MECHANICAL AND STRUCTURAL CHARACTERIZATION OF MINI-BAR REINFORCED CONCRETE BEAMS

A Dissertation

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Sudeep Adhikari

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ABSTRACT

With major breakthroughs in material science in recent years, civil engineering construction technology has simultaneously gone through major paradigm shifts. From the conventional reinforcing material like steel, we witnessed the introduction of fiber reinforced polymer bars (FRP), fiber reinforced polymer wraps and multitudes of chopped reinforcing fiber materials, metallic and synthetic in recent times.

This dissertation covers a comprehensive research program that was undertaken at the University of Akron for the mechanical and structural characterization of concrete reinforced with new type of reinforcing fiber material, called as Basalt minibar. Basalt minibar is a non-corrosive structural macro fiber made from basalt fiber reinforced polymer (BFRP) bars. It is manufactured using a simplified automated method called wet-ley up process. Basalt mini-bar possesses higher tensile strength and stiffness compared to other standard synthetic fibers and at the meantime it is non-corrosive. Fibers act as the proactive reinforcement that provides the immediate tensile load carrying capacity as soon as micro cracks develop in concrete. Fiber reinforcement can be used as the proactive load carrying element and can also be used as supportive reinforcement to control issues like shrinkage in concrete. This demands for a type of a material which is durable, has adequate strength and stiffness to impart sufficient toughness to the structure and can mix well with concrete, developing good bond strength.
The primary motivation behind the research is to characterize Basalt mini-bar as a new hybrid-material which possesses the beneficial features of both metallic and synthetic fiber. The overall goal of the project was achieved through tensile strength characterization of Basalt fiber reinforced polymer (BFRP) bars, followed by mechanical and toughness characterization of minibar and minibar reinforced concrete (MRC), leading to strength characterization of minibar reinforced concrete (MRC) beams, finally completed with some preliminary studies on thermal behavior of minibar reinforced concrete (MRC) slab.

Based on the experimental test results and the subsequent analyses findings, it was observed that Basalt fiber reinforced polymer (BFRP) bars exhibit strength size effect. The flexural tensile strength and post-cracking tensile strength of MRC were found to increase significantly with minibar dosage within the range considered (2 to 10%). It was observed that post-cracking behavior of beams (ductility, deflection) was significantly improved due to the addition of basalt minibar. The beams were capable of undergoing larger deformation, thus exhibiting increased maximum tensile strain at failure. Theoretical models were developed to account for the number of minibar across the crack plane, maximum load carrying capacity of MRC beams and the increase in shear strength due to the addition of minibar. The structural response of MRC slab under high temperature was found not to be satisfactory.

It is believed that the dissertation provides insight into essential mechanical properties of minibar and minibar reinforced concrete. This can be further used in coming times by the new generation of engineers to study long term behavior of these materials for various durability characterizations.
DEDICATION

I'd like to dedicate this dissertation to my advisor Dr. Anil Patnaik, my parents: Bishwo Nath Adhikari and Prabha Adhikari, my brother: Sujan Adhikari, and all my dear friends.
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CHAPTER I
INTRODUCTION

The last 100 years had been the most eventful epoch in the human history in regard to major and groundbreaking paradigm-shifts in the world-views, technology advances and material-science innovations during the period. If the development of quantum mechanics and relativity can be regarded as the major shift in our material world-view, it won’t be an over exaggeration if it is considered that the introduction of composite materials to be the paradigm-shift in the field of materials. The past 100 years represents the evolution of composite materials in the similar way iron and then steel characterized 19th century [1].

Apart from their multifarious application, particularly in the performance-based sectors such as defense and aerospace, composite materials are gradually finding their way to establish themselves to be a very good alternative material for the civil-engineering structural applications. Over the last 20 years, composite materials have developed into economically and structurally viable construction material for buildings and bridges [2]. They have proved themselves to be a very ingenious substitution to the conventional metallic materials even in civil engineering application, where both strength and stiffness play pivotal roles. In civil engineering, composites have found its application, particularly in the form of FRP (fiber reinforced polymer) bars, planar
composite wraps and FRC (fiber reinforced concrete) which can comprise non-metallic discrete fibers randomly distributed in concrete matrix. All of these forms serve definite functional purposes, depending on their nature of application and structure itself. The scope of the current research revolves around the structural behavior of concrete reinforced with a special form of non-metallic fiber made from basalt rock, here forth addressed as MRC (mini-bar reinforced concrete). Minibar is a new non-corrosive structural macro fiber made from basalt fiber reinforced polymer material that has high tensile strength and other attractive physical and mechanical properties. The introduction of basalt minibar can serve very important structural purposes in construction industry. Since minibar is a high strength polymer material, non-corrosive and light weight and stiffer than other conventional polymer fibers, it has the salient features of both stiffer steel fiber and non-stiff regular non-metallic fibers. It is a well-known fact that concrete’s tensile strength is very low compared with its compressive strength. The basic philosophy of fiber reinforced concrete is the addition of discrete metallic/non-metallic fibers to the concrete matrix such that overall post-cracking tensile behavior can be imperatively enhanced. From past research, this fact is well established that addition of fibers increases the post-cracking tensile strength of the concrete and alteration of failure mode from brittle to ductile thus leading to improved toughness characteristics. The increased energy absorption capacity of FRC can find its important application, especially when structure is subjected to seismic excitation and other forms of loadings such as impact or fatigue. The primary intention behind the introduction of minibar is not only to alter the failure modes of concrete, but also to enable concrete to be designed for loadbearing capacity with high flexural strength and average residual strength. It was inspired by the results
from previously conducted tests on basalt fiber reinforced polymer (BFRP) bars related to their suitability for reinforced concrete applications. It was observed that BFRP bars were identified to have tensile strength, modulus and rupture strain to be in the ranges that are suitable for civil engineering applications [3]. With the tests on BFRP reinforced concrete beams, it was revealed that wet layup production process may have provided the required roughness and unevenness resulting in adequate bond between the bars and the surrounding concrete [4]. The higher tensile strength, better corrosion resistance and capability of BFRP bar to develop adequate bond with concrete motivated the production of minibar as a random fiber reinforcement in concrete using the same method. Apart from the structural applications, it was observed that basalt minibar has other functional advantages that may establish this material as an excellent with a density of 1.9gm/cc, minibar has a density closest to concrete than the density of steel or synthetic fiber [5]. This gives minibar an advantage over others fibers during mixing of concrete. Some of the main characteristics of minibar reinforced concrete that were envisioned at the onset of this development were:

