ANALYTICAL AND EXPERIMENTAL STUDY ON STEEL FIBER REINFORCED CONCRETE CORBEL

By

MEHTA ANAND

08MCL021

DEPARTMENT OF CIVIL ENGINEERING

AHMEDABAD-382481

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ANALYTICAL AND EXPERIMENTAL STUDY ON STEEL FIBER REINFORCED CONCRETE CORBEL

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(Computer Aided Structural Analysis and Desiging)

By

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May 2010

Declaration

This is to certify that

i) The thesis comprises my original work towards the degree of Master of Technology in Civil Engineering at Nirma University and has not been submitted elsewhere for a degree.

ii) Due acknowledgement has been made in the text to all other material used.

MEHTA ANAND

Certificate

This is to certify that the Major Project entitled "Analytical and Experimental Study on Steel Fiber Reinforced Concrete Corbels" submitted by Mr. Mehta Anand R., 08MCL021, towards the partial fulfillment of the requirements for the degree of Master of Technology in Civil Engineering of Nirma University of Science and Technology, Ahmedabad is the record of work carried out by him under my supervision and guidance. In my opinion, the submitted work has reached a level required for being accepted for examination. The results embodied in this major project, to the best of my knowledge, haven't been submitted to any other university or institution for award of any degree or diploma.

Mr. Himat Solanki
Guide, Faculty,
Department Civil Engineering,
Institute of Technology,
Nirma University, Ahmedabad

Prof. P. H. Shah
Professor and Head,
Department of Civil Engineering,
Institute of Technology,
Nirma University, Ahmedabad

Dr K. Kotecha
Director,
Institute of Technolog
Nirma University,
Ahmedabad.

Abstract

The research program for this thesis studies the benefits of using steel fiber in reinforced concrete corbel. By using Steel Fiber Reinforced Concrete corbel, some of the difficulties associated with it's construction can be overcome and a greater seismic strength can be provided.Use of steel fiber provides ductility to the section,it reduces crack width and also increases the ultimate shear capacity of the section. Several points emerge from a study of the behavior of corbel under vertical load that has been taken into account to develop a method of design.The main factors influencing the behavior of corbel section under ideal condition are a_v/d(Shear span/depth) ratio , Percentage of main reinforcement , Concrete strength and Fiber percentage in the section.

Various guidelines as provided by different codes i.e. IS-456, ACI-318 and EC-2 were studied and comparison between those guidelines was done.It was observed that the information for designing the section with use of steel fiber was lacking or has not been incorporated in the guidelines.Thus, An analytical method was proposed based on Truss Analogy for estimating the ultimate load bearing capacity of corbel specimen under vertical loads which takes into account various parameters like bond stress,fiber stress,fiber length/diameter ratio(Aspect ratio).

A comparison with 51 test results reported in the literature is made and proposed theory is also verified with experimental work.Experimental work comprises of Four corbel specimen constructed without addition of steel fiber in order to reflect current building code , Four steel fiber in reinforced concrete corbel were constructed with a fiber percentage of 0.5 % while Four steel fiber in reinforced concrete corbel were constructed with a fiber percentage of 1.0 % by varying the percentage of main tensile reinforcement as 0.8 % for 6 specimen and 1.2% for remaining 6 specimen.Xorex steel fibers with a length of 30-mm, a diameter of 0.50-mm and an aspect ratio of 60 were

used.

Design charts for reinforced concrete corbel for various codes were discussed and same charts were modified for steel fiber reinforced concrete corbel.with the use of modification made in chart one can easily derive the increase in strength of section with increase in percentage of steel fiber.The modification factor is found based on Aspect ratio and percentage of steel fiber. .

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- MEHTA ANAND

08MCL021

Abbrevation Notation and Nomenclature

A_s .... Area of tension bar reinforcement.

a .... Depth of rectangular compressive stress block.

A_v .... Shear span of corbel.

b .... Width of column/corbel.

c .... compressive force.

d .... Distance from extreme compression fiber to centroid of main bar reinf.

d_f .... Fiber diameter.

E_s .... Modulus of elasticity of steel.

e .... Distance from extreme compression fiber to top of tensile stress block

F_{be} .... Bond efficiency factor.

f_c' .... Cube compressive strength of concrete.

f_y .... Yield strength of concrete.

h .... Overall depth of corbel section.

l .... Fiber length.

p_e .... Effective volume percentage of fibers.

P .... Percentage of volume of fiber.

T_{fc} .... Tensile forces of fibrous concrete.

T_{rb} .... Tensile force for bar reinforcement.

X .... Distance from extreme compression fiber to neutral axis.

β .... Angle of inclination of compression strut with respect to N.A.

$\beta 1$ .... Factor used to calculate the depth of rectangular stress block.

εs .... Tensile strain in steel.

εc .... Compressive strain in concrete.

σt .... Tensile stress in fibrous concrete.

ζd .... Dynamic bond stress between fiber and matrix.

α .... Modification factor for compressive strength.

A_{st} .... Area of reinforcement

DL .... Dead load

LL .... Live load

L .... Length of member

f_y .... Yielding strength of steel

V_u .... Ultimate Shear

E_s .... Modulus of Elasticity of Steel.

ϕ .... Diameter of Bar

σst .... Permissible Stress in Steel

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Chapter 1

Introduction

1.1 General

A corbel or a bracket is a short cantilever projection which supports a load bearing member
where :

a. The distance a_v between the line of reaction to the supported load and the root of the corbel is less than d (Figure 1.1).

b. The depth at the outer edge of the contacted area of the supported load is not less than one half of the depth at the root of the corbel.

Corbel is a structural element to support various structural components of structure (i.e. beams, columns, gantry, heavy beams of parking structures, etc.). Now-a-days, it is widely used in precast construction. The corbel is cast monolithic with the column element or wall element. The depth of the corbel at it's outer edge should not be less than one-half of the required depth d at the support.
Typically, reinforcement as shown in figure 1.1 for corbel in past consisted of several bars across the width of the bracket bent. The depth of a bracket or corbel at its outer edge should not be less than one-half of the required depth d at the support. Reinforcement should consist of main tension bars with area As and shear

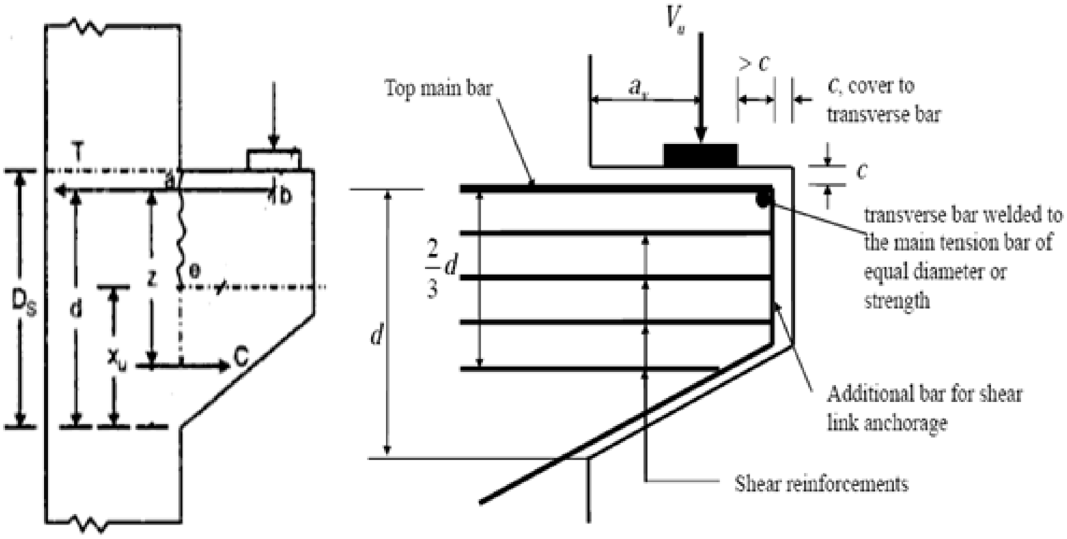


Figure 1.1: Typical sketch of Corbel

reinforcement with area A_h. The shear reinforcement should consist of closed ties parallel to the main tension reinforcement. The area of shear reinforcing should uniformly distributed within two-thirds of the depth of the bracket/corbel adjacent to the main tension bars. It is good practice to anchor main tension reinforcement bars as close as possible to the outer edge by welding a crossbar or steel angle to the main tension bars. Tension Reinforcement, As should be adequate at the face of the support to resist the moments due to Vu the vertical load and any Nu horizontal forces. This reinforcement must be properly developed to prevent pull-out, by proper anchorage within the support and by a crossbar welded to the main tension bars at the end of the bracket/corbel.

1.2 Behaviour of reinforced concrete corbel

1.1.1 BEHAVIOUR OF REINFORCED CONCRETE CORBEL:-

The following are the major points which describe the behavior of the reinforced concrete corbel, as follows:

a. The ***shear span/depth*** a_v/d ***ratio is less than 1.0***; it makes the corbel behave in two-dimensional manner.

b. ***Shear deformation*** is significant in the corbel.

c. There is ***large horizontal force*** transmitted from the supported beam result from ***long-term shrinkage*** and ***creep deformation***.

d. ***Bearing failure*** due to large concentrated load.

e. The cracks are usually ***vertical*** or ***inclined pure shear cracks***.

f. The mode of failure of corbel are shown in Figure 1.2.

- Diagonal shear(shear failure)
- Shear friction(Yeilding of tension bars)
- Anchorage failure(failure of concrete by compression or shearing)
- Vetrical splitting(bearing failure)

The followings figure shows the mode of failure of corbel.

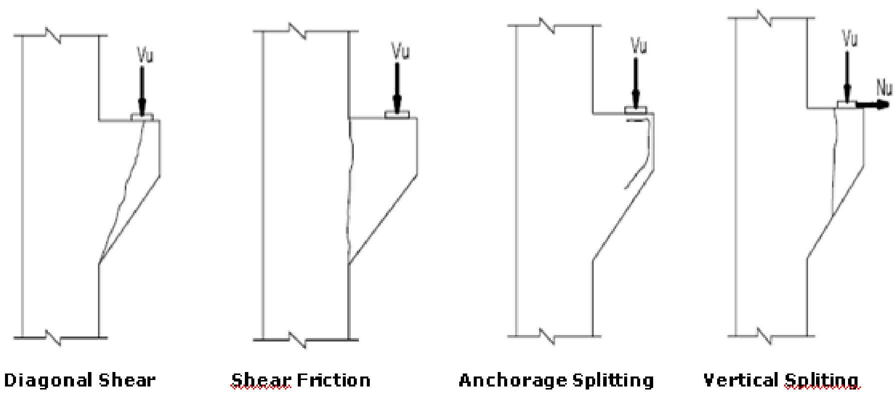


Figure 1.2: Failure modes of R.C.Corbel

1.3 Background

Concrete is one of the most versatile building materials. It can be cast to fit any structural shape from a cylindrical water storage tank to a rectangular beam or column in a high-rise building. It is readily available in urban areas at relatively low cost. Concrete is strong under compression yet weak under tension. To enhance the tension in the concrete some form of reinforcement is needed. The most common type of concrete reinforcement is the use of steel bars. The advantages, of using concrete are due to high compressive strength, good fire resistance, high water resistance, low maintenance, and long service life. The disadvantages to using concrete are due to poor tensile strength, and formwork requirement.

The intrinsic problem of normal concrete is its brittle nature which may cause collapse in non-seismically detailed structural members after the first crack during a large earthquake. The use of steel fibers into the reinforced concrete member may convert the brittle characteristics to ductile ones. The principal role of fibers is to bridge cracks and resist their formation to control plastic shrinkage cracking and drying shrinkage cracking. Therefore a considerable improvement in tensile strength and higher ultimate strain can be obtained. When steel fiber reinforcement is added into the reinforced concrete structural member generally fibers do not increase the flexural strength of concrete, so it cannot replace structural steel reinforcement.

Steel Fiber Reinforced Concrete (SFRC) has an untapped potential application in Building frames due to its high seismic energy absorption capability and the construction technique is relatively simple. To tap such potential, the existing body of knowledge on SFRC must be expanded to provide a proper basis for officials to add this method of construction to the provisions of the building code. The amount of fibers added to a concrete mix is measured as a percentage of the total volume of the composite (concrete and fibers) termed V_f . V_f typically ranges from 0.1 to 3%. Aspect ratio (l_f/d_f) should also be considered and it can be calculated by

Dividing fiber length (l) by its diameter (d) of fiber. In case of Fibers with a non-circular cross section is used an equivalent diameter of fiber should be considered for the calculation of aspect ratio. If the modulus of elasticity of the fiber is higher than the matrix (concrete or mortar binder), it help to carry additional load due to increase in the tensile strength of the material. Moderate increase in the aspect ratio of the fiber usually is resulted into a higher flexural strength and toughness of the matrix. However, fibers which are too long or large aspect ratio end to "ball" in the mix and create workability problems.

This research focuses on behavior of Reinforced concrete corbel steel using steel Fibers. Normally steel fiber length ranges from 1.5 to 75 mm, a typical diameter Lies In the range of 0.25 to 0.75 mm and aspect ratio ranges from 30 to 100. Fibers are drawn wire from mild steel, conforming to IS-280-1976 with the diameter of wire varying from 0.3 to 0.5 mm have been practically used in India. Steel fibers Having a rectangular cross-section are produced by silting the sheets about 0.25 mm thick. Round steel fibers are produced by cutting or chopping the wire, flats Sheet fibers having a typical cross-section ranging from 0.15 to 0.41 mm in Thickness and 0.25 to 0.90 mm in width are produced by silting flat sheets .Fiber shapes are illustrated in Figure 1.3. Addition of steel fibers does not significantly increase the compressive strength, but it increases the tensile toughness and ductility of the reinforced concrete structural member. It also increases the ability to withstand stresses after significant cracking (damage tolerance) and shear resistance. This finding is very important so that the ductility increases when concrete is reinforced with fiber.

This research aims to improve knowledge by addition of steel fiber reinforced concrete structural member(corbels)through experimental investigation and analysis. Based on the several research have concluded that the micro fibers have better impact in resistance as compares to the longer fiber.

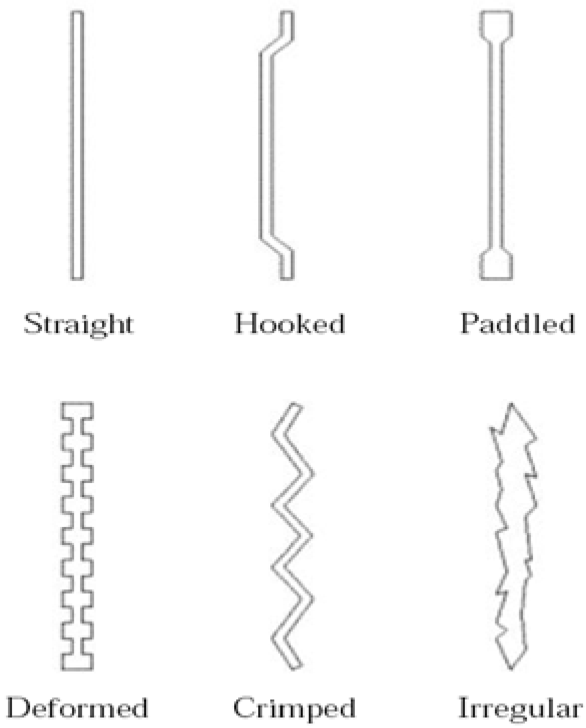


Figure 1.3: Various shapes of Steel fibres

1.4 Objective of work

The objectives of this thesis are:

a. To experimentally investigate the behavior of Reinforced concrete corbel with steel fiber under a vertical load.

b. To investigate the shear resistance capacity of reinforced concrete corbel with or without steel fiber.

c. To develop a simplified analytical method and design procedure based on Strut and Tie truss model and compare its results with various available codes and experimental work.

d. To develop a nomograph for single corbel to obtain percentage of steel reinforcement required for various section of corbel with steel fibers from the results obtained from analytical and experimental work.

1.5 Scope of work

Study of corbel has been mainly divided into two parts,

1, Analytical study.

2, Experimental study.

1.5.1, Analytical study: Designing of single and double corbel using different approaches

a. Designing of corbel using various codes :

(1) According to IS-456 method [Truss theory].

(2) According to ACI method [Shear friction theory].

(3) According to European code.

(4) Using strut and tie method with concrete strain softening.

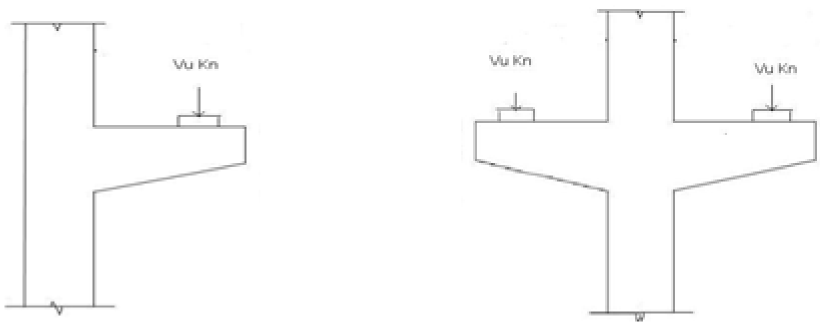


Figure 1.4: Typical sketch of single and double corbel

Based on the given vertical load and other parameters of corbel the tensile reinforcement will be calculated by using various specified methods including the proposed method for various grades of concrete including fiber reinforced steel. A simplified nomograph will be developed for calculating percentage of main reinforcement required for various corbel section.

a. Experimental study : various specimen with M25 grade concrete(25 Mpa) and Reinforcement detailing as specified below will be casted to study the behavior of corbel and comparing various analytical results with experimental results under vertical loading condition.

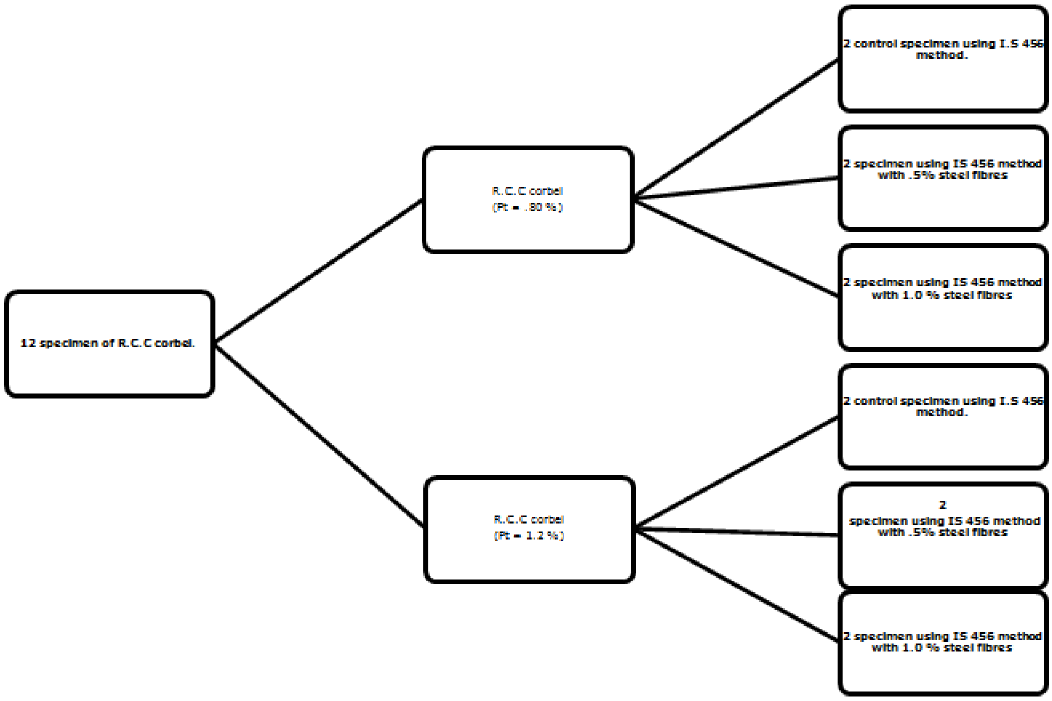


Figure 1.5: Matrix of Experimental work

1.6 Thesis Organization

The thesis is divided into seven chapters. In **Chapter 1** general aspects of Reinforced concrete corbel discussed. It also includes objectives of study and scope of work.

Chapter 2 describes the literature review base on previous research work related to the topics. Both analytical and experimental components of past research are described; it also includes various criteria's as suggested by different codes.

Detailed of proposed theory of the present work along with example and various codal provisions by IS-456, EC-2 and ACI-318 is discussed in **Chapter 3**.

Chapter 4 describes the experimental work. Material and equipment used in the test program, the specimen details along with photographs and the test procedure used are reported hear.

Chapter 5 deals with the available result in the literature are compared with analytical predictions, and the shear strengths are also compared with code predictions.

Chapter 6 discuss about the design charts proposed on basis of various code, together with Nomograph for R.C.C corbel with steel fibers.

Chapter 7 Based on the research work various conclusions were drawn and are presented in this chapter and Recommendations for future research.

Complete test data and crack pattern of test Corbel are given in
Appendix – A Test data and Strain readings.
Appendix – B Failure pattern of test corbels.
Appendix – C List of useful websites.

Chapter 2

Literature Survey

2.1 General

In the past, many researchers have presented on shear strength of Reinforced Concrete corbel with or without application of steel fibers. This chapter focuses on recent theoretical concepts for shear strength in reinforced concrete corbels. The review of various papers related to the shear strength of R.C. corbel is also described to better understand its behavior.

A brief review of the shear provisions in the Indian Standard IS 456:2000 [2], American Concrete Institute Building Code ACI 318-05 [1], and Euro code 2 Part I [3] are also outlined.

Conventional concrete made with Portland cement is relatively strong in compression but weak in tension. It is the reason why reinforcing bars are used in concrete to overcome its weakness in tension. However, with the fiber technology developed, the weakness in tension can be partly surmounted by the inclusion of a sufficient volume of fibers. The concrete incorporated with sufficient fibers can improve the post-cracking behavior of the fiber matrix composites, thereby improving its toughness (**Figure 2-1**). Fibers are used in not only structural areas but also in other special applications such as reducing cracking, drying cracking, chemical resistance, abrasion resistance, and fire resistance. **Table 2-1** shows the application of various fiber reinforced concrete for which fibers may be used to enhance the structural behavior of the reinforced concrete structural members.

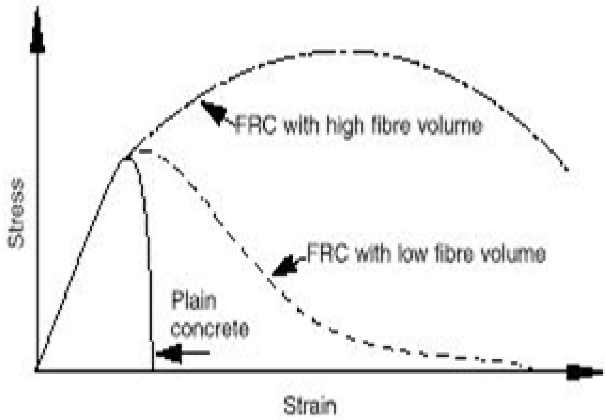


Figure 2.1: Comparision between plain and fibre reinforced concrete

2.2 Experimental and analytical study on shear strength of reinforced concrete corbel

Corbels with shear reinforcement tested by other researchers are considered here. The shear strength and details of other parameters capacities of these corbel sections are given in the following subsection.

The shear failure load of corbel is principally by development of vertical crack starting from the re-entrant corner proceeding towards the lower fiber. The bearing failure can also occur if adequate bearing area is not provided. It is necessary to provide a required amount of shear reinforcement, which should prevent sudden shear failure on the formation of first diagonal tension cracking and, in addition, would adequately control the diagonal tension cracks at service load levels. To improve the structural behavior of reinforced concrete corbels, steel fiber should be added to the conventional

Table 2.1: Application of fibre reinforced concrete

Fiber Type	**Application**
Steel	Seismic-resistant structures, bridge decks, cellular concrete roofing units, pavement overlays, concrete pipe, airport runways, pressure vessels, tunnel linings, ship-hull construction.
Glass	Precast panels, small containers, sewer pipe, thin concrete shell roofs, wall plaster for Concrete block. Agriculture, architectural cladding and components.
Carbon	Single and double curvature membrane structures, boat hulls, scaffold boards.
Polypropylene, Nylon	Foundation piles, prestressed piles, facing panels, floatation units for walkways and moorings in marinas, road-patching material, heavyweight coatings for underwater pipes.
Natural fibers	Roof tiles, corrugated sheets, pipes, silos and tanks

concrete mix. This will not only improve its structural behavior but also reduces the amount of shear reinforcement

2.2.1 Shailendra Kumar [4]

Paper discuss an analytical study on the ultimate shear strength of steel fibrous reinforcement concrete corbel without shear reinforcement and tested under vertical loading only. Paper compared the results of 55 specimens in literature. Based on a comparison he developed a semi-empirical characteristics equation and then he verified to measure the shear strength involving various parameters.

