# EXPERIMENTAL INVESTIGATION OF ANCHOR EDGE DISTANCE EFFECTS ON CONCRETE BREAKOUT STRENGTH WITHIN FRC

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Presented to the Faculty of the Graduate School of

The University of Texas at Arlington in Fulfillment

of the requirements for the Degree of

**MASTER OF SCIENCE IN CIVIL ENGINEERING**

**STRUCTURAL ENGINEERING**

THE UNIVERSITY OF TEXAS AT ARLINGTON

December 2023

# Acknowledgment

The first person to whom I would like to express my heartfelt appreciation is Dr. Azzawi for his guidance and significant support and for continuing to drive me throughout my graduate academic career. Dr. Azzawi has been a great mentor to me, providing invaluable and crucial technical information and educational guidance throughout this research. Dr. Azzawi has an excellent background education and knowledge and is ready to offer and share with me without any tiny doubt. His advice has always provided me with massive inspiration and motivation in dealing with academic and personal challenges. I can't forget his efforts to make this research straightforward.

Also, I would like to express my gratitude and thank other faculty members. Thank you to Dr. Tamer Aljufout and Dr. Juan Antonio Balderrama for their support, comments, advice, ideas, and time.

I would also like to thank Balaram Lutil, Mossaab Lnu, Mahfooz Akbri, Sujan Kumar, Sadat Morshed, Dr. Park, and Samed Alkafaji, who provided me with invaluable support and assistance throughout this project.

Appreciation is also extended to my family, especially my parent and siblings. I am deeply indebted to them for their unwavering love, understanding, support, encouragement, and patience during my graduate student career. At the same time, I offer my special thanks to my best friends, Atheer Alkafaji, Mohammed Aljboori, and Ahmed Al Ateeq, for their encouragement and guidance.

November 22, 2023

## Abstract

**EXPERIMENTAL INVESTIGATION OF ANCHOR EDGE DISTANCE EFFECTS ON CONCRETE BREAKOUT STRENGTH WITHIN FRC**

**The University of Texas at Arlington, 2023**

Supervising professor: Dr. Raad Azzawi

This research investigates the effect of edge on concrete breakout strength within steel fiber reinforced concrete (SFRC) under pure tension load. Single and group sets of anchors with high-strength steel-headed studs type (F1554 Grade 105) were cast in place within concrete specimens of different amounts of steel fibers and varied edge distances. Steel fibers were used in three percentages (0%, 0.5%, and 1%) by a weight-volume fraction of the mixture to produce three types of concrete mix designs in the lab. The physical and mechanical properties of steel fibers concrete were calculated and measured by testing specimens at the Civil Engineering Laboratory Building (CELB). In total, nine-cylinder specimens of 4-inch diameter and 8-inch height for compressive strength, nine-cylinder specimens of 6-inch diameter and 12-inch height for split tensile test, and nine beam specimens of 6*6*20 inch for modulus of rupture and flexural behavior. Nine concrete pedestals of 2.5*7.5*20 connected to three concrete beams of 7.5*9*55, nine concrete pedestals of 5*10*20 connected to three concrete beams of 10*9*55, nine concrete pedestals of 7.5*12.5*20 connected to three concrete beams of 12.5*9*55 were cast-in-place with 27 sets of anchor groups and 27 of single anchor were installed and tested after 28 days of curing. The two factors of embedment depth and distance between anchors for all sets are kept constant without changing. The adequate embedment depth and the spacing between two anchors in grouping action are followed as specified and defined clearly in ACI 318-19.

The experiments revealed that the increase of the amount of the steel fiber fraction increases the concrete breakout strength of anchor group in tension by (14.3%, 3.43%, and 8.21 %) corresponding to the change in edge distance (0.5 hef, 1 hef, and 1.5 hef) respectively for increasing of steel fiber

from (0%-0.5%) and (44.88%, 33.3%, and 14.28%) corresponding to the change in edge distance (0.5 hef, 1 hef, and 1.5 hef) respectively for increasing (0%-1%). In the case of a single anchor, the concrete breakout increasing of increasing steel fiber fraction from (0.0%-0.5%) is (1.92%, 1.12 %, and 1.1%) corresponding to the change in edge distance (0.5 hef, 1 hef, and 1.5 hef) respectively. The increase from (0.0%-1.0% SFRC) is (6.76%, 16.6%, and 1.4%) corresponding to the change in edge distance (0.5 hef, 1 hef, and 1.5 hef), respectively.

Also, the increase in steel fiber from (0.0%-0.5%) of the anchor group is causing an increase in strain around (+1.58%, +19.37%, and +1.89%) and displacement approximately (+25. 39%, +7.95%, and +22.33%) corresponding to the change in edge distance (0.5 hef, 1 hef, and 1.5 hef) respectively. The increase in steel fiber from (0.0%-1.0%) is causing an increase in strain around (+46.0%, +65.16%, and +11.79%) and displacement approximately (+49.21%, +15.91%, and 40.77%) corresponding to the change in edge distance (0.5 hef, 1 hef, and 1.5 hef) respectively. Similarly, the increase in steel fiber of single anchor from (0.0%-0.5%) is causing the rise in displacement around (+39. 73%, +56.86%, and +44.38%) corresponding to the change in edge distance (0.5 hef, 1 hef, and 1.5 hef) respectively. The increase in steel fiber from (0.0%-1.0%) is causing the rise in displacement around (+56.76%, +76.47%, and 68.75%) corresponding to the change in edge distance (0.5 hef, 1 hef, and 1.5 hef) respectively.

The concrete breakout strengths for single and group anchors were compared and tested in the same conditions. The concrete breakout strength of the group anchors effect will differ by increasing steel fiber from (0.0%, 0.5%, and 1.0%). The differences are (+63.74%, +86.5%, +89.21%, +72.5%, +88.5%, +95.48%, +86.5%, +98.92%, and +100.0%) respectively, and corresponding to (0.5 hef, 1 hef, and 1.5 hef). Concrete average compressive strength increased by increasing steel fiber. The growth in the average strength from (0.0% -0.5%) is (3.04%) and the increase from (0.0% - 1.0%) is (9.62%). The split tensile strength increased by increasing the steel fiber. The rise of tensile strength from (0.0%-0.5% SFRC) is (9.05%) while the increase of strength from (0.0%-1.0% SFRC) is around (10.67%). The flexural strength increased by increasing the steel fiber. The rise of steel fibers by

(0.5% and 1.0%) led to increased flexural strength by (9.5% and 38.5%) respectively. Finally, compare the experimental results of the concrete breakout strength with the modified Concrete Capacity Design Method (CCD).

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# CHAPTER ONE

## INTRODUCTION

In the construction industry, concrete is used in different elements in structures such as foundation. The primary purpose of the foundation base is to transfer loads from the superstructure to the soil underneath, where most foundations are made from concrete. The concrete material consists of cement, sand, and gravel, and water helps in the chemical process and makes this mix hardened entirely. Each material in the world has engineering properties to distinguish. Hardened concrete mix is considered a brittle material with low tensile strength but good behavior in compressive strength. Thus, the brittleness of the material can be reduced by improving the behavior of the tensile strength of concrete by adding fibers, which is the topic of this research. Using an adequate amount, high quality, and appropriate shape of steel fibers increases the tensile strength and the ductile behavior of the concrete matrix. There are different types of steel fiber produced in the world for specific goals. This steel fiber, like any other material, has engineering properties. In many structural applications, the best connection between two different materials is to use the anchor. So, steel anchors serve a magnificent purpose in structural design and construction. According to the applied load, the failure could be in anchor or concrete, depending on which one is weaker; thus, it cannot carry the applied load. Comparing steel anchors with concrete, the anchor has better properties than concrete. So, most of the failures happened in concrete. Hence, it is necessary to understand the behavior of the single and group anchors erected in concrete under tension force, which will be studied in this research.

Development in fasting technology plays a significant role in civil and structural engineering. For example, steel anchors connect structural timber, steel, or any other materials with concrete. This means anchorage can be used in many applications, requiring deep study and investigation of concrete, which bonds these anchors. In this way, the steel anchor connections are a critical component responsible for transferring loads between these top structural elements and concrete members. This area can effect on structural performance due to the heavy magnitude of loads transmitted in this small volume of members, which requires providing high strength. Generally, loads in civil engineering are not limited to one type. The anchor-concrete area could be exposed to axial, shear, and moment. This can lead to various kinds of failures of anchored connections that can occur under tension, shear, moment, or the combination between these loads.

Anchorage can be classified into two types depending on the way of installation and the shape of the anchor. According to the method of installation, there are two main types of anchors: cast-in-place anchorages, which are installed in structures during the formwork stage and before concrete pouring, and post-installed anchorages, which are placed in hardened concrete after drilled concrete more than the diameter of anchor and using adhesive material to increase the bond between the anchor and concrete. Anchors can be divided into two groups depending on the way of installation, which are cast in place and post-installed. The cast-in-place anchors come in several shapes and sizes, like Hex-headed bolts, hooked J and L bolts, and welded-headed studs. Hex-headed stud is an anchor that typically comes with a small washer and hex nut to increase the zone area of concrete strength as specified in ASTM F1554 outlines with different grades. The grade of anchor used in this research is F1554 G105, which is considered high strength to make sure failure will be concrete. Some of these anchors are threaded, and some are not.

There are many modes of failure in concrete anchorage areas. Mainly, applying tension loads to the anchor may make this area fail due to anchor steel failure, pullout, pry-out, splitting, side-face blowout, and concrete cone failure. Concrete cone failure is a standard breakout mode of anchors in concrete that erupted by a tensile force. Applying pure tensile load on the anchor makes this anchor subjected to a uniform distributed load all over the cross-section of the steel area. Then, this load is transmitted to concrete, which surrounds this anchor. The concrete tries to resist and provide the capacity for different types of stress. Friction, shear, and tension stress are the types of stress that are initiated in this kind of load. Concrete material has limits as any other material for these types of stresses. Which one reaches the highest limit will cause the failure of concrete. Then, the anchor cannot carry any more loads so the failure will erupt in the component system. The failure under any stresses is governed by crack growth in concrete. A concrete breakout occurs when the applied load is resisted by the cone of influence more remarkable than the force generated between the concrete and the steel anchor. The pullout and steel failure happens when the anchor has a short embedded length or the anchor-yielding properties reach ultimate strength before the concrete, causing a cut of an anchor. Pry out is happening in the concrete part, especially when using less the requirement of edge distance.

Meanwhile, splitting is happening in group anchors only, and stresses are very high, which can cause surface cracks before induced cone influence. Lastly, side-face could occur when the anchor has an extended embedded depth and is close to the edge under high-tension loads. In all of these cases of a concrete breakout of anchor groups, the anchor spacing, anchor diameter, adequate embedment depth, eccentricity of anchors, and the substantial edge distance have a significant influence on the load-bearing capacity of the group. The concrete breakout strength of single and group anchor sets was studied carefully in this research. The obtained breakout values of testes are compared with the nominal definite breakout strength equations based on the Concrete Capacity Design (CCD) method as specified in ACI 318-19 and modifying the nominal concrete breakout strength of group anchor sets with the modification factor related to the steel fiber reinforced concrete (SFRC) and edge distance factor.

One way to improve the fracture behavior and tensile strength of concrete is by adding steel fibers to the concrete mix. Thus, the brittle characteristic of plain concrete (PC) can be improved and switched to a composite material to make it behave as a ductile material to a certain level. This steel fiber can be reinforced in concrete in a randomly oriented distribution and with a short discontinuity of steel fiber geometry compared to rebar. Steel fiber has less unit weight than rebar and does not need to prepare work before casting as rebar. The steel fiber reinforced concrete shows improved flexural tensile strength under the tension load, an essential factor affecting the concrete breakout strength. Adding steel fibers and changing the edge distance shows that the change happened in substantial failure mode, and this is due to the increase in the tensile strength of the concrete.

Nowadays, steel fiber is a prevalent technical solution in civil engineering for all the advantages that it provides. Economically, steel fiber can save a lot compared to conventional reinforced concrete. Steel fibers can play a significant role in reducing concrete cracks due to shrinkage and thermal variations. All these advantages can be helpful and reflected in concrete by improving the durability of concrete structures.

Concrete is plain material in engineering terminology. As soon as steel fiber is added, it will convert and change the status of plain concrete from plain to composite. SFRC is a composite material that combines a cementers' mix with a discontinuous reinforcement. Steel fibers can vary by strength, unit weight, surface finish, type of fibers, shapes, length, and diameter. Steel fiber is similar to additive material. So, it has advantages and disadvantages. There are many advantages of using steel fibers in concrete in a wise way; for example, it increases the tensile strength, increases the ductility of concrete, reduces the shrinkage cracking, reduces concrete deformations by improving cohesion, and increases the toughness and fatigue strength. The steel fiber used in this study is (DRAMIX) 3D 45/50, as shown in Figure (1.1). Using this type of steel fiber is to study the effect of low-strength fibers on the mechanical properties of concrete and compare these properties with the experimental results of single and group anchors in concrete breakout strength.


Figure 1.1 Type of Steel Fiber Used in This Study

### 1.1 Objectives

The main objective of this research is to investigate the effect of anchor edge distance on the concrete breakout strength within changing contents of steel fiber reinforced concrete under pure tension load, as well as comparing the concrete breakout strength of group anchors with single sets. To meet this objective, eighteen concrete specimens with different amounts of steel fibers were cast-in-place with fifty four sets of single and group anchor (F1554 Grade 105, steel-headed studs) were produced and tested at the Civil Engineering Laboratory Building (CELB) at the University of Texas in Arlington. Anchors are setting up and lined up by using holders before pouring of concrete and all of the specimens created from the same design mixtures were also tested for their physical properties.

Table (1.1) The Breakdown Structure of the research

| | | |
|---|---|---|
| Specimens Testing | 4"x8" (9 Cylinders) Tested for Compressive Strength | Three cylinders of each mixture tested for (0.0%, 0.5%, and 1.0% SRFC) |
| | 6"x12" (9 Cylinders) Tested for Split Tensile Strength | Three cylinders of each mixture tested for (0.0%, 0.5% and 1.0% SRFC) |
| | 6"x6"x20" (9 Beams) Tested for Flexural | Three beams of each mixture tested for (0.0%, 0.5% and 1.0% SRFC) |
| Concrete Beams Specimens | 1 Beam (7.5"x9"x55") with 3 pedestals 2.5"x7.5"x20" has 3 Sets of Cast-in-place Group Anchor | 3 Sets of Anchor Groups vs SFRC |
| | 1 Beam (10"x9"x55") with 3 pedestal 5"x10"x20" has 3 Sets of Cast-in- place Group Anchor | 3 Sets of Anchor Groups vs SFRC |
| | 1 Beam (12.5"x9"x55") with 3 pedestal 7.5"x12.5"x20" has 3 Sets of Cast-in-place Group Anchor | 3 Sets of Anchor Groups vs SFRC |
| Concrete Beams Specimens | 1 Beam (7.5"x9"x55") with 3 pedestals 2.5"x7.5"x20" has 3 Sets of Cast-in-place Single Anchor | 3 Sets of Anchor Groups vs SFRC |
| | 1 Beam (10"x9"x55") with 3 pedestal 5"x10"x20" has 3 Sets of Cast-in- place Single Anchor | 3 Sets of Anchor Groups vs SFRC |
| | 1 Beam (12.5"x9"x55") with 3 pedestal 7.5"x12.5"x20" has 3 Sets of Cast-in-place Single Anchor | 3 Sets of Anchor Groups vs SFRC |

## 1.2 Research Contribution

Using steel fiber in concrete increases the concrete breakout strength and reduces the edge distance, which is a significant contribution of this research. Mechanical properties of the concrete will be changed by adding steel fiber in different directions. The compressive, split tensile and flexural strength will increase by increasing of using this specific type of steel fiber. Accordingly, this research revealed of use less area and lighter concrete mass required to produce the same anchorage strength comparing with the strength without fibers. Comprehensively, two main issues delivered in this research: the economy and providing more safety to the structures. Increasing concrete strength will allows using higher grade anchors and this will drive designers to decrease cost, unnecessary heavy mass concrete and big cross section area. Anchors are using in foundation of many structural applications like guardrail bridges, traffic light poles, road sign boards, power lines tower, telecom tower and so on. These anchors play an important part to transmitted loads to the concrete. This technique of adding steel fiber will help designers to consider additional strength, and figure out the capacity within less required edge distance according to ACI specification. This will allow more factors of safety and stability for structure.

## 1.3 Outline of the Thesis

Five chapters will be present entire study for this thesis, as below:

Chapter 1- Introduction: The concept of concrete breakout behavior of single and group anchors under pure tension load as will be explained in this chapter and how steel fibers will influence on the strength as well as changing edge distance.

Chapter 2- Literature Review: This chapter will present the concept of anchors, steel fiber reinforced concrete, and covers past researches on the concrete breakout with SFRC.

Chapter 3- Experimental Works: This chapter encompasses the design requirements of single and group anchors, concrete mix design with or without steel fibers, and testing of all specimens.

Chapter 4- Experimental Results: The test results and all specimens data introduced in chapter 3 will be present in detail in this chapter.

Chapter 5- Conclusions and Recommendations: The conclusions and recommendations by the researcher and proposals for further researches are going to be addressed in this chapter.

# CHAPTER TWO

# LITERATURE REVIEW

## 2.1 Previous Research and Design Practices

There are many researches available on the behavior of anchorages in SFRC. The reason is the anchorage serves an important role in connecting and attaching various components of concrete structures. It is essential to understand the behavior of anchorages in concrete mixed with steel fiber and to validate the applicability of the current study for the design of anchorages for use in structural concrete. This chapter summarizes the work that has been conducted by other researchers on the anchor groups in SFRC and its structural performance and the concrete breakout strength of SFRC.

### 2.1.1 Concrete Grouping Anchors

(Tóth et al. 2019) presented one of the paper which it shows the results of experimental investigations was done for plain concrete and SFRC on tension and shear loaded steel anchors. This inclusive investigation includes 62 pull-out and shear loading tests on single anchors and anchor groups. The test revealed that the SFRC has generally positive effect on the load-displacement behavior of anchorage. Developing crack bridging mechanism of the SFRC and more ductile behavior can lead to a good anchor fasting and considered as a great advantage. What’s more, in case of concrete failure, the ultimate load failure will be higher in SFRC comparing to plain concrete in some certain application and parameter combinations. The scope study of this paper was to perform and investigate the influence of SFRC on concrete cone and concrete edge failure loads for tension and shear loading tests on both single anchors and on anchor groups. Tests were carried out in normal-strength PC and in SFRC using 30 kg/m3 and 50 kg/m3 of steel fibers. Also, testes were carried out on anchor groups, composed on a single row of three anchors (1 × 3) for centric (e = 0 mm) and eccentric tension tests (e = 60 mm, e = 120 mm). According to general performance of this investigation, the anchors installed in SFRC have better strength than one installed in plain concrete. The conclusion in this investigation was along increasing of use SFRC will lead to increase in the ultimate concrete cone capacity and concrete edge resistance of the anchors which is based on all results data. The test results shows that anchors group behavior in SFRC is affected by load displacement behavior of the anchors as well as affected by load bearing capacity and their geometrical arrangement.

A paper by (Qian et al. 2018), studied investigation of the tensile capacity for anchor groups with different spacing between cast-in-place headed anchors of high strength and deep embedment. Twelve reinforced concrete specimens used to conduct this experimental tensile load of cast-in-place headed anchor bolts type (42CrMo). This anchor bolts has diameters 36, 48, and 60 mm and embedded identically by 35 times the anchor diameter in these specimens. The spacing between outside anchor diameters of the test specimens is ranging between 2 and 5 times diameter. The construction steps was mainly are binding steel bars, fixing anchor bolts, supporting wall framework, pouring concrete, removing template and load test. The ready-mixed sulfate resistant concrete was transported from an industrial plant by mobile mixers to the site. The compressive strength of cubic 150-mm at age 28-day was 25MPa. The tensile load-displacement curves followed the same pattern in regardless of the anchor spacing and diameter of the anchor groups. This curve can be simplified into three typical regions: an initial linear segment, a curvilinear transition, and a final linear sector. “The interpreted load- carrying capacity of the elastic limit of an anchor group of cast-in-place headed anchors with high strength and deep embedment in reinforced concrete increased as the anchor spacing increased”. The measured axial steel strains did not exceed the yield strain of 42CrMo alloy steel of all the three anchor bolts of each specimen when were pulled out from the concrete column and even at the applied maximum tensile load.

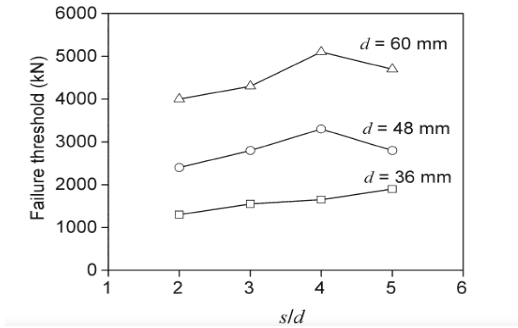



Figure (2.1) The Concrete Cone Failure and Ratio of Spacing to Diameter Anchor

The recommendations and guidelines regarding the design of anchorage in concrete are mainly proposed in (ACI 318-19). These design provisions in the ACI standards are generally based on the assumption of a ductile failure mode of cast-in-place anchors. The failure can be determined by tensile capacity of the anchor bolt which is governed by the tensile yield and fracture of the anchor steel or by the tensile breakout of concrete where the anchor was embedded. The average breakout capacity of a headed anchor is determined by the Concrete Capacity Design (CCD) Method, and the breakout strength calculations are based on a model suggested in the Kappa Method. Figure (2.2) shows the concrete cone failure and how to determine the angle of failure (∅).

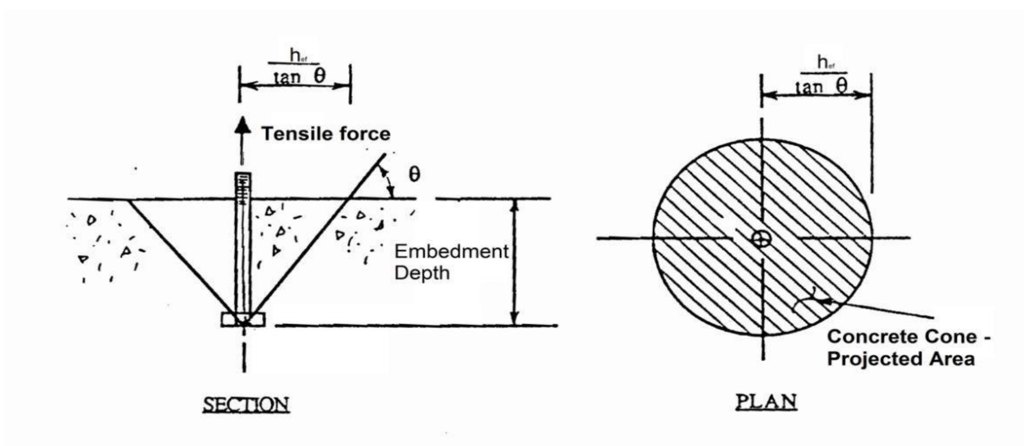



Figure (2.2) The Concrete Cone Failure and Angle of failure

The edge distance of anchorage from the concrete-free edge and the distance of adjacent anchors might affect the anchorage capacity and performance (Nilforoush R., 2017). The ductile behavior of anchors can be affected by the size of the concrete cone failure load (Lee N.H. et al. 2007). Cast-in-place anchors subjected to tension and combined tension and shear interaction, edge conditions, and group effects of cast-in-place anchors should be taken into consideration by examining the concrete failure modes.

According to (ACI 318-19) standard, one of the recommendations to preclude the splitting failure concrete is to use spacing between anchors more than four times of the outside diameter of an anchor. (Klug et al. 2002) performed tension and shear loading tests using expansion anchors, undercut anchors, and bonded anchors in PC and SFRC. The embedment depth of the tested anchors ranged between 50 and 60 mm. The ultimate load for the tested anchor types wasn't increased and it was concluded that the structural behavior of fastenings is not improved in SFRC (L = 35 mm, wavy steel fiber and L = 50 mm, d = 0.8 mm hooked-end steel fiber) compared to plain concrete. However, in most of the cases, it was observed that the failure mode of the anchors was different from concrete breakout failure. Therefore, the beneficial effects of anchoring in SFRC could not be recognized. Furthermore, the author assumed that the fiber orientation might have been parallel to the component surface. Consequently, the amount of fibers, which were intercepting the concrete breakout body, was not sufficient to improve the load-bearing behavior of the fastening system.

A paper by (Nam Ho Lee et al 2007) is to study and evaluate the performance of headed anchors in tension with large diameter and deep embedment depth in concrete. The anchor diameter and embedded depth were greater than 2'' and 25'' respectively. These two parameters not addressed or mentioned in ACI 318, Appendix D and ACI 349,

Appendix B. So, it used the existing design equation in ACI codes to assess applicability and how much compared to large diameter and greater embedded depth differ. Usually, these great diameter and embedded depth use in nuclear plants and anchorage of tanks. The specification of anchor was used in this paper: diameters 2.75'' to 4.25'', yielding strength equal to 140 Ksi and ultimate strength is equal to 155 Ksi. The embedded depth was tested from 25'' to 45'' from surface of concrete. The paper is considered very important because it is represent the first experimental tensile test for anchors embedded more than 21'' and which application can use this dimension of embedded. ACI 318-05, Appendix D, include equation that can estimate the concrete breakout of headed anchors with embedded depth more than 11'' in un-cracked concrete. There are five equations are proposed in that paper to predict concrete breakout. The test contains five test configurations (T1, T2, T3, T4, and T5) and four test replicates per each configuration. Large sample was used to avoid any splitting failure and to minimize of shrinkage cracks; the top and bottom rebar was used in both directions for some of samples. Anchors were placed correctly to their position by using wooden frame and steel. The concrete compressive strength designed to achieve 5500 psi. The test set up is similar to set up in this research. The load increment was 3.5% of ultimate strength. Load, strain and displacement were measured. The failure for T1 to T3 was under cone breakout, while specimen T4 and T5 with supplementary reinforcement were not test to failure. It was noticed the load displacement relationship is varied based on concrete strength at time of testing. According to ACI for allowed head size, the concrete breakout is increasing proportionally with embedded depth to 1.5 power. So, for much head size, it is require increasing power of embedded depth more than 1.5 due to underestimation. It was found during the evaluation of T1, T2, and T3 through proposed equation 5 that the concrete breakout increase in proportion to ($h^2_{ef}$ ). Consequently, the predicted capacities $N_{u,calc}$ are much higher than measure values $N_{u,test}$ and the ratio $N_{u,test}$ /$N_{u,calc}$ decreases with increasing embedded depth. It can be conclude that proposed equation (4) is allow to be used in un-cracked for embedded anchor more than 10'' only if the head size is large which make pressure under head is $p_n \leq 3^*fc'$, but in cracked concrete, it was observed that the concrete capacity was lower. Therefore, the embedded depth to power 1.67 was used due to the reducing of concrete capacity of cracked concrete compared to un-cracked by (20%) which is mean obtain concrete capacity of cracked is multiply by factor 0.8 and should only be used for deep anchors if the pressure under head is $p_n \leq 2.4^*fc'$. The test shows that reinforcement used in specimen T4 was effectively acted in anchorage of concrete and resist applied tension load that leads to increase in capacity due to supplementary reinforcement compared with T1 by approximately (60%) of the calculated yield strength of rebar. Also, the test showed that mean tested capacity was much smaller of the summation of reinforcement and concrete breakout strength CCD. Both T4 and T5 were increased proportionally to the amount of supplementary reinforcement. It was conclude that equation 5 is overestimate the tensile breakout capacity and equation 1 is more conservative for large anchors which is addressed by Fuch in ACI. Also, it was concluded that the cone angle is not 35 but its ranging 25-30 degrees. The supplementary reinforcement should be designed using a strut-and-tie model.

Table (2.1) The Proposed Equations of Concrete Breakout

| Equation number | Predictor | Remark |
|---|---|---|
| (1) | $N_{u,m} = 40\sqrt{f_c'}\,h_{ef}^{1.5}$ (lb) | Mean breakout strength, CCD-method with exponent 1.5 on $h_{ef}$ |
| (2) | $N_u = 30\sqrt{f_c'}\,h_{ef}^{1.5}$ (lb) | Nominal breakout strength, ACI 318-05, Appendix D |
| (3) | $N_{u,m} = 26.7\sqrt{f_c'}\,h_{ef}^{1.67}$ (lb) | Mean breakout strength for anchors with $h_{ef} \geq 10$ in. (254 mm), CCD-method with exponent 1.67 on $h_{ef}$ |
| (4) | $N_u = 20\sqrt{f_c'}\,h_{ef}^{1.67}$ (lb) | Nominal breakout strength for anchors with $h_{ef} \geq 10$ in. (254 mm) according to ACI 318-05, Appendix D |
| (5) | $N_u = 4\sqrt{f_c'}\,\pi h_{ef}^2\,(1 + d_k/h_{ef})$(lb) | Nominal breakout strength, ACI 349-97 (45-degree cone model) |

Note: $f_c'$ = specified concrete compressive strength (psi); $h_{ef}$ = effective embedment (in.); and $d_b$ = diameter of anchor head (in.).