- **Tensile Strength in excess of 15 MPa**
- **Average residual strength (post-cracking strength) up to 17 MPa**
- **Elimination of reinforcing steel including wire welded mesh and steel rebars in select applications**
- **Zero corrosion and stronger concrete means thinner elements are possible**
- **The selection of the raw materials to eliminate the effects of alkaline attack on the minibar**
Improved usability with the density similar to concrete. This has several advantages such as random placement, even distribution, easy mixing, good flowability and pumping, no floating or sinking, possibility of high volume fractions and high fiber count.

All of the above mentioned features which were studied during the research program can introduce basalt minibar as a novel and attractive composite material in the field of civil-engineering, owing to its innate capability to meet more functional requirements along with stringent structural demands. The simplicity associated with the application of minibar reinforced concrete can attract various applications in civil-engineering, especially when dead-weight, space or time constraint, failure modes and energy absorption capacity are of primary concern.

1.1 Research Significance

Fiber reinforced concrete (FRC) has already established itself as a state-of-the-art construction material in contemporary civil-engineering construction industry. There are various types of fibers currently being used with different materials, geometry and surface characteristics. The increased inquisitiveness and zeal among scientists and engineers to come up with better fiber, optimized to best possible structural response is the primary motivation. The significance of current research lies in the material characterization of a new type of non-metallic fiber made from basalt rock, here forth called as mini-bar. Several practical applications have been successfully demonstrated in Europe showing the functional and economic advantages of minibar reinforced concrete for constructing pontoons, inner walls and façade walls [3].
Table 1.1: Advantages of Minibar Reinforced Concrete

<table>
<thead>
<tr>
<th>Advantage</th>
<th>Pontoon A BFRP</th>
<th>Pontoon B BFRP and MiniBars</th>
<th>Façade wall G3 MiniBars</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reduced weight due to less cover layer</td>
<td>40% less weight, enables lower costs through value chain</td>
<td>40% less weight, enables lower costs through value chain</td>
<td>Thickness can be reduced but cost effective even without reduction</td>
</tr>
<tr>
<td>Reduced labor</td>
<td>From 2 days to 4 hours</td>
<td>From 2 days to 1 hour</td>
<td>Labor reduced but cost effective even without</td>
</tr>
<tr>
<td>Lower Cost of finished concrete application</td>
<td>Same cost at factory 5% saving installed</td>
<td>Lower Cost at Factory</td>
<td>Simply less cost than the steel nets 26% lower costs overall</td>
</tr>
<tr>
<td>Longer Life</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
</tr>
</tbody>
</table>

Minibars have been found competitive for elimination of wire welded mesh for all three examples. This resulted on significant cost savings, faster constructability and production [Table 1.1]. With the advantage of reduction in dead weight, the overall efficiency of the construction was found to have improved.

Identification of pertinent mechanical properties and their level of applicability to serve the pre-deemed structural purposes comprise the overall significance of the project. Another significance of this research is the goal to develop sufficient fundamental background for further study of long-term behavior of this particular mini-bar reinforced concrete material and other intricate structural responses. For instance, characterization of primary short-term properties can be further used for fire-characterization, punching shear-strength and dynamic-characterization pertaining to fatigue and seismic loadings.

1.2 Research Objectives

The primary research objectives of the research presented in this dissertation are as follows:
• Material characterization of basalt minbar and minbar reinforced concrete (MRC)
• Toughness characterization of MRC based on material characterization
• Flexural and shear strength characterization of MRC beams based on material properties
• Development of analytical models to predict strength characteristics of MRC and MRC beams
• Flexural strength of MRC at elevated temperature
• Development of an analytical model to predict thermal characteristics of MRC

1.3 Research Methodology

This study includes the determination of mechanical properties of basalt mini-bar, basalt MRC, toughness characterization of MRC, shear and flexural characterization of MRC, finally followed by the study of its fire-response. For the determination of uniaxial tensile strength of basalt mini-bar, a special test set-up was devised and modulus of elasticity was calculated based on respective ASTM standard pertaining to uniaxial tensile strength test of fiber reinforced plastic (FRP) bars. Uniaxial tensile strength tests were performed on basalt fiber reinforced polymer bars (BFRP) of different sizes for the study of associated size-effect. Corresponding ASTM test procedures for the determination of tensile strength and modulus of elasticity of fiber reinforced plastic (FRP) bars were strictly followed during the test. All the mechanical properties of MRC were determined based on standard procedures conforming to ASTM test methods for the determination of mechanical properties of hardened concrete.
Toughness characterization which comprises determination of flexural strength, average residual strength and energy absorption capacity of MRC, relevant test methods conforming to ASTM procedures for testing of fiber reinforced concrete (FRC) were followed. All the MRC beams of varying shear span to depth ratio were tested in four-point bending and the ultimate load carrying capacities were predicted based on a theoretical model developed alongside. The theoretical model incorporates ACI rectangular stress-block and strain-compatibility method and is based on using the discrete tensile property of basalt mini-bar, in contrast to smeared tensile properties of fiber reinforced concrete.