2.2.2 Nijad I. Fattuhi [5]

Nijad studied the behavior of twenty-five reinforced concrete corbel with the application of steel fiber as shear reinforcement and also discussed on behavior of corbel on variation of some principal parameters like main tension reinforcement, shear span and ratio of horizontal to vertical load Fig.2.1 and Table-2.2a. Based on the experimental study, he concluded that addition of steel fibers to concrete improves the performance of corbel and addition of 1 % by volume of steel fiber can increase an average strength by 25 %, while the application of horizontal load in addition to vertical load resulted in decrease in load bearing capacity of corbel. He also concluded that there is no apparent effect of fiber length to corbel depth ratio on fiber efficiency. Also developed an expression for estimating maximum load carried by corbel and load at a particular crack width.

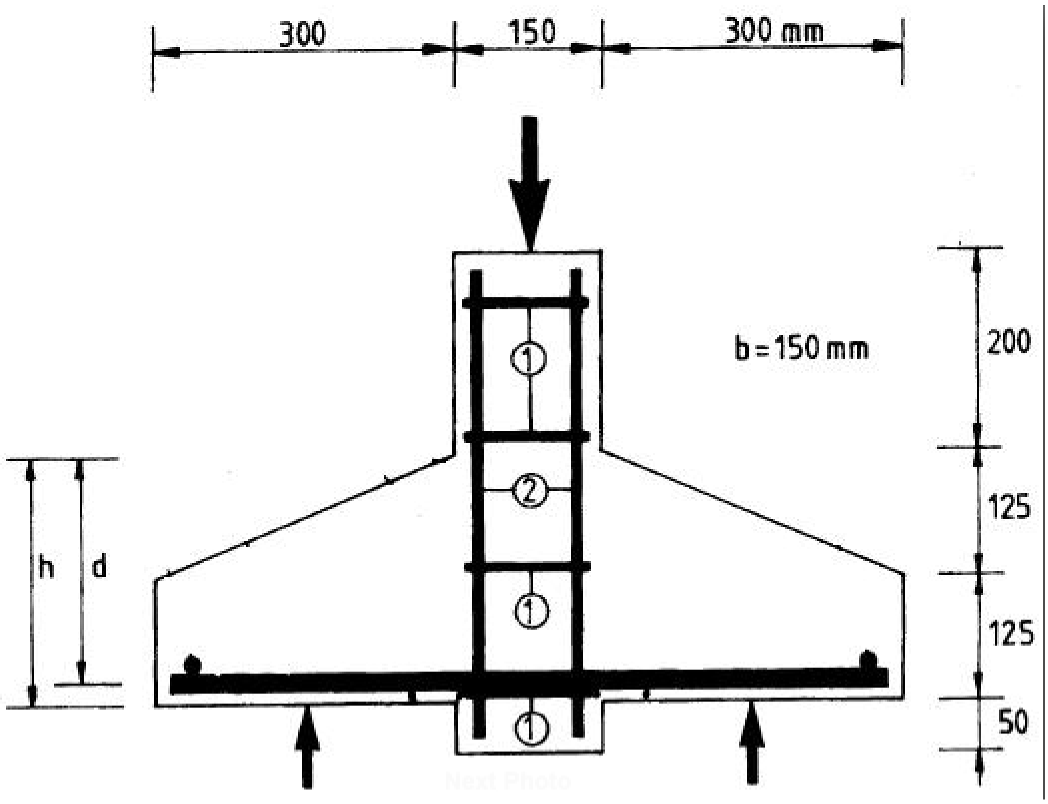


Figure 2.2: Details of specimen for experimental work[5]

Table 2.2: Reinforcement details and ultimate load capacity of specimens [5]

Corbel	Reinforcement		a,mm	b,mm	d,mm	h,mm	H*/v
	Main bars,mm	Vf					
89	2x10	0	150	152.5	224	249	0
90	2x10	0	200	154	226	251	0
91	2x12	0	200	151.5	225	251	0
92	2x12	0	150	150.2	223	249	0
93	2x10	0	100	151.9	225	250	0
94	2x12	0	100	151	223	249	0
95	2x10	0	150	151.1	223.5	248.5	0
96	2x10	1	200	152.6	226	251	0
97	2x12	1	150	150.7	223.5	249.5	0
98	2x12	1	200	151	222	248	0
99	2x12	1	100	151.4	223	249	0
100	2x10	1	100	151.5	224.8	249.8	0
101	2x18	1	165	151	217.2	248.2	0
102	2x18	0	165	154	214.1	250.1	0
103	2x18	1	230	152	221.8	250.8	0
104	2x18	0	120	154.5	210	249	0
105	2x14	1	150	150.8	230	249.8	0
106	2x14	1	100	151.2	230.5	249.5	0
107	2x14	1	150	150.9	231	250	0.2
108	2x14	1	100	152	231	250	0
109	2x14	1	150	152	231	250	0.2
110	2x14	1	150	153.5	229	248	0.5
111	2x14	1	150	151.5	231	250	0.5
112	2x14	1	150	151	229	248	0
113	2x14	0	110	153.6	219	250.9	0.2

2.2.3 Hashim M.S Abdul-Wahab [6] Hashim studied six specimens the effect of addition of steel fiber by adding the percentage of steel fiber with various shear span-span ratio. Based on the test results he concluded that 0.5 % of addition of steel fiber by volume of concrete leads to 21 % in increase in shear strength while addition of 1.0 % of steel fiber by volume of concrete leads to 40 % of increase in shear strength he also compared with the analytical methods as specified in ACI code and Truss analogy and concluded that these two methods were very conservative and such approaches may not lead to economic designs. When steel fiber reinforced is added further he proposed a method for predicting the contribution of fiber in shear resistance which contributed in less congested and conventional reinforced corbels.

2.2.4 Giuseppe Campione ,Lidia La Mendola & Maria Letizia Mangiavillano [7] Giuseppe et al. discuss about the flexural behavior of corbel in plain and fibrous concrete. Based on experimental study they developed various result in form of load-deflection curves and crack pattern and an effectiveness of fibrous concrete to reinforced concrete. Also they developed an analytical model using equivalent truss model and its results were compared with various test results available in literature and their test results with variation of steel fibers as 0.5 % and 1.0 % by volume of concrete. They concluded that fibrous concrete activates flexural failure and improves the ductility of corbel and superior performance even in terms of strength.

2.2.5 Nijad I. Fattuhi and Barry P. Hughes [8]

14 specimens under vertical loading with main reinforcement and secondary reinforcement with variation of steel fibers or stirrups have been studied by Fattuhi and Hughes as shown in Figure 2.3 and Table 2.3. They found that corbel with fibers failed in most gradual and ductile manner and failure mode changed from diagonal splitting to flexural mode when fiber were used as secondary reinforcement . They observed an elastic-plastic behavior for the corbels

reinforced with relatively large number of steel fibers.

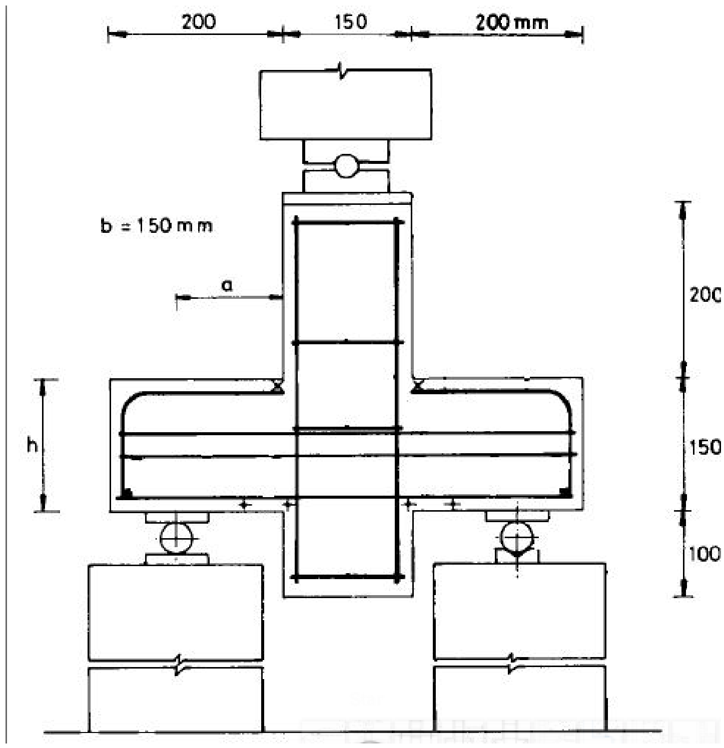


Figure 2.3: Details of specimen for experimental work

2.2.6 Nijad I. Fattuhi and Barry P. Hughes [9] 18 specimens under vertical loading were tested by varying volume of main bars and shear-span to depth ratio as shown in Figure 2.4 and Table 2.4. They suggested an empirical equation for estimating the maximum vertical load for reinforced concrete corbels made from plain or steel fibrous concrete. They concluded that a specimen with only main bars failed catastrophically and the mode of failure was generally diagonal splitting or constrained shear. While corbel with steel fiber failed gradually and in a ductile manner. They almost noticed an elastic-plastic behavior for the corbels reinforced with low volume of main bars and at a relatively large shear span-to depth ratio.

Table 2.3: Reinforcement details and ultimate load capacity of specimens [8]

Corbel no	Reinforcement		
	Tension bars,dia	Stirrups	Fibers(equivalent to)
T1	2X10 mm	——————-	——————————————
T2	2X10 mm	1x10mm^2	——————————————
T3	2X10 mm	——————-	No. 1 stirrup(10mm^2)
T4	2X10 mm	——————-	No. 2stirrup(10mm^2)
T5	2X10 mm	——————-	No. 3stirrup(10mm^2)
T6	2x12 mm	——————-	——————————————
T7	2x12 mm	1x10mm^2	——————————————
T8	2x12 mm	2x10mm^2	——————————————
T9	2x12 mm	2x10mm^2	——————————————
T10	2x12 mm	——————-	No. 1 stirrup(10mm^2)
T11	2x12 mm	——————-	No. 2stirrup(10mm^2)
T12	2x12 mm	——————-	No. 3stirrup(10mm^2)
T13	——————-	——————-	——————————————
T14	——————-	——————-	——————————————

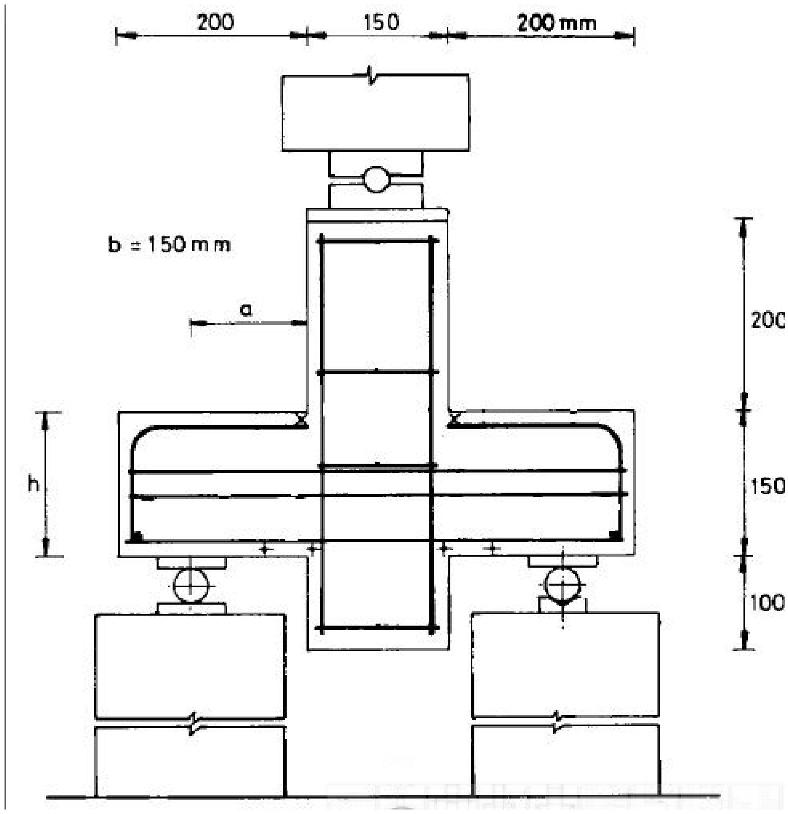


Figure 2.4: Details of specimen for experimental work

2.2.7 Giuseppe Campione [10]

Since a softened STM macro model which can reproduce the flexural behavior of corbels in plain and fibrous concrete for determination of load deflection curves of corbels subjected to the coupled effect of horizontal and vertical loading they considered a softened strut and tie micro model in their work .Results of their study show considerable reduction in crushing of compression region by use of fiber instead of stirrups and also improvement in ductility by activating flexural failure when fibrous concrete is considered. Design considerations on use of fibrous reinforced concrete in RC corbel was given. The results obtained showed the effectiveness of FRC resulted into improvement of maximum strength and also showed that FRC behavior is very ductile and in most cases flexural failure

Table 2.4: Reinforcement details and ultimate load capacity of specimens [9]

Corbel no.	Rein(Main bars)	Rein(Fibers,mm)	a/h ratio	LOAD
C27	2-8mm	0.65*60	0.35	125.8
C28	2-8mm	Dual-form	0.59	88.2
C29	2-8mm	Dual-form	0.83	65.9
C30	2-10mm	Dual-form	0.35	171
T3	2-10mm	Dual-form	0.59	133
C4	2-10mm	Dual-form	0.83	91.8
C31	2-12mm	Dual-form	0.43	179
T10	2-12mm	Dual-form	0.59	138
C32	2-12mm	Dual-form	0.83	110.1

occurs in such specimen.

2.2.8 L.B.Kriz & C.H.Raths [11]

This paper discusses the development of design criteria for R.C. corbel. This paper has three parts Part 1 discusses about the Design criteria, Design aids and Examples. Part 2 describes various tests on proposed criteria as described in Part 1 corbel with test specimen 124 with vertical load and 71 corbels with vertical and horizontal load. While Part 3 includes discussion and analysis of experimental data and derivation of design equations.

The experimental evidence presented in this paper indicates that the nominal ultimate shear stress, V_{UF} in corbels with a shear span to effective depth ratio less than one may exceed the maximum shear stress allowed by Chapter 17 of the AC1 Code (AC1 318.6.3). For corbels with a_v/d ratio greater than one, the nominal ultimate shear stress in a corbel is a function of the ratio of the shear span to the effective depth, of the reinforcement ratio, of the concrete strength, and of the ratio of the horizontal and vertical components of the applied loads. They observed that horizontal forces acting outward from the column significantly reduce corbel strength, and should be considered in the design of a corbel unless special provisions are made for free movements of the supported beams. They also observed that tension reinforcement and horizontal stirrups are equally effective in increasing the strength of a corbel subject to vertical loads only. However, the effective amount of reinforcement should be limited. They noticed that load carried by a column do not affect the corbel strength, nor does the amount or arrangement of column reinforcement. The results of this investigation have been used as a basis for the formulation of "Proposed Criteria for the Design of Corbels".

2.2.9 Alan H. Mattock [12]

Simple new and modified shear friction design proposals for normal weight and lightweight reinforced concrete corbels are presented based on previously reported experimental studies. A Model Code Clause embodying the design proposals in detailed, together with a step-by-step procedure for practical application are presented. Design examples are included for both normal weight and lightweight concrete corbels using both the ACI 318-71 shear-friction provisions and the modified shear-friction theory. It also contains charts to facilitate proportioning of the corbel reinforcement. Use of the design proposals can lead to savings in reinforcement, particularly if the modified shear-friction theory is used for shear design of reinforced concrete corbel.

2.2.10 Himat Solanki and Gajanan M. Sabnis [13]

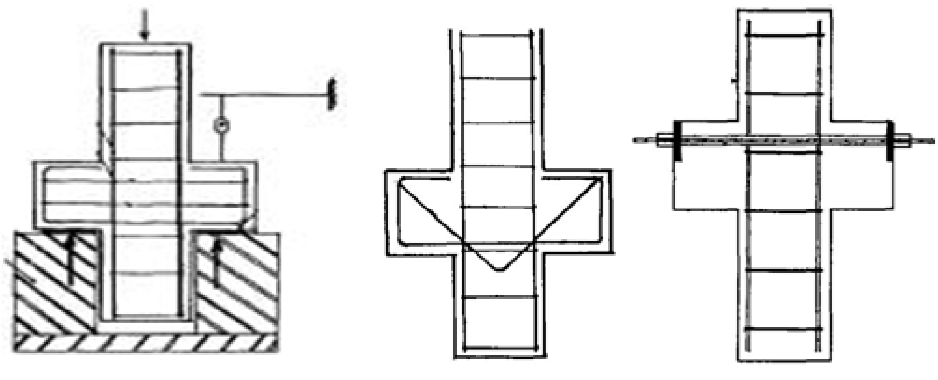
They simplified the truss analogy based on the summary of previous work. They reviewed test series of 16 different investigations and were analyzed and then were calculated with the proposed truss analogy. Proposed a general design approach covering the practical range of a/d values from 0.1 to 1.0 and for horizontal or inclined reinforcement and eccentric loads.

2.2.11 Mohamed A. Ali ,Richard N. White [14]

Paper discuss about Strut and Tie Model with compression stress bulging of the strut to its degradation. They compared available 34 test specimen results under horizontal and vertical loading done by Alan.H.Mattock, Chakraborti and balaguru with analytical work and concluded on various variations of results of different methods.

2.2.12 Pinaki R. Chakrabarti , Davood J.Farahani and Shihadeh I.Kashou [15]

Short concrete members like brackets, corbels and ledger beams are subjected to direct shear with reinforcement normally selected using the shear friction theory from ACI318-83.the study was done in two phases .phase-1 include eight corbels divided to four series with depth varying between 228.60 and 254.00 mm with closed and inclined stirrups. While in phase-II, nine corbels divided into three series 228.60 mm depth with and without confining steel, and with varying amount of precompression were studied. in both cases, the shear span-to-depth ratio was less than 0.5 he found that the limits the ultimate shear strength criteria were very conservative. Cracks appears around 70 % of the collapse load.



Series TA Series Tb Series s

Figure 2.5: Details of specimen for experimental work

Table 2.5: details and ultimate load capacity of specimens [9]

Specimen	a/d	F	Vu(t)	Vu(l)
		MPa	test,	calculated,
		(psi)	kN	kN(kips)
			(kips)	
TA-1	0.37	28.96	322.48	199.3
TB-1	0.37	28.5	333.6	238
TA-2	0.32	28.27	333.6	199.3
TB-2	0.32	28.06	333.6	238
TA-3	0.37	34.64	356	199.3
TB-3	0.37	33.64	360.3	238
TA-4	0.32	34.86	382.53	199.3
TB-4	0.32	33.64	378.1	238
Specimen	a/d	Fc	Vn(t) test	Vn(a) calculated
		MPA	KN	kN(kips)
SA-1	0.37	34.6	266.9	190
SA-2	0.37	33.9	278	190
SA-3	0.37	33.2	266.9	190
SB-1	0.37	56.4	422.6	258
SB-2	0.37	65.7	433.7	258
SB-3	0.37	55.3	411.5	258
SC-1	0.37	37.1	367	252.2
SC-2	0.37	38.3	422.6	252.2
SC-3	0.37	41.5	433.7	252.2

2.2.13 Bhupinder Singh, Yaghoub Mohammadi and S.K. Kaushikf [16] Strut and tie method have been discussed based on assumption that pin-jointed trusses consisting of strut and tie connected at nodes based on ACI codes. Also discussed about the selection of strut and ties and determination of truss forces, various check on strut and crack control reinforcement.

2.2.14 Saeed Ahmad and Attaullah Shah[17]

Ahmad and attaullah discussed an Strut and Tie method for double corbel based on the research work done on nine double corbel experimentally and compared the results with Strut and Tie method for high strength concrete and concluded that experimental results obtained for high strength reinforced concrete corbels are quite close to the calculated theoretical values using Strut and Tie method for the shear strength of corbel.

2.2.15 Michale Chilvers and Sam Fragomeni[18]

Strut and Tie method as specified in AS 3600-2000 concrete structure code, a trial and error procedure was carried out to determine the amount of tension reinforcement required in corbel to resist anticipated vertical load. They developed chart to facilitate the designer to eliminate an iterative design process for determining the amount of tension reinforcement in the corbels.

2.2.16 Himat Solanki [19] The design chart of reinforced concrete corbel based on truss analogy method having different span to depth ratio (av/d) of one or less and subjected to a combination of horizontal and vertical loads such that Nu/Vu $<=$ 1.0. Based on the criteria a design chart was proposed and was developed so as to be compatible with safety provisions for flexure and shear transfer as contained in ACI (318-77).

2.2.17 Charles H. Henager and Terrence J. Doherty [20] They discussed about the analytical method that is based on the ultimate strength approach and takes into account the Bond stress, Fiber stress, Fiber length to diameter ratio and volume fraction of the fiber. The strength computed for the fibrous concrete is added to the strength contributed by the reinforcing bars to obtain the theoretical ultimate moment. The method shows good correlation between predicted strength and experimental values. They found that shear strength increase by 25 % for the particular beams tested also commented that crack width and crack spacing were less in reinforced fibrous concrete and higher laod was achieved. The post-cracking strength stiffness of the reinforced fibrous beams was greater than a conventional reinforced beam.

2.2.18 Glyn Jones [21] The design chart was presented based on BS-8110 for av/d <0.6.they gave an graphical representation of the upper and lower limits imposed on the reinforcement (max 1.3%). They found accurate modulus of elasticity values for fiber reinforced composites are difficult to determine from flexural tests. However, by allowing for shear deflections the central deflection readings indicated modulus of elasticity values between 12.3 and 16.7 GN/m2 for all beams tested.

2.2.19 Himat T. Solanki [22] The paper discussed about the development of the design charts for reinforced concrete corbels based on the truss analogy having shear span to depth ratio less than 1.0. The proposed design charts presented here, were developed so as to be compatible with safety provision for flexure and shear transfer as contained in ACI 318-77

2.2.20 G. Somerville [23] This report reviews the available test data on corbels to determine the major parameters that influences behavior are evaluated and compared. A design approach is developed which is consistent with that in the draft unified code of practice and which is capable of dealing with horizontal forces and with ratios of shear span to effective depth which are greater than 1.0. the system of forces involved is that of a simple Strut and Tie. Design and detailing is particularly important with regard to anchoring the main steel and providing secondary reinforcement.

2.2.21 Nguyen Van chanh [24] In this paper, the mechanic properties, technologies, and applications of steel fiber reinforced concrete are discussed. he also discussed and gave Mix design for steel fiber reinforced concrete and gave procedure for producing steel fiber reinforced concrete and discuss various structural use and application of steel fiber reinforced concrete.

2.3 Summary

During the last three decades, steel fibers have been applied in pavement and shotcrete linings. However the use of steel fiber reinforced concrete (SFRC) in real seismic design is restricted because of the lack of validated design formulae and appropriate codes. Recently, a range of steel fibers and SFRC products are commercially available, and the use of steel fiber reinforced concretes (SFRC) in structure has developed progressively. Steel fiber reinforced concrete is a concrete mix that contains discontinuous, discrete steel fibers that are randomly dispersed and uniformly distributed. The quality and quantity of steel fibers influence the mechanical properties of concrete. It is generally accepted that addition of steel fibers significantly increases tensile toughness and ductility. The benefits of using steel fibers become apparent after concrete cracking because the tensile stress is then redistributed to fibers. For structural design purpose less than 0.5 % fiber dosage rates are not helpful to withstand stresses after significant cracking.

Chapter 3

Theory

3.1 General

This chapter present a theory developed to predict the shear strength of reinforced concrete corbel with steel fibers. The theory is simplified theory based on the stress analysis of Strut- and Tie model with concrete softening effect. This chapter also includes various codal provisions for design of reinforced concrete corbel.