(Rasoul Nilforoush et al. 2017) studied the evaluation of tensile behavior of single cast-in-place anchor bolts in plain and steel fiber-reinforced normal- and high-strength concrete. The studied was covered the influence of concrete thickness member, concrete strength, and the addition of steel fiber, on anchorage capacity and performance was evaluated as well. Nineteen single cast in place headed anchors were tested in plain and SFRC for normal and HPC. So t, the results of this investigation was exercised in terms of load displacement curve, anchorage ultimate load, and anchor displacement at ultimate load and at load corresponding to initiation of concrete cone cracking. The Concrete Capacity Design (CCD) was evaluated to prophesy concrete breakout of anchor bolts for normal and HPC in plain and SFRC. The concrete cone angle was assumed to be equal 35 degree with respect to concrete surface which is lead to $3h_{ef}x3h_{ef}$ projected cone area on concrete surface. The dimension of concrete samples was $h_{ef}$=220mm, L=W=1300mm, and the height of concrete slab 1.5-3 times the embedded depth of anchor. The diameter of steel headed anchor is 36mm and covered threads by plastic tube to get rid-off any friction forces generated between anchor and concrete. Plain and steel fiber is two mixes designed was used in this investigation. Fiber content was 80 kg/$m^3$. The pull out test after 60 days was loaded under displacement control at constant rate 1mm/min. the general behavior was the anchorage capacity and displacement at ultimate load increased slightly by increasing member thickness, as same as, compressive strength. Meanwhile, the displacement at ultimate load and ductility will decrease. Anchors in HPC have higher concrete strength capacity than anchors in normal plain concrete mix (NPC) but lower displacement; this might be attributed to brittle material nature. No cracks were observed during the test till failure happen suddenly and abruptly without any notification. The maximum load and anchor displacement at ultimate load will increase remarkably by increasing steel fiber in concrete whatever member thickness. So, the displacement after ultimate load which was used for evaluating ductility was larger in SFRC than PC. Apparently, the concrete cone in SFRC is influenced by the fracture properties of concrete (compressive, split tensile, flexural). The concrete failure mod will be transmitted from mixed mode concrete cone and splitting to pure cone breakout by increasing member thickness and global bending stiffness. The concrete cone diameter in SFRC is less than the cone diameter in PC. The tensile breakout capacity is increased by increasing member thickness and concrete strength. The experimental results show that CC method is underestimating the tensile breakout capacity in SFRC.

Another paper by (A.F. Ashour et al 2004) is to study the concrete breakout strength of single anchors of cast in place and post installed in tension using neural networks. Three different techniques which are training, validation, and testing are adopted to represent anchor installation system in neural network input layer. The concrete breakout and other different influencing parameter obtained from this neural method were in general agreement with ACI 318-02 for two types of anchor installation and it showed the concrete breakout is proportional to the embedment depth of 1.5 power and marginally affected by changing the anchor head diameter. To enhance anchorage properties, cast in anchors installed before do casting which is require to take care of location of these anchors because these anchors are non-adjustable like post installed which provide more flexibility.  Artificial Neural Networks (ANNs) have been used for many years to predict the performance of concrete and it was observed providing accurate and reliable results compared with experiment data. The ANN was applied to figure out the compressive strength at age 28 by Hong-Guang and Ji-Zong. Also it was used by Flood to find out the deflection of reinforced concrete beams strengthened by FRP. The aim of using ANN with single anchors is to develop multi-layer feed forward neural networks trained back with back propagation algorithm to modal nonlinear relationship, to employ and compare three different technique, and to conduct a parametric study to find the importance of different input parameters on concrete breakout. Failure mode for the both type of anchor installation will depend on the embedment depth, edge distance and steel strength of the anchor. Bursting failure occurs when the anchor is positioned close to edge and has large embedment that may preclude the concrete cone failure. Most of these approaches to find out the breakout capacity use embedment depth and compressive strength as index for tensile strength (0.33 * fc'^0.5). The ANN used Levenberg-Marquardt algorithm which is built in Mat Lab version 6 and the ANN find agrees well with CCD method that ignores anchor diameter effect. The conclusion of that ANN method was anchor diameter has negligible effect on concrete breakout, the concrete breakout is proportional with the embedment depth to power of 1.5, and the prediction from ANN was complying with AI 318-02.

Research by (Saad Ali AlTaan et al. 2012) studied the breakout capacity of cast-in-place single short-headed anchor bolts embedded in both normal concrete (NC) and steel fiber reinforced concrete (SFRC). The volume fractions of steel fiber (Harex) of shelled deformed cross section were used in this paper is (0.4%, 0.8%, 1.2%, and 1.6%) and aspect ratios (19.63, 36.33) which means two length used in this test. The anchor bolts diameters (8, 10, 12mm) and embedment depths (25, 37.5, 50, 62.5mm) were used. The test was performed under three concrete mix proportions had been named Mix M1, Mix M2, and Mix M3. The SFRC effect on the workability so the super plasticizer was used in high strength concrete. The One eighty specimens had been tested and the failure as follow: The plurality of failure specimens which represent (159) were concrete cone failure and in some cases the cone breaks into pieces. The rest of others failure (21) was bolts yielding or fracture (steel failure). The concrete angle cone is increasing by increasing embedment depth and steel fiber index, but decreasing by increase compressive strength according to the test results. Also, it was noticed the concrete capacity of the anchors were increased by increasing steel fiber content and the size of cones were smaller in SFRC than the NC. Loads transfer from anchors to concrete by one or more of the following mechanisms: friction, chemical adhesion, mechanical interlock and breaking against head (Fuches et al., 1995). The average tensile stress was assumed equal to ($f_t$=f'c^0.5 / 31). The anchors with edge distance ($c<h_{ef}$) and/or anchors affected by other concrete breakout cones. Two methods were proposed to calculate breakout capacity. The first method proposed by Mohammed 2006 to calculate the breakout capacity for anchors embedded in SFRC. it is represent lateral surface area multiplied by the post cracking tensile strength ($\sigma_{ct}$) of the steel fiber concrete. This method also estimate the failure angle which proposed by Mohammed 2006 and Al-Jaffal 2007. The second method is to predict the breakout capacity of headed anchors embedded in SFRC in NC with strength ranging from 27.4 to 34.5 and it's derived from the regression analysis. The researcher concluded failure angle increased with embedment depth and SFRC, but decreased with concrete strength. The two methods was acceptable accuracy compared to the experimental breakout capacity of short headed anchors embedded in normal and high strength concrete.

A recent research was done to investigate the effects of steel fibers on the concrete breakout of the cast-in-place headed stud anchors in tension by (Karthik Vidyaranya 2019). Three dosages of steel fiber (0.0, 0.5, and 1.0%) were used to predict concrete breakout with same concrete mix proportions. The type of headed anchor stud was high strength anchors (F1554 G105). All the work was done in the Civil Engineering Laboratory in University of Texas at Arlington to figure out the mechanical and physical properties. The mechanical properties cover three cylinders 4''x8'' to calculate compressive strength, three cylinders 6''x12'' to calculate split tensile strength, and three beams 6''x6''x20'' to calculate the flexural strength. All testes were done at age 28 days. Nine studs were installed in three beams (54"x16"x10") which is mean three anchors for each beam. These anchors were installed individually to be test for concrete breakout as single anchors within concrete. No group action was associated to that research. The embedment depth was kept constant for all anchors. The concrete mix design was intended to produce (4000 psi) compressive strength in plain concrete. The calculation for concrete breakout was followed the method in Concrete Capacity Design (CCD) as in (ACI 318-14) and modified to be accurately with the experimental results. It was noticed that the compressive strength increased by 35% and 48% for 0.5% and 1% of steel fiber content respectively as exterminate revealed compared with plain concrete. The split tensile test showed increasing in tension by 77% and 107% for 0.5% and 1% of steel fiber content respectively compared with concrete without any steel fiber. It was observed that the flexural strength increased by 15% and 37.4% for 0.5% and 1% of steel fiber content respectively as exterminate revealed compared with plain concrete. Lastly, it was discovered that by increasing steel fiber percentage will decrease the diameter of concrete cone failure and increased failure angle.

Another research was done by (Atheer Alkhafaji) is to investigate the group action of anchors in tension with SFRC and compare these results of concrete breakout capacity with the single anchors by (Karthick). It was use same anchor properties and embedded depth 2.5''. The compressive strength designed to be (4000 Ksi). The dimensions of beams are similar for both researches. The concrete mix proportions were made (0%, 0.5%, 1%, and 1.5%). The research revealed that the concrete breakout increases with increase of steel fiber dosage by 43%, 73%, and 81% respectively. Also, it showed that the concrete cone failure decreases by increasing SFRC by 14%, 48%, and 70% respectively. But when the results compared with single test, the group anchor effect reduced by 19%, 16%, 15%, and 14% corresponding to SFRC dosages. On the other hand, it was noticed that mechanical properties were increased for compressive, split tensile, flexural strength by different percentages.

### 2.1.2 Steel Fiber Reinforced Concrete (SFRC)

A paper by (Z Marcalikove et al. 2019), studied the comparison of material properties of steel fibers reinforced concrete with two types of steel fiber. The same concrete mix proportions were used for these two types of fiber. These two fibers are differing in shape, mechanical and physical properties, one is short and straight (Dramix OL 13/20) and the other is long and bend (Dramix 3D 55 30BG). They added by two dosages 40 and 75 kg/m$^3$. Figure (2.3) shows the two types of steel fiber used in Marcalikove research.

| Property | Dramix® OL 13/20 | Dramix® 3D 55/30BG |
|---|---|---|
| Length [mm] | 13 | 30 |
| Diameter [mm] | 0.21 | 0.55 |
| Flexural strength [N/mm$^2$] | 2750 | 1345 |
| Dosage [kg/m$^3$] | 60 | 25 |
| Modulus of elasticity [GPa] | 200 | 200 |

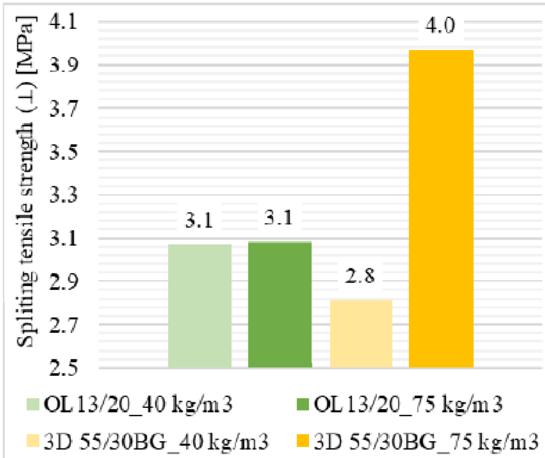
Dramix® OL 13/20　　　　Dramix® 3D 55/30BG

Figure (2.3) Two Types of Steel Fiber

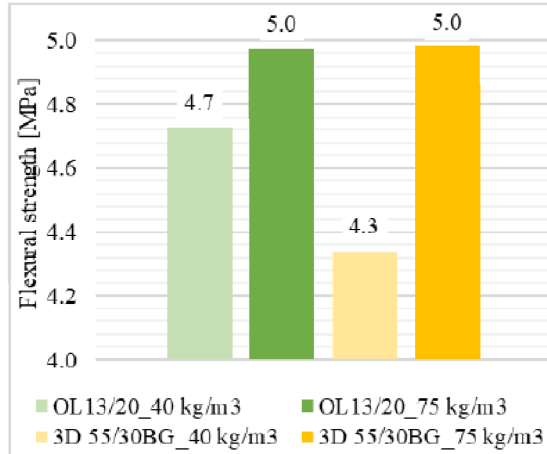
The mechanical properties intended to test was compressive, split tensile and flexural strength. Fiber concrete can vary by two factors: class of concrete and type of fibers. The last one has many physical properties that can effect on fiber concrete like shape, length, diameter, surface finish. It is more favorable to use steel fiber in building structure like floors, foundation, and tunnel linings than normal concrete. The benefit of using SFRC is to increase tensile strength and ductility of concrete. The other advantage is to reduce shrinkage cracking, concrete deformation, and increase toughness and fatigue beside for improving of cohesion. As mentioned above, one of the mechanical properties to measure is compressive strength which is typically sized on cube 150x150x150mm. The compressive cube specimens always tested in perpendicular way of filling. Meanwhile, two variants were chosen: perpendicular and parallel to the way of filling to measure split tensile strength. The reason stand behind that is the checking homogeneity of fibers in test samples. Three point bend was picked to find out the fracture mechanic parameter. In order to get accurately result, it is require to use fiber with 60mm length and suitable concrete with grain fraction of maximum 32mm. The size of aggregate fraction has larger impact on the value of fracture energy. Twelve test specimens were used to predict the compressive strength and to cover all variants of dosages (40, 75 kg/m$^3$) and types of steel fiber (OL, 3D). The compressive strength in 75 kg/m$^3$ dosage was higher than the compressive strength in 40 kg/m$^3$, but the difference between two types of steel fiber was relatively small. The compressive strength for the 40 kg/m$^3$ was 36.4 Mpa for Dramix OL13/20 and 38.5 Mpa for Dramix 3D 55/30BG, but the compressive strength for dosage 75 kg/m$^3$ was 40.1 MPa for Dramix OL13/20 and 40.9 MPa for Dramix 3D 55/30BG. Twenty specimens were used to predict split tensile strength and divided by 12 sample tested perpendicularly to the filling way and rest of them parallel to filling way. The splitting tensile strength was minimal for both types of loading: perpendicular and parallel and dosages. It was noticed that the difference in split tensile strength was greater in the perpendicular direction than parallel for high dosages. The specimens of flexural beam were prepared in 12 beams and divided by three for each steel fiber dosage and type. These beams tested at age 28 days as similar to compressive and tensile strength. The flexural strength was very similar for both type of dosing. It was ranging (4.7 to 5, and 4.3 to 5 Mpa) for OL and 3D respectively. It was observed that the load - deformation curve was very similar but slightly greater residual strength occurs at higher dosages of steel fiber and it’s clearly pronounced for Dramix 3D 55/30BG. The test showed that flexural strength in parallel filling direction is lower than perpendicular.

Table (2.2) The Strength Corresponding to Two Types Fibers

Splitting Tensile Strength Perpendicular to Filling Direction




Flexural strength




In conclusion, it is very important to know the mechanical and fracture properties of steel fiber reinforced concrete members to enhance of structural design. It is worthwhile that these properties have to be evaluated on standard specimens and with standard recommendations. The proposed way to estimate and analyze the post-cracking behavior in tension and toughness properties was use different types of specimens, experimental test procedures and parameters. It is clearly that the steel fibers have a positive and negative impact once added to the concrete as shown from previous studies in literature reviews.

## 2.2 Advantages vs Disadvantages of Steel Fiber in concrete

The advantages of using steel fibers in concrete can be point out as following:

1. Increasing the compressive strength.
2. Increasing the tensile strength.
3. Increasing the flexural strength.
4. High durability.

5. Reducing the shrinkage cracking in concrete.
6. Reducing the concrete deformations.
7. Increasing the toughness and fatigue strength.
8. Improving the cohesion.
9. More Ductility of the concrete.

Disadvantages can be listed as following:

1. Reducing the workability of concrete.
2. Fibers may get concentrated at few places which is not ideal and turn results in poor quality of concrete.
3. Using of SFRC requires more accurate configuration as opposed to normal concrete.
4. Fiber-reinforced concrete tends to be more expensive than ordinary concrete.
5. Fiber-reinforced concrete is heavier than non-fiber concrete.
6. Steel fiber is difficult to self-mix. Generally, a contractor will mix and pour or spray this type of concrete.
7. Fibers in concrete make concrete very harsh and it is difficult to handle and pose problems during placement.

# CHAPTER THREE

# EXPERIMENTAL WORKS

## 3.1 Fabrication of Test Specimens

### 3.1.1 Design of Formwork Specimens

Five types of specimens were prepared according to the test: compression test, split tensile test, modulus of rupture, single anchor, and finally, the anchor groups' tension test. Six types of formwork specimens were designed according to the specifications of ACI 318-19. The size of the formwork beam is 53’’x7.5’’x9" with pedestal sized 2.5’’x7.5’’x20", 53’’x10’’x9" with pedestal sized 5’’x10’’x20", and 53’’x12.5’’x9" with pedestal sized 7.5’’x12.5’’x20" to ensure the anchor groups and single are satisfied. For both group and single pullout tests, the formwork pedestal sizes were prepared according to the specifications of ACI 318-19. The edge distance, according to the ACI code, must be 1.5 times the embedded depth. The embedded depth is maintained to be the same for all specimens except the limit for the edge distance, which is picked to be in three limits. These limits are 1.5 times embedded depth, one times embedded depth, and 0.5 times embedded depth to ensure that the edge distances effect can be studied. The spacing between two anchors in one set of tests was set up according to ACI to ensure they were satisfied. This large beam provides enough housing for the three sets of anchors of one certain edge distance in one shaft, which can allow setting up the hydraulic ram on the beam. Without any interaction or dissipation, the stress with other parts is not included in the study. The reaction of beams to the set-up base plate or fixture of hydraulic ram cannot be involved and affect the surrounding area of the anchor if it is set in a group or single. The adequate embedment depth of the anchor's group is (2.5 inches) which represents five times the diameter of the anchor, and the spacing between two anchors is (5 inches) which means two times the embedded depth as per specifications of ACI 318-19 Chapter 17. See Figure 3.1to Figure 3.14. All dimensions are in inches.

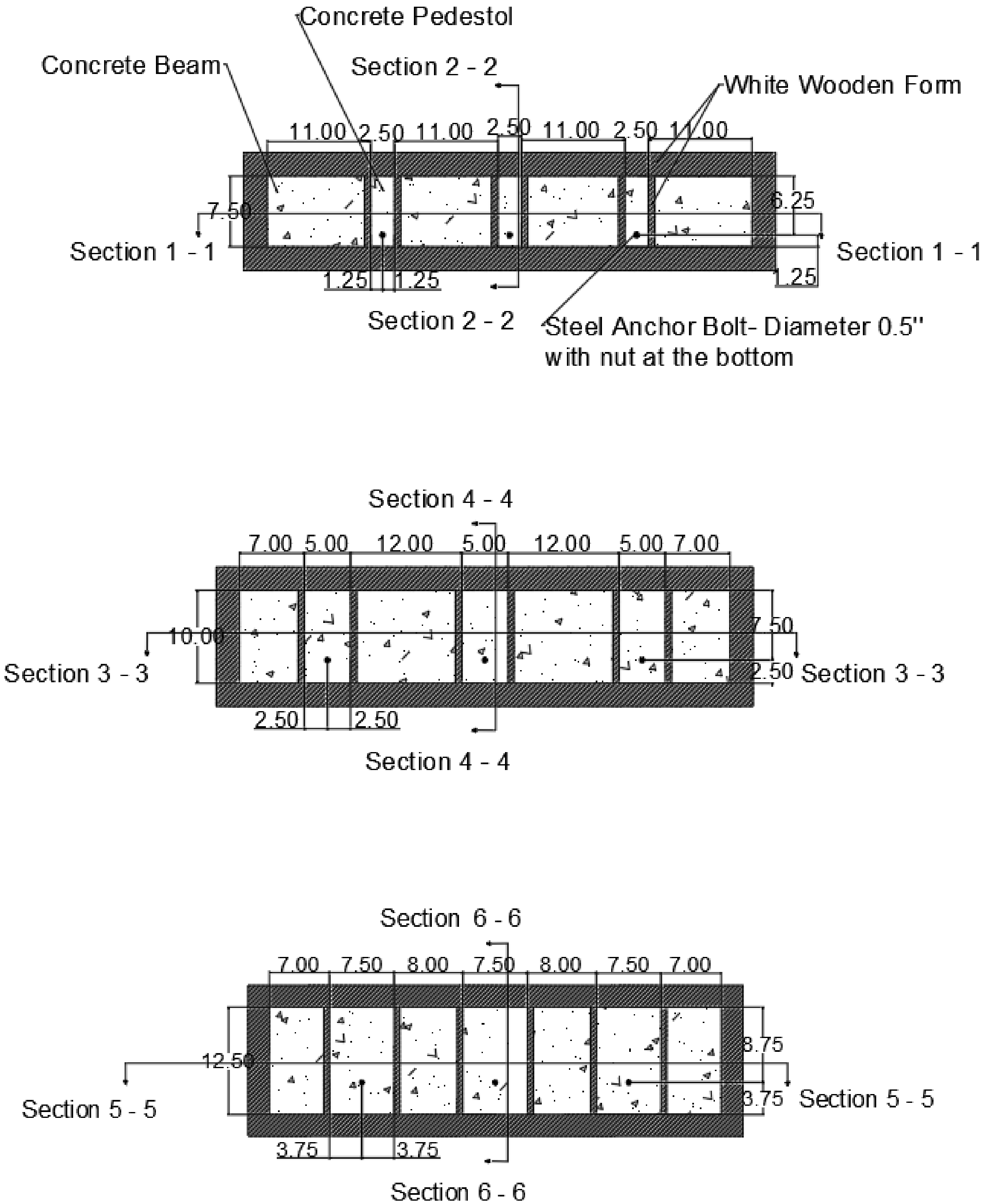



Figure 3.1 Specimen Plans View for Single Anchor Sets, all dimensions in inch.

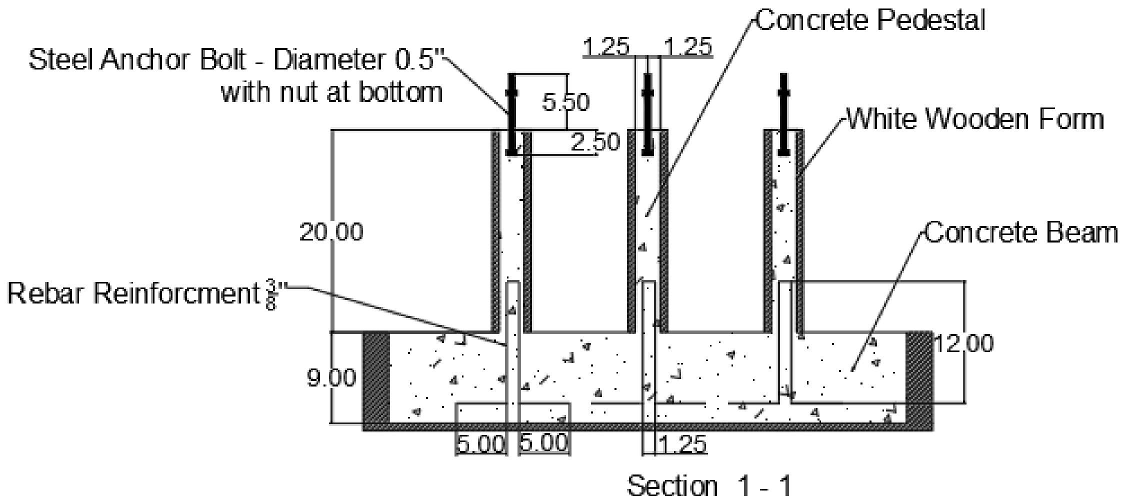



Figure 3.2 Specimen Sections (1-1), all dimensions in inch.

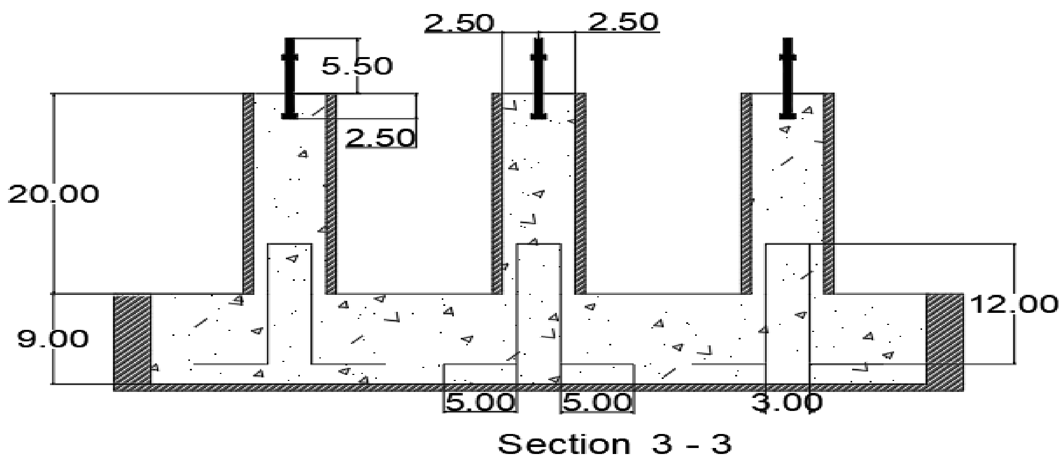



Figure 3.3 Specimen Sections (3-3), all dimensions in inch.

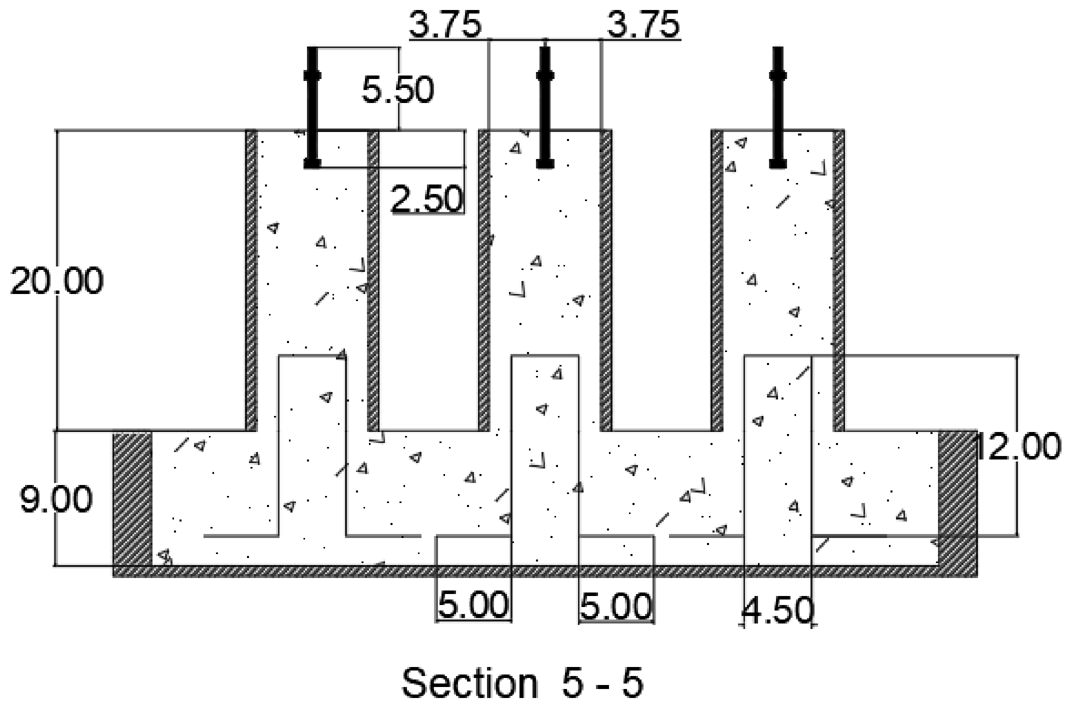



Figure 3.4 Specimen Sections (5-5), all dimensions in inch.