Fire-characterization of MRC slab was performed conforming to relevant ASTM standard for the evaluation of fire-performance of steel reinforced concrete slabs.

1.4 Document Outline

This dissertation comprises four different major sections. Each section consists of a summary of pertinent literature-review, test procedures, results and analysis, finally followed by conclusion and recommendations. The outline is subdivided as below:

- Literature review
- Experimental program
  1. Size effect associated with basalt fiber reinforced polymer (BFRP) bars
  2. Mechanical Characterization of Basalt Mini-Bar and Mini-Bar Reinforced Concrete (MRC)
  3. Toughness Characterization of MRC
  4. Flexural and shear characterization of MRC Beams
5. MRC in the context of elevated temperature

- Materials and Procedures
- Experimental Method
- Analytical Models
- Results and Conclusions
CHAPTER II
SIZE-EFFECT AND ITS SIGNIFICANCE ON FRP REBAR MATERIAL UTILIZATION

This section will encompass the detailed literature review done in regard to the study of size-effect associated with fiber reinforced polymer (FRP) bars and its relevance on the material characterization. Basalt fiber reinforced polymer (BFRP) bar was investigated, experimental program was carried out to address the objective, finally followed by the development of results, discussion and conclusions.

2.1 Literature Review

Size effect is ubiquitous in nature. It can be defined as a positive or negative change in mechanical characteristics of a material when tested on different scales but with same geometry. Physical properties of a material more or less forms a constant set of parameters, irrespective of the size of the specimen selected for the test. For instance, density of concrete will be constant for a particular temperature and pressure, irrespective of the geometry and size of the specimen. However, it is well-known that structural properties of a material may not be fundamentally intrinsic and need to be viewed as a convoluted response of the material property, geometry and size of the specimen and the subjected test conditions. Continuum mechanics is based on the fundamental assumption
of isotropy and homogeneity which ensure the preservation of micro-structural symmetry in varying directions. These factors further account for the constant material property in different directions. The constancy of material property in different directions and size can render the structure response of the material under test to be independent of geometry and scale. Investigation of size-effect was therefore found not very rigorous in the field of solid mechanics [6]. However, as new levels of material complexity is studied, for instance composites, more intricate behavioral response is encountered. A composite is a multi-phase material system whose mechanical properties are the synergistic response of its constituents. It is not difficult to surmise that the micro-mechanical response of these materials is inherently anisotropic and heterogeneous. Concrete and FRP rebar are the striking examples that are pertinent to this study. Size effect has been found to have pronounced effect on the mechanical characteristics of such materials. To a certain extent, every isotropic material is a mathematical idealization. If the micro and mesoscopic scales are probed, any material can be considered to be heterogeneous which actually contributes for the actuation of different toughening mechanism during failure. In case of brittle composite materials, these heterogeneities will be definitely magnified owing to the variability in material properties, construction procedures, interaction mechanisms and distribution of unavoidable defects and flaws in the material continua. One of the implications of these factors from the fracture mechanics point of view is the increase in effective fracture surface area than the nominal one [7]. It should be noted that the variation or scatter in material properties with size is more pronounced in composite materials compared to their metallic counterparts where plastic flow relieves the stress-intensifying effect of the flaw, thus ensuring lesser scatter in the strength properties [8].
In the case of polycrystalline metals, the characteristic micro-structure doesn’t vary significantly in the small tested specimens which render their strength characteristics more or less volume-independent. It has been regularly observed that the mechanical strength of composites such as FRP bar increases with decrease in size. These observations lead to an important conclusion that size and scale effect in continuum and composite materials is the global response of the internal micro-structure of their constituents. It will not be an exaggeration to state that the intensity of size effect varies inversely with isotropy of the micro-structural constitution of the respective material. This is supported by the fact that size-effects are more pronounced in composites such as concrete and FRP bar in contrast to their homogenous metallic counterparts.

The phenomenological origin of size-effects can be traced back to mechanistic observations, however, the scientific formalism dealing with their quantification hinges on the associated statistical parameters. Bazant [6] stated that indulgence of structural engineers on non-statistical mechanistic size effect is due to the fact that classical elasticity and plasticity theories in which failure criterion is expressed in terms of stress and strain are independent of size effect. However, it is to be noted that size effects also need to be dealt with statistically, regarding the non-deterministic and stochastic nature of the internal micro-structure of the material. This was ingeniously observed by Mariotte in 1686, dealing with the strength of a long and short rope of the same size. Investigating on the strength of a wire of same size but of varying length, he concluded that a long and short rope always support the same load unless there is a presence of faulty place in long rope in which it will break sooner than in the shorter [6]. This simple observation can throw light on the origin of size-effect on composite materials, which are inherently
anisotropic and heterogeneous, hence more susceptible to statistical effects due to avoidable presence of imperfections and unaccounted interactions. As the size of the specimen increases, it is reasonable to assume that there is more probability of the inclusion of more imperfections, thus giving rise to reduced strength characteristics. Griffith experimentally demonstrated that the nominal strength of glass fiber increased from 42,300 psi to 491,000 psi when the diameter is reduced to 0.00013 in from 0.0042 in [6]. The reduced strength was apparently attributed to the increased number of voids and imperfections in bigger-sized diameter glass fiber.