3.1.1 Factors affecting properties if fiber reinforced concrete

Fiber reinforced concrete is the composite material containing fibers in the cement matrix in an orderly manner or randomly distributed manner. Its properties would obviously, depend upon the efficient transfer of stress between matrix and the fibers. The factors are briefly discussed below:

- *Relative Fiber Matrix Stiffness:* The modulus of elasticity of matrix must be much lower than that of fiber for efficient stress transfer. Low modulus of fiber such as nylons and polypropylene are, therefore, unlikely to give strength improvement, but the help in the absorbsion of large energy and therefore, impart greater degree of toughness and resistance to impart. High modulus steel fibers impart strength and stiffness to the composite. Interfacial bond between

Table 3.1: Aspect ratio of the fiber

Type of concrete	Aspect ratio	Relative strength	Relative toughness
Plain concrete	0	1	1
With fibers	25	1.5	2.0
Random fibers	50	1.6	8.0
Dispersed fibers	75	1.7	10.5
	100	1.5	8.5

the matrix and the fiber also determine the effectiveness of stress transfer, from the matrix to the fiber. A good bond is essential for improving tensile strength of the composite.

- *Volume of Fibers:* The strength of the composite largely depends on the quantity of fibers used in it. Variation of fibers in concrete mix shows the effect of volume on the toughness and strength. The increase in the volume of fibers, increase approximately linearly, the tensile strength and toughness of the composite. Use of higher percentage of fiber is likely to cause segregation and harshness of concrete and mortar.

- *Aspect Ratio of the Fiber:* Another important factor which influences the properties and behavior of the composite is the aspect ratio of the fiber. It has been reported that up to aspect ratio of 75, increase on the aspect ratio increases the ultimate concrete linearly. Beyond 75, relative strength and toughness is reduced. Table 3.1 shows the effect of aspect ratio on strength and toughness.

- *Orientation of Fibers:* One of the differences between conventional reinforcement and fiber reinforcement is that in conventional reinforcement, bars are oriented in the direction desired while fibers are randomly oriented. To see the effect of randomness, mortar specimens reinforced with 0.5% volume of fibers were experimentally tested in past by various researchers. In one set specimens, fibers were aligned in the direction of the load, in another in the direction perpendicular to that of the load, and in the third randomly distributed. It was observed that the fibers aligned parallel to the applied load offered more tensile strength and toughness than randomly distributed or perpendicular fibers.

- *Workability and Compaction of Concrete:* Incorporation of steel fiber decreases the workability considerably. This situation adversely affects the consolidation of fresh mix. Even prolonged external vibration fails to compact the concrete. The fiber volume at which this situation is reached depends on the length and diameter of the fiber. Another consequence of poor workability is non-uniform distribution of the fibers. Generally, the workability and compaction standard of the mix is improved through increased water/ cement ratio or by the use of some kind of water reducing admixtures.

- *Size of Coarse Aggregate:* Maximum size of the coarse aggregate should be restricted to 10mm, to avoid appreciable reduction in strength of the composite. Fibers also in effect, act as aggregate. Although they have a simple geometry, their influence on the properties of fresh concrete is complex. The inter-particle friction between fibers and between fibers and aggregates controls the orientation and distribution of the fibers and consequently the properties of the composite. Friction reducing admixtures and admixtures that improve the cohesiveness of the mix can significantly improve the mix.

- *Mixing:* Mixing of fiber reinforced concrete needs careful conditions to avoid balling of fibers, segregation and in general the difficulty of mixing the materials uniformly. Increase in the aspect ratio, volume percentage and size and quantity of coarse aggregate intensify the difficulties and balling tendency. Steel fiber content in excess of 2% by volume and aspect ratio of more than 100 are difficult to mix.

3.2 Analytical model

Method for calculating the ultimate load carrying capacity of steel fiber reinforced concrete corbels:- Various factors taken into considerations

a. Bond stress

b. Fiber stress

c. Fiber length/ Diameter ratio

d. Volume fraction of fiber

3.2.1 Description of method:

Assumption: - The following assumptions are made for the analysis method:-

a. The compressive stress is represented by a rectangular stress block as used in the ultimate strength method.

b. At the extreme concrete compression fiber, the maximum strain are 0.0035 mm/mm.

c. There is no slip between concrete and steel.

d. The fiber composite contributes to the tensile strength of fibrous concrete and is represented by a tensile stress block equal to the force required to develop the Dynamic bond stress of the fibers that are effective in that portion of the corbel cross section.

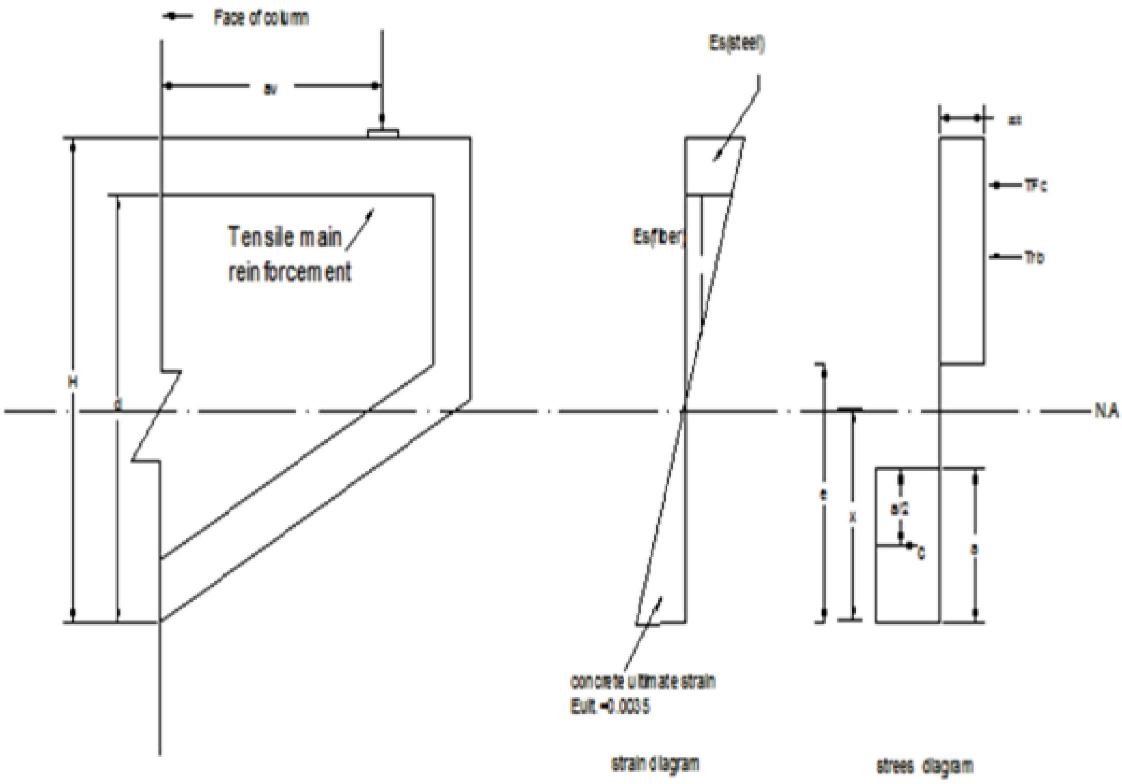
e. Stress distribution in the compression bending zone follows the idealized Stress-Strain curve for concrete.

f. The bond stress that is developed during fiber pull out is defined as Dynamic Bond Stress. The ultimate strength of the corbel occurs along with considerable cracking with that fiber pullout is occurring at that place/point. Values of dynamic bond stress are taken as 3.6 N/mm^2 which gives fiber stresses in the range of 331,000 to 586,000 N/mm^2 .

g. The tension is taken as the area with a minimum tensile strain of σf/Es. σ f =Stresses in fiber at the assumed bond stress. Es =Modulus of elasticity of steel.

h. A bond efficiency factor is assumed as 1.0 for smooth, straight, round or rectangular fiber 1.2 for deformed fibers.

The basic assumptions are shown in the figure.3.1.



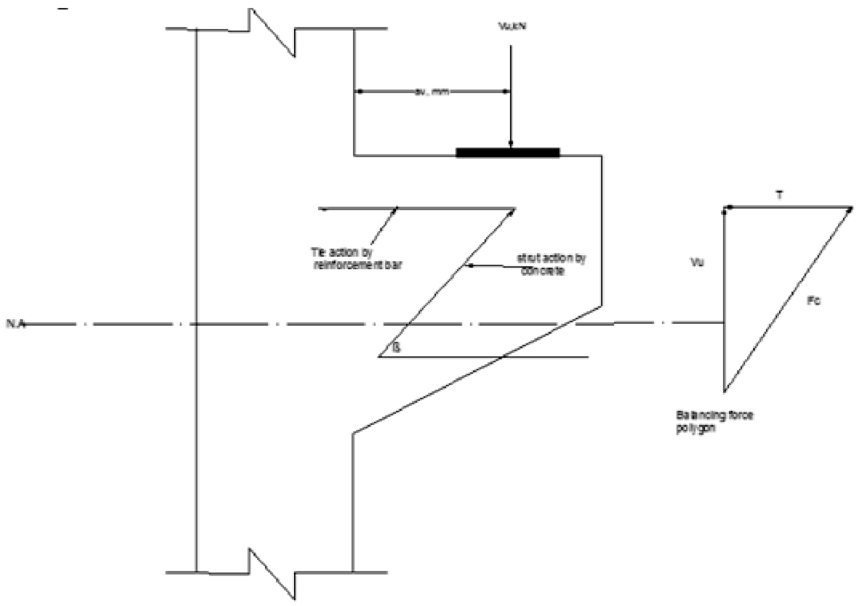


Figure 3.1: Force distribution in corbel section

3.2.2 Mathematical Formulation of Method:-

Various methods has been proposed to calculate the ultimate strength of reinforced steel fiber – concrete (RSFC) corbels, studies have shown that corbels with plain concrete failed in a brittle manner soon after reaching their ultimate looks where as corbels with steel fiber reinforced concrete failed more gradually, showing elastic-plastic behavior. The observed failure modes were classified in to four types i.e.- diagonal Shear, Shear Friction, Anchorage Splitting and Vertical Splitting. In view of the complexity involved in the different kinds of failure modes involving large number of variables, a systematic and rational analysis of reinforced steel – fiber concrete corbels subjected to vertical loading for shear capacity is much necessitated.

Henager and Doherty [19] presented an analytical method based on the usual sectional analysis and fully plasticized stress blocks and forces. The method accounts for fiber resistance in tension by adding a rectangular stress block in the tension zone. The effect of fibers on the strength and ductility of steel – fiber concrete in compression was neglected.

Henager and Doherty [19] referred "Prediction of the flexural strength properties of steel fibrous concrete by Lankard B.R. For a complete analysis of the derivation of the various equation for steel fibrous concrete.
The following procedure has been used to develop the ultimate load (shear strength) carrying capacity of steel fiber reinforced concrete corbels.

Step 1 Calculation of compressive forces

$$C = \alpha f_{cube} ab \tag{3.1}$$

Where,

$\alpha = 0.85$ For plain concrete.

$= 0.90$ for steel fiber concrete.

f_{cube} = compressive strength of concrete.

a = Depth of compressive zone = $\beta 1$ X.

b = Width of corbel.

Here, $\beta_1 = 0.85$ for fcube Note:

- $\beta 1 = 0.9$, has been recommended in BS8110:1997.
- $\beta 1 = 0.8$, has been recommended in EC-2, part-I:2004.

Thus, equation 3.1 can be modified as

$$C = \alpha f_{cube} \beta_1 \beta b \tag{3.2}$$

Now resolving various forces as shown in figure-1

$$C = T_{rb} + T_{fc} \tag{3.3}$$

Where,

C = Compressive forces.

T_{rb} = Tensile force of bar reinforcement.

T_{fc} = Tensile force of fibrous concrete.

Step 2 Calculation of tensile strength

$$T_{rb} = A_s + f_y \tag{3.4}$$

Where,

A_s = area steel reinforcement provided.

f_y = yield strength of reinforcing bar steel.

Calculating various parameters for T_{fc} :

$$T_{fc} = \sigma_t b(H - e) \tag{3.5}$$

where,

σ_t= Tensile Strength of Fibrous concrete.

H = Total Depth of corbel section

e = overall depth minus the depth of tension zone.

- Calculation of tensile stress developed in fiber during pullout.

$$\sigma_f = \frac{\tau_d F_{be} \frac{1}{2} \pi d_f}{\frac{\pi d_f^2}{4}} = \frac{2 \tau_d F_{be} l}{d_f} \tag{3.6}$$

Henager and Doherty (1976) have considered 2.3 N/mm2 dynamic bond strength of fiber reinforced While Swamy et al.(1993), Lok and Ziao (1999) have used 4.15 N/mm2 . In this model the dynamic bond stress value of 3.6 N/mm_2 was considered.

- Calculation of Tensile stress of fibrous concrete

$$\sigma_t = P_e F_{be} \frac{l_f}{d_f} P \tag{3.7}$$

Where,

σ_t = Tensile strength of fibrous concrete.

Pe = Effective percentage of steel fiber.

P = Percentage of steel fiber.

(l_f/d_f) = aspect ratio of fiber.

F_{be} = Bond efficiency factor of fiber.

= 1.0 , for smooth fiber.

= 1.2 , for duoform fiber.

Step 3 Calculation of the distance between extreme compression fibers to top of tensile stress block of fibrous concrete.

$$e = \frac{\varepsilon_s + 0.0035}{0.0035} x \tag{3.8}$$

Where,

$$\varepsilon_s = \frac{\sigma_f}{E_s} \tag{3.9}$$

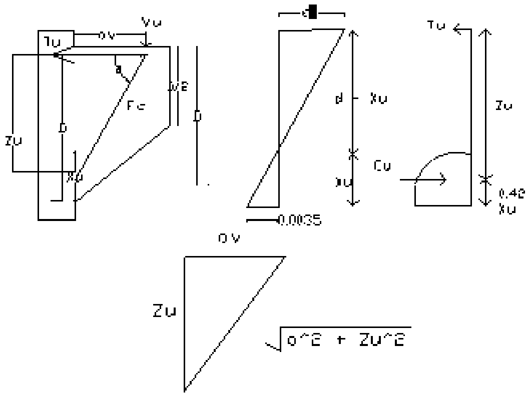


Figure 3.2: Force distribution in corbel section

Where,

σ_f= Tensile stress developed in fiber during pullout.

E_s= Modulus of elasticity of steel.

Now , substituting values of various equation Eq-3.1, Eq-3.4, Eq-3.5, Eq-3.7 ,and Eq-3.8 in Eq-3.3 and simplifying it we get ,

$$\text{C} = T_{fc} + T_{rb}.$$

i.e.

$$\alpha f_{cube} ab = \sigma_t b(H - e) + A_s f_y.$$

$$\alpha f_{cube}(\beta_1 x)b = P_e \frac{l_f}{d_f)} PF_{be} * b(H - e) + A_s f_y. \tag{3.10}$$

Now, modifying the equation to find the depth of neutral axis.

Thus , Neutral axis ,x

$$x = \frac{A_s f_y + \sigma_t bH}{\alpha f_{cube} b\beta + \sigma_t be} \tag{3.11}$$

Step 4 Calculate the ultimate load capacity(shear strength) of corbel.

Substituting value of X from Eq.3.11. And from figure 3.1 , the value of cos β , into Eq.3.2. Here, $\alpha = \sin\beta\cos\beta$. $sin\beta = fracz_u\sqrt{a^2 + \ Z_u^2}, cos\beta = \frac{a}{\sqrt{a^2+Z_u^2}}$ Thus, substituting all the values we get,

$$Vu = k\frac{f_c uba_v(d - \frac{1}{2}\beta_x)}{{a_v}^2 + (d - \frac{1}{2}\beta_x)^2} \tag{3.12}$$

Where,

V_u = Ultimate load carrying capacity of corbel.

k = 1.0 for av/d > 1.0 , 0.5 % < Ast < 1.0 %

= 1.3 for av/d < 1.0 , Ast < 0.8 %

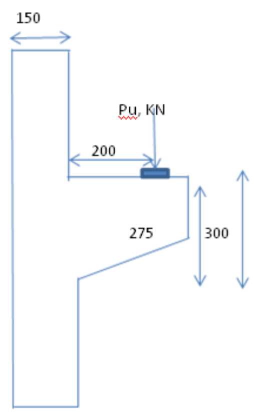
= 1.64 for 0.5 < av/d < 1.3 , 0.5 % < Ast >= 1.4 %

For the particular section the ultimate shear capacity of corbel was found 160 KN while using the proposed method it was calculated as 148 KN.

Thus,

$\frac{V_{u,test}}{V_{u,calculated}} = 0.925$

3.2.3 Design example for proposed method.



INPUT DATA :-

Dimension column			
b	=	150	mm
d	=	150	mm
Dimension of corbel			
b	=	150	mm
h	=	300	mm
d	=	275	mm
shear span (a_v)	=	200	mm
Various parameters			
	=		
	=		
a_v / d	=	0.727273	
Effective percentage of fibres (P_e)	=	0.8	%
Percentage by volume of fibers (p)	=	1	%
Area of steel provided	=	1.2	%
Bond efficiency factor (F_{be})	=	1	smooth surfaced fibers
α	=	0.9	
Fiber length	=	60	mm
Dynamic bond stress between fiber and the matrix	=	3.6	KN/m^2
Fiber diameter (d_f)	=	0.7	mm
ratio of Fiber length to Fiber diameter	=	60	
Tensile stress developed in fiber during pullout	=		
f_y	=	415	
f_c'	=	24.68	24.7
E_s	=	200000	

STEP 1 Calculation of compressive forces			
C = a * f_c' (0,75 c) b	=	0.049	
C = $T_{rb} + T_{fc}$			
step 2 Calculation of tensile strength			
$T_{rb} = A_s\ f_y$	=	49800	
calculation of various parameters for T_{fc}			
σ_f	=	432	N/mm^2
σ_t	=	173	N/mm^2
step 3 calculation of the distance			
between extreme compression fiber to			
top of tensile stress block of fibrous concrete			
s = σ_f / E_s	=	0.002	
e = s +0.0035 /0.0035 c	=	1.617	x
x = $A_s\ f_y$ + σ_t b D / [a f'c b (β) + st b (e)	=	50.13	mm
Vu =	=	90.23	KN
Step 4 Ultimate shear capacity			
			FACTOR
Vu (KN) =	=	**90.23**	**1**
	=	**117.3**	1.3
	=	**148**	**1.64**

3.3 Design Of Corbels or Brackets by IS-456 :

The corbel or bracket is a shorter cantilever projection which supports a load bearing member which is below the corbel. In the case of corbel the av/d ratio is less than unity and the depth at the outer edge of the contact area should not be less than one half of the depth at the root of the corbel. The failure of the corbel is principally by development of vertical crack starting from the reentrant corner proceeding towards its lower fiber. Bearing failure can also occur if adequate bearing area is not provided.

3.3.1 Theory

The design of corbels comes under the case of shear design with av/d ratio less than 0.6. The design of corbel was not included in earlier Code IS:456-1978. Now the provision has been incorporated in IS 456-2000 which is almost the same as given in BS:8110.

The design can be done as follows:

The width "b" can of bracket shall be decided from the practical considerations and the size of the

Bearing plate based on bearing strength.

$$Area of bearing plate = \frac{V_u}{\sigma_{cbr}} \tag{3.13}$$

Where,

σ = bearing strength of concrete = $0.45 f_{ck}$

a. Section Design : design shear V_{uD} = load carried by corbel.

(1) Width of corbel b = width of column.

(2) Depth shell be decided by following two criteria :

(3) For corbels ,

$$\frac{a_v}{d} < 0.6 \tag{3.14}$$

(4) For shear ,

$$d \geq \frac{V_u}{\tau_{uc.max}.b} \tag{3.15}$$

Total depth D = d + effective cover

Depth at free edge > D/2

(1) Main steel : A_{st}

The basis of designing horizontal main steel at top shall be, that it behaves as a tie member in a simple triangular tie-strut system shown in Fig. 3.3 in which the force in the inclined compression strut Fc , the eccentricity " a", the force Vu and the depth "d" governs the design . From force triangle ,

$$V_u = F_c sin\beta \tag{3.16}$$

$$F_c = 0.36 f_{ck} b(X_u cos\beta) \tag{3.17}$$

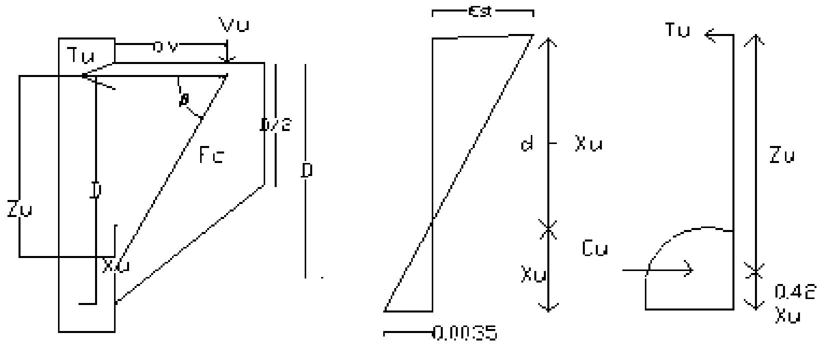


Figure 3.3: Tie and Strut system under vertical load

$$cot\beta = fracaZ_u, sin\beta = fracz_u\sqrt{a^2 +\ Z_u^2}, cos\beta = \frac{a}{\sqrt{a^2 + Z_u^2}} \tag{3.18}$$

$$V_u = F_c sin\beta \tag{3.19}$$

Thus ,

$$V_u = 0.36 f_{ck} b (X_u cos\beta) sin\beta \tag{3.20}$$

Substituting the value of cosβ and sinβ from Eq.3.17

$$V_u = 0.36 f_{ck} b X_u \frac{a z_u}{\sqrt{a^2 + \ Z_u^2}} \tag{3.21}$$

$$Z_u = d - 0.42 X_u. \tag{3.22}$$

$$T_u = C_u = F_c cos\beta = (V_u/sin\beta) X cos\beta = V_u cot\beta = V_u * (a/Z_u) \tag{3.23}$$

Using Eq. 3.20 and 3.21 , the depth of N.A.(x_u) will first be obtained by trial and error procedure .Having known X_u ,the tension in horizontal steel will be obtained by using eq.3.22.

The area of steel will be given by :

$$A_{st} = \frac{T_u}{f_{st}} \tag{3.24}$$

Where,

f_{st} = stress in steel corresponding to E_s to be obtained from the strain diagram shown in Fig.3.3, by using the following relation :

The main tension steel shall not be less than 0.4 % and not more than 1.3 % of the section at the face of the supporting member and should be adequately anchored. Adequate anchoring is effected either by welding the reinforcement at the face of the corbel or by bending back the bars to form a loop. In either case the bearing area of the load should not project beyond the straight portion of the bars of main steel.

b. **Shear Design :**

Shear to be resisted by horizontal stirrups of area Ash is given by :

$V_{ush} = V_u - V_{uc}$

Where ,

V_{uc} is that corresponding to (ζ_{uc} * 2d / a) and ζ_{uc} is corresponding to A_{st}

Vertical spacing of these stirrups is given by :

s = 0.87 f_y A_{sh} d / V_{ush}

Also, total area of horizontal ties shall not be less than $A_{st}/2$.

$$\frac{A_{sh}}{s}\frac{2d}{3} \nleq \frac{A_{st}}{2} \tag{3.25}$$

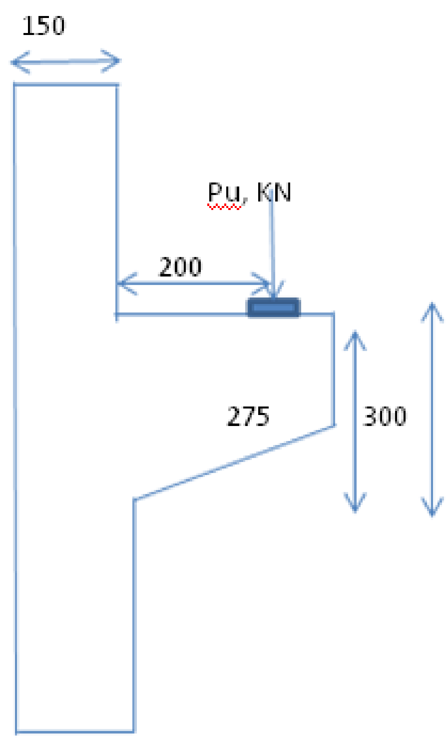
Therefore ,

$$s \gneqq \frac{A_{sh}\frac{4d}{3}}{A_{st}} \tag{3.26}$$

These will be provided over a distance 2d/3 from A_{st}.

c. **Development Length**

Main steel A_{st} shall be anchored an anchorage length equal to L_d on both sides of the face of support.