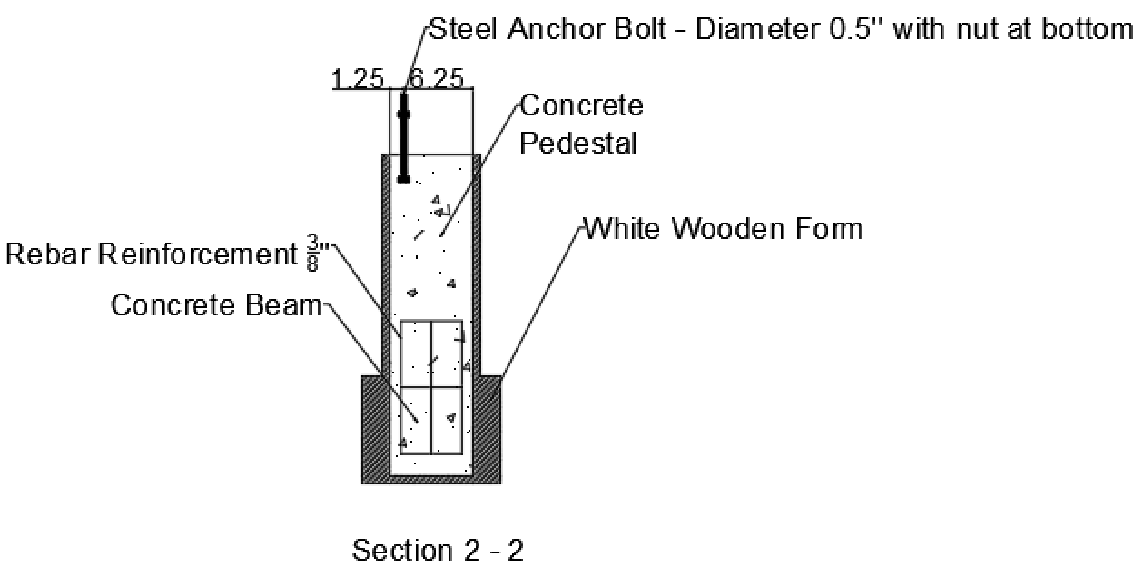



Figure 3.5 Specimen Sections (2-2), all dimensions in inch.

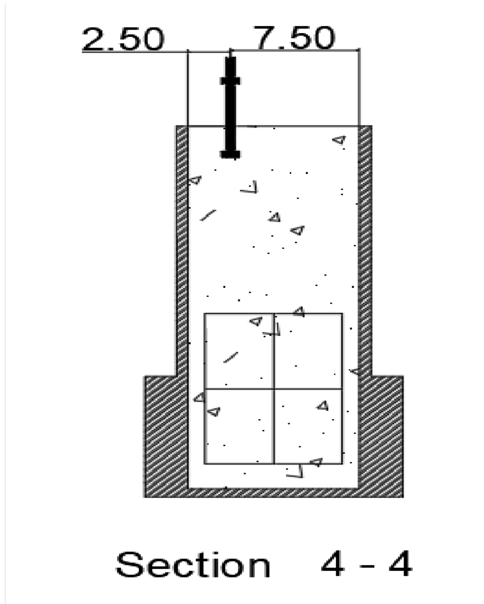



Figure 3.6 Specimen Sections (4-4), all dimensions in inch.

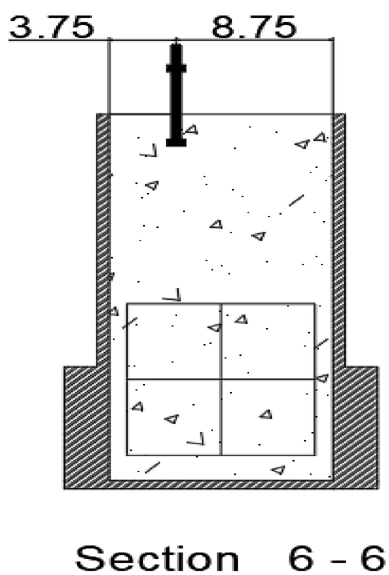



Figure 3.7 Specimen Sections (6-6), all dimensions in inch.

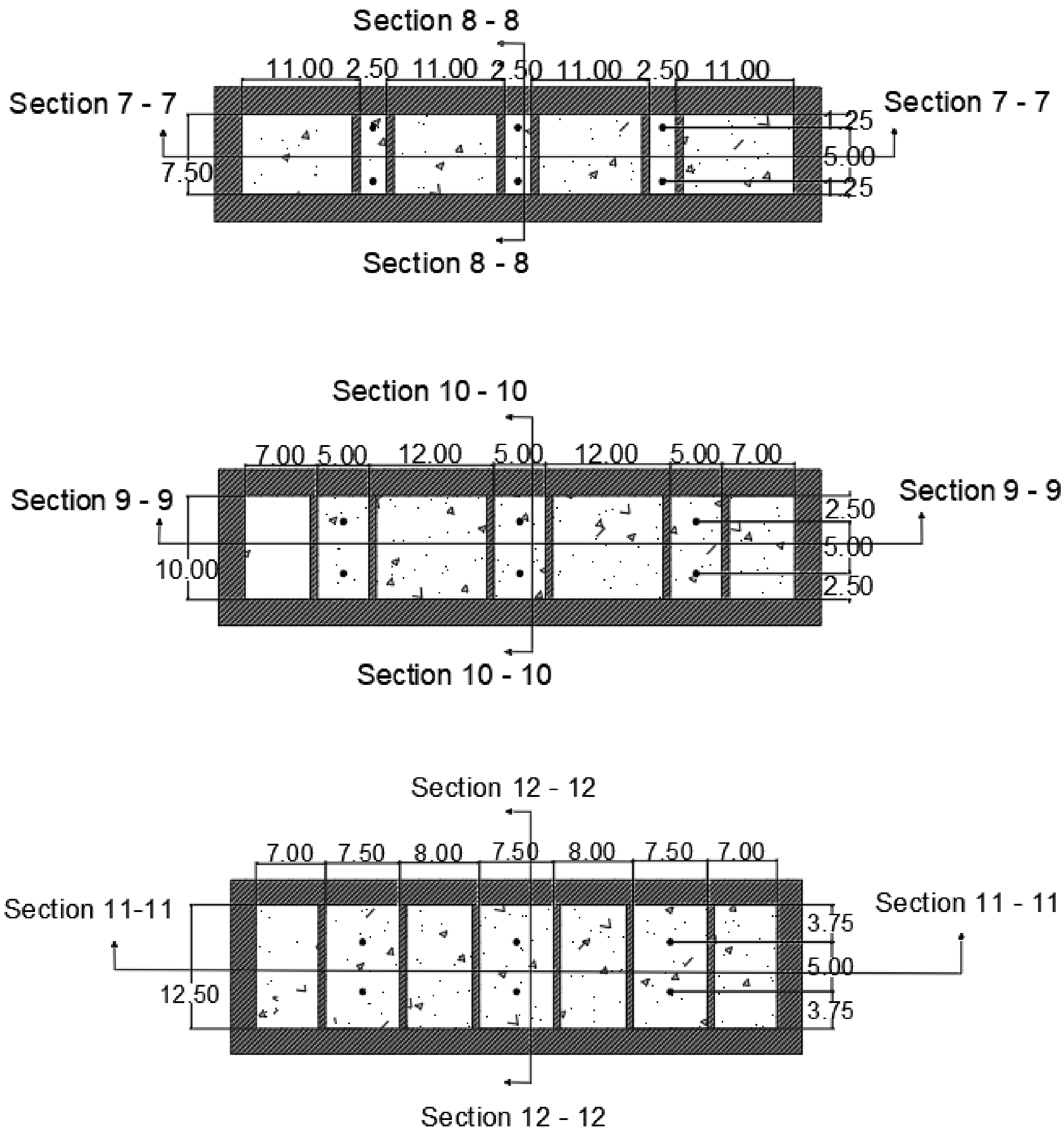



Figure 3.8 Specimen Plans View for Group Anchors Sets, all dimensions in inch.

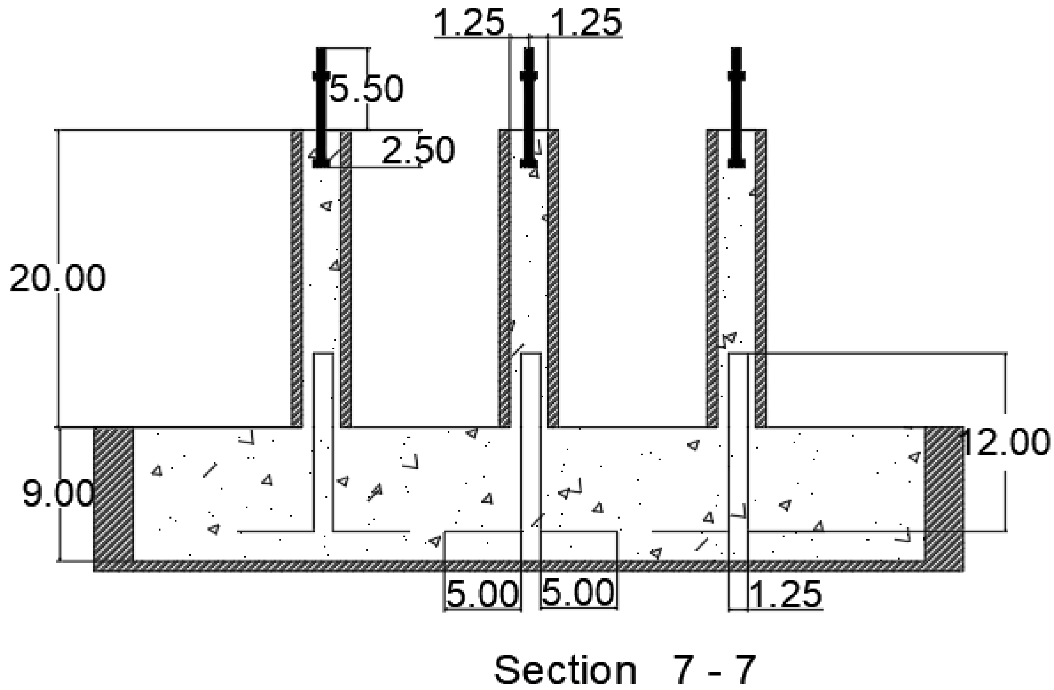



Figure 3.9 Specimen Sections (7-7), all dimensions in inch.

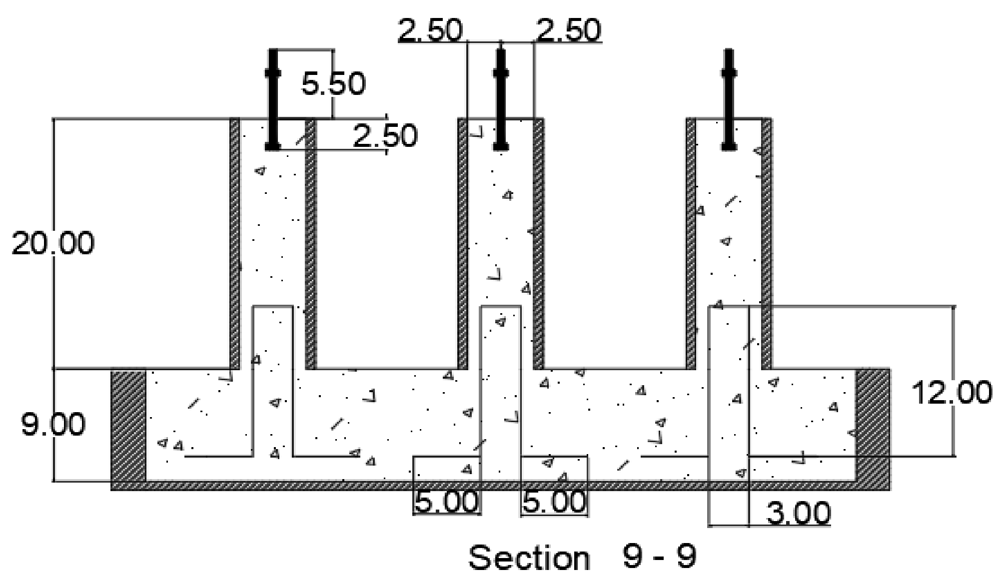



Figure 3.10 Specimen Sections (9-9), all dimensions in inch.

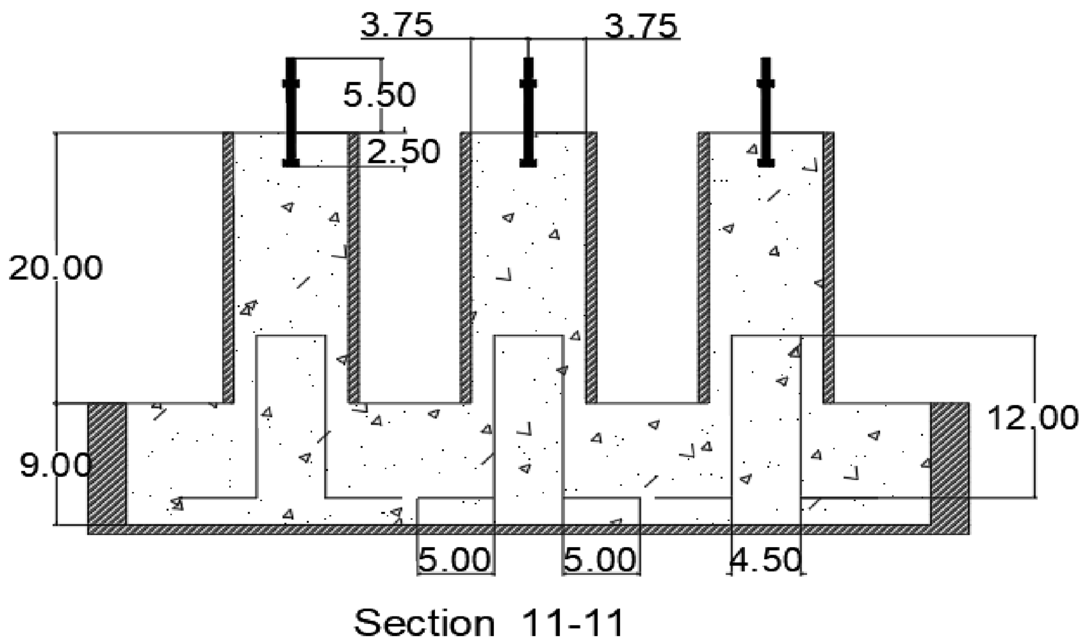



Figure 3.11 Specimen Section (11-11), all dimensions in inch.

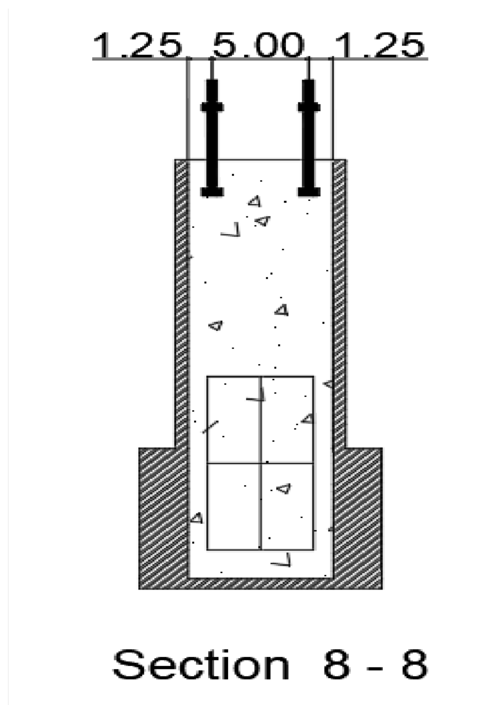



Figure 3.12 Specimen Sections (8-8), all dimensions in inch.

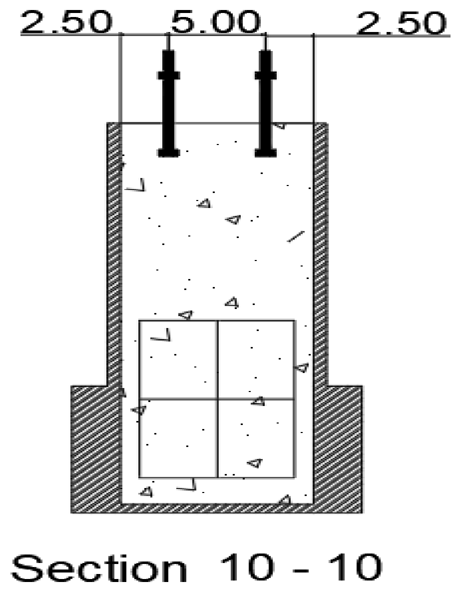



Figure 3.13 Specimen Section (10-10), all dimensions in inch.

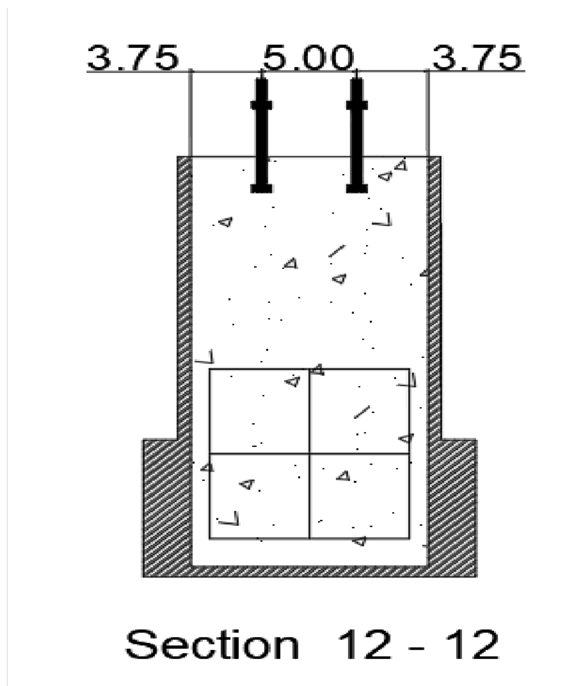



Figure 3.14 Specimen Section (12-12), all dimensions in inch.

### 3.1.2 Preparing of Formwork

Wooden formwork specimens were constructed for 18 concrete beams and illustrated in Figure (3.15). Preparing a typical white wood section (2” x 9”) as per the design of the formwork specimen was required. A Typical (2”x9”) wood section was cut and nailed together to create the formwork specimen. As well as, (32/48”) plywood was cut and fixed to the sides of the entire pedestal frame. Additional (1/2”) plywood was nailed to the exterior faces of the frame to ensure that the pressures from the concrete beam during the pouring stage would not affect the created frame. The work was done professionally to ensure all sides were connected and sealed firmly to prevent any loosing or discharge of concrete. The interior panels of the formwork frame are oiled up to avoid any sticking between the frame and the poured concrete by using WD-40.

Besides the formwork, the rebar was used in this research. The size of the rebar is 3/8’’, which was adequate to resist any separation of beam and pedestal. The rebar was shaped as shown below and connected by using wire. It is essential to maintain the concrete cover for the rebar. This rebar was placed away from anchors by 15’’ to avoid any interlocking or contraction of stresses.

Figure 3.15 Steps of Timber Formwork Specimens

### 3.1.3 Material and Mix Design

The experimental study aims to understand the material's behavior of both concrete and the action of the anchor single and groups under the tensile test. The concrete and steel fiber are the main materials used in this test; the concrete itself is a composite material that its constituents contain cement, aggregate, and water. The concrete mixture was prepared from the cement, sand, gravel, and adding of water. The cement specification was type I / II. Clean and good quality of fine and coarse aggregate which were used in the mixing was provided by UTA in the construction stack yard. The size of fine aggregate were used was ranging between (3/8'' - #100) and for coarse aggregate was ranging between (3 1/2'' – 1 1/2''). In this process is very important to weight all material and make sure the quantity control is followed correctly to ensure achieving of good strength. The water cement ratio were used was (0.4). The concrete was mixed by mixers available at the CELB lab. The manufacture brand of concrete mixer is MultiQuip and model is MC94PE. The size of concrete mixer is nine (cf). The electrical vibrator was used to make sure the concrete material well compacted and avoid any voids; Figure (3.16) shows the on-site concrete mix. The targeting design of the compressive strength plain concrete was intend to be (4000 psi), Table (3.1) illustrate the mix proportions of the concrete mixture.

Figure 3.16 On-Site Concrete Mix

The procedure which is followed in this research is preparing cylindrical specimens (4”x8” and 6”x12”) and beams (6”x6”x20”) per each batch, were cast and tested after 28 days of curing to determine the compressive strength, split tensile strength, and flexural strength of the concrete. All the specimens were de-mold after casting in the next day and placed in the curing room for the CELB. Figure (3.17) shows the cylindrical specimens and beams as well.

Figure 3.17 Cylindrical and beam specimens

Table (3.1) Mix Proportions of the Concrete Mixture

| Component | Density (Ibs/cf.) | Weight (Ibs) | Volume (cf.) |
|---|---|---|---|
| Portland Cement I/II | 196 | 1464 | 7.47 |
| Coarse Aggregate (3 1/2''- 1 1/2'') | 161 | 1025 | 6.36 |
| Fine Aggregate (3/8'' - #100) | 176 | 1581 | 8.98 |
| Water | 62.4 | 585 | 9.37 |
| Water/Cement | - | 0.4 | - |
| Concrete Mix Total | 144.7 | 4657 | 32.0 |

The experiment was conducted to give sight and ideas into the real material behavior of the steel fiber reinforced concrete. Also, to show and measure the type of concrete breakout strength under the ultimate tensile load where applied to the anchor groups and individuals. The steel fibers with end bends are used in this study and is called Dramix 3D. Most of the steel fiber industries recommend to use (0.5 % - 2.0 %) of the total weight of concrete and according to previous studies. Figure (3.18) shows the type of steel fiber, Figure 3.19 Data Sheet of Steel Fiber 3D 45/50 BL, and Table (3.2) illustrates the properties of steel fiber as well.

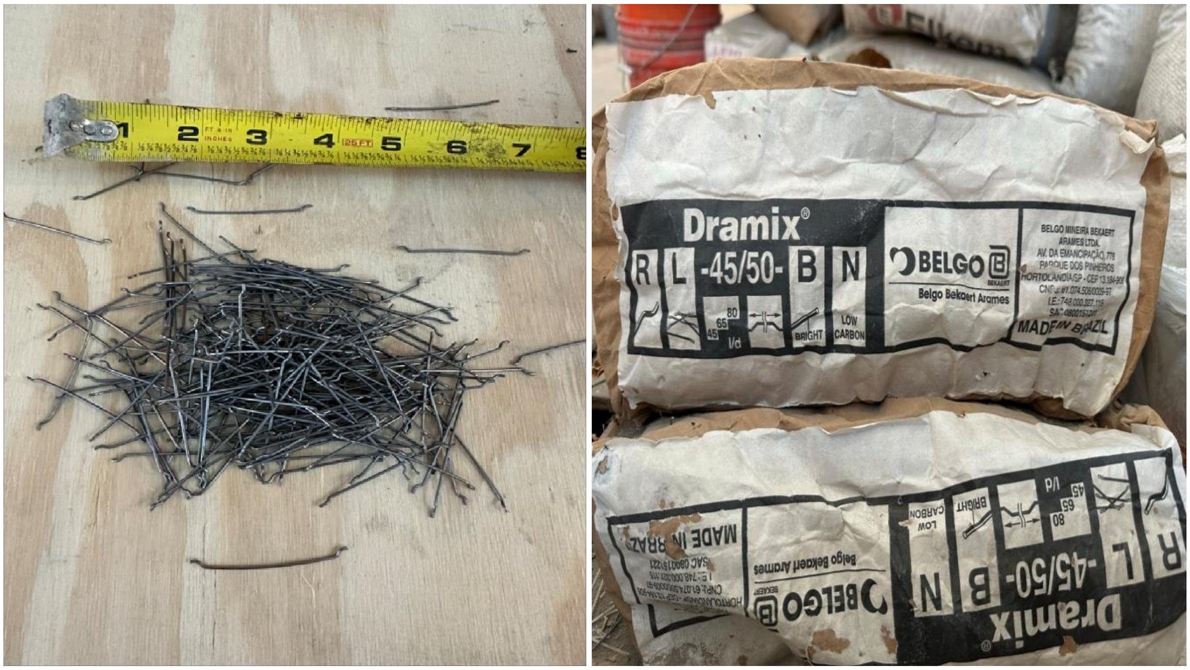



Figure 3.18 Type of Steel Fiber 3D 45/50 BL

Table (3.2) Properties of Steel Fiber

| Type of Fiber | Technical Category Name | Length mm (in) | Diameter mm (in) | Aspect Rastio (λ=L/D) | Tensile Strength N/mm^2 (lb/in^2) |
|---|---|---|---|---|---|
| Bright, Low Carbon, Round Wire/Straight with Bend Ends | 3D 45/50 BL | 50 (2) | 1.05 (0.04) | 45 | 1115 (161717) |

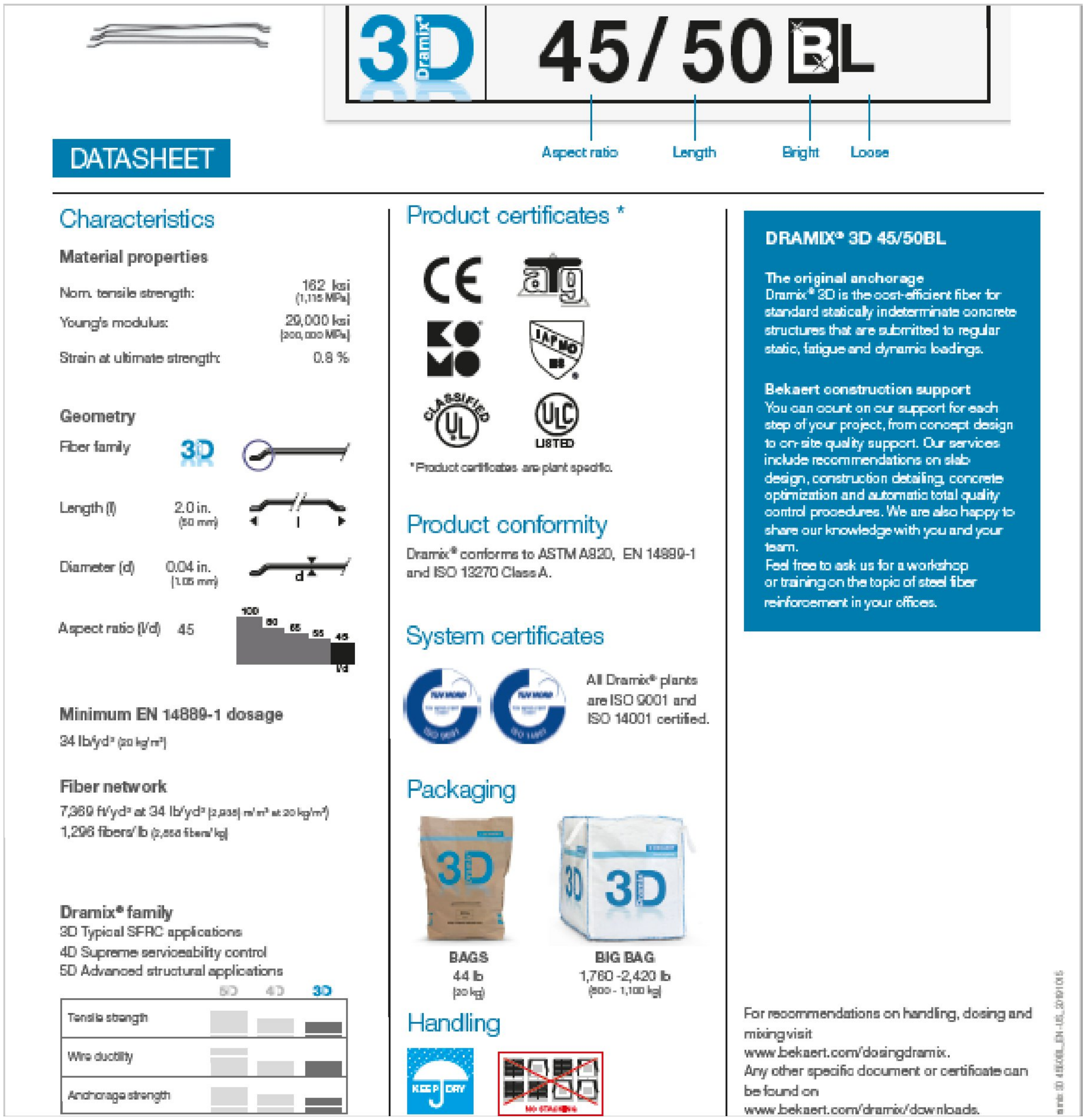



Figure 3.19 Data Sheet of Steel Fiber 3D 45/50 BL

The quantity of RC is the same proportions mix are used with different amount of steel fibers; the calculation of adding steel fiber is according to the percentage of steel fiber required multiplied by the total weight of concrete per designating volume in (Ibs) of the mass concrete intend to be cast. Table (3.3) illustrates the weight of steel fiber for each concrete mix.