This leads to the observation that size effect associated with heterogeneous materials can be viewed under two different schools of thought:

1. Mechanistic Size-Effect-The mechanistic size effect can be primarily attributed to the losses generated by the interaction mechanisms that are actuated during the force transfer through the composite materials. The integral strength response of a composite bar is the combined effect of the load carried by each fibers facilitated by the load-transfer from fiber to matrix and vice versa. As the size of FRP bar increases, the effective number of fibers also increases thus the transfer mechanisms will be more tortuous. This will induce unavoidable eccentricity in the member thus giving rise to other internal force resisting mechanism such as interlaminar and transverse shear, leading to micro-buckling of fibers. Fiber composites with unidirectional reinforcement (FRP bars) have been observed to undergo a typical compression failure. It involves the transverse propagation of a kink band constituting micro-buckling of individual fibers. Axial shear-splitting cracks are liable to form between the fibers during the process [6]. Since
composite material is inherently weak in transverse direction, the effective load-carrying capacity is further attenuated. It has also been observed that the size of the specimen can have a qualitative influence on the mode of failure. Smaller structures are found to follow a more ductile failure pattern accompanied by distributed cracking and strain-softening. The stress redistribution caused by these factors gives rise to size effects of mechanistic origin, termed as “Energetic size-effect” [9]. Mechanistic size-effect, as it can be observed, will be considerably of lesser degree in case of homogenous materials like metals. This can be clarified by a simple explanation as below:

2. Statistical Size-Effect: Statistical size effect is a quantitative measure of the smeared effect of the inherent imperfections and flaws in the composite material. As the member size increases, there is every possibility that more imperfections will be incorporated into the system. The increased curing time for the resin, increment in the mean fiber length per unit length of the FRP specimen and increase in the volume of matrix, all may add up to the smeared effect leading to reduced FRP strength. Scaling-problem is particularly rendered more rigorous in case of composites due to the intricacy in their micro-structure, material properties and manufacturing-processes and techniques [10]. Here lies the fundamental of the famous Weibull theory of size effect. Weibull’s weakest link concept states that the probability of finding a critical imperfection in a given material increases with increasing volume [11]. However, Z.P. Bazant was of the opinion that weibull theory is not totally applicable to quasibrittle material like concrete and composite where power law defines size effect, implying absence of
any characteristic length of the material or structure. In Weibull theory, the size-effect associated with nominal strength characteristics arises due to the increased probability of encountering a material element of critical length within the volume [6]. Considering the heterogeneous nature of the material distribution in composites, it is very likely that the presence of critical length is a distinct possibility for composites. Size effect not only translates to change in strength characteristics but it may alter the possible structural behavior near failure. When strain-softening and strain-localization is taken into account, the material behavior ranges from brittle to ductile by merely increasing the size, materials properties and geometry remaining constant [11]. Bazant [9] stated that brittle materials whose size exceeds the fracture process zone (FPZ), failure is completely brittle and the size-effect is generated statistically governed by the weakest link in the structure. Carpinteri considered two models for the analysis of size-effect associated with concrete members. The first one considers an ideal material with a random distribution of microdefects and the second one considers a cohesive-crack zone constitutive model. It was observed that both models predict a decrease in apparent strength with the increase in size. From the studies on tensile strength of carbon fibers, T. Tagawa et al. [12] reported the apparent decrease in the tensile strength of carbon fiber with increasing size. The effects on fracture stress with variable gage-length and diameter were considered. Size effect was found to exhibit on both cases, however, the effects were markedly pronounced in the case of change in diameter. It was observed that the dependency on the diameter is almost 10 times of that of dependency on gage-length [12]. This
anisotropy on size-effect was indicative of the non-uniform distribution of fracture sources in the specimens. It can be inferred that the distribution of critical flaws in radial direction is more significant than in the axial direction. The author provided a mechanistic explanation for the variation in strength-characteristics in axial and radial directions based on the variation of graphitization in different direction, which is typical in case of carbon fibers.

### 2.1.1 Analysis of Strength Size Effects

Statistical weakest link theory has been a well-formed discipline providing the basic formalism for the analysis of conventional fracture study for a long time [10]. The concept was used for the statistical study of many brittle materials and Weibull made the greatest contribution by providing a probability distribution function and extending the applicability of the theory to many brittle materials [10]. The motivation of the current research to study the size-effect associated with tensile strength of basalt FRP bar in light of Weibull weakest link theory. Weakest link theory assumes that the material is constituted of smaller elements linked together and that failure of the material is followed by failure of the material or the links. The probability of failure of each link is then expressed as

\[
F(\sigma) = 1 - \exp\left\{ -\frac{1}{V_0} \int_V \phi[\sigma(r)] \, dV \right\}
\]

Where it is also assumed that \( F(\sigma) \) is an independent randomly distributed variable and it describes the strength distribution for every element. Equation 1 may be generalized for the constant uniaxial case (as in the case of basalt FRP bars in uniaxial tension)
For testing under uniform tensile stress, $\sigma(r) = \sigma = \text{constant}$, hence the equation reduces to

$$F(\sigma) = 1 - \exp \left\{ -\frac{V}{V_0} \phi(\sigma) \right\}$$  \hspace{1cm} (2)

Applying Equation 2 to two different volumes $V_1$ and $V_2$ of the same material, we can get

$$F_1(\sigma) = 1 - \exp \left\{ -\frac{V_1}{V_0} \phi(\sigma) \right\}$$  \hspace{1cm} (3)

$$F_2(\sigma) = 1 - \exp \left\{ -\frac{V_2}{V_0} \phi(\sigma) \right\}$$  \hspace{1cm} (4)