3.3.2 Spreadsheet for calculation of Corbel section using IS-456[2]

DATA :-				
Factored load	P	=	160	KN
Distance of load from the face of Col.	a_v	=	200	mm
Size of column	B	=	150	mm
	D	=	150	mm
Concrete grade	F_{ck}	=	24.68	N/mm^2
Steel grade	F_y	=	415	N/mm^2

Step 1	Dimensioning of corbel			
	Area of bearing plate	=	11801.73	
	bearing strength of concrete $\sigma_{bcr} = 0.45\ f_{ck}$	=	11.106	
	τ_{ucmax}	=	3.1	N/ mm^2
	width of corbel = b mm	=	150	mm
	depth of corbel			
	d ¿= a/ 0.6	=	333.333333	mm
	d ¿= V_u/τ_{ucmax} * b	=	344.086022	mm
	d	=	275	mm
	X_u	=	127.5	mm
	Z_u	=	246.45	mm
	V_u	=	124.711805	KN
step : 2	**Main steel**			
	T_u	=	194.765673	KN
	f_{st}	=	809.803922	
	A_{st}	=	240.509668	mm^2
or	P_t	=	0.8	%
	A_{st} provided	=	330	mm^2
	min A_{st} provided	=	165	
	¿0.4% and ¡1.3 %			
			nos	mm dia bars
	thus A_{st}	=	3	12
			A_{st}	339.12
step : 3	**Shear design**			
	F_{ck}	=	25.42	
	P_t	=	0.4	
	τ_{uc}	=	0.584	N/mm^2
	V_{uc}	=	1.5768	N/mm^2
	$\tau_{uc,max}$	=	3.1	N/mm^2
	V_{uc}	=	63.86	KN
	spacing for 8 mm 2 legged stirrups			
	A_{sv}	=	100.48	
	S	=	604.63	mm
	min of A_{st}	=	226.08	mm^2
thus	S	=	106	mm
step : 4	**Development length**			
	L_d			
	L_d = + es	=	780	mm
	4 T_{bd}			

3.4 ACI-318-08 Method.

3.4.1 General :

Since the corbel is cast at different time with the column element then the cracks occurs in the interface of the corbel and the column. To avoid the cracks we must provide the shear friction reinforcement perpendicular with the cracks direction.
ACI code uses the shear friction theory to design the interface area.

3.4.2 Shear Friction Theory

In shear friction theory we use coefficient of friction μ to transform the horizontal resisting force into vertical resisting force. The basic design equation for shear reinforcement design is :

$$\Phi V_n = V_u \tag{3.27}$$

where :

V_n = nominal shear strength of shear friction reinforcement

V_u = ultimate shear force

φ = strength reduction factor ($\varphi = 0.85$)

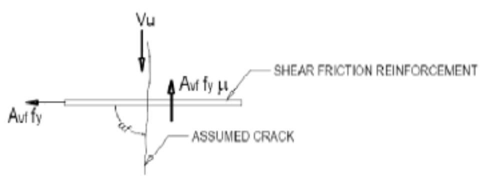


Figure 3.4: Shear Friction theory

The nominal shear strength of shear friction reinforcement is :

Sr no	As	PRIMARY	CLOSED	
		REINFORCEMENT	STIRRUP	
			Ah	LOCATION
1	As¿=2/3Avt+An	As=2/3Avt+An	Ah=1/3Avt	2/3d
2	As¿=Af+An	As=Af+An	Ah=1/2Af	2/3d

METHOD	COEFFICIENT
	OF FRICTION
	M
Concrete Cast Monolithic	1.4λ
Concrete Placed Against Roughened	1.0λ
Hardened Concrete	
Concrete Placed Against Unroughened	0.6λ
Hardened Concrete	
Concrete Anchored To Structural Steel	0.7λ

where :

V_n = nominal shear strength of shear friction reinforcement

A_{vf} = area of shear friction reinforcement

f_y = yield strength of shear friction reinforcement

μ = coefficient of friction

3.4.3 Coefficient of Friction Method

The value of λ is :

$\lambda = 1.0$ normal weight concrete

$\lambda = 0.85$ sand light weight concrete

$\lambda = 0.75$ all light weight concrete

The ultimate shear force must follows the following conditions :

$$Vu = \varphi(0.2f'_c)b_w d \tag{3.28}$$

where :

Sr no	As	PRIMARY	CLOSED	
		REINFORCEMENT	STIRRUP	
			Ah	LOCATION
1	As¿=2/3Avt+An	As=2/3Avt+An	Ah=1/3Avt	2/3d
2	As¿=Af+An	As=Af+An	Ah=1/2Af	2/3d

V_u = ultimate shear force (N)

f'_c = concrete cylinder strength (MPa)

b_w = width of corbel section (mm)

d = effective depth of corbel (mm)

3.4.4 STEP – BY – STEP PROCEDURE

The followings are the step – by – step procedure used in the shear design for corbel , as follows :

a. Calculate the ultimate shear force Vu.

a. Check the ultimate shear force for the following condition, if the following condition is not achieved then enlarge the section.

$$Vu = \varphi(0.2f'_c)b_w d \tag{3.29}$$

a. Calculate the area of shear friction reinforcement A_{vf}.

a. Calculate the shear strength V_u as described in step 3. The design must be follows the basic design equation as follows :

$$\Phi V_n = V_u \tag{3.30}$$

3.4.5 Flexural Design of Corbel

3.4.6 General

The corbel is design due to ultimate flexure moment result from the supported beam reaction $V_u$and horizontal force from creep and shrinkage effect N_u.

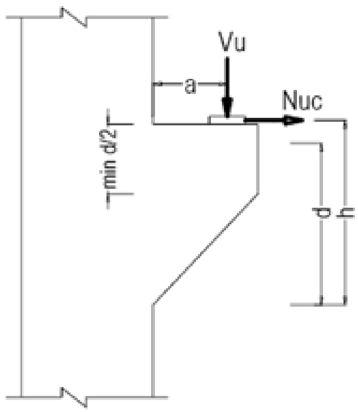


Figure 3.5: Design Force of Corbel Tension Reinforcement

3.4.7 Design of corbel :

The ultimate horizontal force acts in the corbel **Nuc** is result from the creep and shrinkage effect of the pre-cast or pre-stressed beam supported by the corbel.
This ultimate horizontal force must be resisted by the tension reinforcement as follows :

$$A_n = \frac{N_{uc}}{\phi f_y} \tag{3.31}$$

where :

A_n = area of tension reinforcement

N_{uc} = ultimate horizontal force at corbel

f_y = yield strength of the tension reinforcement

φ = strength reduction factor ($\varphi = 0.85$)

Minimum value of N_{uc} is 0.2 V_{uc}.

The strength reduction factor is taken 0.85 because the major action in corbel is dominated by shear.

Flexural Reinforcement

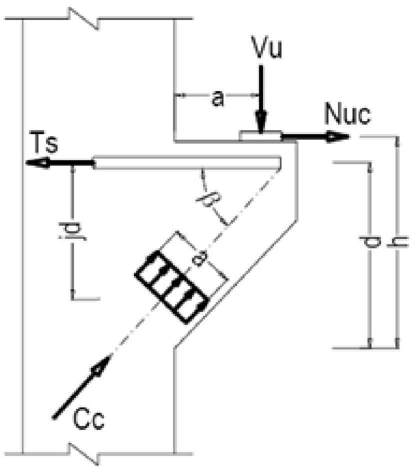


Figure 3.6: Ultimate Flexure Moment at Corbel

The ultimate flexure moment **Mu** result from the support reactions is :

$$Mu = V_u(a) + N_{uc}(h - d) \quad (3.32)$$

where : M_u = ultimate flexure moment V_u = ultimate shear force a_v = distance of V_u from face of column N_{uc} = ultimate horizontal force at corbel h = height of corbel d = effective depth of corbel The resultant of tensile force of tension reinforcement is :

$$T_f = A_f f_y \quad (3.33)$$

where : T_f = tensile force resultant of flexure reinforcement A_f = area of flexure reinforcement f_y = yield strength of the flexure reinforcement The resultant of compressive force of the concrete is :

$$C_c = 0.85 f_c' ba(cos\beta) \tag{3.34}$$

where : C_c = compressive force resultant of concrete f_c' = concrete cylinder strength b = width of corbel a = depth of concrete compression zone The horizontal equilibrium of corbel internal force is :

$$\Sigma \text{ H} = 0 \Rightarrow C_c = T_s \ 0.85 f_c' ba(\cos\beta) = A_f f_y$$

$$a = \frac{A_f f_y}{0.85 f_c' b \cos\beta} \tag{3.35}$$

The Flexure Reinforcement area is :

$$A_f = \frac{M_u}{\phi f_y (d - \frac{a}{2})} \tag{3.36}$$

$$A_f = \frac{M_u}{\phi f_y (d - \frac{\frac{A_f f_y}{0.85 f_c' b \cos\beta}}{2})} \tag{3.37}$$

$\cos\beta$ value can be calculated based on the $\tan\beta$ value as follows :

$$\tan\beta = \frac{jd}{d} \tag{3.38}$$

where :

a = distance of V_u from face of column

jd = lever arm

Based on the equation above we must trial and error to find the reinforcement area A_f.

For practical reason the equation below can be used for preliminary :

where :

A_f = area of flexural reinforcement

M_u = ultimate flexure moment at corbel

Table 3.2: Distribution of Corbel Reinforcements Closed

Sr no	As	PRIMARY	CLOSED	
		REINFORCEMENT	STIRRUP	
			Ah	LOCATION
1	As?2/3Avt+An	As=2/3Avt+An	Ah=1/3Avt	2/3d
2	As?Af+An	As=Af+An	Ah=1/2Af	2/3d

f_y = yield strength of the flexural reinforcement

φ = strength reduction factor (φ=0.9)

d = effective depth of corbel

jd = 0.85 (assume)

3.4.8 Distribution of Corbel Reinforcements

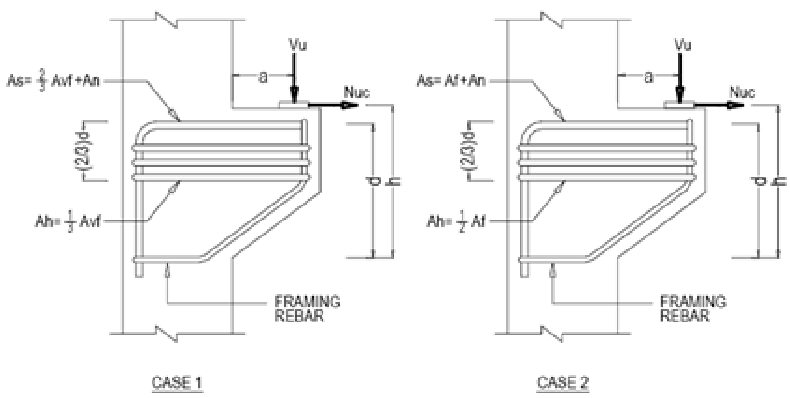


Figure 3.7: Distribution of Corbel Reinforcements

From the last calculation we already find the shear friction reinforcement Avf, tension reinforcement An and flexural reinforcement Af. We must calculate the primary tension reinforcement As based on the above reinforcements.

where :

A_s = area of primary tension reinforcement

A_{vf} = area of shear friction reinforcement

A_n = area of tension reinforcement

A_f = area of flexure reinforcement

A_h = horizontal closed stirrup

d = effective depth of corbel

The reinforcements is taken which is larger, case 1 or case 2, the distribution of the reinforcements is shown in the figure above.

3.4.9 Limits of Reinforcements

The limits of primary steel reinforcement at corbel design is :

$$\rho = \frac{A_s}{bd} \geq 0.04\frac{f_c'}{f_y} \tag{3.39}$$

where :

As = area of primary tension reinforcement

b = width of corbel

d = effective depth of corbel

The limits of horizontal closed stirrup reinforcement at corbel design is :

$$A_h \geq 0.5(A_s - A_n) \tag{3.40}$$

3.4.10 Step by Step procedure

The followings are the step – by – step procedure used in the flexural design for corbel , as follows :

a. Calculate ultimate flexure moment Mu based on eq.3.4.5 .

$$M_u = V_u a + N_{uc}(h - d) \tag{3.41}$$

Table 3.3: Distribution of Corbel Reinforcements Closed

Sr no	As	PRIMARY	CLOSED	
		REINFORCEMENT	STIRRUP	
			Ah	LOCATION
1	As?2/3Avt+An	As=2/3Avt+An	Ah=1/3Avt	2/3d
2	As?Af+An	As=Af+An	Ah=1/2Af	2/3d

b. Calculate the area of tension reinforcement An based on eq.3.4.4.

c. Calculate the area of flexural reinforcement **Af** based on eq.3.4.9.

$$A_f = \frac{M_u}{\phi f_y(0.85d)} \tag{3.42}$$

a. Calculate the area of primary tension reinforcement **As** and stirrup reinforcement **An**.

b. Check the reinforcement for minimum reinforcement based on eq.3.4.13 and eq.3.4.14.

3.4.11 Spreadsheet for calculation of Corbel section using ACI-318[[1]].

Vertical load	V_U	=	139000	N
Vertical live load	V_L		25	kips
Horrizontal load	N_{uc}	=	0	N
distance of load	a_v	=	200	mm
from the face of column				
size of column	B	=	150	mm
	D	=	150	mm
	h	=	300	mm
concrete cover		=	25	mm
	d	=	275	mm
concrete grade	M	=	k-300	
concrete compressive strength	f_{ck}	=	25.48	
concrete cylinder strength	$f'_c = 0.83$ x f_c	=	21.1484	Mpa
steel grade	f_y=	=	415,000	Mpa

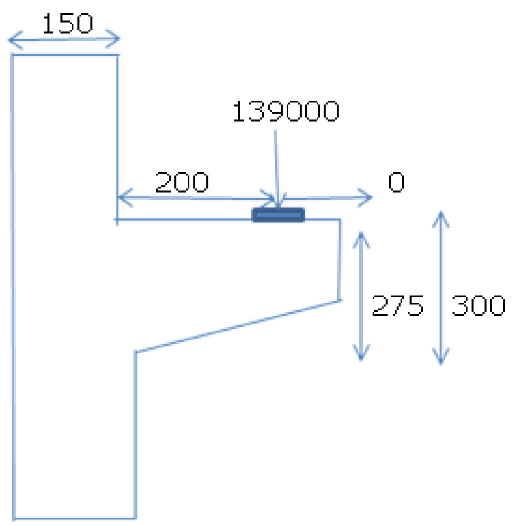


Figure 3.8: Reinforced concrete Corbel dimensions

Step 1			Factored loads		
	V_U =		139000	N	
	N_{uc}	=	27800	N	
	M_u	=	V_u a + N_{uc} (h - d)	(ACI-318(05)/cl. 11.9.3/pg 177	
	M_u	=	28495000	N-mm	
N_{uc}/V_u =	0.2	= ¿		0.2 (ACI-318(05)/cl. 11.9.3.4/pg 178	CHECK
Step 2			Preliminary Corbel size		
	V_U =	=		(0.85 f'c) A1	
		=	0.85		
	Bearing plate width	=	V_U / (0.85 f'c) A1		
			60.64684	mm	
			60.7	mm	
	Area of bearing plate	=	2 + 1/2 (bearing plate width)		
		=	32.35		
Step 2	Determine depth of bracket for shear				
	max V_n =	=	0.2 f'c bud	¡=	800 b_u d
	max V_n	=	800		
	Min d	=	V_u / b (max V_n)		
			1362.75		
			53.65157	mm	
Assume	if h	=	15		
	d	=	53.65157	mm	
	check for shear span to depth ratio				
	a_v / d	=	0.666667	¡	1
			shear friction method can be applied		
Step 3	Determine flexural reinforcement				
	M_u	=	28495000	N-mm	
Requires	A_f	=	M_U / f_y (0.85*d)		
	A_f	=	326.3822	mm2	
	Required ρ	=	0.0035		
	Min ρ	=	0.04 *(f'_c / f_y)	(ACI-318(05)/cl. 11.9.5/pg 178	
	Min ρ	=	0.002		

Step 4	Determine the Shear-friction reinforcement			
	A_{vf}	=	V_u / f_y	(ACI-318(05)/cl. 11.7.4.1/pg 172
		=	1.4	(ACI-318(05)/cl. 11.7.4.3/pg 173
	A_{vf}	=	281.46	mm^2
Step 5	Determine Main tension reinforcement As			
Calculation for	the required area for flexure			
	M_u	=	V_u a + N_{uc} (h - d)	(ACI-318(05)/cl. 11.9.3/pg 177
	M_u	=	28495000	N /mm^2
Requires	A_f	=	326.3822	mm^2
Step 6	Determine additional reinforcement An for axail tension			
	A_n	=	N_{uc} / f_y	
	A_n	=	78.80936	mm^2
	Requirement for Main steel As			
	A_s	=	2/3 A_{vf} + An	(ACI-318(05)/cl. 11.9.3.5/pg 178
	A_s	=	266.45	mm^2
	or			
	A_s	=	A_f +A_n	
	A_s	=	405.1915	mm^2
provide	suitable reinforcement			
	use	=	3 - ♯16	
	Area provided =	=	603	mm^2
Step 7	Design of closed stirrups or ties			
	Required A_h	=	0.5 (A_s - A_n)	(ACI-318(05)/cl. 11.9.4/pg 178
	Thus	=	187.6406	mm^2
	A_h	=	1/3 * A_{vf}	
	A_h	=	93.82	mm^2
	provide A_h	=	187.6406	mm^2
provide	suitable reinforcement			
	use	=	3 - ♯10	
	Area provided =	=	471	mm^2

3.5 Euro code (EC 2) part 1 :2004 method:

3.5.1 General

3.5.2 Code philosophy

The Eurocode is less empirical and more logical in its approach. For example, variables such as partial factors for materials are shown within formulae, rather than being "built in" as part of an obscure number. If one wishes to go into greater detail, there are appendices to the code that give derivation formulae for items such as creep coefficients and shrinkage strains, which are most helpful when attempting to automate the design process.

EC2 makes no attempt to be a design "guide"; it is a code giving general rules. There are no simplified tables of moment or shear factors for example, as one would be expected to look for these in separate design guides or standard textbooks.

In appears that, EC2 has great potential of being accepted. Due to it's superiority and economic advantages , EC2 will be universally recognized.

3.5.3 Strut-and-tie models

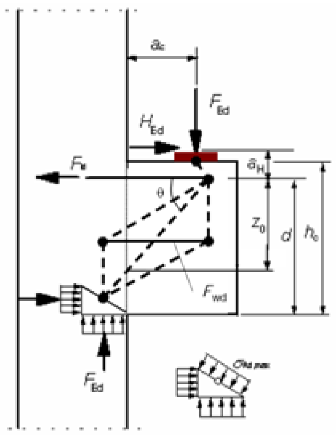


Figure 3.9: Typical node model for corbel section.

3.5.4 Partial factors for materials:

a. Partial factors for materials for ultimate limit states, γ_c and γ_s should be used.

The recommended values for 'persistent & transient' and 'accidental, design situations are given in Table 3.4 . These are not valid for fire design for which reference should be made to EN 1992-1-2. For fatigue verification the partial factors for persistent design situations given in Table 3.4 are recommended for the values of $\gamma_{c,fat}$ and $\gamma_{c,fat}$.

Table 3.4: Partial factors for materials for ultimate limit states

design	γ_c	γ_s	γ_s
situation	for	reinforcing	prestressing
	concrete	steel	steel
persistent			
and	1.5	1.15	1.15
transient			
accidental	1.2	1	1

Design situations γ_c for concrete γ_s for reinforcing steel γ_s for prestressing steel

b. The values for partial factors for materials for serviceability limit state verification should be taken as those given in the particular clauses of this Eurocode.

Note: The values of γC and γS in the serviceability limit state for use in a Country may be found in its National Annex. The recommended value for situations not covered by particular clauses of this Eurocode is 1.0.

c. Lower values of γC and γS may be used if justified by measures reducing the uncertainty in the calculated resistance.

3.5.5 Analysis of corbel with strut and tie models:

a. Strut and tie models may be used for design in ULS of continuity regions and for the design in ULS and detailing of discontinuity regions. In general these extend up to a distance h (section depth of member) from the discontinuity. Strut and tie models may also be used for members where a linear distribution within the cross section is assumed, e.g. plane strain.

b. Verifications in SLS may also be carried out using strut-and-tie models, e.g. verification of steel stresses and crack width control, if approximate compatibility for strut-and-tie models is ensured (in particular the position and direction of important struts should be oriented according to linear elasticity theory)

c. Strut-and-tie models consist of struts representing compressive stress fields, of ties representing the reinforcement, and of the connecting nodes. The forces in the elements of a strut-and-tie model should be determined by maintaining the equilibrium with the applied loads in the ultimate limit state. The elements of strut-and-tie models should be dimensioned according to the rules given in 6.5.

d. The ties of a strut-and-tie model should coincide in position and direction with the corresponding reinforcement.

e. Possible means for developing suitable strut-and-tie models include the adoption of stress trajectories and distributions from linear-elastic theory or the load path method. All strut-and-tie models may be optimized by energy criteria.

For members with shear reinforcement, the shear resistance, V_{Rd} can be calculated as:

$$V_{Rd,max} = \alpha_{cw} b_w Z V_1 \frac{f_{cd}}{\cot\theta + \tan\theta} \tag{3.43}$$

where:

A_{sw} = the cross-sectional area of the shear reinforcement

s = spacing of the stirrups

f_{cd}= is the design value of the concrete compression force in the direction of the longitudinal member axis

$S\nu_1$ = a strength reduction factor for concrete cracked in shear

α_{cw} = a coefficient taking account of the state of the stress in the compression chord

Z = is the inner lever arm, for a member with constant depth , corresponding to the bending moment in the element under consideration. In the shear analysis of reinforced concrete without axial force, the approximate value Z = 0.9 d may normally be used

Θ = is the angle between the concrete compression Strut and the beam axis perpendicular to the shear force

Note 1: The value of ν_1 and α_{cw} for use in a Country may be found in its National Annex. The recommended value of ν_1 is ν (see EC-2/pg 87/Expression (6.6N)).

$$\vartheta = 0.6[1 - \frac{f_{ck}}{250}] \tag{3.44}$$

Note 2: If the design stress of the shear reinforcement is below 80% of the characteristic yield stress f_{yk}, ν_1 may be taken as:

$$\vartheta = 0.6 for f_{ck} \leq 60Mpa \tag{3.45}$$

$$\vartheta = 0.9[1 - \frac{f_{ck}}{200}] > 0.5 for f_{ck} \geq 60Mpa \tag{3.46}$$

Note 3: The recommended value of αcw is as follows:

1 for non-prestressed structures

$$(1 + \frac{\sigma_{cp}}{f_{cd}}) for 0 < \sigma_{cp} \leq 0.25 f_{cd} 1, 25 for 0.25 f_{cd} < \sigma_{cp} \leq 0.5 f_{cd} \tag{3.47}$$

$$2.5(1 + \frac{\sigma_{cp}}{f_{cd}}) for 0.5 f_{cd} < \sigma_{cp} \leq 1.0 f_{cd} \tag{3.48}$$

where:

σ_{cp} is the mean compressive stress, measured positive, in the concrete due to the design axial force.This should be obtained by averaging it over the concrete section taking account of the reinforcement.The value of σ_{cp} need not be calculated at a distance less than $0.5d$ cot θ from the edge of the support.

The maximum effective cross-sectional area of the shear reinforcement, Asw,max, for cotθ =1 is given by:

$$\frac{A_{sw,max}}{f_{ywd}} b_w s \leq \frac{1}{2} \propto_{cw} \vartheta_1 f_{cd} \tag{3.49}$$

3.5.6 Forces in tie :-

To calculate forces in tie member following expression can be used:

$$F_{td} = F_{sd} \frac{a_c}{Z_O} + H_{sd} \frac{a_H + Z_O}{Z_O} \tag{3.50}$$

Where ,

F_{td} is the design value of the tensile force in the longitudinal reinforcement

F_{sd} is the design value of the concrete compression force in the direction of the longitudinal member axis.

H_{sd} is the design value of the concrete compression force in the direction perpendicular to the longitudinal member axis.

a_c = distance of line of action of vertical force from the face of column.

a_H =distance between the lever arm and the tensile tie formed due to loading.

Area of steel required for tie can be calculated as ,

$$A_{st} = \frac{F_{Td}}{F_{yd}} \tag{3.51}$$

Where,

A_{st} = area of steel required for tie .

F_{td} = is the design value of the tensile force in the longitudinal reinforcement

F_{yd} = yield strength of steel used.

3.5.7 Forces in links :-

a. Corbels a_c ¡ z_0 may be designed using strut-and-tie models as described in section 6.5 . The inclination of the strut is limited by 1,0 = tanθ = 2.5.