Table (3.3) Weight of Steel Fiber for Each Percentage per 27 c.f

| 0.0% Steel Fiber | 0.5% Steel Fiber | 1.0% Steel Fiber |
|---|---|---|
| 0 lb (Plain concrete) | 20.25 lb | 40.5 lb |

### 3.1.4 Casting of Concrete

To keep the shape of all casted concrete clean and free of any loose parts, the formworks and the small specimens are prepared by spraying the inside faces with WD-40. The WD-40 uses in many concrete work as a concrete releasing agent and prevents the sticking between the formwork and the poured concrete so it will make it easy to remove the form and reduce any damage could be happen to samples. Before start casting, it must make sure the fine and coarse aggregate are free from any debris or dirties. The purified water must be use in mixing and should free from any salt. The concrete was mixed by using of the mixer available at the CELB; the following steps are made to produce the concrete:

1- In the beginning, add the coarse aggregate to the concrete mixer.

2- Adding of the fine aggregate to the mixture.

3- It's preferred to add Portland cement after well mixing of fine and coarse aggregate together to make sure perfect particle distributing through the entire mix and to prevent any sudden harden or agglomeration.

4- Gradually add water to the mixture to the appropriate amount base on the designed w/c ratio to obtain good workability.

5- In the case of SFRC, the steel fibers added (% by weight of concrete) before adding the water to allow the proper distribution of the fibers in the mixture. The purified water was used to cure the samples that casted in the CELB. Figure (3.20) shows the concrete during the pouring stage and after that.

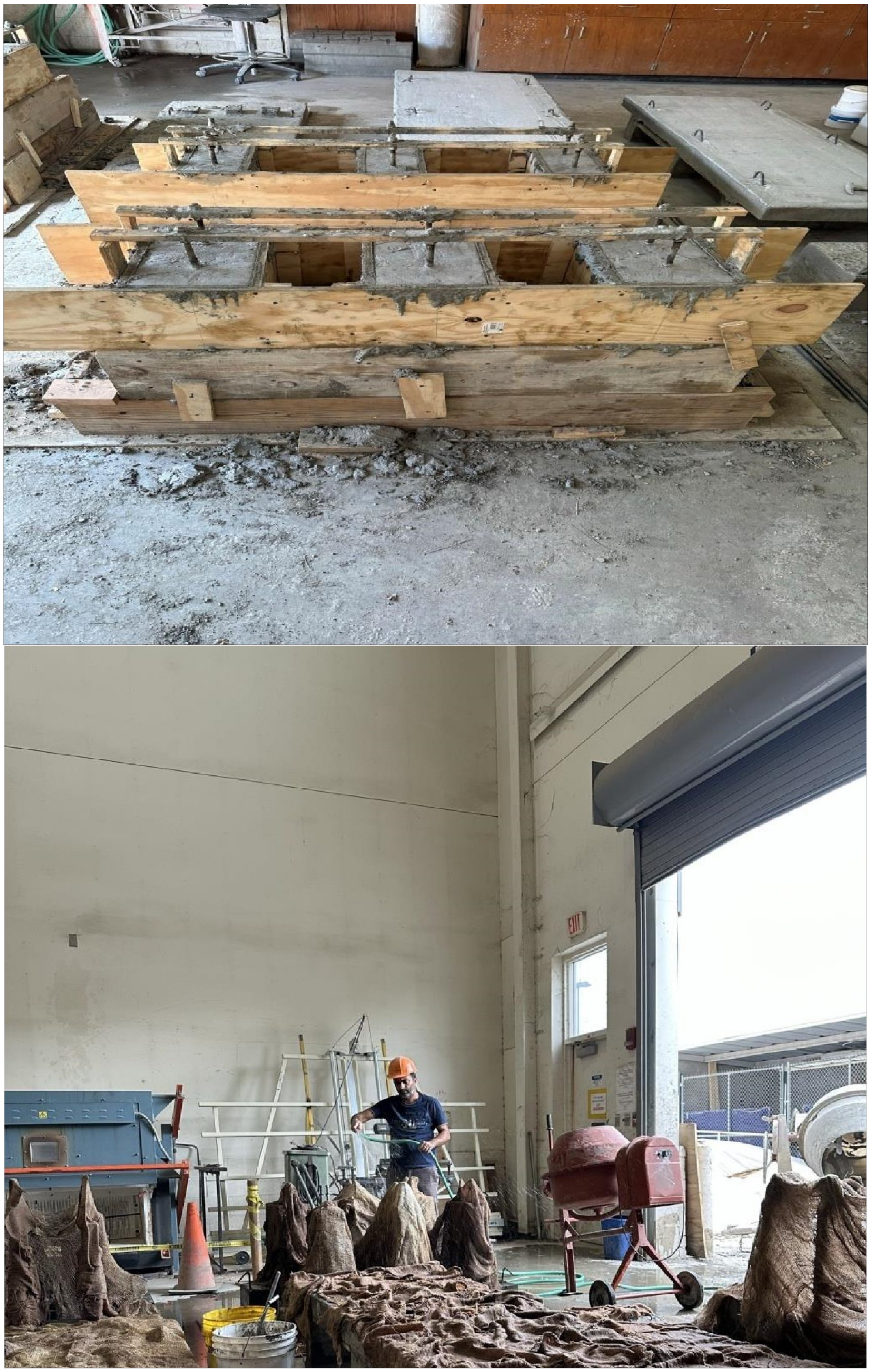


Figure 3.20 SFRC During the Pouring Day and After

One of the required concrete tests is slump which is usually give scale and value itself can demonstrate the workability of concrete. The slump test was performed according to the ASTM C143 (Standard Test Method for Slump of Hydraulic-Cement Concrete). The device of slump test is including base plate, cone, and bar. To measure the slump; the standard cone is 8" base, 4" top, and 12" height was used. The slump cone is used on the steel base plate and the concrete is poured inside the cone in three layers. For each layer, the concrete is tamped with steel rod 25 times per each layer. When the cone is filled with concrete, the top surface is made to be smooth. Then, lifting of the cone should be within 5 seconds. It is usually flip the cone and placed next to the concrete to use cone as reference height. Distance from the top of the cone to the top of poured concrete is considered the slump value.

It was discovered that the slump value is changing with the same w/c ratio for all the concrete mixtures. The consistency of the mix was influenced due to adding steel fibers, hence the slump value decreases. It was noticed the relationship between percentage of steel fiber and slump is conversely. The slump was decreasing as the percentage of the steel fibers increased and the workability of the SFRC was less than the plain concrete due to these reasons. This effect was clearer and more visible with the addition of a higher percentage of fibers; Table (3.4) illustrates the slump test values for a different amount of steel fiber.

Table (3.4) The Slump Test Value for Different percentage of Steel Fiber

| **Mixture** | **PC (0.0% SF)** | **0.5% SF** | **1.0% SF** |
|---|---|---|---|
| **Slump Value** | 7.5 | 5.5 | 4 |

The concrete specimens were de-molded after 24 hours. It should be take care of de-mold specimen processes and avoid any damage that could effect on the results of concrete test. The specimens should be labeled with the percentage of concrete to avoid any confusion. Finally, the specimen is placed inside the curing room with specific temperature to the date of the test (after 28 days). Figure (3.21) shows the concrete specimens in the curing room.


Figure 3.21 Concrete Specimens in the Curing Room

### 3.2 Test Set-Up

The Civil Engineering Laboratory Building at the University of Texas at Arlington is featured with all necessary equipments to conduct and perform all tests. The cylinder compression test, split tensile test, flexural test, and finally, the anchor single and groups pull out test will be discussed technically below per each one of these tests.

#### 3.2.1 Cylinder Compression Test

The standard size of cylinders to conduct compression test is (4’’x 8’’). Concrete cylinders are tested under a uni-axial compression load. This uni-axial load applied parallel to the long direction (8’’), in other word, the load was applied on the surface area with 4’’-diamter. The (400 kips) compression machine was used in this test and found adequate or compatible to the ASTM C39. The standard procedure of the compression test was followed to perform this test. The concrete specimens were loaded at a load rate of (440 Ib/sec) (35 psi/sec) and the ultimate load was recorded. Figure (3.22) shows the images of the compression test set-up and the instrumentation.

Figure 3.22 Compression Test Set-Up

After completing the concrete cylinder's compression test, clearly, the compressive strength was increased by increasing the fraction of steel fibers. Increasing of the compressive strength from SFRC (0%) to SFRC (0.5%) is noticeable; while the increase in strength from (0.5%) SFRC to (1.0%) SFRC is lesser. This is because of increasing the air voids between the concrete and the steel fiber. Figure (3.23) shows the concrete cylinders compression failure for different types of SFRC.

(0.0 % SFRC)

(0.5 % SFRC)

(1.0 % SFRC)

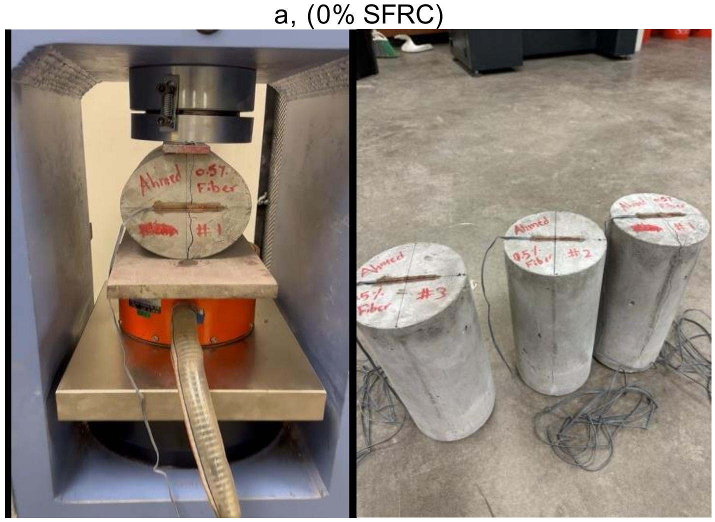
Figure 3.23 Compression Failure vs. Different SFRC

### 3.2.2 Cylinder Split Tensile Test

The standard size of concrete cylinders to conduct split tensile strength is (6”x12”). The concrete cylinders are tested by using the (500 kips) compression machine based on the ASTM C496. The standard procedure was followed to perform the test. In this test, the specimen lay down horizontally and the load was applied across the length of concrete cylinders. The specimens were loaded at an approximate load rate of (190-380 Ib/sec) (100-200 psi/min) until the concrete specimens develop a tension crack along their diameter beneath applied load all over the length. The ultimate load due to the triaxial compression force is used in determining the split tensile strength; Figure (3.24) shows the tensile test set-up and the instrumentation. The strain gauge was attached to the one side of the cross section to measure strain. After all, the split tensile strength is increased significantly by increasing the content of the steel fibers and the increase for the strength from (0%-0.5%) has been more pronounced than the increase in strength from (0.5%-1.0%). On the hand, the strain in plain concrete is less than the strain in the cylinders that has steel fiber in it. In despite of the small difference in strain between plain concrete and the one who has fiber, but the strain for the cylinders with steel fiber is similar. The strain increased 30% for the cylinders that has steel fiber from (0%-0.5%). See Figure (3.25) shows the specimen's tensile failure for different types of SFRC.


Figure 3.24 Split Tensile Test Set-Up

a, (0% SFRC)



b. (0.5 % SFRC)

c. (1.0 % SFRC)

Figure 3.25 (a-c) Split Tensile Failure vs Different SFRC

### 3.2.3 Flexural Beam Test

Flexural beam test is another indirect method of testing and evaluating the tensile strength of the concrete. The standard size of flexural beams based on the ASTM C78 is (6”x6”x20”). The standard method ASTM C78 was followed for this test to obtain the flexural strength of concrete. The test is called third-points bending test and the clear span was set to (18”) center to center of supports. The center of upper bearer distance was set to (9”) from the center of any supports. The upper bearer has two pin loads and the distance was set to (6’’). Strain gauge was attached at the center of clear span in the farthest bottom fiber of beam to measure strain in tension fiber. Also, the LVDT was set up to measure displacement of the top fiber which is exposed to compression stresses. The concrete beams were loaded at an approximate load rate of (100 Ib/sec) and the ultimate load was recorded to determine the modulus of rapture. Figure (3.26) shows the flexural test set-up and instrumentation.


Figure 3.26 Flexural Test Set-Up

The flexural strength for the concrete beams is showing that the result increasing proportionally by the increase of the steel fibers. Consequently, the increase of steel fiber leads to the highest flexural strength. To conclude from these results, the tension capacity in the cross section below the neutral axis is increased gradually by increasing fiber. So, this type of steel fiber can make positive effect on the flexural strength. It was noticed that the displacement is increasing by increasing the steel fiber. In despite of the small difference of displacement between plain concrete and the one that has fiber, but the displacement for the beams with steel fiber is try to be similar. On the hand, the strain in plain concrete is much more than the strain in the beam that has steel fiber in it. The flexural strength is reducing corresponding to the decreasing in strain and displacement. Figure (3.27) shows the flexural failure for different types of SFRC.


a. (0.0 % SFRC)


Ahmed
0.5%
Fiber
#1
Ahmed
0.5% Fiber
#2
Ahmed
0.5% Fiber
#3
Ahmed
0.5% Fiber
#2
Ahmed
0.5%
Fiber
#1

b. (0.5 % SFRC)


c. (1.0 % SFRC)

Figure 3.27 (a-c) Flexural Failure vs Different SFRC

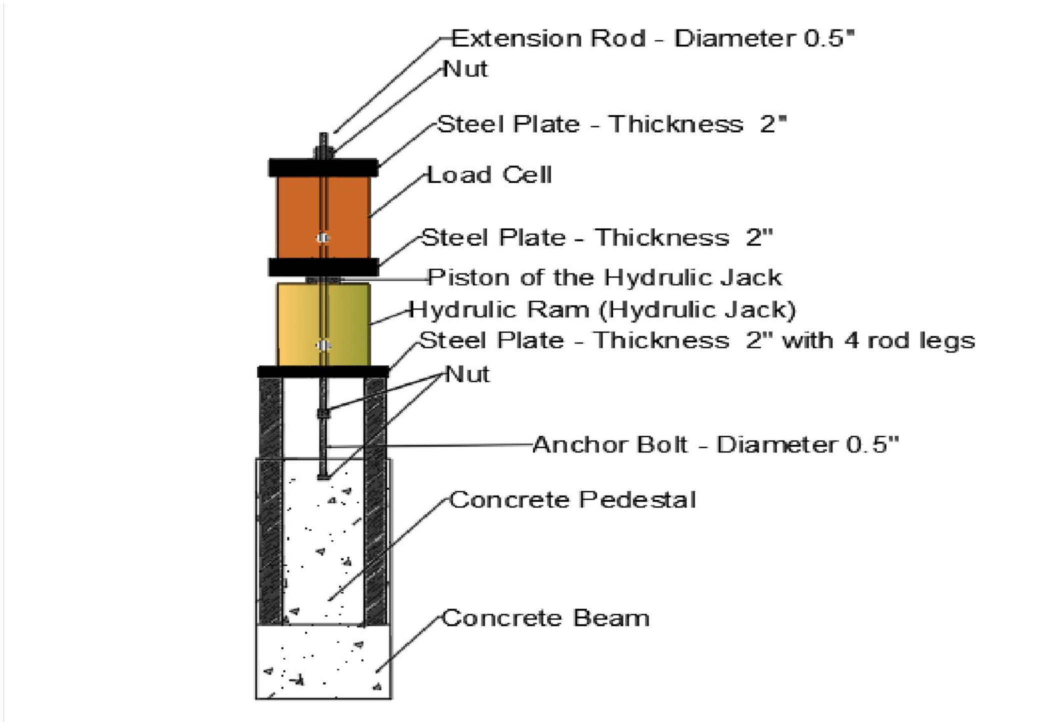
### 3.3 Single Anchor Pull-out Test

The anchor bolt with type (BLK F1554 GRADE 105 ROD, 3” THREADED EACH END) was used in this pullout test (pure tension test) and were performed at the CELB Building. The single anchors were 8” length and 3” threaded and they were embedded vertically without any inclination to depth 2.5” in the concrete beam. The nine sets of single anchors were placed separately. The edge distances are (1.25’’, 2.5’’ and 3.75”) for three sides of the pedestal to every mix design with different steel fiber content. The individual anchor was placed in the wooden frame before the day of pouring by using (1”x2.5”) which was nailed to the frame and holes were drilled based on the requirements. Finally, the bottom part of single anchor was placed with using nuts (BLK A194-2H HVY HX NUT) which should be embedded in concrete to create head stud effect or CCD. Figure (3.28) shows the steel headed stud with the nuts and washers.

Figure 3.28 Steel Headed Stud (F1554 G105)

The standard test method for strength of anchors in concrete elements (ASTM E488) was followed to conduct this test. All single anchor sets were tested individually. The test conducted by placing set-up equipment which includes (steel frame, hydraulic ram, load cell, and steel rod). Figure (3.29) and Figure (3.30) shows the test set-up. This set-up was used on all the anchor sets and set-up includes the following steps:

1- Steel Frame: The steel frame is consisted of one steel plate (2” thickness) and four steel rod legs (30” height and 2.5’’ diameter) and it carries the hydraulic ram and load cell. This steel frame has hole in center to make extension rod bar pass through from anchor to the top of the fixture set up.
2- Hydraulic Ram: This hydraulic jack is set on the steel frame and carry load cell. Hydraulic jack is connected to the Hydraulic pump which is pumping fluid to the hydraulic jack and makes the piston of the hydraulic raise up to compress load cell.
3- Load cell: Load cell is a device used to records the tensile force which is applied to the anchor component. Figure (3.31) shows the load cell used in this test.
4- Extension Rod: The steel rod (0.5” Diameter) x (36” long high) is connected to the individual anchor. This steel rod passes through the hole in hydraulic ram and load cell to the top plate of the fixture set-up.
5- Steel Plate: Couple of steel plates (2’’ thick) used separately to make sure the load uniformly distributed between parts.



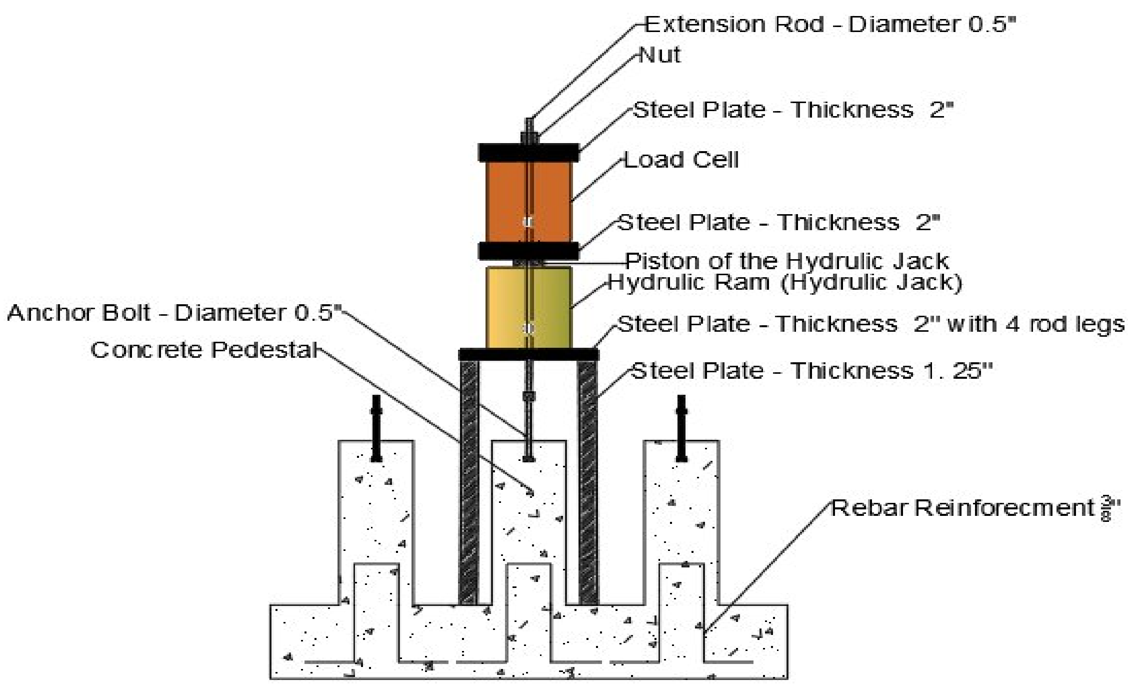




Figure 3.29 Single Anchor Pullout Test Set-Up (Schematic)

Figure 3.30 Single Anchor Pullout Test Set-Up (Lab)

Figure 3.31 Load Cell (500KN-2MN)


Figure 3.32 Research Team during the test day

After the anchor had been tested, the ultimate applied tensile load on anchors was recorded and the breakout or cracked area around the individual anchor was monitored and registered. Figures (3.33) to (3.35) show the concrete breakout and the anchor group's failure for different SFRC.

Figure 3.33 Concrete Breakout Failures (0.0% SFRC)

Figure 3.34 Concrete Breakout Failures (0.5% SFRC)


Figure 3.35 Concrete Breakout Failures (1.0% SFRC)

### 3.4 Anchor Group Pullout Test

The anchor bolt with type (BLK F1554 GRADE 105 ROD, 3" THREADED EACH END) was used in this pullout test (pure tension test) and were performed at the CELB Building. The anchors were 8" length and 3" threaded and they were embedded vertically without any inclination to depth 2.5" in the concrete beam. The nine sets of group anchors were placed separately. The edge distances are (1.25'', 2.5'' and 3.75") for three sides of each anchor in the pedestal to every mix design with different steel fiber content. The group anchor sets was placed in the wooden frame before the day of pouring by using (1"x2.5") which was nailed to the frame and holes were drilled based on the requirements. Finally, the bottom part of group anchor was placed with using nuts (BLK A194-2H HVY HX NUT) which should be embedded in concrete to create head stud effect or CCD. Figure (3.19) shows the steel headed stud with the nuts and washers.

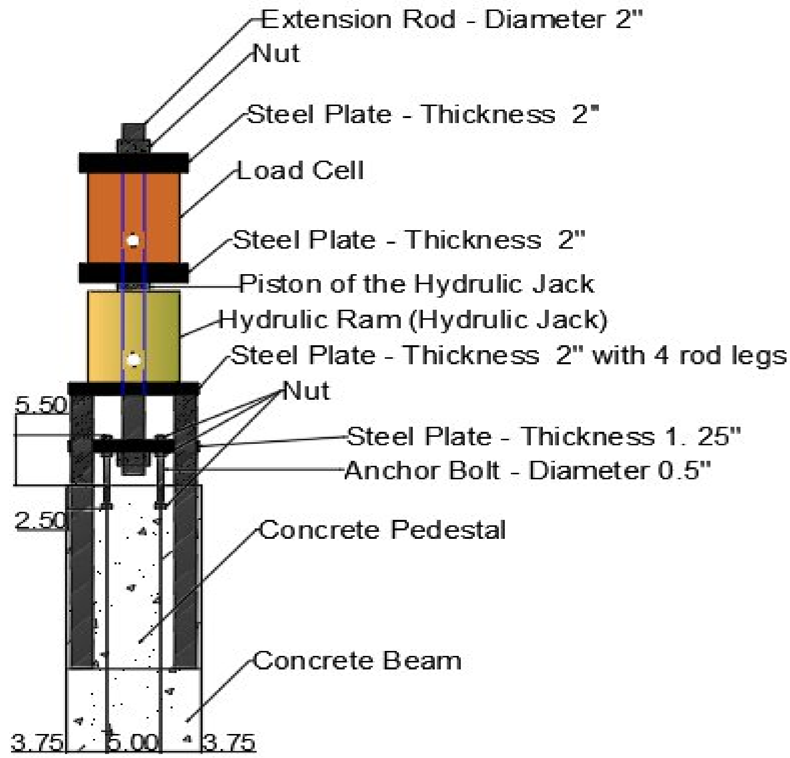
The standard test method for strength of anchors in concrete elements (ASTM E488) was followed to conduct this test. All group anchor sets were tested individually. The test conducted by placing set-up equipment which includes (steel frame, hydraulic ram, load cell, small steel plate, and steel rod). Figure (3.36) and Figure (3.37) shows the test set-up. This set-up was used on all the anchor sets and set-up includes the following steps:

1- Steel Frame: The steel frame is consisted of one steel plate (2" thickness) and four steel rod legs (30" height and 2.5'' diameter) and it carries the hydraulic ram and load cell. This steel frame has hole in center to make extension rod bar pass through from anchor to the top of the fixture set up.
2- Hydraulic Ram: This hydraulic jack is set on the steel frame and carry load cell. Hydraulic jack is connected to the Hydraulic pump which is pumping fluid to the hydraulic jack and makes the piston of the hydraulic raise up to compress load cell.
3- Load cell: Load cell is a device used to records the tensile force which is applied to the anchor component. Figure (3.38) shows the DAQ instrument used in this test.
4- Extension Rod: The steel rod (0.5" Diameter) x (36" long high) is connected to the group

anchor. This steel rod passes through the hole in hydraulic ram and load cell to the top plate of the fixture set-up.

5- Steel Plate: Three of steel plates, two of them (2'' thick) used separately to make sure the load uniformly distributed between parts and the third one is (1.25''thick) which is used to connect two anchors and tied to extension rod.

Some of technical equipment's are used to collect data during test. These equipment's can be call by Computer, DAQ (Data Acquisition), Strain gauges, and LVDT (Linear Variable Differential Transformer). See Figure 3.39.



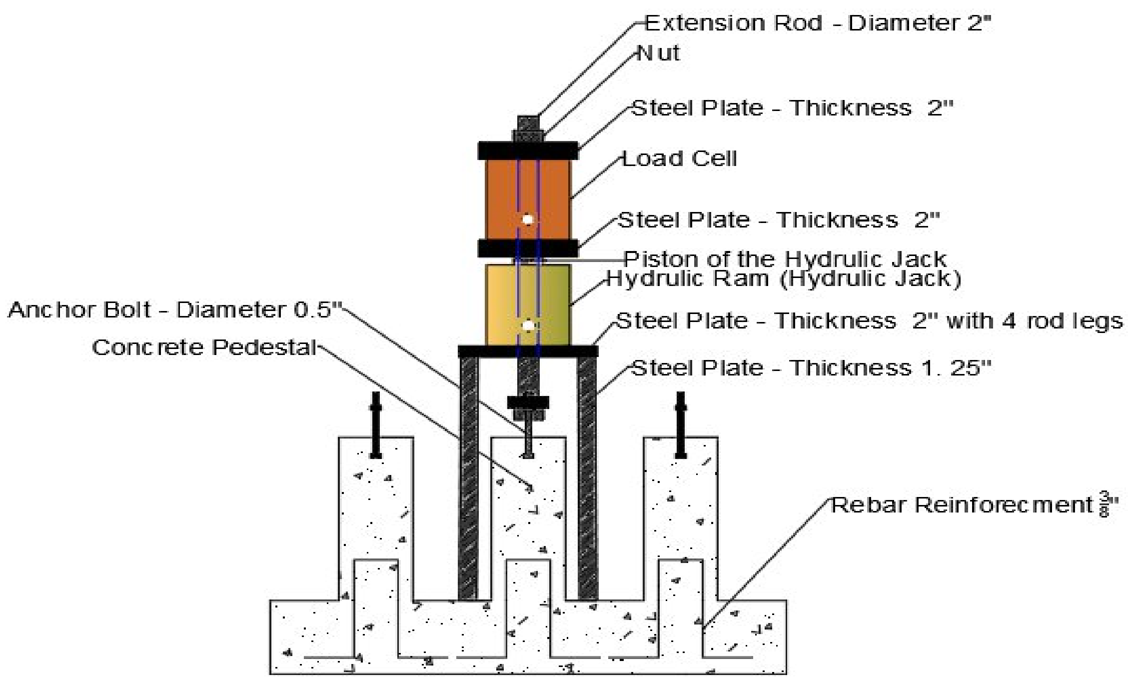




Figure 3.36 Anchor Group Pullout Test Set-Up (Schematic)

Figure 3.37 Anchor Group Pullout Test Set-Up (Lab)

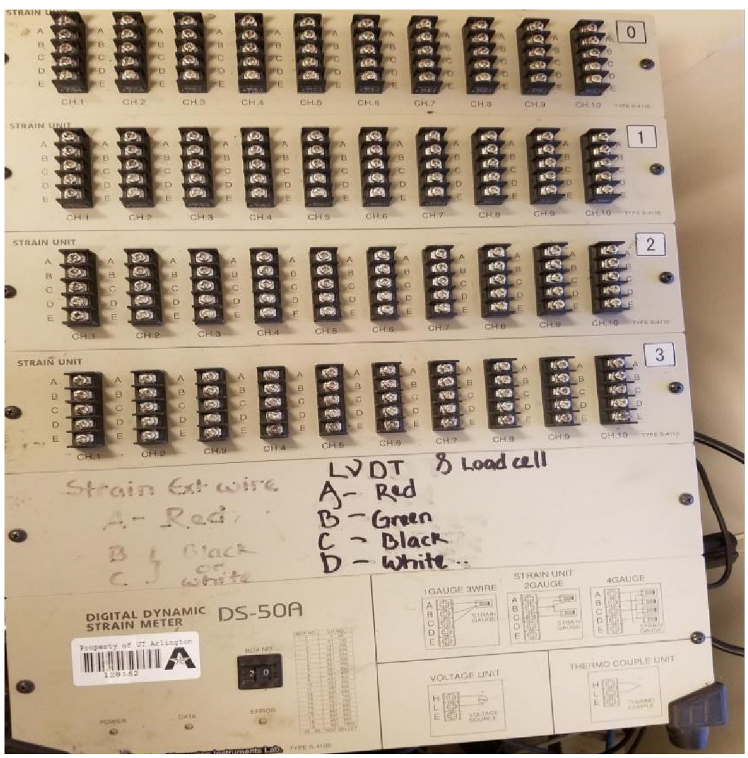



Figure 3.38 DAQ Instrument

Figure 3.39 Research Team during the test day

After the anchor groups had been tested and pulled out. The ultimate applied tensile load on the anchor groups was recorded and the breakout or failure area around the anchor groups was monitored and registered. Figures (3.40) to (3.42) show the concrete breakout and the anchor group's failure for different SFRC.