Elimination of $\phi(\sigma)$ between the two equations leads to

$$F_1(\sigma) = 1 - \left[ 1 - F_1(\sigma) \right]^{V_2/V_1}$$  \hspace{1cm} (5)

If $V_2 > V_1$, Equation 5 implies $F_2(\sigma) > F_1(\sigma)$. If we consider two bodies of different volumes (as in the case of two specimens of same materials but of different diameter) subjected to a constant stress field (as in the case of uniaxial tensile test of FRP bars) such that both volumes have an equal probability of fracture, it implies

$$F_1(\sigma) = F_2(\sigma)$$  \hspace{1cm} (6)

This leads to

$$1 - \exp \left\{ -\frac{V_1}{V_0} \phi(\sigma_1) \right\} = 1 - \exp \left\{ -\frac{V_2}{V_0} \phi(\sigma_2) \right\}$$  \hspace{1cm} (7)

This further reduces to

$$V_1 \phi(\sigma_1) = V_2 \phi(\sigma_2)$$  \hspace{1cm} (8)

If the three parameter Weibull function is used as expressed below,

$$\phi(\sigma) = \left\{ \left( \frac{\sigma - \sigma_L}{\sigma_0} \right)^m \right\} \quad \sigma \geq \sigma_L$$  \hspace{1cm} (9)

$$= 0 \quad \sigma < \sigma_L$$
Where $\sigma_L$ is the stress level below which there is no failure while $m$ and $\sigma_0$ are called shape parameter and scale parameter respectively. They depend on the manufacturing process of the material [13]. This will further lead to

$$V_1 \left[ \frac{(\sigma_1 - \sigma_{L1})}{\sigma_{01}} \right]^{m_1} = V_2 \left[ \frac{(\sigma_2 - \sigma_{L2})}{\sigma_{02}} \right]^{m_2}$$

(10)

If the bodies are of the same material, it implies $m_1 = m_2 = m$, $\sigma_{01} = \sigma_{02} = \sigma_0$ and $\sigma_{L1} = \sigma_{L2} = \sigma_L$ and it leads to

$$\sigma_2 = \left[ \left( \frac{V_1}{V_2} \right)^{\frac{1}{m}} \left( \frac{\sigma_1}{\sigma_L} - 1 \right) + 1 \right] \sigma_L$$

(11)

The above equation expresses the linearity of the fracture stresses of equal probability [13]. When the lower limit-stress is null $\sigma_L = 0$ and if the specimens are of the same material

$$\sigma_2 = \left( \frac{V_1}{V_2} \right)^{\frac{1}{m}} \sigma_1$$

(12)

As we can see, this Equation 12 relates the variation of strength with the volume of the material, hence, provides a quantitative measure of the size-effect. By its very nature, we can see that it is strictly of statistical origin. The shape parameter, $m$, can also be approximated with the coefficient of variation by the following relation [10].

$$m \approx \frac{1.2}{C.V.}$$

(13)

Size-effect in the case of uniaxial FRP bars can have different contextual significance. The first one is generated by the scaling effect between actual structure size and the laboratory prototype. When we are supposed to investigate the response of a structural member under particular loading-condition, we simulate the condition that can be implemented within a laboratory experimental program. This is achieved by scaling
the actual size of the structure to a smaller magnitude. The scaling generally constitutes of higher magnitude in the case of civil engineering structures. This can be termed as extrinsic size-effect. The extrinsic size-effect is solely due to highly reduced size of the prototype during the conventional experimental programs. For instance, a FRC beam tested for average residual strength as per ASTM is a highly rarefied experimental simplicity that is very unlikely to represent the reinforced concrete structures in actual field. Hence, it can be observed that extrinsic size-effect arises due to the huge difference in the size of the actual structure and its prototype. It may not play a significant role when the sizes of two specimens are not varying by considerable degree. In the case of uniaxial FRP rebars, the size-effect finds a different kind of expression. It arises not just because of the scaling effect between the structure and the prototype, but the intrinsic behavior of the material system itself. Intrinsic size-effect manifests itself in the case of FRP uniaxial rebars where it is not the result of scaling but the actual variation in the fracture-strength of the material with size. It can be inferred that intrinsic size-effect can have more pronounced effect on the structural behavior of the system as they are more of a mechanistic origin. Conclusively, Size-effect is finally categorized as follows:

1. Extrinsic Size Effect: Arises from the scaling phenomena; give rise to statistical effects

2. Intrinsic Size-effect: Arise from the material-system; more of a mechanistic origin
2.2 Materials

This sub-section will provide a detailed description of the primary material, which is basalt fiber reinforced polymer (BFRP) bar, and the general description of the secondary materials used for the preparation of specimen.

2.2.1 BFRP Bars

The basalt bars which were used for the research were provided by the sponsor Refrotech (Norway) and were of nominal diameters of 3mm, 5mm, 7mm, 12mm and 15mm. All the sizes correspond to the gross size governed by the fibers, including the effect from polymeric resin. Wet ley-up process is a very simple of producing FRP composite materials which essentially consists of laying up the fibers and impregnating them with the polymeric resin such that it yields the usable composite material when cured. The fibers for the case of basalt FRP bars were extracted from the igneous rock called basalt. Primary composition of basalt rock generally includes various forms of oxides, silica-oxide being the most abundant one. The percentage of silica oxide is generally between 51.6 to 57.5 percent and generally the basalt with the silica-oxide content above 46 percent (acid-basalt) is considered good for fiber-production. Polymeric resin that was used as the matrix was vinylester.