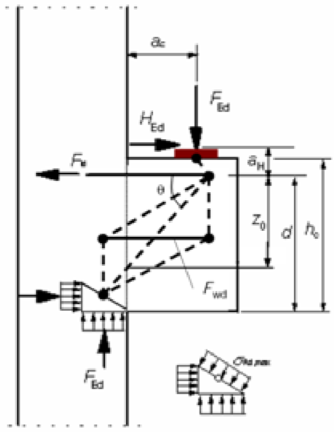


Figure 3.10: Corbel Strut and Tie model

b. If a_c ¡ 0.5 h_c closed horizontal or inclined links with $A_{s,lnk} \geq A_s$,main should be provided inaddition to the main tension reinforcement.

c. If a_c ¿ 0.5 h_c and F_{Ed} ¿ V_{RD}, (see 6.2.2), closed vertical links $A_s \geq k_2\ F_{Ed}/f_{yd}$ should be provided in addition to the main tension reinforcement.

d. The main tension reinforcement should be anchored at both ends. It should be anchored in the supporting element on the far face and the anchorage length should be measured from the location of the vertical reinforcement in the near face. The reinforcement should be anchored in the corbel and the anchorage length should be measured from the inner face of the loading plate.

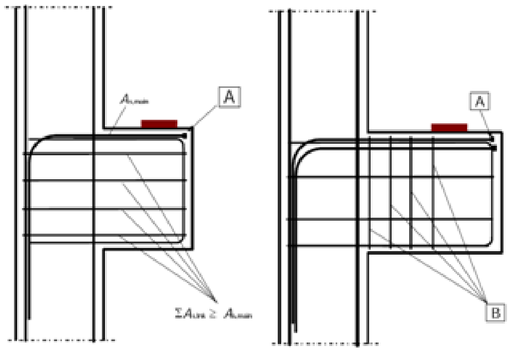


Figure 3.11: Corbel detailing

e. If there are special requirements for crack limitation, inclined stirrups at the re-entrant opening will be effective.

3.5.8 Spreadsheet for calculation of Corbel section using EC-2[3].

Following is the data considered for the calculation of corbel section.

DATA				
Dimension column				
h_w	=	150	mm	
b_w	=	150	mm	
dimension of corbel				
b	=	150	mm	
h	=	300	mm	
concrete cover	=	25	mm	
d	=	275	mm	
shear span (ac)	=	200	mm	
Various parameters				
V_u (live)	=	139	KN	
partial safety factor for concrete = γ_c	=	1.5		EC-2/Ch-2/Table-2.1N/pg-24
partial safety factor for steel = γ_s	=	1.15		EC-2/Ch-2/Table-2.1N/pg-24
partial safety factor for action = γ_G	=	1.35		EC-0/Annex-A1
partial safety factor for action = γ_Q	=	1.5		EC-0/Annex-A1
a_c / h_c		0.727273		
dimension of bearing plate		150 x 150 x 20		mm
Thickness of bearing plate (t)	=	20	mm	
f_y	=	415000		
F_{cd}	=	25.97		
f_{ck}	=	25.97		
diameter of bars used	=	12	mm	

PRELIMINARY design model

a_c / h_c ¡ 0.4	=	0.666667		
dimensioning with	=	500	mm	
h = 2.50 * a_c				

Design using a strut and tie model

effective depth (d)				
= h - cover - 2	=	451	mm	
a_H = cover + 2 + t	=	69	mm	
d / ac = tanθ	=	2.255		
?	=	66.09		
total vertical force F_{sd}	=	209.85	KN	
total horrizontal force				
H_{sd} = 0.200 x F_{sd}	=	41.97	KN	

Concrete strut capacity V_{rdmax}

V_{rdmax}

a_{cw} * b_w * z * f_{cd} / ($\cot\theta + \tan\theta$)

a_{cw}	=	1		
z = 0.9 * d	=	405.9	mm	
cotθ	=	2.5		
tanθ	=	0.4		
?1	=	0.6		
f_{cd}	=	25.97		
Thus ,				
V_{rdmax}	=	327.1414	KN	¿
		The check is verified		

Force in tie

Z_o = d (1 - 0.4 V_{sd} / V_{rdmax})	=	335.2796	mm	
Calculating forces in Tie				
$F_{td} = F_{sd}$ *(a_c /Z_o) + H_{sd} * (a_H + Z_o / Z_o)	=	125.1791	KN	
Area of steel required				
As reqd. = F_{td} / f_{yd}	=	301.6364	mm^2	
		no	diameter	
provide main tension reinforcement	=	3	12	bars
Area provided	=	339.228	mm^2	
		Safe		

Forces in Links

a_c / h_c	=	0.666667	¡	0.5
Inclined closed links are required as shown in EC-2/Annex j.3/Figure - b/pg-224				
Total area required for links (A_{sw})	=	75.4091	mm^2	
provide main tension reinforcement	=	5	8	bars
Area provided	=	251.28	mm^2	
		safe		

check pressure under bearing plate

Mean compressive stress = σ_c = F_{sd} / A_c ¡=				EC-2/ch-6/eq 6.61/pg-106
υ = 1 - F_{ck}/250	=	0.89612		EC-2/ch-6/eq 6.57N/pg-107
$\sigma_c = F_{sd}$ / A_c	=	5.596		
σ_{rdmax} = 0.60 υ F_{cd}	=	13.96334		
		The check is verified		
Reinforcement anchorage / Development length required				
minimum mandrel diameter of main reinforcement				
= 4 *	=	48	mm	EC-2/ch-8/ Table-8.1N/pg-13
Required corbel width b_{req}				
1.5 * 4 + 2 * + 2* cover	=	146	mm	
$\sigma_{sd} = F_{td}$/ As, provided	=	369.0117	N/mm^2	EC-2/ch-8/ Cl-8.4.2/pg-133
f_{bd} = 2.25 a_{cd} η_1 η_2 f_{ctk} / γ_c	=	2.7	N/mm^2	
Basic anchorage length				
lb,reqd = (/4)*(ssd/fbd)				
necessary anchorage	=	410.02	mm	
length provided	=	410	mm	

3.6 Summary

This chapter comprises of design provisions as suggested by various codes IS-456[2] , ACI-318[1] and EC-2[3]. The procedure specified has been studied and described and an example with an spreadsheet program has been incorporated. An analytical model was proposed to evaluate the strength of steel fiber reinforced concrete corbel has also been described and also an spreadsheet program is shown for example.

Chapter 4

Experimental work

4.1 General :-

This chapter describes the experimental work. To better understand the response of reinforced concrete corbel with steel fibers failing under vertical load, twelve reinforced concrete beams were tested at the Structural Laboratory of the Department of Civil Engineering at the College of Nirma Institute of Technology, Ahmedabad. Experimental work related to shear strength and compressive Test has been carried out on the reinforced concrete corbel specimens which were tested under vertical loading conditions.

This chapter describes the objectives of the experimental campaign, details of the reinforced concrete corbel specimens, their construction, material properties, the instrumentation utilized, and the testing procedure that was used. The results of the tests and a discussion are presented in Chapter 5.

4.2 Objective of experimental campaign

The main objectives of the experimental campaign carried out were:

- To study the behavior of reinforced concrete corbel under vertical loading condition with use of steel fibers.
- Strain Measurement & Deflection measurement
- Ultimate Failure Load
- Crack & failure patterns.

4.3 Design of test specimen

The reinforced concrete corbel is subjected to vertical loading placed at an distance a_v from the face of column with variation in percentage of steel fibres. calculate ultimate load. The following data apply :

Data :

factored load	P	=	120	KN
distance of load from the face of Col.	a_v	=	200	mm
size of column	B	=	150	mm
	D	=	150	mm
concrete grade	M	=	25	N/mm^2
steel grade	F_y	=	415	N/mm^2

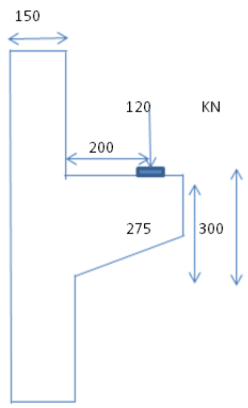


Figure 4.1: A Typical section of corbel

step : 1	**Dimensioning of corbel**				
Bearing length =	size of column	=	150	mm	
Note :	bearing on steel plate is considered				
	bearig strength	=	0.45 Fck	11.25	N/mm2
	width of plate	=	91.1111	mm	
provided	width of plate	=	65	mm	
	Estimation of depth		d		
	$T_c$max	=	3.1	N/mm2	
	T_c	=	3.1	N/mm2	
	d	=	258.065	mm	
	dprovided	=	270	mm	
	Dtotal	=	300	mm	
	Depth at the face	=	150	mm	
Check for strut action					
	a/d	=	0.74074		
			O.K		

step 2:	**Determination of lever arm**			
	k	=	0.134	
	equation for lever arm			
	is quadretic			
	in term of z/d			
	a	=	1	
	b	=	0.846	
	c	=	0.084	
	z/d	=	0.73	
	lever arm z	=	197.265	mm
	depth of N.A x	=	161.631	mm
	x/d	=	0.5986	
	adequate compression steel required			
step 3:	**Resolution of forces**			
	Ft	=	121.6638	KN
	1/2 * Fv Ft	=	60	KN
	design Ft	=	121.6638	KN
	Fh	=	0	KN
step 4:	**Area of tension steel**			
	strain in steel Es	=	0.002347	
	from SP 16 Fs	=	350	N/mm2
	Ast	=	347.6107	mm2
	Dia. of bar provided	=	12	mm
	No. of bar req.	=	3.075113	
	No. of bar pro.	=	3	
	Ast pro.	=	339.12	mm2

step 5:	**Check for mini. And maxi.**			
	% of steel			
	(100 x Ast)/(b x d)	=	0.8373333	
			¡ 0.4	O.K.
			¿ 1.3	O.K.
step 6:	**Area of shear steel**			
	Asv min	=	226.08	mm2
	Dia. of bar provided	=	8	mm
	No. of bar req.	=	4.5	
	No. of bar pro.	=	4	
	spacing	=	45	mm

step 7: **Shear capacity of section**

Tc	=	0.5944	N/mm2
a/d	=	0.7407407	
increased shear strength	=	1.60488	N/ mm2
Tc'	=	1.60488	N/mm2
Vuc	=	64.99764	KN
Vus	=	217.66982	KN
Total shear capacity (V)	=	282.667	KN

step 8 : **Development length**

Ld	=	780	mm

Ld = + es

4 Tbd

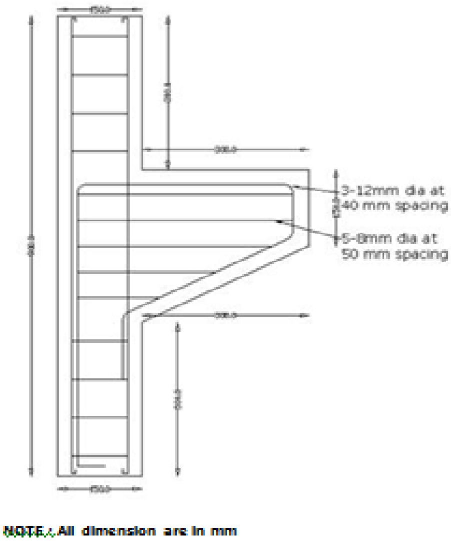


Figure 4.2: Reinforcement detailing of a Reinforced Concrete corbel

4.4 Specimen details

Geometry of reinforced concrete corbel is decided based on extensive literature survey. A total of twelve reinforced concrete corbel specimens of dimension as shown in the line sketch diagram in figure 4.3 which will be tested under vertical loading. The reinforcement cages are also required to be prepared their details are shown in Fig. 4.4 Other details of specimens are summarized in Table 4.1.

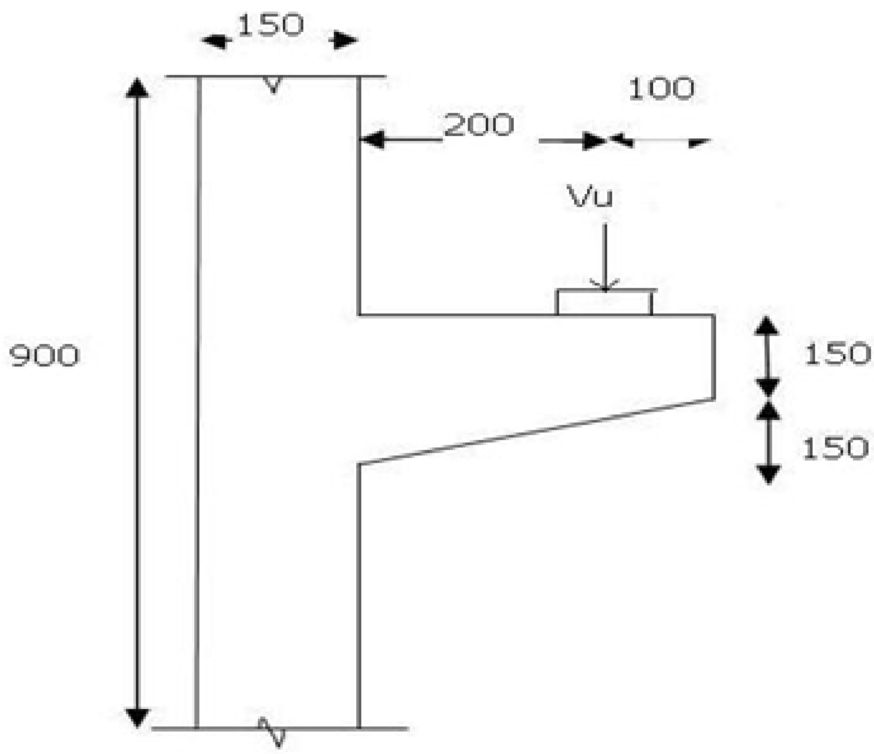


Figure 4.3: line sketch of corbel specimen

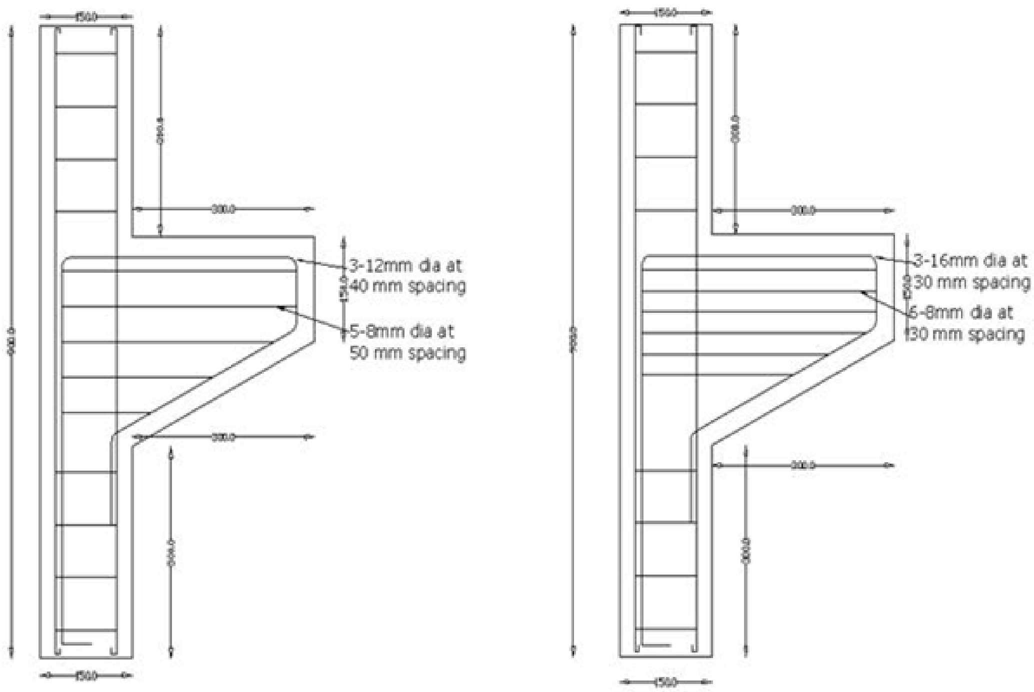


Figure 4.4: Reinforcement detailing of specimen

Table 4.1: Details of various specimen

Test series	specimen mark	av (mm)	d (mm)	D (mm)	b (mm)	Vf %
1	cra1	200	300	150	150	0
	cra2	200	300	150	150	0
	cra3	200	300	150	150	0.5
	cra4	200	300	150	150	0.5
	cra5	200	300	150	150	1
	cra6	200	300	150	150	1
2	crb1	200	300	150	150	0
	crb2	200	300	150	150	0
	crb3	200	300	150	150	0.5
	crb4	200	300	150	150	0.5
	crb5	200	300	150	150	1
	crb6	200	300	150	150	1

4.5 Material properties

4.5.1 Concrete properties

Concrete mix proportion were prepared base on to study the various available research paper. A maximum aggregate size of 20 mm was used in all twelve reinforced concrete corbels. Standard 150 mm x 150 mm cubes were cast with the specimens to obtain the compressive strength of each concrete mix. These cubes were kept under the same environment conditions as the beam specimens until the time of testing. The mix proportions for M-25 concrete mix is presented in Table 4.2.

Table 4.2: Mix proportion of concrete

w/c ratio	sand (kg/mˆ3)	Coarse aggregate	Fine aggregate	Steel fiber
0.41	1.48	1.084	1.626	Ranges from 0, 0.5 to 1.o

Type I Portland cement was used with locally available natural sand having a fineness modulus of 2.62 and a specific gravity of 2.59 was used as a fine aggregate. Crushed granite with maximum size of 20 mm and specific gravity 2.89 was used as a coarse aggregate. Although the same mix was used throughout the programmed, different average compressive strengths were obtain by testing the cube specimen at different ages.

4.5.2 Reinforcing Steel Properties

Details of longitudinal reinforcement and shear reinforcement are summarized in Table 4.3 .Permissible stresses for main reinforcement 12 and 16 mm diameter Grade Fe - 415 (415 MPa) and 8 mm diameter steel bar Grade Fe -415 were used for shear reinforcement.

Table 4.3: Reinforcement detailing of various specimen

group	Specimen mark	Main reinf.	Shear reinf.	% of reinf.	% of fiber
	cra1	3-12	5-8	0.8	0
	cra2	3-12	5-8	0.8	0
1	cra3	3-12	5-8	0.8	0.5
	cra4	3-12	5-8	0.8	0.5
	cra5	3-12	5-8	0.8	1
	cra6	3-12	5-8	0.8	1
	crb1	3-16	6-8	1.2	0
	crb2	3-16	6-8	1.2	0
2	crb3	3-16	6-8	1.2	0.5
	crb4	3-16	6-8	1.2	0.5
	crb5	3-16	6-8	1.2	1
	crb6	3-16	6-8	1.2	1

4.5.3 Details of Form work

For casting specimen form-work are required to be prepared. The dimensions and details of form-work are as shown in Fig. 4.5.

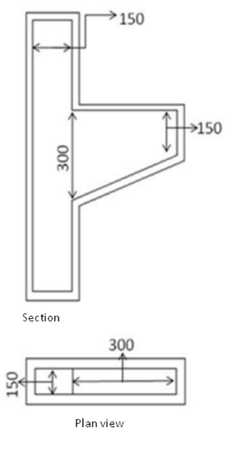


Figure 4.5: Formwork for specimens

Figure 4.6: Reinforcement cage in mold

Figure 4.7: procedure for casting of specimen

4.5.4 Manufacture of the Test Specimens

Twelve corbel were manufactured at the Structural Laboratory of the Department of Civil Engineering, Nirma Institute of Technology, Ahmedabad. Concrete was placed in layers into the timber moulds (Fig. 4.6).

Hand-held mechanical vibrators were used to compact the fresh concrete(Fig. 4.7). Control cubes were compacted in layers on a vibrating table.The concrete components,reinforcement bars, moulds, and procedures were those actually used at that plant. Fig. 4.7 shows some picture of the fabrication of the specimens.

4.5.5 Concrete Compressive Strength

In each specimen, 150 mm x 150 mm cubes were tested in compression at various ages at the time of beam test. The results of the concrete compressive strength tests are given in Table 4.4.

Table 4.4: compressive strength of various specimen

	compressive strength (KN)	
specimen mark	cast date	28 days
cra1	5/12/2009	24.8
cra2	7/12/2009	25.2
cra3	8/12/2009	24.48
cra4	9/12/2009	24.26
cra5	10/12/2009	24.68
cra6	11/12/2009	24.50
crb1	12/12/2009	25.97
crb2	14/12/2009	25.48
crb3	15/12/2009	24.89
crb4	16/12/2009	24.90
crb5	17/12/2009	25.3
crb6	18/12/2009	23.80

4.6 Test setup

Testing of column has been carried out on loading frame in concrete technology laboratory of 1000 KN capacity. As discussed the specimen are required to be tested under vertical point load applied by the help of hydraulic jack at concrete technology laboratory which is of 500 KN capacity. Detail of test set-up is as shown in Fig. 4.10, where the ratio of shear span (a) to depth (d) ratio was 1.51. The load was applied at mid-span of the beam specimen and transferred from jack to steel I beam to supporting plate to knife plate and finally on fix support. As shown in Fig. 4.8 the specimen is to be placed simply supported on either side by some sort of support .

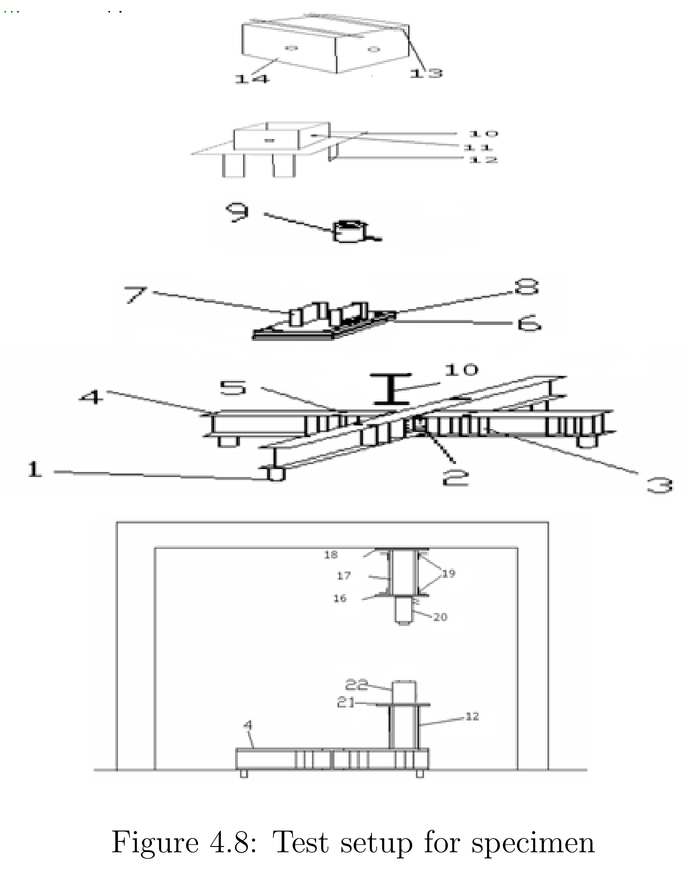


Figure 4.8: Test setup for specimen

To monitor the behaviour of the tested corbel, the applied loads, strains at the

Table 4.5: Parts of test set-up

Description	
1	90mm Out side Diameter hollow pipe for anchoring
	of assembly with concrete flooring, with length
	of 100mm, 4nos
2	Connecting angles ISA 5050, thickness 6mm, length of
	angle is 200mm, connected with 3-18mm bolting,
	number of angel 4, and number of bolts 12.
3	Stiffeners 280*65mm, 18mm th., 100mm c/c, 30nos
4	ISMB 300, total length 4m.
5	19mm drill, 20nos.
6	Connecting angles ISA 5050, thickness 6mm, length of
	angle is 200mm, Connected with 8 mm fillet weld.
	2 nos.
7	ISMB 300, length 300mm, 2nos.
8	Base plate 460*460mm, 2*18mm thick.
9	Hydraulic jack 1000 kN capacity, base diameter
	230mm, piston diameter 155 mm,height at rest
	condition 262 mm.
10	ISMB 200, 3 nos,length 200mm
11	Bolts on 4 side of attachment
12	side plates or fissures attached with box section
13	metal plates at the base of box section with 4 nos 19
	mm dia holes to fix it with the loading frame
14	upper box section for holding column section of
	corbel
15	750 x 750 mm square plate
16	metal plate at the upper end of loading frame to fix
	up the upper box attachment with the frame
17	ISMB-300 for adjusting height according to specimen
	requirement
18	ISMB-300 connected to the main loading frame
19	angle section used to connect ISMB 300 with loading frame
20	upper box section for holding column section of corbel
21	Metal plate at thr bottom box section to fix it up on
	ISMB- 200 to provide fixidity to section at bottom of
	frame
22	box section at the bottom of frame

external surface of concrete, and displacement were measured using different instruments such as Linear Variable Differential Transformers (LVDT), dial gauge and P-3 Electrical strain gauges. Photography and video equipment were also utilized. LVDT is used to measure deflection of the beam from the bottom. LVDT is kept in such a way that it remains in contact with top edge of reinforced concrete corbel.