Figure 3.40 Concrete Breakout Failures (0.0% SFRC)

Figure 3.41 Concrete Breakout Failures (0.5% SFRC)


Figure 3.42 Concrete Breakout Failures (1.0% SFRC)

# CHAPTER FOUR

# EXPERIMENTAL RESULTS

## 4.1 Results of Compression Test

### 4.1.1 Compression Test Data

The ultimate compressive strength (fc`) was determined by using the following equation:

$$fc` = \frac{P}{\pi * r^2} \quad \text{(psi)}$$

Where: (P) is represent the applied ultimate load in (Ibs) over the section area of cylinder (4" diameter).

(r) is represent the radius of the cylinder (2"). Table (4.1) illustrates the compression test results.

Table (4.1) Compressive Strength Test Data

| Steel Fiber Content in The Concrete Mix | Specimen Number | Ultimate Compressive Load (lb) | Compressive Strength fc` = P/ π r² (psi) | Mean Compressive Strength Ẍ (psi) | Standard Deviation $S_x = (\sum_{i=1}^{N}(X-\ddot{X})^2/(N-1))^{0.5}$ | Coefficient of Variation C.V= (Standard Deviation / Mean)*100 | Average Strength (psi) |
|---|---|---|---|---|---|---|---|
| 0% | 1 | 50449 | 4016 | 3874 | 129 | 3.33 | 3875 |
| | 2 | 48282 | 3844 | | | | |
| | 3 | 47256 | 3762 | | | | |
| 0.5% | 1 | 53546 | 4263 | 3991 | 280 | 7.016 | 3993 |
| | 2 | 46516 | 3703 | | | | |
| | 3 | 50334 | 4007 | | | | |
| 1.0% | 1 | 53800 | 4283 | 4246 | 289 | 6.806 | 4248 |
| | 2 | 49500 | 3941 | | | | |
| | 3 | 56728 | 4516 | | | | |

### 4.1.2 Compression Test Results

The values of compressive strength can be studied to understand the effect of steel fiber on it. The compressive strength has an increasing trend with the increase of a fraction of steel fibers in concrete. The increasing in the average strength from (0.0% -0.5%) is (3.04%) and the increasing from (0.0% - 1.0%) is (9.62%). This could be attributed to the air content of the concrete, which many researchers believe that the air content increases with the increase of the steel fibers volume fraction. Also, the second reason could be the specification of steel fiber which plays a big role in changing the behavior of concrete and mechanical properties as well. Figure (4.1) shows the average compressive strength for different SFRC. The coefficient of variation (C.V %) is (3.33% - 7.016%) within the limits of ASTM C39 which is (10.6%).

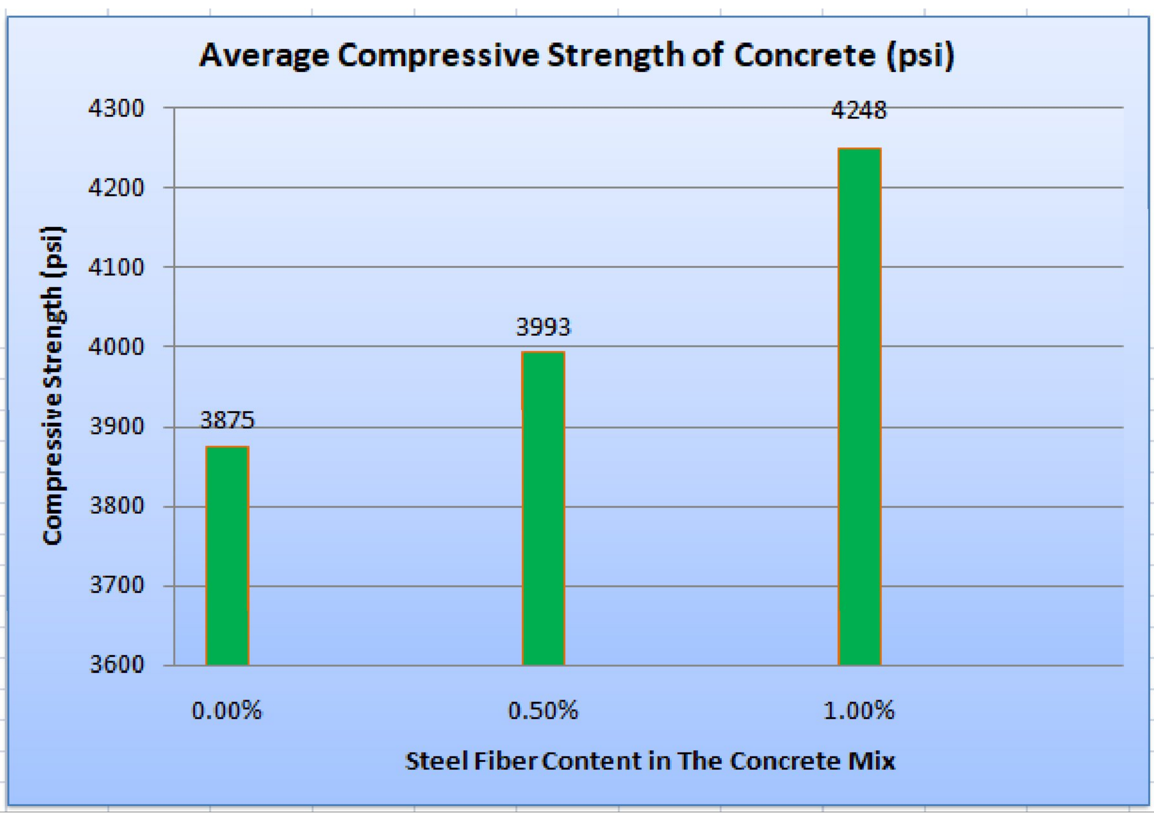



Figure 4.1 Averages Compressive Strength vs. SFRC

## 4.2 Results of Split Tensile Test

### 4.2.1 Tensile Test Data

The split tensile strength ($f_t$) was determined by using of the following equation:

$$f_t = \frac{2*P}{\pi*L*D} \qquad \text{(psi)}$$

Where: (P) is represent the Ultimate applied load in (Ibs) along length of cylinder.

(L) is represent the length of the cylinder (12").

(D) is represent the diameter of the cylinder (6"). Table (4.2) illustrates the tensile strength test results.

Table (4.2) Split Tensile Strength Test data

| Steel Fiber Content in The Concrete Mix | Specimen Number | Ultimate Split Tensile Load (lb) | Maximum Strain at failure Load (μ) | Tensile Strength $f_t = 2P / \pi LD$ (psi) | Mean Tensile Strength $\ddot{X}$ (psi) | Standard Deviation $S_x = (\sum_{i=1}^{N}(X-\ddot{X})^2/(N-1))^{0.5}$ | Coefficient of Variation C.V= Standard Deviation / Mean | Average Strength (psi) | Average Strain (μ) |
|---|---|---|---|---|---|---|---|---|---|
| 0.0% | 1 | 46126 | 62.85 | 408 | 427.6 | 64 | 14 | 431 | 61.263 |
| | 2 | 42456 | 49.52 | 375 | | | | | |
| | 3 | 56609 | 71.42 | 500 | | | | | |
| 0.5% | 1 | 53988 | 78.09 | 477 | 468.3 | 32 | 6 | 470 | 79.68 |
| | 2 | 48921 | 76.19 | 432 | | | | | |
| | 3 | 56085 | 84.76 | 496 | | | | | |
| 1.0% | 1 | 55735 | 92.38 | 493 | 477.6 | 13 | 2 | 477 | 80.313 |
| | 2 | 53289 | 95.23 | 471 | | | | | |
| | 3 | 53114 | 53.33 | 469 | | | | | |

### 4.2.2 Tensile Test Results

The values of tensile strength can be studied to understand the effect of steel fiber on it. It is evident that the split strength of the concrete is increased with the increase of steel fiber. The increasing of tensile strength from (0.0%-0.5% SFRC) is (9.05%) while the increasing of strength from (0.0%-1.0% SFRC) is (10.67%). The split tensile strength for these certain amount of steel fiber has the same strength behavior. Overall, it is important to study how the steel fibers effect on the failure of the concrete. Figure (4.2) shows the average tensile strength for different SFRC. The coefficient of variation (C.V %) is (2% - 14%) within the limits of ASTM C496 which is (0.0%>C.V.>14%).

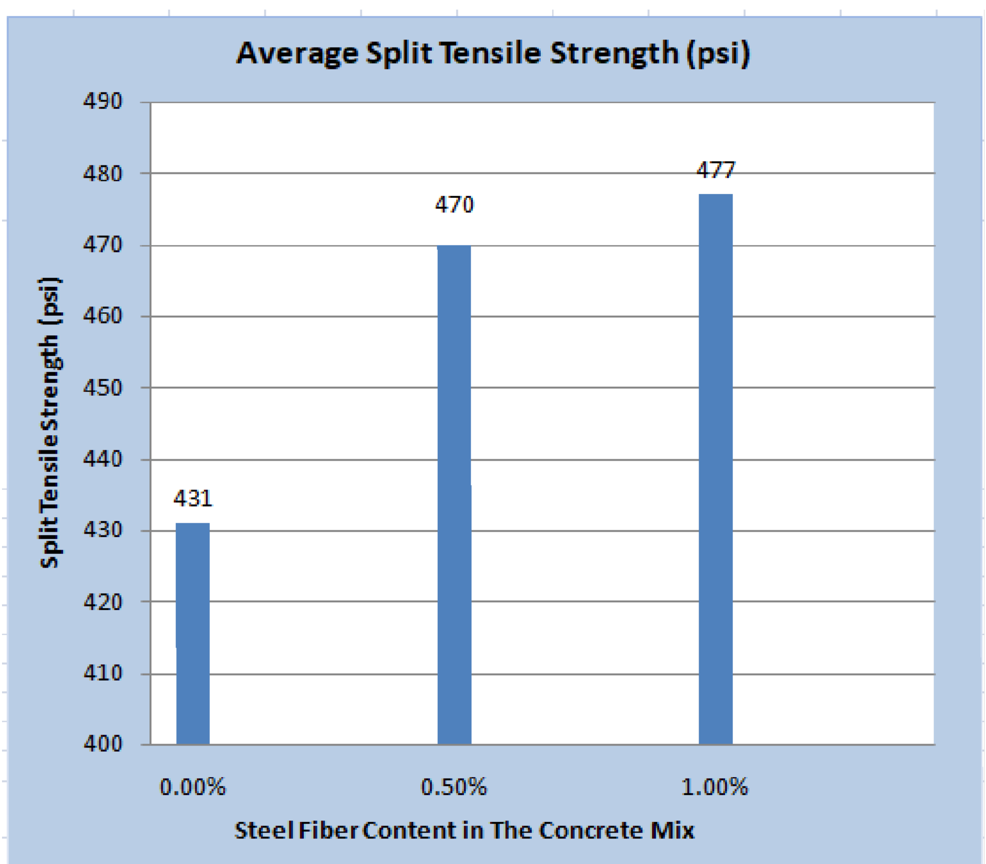



Figure 4.2 Average Split Tensile Strengths vs. SFRC

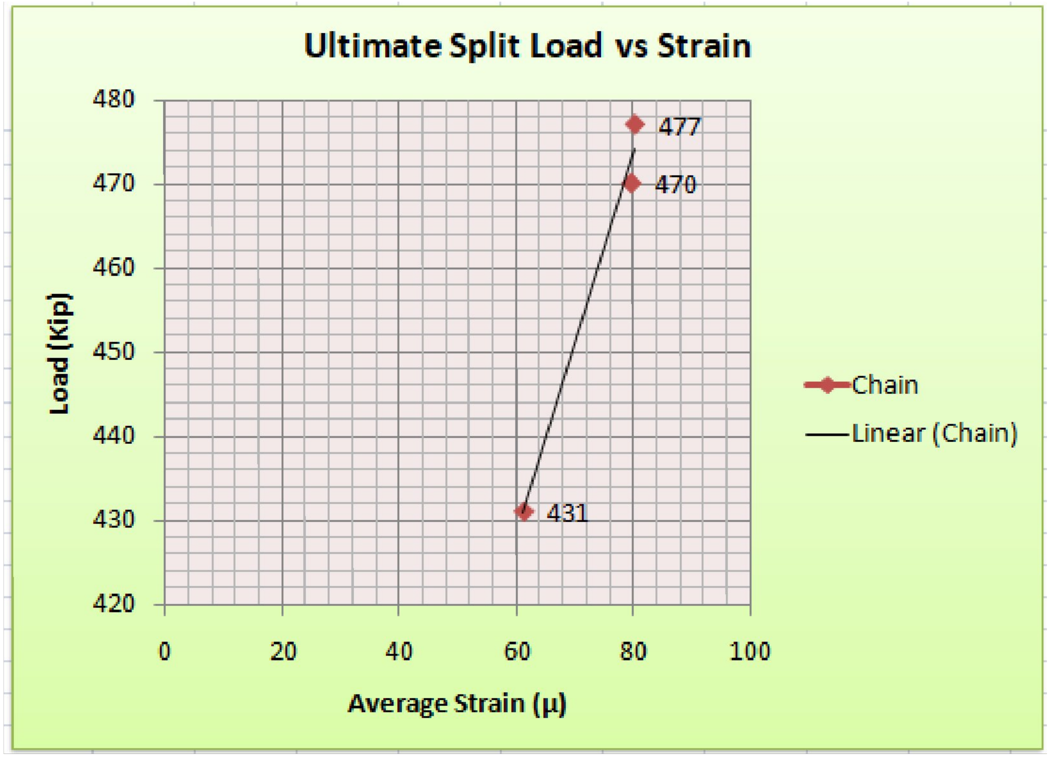



Figure 4.3 Average Strains vs. Load

### 4.3 Results of Modulus of Rupture

#### 4.3.1 Modulus of Rupture Test Data

The flexural strength ($f_r$) was determined by using the following equation:

$$f_r = \frac{P*L}{B*D^2} \quad \text{(psi)}$$

Where: (P) is represent the Ultimate applied load in (Ibs) along the width face 6’’

(L) is represent the length of the beam specimen, clear span from c/c of support (18”).

(D) is represent the depth of the beam (6”).

(B) is represent the width of the beam (6”). Table (4.3) illustrates the flexural strength test results.

Table (4.3) Flexural Strength Test data

| SF Content in The Concrete Mix | Specimen Number | Ultimate Rapture Load (lb) | Maximum Strain at failure Load (μ) | Average Strain (μ) | Maximum deflection (in) | Average Ultimate Displacement (in) | Flexure Strength $f_r = P\,L / (B\,D^2)$ (psi) | Mean Strength $\ddot{X}$ (psi) | Standard Deviation $S_x = (\sum_{i=1}^{N}(X-\ddot{X})^2/(N-1))^{0.5}$ | Coefficient of Variation C.V= Standard Deviation / Mean | Average Strength (psi) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0% | 1 | 5416 | 41.9 | 138.853 | 0.0476 | 0.0365 | 225 | 196 | 26.21 | 13.372 | 200 |
| | 2 | 4542 | 328 | | 0.0316 | | 189 | | | | |
| | 3 | 4193 | 46.66 | | 0.0304 | | 174 | | | | |
| 0.5% | 1 | 5591 | 169.52 | 351.707 | 0.0424 | 0.0364 | 232 | 217 | 19.08 | 8.793 | 219 |
| | 2 | 4717 | 6.6 | | 0.0328 | | 196 | | | | |
| | 3 | 5416 | 879 | | 0.034 | | 225 | | | | |
| 1.0% | 1 | 6289 | 1157 | 583.127 | 0.0448 | 0.043 | 262 | 276 | 12.12 | 4.391 | 277 |
| | 2 | 6814 | 580 | | 0.0416 | | 283 | | | | |
| | 3 | 6814 | 12.38 | | 0.0428 | | 283 | | | | |

### 4.3.2 Modulus of Rupture Test Results

In this table, the results of the average flexural strength is clearly show that the flexural strength increases proportionally with the increase of the steel fibers. The increasing of steel fibers for (0.5%, and 1.0%) lead to increase the flexural strength by (9.5%, and 38.5%) respectively with respect to 0%. It is clear that the behavior of (0.5% and 1.0%) isn’t close to each other regarding the flexural strength. Overall, with (0.5% and 1.0%) will give flexural strength about (1/10 and 4/10 times) of Strength of plain concrete, respectively, and this will give greater strength capacity for the concrete under flexural loading. Figure (4.4) shows the average flexural strength for different SFRC and Figure (4.5) shows the average flexural strength beams verse displacement.

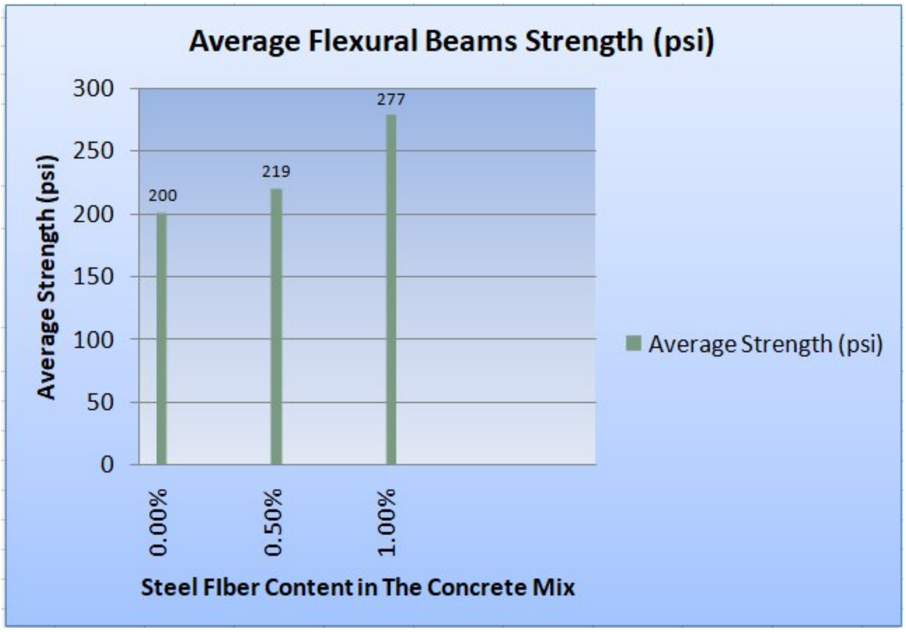



Figure 4.4 Averages Flexural Strength vs. SFRC

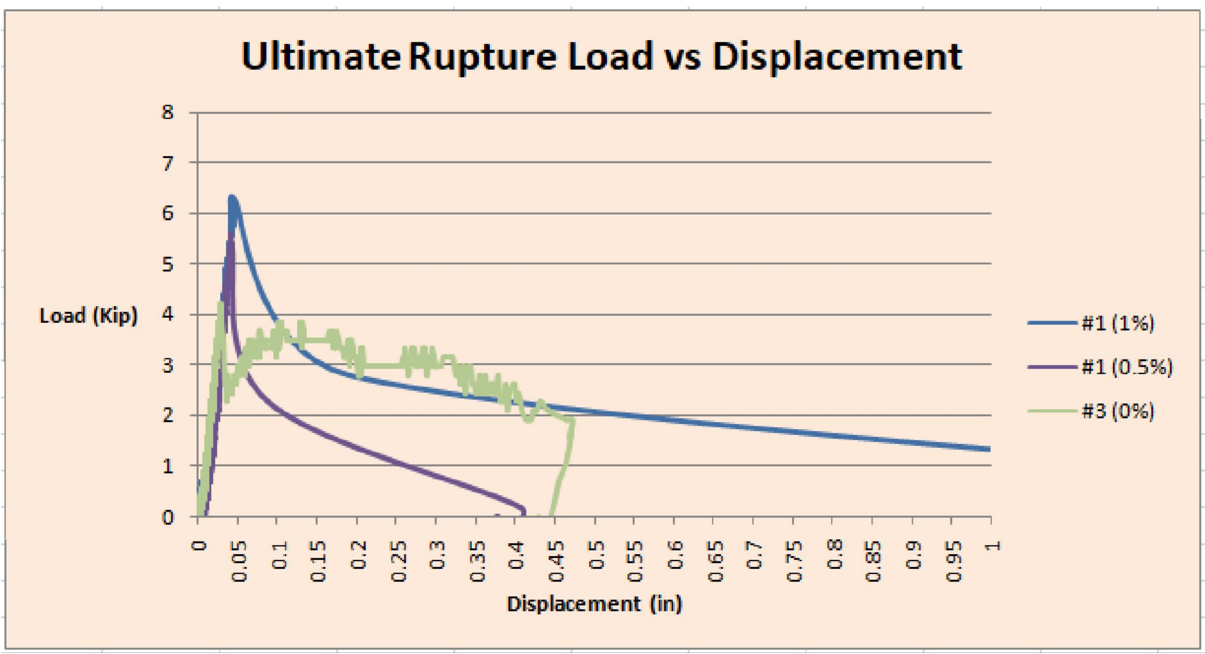



Figure 4.5 Averages Flexural Strength vs. Displacement

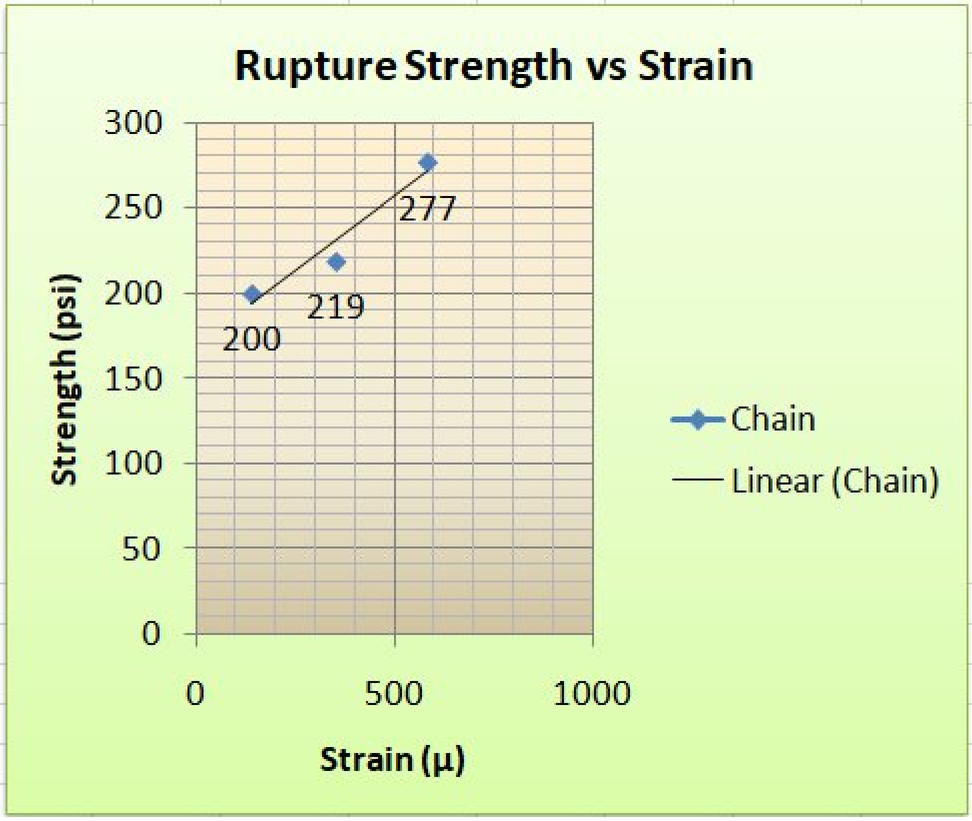



Figure 4.6 Averages Flexural Strength vs. Strain

The next table will show the summary results of the specimens' strength test. The ratio between modulus of rupture and the split tensile strength is almost the same for all mixtures and is ranging between (0.46-0.58). Briefly, the plain concrete has the highest ratio comparing with others and the (0.5% SFRC) and (1.0% SFRC) which are close in the ratios. Table (4.4) illustrates the summary of all strength results and Figure (4.7) shows the summary of the average strength for different SFRC.

Table (4.4) Summary of the Strength Results for Different SFRC

| Type of Test | Plain Concrete (0.0% SFRC) | (0.5% SFRC) | (1.0% SFRC) |
|---|---|---|---|
| Average Compressive Strength (psi) | 3875 | 3993 | 4248 |
| Increasing % | N/A | 3.04% | 9.62% |
| Average Tensile Strength (psi) | 431 | 470 | 477 |
| Increasing % | - | 9.05% | 10.67% |
| Average Flexural Strength (psi) | 200 | 219 | 277 |
| Increasing % | N/A | 9.5% | 38.5% |
| Absolute ratio for the Modulus of Rupture/Split Tensile Strength | 0.46 | 0.47 | 0.58 |

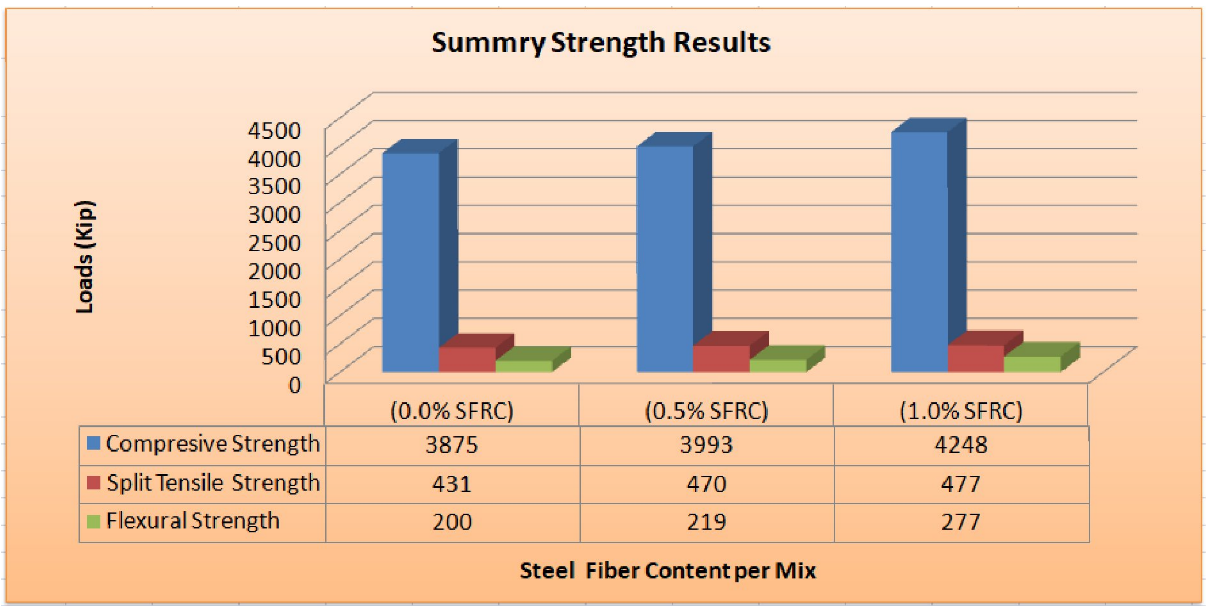



Figure 4.7 Summary of Average Strength vs. SFRC

### 4.4 Concrete Breakout and Anchor Group Test Results

#### 4.4.1 Concrete Breakout Strength in Tension Data (Group)

After the anchor group sets had been successfully tested. The ultimate applied tensile load was recorded and the breakout / cracked area around the anchor were registered. The displacement and strain of each anchor was recorder as well. Table (4.5) illustrates the concrete breakout strength in tension tests for different SFRC.