2.2.2 Steel Anchorage Tube

Since basalt FRP bar is an anisotropic composite material, its strength in transverse direction is low compared to its high axial tensile strength. This necessitates
the provision of some kind of anchor at the end of the specimen while applying the load during the tensile test. To meet this purpose, two steel tubes at each end, as dictated by the test methodology, were provided. Steel tube used for the anchor are black-welded steel with threads at both ends to facilitate the positioning of caps. Length of the steel tube grip was varied from 12 inches to 24 inches depending upon the size of the BFRP bar being tested. Lengths of grips were determined based on the judgment of the experimenter, which was governed by the observation that the length must be sufficient enough to prevent any premature bond-failure at the BFRP bar and grouting material interface.

2.2.3 Grouting Material-Epoxy

Epoxy used for the purpose of grouting steel tubes with basalt FRP bar is a commercial structural epoxy, designated as AKA-Epoxy system. The mix ratio for the particular epoxy recommended by the manufacturer is 1 part resin to 1 part hardener by volume, with gel time as per specified by the manufacturer was 180 minutes. Viscosity of the epoxy, as reported by the manufacturer was around 2300 centipoises.

2.3 Test Procedures

This section encompasses the entire set of laboratory tasks which were performed to determine the tensile strength of basalt FRP bars of different sizes, finally leading to an insight into its size-effect response. Tensile strength test of basalt FRP bar was performed conforming to ASTM D 7205, the standard method for the determination of uniaxial tensile strength of the FRP pultruded bars. This essentially consists of applying the tensile
load on the bar in the universal testing machine and loading it up to rupture. Experimental test program primarily consisted of the preparation of basalt FRP test specimen of different sizes, uniaxial tensile strength test in the lab conforming to the standard test procedure, and finally the data retrieval. All the related work is described the following sub-sections. A general schematic for the uniaxial tensile strength of basalt FRP bar is shown on Figure 2.1 below.

Figure 2.1: Schematic of the Tensile Test Set up for Basalt FRP Bar

As depicted in Figure 2.1, the steel anchors provided at the end of specimen were engaged in the grips of universal testing machine. The test specimen and was
monotonically loaded to failure at a uniform rate, till rupture. An extensometer was provided at the mid-length of the specimen which was connected with data acquisition system for continuous data recording. It should be noted that the lengths of grip were varied depending on the size of basalt FRP bar to prevent premature debonding at the interface of FRP bar and grouting material.

The elevation and cross-sectional details of a tensile test specimen for a particular size is depicted in Figure 2.2.

![Elevation and Cross-Sectional Details of a Tensile Test Specimen](image)

Figure 2.2: Longitudinal and Cross-Sectional Details of a Typical Basalt FRP Tensile Test Specimen (Note: Free and anchorage length varies with the size of Basalt FRP bar)

2.3.1 Preparation of Specimens

The first step undertaken for the accomplishment of the tensile strength test of the basalt FRP bar was the preparation of a wooden framework to hold the tensile test specimens vertically to facilitate the grouting of the anchorage tubes with epoxy. A wooden framework, as shown in Figure 2.3 was prepared in the material lab of the Civil Engineering department at the University of Akron.

As the free-length and anchorage length is a function of bar size, the overall lengths of tensile test specimens for varying bar sizes were different. As per the test
standard, the free length needs to be greater than or equal to 40 times the diameter of the bar to be tested. Anchorage length for different sizes were judiciously selected based on the tests performed previously. The required bar specimens were cut to the required lengths, were positioned on the anchorage steel tube and steel caps were attached at the ends. Specimens were sufficiently clamped to the wooden framework. A commercial structural epoxy system, named as AKA-Epoxy system was used to grout the steel tubes. The specimens were kept in the same position for 24 hours to allow for the epoxy to set, inverted and similarly grouted at the other end. Figure 2.4 and Figure 2.5 show the prepared tensile test specimens of two different sizes.
2.3.2 Experimental Program

All the specimens were tested in civil engineering material lab on a universal testing machine (UTM), brand name of “BALDWIN” with a capacity of 300,000 lb. Schematic representation of the arrangement of the specimens on the testing machine is depicted in Figure 2.1. Extensometer is positioned approximately at the center of the free
length. The extensometer used for the experiment is large-gage length model extensometer, model 3543-SR-0300-200T-ST with a gage-length of 2”. The extensometer is hooked up with the data acquisition system, ADMAT. Specimen was loaded at a constant rate of 15 pounds per second until it is stressed to the complete rupture of the specimen. Figure 2.6 shown below shows the positioning of the strain-gage on the tensile test specimen.

![Image: Strain-Gage Positioning on Tensile Specimen]

2.4 Results and Analysis

This sub-section includes the description of the test results obtained from the tensile test performed on the basalt FRP bars of five different sizes and the associated
size effect. The tensile strength test provided the useful information about the tensile strength of basalt FRP bar of different diameters, their failure modes and stress-strain behavior. Results, therefore, consisted of stress-strain curves, their ultimate tensile strengths, rupture strains and modulus of elasticity. However, the primary motivation of this phase of project was to find out if basalt FRP bars exhibit reduction in their uniaxial tensile strength with the increase in their size, hence the size-effect.

2.4.1 Strength Characteristics

The strength characteristics which comprises uniaxial tensile strength, rupture strain and modulus of elasticity of basalt BFRP bars of different sizes are tabulated below. It can be observed that the uniaxial tensile strength is decreasing with increasing size of basalt FRP bar.