One day before testing, the test specimens and their respective 150 x 150 mm control cubes were taken out of the moist room and allowed to dry. The nextday, the control cubes were capped and tested in compression to determine the strength of the concrete at that time. The specimen were loaded to failure in a 500 kN capacity, under vertical point loads at 200 mm spacing from the face of column. The specimens were tested to their maximum load-carrying capacity by monotonically increasing the load with the help of hydraulic jack. At each load increment three different measurements were taken: the vertical deflection at free-end of corbel, the strains in the main tensile steel bars, and the strains in the stirrups. At
each loading stage, the crack pattern in the clear span was also observed and recorded. The collapse load is defined as the load that caused failure of the test specimen.

4.6.1 Specimen test

One day before testing, the test specimens and their respective 150 x 150 mm control cubes were taken out of the moist room and allowed to dry. The next day, the control cubes were capped and tested in compression to determine the strength of the concrete at that time. The specimen were loaded to failure in a 500 KN capacity, under vertical point loads at 200 mm spacing from the face of column. The specimens were tested to their maximum load-carrying capacity by monotonically increasing the load with the help of hydraulic jack. At each load increment three different measurements were taken: the vertical deflection at mid-span, the strains in the longitudinal bars, and the strains in the stirrups. At each loading stage, the crack pattern in the clear span was also observed and recorded. The collapse load is defined as the load that caused failure of the test specimen.

4.6.2 Test procedure

General arrangement of testing setup is shown in Fig. 4.9

Figure 4.9: Vetical load applied to specimen

All specimens were loaded to failure. In each corbel section, initially exercised by applying a column load to fix both the column ends and then the load on the cantilever part was applied to ensure that the test set-up and the instrumentation worked properly. The specimen was then unloaded and datum readings were taken. Initially, the corbel was loaded in increments of 10 KN until the load reached at failure of specimen. After failure, each corbel section was photographed to show the crack pattern and the mode of failure. Appendix B contains photographs of all the corbel section after failure. The test results are presented in the next chapter

4.7 Instumentation

Strain at different heights of the column and loads are measured during the experiments by making use of various instruments. Different instruments used in experimental work are as follows:-

1. LVDT (Linear Variable Differential Transducer)
2. Hydraulic Jack
3. P-3 Electrical Strain Gauges

4.7.1 LVDT (Linear Variable Differential Transducer)

LVDT is used to measure displacement of the r.c.c. corbel when the load is being applied on it. LVDT is attached at the position where deflection is to be measured. Attachment of LVDT and digital displacement indicator is shown in Fig. 4.10 and 4.11 respectively. Strength of the LVDT sensor's principle is that there is no electrical contact across the transducer position sensing element for which the user of the sensor means clean data, infinite resolution and a very long life.

Figure 4.10: Attachments of LVDT and digital display

4.7.1 Hydraulic Jack

Hydraulic jack of capacity of 1000 kN is used . it is placed on the top edge of the corbel specimen and then loading is applied on the specimen.

Figure 4.11: Hydraulic Jack

4.7.2 P-3 Electrical Strain Gauges

The Model P3 Strain Indicator and Recorder is a portable, battery-operated instrument capable of simultaneously accepting four inputs from quarter-, half-, and full-bridge strain-gage circuits, including strain-gage-based transducers. Water-resistant grommets in the hinged cover allow the lid to be closed with lead-wires attached. Designed for use in a wide variety of physical test and measurement applications, the P3 functions as bridge amplifier, static strain indicator, and digital data logger . The Model P3 Strain Indicator and Recorder, utilizing a large LCD display for readout of setup information and acquired data, incorporates many unique operating features that make it the most advanced instrument of its kind. An extensive, easy-to-use menu-driven user interface operates through a front-panel keypad to readily configure the P3 to meet your particular measurement requirements. Selections include active input and output channels, bridge configuration, measurement units, bridge balance, calibration method, and recording options, among others.

Data, recorded at a user-selectable rate of up to 1 reading per channel per second, is stored on a removable multimedia card and is transferred by USB to a host computer for subsequent storage, reduction and presentation with third-party software. The P3 can also be configured and operated directly from your PC with a separate software application included with each instrument. Additionally, a full set of ActiveX components is provided for creating custom applications in any language supporting ActiveX.

Figure 4.12: Strain Gauge of 120 Ω with P3 Electrical strain guage

A highly stable measurement circuit, regulated bridge excitation supply, and precisely settable gage factor enable measurements of ±0.1% accuracy and 1 micro-strain resolution. Bridge completion resistors of 120, 350 and 1000 ohms are built in for quarter-bridge operation. Also, input connections and switches are provided for remote shunt calibration of transducers and full-bridge circuits. The P3 operates from an internal battery pack of two readily available D cells. Battery life depends upon mode of operation but ranges up to 600 hours of continuous use for a single channel.

It can also be powered by connection to an external battery or power supply, a USB port on a PC or with an optional external line-voltage adapter.

Chapter 5

Presentations and Discussion of Results

5.1 General

The test results and the effects of various parameters on the shear strength of plain and fiber reinforced concrete corbel has been discussed based on available research and literature data are elaborated in this chapter. The behavior and failure pattern of the test corbels is discussed and the shear strength is tabulated.

The available test results were compared with proposed theory outlined in this report. Comparisons of available test results from previous investigations with predictions from the theory are also given.

The test data available without steel fiber were compared with various code provisions as suggested in IS 456:2000 [1], ACI 318-05 [2], and Eurocode EC2 Part I [3].

Table 5.1: Summary of Test Results

sr. no	av	b	d	av / d	Ast %	vf %	lf/df	fck	Vu,expe.
CRA-1	200	150	275	0.727	0.8	0	60	24.8	—
CRA-2	200	150	275	0.727	0.8	0	60	25.2	127
CRA-3	200	150	275	0.727	0.8	0.5	60	24.48	136
CRA-4	200	150	275	0.727	0.8	0.5	60	24.26	144
CRA-5	200	150	275	0.727	0.8	1	60	24.68	160
CRA-6	200	150	275	0.727	0.8	1	60	24.5	155
CRB-1	200	150	275	0.727	1.2	0	60	25.97	132
CRB-2	200	150	275	0.727	1.2	0	60	25.48	139
CRB-3	200	150	275	0.727	1.2	0.5	60	24.89	143
CRB-4	200	150	275	0.727	1.2	0.5	60	24.9	166
CRB-5	200	150	275	0.727	1.2	1	60	25.3	172
CRB-6	200	150	275	0.727	1.2	1	60	23.8	144

5.2 Test results

5.2.1 Behaviour of Test Corbels

All the corbels failed in Diagonal shear. A summary of experimental results are given in Table 5.1. Complete details are given in Appendix A.

The behavior of all tests Corbel was similar. Initially diagonal shear cracks were observed at the point of load applied on corbel which gradually develops and then travel towards the column-corbel junction. Subsequently, the cracks at the corbel-column junction were observed . However, the cracks at the junction progresses gradually towards the centre of the column with increase in load until the failure of the Corbel.

Crack patterns and failure modes of all specimens photographs are given in 5.3.

5.2.2 Effects of Test Parameters

The effect of test parameter on the shear strength is discussed below

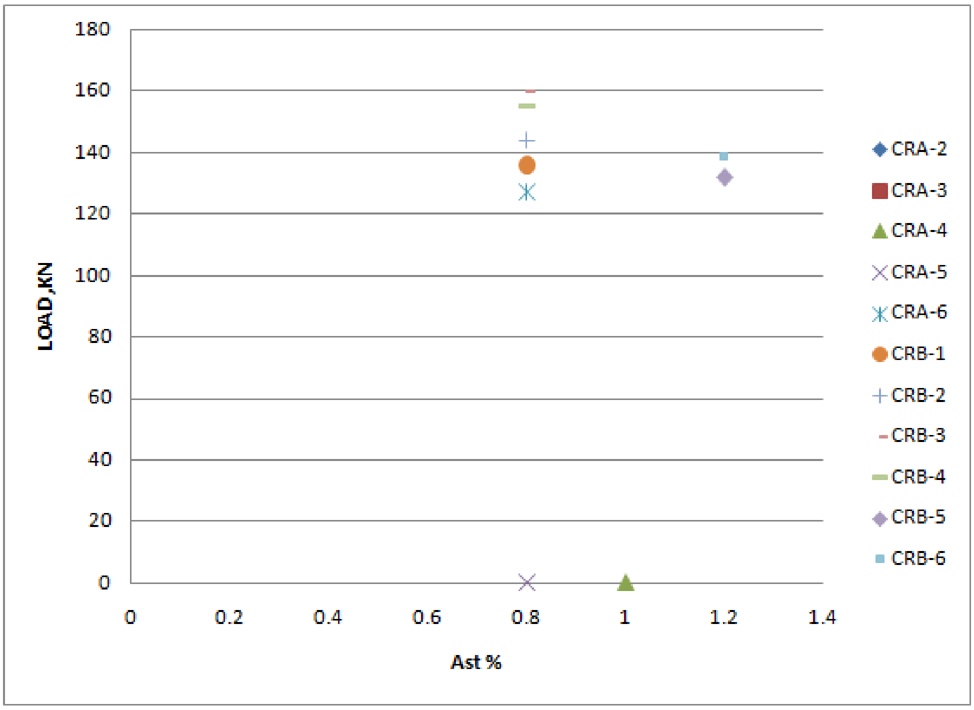


Figure 5.1: Increase in shear strength at same tensile Reinforcement with variation of steel Fiber percentage

5.2.3 Increase in shear strength based on increase in fiber percentage:

It shows, trend of increasing in shear strength with increase in the percentage of Steel Fiber with same tensile reinforcement

5.2.4 Load Vs Defelction results comparison.

Fig. 5.2 shows the Load(shear strength) versus free end deflection curve for specimen which are typical for the test Corbels . Complete test data of free end deflection of corbel specimen are described in Appendix A.

Table 5.2: Load or Shear Strength of Corbel Section

sr. no	Ast %	vf %	Vu,expe.
CRA-1	0.8	0	—
CRA-2	0.8	0	127
CRA-3	0.8	0.5	136
CRA-4	0.8	0.5	144
CRA-5	0.8	1	160
CRA-6	0.8	1	155
CRB-1	1.2	0	132
CRB-2	1.2	0	139
CRB-3	1.2	0.5	143
CRB-4	1.2	0.5	166
CRB-5	1.2	1	172
CRB-6	1.2	1	144

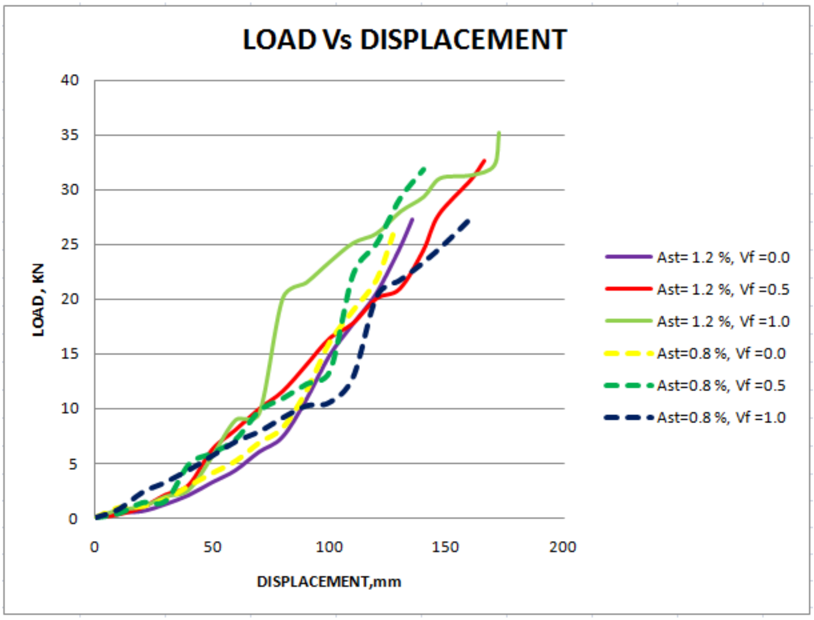


Figure 5.2: Load Vs Displacement Graph

5.2.5 Load Vs Strain results comparison.

Fig. 5.3 to 5.4 shows some typical curve of shear forces versus strain in tension and compression zone of corbel section . For figure note that Strains gauges are attached at different position of corbel are shown Chapter 4.(Fig. 4.10 to 4.11). Details of test data are illustrate in Appendix A.

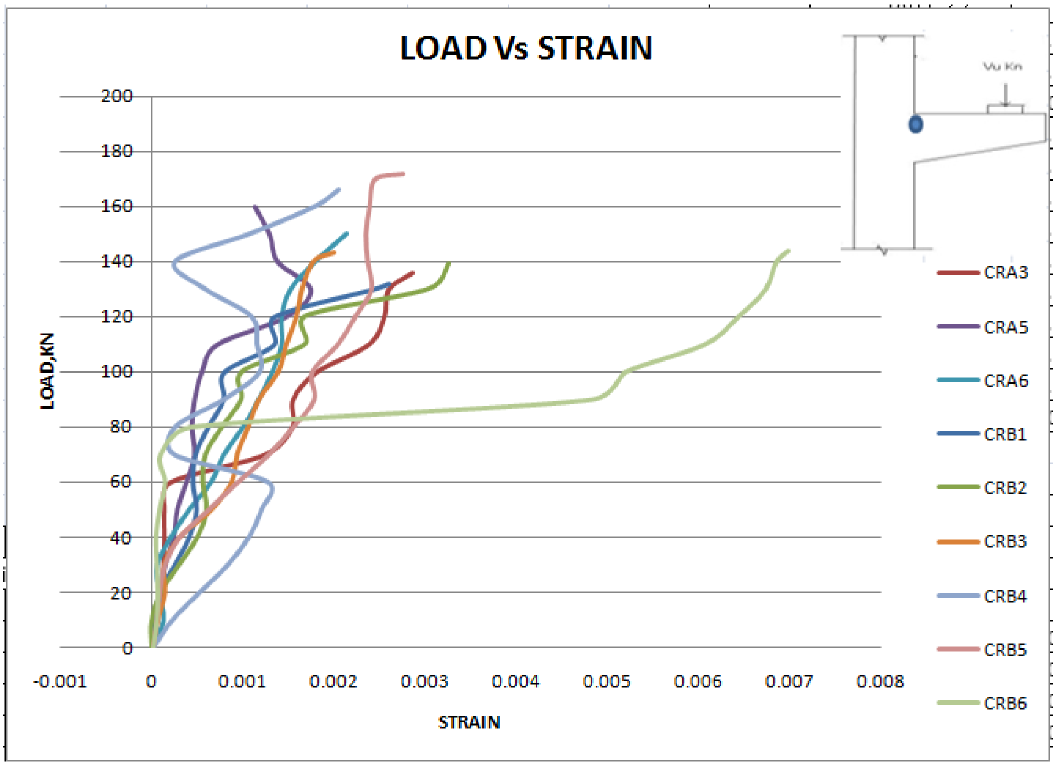


Figure 5.3: Load Vs Strain diagram for Tension zone

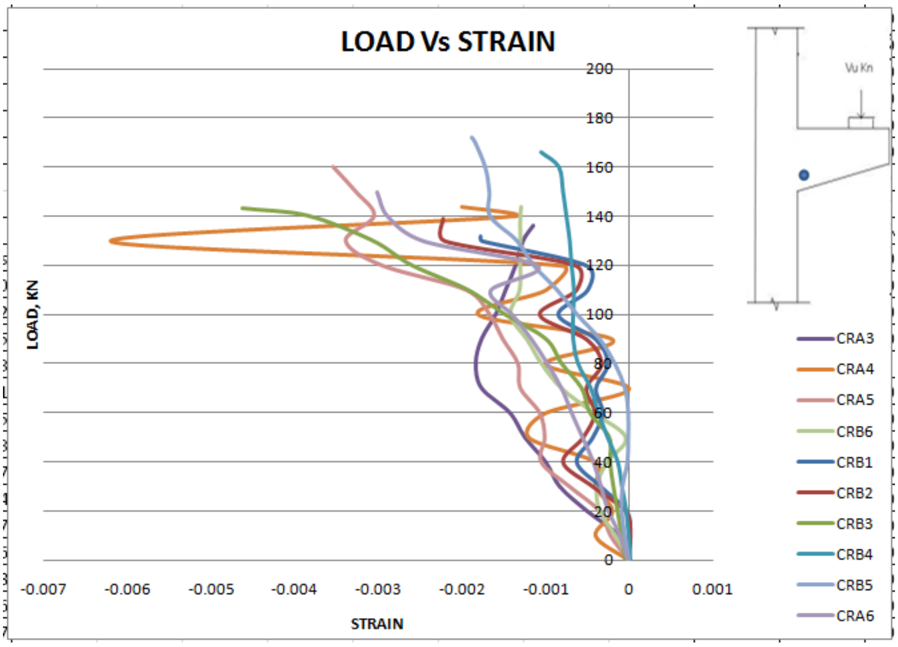


Figure 5.4: Load Vs Strain diagram for Compression zone

5.3 Failure pattern:

Details of the failure mode of corbel section has been listed in Table 5.3.

In specimen CRb-5 and CRb-6,shear-flexure failure mode was observed remaining all specimen showed the diagonal shear (D.S) failure mode , in Corbel specimen the initial crack were observed at the point above load and at the junction of the corbel. various failure mode for corbel section observed as an part of experimental work has been given in Appendix B. Note: D.S = Diagonal shear. FL = Flexure

Table 5.3: Failure mode of corbel

sr. no	av	b	d	av / d	Ast %	vf %	lf/df	fck	Failure pattern
CRa-1	200	150	275	1	0.8	0	60	24.8	D.S
CRa-2	200	150	275	1	0.8	0	60	25.2	D.S
CRa-3	200	150	275	1	0.8	0.5	60	24.48	D.S
CRa-4	200	150	275	1	0.8	0.5	60	24.26	D.S
CRa-5	200	150	275	1	0.8	1	60	24.68	D.S
CRa-6	200	150	275	1	0.8	1	60	24.5	D.S
CRb-1	200	150	275	1	1.2	0	60	25.97	D.S
CRb-2	200	150	275	1	1.2	0	60	25.48	D.S
CRb-3	200	150	275	1	1.2	0.5	60	24.89	D.S
CRb-4	200	150	275	1	1.2	0.5	60	24.9	D.S
CRb-5	200	150	275	1	1.2	1	60	25.3	D.S+ FL.
CRb-6	200	150	275	1	1.2	1	60	23.8	D.S +FL.

5.4 Comparison of results of analytical model with literature data and Experimental work.

5.4.1 Comparison of results of analytical model with literature data:

The shear strength of test beams was calculated using the theory presented in **Chapter 3**. The specimens from previous investigations were included in the shear strength comparisons. The test results of the previous studies were given in **Chapter 2.**

Corbels with only vertical loads were considered. Before studying the correlation between measured and predicted shear strength, the following points need attention:

a. The shear span to depth ratio(av/d) of corbel section ranges from 0.4 to 1.13 for various section while the reinforcement ranges from 0.5 to 1.4 % with variation of steel fiber ranging from 0.7 to 2.5 %.

b. The concrete compressive strength of the test corbel from various investigators ranged from 34.22 to 56.60 Mpa.

c. All the beams considered were only subjected to vertical loads no horizontal load was applied to the section.

Comparisons of test shear strength to prediction by the theory are presented in Table 5.4. There were 51 test results altogether. The mean Vu(calc)/Vu(test) value of the ultimate shear strengths is 0.96 with a coefficient of variation of 0.16. Also, Correlation of test and predicted shear strengths in present study are presented in Table 5.4 (Present Study). The mean Vu(test)/Vu(calc) value of the ultimate shear strengths is 1.06 with a coefficient of variation of 0.18.

Table 5.4: Comparison of Proposed Theory with Literature data

SR. NO	SP. NO	Resercher	SP. NO.	av	b	d	av/d	As/bd	?f	Vu test	Vu, calc	Ratio	Ratio
				mm	mm	mm		%	%	KN	KN	Vutest/ Vu calc	Vu calc / Vu test
1	1	Hughes	C2	125	152	120	1	0.861	0.7	84.5	99.34	0.851	1.18
	2		C3	125	152	119	1.1	0.868	0.7	82.9	73.75	1.124	0.89
	3		C4	125	151	123	1	0.846	0.7	91.8	84.68	1.084	0.92
	4		C5	125	152	119	1.1	0.868	0.7	96	98.11	0.978	1.02
	5		C6	125	156	117	1.1	0.861	0.7	75.2	77.59	0.969	1.03
3	6	FATTUHI	C27	52.5	153	121	0.4	0.543	0.7	125.8	124.5	1.01	0.99
	7		C28	89	151	124	0.7	0.537	0.7	88.2	86.22	1.023	0.98
	8		C29	125	153	130	1	0.505	0.7	65.9	93.14	0.708	1.41
	9		C30	52.5	154	121.5	0.4	0.84	0.7	171	107.8	1.587	0.63
	10		C31	64.5	153	118	0.6	1.253	0.7	179	140.3	1.276	0.78
	11		C32	125	153	118	1.1	1.253	0.7	110.1	104.1	1.058	0.95
3	12	FATTUHI	T3	89	152	122	0.7	0.847	0.7	133	126.4	1.052	0.95
	13		T4	89	151	123	0.7	0.846	1.4	142.5	135.1	1.055	0.95
	14		T10	89	151	117	0.8	1.28	0.7	138	130.1	1.061	0.94
	15		T11	89	152	121	0.7	1.23	1.4	160.2	136.6	1.173	0.85
	16		T12	89	152	121	0.7	1.23	2.1	171.2	136.9	1.251	0.8
4	17	FATTUHI	1	80	152.5	123	0.7	1.206	1.7	153	120.4	1.271	0.79
	18		2	80	155	124	0.7	1.177	1.7	160	128.5	1.245	0.8
	19		3	80	152.5	126	0.6	0.523	1.7	91.2	95.37	0.956	1.05
	20		4	80	155	125	0.6	0.519	1.7	93	93.32	0.997	1
	21		5	140	155	123	1.1	1.186	1.7	103	98.04	1.051	0.95
	22		6	140	154.5	124	1.1	1.186	1.7	95.7	92.45	1.035	0.97
	23		7	140	153	126	1.1	0.521	0.7	53.3	66.89	0.797	1.26
	24		8	140	153	125.5	1.1	0.524	0.7	53.1	72.78	0.73	1.37
	25		9	80	152.5	123	0.7	1.206	1.7	152.9	100.3	1.524	0.66

SR. NO	SP. NO	Resercher	SP. NO.	av	b	d	av/d	As/bd	?f	Vu test	Vu, calc	Ratio	Ratio
				mm	mm	mm		%	%	KN	KN	Vutest/ Vu calc	Vu calc / Vu test
	26		10	140	155.5	123	1.1	1.183	1.7	102.9	113.6	0.906	1.1
	27		11	140	153	126	1.1	0.521	0.7	56	70.85	0.79	1.27
	28		12	80	154	125	0.6	0.522	0.7	92	93.73	0.982	1.02
	29		13	110	154.7	123	0.9	1.189	1.7	111.7	106.1	1.053	0.95
	30		14	110	153.5	125	0.9	0.524	0.7	68.3	73.42	0.93	1.08
	31		15	110	152.5	126	0.9	0.523	0.7	67.2	78.4	0.857	1.17
	32		16	110	154.5	123.5	0.9	1.185	1.7	114.3	117.8	0.97	1.03
	33		18	89	154	124.5	0.7	1.18	1	119	80.94	1.47	0.68
5	34	FATTUHI	20	110	153	123.5	0.9	1.197	1.8	126	119	1.058	0.95
	35		21	110	156	122	0.9	1.188	1.5	118	116.7	1.011	0.99
	36		22	100	153	123	0.8	0.835	1.5	108.5	112.7	0.962	1.04
	37		23	110	153	122.5	0.9	1.207	2	126.5	102.6	1.232	0.81
	38		24	80	153	124	0.7	0.828	2	131.5	96.6	1.361	0.74
	39		27	80	153.5	123.5	0.7	1.193	2.5	171.5	120.8	1.419	0.7
	40		30	120	153.9	120.2	1	0.749	1	86.5	92.69	0.933	1.07
	41		31	135	154.5	124	1.1	1.443	2	119.5	124.6	0.959	1.04
	42		32	120	154	120.2	1	1.494	2	132.5	122.7	1.08	0.93
	43		35	135	155.1	122.5	1.1	1.786	1.5	124.5	122.9	1.013	0.99
	44		37	135	153.8	123.1	1.1	1.792	2	140	121.2	1.155	0.87
	45		38	110	152.2	124	0.9	0.533	2	74	72.37	1.023	0.98
	46		39	110	153.5	154	0.7	1.45	2.3	144.5	137.7	1.049	0.95
	47		40	125	155.5	122.8	1	1.777	2.3	142	119.6	1.187	0.84
	48		44	135	153.8	122.6	1.1	1.466	1.5	109.5	110.1	0.995	1.01
	49		45	135	153	122.3	1.1	1.183	1	120	113	1.062	0.94
	50		46	75	154.5	92	0.8	0.707	1	74.5	70.72	1.053	0.95
	51		48	80	155.5	93.2	0.9	1.084	2	100	88.43	1.131	0.88

Table 5.5 shows the results of the comparison made between the results obtained by the present experimental work with the results obtained by the proposed theory as discussed in **chapter 3**.