Table (4.5) Concrete Breakout Strength Test Results for Different SFRC of Anchor Group

| SF Content in Concrete | Edge Distance (in) | Sample Number | Failure Load (Kip) | Average Load (Kip) | Strain (μ) | Average Strain (μ) | Displacement (in) | Average Displacement (in) |
|---|---|---|---|---|---|---|---|---|
| 0 % Fiber-Plain | 0.5 hef | #1 | 3.754 | 4.115 | 348.373 | 411.679 | 0.016 | 0.0210 |
| 0 % Fiber-Plain | 0.5 hef | #2 | 4.559 | | 487.618 | | 0.026 | |
| 0 % Fiber-Plain | 0.5 hef | #3 | 4.033 | | 399.047 | | 0.021 | |
| 0 % Fiber-Plain | 1.0 hef | #1 | 10.521 | 9.515 | 845.739 | 691.563 | 0.032 | 0.0293 |
| 0 % Fiber-Plain | 1.0 hef | #2 | 10.483 | | 708.844 | | 0.031 | |
| 0 % Fiber-Plain | 1.0 hef | #3 | 7.540 | | 520.105 | | 0.025 | |
| 0 % Fiber-Plain | 1.5 hef | #1 | 13.151 | 13.093 | 1522.090 | 1525.775 | 0.034 | 0.0343 |
| 0 % Fiber-Plain | 1.5 hef | #2 | 13.677 | | 1643.808 | | 0.04 | |
| 0 % Fiber-Plain | 1.5 hef | #3 | 12.450 | | 1411.427 | | 0.029 | |
| 0.5 % Fiber | 0.5 hef | #1 | 4.892 | 4.705 | 480.003 | 418.204 | 0.027 | 0.0263 |
| 0.5 % Fiber | 0.5 hef | #2 | 4.952 | | 544.592 | | 0.03 | |
| 0.5 % Fiber | 0.5 hef | #3 | 4.270 | | 230.018 | | 0.022 | |

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| 0.5 % Fiber | 1.0 hef | #1 | 8.037 | 9.842 | 650.043 | 825.489 | 0.028 | 0.0317 |
| 0.5 % Fiber | 1.0 hef | #2 | 11.182 | | 1083.011 | | 0.035 | |
| 0.5 % Fiber | 1.0 hef | #3 | 10.308 | | 743.412 | | 0.032 | |
| 0.5 % Fiber | 1.5 hef | #1 | 14.851 | 14.168 | 1936.189 | 1554.549 | 0.046 | 0.0420 |
| 0.5 % Fiber | 1.5 hef | #2 | 14.640 | | 1500.790 | | 0.043 | |
| 0.5 % Fiber | 1.5 hef | #3 | 13.014 | | 1226.667 | | 0.037 | |
| 1% Fiber | 0.5 hef | #1 | 6.313 | 5.962 | 560.419 | 601.057 | 0.023 | 0.0313 |
| 1% Fiber | 0.5 hef | #2 | 7.014 | | 646.497 | | 0.05 | |
| 1% Fiber | 0.5 hef | #3 | 4.559 | | 596.256 | | 0.021 | |
| 1% Fiber | 1.0 hef | #1 | 13.327 | 12.684 | 1296.722 | 1142.180 | 0.04 | 0.0340 |
| 1% Fiber | 1.0 hef | #2 | 12.801 | | 1160.181 | | 0.033 | |
| 1% Fiber | 1.0 hef | #3 | 11.924 | | 969.638 | | 0.029 | |
| 1% Fiber | 1.5 hef | #1 | 14.203 | 14.963 | 1502.361 | 1705.737 | 0.039 | 0.0483 |
| 1% Fiber | 1.5 hef | #2 | 15.256 | | 1706.948 | | 0.049 | |
| 1% Fiber | 1.5 hef | #3 | 15.431 | | 1907.902 | | 0.057 | |

### 4.4.2 Pure Ultimate Tensile Load of Anchor Groups (Group)

Many important things can be found in this table and need to be studied carefully. The average concrete breakout strength of the anchor group's values is clearly increases by increasing the fraction of the steel fibers in concrete as well as by increasing the edge distance of anchor bolts.

The increasing in strength regarding steel fiber from (0.0%-0.5%) is (14.33%, 3.43%, and 8.21 %) corresponding to the change in edge distance (0.5 hef, 1 hef, and 1.5 hef) respectively, and the increasing from (0.0%-1.0% SFRC) is (44.88%, 33.3%, and 14.28%) corresponding to the change in edge distance (0.5 hef, 1 hef, and 1.5 hef) respectively. Overall, the increase in concrete breakout strength for (0.5% and 1.0% SFRC) is trend to increase with a great difference of the breakout strength. Figure (4.8) shows the average concrete breakout strength for different SFRC.

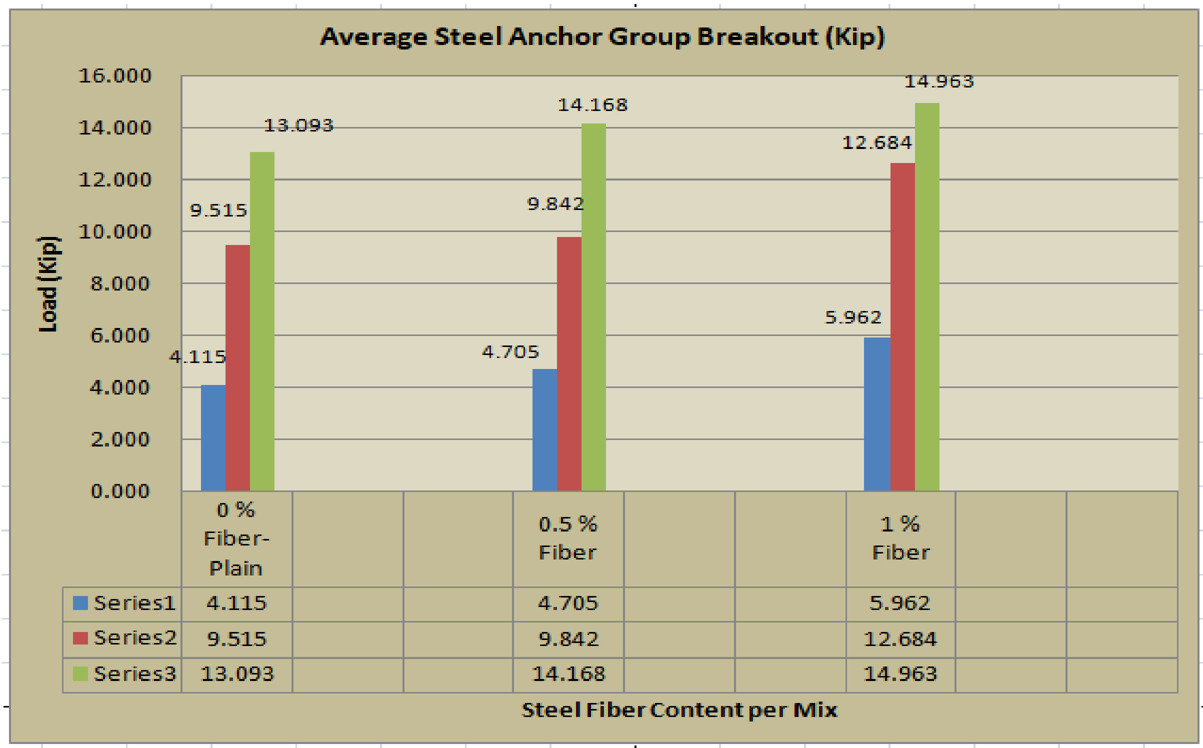
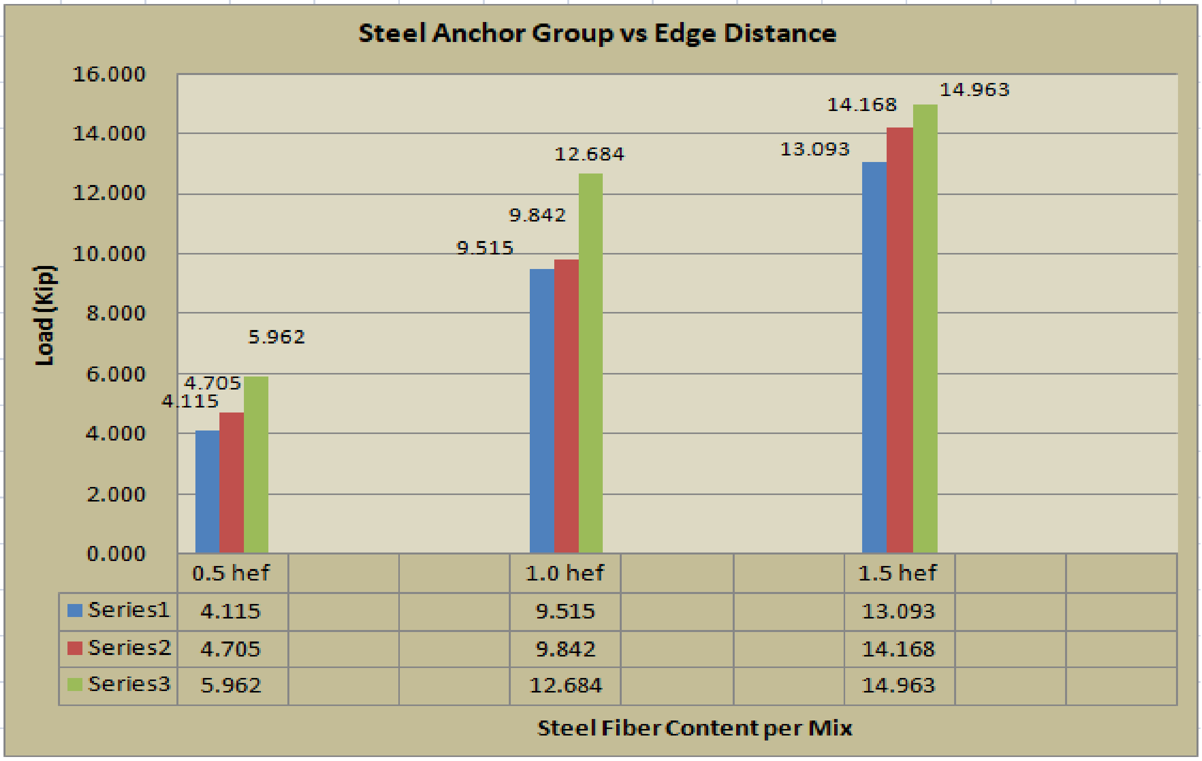


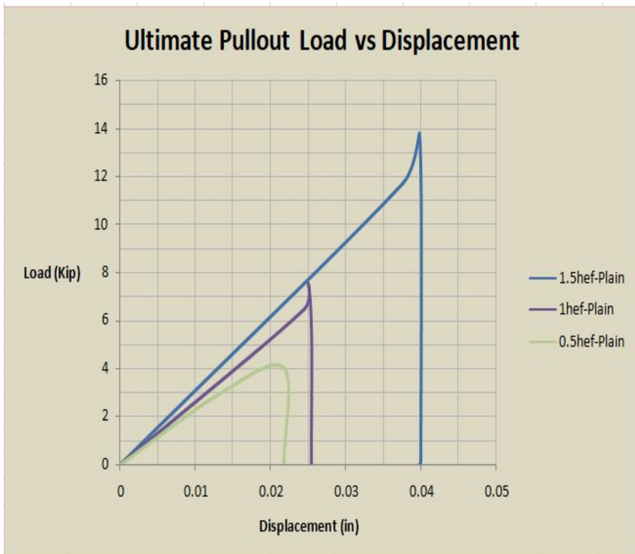

Figure 4.8 Average Concrete Breakout Strength vs. SFRC



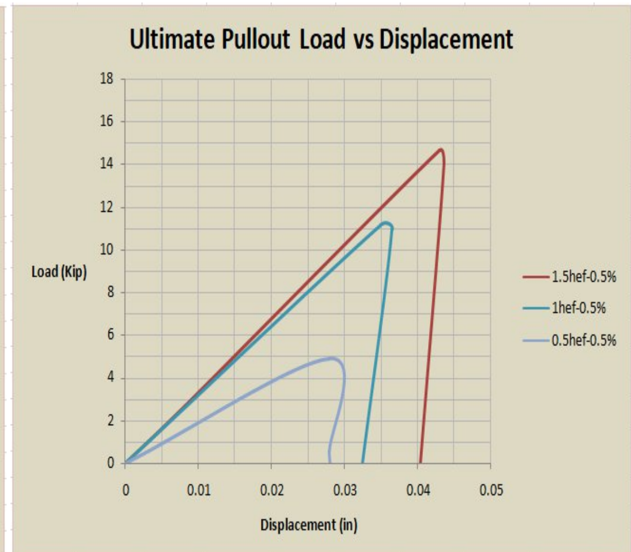

| | 0.5 hef | | | 1.0 hef | | | 1.5 hef | | |
|---|---|---|---|---|---|---|---|---|---|
| Series1 | 4.115 | | | 9.515 | | | 13.093 | | |
| Series2 | 4.705 | | | 9.842 | | | 14.168 | | |
| Series3 | 5.962 | | | 12.684 | | | 14.963 | | |

Figure 4.9 Average Concrete Breakout Strength vs. Edge Distance

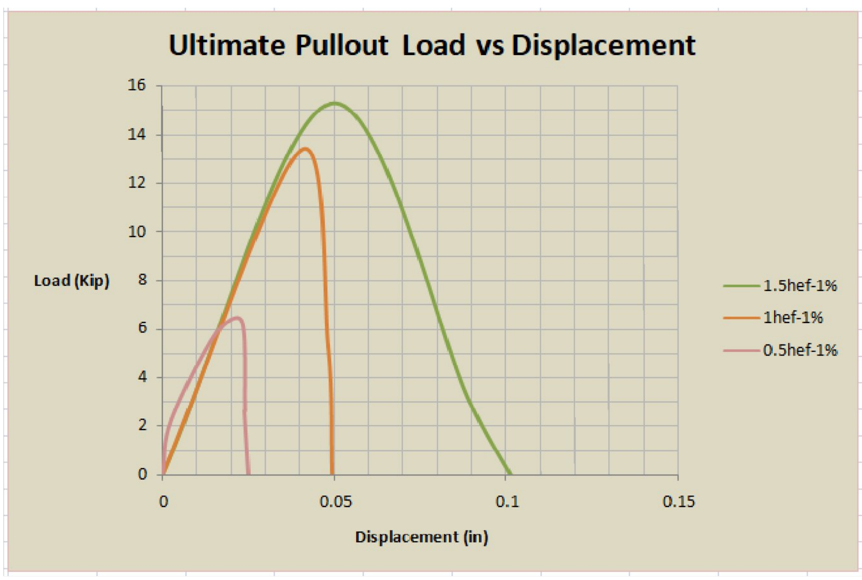
On the other side, the average concrete breakout strength of the anchor group's values is clearly increases by increasing strain and displacement as well as by increasing the edge distance of anchor bolts. The frequency was used in the DAQ for recording is (100 HZ). The increasing in steel fiber from (0.0%-0.5%) is causing increase in strain around (+1.58%, +19.37%, and +1.89%) and displacement around (+25. 39%, +7.95%, and +22.33%) corresponding to the change in edge distance (0.5 hef, 1 hef, and 1.5 hef) respectively. The increasing in steel fiber from (0.0%-1.0%) is causing increase in strain around (+46.0%, +65.16%, and +11.79%) and displacement around (+49.21%, +15.91%, and 40.77%) corresponding to the change in edge distance (0.5 hef, 1 hef, and 1.5 hef) respectively. Figure (4.10) shows the average concrete breakout strength for different SFRC against displacement and figure (4.11) shows the average concrete breakout strength for different SFRC against strain. In case of anchor group, the common failure for 0.5 times hef was breakout total concrete area way below the bottom anchor in plain concrete and it's tried to change to be pry out when steel fiber was added. For the 1 and 1.5 times hef, the failure was pry out and it's tried to be splitting when steel fiber was added.




Plain

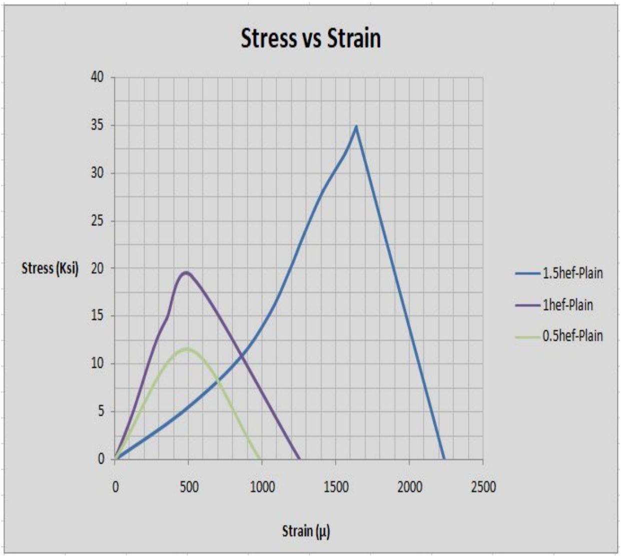



0.5%

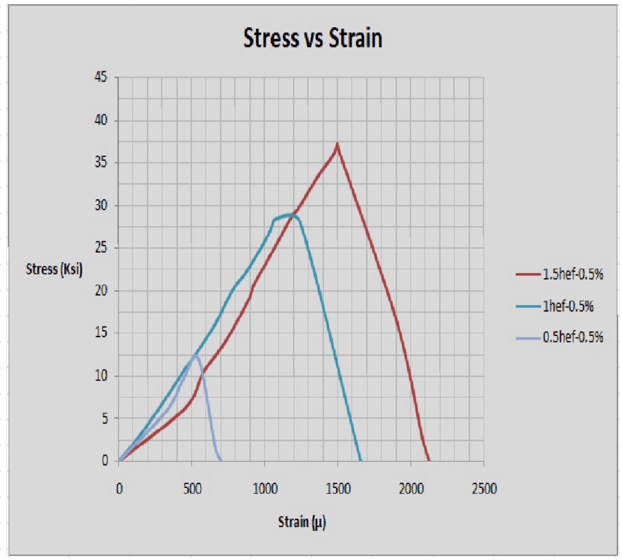



1.0%

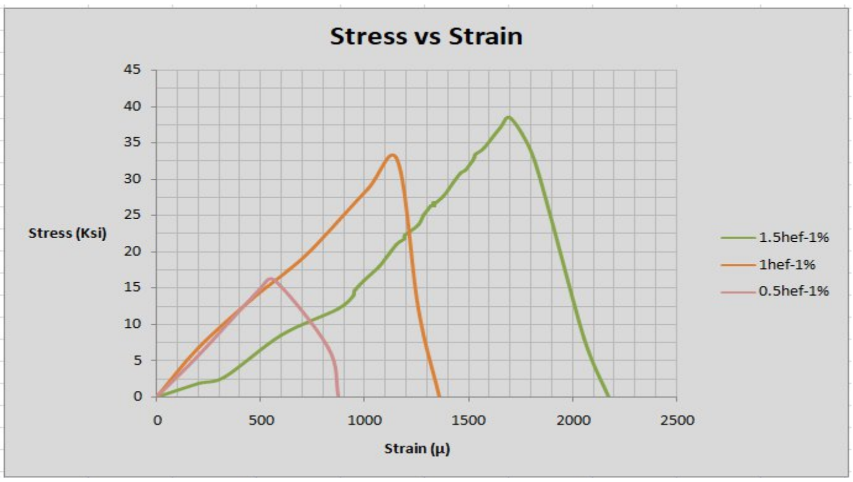
Figure 4.10 Average Concrete Breakout Strength of Anchor Groups vs. Displacement




Plain




0.5%




1.0%

Figure 4.11 Average Concrete Breakout Strength of Anchor Groups vs. Strain

## 4.5 Concrete Breakout and individual Anchor Test Results

### 4.5.1 Concrete Breakout Strength in Tension Data (Single)

After the individual anchor sets had been successfully tested. The ultimate applied tensile load was recorded and the breakout / cracked area around the anchor were registered. The displacement and strain of each anchor was recorder as well. Table (4.6) illustrates the concrete breakout strength in tension tests for different SFRC.

Table (4.6) Concrete Breakout Strength Test Results for Different SFRC of Single Anchor

| SF Content in Concrete Mix | Edge Distance (in) | Sample Number | Failure Load (Kip) | Average Load (Kip) | Strain (μ) | Average Strain | Displacement (in) | Average Displacement (in) |
|---|---|---|---|---|---|---|---|---|
| 0 % Fiber-Plain | 0.5 hef | #1 | 3.507 | 3.228 | N/A | N/A | 0.015 | 0.012 |
| 0 % Fiber-Plain | 0.5 hef | #2 | 3.156 | | N/A | | 0.012 | |
| 0 % Fiber-Plain | 0.5 hef | #3 | 3.020 | | N/A | | 0.010 | |
| 0 % Fiber-Plain | 1.0 hef | #1 | 5.085 | 5.498 | N/A | N/A | 0.017 | 0.017 |
| 0 % Fiber-Plain | 1.0 hef | #2 | 4.910 | | N/A | | 0.015 | |
| 0 % Fiber-Plain | 1.0 hef | #3 | 6.500 | | N/A | | 0.019 | |
| 0 % Fiber-Plain | 1.5 hef | #1 | 7.365 | 7.338 | N/A | N/A | 0.020 | 0.021 |
| 0 % Fiber-Plain | 1.5 hef | #2 | 5.962 | | N/A | | 0.017 | |
| 0 % Fiber-Plain | 1.5 hef | #3 | 8.688 | | N/A | | 0.027 | |
| 0.5 % Fiber | 0.5 hef | #1 | 3.620 | 3.290 | N/A | N/A | 0.022 | 0.017 |
| 0.5 % Fiber | 0.5 hef | #2 | 2.970 | | N/A | | 0.013 | |
| 0.5 % Fiber | 0.5 hef | #3 | 3.280 | | N/A | | 0.017 | |
| 0.5 % Fiber | 1.0 hef | #1 | 5.591 | 5.560 | N/A | N/A | 0.026 | 0.027 |
| 0.5 % Fiber | 1.0 hef | #2 | 6.289 | | N/A | | 0.029 | |
| 0.5 % Fiber | 1.0 hef | #3 | 4.800 | | N/A | | 0.025 | |
| 0.5 % Fiber | 1.5 hef | #1 | 7.540 | 7.419 | N/A | N/A | 0.034 | 0.031 |
| 0.5 % Fiber | 1.5 hef | #2 | 7.365 | | N/A | | 0.032 | |
| 0.5 % Fiber | 1.5 hef | #3 | 7.353 | | N/A | | 0.026 | |

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| 1% Fiber | 0.5 hef | #1 | 4.368 | | N/A | | 0.025 | |
| 1% Fiber | 0.5 hef | #2 | 2.440 | 3.446 | N/A | N/A | 0.013 | 0.019 |
| 1% Fiber | 0.5 hef | #3 | 3.530 | | N/A | | 0.020 | |
| 1% Fiber | 1.0 hef | #1 | 5.611 | | N/A | | 0.024 | |
| 1% Fiber | 1.0 hef | #2 | 5.962 | 6.411 | N/A | N/A | 0.029 | 0.030 |
| 1% Fiber | 1.0 hef | #3 | 7.660 | | N/A | | 0.037 | |
| 1% Fiber | 1.5 hef | #1 | 7.163 | | N/A | | 0.028 | |
| 1% Fiber | 1.5 hef | #2 | 7.163 | 7.441 | N/A | N/A | 0.038 | 0.036 |
| 1% Fiber | 1.5 hef | #3 | 7.997 | | N/A | | 0.042 | |

*N/A means not measured or placed strain gauge because of individual anchor.

#### 4.5.2 Pure Ultimate Tensile Load of Individual Anchor (Single)

Many serious things can be found in this table and need to be studied carefully. The average concrete breakout strength of the individual anchor values is clearly increases slightly by increasing the fraction of the steel fibers in concrete as well as by increasing the edge distance of anchor bolts. The increasing in strength regarding steel fiber from (0.0%-0.5%) is around (1.92%, 1.12 %, and 1.1%) corresponding to the change in edge distance (0.5 hef, 1 hef, and 1.5 hef) respectively. and the increasing from (0.0%-1.0% SFRC) is around (6.76%, 16.6%, and 1.4%) corresponding to the change in edge distance (0.5 hef, 1 hef, and 1.5 hwf) respectively. Overall, the increase in concrete breakout strength for (0.5% and 1.0% SFRC) is trend to increase with a noticeable difference in concrete breakout strength. Figure (4.12) shows the average concrete breakout strength for different SFRC.