Table 2.1 shows the details of the uniaxial tensile strength tests performed on R4 basalt FRP bar WITH 4.3 mm gross (nominal) diameter (referred as R4 by the sponsors). Result includes maximum load till rupture, maximum stress (tensile strength), rupture at failure (rupture strain) and longitudinal modulus of elasticity with their average value and standard deviation.
Table 2.1: Tensile Test Summary of R4 BRFP Bars

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Max Load</th>
<th>Rupture Strain</th>
<th>Stress</th>
<th>Modulus</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>lb</td>
<td>in/in</td>
<td>ksi</td>
<td>ksi</td>
</tr>
<tr>
<td>1</td>
<td>3635</td>
<td>0.024</td>
<td>161.86</td>
<td>7445</td>
</tr>
<tr>
<td>2</td>
<td>3734</td>
<td>0.031</td>
<td>166.07</td>
<td>6025</td>
</tr>
<tr>
<td>3</td>
<td>3615</td>
<td>0.028</td>
<td>160.85</td>
<td>6080</td>
</tr>
<tr>
<td>4</td>
<td>3055*</td>
<td>0.022*</td>
<td>136.60*</td>
<td>7147</td>
</tr>
<tr>
<td>5</td>
<td>3512</td>
<td>0.027</td>
<td>156.21</td>
<td>7997</td>
</tr>
<tr>
<td>6</td>
<td>4038</td>
<td>0.036</td>
<td>179.56</td>
<td>6859</td>
</tr>
<tr>
<td>7</td>
<td>3846</td>
<td>0.031</td>
<td>171.14</td>
<td>5903</td>
</tr>
<tr>
<td>8</td>
<td>3516</td>
<td>0.033</td>
<td>156.35</td>
<td>5796</td>
</tr>
<tr>
<td>9</td>
<td>4092</td>
<td>0.029</td>
<td>182.17</td>
<td>6155</td>
</tr>
<tr>
<td>10</td>
<td>3552</td>
<td>0.028</td>
<td>157.95</td>
<td>6231</td>
</tr>
<tr>
<td>11</td>
<td>3891</td>
<td>0.029</td>
<td>173.18</td>
<td>6387</td>
</tr>
<tr>
<td><strong>Mean</strong></td>
<td>3743</td>
<td><strong>0.03</strong></td>
<td><strong>167</strong></td>
<td><strong>6548</strong></td>
</tr>
<tr>
<td><strong>Std Dev</strong></td>
<td>214</td>
<td><strong>0.003</strong></td>
<td><strong>9.5</strong></td>
<td><strong>715</strong></td>
</tr>
</tbody>
</table>

Table 2.2 shows the details of the uniaxial tensile strength tests performed on R7 basalt FRP bar of 7.1 mm gross (nominal) diameter (referred as R7 by the sponsors). Result includes maximum load till rupture, maximum stress (tensile strength), rupture at failure (rupture strain) and longitudinal modulus of elasticity with their average value and standard deviation.
Table 2.2: Tensile Test Summary of R7 BFRP Bars

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Max Load</th>
<th>Rupture Strain</th>
<th>Stress</th>
<th>Modulus</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>10132</td>
<td>0.027</td>
<td>164.76</td>
<td>6045</td>
</tr>
<tr>
<td>2</td>
<td>10804</td>
<td>0.032</td>
<td>175.64</td>
<td>5831</td>
</tr>
<tr>
<td>3</td>
<td>8763</td>
<td>0.026</td>
<td>142.43</td>
<td>5981</td>
</tr>
<tr>
<td>4</td>
<td>9064</td>
<td>0.026</td>
<td>147.36</td>
<td>5860</td>
</tr>
<tr>
<td>5</td>
<td>9462</td>
<td>0.026</td>
<td>148.95</td>
<td>5875</td>
</tr>
<tr>
<td>6</td>
<td>10209</td>
<td>0.027</td>
<td>153.74</td>
<td>5773</td>
</tr>
<tr>
<td>7</td>
<td>9820</td>
<td>0.031</td>
<td>165.92</td>
<td>7114</td>
</tr>
<tr>
<td>8</td>
<td>9815</td>
<td>0.026</td>
<td>159.54</td>
<td>6682</td>
</tr>
<tr>
<td>9</td>
<td>10420</td>
<td>0.024</td>
<td>159.54</td>
<td>6518</td>
</tr>
<tr>
<td>10</td>
<td>10420</td>
<td>-</td>
<td>169.40</td>
<td>6261</td>
</tr>
</tbody>
</table>

**Mean**            | **9891** | **0.027**      | **159**  | **6194** |
**Std Dev**         | **641**  | **0.003**      | **11**   | **445**  |

Table 2.3 shows the details of the uniaxial tensile strength tests performed on R10 basalt FRP bar of 9.8 mm gross (nominal) diameter (referred as R10 by the sponsors). Result includes maximum load till rupture, maximum stress (tensile strength), rupture at failure (rupture strain) and longitudinal modulus of elasticity with their average value and standard deviation.
Table 2.3: Tensile Test Summary of R10 (10 mm) BFRP Bars