12 specimen were tested with details as specified in the Table 5.5. The mean Vu(calc)/Vu(test) value of the ultimate shear strengths is 1.06 with a coefficient of variation of 0.07. Also, Correlation of test and predicted shear strengths in present study are presented in Table 5.4 (Present Study). The mean Vu(test)/Vu(calc) value of the ultimate shear strengths is 0.95 with a coefficient of variation of 0.06.

Table 5.5: Comparison of Proposed Theory with Present experimental work

sr. no	av / d	Ast %	vf %	lf/df	fck	Vu, anly.	Vu,expe.	Vu,e/Vu, a..	Vu,a/Vu,e..
CRA-1	0.73	0.8	0	60	24.8	132.1	————		
CRA-2	0.73	0.8	0	60	25.2	134.3	127	0.95	1.06
CRA-3	0.73	0.8	0.5	60	24.5	146.3	136	0.93	1.08
CRA-4	0.73	0.8	0.5	60	24.3	146.9	144	0.98	1.02
CRA-5	0.73	0.8	1	60	24.7	163.4	160	0.98	1.02
CRA-6	0.73	0.8	1	60	24.5	162.2	155	0.96	1.05
CRB-1	0.73	1.2	0	60	26	137.4	132	0.96	1.04
CRB-2	0.73	1.2	0	60	25.5	134.8	139	1.03	0.97
CRB-3	0.73	1.2	0.5	60	24.9	157.1	143	0.91	1.1
CRB-4	0.73	1.2	0.5	60	24.9	157.1	166	1.06	0.95
CRB-5	0.73	1.2	1	60	25.3	186.5	172	0.92	1.08
CRB-6	0.73	1.2	1	60	23.8	175.5	144	0.82	1.22

5.5 Comparison of results of Experimental work with various codes.

Various code provisions for shear strength of concrete Corbels were described in Chapter 3. The experimental shear strength of the 12 corbels were tested in the Present study is compared to the predictions by the following:

1. Indian Standard IS 456: 2000 [1]

2. American Concrete Institute Building Code ACI 318-05 [2], and

3. Euro code EC2 Part I [3].

The comparisons of test shear strengths to predictions by the IS 456: 2000 [1], ACI 318-05 [2] and EC2 Part-I [3] codes are given in Table 5.6.

The summary of correlation in Table 5.6 indicates significant scatter in the predictions by the codes. For the three methods of prediction, the coefficient of variation ranged from 0.037 (ACI 318-05 [2]) to .16 (IS 456: 2000 [1]).

Proposed method gave the best prediction with the smallest scatter. The mean Vu(Exp.)/Vu value is 1.0325 with a coefficient of variation of 0.04.

All other code methods apart from ACI 318-05 [2] gave overall conservative

Table 5.6: Comparison of Proposed Theory and various codes with Present experimental work

specimen	Vu,IS456/ Vu,test	VuACI318/ Vu,test	Vu,EC-2/ Vu,test	Vu,anl./ Vu,test
cra1	2.4094488	1.136575	0.888031	1.06
cra2	2.4094488	1.145669	0.883465	1.06
crb1	2.7324242	1.145076	0.88553	1.04
crb2	2.6489209	1.066906	0.900504	0.97
mean	2.5500607	1.123557	0.889382	1.0325
standard	0.1659048	0.037995	0.076456	0.04272
deviation				

predictions. The most conservative in estimating the shear strength design of corbel were given by the IS 456:2000 [1].

5.6 Summary

The report presented the analytical investigation on shear strength of reinforced concrete corbel subjected to vertical loading only. In all, 63 corbel were analyzed. The analytical study comprised the development of a theory based on propose theory. The following conclusions are drawn:

1 The shear strength of corbel increased with an increase in the fiber percentage.

2 The shear strength also increased with an increase in the tensile reinforcement.

3 The shear span-to-depth ratio a_v/d have a significant effect on the shear strength of reinforced concrete corbel.

4 The ultimate shear strengths predicted by the theory correlated with the test results of other specimens available in the literature.

5 As the amount of tensile reinforcement along with fiber percentage is increased, the corbel fails in a flexure mode and in brittle manner.

6 If the amount of tensile reinforcement is same then with increase in steel fiber percentage to 1.0 shows more ductility for particular corbel section.

Chapter 6

Design Chart for Corbel

6.1 General

Various design charts are proposed by various researches in order to simply method for calculating the reinforcement required. Such design charts based on various codes such as ACI-318[2] , BS-8110[3] and IS-456 [1] were proposed for reinforced concrete corbel section. We have modified a chart proposed by ACI-318 to overcome its limitation and the same chart can be used to calculate the fiber reinforced concrete corbel sections.

6.2 Design chart for reinforced concrete corbel as per ACI-318.

6.2.1 Design chart:

By using Truss analogy method and applying laws of statics, the following requirements must be satisfied for the corbel to remain in equilibrium.

a. $F_y = 0$

$$i.e.V_u \leqslant Csin\beta \tag{6.1}$$

b. $F_x = 0$

$$i.e.N_u \leqslant T - Ccos\beta or N_u <= A_s f_y - Ccos\beta \tag{6.2}$$

c. M = 0

$$\frac{V_u}{bd} = \frac{a}{d} + \frac{N_u}{V_u} h/d - 1 + \frac{jd}{d}\frac{d}{jd} \tag{6.3}$$

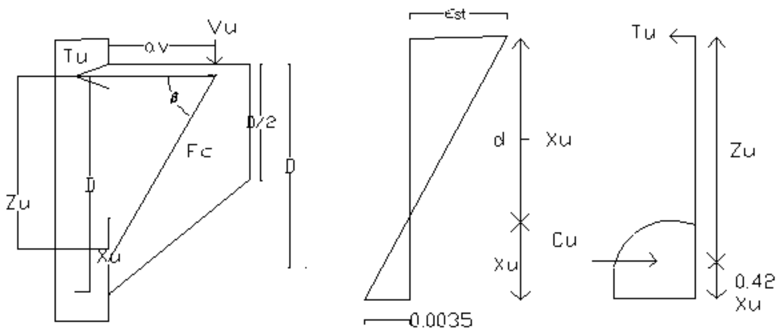


Figure 6.1: Reinforced concrete Corbel force diagram

To be on the conservative side, neglect the contribution of the stirrup force As fsy to the resistance moment and equation 6.3 could be expressed as,

$$V_u a + N_u(h - d + jd) \leq A_s f_y jd \tag{6.4}$$

Now, it is assumed that the compressive force 'C' , lies at one half the depth of the equivalent rectangular stress block.

Thus, the depth of the rectangular stress block, $\boldsymbol{X}$,could be expressed as

$$x = \frac{2d}{\beta_1}(1 - \frac{jd}{d} \tag{6.5}$$

The compression force, 'C' resisted by a rectangular stress block with a vertical height $\beta_1\boldsymbol{X}$:

C = 0.85 β_1 f'_c (b $\boldsymbol{X}$ cos β)

Or

$$C = 1.7bdf'_c(1 - \frac{jd}{d}cos\beta \qquad (6.6)$$

From equation 6.1

$$C = fracV_u sin\beta \qquad (6.7)$$

Now equating 6.7 and 6.8, we get

$$C = 1.7bdf'_c(1 - \frac{jd}{d})sin\beta cos\beta \qquad (6.8)$$

Or

$$\frac{V_u}{1.70\alpha f'_c ba}\frac{a}{d} = 1 - \frac{jd}{d} \qquad (6.9)$$

Where,

$$\alpha = \sin\beta\cos\beta \qquad (6.10)$$

can be taken as 0.8 d/a .

ch1rt 1 is a graphical representation of equation 6.9 for respective value of jd/d. From the strain diagram, using $\varepsilon_c = 0.003$ as shown in figure 6.1, steel strain can be expressed as

$$\varepsilon_s = 0.003(2(\frac{jd}{d} - 1))2(1 - \frac{jd}{d}) \qquad (6.11)$$

ch2rt 2 represent jd/d against steel strain ε_s Stress-strain curve for steel having a yield strength of 40 Ksi and 60 Ksi are given in Chart 3. From equation 6.4

$$\phi A_s = \frac{(V_u a + N_u(h - d + jd))}{f_y jd} \qquad (6.12)$$

Hence,

$$\frac{100A_s}{bd} = \frac{V_u}{bd}\frac{a}{d} + \frac{N_u}{V_u}\frac{h}{d} - 1 + \frac{jd}{d}\frac{d}{jd}\frac{100}{f_y} \qquad (6.13)$$

Chart 4 is a graphical representation of equation 6.2.11.

6.2.2 Design Procedure:

a. Select material property f'_c , and sectional properties b and a.

b. Estimate effective depth d, in accordance with sections 11.14 or 11.15 of ACI-318-08 code.

c. Calculate $V_u = 1.7 \propto f'_c ba$ and a/d.

$\propto$ =(a)(jd)/(a^2 + jd^2)j

Where, jd = 0.8d

d. Determine jd/d from chart 1 .

e. Calculate $\frac{v_u}{bd}[\frac{a}{d} + \frac{N_u}{V_u}(\frac{h}{d}) - 1 + \frac{jd}{d}\frac{d}{jd}$

f. Determine f_y and $100\frac{A_s}{bd}$ from charts 2,3 and 4 respectively .

g. Check that 100As/bd complies with restrictions on the amount of Main tension (detailing rules given in Sections 11.14.2 and 11.14.5 Of ACI code).

h. Calculate 1ooAs/bd in accordance with Section 11.14.4 of ACI code .

i. Check anchorage requirement for As in accordance with Section 12.5(a) Of ACI code.

6.2.3 Nomograph for Reinforced concrete Corbel according to ACI-318.

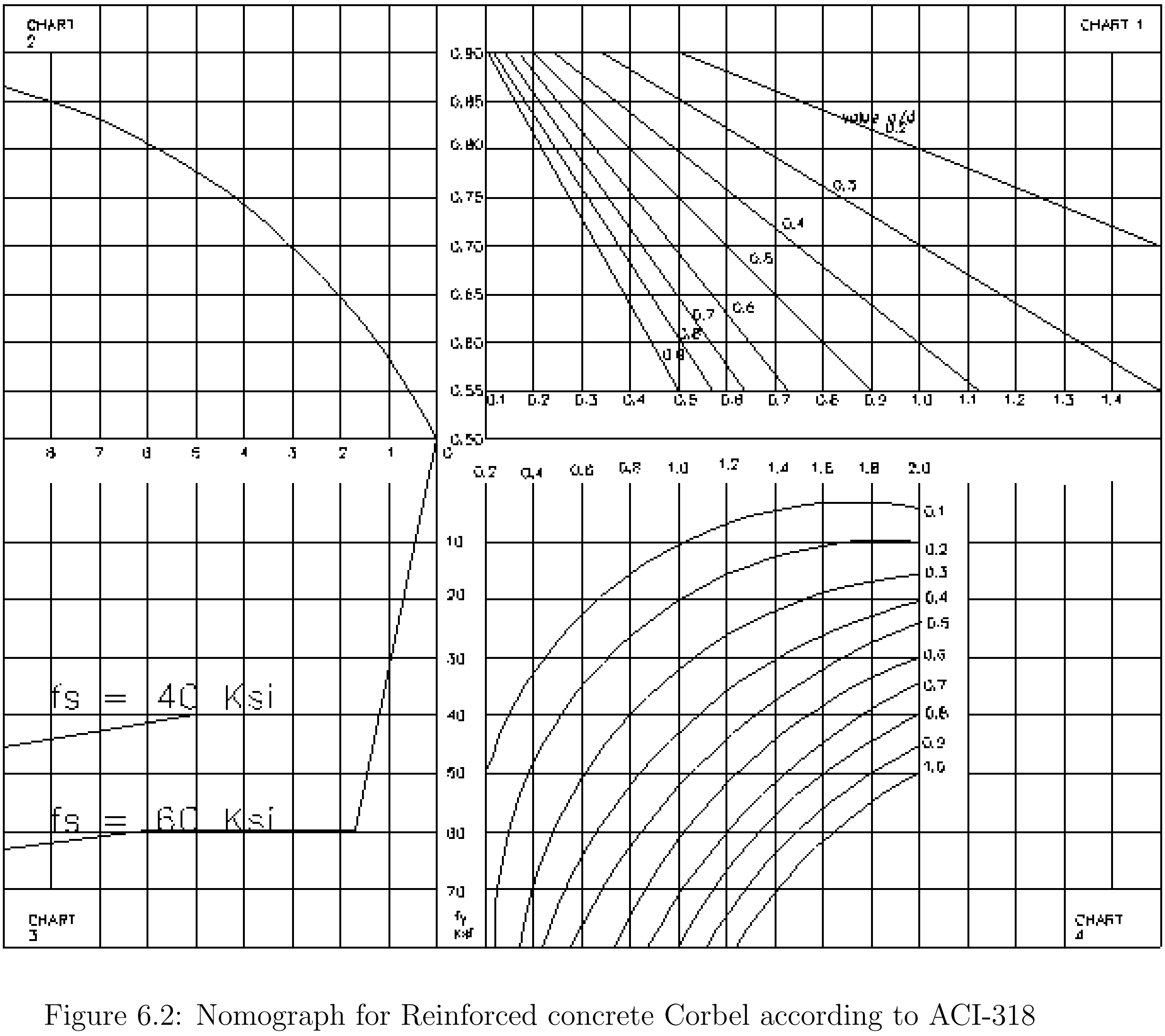


Figure 6.2: Nomograph for Reinforced concrete Corbel according to ACI-318

6.3 Design chart for reinforced concrete corbel as per BS-8110.

6.3.1 Design chart:

Consider the loaded corbel shown in figure 6.3.

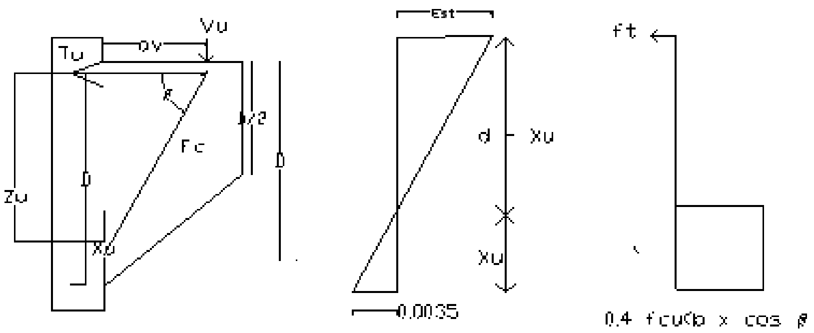


Figure 6.3: Reinforced concrete Corbel force diagram

The neutral axis depth

$$X = 2d(1 - \frac{z}{d}) \tag{6.14}$$

The total compressive force in the concrete Fc can be calculated as,

$$F_c = 0.4 f_{cu} bd \cos\beta \tag{6.15}$$

Now, substituting value of $\boldsymbol{X}$ or substituting value of eqn 6.14 in eqn 6.15

We get,

$$F_c = 0.8 f_{cu} bd(1 - \frac{z}{d}) a_v \sqrt[2]{{a_v}^2 + z^2} \tag{6.16}$$

Now, based of figure 6.3

$$\sin\beta = V_u F_c = \frac{z}{\sqrt[2]{{a_v}^2 + \ z^2}} \tag{6.17}$$

Thus,

$$F_c = V_u \frac{\sqrt[2]{{a_v}^2 + \ z^2}}{z} \tag{6.18}$$

On comparing equation 6.16 and equation 6.18 we get,

$$(\frac{Z}{d})^2 - \frac{1}{1+k}(\frac{Z}{d}) + \frac{1}{1+\frac{1}{k}}(\frac{a_v}{d})^2 = 0 \tag{6.19}$$

Where,

$$k = V_u/0.8 f_{cu} b a_v \tag{6.20}$$

Chart 1 is the graphical representation of equation 6.19 for admissible values of (Z/d).

Steel strain

$$\varepsilon_s{=}0.0035\ \frac{(2\frac{Z}{d}-1)}{2(1-\frac{Z}{d})}$$

Chart 2 plot of (z/d) against ε_s.

Stress versus strain curves for steels having characteristics strengths of 250 and 460 N/mm2 are represented in Chart 3.

The curves in Chart 4 are derived as follows:

$\tan\beta{=}\frac{V_u}{F_t} = \frac{Z}{a_v}$ So,

$$F_t = V_u \frac{a_v}{Z} \tag{6.21}$$

And

$$A_s = \frac{F_t}{f_s} = V_u \frac{a_v}{Z f_s} \tag{6.22}$$

Hence,

$$100\frac{A_s}{bd} = \frac{100}{f_s} V_u \frac{a_v}{bd^2}\frac{d}{Z} \tag{6.23}$$

Chart 4 is a graphical representation of equation 6.23

6.3.2 Design Procedure:

1 Estimate the effective depth (d) using table 6 of BS8110.

2 Calculate (Vu /0.8fcu b.av) and (av/d).

3 Determine (z/d) from chart 1.

4 Calculate (Vu.av/bd$\hat{2}$)(d/z).

5 Determine x,fs, and 100 As/bd from charts 2,3 and 4 respectively.

6 Check that 100 As/bd complies with the restrictions on the amount of main tension steel (detailing rules given in BS8110).

7 Calculate 100 Asv/bd in accordance with table 5 and the detailing rules given in BS8110.

6.3.3 Nomograph for Reinforced concrete Corbel according to BS-8110.

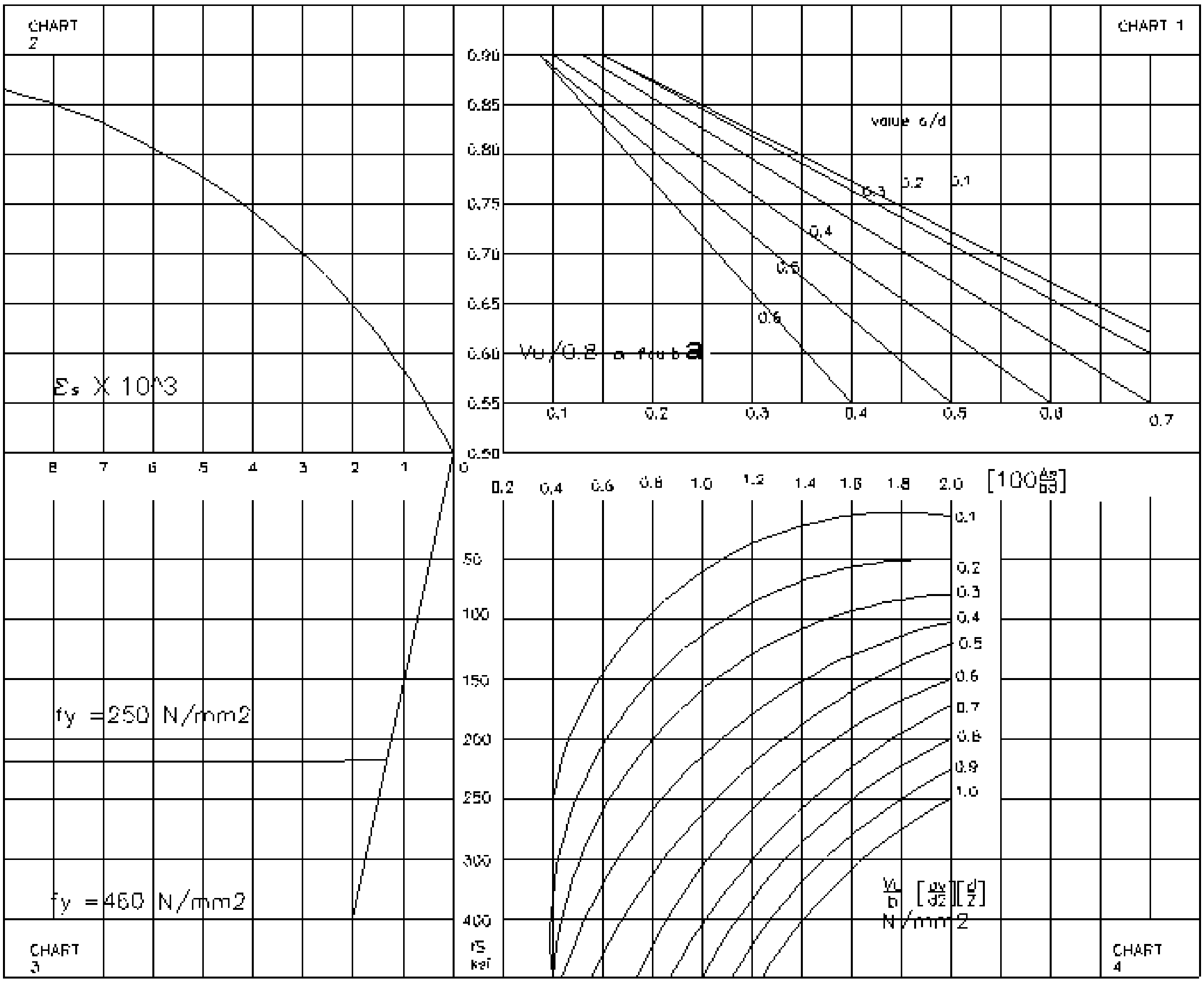


Figure 6.4: Nomograph for Reinforced concrete Corbel according to BS8110

6.4 Design chart for reinforced concrete corbel as per IS-456

6.4.1 Design chart:

Consider the loaded corbel shown in figure 6.5.

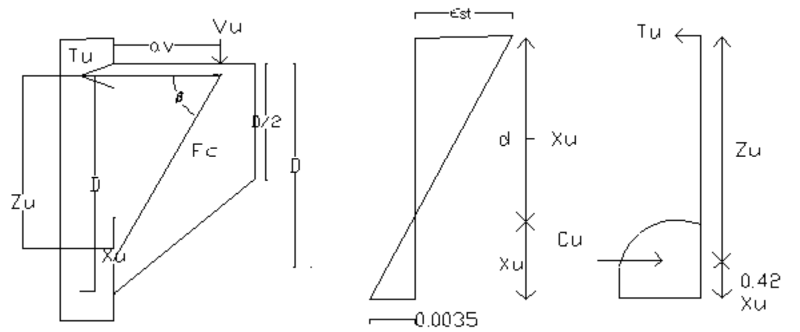


Figure 6.5: Reinforced concrete Corbel force diagram

The neutral axis depth

$$X = 2d(1 - \frac{z}{d}) \tag{6.24}$$

The total compressive force in the concrete Fc can be calculated as,

$$F_c = 0.4 f_{cu} bd \cos \beta \tag{6.25}$$

Now, substituting value of $\boldsymbol{X}$ or substituting value of eqn 6.24 in eqn 6.25

We get,

$$F_c = 0.8 f_{cu} bd(1 - \frac{z}{d}) a_v \sqrt[2]{{a_v}^2 + z^2} \tag{6.26}$$

Now, based of figure 6.5

$$\sin \beta = V_u F_c = \frac{z}{\sqrt[2]{{a_v}^2 + \ z^2}} \tag{6.27}$$

Thus,

$$F_c = V_u \frac{\sqrt[2]{{a_v}^2 + \ z^2}}{z} \tag{6.28}$$

On comparing equation 6.26 and equation 6.28 we get,

$$(\frac{Z}{d})^2 - \frac{1}{1+k}(\frac{Z}{d}) + \frac{1}{1+\frac{1}{k}}(\frac{a_v}{d})^2 = 0 \tag{6.29}$$

Where,

$$k = V_u/0.8 f_{cu} b a_v \tag{6.30}$$

Chart 1 is the graphical representation of equation 6.19 for admissible values of (Z/d).