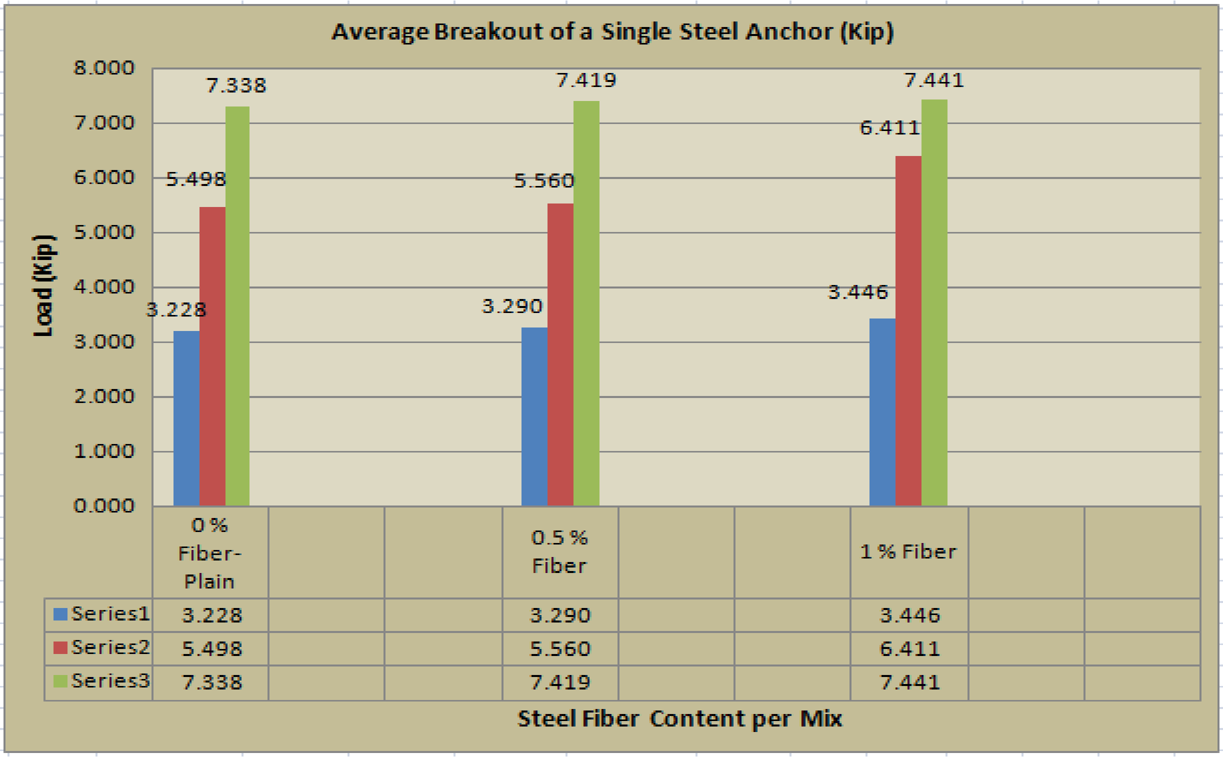



Figure 4.12 Average Concrete Breakout Strength vs. SFRC

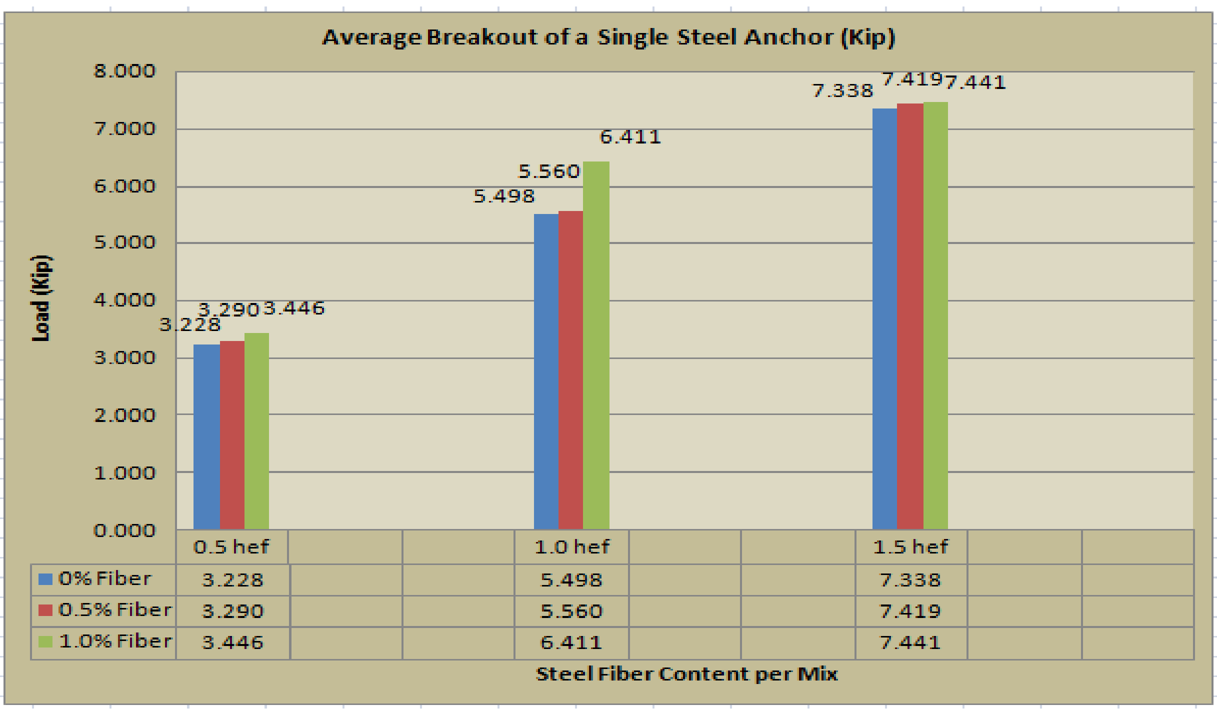



Figure 4.13 Average Concrete Breakout Strength vs. Edge Distance

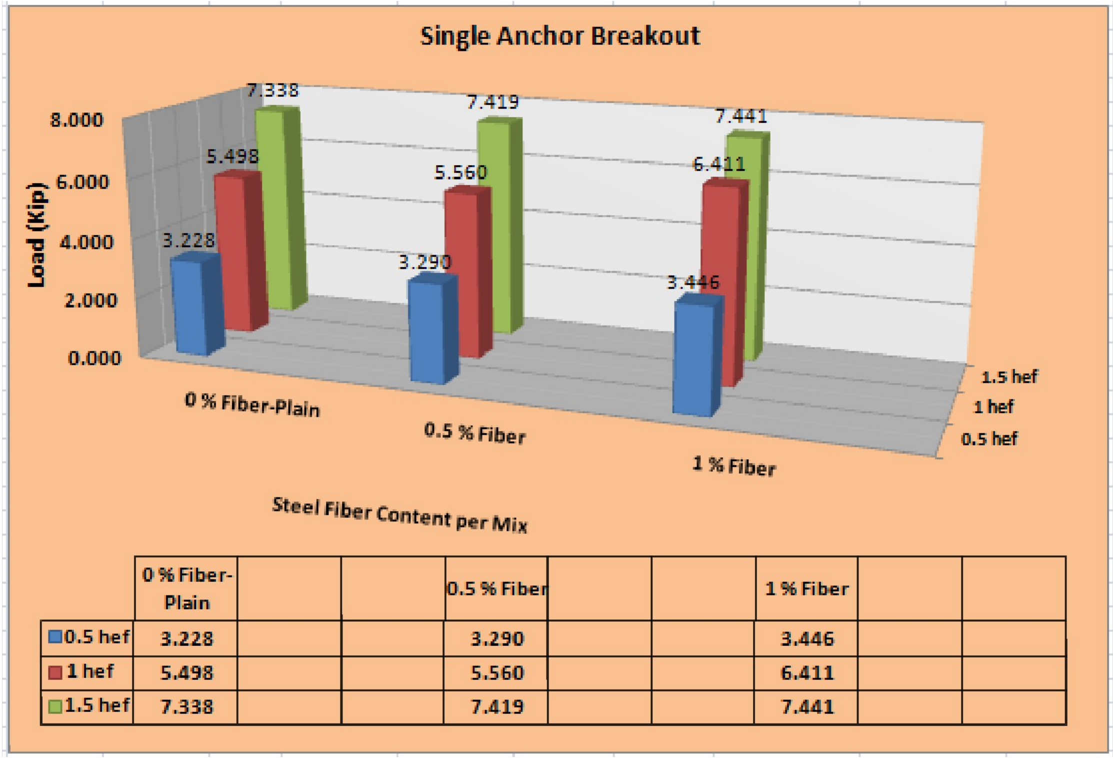



Figure 4.14 Summary of Average Concrete Breakout Strength

On the other hand, the average concrete breakout strength of the individual anchor values is clearly increases by increasing displacement as well as by increasing the edge distance of anchor bolts. The increasing in steel fiber from (0.0%-0.5%) is causing increase in displacement around (+39. 73%, +56.86%, and +44.38%) corresponding to the change in edge distance (0.5 hef, 1 hef, and 1.5 hef) respectively. The increasing in steel fiber from (0.0%-1.0%) is causing increase in displacement around (+56.76%, +76.47%, and 68.75%) corresponding to the change in edge distance (0.5 hef, 1 hef, and 1.5 hef) respectively. The strain for individual anchor sets didn't measure like anchor groups. Figure (4.15) shows the average concrete breakout strength for (1% SFRC) against displacement. In case of single anchor, the dominate failure of 0.5, 1, and 1.5 times hef was pry out and it's maintain to stay as pry out when steel fiber added but with increasing of the crack slop.

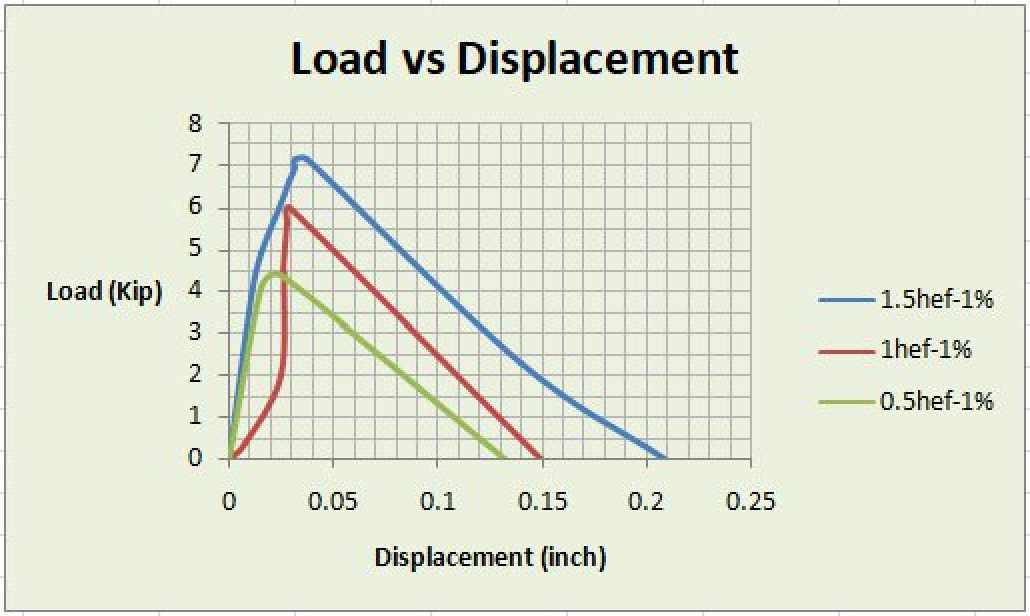



Figure 4.15 Average Concrete Breakout vs Single Anchor Displacement

### 4.6 Anchor Group vs. Single Anchor in Tension Test

After all work done previously, now, it needs to make comparison between the results of experimental investigations on concrete breakout strength of cast-in-place steel anchor groups within Steel Fiber Reinforced Concrete (SFRC) under pure tension load with the results of the breakout strength for single anchors under the same conditions and specifications. The concrete break strength of anchor groups increased by increasing the steel fibers to certain limit. In general, the strength for group anchor for different steel fiber content or different edge distance will be less than the strength of double single anchor. The differences in the concrete breakout strength of anchor groups compared with two single anchors will be named as group effect and it shows the strength increased by increasing of steel fibers in concrete, to be more detail, adding this type of steel fiber make strength fluctuate and doesn't show any decent pattern. The ratios for (0.0%, 0.5%, and 1.0%) is (+63.74%, +86.5%, +89.21%, +72.5%, +88.5%, +95.48%, +86.5%, +98.92%, and +100.0%) respectively and corresponding to (0.5 hef, 1 hef, and 1.5 hef). This turbulence can be attributed to the interaction of the stresses from the grouping action not like the single anchor that works by itself without any other stresses. Overall, good results of strength can be obtained by increasing of steel fibers in concrete as illustrated in Table (4.7).

Table (4.7) Average Concrete Breakout Strength of Anchor Groups vs. 2 Single Anchors

| Edge Distance | Concrete Mix | Average Concrete Breakout Strength (Kip) of Anchor Group with Group Effect | Average Concrete Breakout Strength of Single Anchors (Kips) without Grouping effect | Double Single | Group Effect Factor |
|---|---|---|---|---|---|
| 0.5hef | 0.00% | 4.115 | 3.228 | 6.456 | 0.637 |
| 1.0hef | 0.00% | 9.515 | 5.5 | 11 | 0.865 |
| 1.5hef | 0.00% | 13.093 | 7.338 | 14.676 | 0.892 |
| 0.5hef | 0.50% | 4.705 | 3.29 | 6.58 | 0.715 |
| 1.0hef | 0.50% | 9.842 | 5.56 | 11.12 | 0.885 |
| 1.5hef | 0.50% | 14.168 | 7.419 | 14.838 | 0.955 |
| 0.5hef | 1.00% | 5.962 | 3.446 | 6.892 | 0.865 |
| 1.0hef | 1.00% | 12.684 | 6.411 | 12.822 | 0.989 |
| 1.5hef | 1.00% | 14.963 | 7.441 | 14.882 | 1.000 |

The study was made to prove the beneficial effects of adding steel fiber as reinforcement in concrete structures. This addition of steel fibers to the concrete mix can lead to better mechanical and physical concrete proprieties, including higher fracture energy. Also, this positive can change the shape of the crack or failure and thus leads to an increase in the concrete breakout strength of anchors in this study. Finally, this table explains clearly that using twice strength of single anchor not as same as use group anchor in behavior and values of concrete breakout strength which is always less than double single anchor, but there is a development effect by increasing of the steel fibers. Comprehensively, this study represents another prove that the concrete breakout strength of anchor groups will not be twice of concrete breakout strength of double single anchors. Figure

(4.16) and (4.17) shows the results of the concrete breakout strength of anchor groups and two single anchors with different SFRC. Also, figure (4.18) shows the group effect ratio towards steel fiber content for different cross section area.

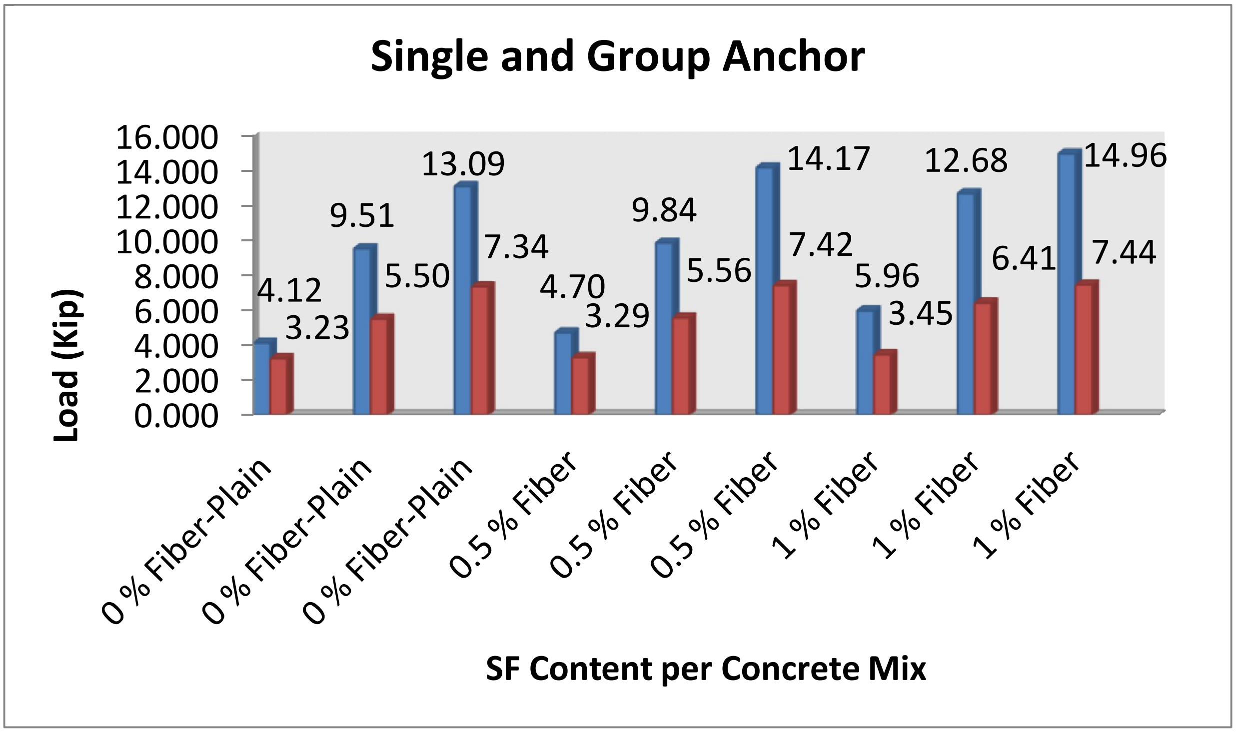
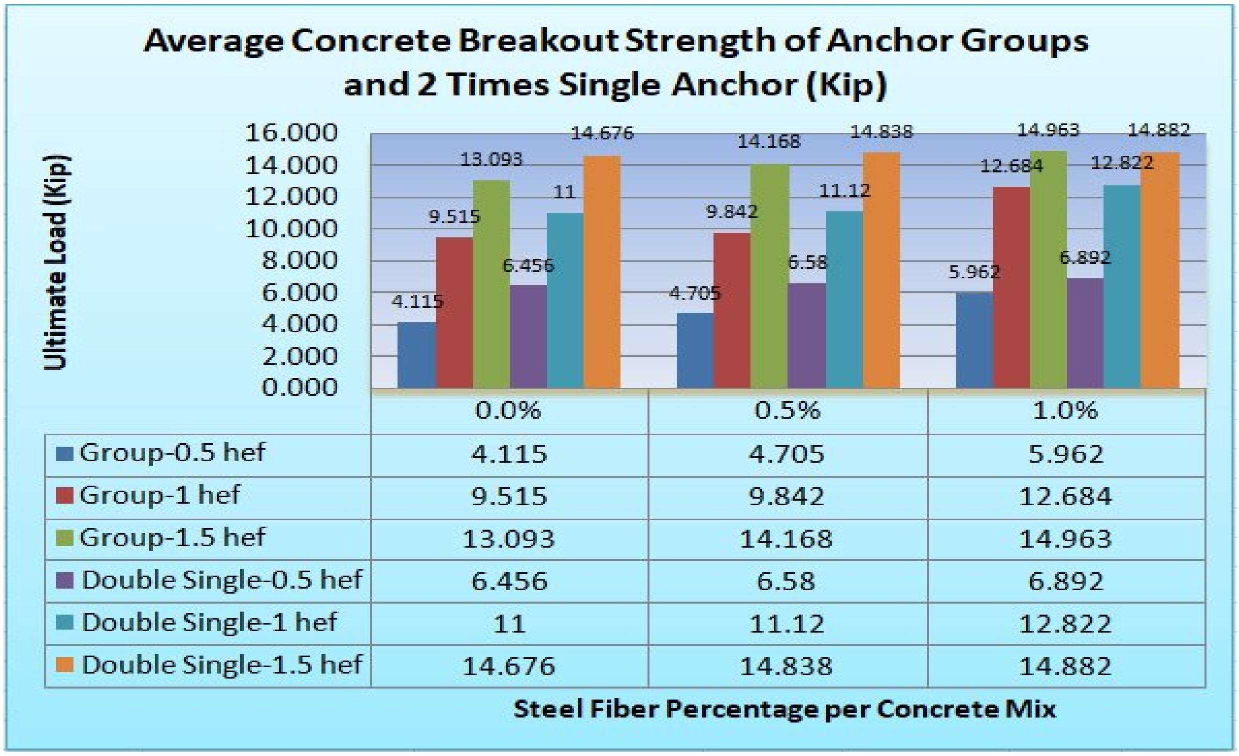



Figure 4.16 Concrete Breakout Strength of Anchor Groups and Single Anchors vs. SFRC




| | 0.0% | 0.5% | 1.0% |
|---|---|---|---|
| Group-0.5 hef | 4.115 | 4.705 | 5.962 |
| Group-1 hef | 9.515 | 9.842 | 12.684 |
| Group-1.5 hef | 13.093 | 14.168 | 14.963 |
| Double Single-0.5 hef | 6.456 | 6.58 | 6.892 |
| Double Single-1 hef | 11 | 11.12 | 12.822 |
| Double Single-1.5 hef | 14.676 | 14.838 | 14.882 |

Figure 4.17 Concrete Breakout Strength of Anchor Groups and 2 Single Anchors vs. SFRC

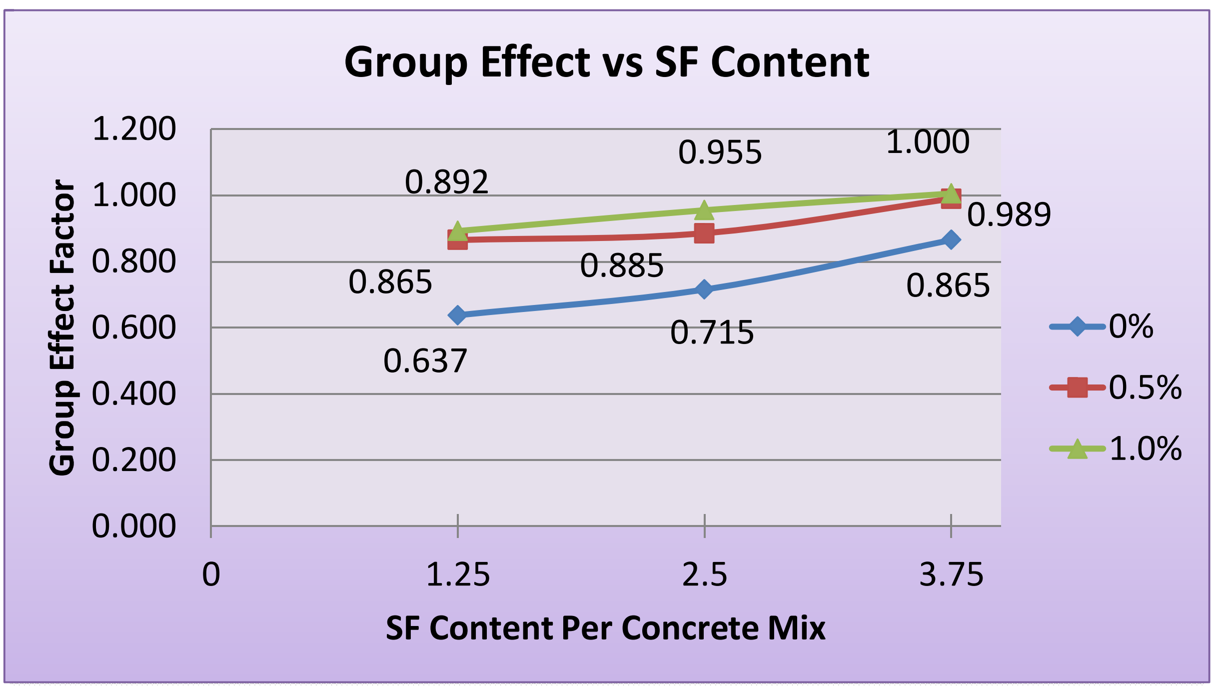



Figure 4.18 Group effect ratio for different edge distance vs. SFRC

## 4.7 Experiment Summary

### 4.7.1 Experiment Data and Results

From the previous lab work, all tests were conducted and discussed during this research. It was evident that the use of the steel fibers had a good effect on the performance of the plain concrete on all mechanical aspects of the concrete split tensile strength and on the concrete breakout strength of anchor groups or single anchor, and it has positive effect on the compressive strength and flexural strength. Broadly, the increasing of the steel fibers from (0.0%-0.5%) showed better performance and achievement for split tensile but the increasing of (1.0% SFRC) showed slight differences and for the rest of tests showed the positive impact as in the compressive and flexural strength where both strengths increased for all the ways forward by adding steel fiber and its clearly observed for the first time steel was added. Adding steel fiber tend to make concrete material to behave as more ductile and this is clearly shown under the split tensile by giving concrete more capacity under this load. The results show that the optimum value of this type of steel fibers is suggested to be (0.5%) as it affects mechanical performance greatly. On the other hand, it should be noted that the increase in the amount of steel fibers decreases the consistency and workability of the concrete which can be undesirable in construction. Therefore, the using of additives to enhance workability would be essential and need to be study as well. In conclusion, using (0.5% SFRC) by the weight volume fraction of the concrete is satisfactory.

## 4.8 Discussion of Results

### 4.8.1 Small Specimen Contributions

To review and deal with all the data of the test results were conducted and gained in this study; concrete in general is known as a brittle material and very weak or poor resistance in tension. Adding of the steel fibers instead of rebar to the concrete can change the tensile strength and behavior of concrete. The addition of the steel fiber introduces a great tensile strength to the concrete due to the bond between fibers and concrete. Therefore, the increase in the fiber fraction might be increased compressive, tensile, and flexural strength. The concrete mixture in this study was designed for compressive strength of (4000 psi) and adding steel fibers increased the compressive strength as expected. As well as the flexural strength increased, in the same way, the split tensile strength increased by increasing steel fiber and this is giving concrete more ductility and preventing sudden failure. Finally, it will give more factors of safety. This was also noticed in the shape of failure during the flexural strength test. Furthermore, the concrete breakout strength of anchor groups and individual increased by increasing steel fibers in concrete mix.

### 4.8.2 Anchorage Presumptions and Hypothesis

The design requirements of all standards need to be studied carefully. So, in this section is very necessary to introduced, provide and explain the design requirements for anchor groups in concrete which are used to transmit test loads using pure tension without any other type of loads like shear or / and moment. The requirements that will cover the concrete breakout failure mode and calculations of the nominal breakout strength as specified in (ACI 318-19) is as follow:

1- What makes anchor bolts work as group or individual? The clear answer for this question is anchor spacing which is defined by distance between anchors and should be less than 3 times the embedded depth of the anchor. Any spacing more than this limit will make anchor considered as single anchors. In this research is assumed to use 2.5'' embed depth, so (3x2.5”=7.5”). The 5'' spacing was used in this study which is less than

7.5''.

2- There are two limits to specify the edge distance. One is rely on the anchor diameter and the other one is depend on the embedded depth of anchor. The anchor diameter is equal to 0.5''. The minimum edge distance is (6x diameter of an anchor). So, (6x0.5" =3"). For the embedded depth, the edge distance should be more than 1.5 of embedded anchor under pure tension. So, (1.5x2.5=3.75''). This research is to study the edge distance effect on the breakout strength of group and individual anchors which is beyond this limit and not covered in the ACI standards. So the edge distances which are used 1.5, 1, and 0.5 of the embedded depth.

3- The last requirement need to be consider in this research is the effective embedment depth of anchors and shall not exceed the minimum of (2/3 x member thickness or member thickness – 4) so (2/3 x 20" member thickness= 13.33" and 20"-4" = 16"). In this study, the embedment depth of anchors is (2.5" < 13.33").

While requirements for the design of anchors was introduced previously. There are relationship between design requirements and failure of design. This mean, the individual or group anchors could be exposed for various types of concrete failure modes for anchor groups as follows:

1- Steel failure of anchor groups under pure tension.
2- Pullout failure of cast-in anchors in pure tension.
3- Concrete side-face blowout failure of headed anchors in tension.
4- Concrete breakout failure of anchor groups under pure tension.
5- Concrete splitting failure of anchor groups under pure tension.

So, the design requirements can specify the category of failure and control the concrete strength range.

The nominal concrete breakout strength in tension of a group of anchors ($\mathbf{N_{cbg}}$) shall not exceed the following as per the design specifications of (ACI 318-19).

**$N_{cbg}$** = ($A_{Nc}$/$A_{Nco}$) x $\Psi_{ec,N}$ $\Psi_{ed,N}$ $\Psi_{c,N}$ $\Psi_{cp,N}$ * $\Psi_{ga}$* $N_b$

ANco=(2x0.5hef)x(2x0.5hef)=1hef^2

ANco=(2x1.0hef)x(2x1.0hef)=4hef^2

ANco=(2x1.5hef)x(2x1.5hef)=9hef^2

Where:

**$N_{cbg}$**: The nominal concrete breakout strength of a group of anchors.

**$A_{Nc}$**: The total projected concrete failure area of group of anchors that shall be approximated based on the geometrical failure figure.

**$A_{Nco}$**: The projected concrete failure area of a single anchor with an edge distance equal to or lesser than (1.5 hef), where (hef; effective embedment depth of anchor). In case of grouping anchors, the ($A_{Nc}$ < $A_{Nco}$***n**, where **n**; No. of anchors in one group).

**$\Psi_{ec,N}$**: Modification factor for anchor groups loaded eccentrically in tension and shall not be taken greater than (1.0). In this study assumed to be (1.0), no eccentricity.

**$\Psi_{ed,N}$**: Modification factor for edge effects for anchor groups loaded in tension. For (Ca, min >1.5 hef, then **$\Psi_{ed,N}$** = 0.8, 0.9, and 1.0).

**$\Psi_{c,N}$**: Modification factor for no cracking at service load levels and in case of cracking at service load levels, **$\Psi_{c,N}$** shall be taken as (1.0). In this case (**$\Psi_{c,N}$ =1.25**) because (ft<fr) indicates no cracking at service load levels.

**$\Psi_{cp,N}$**: Modification factor for post-installed anchors designed for uncracked concrete. For cast-in anchors, **$\Psi_{cp,N}$** shall be taken as (1.0).

**$\Psi_{ga}$**: Modification factor for anchor groups in steel fiber reinforced concrete. ($\Psi_{ga}$ = 0.63, 0.86, 0.89) for (0.5 hef, 1 hef, and 1.5 hef) of edge distance.

**$N_b$:** Basic concrete breakout strength in tension of a single anchor in cracked concrete (lb), shall not exceed the following as specified in (ACI 318-19, Ch.17).

**$N_b$** = $K_c$*$\lambda_a$* $\sqrt{fc`}$ * $(hef)^{1.5}$

Where:

**$K_c$** =24 for cast-in anchors and 17 for post-installed anchors. In this study (**$K_c$ = 24**).

**$\lambda_a$** : Modification Factor for lightweight concrete shall be taken as (1.0λ) for cast-in anchors. The value of (λ) shall be based on the composition of the aggregate in the concrete mixture as specified in (ACI 318-19, Ch.17), (λ) for normal weight concrete is (1), (λ) for all light weight concrete is (0.75), and (λ) for sand-lightweight concrete is (0.85). in this study the value of (**$\lambda_a$ =1**).

**fc`**: The compressive strength of the concrete, based on the design mix in this study (fc`=4000 psi).

**$h_{ef}$**: The effective embedment depth of the anchor groups, in this study (**$h_{ef}$ = 2.5"**).

By using the Concrete Capacity Design Method (CCD) specified in the (ACI 318-19) with modification of the equation based on the steel fiber factor of anchor groups and edge distance obtained from this experimental then will obtained the following equation:

$$N_b = K_c * \lambda_a * \sqrt{fc`} * (hef)^{1.5} * (1+Z\ \sqrt{fc`})$$

Where:

Z: Modification factor for percentage of steel fiber in concrete as shown in table (4.8).