Steel strain

$$\varepsilon_s{=}0.0035\ \frac{(2\frac{Z}{d}-1)}{2(1-\frac{Z}{d})}$$

Chart 2 plot of (z/d) against ε_s.

Stress versus strain curves for steels having characteristics strengths of 250 and 415 N/mm^2 are represented in Chart 3.

The curves in Chart 4 are derived as follows:

$\tan\beta{=}\frac{V_u}{F_t} = \frac{Z}{a_v}$ So,

$$F_t = V_u \frac{a_v}{Z} \tag{6.31}$$

And

$$A_s = \frac{F_t}{f_s} = V_u \frac{a_v}{Z f_s} \tag{6.32}$$

Hence,

$$100\frac{A_s}{bd} = \frac{100}{f_s} V_u \frac{a_v}{bd^2} \frac{d}{Z} \tag{6.33}$$

Chart 4 is a graphical representation of equation 6.30

6.4.2 Design Procedure:

1 Estimate the effective depth (d) using table 6 of IS-456.

2 Calculate (Vu /0.8fcu b.av) and (av/d).

3 Determine (z/d) from chart 1.

4 Calculate (Vu.av/bd$\hat{2}$)(d/z).

5 Determine x,fs, and 100 As/bd from charts 2,3 and 4 respectively.

6 Check that 100 As/bd complies with the restrictions on the amount of main tension steel (detailing rules given in IS-456).

7 Calculate 100 Asv/bd in accordance with table 19 of IS-456 .

6.4.3 Nomograph for Reinforced concrete Corbel according to IS-456.

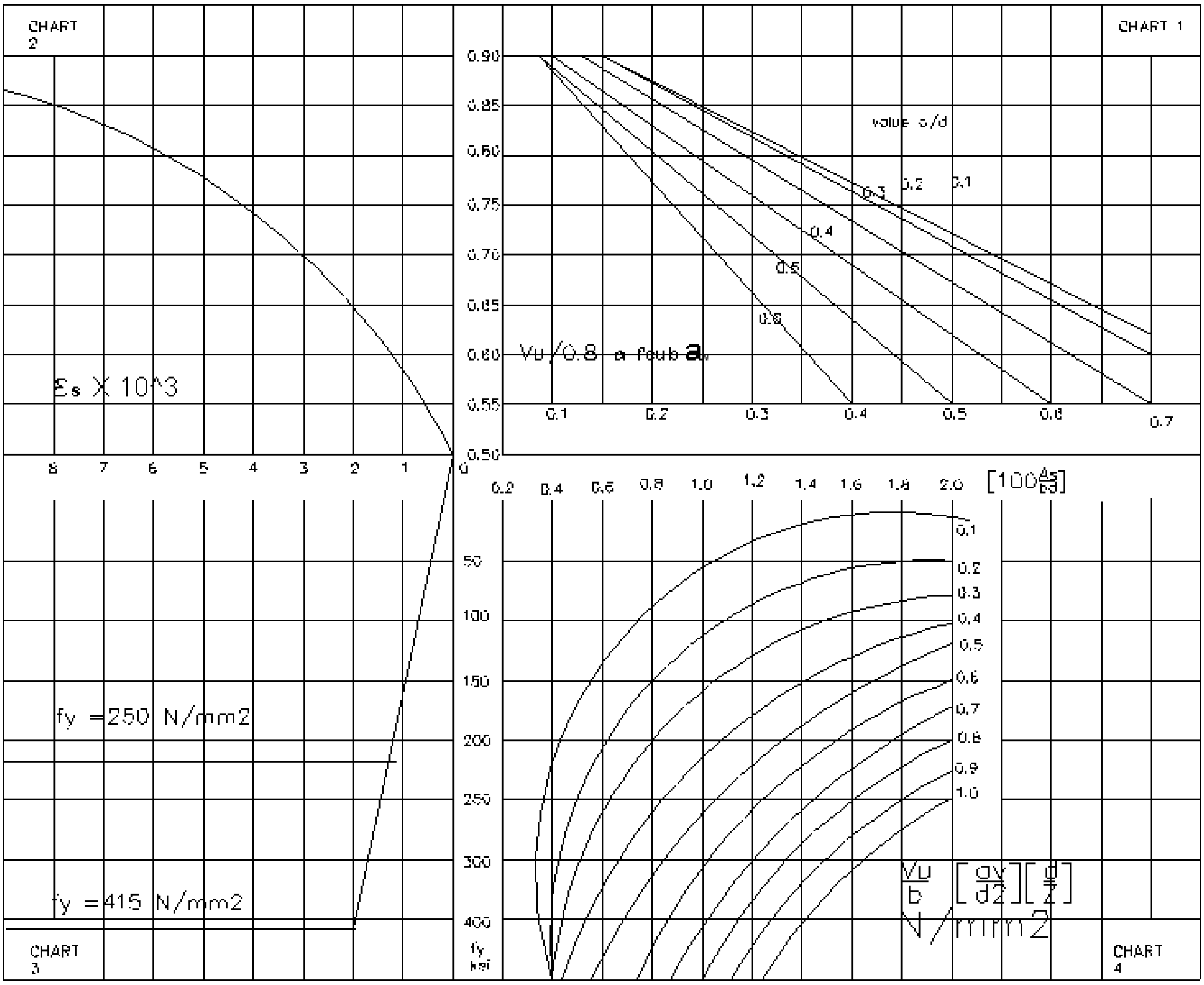


Figure 6.6: Nomograph for Reinforced concrete Corbel according to IS-456

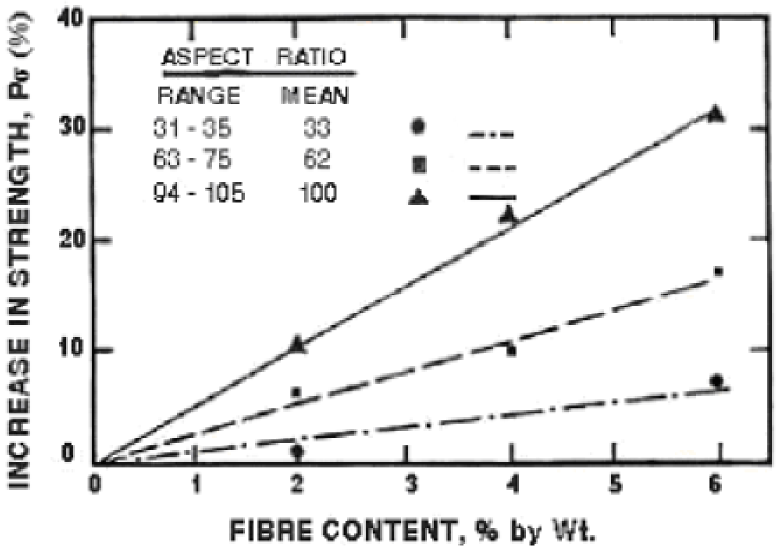


Figure 6.7: Modification factor for steel fiber reinforced concrete.

6.5 Modified Nomograph for steel fiber reinforced concrete.

Figures 6.2 , 6.4 and 6.6 are for calculating the main tensile reinforcement of the reinforced concrete corbels. These figure do not take into account the steel fibers, the compressive strength of concrete need to be modified. Figure 6.7 shows percentage increase in compressive strength of concrete with respect to the percentage of steel fiber reinforcement and the aspect ratio (lf/df). Based on this figure a modification should be made. This new value of concrete compressive strength should be used in lieu of plain concrete compressive strength in Chart-1.

For example: Referring to chart 6.5 [IS-456[1]]. For following data we get, Data:-

Vu = 450 KN.

fcu = 30 N/mm2

b = 300 mm.

d = 425 mm

av = 150 mm

$$\frac{Vu}{0.8fcu\ b.av} = 450 * 103/0.8 * 30 * 300 * 150 = 0.42$$

av / d = 150/390 = 0.394

$$\frac{Vu\ \ av}{b\ d^2}\ \{\frac{d}{z}\} = 450 * 103 * 150/300 * 3902 * 0.64 = 2.31.$$

Referring to Figure 6.5 we get Ast required is equal to 0.8 % of section.

Now, modifying the same example by adding 2.0 % of steel fiber to the section with aspect ratio equal to 90.

Modified concrete compressive strength from figure 6.7 is increased by 10 %

fc = 33 N/mm2 .

Thus,

$$\frac{Vu}{0.8fcu\ b.av} = 450 * 103/0.8 * 33 * 300 * 150 = 0.37.$$

av / d = 150/390 = 0.394

$$\frac{Vu\ \ av}{b\ d^2}\ \{\frac{d}{z}\} = 450 * 103 * 150/300 * 3902 * 0.64 = 2.31.$$

Referring to Figure 6.8 we get Ast required is equal to 0.62 % of section.

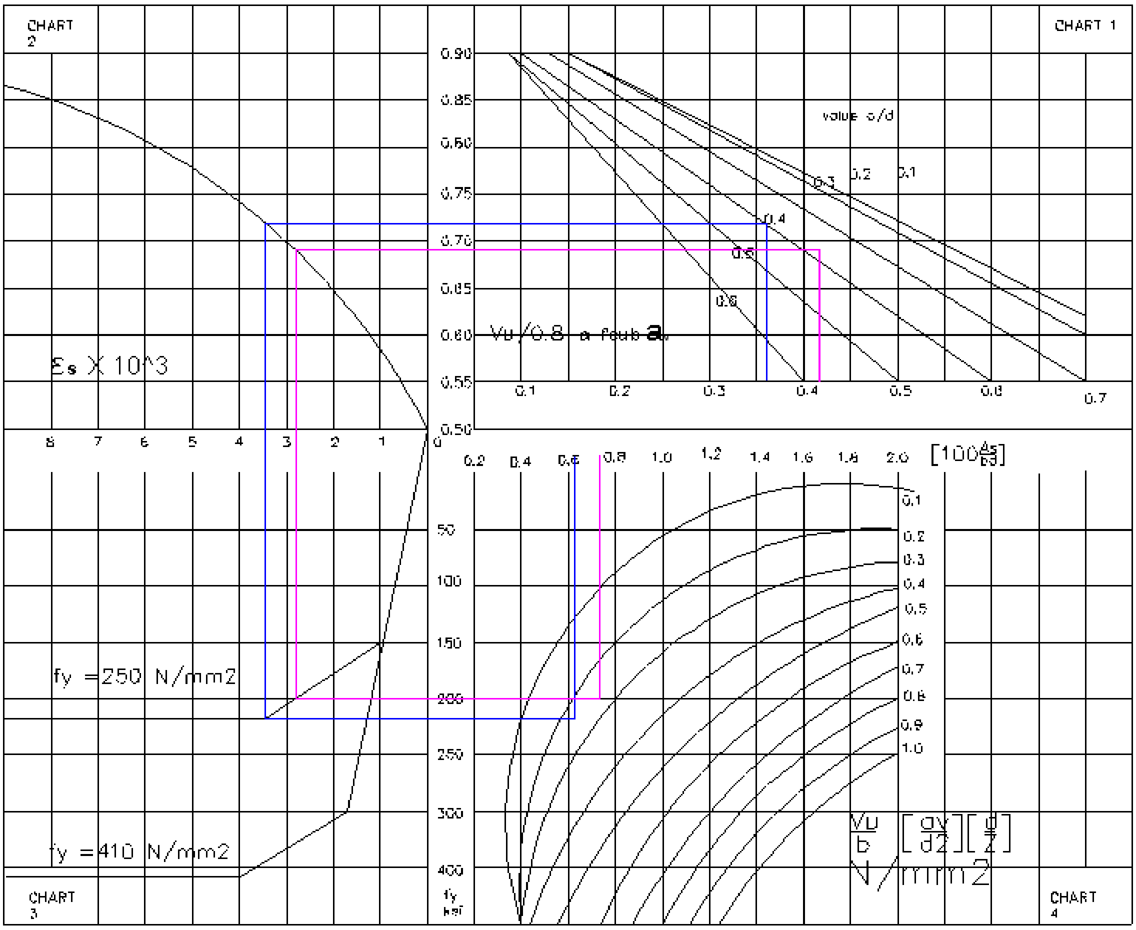


Figure 6.8: shows variation of Ast based on fiber reinforced concrete

Chapter 7

Conclusion and Future Scope

7.1 Conclusion

This chapter presents the conclusions of the present study. The main purpose of this report was to improve the understanding of the behavior of Reinforced concrete corbel section with steel fibers. In general, both the principal and specific objectives as indicated in Chapter 1 have been met. The findings from the experimental and analytical work with regard to the response and shear strength of Reinforced concrete corbels and Steel fiber Reinforced concrete corbels subjected to vertical load has been highlighted here.

The analytical work involved the development of a theory capable of predicting the deformation and shear strength of a Reinforced concrete corbels and Steel fiber Reinforced concrete corbels subjected to vertical load.

The experimental part of the study involved testing of twelve corbels divided into two groups of six each, out of them first two were corbels with plain reinforced concrete corbel, remaining corbel has variation of steel fiber as 0.5 % and 1.0 % respectively All twelve corbel failed in diagonal shear. The deformation in terms of free-end deflection, strain in the main tensile steel bars were monitored during the tests. All the corbels were having an shear span to depth ratio(av/d) equal to 0.72, with an effective span of 200 mm and depth of section was 300 mm. The concrete compressive strength ranged from 23.8 to 25.97 MPa

The results from previous investigations were also studied. The design provisions given by the Indian Standard IS 456:2000 [1], American Concrete Institute Building Code ACI 318-05 [2], and Euro code EC2 Part I [3] were also examined. Comparisons made between the test shear strength and prediction by the various codes of practice.

7.2 Conclusion

a. The test results showed that the addition of steel fibers to concrete resulted in overall improvements in the performance of corbel.

b. Under a specific load, corbel reinforced with steel fibers exhibited smaller cracks widths than those without fibers.

c. Increase in percentage of fiber increase in corbel section increases the depth of neutral axis, ultimate strength and fracture toughness and reducing crack width.

d. Improvement in ductility of corbel resulted from the addition of fiber, where in specimen with 1.2% main tensile reinforcement and 1 % of fiber reinforcement the failure mode changed from diagonal splitting to flexure.

e. Addition of 0.5 % of fiber by volume of steel fiber produced an overall average strength increase of 15 % while if fiber percentage is increased to 1% an overall average strength increase of 25% was observed.

f. The report presented the theoretical investigation of 51 Fiber reinforced concrete corbel under vertical load. The results were compared with the proposed theory and it showed an satisfactory agreement with test results.

g. The IS-456 method predicts highly conservative and uneconomical section and it do not incorporate any design provision for steel fiber reinforced concrete corbels.

h. The design charts can be used to reduce the time required for routine design of reinforced concrete corbels and modified anc then used for steel fiber reinforced concrete corbels.

The ultimate shear strengths predicted by the theory correlated with the test results of other specimens available in the literature and it was found that the overall mean Experiment/predicted shear strength ratio of 1.06 with a coefficient of variation of 0.18 for the 51 test results of literature while for the experimental work it was found that the overall mean Experiment/predicted shear strength ratio of 1.06 with a coefficient of variation of 0.078.

7.3 Future Scope

The following is a list of areas where future research may be directed:

a. Corbel with both vertical and horizontal load should be tested to determine whether the proposed theory is applicable to such corbel.

b. Corbel with Impact load should be studied on aspects as ductility by varying percentage of steel fiber in corbel section.

c. Shear reinforcement required by the section can be replaced by the percentage of steel fibers and it's behavior can be studied.

d. Nomograph can be developed for single and double corbel subjected to vertical and horizontal loads.

Appendix A

Appendix-A

cra1	
load	Disp
0	0
10	0.83
20	0.94
30	1.51
40	2.63
50	3.67
60	4.67
70	6.17
80	7.4
90	10.41
100	14.43
110	17.05
120	19.57
127	23.26

CRA2		STRAIN	
load	disp	tension	Compression
0	0	0	0
10	1.01	0.0066	0.0028
20	1.14	0.016	-0.0096
30	1.85	0.0104	-0.0102
40	3.21	0.01	-0.0056
50	4.49	0.0072	0.0018
60	5.71	0.0064	-0.003
70	7.55	0.0084	0.0044
80	9.04	0.0066	0.0032
90	12.73	0.0064	0.0028
100	17.63	0.0098	-0.0006
110	20.83	0.0102	0
120	23.91	0.007	0.0004
127	28.44	0.0028	-0.0006

CRA3		STRAIN	
load	disp	tension	Compression
0	0	0	0
10	0.29	0.00004	-0.00012
20	1.03	0.00011	-0.0005

30	1.15	0.00013	-0.00083
40	3.68	0.00014	-0.001
50	4.44	0.00014	-0.00125
60	5.39	0.00021	-0.00143
70	7.33	0.00124	-0.00177
80	8.18	0.00154	-0.00184
90	9.15	0.00155	-0.00177
100	10.03	0.00181	-0.0016
110	16.7	0.00238	-0.00148
120	18.8	0.00255	-0.00136
130	21.88	0.0026	-0.00127
136	23.87	0.00286	-0.00115

CRA4		STRAIN	
load	disp	tension	compression
0	0	0	0
10	0.48	0.012	-0.0004
20	1.71	0.006	-0.0002
30	1.92	0.006	-0.0004
40	6.12	0.008	-0.0004
50	7.4	0.012	-0.0012
60	8.99	0.012	-0.001
70	12.21	0.02	0
80	13.64	0.05	-0.001
90	15.25	0.024	-0.0002
100	16.71	0.026	-0.0018
110	27.84	0.03	-0.001
120	31.34	0.036	-0.0008
130	36.46	0.038	-0.0062
140	39.78	0.043	-0.0014
144	41.37	0.026	-0.002

CRA5		STRAIN	
load	disp	tension	compression
0	0	0	0
10	0.86	0.00006	-0.00022
20	2.58	0.00009	-0.00035
30	3.63	0.00015	-0.00071
40	4.88	0.00024	-0.00105
50	6.35	0.00028	-0.00101
60	7.8	0.00038	-0.00107

70	8.81	0.00047	-0.0013
80	10.23	0.00044	-0.00133
90	11.44	0.00047	-0.00151
100	11.79	0.00055	-0.00167
110	14.38	0.00072	-0.00195
120	22.62	0.00147	-0.00296
130	24.3	0.00174	-0.00339
140	26.1	0.00138	-0.00305
150	28.22	0.00129	-0.00327
160	30.58	0.00113	-0.00354

CRA6		STRAIN	
load	disp	tension	Compression
0	0	0	-0.00002
10	0.68	0.00013	-0.00011
20	2.02	0.00008	-0.00025
30	2.85	0.00006	-0.00034
40	3.84	0.00021	-0.00043
50	4.99	0.00041	-0.00056
60	6.12	0.00065	-0.00069
70	6.93	0.00079	-0.00081
80	8.03	0.001	-0.00098
90	8.98	0.00117	-0.00117
100	9.27	0.00132	-0.0014
110	11.3	0.00142	-0.00166
120	17.78	0.00142	-0.0011
130	19.1	0.00152	-0.00246
140	20.5	0.00179	-0.00289
150	22.18	0.00213	-0.00302
155	24.02		

CRB1		Strain	
load	DISP	TENSION	COMPRESSION
0	0	0	-0.00001
10	0.22	0	0.00001
20	0.34	0.00008	-0.00003
30	0.98	0.00024	-0.00034
40	1.82	0.0004	-0.00063
50	2.68	0.00048	-0.00044
60	3.79	0.00044	-0.00032
70	5.46	0.00047	-0.0004
80	5.9	0.00061	-0.00026

90	9.87	0.00078	-0.0004
100	13.93	0.00079	-0.00085
110	16.84	0.00134	-0.0005
120	19.34	0.00134	-0.00051
130	23.45	0.00243	-0.00176
132	24.37	0.0026	-0.00178

CRB2		strain	
load	DISP	TENSION	COMPRESSION
0	0	0	-0.00002
10	0.82	0	0.00002
20	0.94	0.0001	-0.00004
30	1.58	0.0003	-0.00043
40	2.42	0.0005	-0.00079
50	3.88	0.0006	-0.00055
60	4.99	0.00056	-0.00041
70	6.66	0.00059	-0.00051
80	8.94	0.00077	-0.00033
90	11.67	0.00098	-0.00051
100	15.73	0.00099	-0.00107
110	18.64	0.00168	-0.00063
120	21.74	0.00168	-0.00064
130	25.85	0.00304	-0.00221
139	30.31	0.00325	-0.00223

CRB3		strain	
load	DISP	TENSION	COMPRESSION
0	0	0.00001	-0.00005
10	0.17	0.00006	-0.00009
20	0.26	0.00014	-0.00014
30	1.58	0.00013	-0.00019
40	2.04	0.0003	-0.00023
50	4.79	0.00065	-0.00024
60	6.86	0.00087	-0.00047
70	7.2	0.00093	-0.00057
80	8.85	0.00105	-0.00081
90	11.55	0.00117	-0.00100
100	14.26	0.00138	-0.00149
110	16.58	0.00147	-0.00194
120	17.94	0.00158	-0.00261
130	18.27	0.00164	-0.00308
140	23.41	0.00177	-0.00385
143	27.78	0.00199	-0.00463

CRB4		Strain	
load	DISP	TENSION	COMPRESSION
0	0	0.00001	0.00001
10	0.52	0.00022	-0.00001
20	1.68	0.00052	-0.00004
30	2.71	0.00083	-0.00009
40	4.05	0.00106	-0.00014
50	7.78	0.0012	-0.00026
60	9.31	0.00127	-0.00037
70	12.8	0.00024	-0.00046
80	14.35	0.00025	-0.00061
90	16.4	0.00079	-0.00067
100	18.6	0.00118	-0.00067
110	19.05	0.00115	-0.00066
120	22.21	0.00109	-0.00069
130	23.69	0.00058	-0.00071
140	25.56	0.00025	-0.00075
150	27.67	0.00107	-0.00079
160	30.87	0.00179	-0.00084
166	32.67	0.00205	-0.00105

CRB5		Strain	
load	DISP	TENSION	COMPRESSION
0	0	0.00001	0
10	0.54	0.00005	-0.00005
20	0.82	0.00008	-0.00009
30	1.75	0.00014	-0.00008
40	2.37	0.0003	-0.00002
50	5.45	0.00062	-0.00001
60	8.75	0.00094	-0.00001
70	9.15	0.00128	-0.00005
80	20.5	0.00155	-0.00017
90	21.05	0.00178	-0.00035
100	22.95	0.00176	-0.00062
110	24.67	0.00204	-0.00086
120	25.55	0.00223	-0.00114
130	27.4	0.00241	-0.00134
140	28.75	0.00237	-0.00166
150	29.65	0.00235	-0.00167
160	31.25	0.00239	-0.00172
170	32.18	0.00245	-0.00184
172	35.14	0.00276	-0.00188

CRB6		strain	
load	DISP	TENSION	COMPRESSION
0	0	0.00001	0
10	0.99	0.00005	-0.00016
20	1.27	0.00006	-0.00035
30	2.2	0.00005	-0.00039
40	2.82	0.00004	-0.00027
50	5.9	0.00008	-0.00004
60	9.2	0.00014	-0.00042
70	10	0.00009	-0.00081
80	19.6	0.00042	-0.00105
90	21.9	0.00483	-0.00123
100	23.8	0.00519	-0.00143
110	25.52	0.00606	-0.00132
120	26.4	0.00643	-0.00131
130	28.5	0.00673	-0.00131
140	29.85	0.00685	-0.00131
144	32.34	0.00698	-0.0013

Appendix B

Appendix-B

Figure B.1: CRA-3 And CRA-4

Figure B.2: CRA-5 And CRA-6

Figure B.3: CRB-1 And CRB-2

Figure B.4: CRB-3 And CRB-4

Figure B.5: CRB-5 And CRB-6

Appendix C

Appendix-C

list of useful wesite

a. http://www.aci.org
b. http://www.nicee.ac.org
c. http://www.Seminarprojects.com
d. http://www.Sciencedirect.com
e. http://www.iitk.ac.in
f. http://www.asce.com
g. http://www.Theconstructor.blogspot.com

Appendix D

Appendix-D

List of paper communicated

a. Size Effects on the Shear Strength of Fiber Reinforced Concrete element A Theoretical Study, 10th International Symposium on Fiber Reinforced Polymer Reinforcement for Reinforced Concrete Structures (FRPRCS-10), April 2-4, 2011 Marriott Tampa Waterside and Westin Harbor Island (in conjunction with the Spring 2011 ACI Convention)(Abstract selected)

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