Table (4.8) The Steel Fiber Modification Factor

| Embedded \ SFR | 0% | 0.50% | 1.00% |
|---|---|---|---|
| 0.5hef | 0.0190 | 0.0231 | 0.0312 |
| 1.0hef | 0.0240 | 0.0244 | 0.0334 |
| 1.5hef | 0.0220 | 0.0242 | 0.0244 |

By using the following three equations to calculate any value between the tabulated values:

$Z = 77.55x^2 + 0.44x + 0.019$

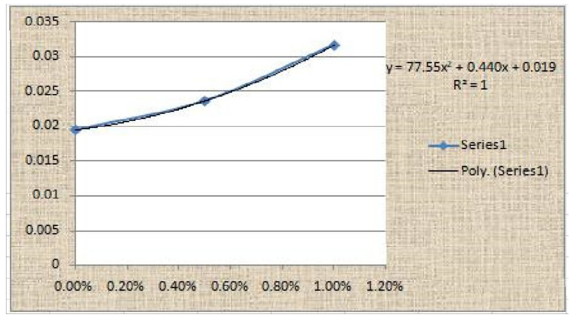



Figure 4.19 Modification Factor (Z) vs. SFRC (%) at (0.5hef)

The above figure shows the magnifier magnitude of concrete breakout with or without steel fiber when the edge distance is represent 0.5 hef.

$Z = 172.0x^2 – 0.78x + 0.024$

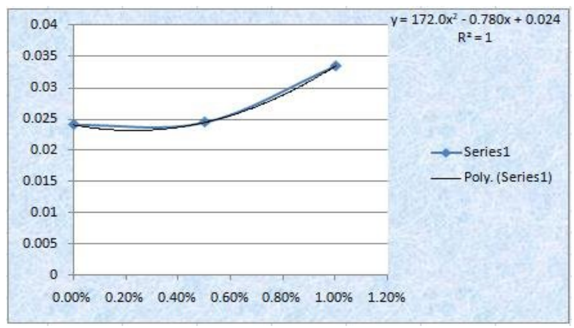



Figure 4.20 Modification Factor (Z) vs. SFRC (%) at (1.0hef)

In the other hand, this above figure shows the magnifier magnitude of concrete breakout with or without steel fiber when the edge distance is represent 1.0 hef.

$Z = -39.71x^2 + 0.633x + 0.022$

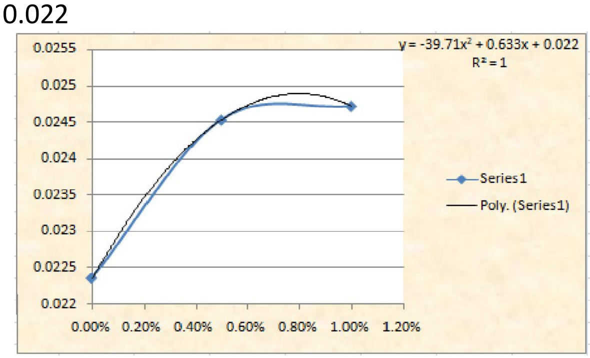



Figure 4.21 Modification Factor (Z) vs. SFRC (%) at (1.5hef)

Finally, the above figure shows the magnifier magnitude of concrete breakout with or without steel fiber when the edge distance is represent 1.5 hef.

By using the previous equations and the modification factors, the following calculations were obtained as shown in table (4.9).

Table (4.9) Concrete Breakout Strength from Experiment and Modified CCD

| Edge Distance | Concrete Mix | Z Obtained from Equations | Z Modification | Term Calculation (1+Z*fc'^0.5) | Capacity Calculated by Modified Equation Z Factor (lbs) | Contrast | Measured from Test (lbs) |
|---|---|---|---|---|---|---|---|
| 0.5 hef | 0.00% | 0.019 | 0.0190 | 2.18258784 | 4059.60 | < | 4115.00 |
| 1.0 hef | 0.00% | 0.024 | 0.0240 | 2.493795167 | 9497.77 | < | 9515.00 |
| 1.5 hef | 0.00% | 0.022 | 0.0220 | 2.369312236 | 12970.06 | < | 13093.00 |
| 0.5 hef | 0.50% | 0.02313875 | 0.0231 | 2.461775766 | 4647.52 | < | 4705.00 |
| 1.0 hef | 0.50% | 0.0244 | 0.0244 | 2.54145443 | 9824.36 | < | 9842.00 |
| 1.5 hef | 0.50% | 0.02417225 | 0.0242 | 2.527066469 | 14040.98 | < | 14168.00 |
| 0.5 hef | 1.00% | 0.031155 | 0.0312 | 3.030101493 | 5900.37 | < | 5962.00 |
| 1.0 hef | 1.00% | 0.0334 | 0.0334 | 3.176388697 | 12664.98 | < | 12684.00 |
| 1.5 hef | 1.00% | 0.024359 | 0.0244 | 2.587265038 | 14827.60 | < | 14963.00 |
| | | | | | | OK | |

The value of the modification factor (Z) is related to the percentage changes of steel fiber in concrete mix from (0.0%-1.0%) which effect on compressive strength, either increasing or decreasing, of the compressive strength of concrete.

The modified (CCD) method equation estimates the nominal concrete breakout strength in tension of anchor groups or single for different edge distance (0.5hef, 1hef, and 1.5hef) within the experimental values with slight differences from (1.02%- 4.07%) thus giving reliable results. Figure (4.22) shows the concrete breakout strength of the experiment and nominal results. The modified (CCD) method equation based on the results that obtained from this study.

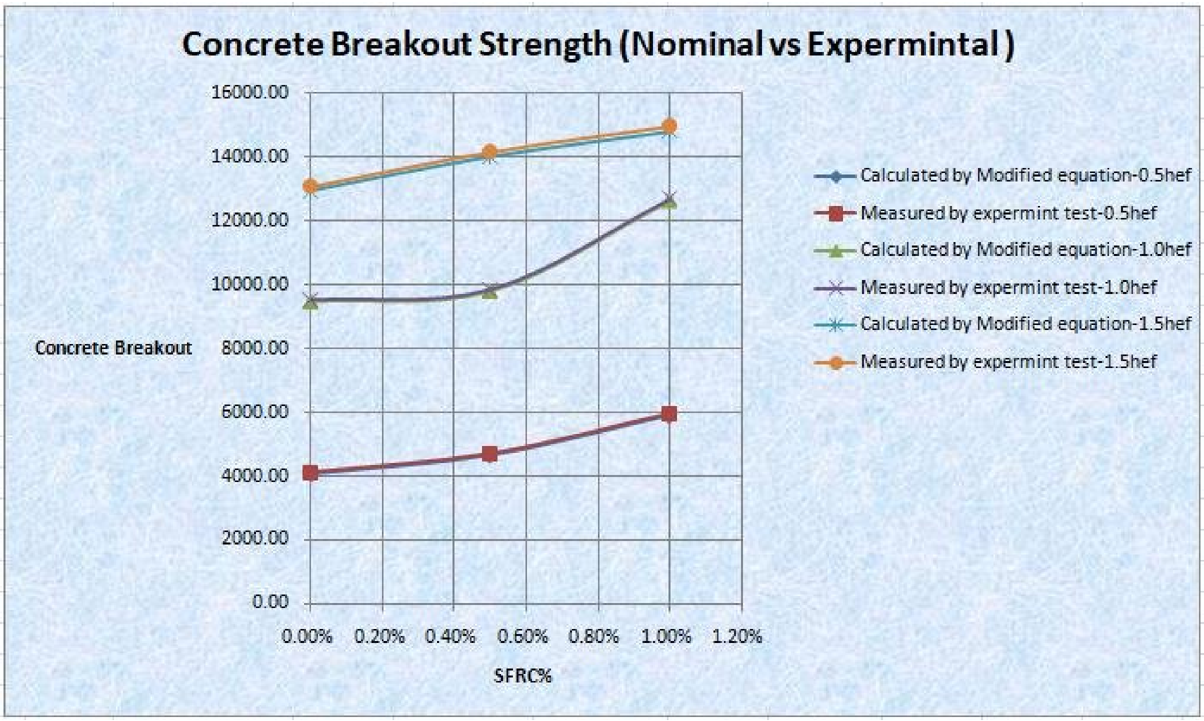



Figure 4.22 Concrete Breakout Strength (Nominal vs Experiment)

# CHAPTER FIVE

# CONCLUSIONS AND RECOMMENDATIONS

## 5.1 Conclusions

1- The increasing in steel fiber will increase breakout strength of anchor group and single. Increasing from (0.0%-0.5%) is (14.3%, 3.43%, and 8.21 %) corresponding to the change in edge distance (0.5 hef, 1 hef, and 1.5 hef) respectively. and the increasing from (0.0%-1.0% SFRC) is (44.88%, 33.3%, and 14.28%) corresponding to the change in edge distance (0.5 hef, 1 hef, and 1.5 hef) respectively. For single, The increasing in strength of single anchor regarding increase steel fiber from (0.0%-0.5%) is (1.92%, 1.12 %, and 1.1%) corresponding to the change in edge distance (0.5 hef, 1 hef, and 1.5 hef) respectively. and the increasing from (0.0%-1.0% SFRC) is (6.76%, 16.6%, and 1.4%) corresponding to the change in edge distance (0.5 hef, 1 hef, and 1.5 hef) respectively.

2- Increase edge distance for plain concrete from (0.5hef - 1hef) and from (0.5hef – 1.5hef) will increase the concrete breakout by (35.7%, and 39.9%) respectively. Increase edge distance for concrete that has 0.5% of SFRC from (0.5hef - 1hef) and from (0.5hef – 1.5hef) will increase the concrete breakout by (22.07%, and 31.7%) respectively. Increase edge distance for concrete that has 1.0% of SFRC from (0.5hef - 1hef) and from (0.5hef – 1.5hef) will increase the concrete breakout by (14.35%, and 16.22%) respectively. Totally, the concrete breakout will improve by increasing edge distance and adding steel fiber.

3- The increasing in steel fiber from (0.0%-0.5%) of anchor group is causing increase in strain around (+1.58%, +19.37%, and +1.89%) and displacement around (+25. 39%, +7.95%, and +22.33%) corresponding to the change in edge distance (0.5 hef, 1 hef, and 1.5 hef) respectively. The increasing in steel fiber from (0.0%-1.0%) is causing increase in strain around (+46.0%, +65.16%, and +11.79%) and displacement around

(+49.21%, +15.91%, and 40.77%) corresponding to the change in edge distance (0.5 hef, 1 hef, and 1.5 hef) respectively. Meanwhile, the increasing in steel fiber of single anchor from (0.0%-0.5%) is causing increase in displacement around (+39. 73%, +56.86%, and +44.38%) corresponding to the change in edge distance (0.5 hef, 1 hef, and 1.5 hef) respectively. The increasing in steel fiber from (0.0%-1.0%) is causing increase in displacement around (+56.76%, +76.47%, and 68.75%) corresponding to the change in edge distance (0.5 hef, 1 hef, and 1.5 hef) respectively. Comprehensively, the ductility and strength will be enhanced by increasing edge distance and adding steel fiber.

4- The effect of anchor group will not be the same as the effect of single anchor. The group effect will reduce by increasing edge distance and steel fiber and then increase concrete breakout. Most of the time, the strength of group will be less than the double single anchor and the ratios for (0.0%, 0.5%, and 1.0%) is (+63.74%, +86.5%, +89.21%, +72.5%, +88.5%, +95.48%, +86.5%, +98.92%, and +100.0%) respectively and corresponding to (0.5 hef, 1 hef, and 1.5 hef).

5- The compressive, tensile, flexural strengths will increase by increasing steel fiber and vice versa. The increasing in the average strength from (0.0% -0.5% SFRC) is (3.04%, 9.05%, and 9.5%) and the increasing from (0.0% - 1.0% SFRC) is (9.62%, 10.67%, and 38.5%). The increasing of these three types of strengths is depending on the specification of steel fiber that is used to improve strengths of concrete by improving resistance, modulus of elasticity and ductility.

6- The tensile strength of concrete has significant efforts on concrete breakout for single and group anchors by effecting on the shape of failure due to changing in concrete mechanical properties. This changing in strengths can be related to compressive resistance and finding the increments when the steel fiber used during this research.

7- The ratio between modulus of rupture and the split tensile strength is almost the same for all mixtures and is ranging between (0.46-0.58). This ratio will increased by increasing steel fiber due to adding steel fiber will increase flexural little bit more than increase tensile strength.

8- The ductility and strain will increase by increasing the steel fiber. So this will increase displacement and strain of anchors to prevent sudden failure.

**5.2 Research Contribution and Impact**

1- Using of SFRC in many of applications like signpost foundations, traffic signal foundations, and the guardrails on the bridges and highways and it's highly recommended to use in elements which exposed to pure tension.

2- Trying to update the relationships between the nominal concrete breakout strength and the experiment breakout strength when the SFRC is intending to be used for many purposes like reduce the size of foundations in many structural applications or to reduce the shrinkage, cracking, and the thermal expansion.

3- Increasing of the steel fibers will try to change the shape of failure and this can be considered as advantage to increase the concrete breakout under pure tension test.

4- This improvement in concrete tensile strength by adding SFRC will give more factor safety for life of construction due to the increase of the concrete properties or can let designer to reduce size of element within the adequate concrete breakout strength.

### 5.3 Recommendations for Further Researches

1- Modeling finite element for steel fiber element under pure tension and different cases of cast-in-place single and group anchors under pure tension depending on the data that obtained in this research.

2- Investigate the behavior of the concrete breakout strength of single and anchor group for more varying edge distance within different amount of steel fibers to investigate the differences between the cast-in-place and post-installed anchor groups.

3- Investigate of using steel fiber for single and anchor group under other loads like Shear, moment, and combination of one or two with tension, in the same way, under impact and seismic loading.

4- Studying single and group of anchors action by using different diameters and embedment depths.

5- Investigate the bond effect of steel fiber on ultimate tensile strength and types of steel fibers in concrete.

6- Investigate the other type of steel fiber (Nano steel Fiber) on ultimate tensile strength and types of steel fibers in concrete.

## APPENDIX A

### LIST OF EQUATIONS

1. $Ncbg = (ANc/ANco) \times \Psi ec,N\ \Psi ed,N\ \Psi c,N\ \Psi cp,N * (Nb)$ (Ibs)

2. $Ncbg = (ANc/ANco) \times \Psi ec,N\ \Psi ed,N\ \Psi c,N\ \Psi cp,N\ \Psi ga,N * (Nb)$ (Ibs)

3. $Nb = Kc*\lambda a* \sqrt{fc`} * (hef)^{1.5}$ (Ibs)

4. $Nb = Kc*\lambda a* \sqrt{fc`} * (hef)^{1.5} * (1+Z\ \sqrt{fc`})$ (Ibs)

5. $\Psi ed,N=0.7+0.3*Ca/(1.5*hef)$

6. $Z = 77.55x^2 + 0.44x + 0.019$ (when 0.5hef)

7. $Z = 172.0x^2 – 0.78x + 0.024$ (when 1.0hef)

8. $Z = -39.71x^2 + 0.633x + 0.022$ (when 1.5hef)

9. $fc` = \frac{P}{\pi * r^2}$ (psi)

10. $f_t = \frac{2*P}{\pi * L * D}$ (psi)

11. $f_r = \frac{P*L}{B*D^2}$ (psi)

**Example** of the nominal concrete breakout strength in tension of a group of anchors (**Ncbg**).

1- For Modification factor (Z=0.019 when Ca=0.5hef) and this is for (0.0% SFRC).

$Z= 77.55x^2 + 0.44x + 0.019$

$= 77.55 (0\%)^2 + 0.44 (0\%) + 0.019= 0.019$

$N_b = Kc*\lambda a* \sqrt{fc`} * (hef)^{1.5} * (1+Z\ \sqrt{fc`})$

$N_b = 24*1*\sqrt{3874}*(2.5)^{1.5}*(1+0.019\ \sqrt{3874}\ ) =$ 12887.62 lbs

$\Psi ec,N =$ 1 (No eccentricity)

$\Psi ed,N$ = 0.7+0.3*Ca / (1.5*hef) = 0.7+0.3*0.5*2.5/(1.5*2.5)= 0.8

$\Psi c,N$ = 1.25 (No cracking at service load)

$\Psi cp,N$ = Ca / 1.5hef = 0.5*2.5/(1.5*2.5)= 0.333

$\Psi ga,N$ = 0.63 ( for group anchors)

ANc = (2(0.5hef)+S)*2(0.5hef)

= (2(0.5*2.5) +5)*2(0.5*2.5) =18.75 $in^2$

ANco= 2(0.5hef)*2(0.5hef)

= 2(0.5*2.5)*2(0.5*2.5) = 6.25 $in^2$

Ncbg = (ANc/n*ANco) * $\Psi$ec,N* $\Psi$ed,N *$\Psi$c,N* $\Psi$cp,N * (Nb) * $\Psi$ga,N

= 18.75 / (2*6.25) * 1 * 0.8* 1.25 * 0.333 *12887.62 * 0.63 = 3990.97 lbs

$N_{cbg}$ = 3990.97 lbs < Ncbg (Experiment) = 4115 lbs <u>O.K</u>

2- For Modification factor (Z=0.023 when Ca=0.5hef) and this is for (0.5% SFRC).

$Z = 77.55x^2 + 0.44x + 0.019$

$= 77.55\ (0.5\%)^2 + 0.44\ (0.5\%) + 0.019 = 0.023$

$N_b = K_c*\lambda_a*\sqrt{fc`}*(hef)^{1.5}*(1+Z\ \sqrt{fc`}\ )$

$N_b = 24*1*\sqrt{3991}*(2.5)^{1.5}*(1+0.023\ \sqrt{3991}) =$ 14701.49 lbs

$\Psi ec,N =$ 1 (No eccentricity)

$\Psi ed,N$ = 0.7+0.3*Ca / (1.5*hef) = 0.7+0.3*0.5*2.5/(1.5*2.5)= 0.8

$\Psi c,N$ = 1.25 (No cracking at service load)

$\Psi cp,N$ = Ca /1.5hef = 0.5*2.5/(1.5*2.5)= 0.333

$\Psi ga,N$ = 0.63 ( for group anchors)

Ncbg = (ANc/n*ANco) * $\Psi$ec,N* $\Psi$ed,N *$\Psi$c,N* $\Psi$cp,N * (Nb) * $\Psi$ga,N

= 18.75 / (2*6.25) * 1 * 0.8* 1.25 * 0.333 * 14701.49 * 0.63 = 4626.33 lbs

$N_{cbg}$ = 4626.33 lbs < Ncbg (Experiment) = 4705 lbs <u>O.K</u>

3- For Modification factor (Z=0.031 when Ca=0.5hef) and this is for (1.0% SFRC).

$Z= 77.55x^2 + 0.44x + 0.019$

$= 77.55\ (1.0\%)^2 + 0.44\ (1.0\%) + 0.019= 0.031$

$N_b = K_c * \lambda_a * \sqrt{fc`} * (hef)^{1.5} * (1+Z\ \sqrt{fc`}\ )$

$N_b =24*1* \sqrt{4246} * (2.5)^{1.5} * (1+0.031\ \sqrt{4246}) = 18668.88$ lbs

Ψec,N= 1 (No eccentricity)

Ψed,N = 0.7+0.3*Ca / (1.5*hef) = 0.7+0.3*0.5*2.5/(1.5*2.5)= 0.8

Ψc,N = 1.25 (No cracking at service load)

Ψcp,N = Ca /1.5hef = 0.5*2.5/(1.5*2.5)= 0.333

Ψga,N = 0.63 ( for group anchors)

Ncbg = (ANc/n*ANco) * Ψec,N* Ψed,N *Ψc,N* Ψcp,N * (Nb) * Ψga,N

= 18.75 / (2*6.25) * 1 * 0.8* 1.25 * 0.333 * 18668.88 * 0.63 = 5874.81 lbs

$N_{cbg}$ = 5874.81 lbs < Ncbg (Experiment) = 5962 lbs <u>O.K</u>

4- For Modification factor (Z=0.024 when Ca=1.0hef) and this is for (0.0% SFRC).

$Z = 172.0x^2 - 0.78x + 0.024$

$= 172.0\ (0\%)^2 - 0.78\ (0\%) + 0.024= 0.024$

$N_b = K_c * \lambda_a * \sqrt{fc`} * (hef)^{1.5} * (1+Z\ \sqrt{fc`}\ )$

$N_b =24*1* \sqrt{3874} * (2.5)^{1.5} * (1+0.024\ \sqrt{3874}) = 14725.22$ lbs

Ψec,N= 1 (No eccentricity)

Ψed,N = 0.7+0.3*Ca / (1.5*hef) = 0.7+0.3*1.0*2.5/(1.5*2.5)= 0.9

Ψc,N = 1.25 (No cracking at service load)

Ψcp,N = Ca / 1.5hef = 1.0*2.5/(1.5*2.5)= 0.6667

Ψga,N = 0.86 ( for group anchors)

ANc = (2(1.0hef) +S)*2(1.0hef)

= (2(1.0*2.5) +5)*2(1.0*2.5) =50 $in^2$

ANco= 2(1.0hef)*2(1.0hef)

= 2(1.0*2.5)*2(1.0*2.5) = 25 $in^2$

Ncbg = (ANc/n*ANco) * Ψec,N* Ψed,N *Ψc,N* Ψcp,N * (Nb) * Ψga,N

= 50 / (2*25) * 1 * 0.9* 1.25 * 0.6667 *14725.22* 0.86 = 9498.24 lbs

$N_{cbg}$ = 9498.24 Ibs < Ncbg (Experiment) = 9515 Ibs <u>O.K</u>

5- For Modification factor (Z=0.0244 when Ca=1.0hef) and this is for (0.5% SFRC).

$Z = 172.0x^2 - 0.78x + 0.024$

$= 172.0\,(0.5\%)^2 - 0.78\,(0.5\%) + 0.024 = 0.0244$

$N_b = K_c*\lambda_a*\sqrt{fc`}*(hef)^{1.5}*(1+Z\sqrt{fc`})$

$N_b = 24*1*\sqrt{3991}*(2.5)^{1.5}*(1+0.0244\sqrt{3991}) = 15080.11$ lbs

Ψec,N= 1 (No eccentricity)

Ψed,N = 0.7+0.3*Ca/(1.5*hef) = 0.7+0.3*1.0*2.5/(1.5*2.5)= 0.9

Ψc,N = 1.25 (No cracking at service load)

Ψcp,N = Ca /1.5hef = 1.0*2.5/(1.5*2.5)= 0.6667

Ψga,N = 0.86 ( for group anchors)

Ncbg = (ANc/n*ANco) * Ψec,N* Ψed,N *Ψc,N* Ψcp,N * (Nb) * Ψga,N

= 50 / (2*25) * 1 * 0.9* 1.25 * 0.6667 * 15080.11 * 0.86 = 9727.15 lbs

$N_{cbg}$ = 9727.15 Ibs < Ncbg (Experiment) = 9842 Ibs <u>O.K</u>

6- For Modification factor (Z=0.0334 when Ca=1.0hef) and this is for (1.0% SFRC).

$Z = 172.0x^2 - 0.78x + 0.024$

$= 172.0\,(1.0\%)^2 - 0.78\,(1.0\%) + 0.024 = 0.0334$

$N_b = K_c*\lambda_a*\sqrt{fc`}*(hef)^{1.5}*(1+Z\sqrt{fc`})$

$N_b = 24*1*\sqrt{4246}*(2.5)^{1.5}*(1+0.0334\sqrt{4246}) = 19635.63$ lbs

Ψec,N= 1 (No eccentricity)

Ψed,N = 0.7+0.3*Ca/(1.5*hef) = 0.7+0.3*1.0*2.5/(1.5*2.5)= 0.9

$\Psi c,N$ = 0.86 (No cracking at service load)

$\Psi cp,N$ = Ca /1.5hef = 1.0*2.5/(1.5*2.5)= 0.6667

$\Psi ga,N$ = 0.86 ( for group anchors)

Ncbg = (ANc/n*ANco) * Ψec,N* Ψed,N *Ψc,N* Ψcp,N * (Nb) * Ψga,N

= 50 / (2*25) * 1 * 0.9* 1.25 * 0.6667 * 19635.63 * 0.86 = 12665.61 Ibs

$N_{cbg}$ = 12665.61 Ibs < Ncbg (Experiment) = 12684 Ibs <u>O.K</u>

7- For Modification factor (Z = 0.022 when Ca=1.5hef) and this is for (0.0% SFRC).

Z = $-39.71x^2 + 0.633x + 0.022$

$= -39.71\,(0\%)^2 + 0.633\,(0\%) + 0.022 = 0.022$

$N_b = K_c * \lambda_a * \sqrt{fc`} * (hef)^{1.5} * (1 + Z\ \sqrt{fc`})$

$N_b = 24*1*\sqrt{3874} * (2.5)^{1.5} * (1+0.022\ \sqrt{3874}) = 13990.18$ Ibs

Ψec,N= 1 (No eccentricity)

Ψed,N = 0.7+0.3*Ca / (1.5*hef) = 0.7+0.3*1.5*2.5/(1.5*2.5)= 1

Ψc,N = 1.25 (No cracking at service load)

Ψcp,N = Ca / 1.5hef = 1.5*2.5/(1.5*2.5) = 1

Ψga,N = 0.89 ( for group anchors)

ANc = (2(1.5hef) +S)*2(1.5hef)

= (2(1.5*2.5) +5)*2(1.5*2.5) =93.75 $in^2$

ANco= 2(1.5hef)*2(1.5hef)

= 2(1.5*2.5)*2(1.5*2.5) = 56.25 $in^2$

Ncbg = (ANc/n*ANco) * Ψec,N* Ψed,N *Ψc,N* Ψcp,N * (Nb) * Ψga,N

= 93.75 / (2*56.25) * 1 * 1* 1.25 * 1 *13990.18 * 0.89 = 12970.06 Ibs

$N_{cbg}$ = 12970.06 Ibs < Ncbg (Experiment) = 13093 Ibs <u>O.K</u>

8- For Modification factor (Z=0.0242 when Ca=1.5hef) and this is for (0.5% SFRC).

Z = $-39.71x^2 + 0.633x + 0.022$

$= -39.71\ (0.5\%)^2 + 0.633\ (0.5\%) + 0.022 = 0.0242$

$N_b = K_c * \lambda_a * \sqrt{fc`} * (hef)^{1.5} * (1+Z\ \sqrt{fc`})$

$N_b = 24*1* \sqrt{3991} * (2.5)^{1.5} * (1+0.0242\ \sqrt{3991}) = 15155.83$ lbs

Ψec,N= 1 (No eccentricity)

Ψed,N = 0.7+0.3*Ca/(1.5*hef) = 0.7+0.3*1.5*2.5/(1.5*2.5)= 1

Ψc,N = 1.25 (No cracking at service load)

Ψcp,N = Ca /1.5hef = 1.5*2.5/(1.5*2..5)= 1

Ψga,N = 0.89 ( for group anchors)

Ncbg = (ANc/n*ANco) * Ψec,N* Ψed,N *Ψc,N* Ψcp,N * (Nb) * Ψga,N

= 93.75 / (2*56.25) * 1 * 1* 1.25 * 1 * 15155.83 * 0.89 = 14050.71 Ibs

$N_{cbg}$ = 14050.71 Ibs < Ncbg (Experiment) = 14186 Ibs <u>O.K</u>

9- For Modification factor (Z=0.0244 when Ca=1.5hef) and this is for (1.0% SFRC).

$Z = -39.71x^2 + 0.633x + 0.022$

$= -39.71\ (1.0\%)^2 + 0.633\ (1.0\%) + 0.022 = 0.0244$

$N_b = K_c * \lambda_a * \sqrt{fc`} * (hef)^{1.5} * (1+Z\ \sqrt{fc`})$

$N_b = 24*1* \sqrt{4246} * (2.5)^{1.5} * (1+0.0244\ \sqrt{4246}) = 16010.334$ lbs

Ψec,N= 1 (No eccentricity)

Ψed,N = 0.7+0.3*Ca/(1.5*hef) = 0.7+0.3*1.5*2.5/(1.5*2.5)= 1

Ψc,N = 1.25 (No cracking at service load)

Ψcp,N = Ca /1.5hef = 1.5*2.5/(1.5*2..5)= 1

Ψga,N = 0.89 ( for group anchors)

Ncbg = (ANc/n*ANco) * Ψec,N* Ψed,N *Ψc,N* Ψcp,N * (Nb) * Ψga,N

= 93.75 / (2*56.25) * 1 * 1* 1.25 * 1 * 16010.334 * 0.89 = 14843 lbs

$N_{cbg}$ = 14843 lbs < Ncbg (Experiment) = 14963 lbs <u>O.K</u>

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