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Max Load, lb</th>
<th>Strain, in/in</th>
<th>Stress, ksi</th>
<th>Modulus, ksi</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>21379</td>
<td>0.030</td>
<td>182.46</td>
<td>6588</td>
</tr>
<tr>
<td>2</td>
<td>19974</td>
<td>0.029</td>
<td>170.56</td>
<td>6630</td>
</tr>
<tr>
<td>3</td>
<td>17542</td>
<td>0.024</td>
<td>149.68</td>
<td>6544</td>
</tr>
<tr>
<td>4</td>
<td>18144</td>
<td>0.018</td>
<td>154.76</td>
<td>6814</td>
</tr>
<tr>
<td>5</td>
<td>17767</td>
<td>#</td>
<td>151.56</td>
<td>6208</td>
</tr>
<tr>
<td>6</td>
<td>18194</td>
<td>#</td>
<td>155.34</td>
<td>6163</td>
</tr>
<tr>
<td>7</td>
<td>16690</td>
<td>0.024</td>
<td>142.43</td>
<td>6715</td>
</tr>
<tr>
<td>8</td>
<td>17928</td>
<td>0.027</td>
<td>153.01</td>
<td>6740</td>
</tr>
<tr>
<td>9</td>
<td>16969</td>
<td>0.024</td>
<td>144.75</td>
<td>6435</td>
</tr>
<tr>
<td>10</td>
<td>17031</td>
<td>0.024</td>
<td>145.33</td>
<td>6450</td>
</tr>
<tr>
<td>11</td>
<td>18299</td>
<td>0.023</td>
<td>156.21</td>
<td>6686</td>
</tr>
<tr>
<td>12</td>
<td>17807</td>
<td>0.024</td>
<td>152.00</td>
<td>6401</td>
</tr>
<tr>
<td>13</td>
<td>20464</td>
<td>0.026</td>
<td>174.63</td>
<td>6316</td>
</tr>
<tr>
<td>14</td>
<td>18677</td>
<td>0.022</td>
<td>159.40</td>
<td>6150</td>
</tr>
<tr>
<td>Mean</td>
<td>18348</td>
<td>0.024</td>
<td>157</td>
<td>6488</td>
</tr>
<tr>
<td>Std Dev</td>
<td>1368</td>
<td>0.003</td>
<td>11.7</td>
<td>221</td>
</tr>
</tbody>
</table>

Table 2.4 shows the details of the uniaxial tensile strength tests performed on basalt FRP bar of 12.6 mm gross (nominal) diameter. Results include maximum stress (tensile strength), rupture at failure (rupture strain) and longitudinal modulus of elasticity with their average value and standard.
Table 2.4: Tensile Test Summary of 12.6 mm BFRP Bars

<table>
<thead>
<tr>
<th>Specimen Type</th>
<th>Specimen ID</th>
<th>$f_{fu}$ ksi</th>
<th>$f_{fu}$ Mpa</th>
<th>$E_f$ ksi</th>
<th>$E_f$ Mpa</th>
<th>$\varepsilon_f$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>145</td>
<td>999</td>
<td>5900</td>
<td>40,651</td>
<td>0.026</td>
</tr>
<tr>
<td>1</td>
<td>2</td>
<td>137</td>
<td>944</td>
<td>6250</td>
<td>43,063</td>
<td>0.024</td>
</tr>
<tr>
<td>4</td>
<td>1</td>
<td>132</td>
<td>909</td>
<td>5900</td>
<td>40,651</td>
<td>0.023</td>
</tr>
<tr>
<td>4</td>
<td>2</td>
<td>134</td>
<td>923</td>
<td>6200</td>
<td>42,718</td>
<td>0.023</td>
</tr>
<tr>
<td>Mean</td>
<td></td>
<td>137</td>
<td>943.93</td>
<td>6063</td>
<td>41771</td>
<td>0.024</td>
</tr>
<tr>
<td>Std Dev</td>
<td></td>
<td>5.72</td>
<td>39.38</td>
<td>189</td>
<td>1300</td>
<td>0.001</td>
</tr>
</tbody>
</table>

From the tables above, it can be observed that the uniaxial tensile strength of basalt FRP bar is decreasing with the diameter, hence, exhibiting size effect. This is in agreement with the pre-established consensus among engineers that uniaxial composite materials exhibit size effect, that is, strength varies inversely with the size. Modulus of elasticity and rupture strain were found to be independent of bar diameter. The observed size-effect can be rationally viewed under the Weibull’s weakest link theory (as discussed in the literature review) which states that the probability of finding more imperfections and weak links within a control volume increases as the size of the specimen increases, which leads to the increased probability of the failure of the specimen at lesser load. Similar to other fiber reinforced polymer (FRP) bars, basalt FRP bar is also a two-phase heterogeneous material system. The less-standard method of manufacturing, non-
uniformity generated by curing history, non-uniform distribution of stress across the section with the increase in size (shear-lag effect) and the intrinsic heterogeneity of individual fibers within a control volume can be regarded to be the primary causes leading to observed size effect. To determine whether the observed size effect is actually statistical or structural in origin, more tests are deemed necessary and this part of research is recommended to be continued in coming days.

Based on the experimental results as discussed above, an empirical equation correlating the variation of uniaxial average tensile strength of basalt FRP bar with its diameter is proposed. The equation is given as below:

\[ Y = 0.23X^2 - 26X + 1250 \]  

(14)

Where

\( Y \)-Uniaxial average tensile strength of basalt FRP bar (MPa)

\( X \)- Diameter of basalt FRP bar (mm)

Figure 2.7 depicts the graphical depiction of the proposed equation and a relative comparison with the similar response exhibited by glass FRP bars.
2.4.2 Failure Mode and Tensile Stress-Strain Curve

For all the basalt FRP bars tested, the stress-strain relationship was found to be linear till failure. The failure was accompanied by the transfer of load to the individual fibers through the vinylester matrix, gradually leading to the debonding between the fibers and polymeric matrix, finally resulting in a volumetrically unstable brittle failure of the complete specimen.

The typical stress-strain curves for different sizes of basalt FRP bars tested in uniaxial tension are shown in figures below.
Figure 2.8: Typical Stress-Strain Curves for 12.6 mm Basalt FRP Bars
Figure 2.9: Typical Stress-Strain Curves for 5 mm FRP Bars

Figure 2.10: Typical Stress-Strain Curves for 3 mm Basalt FRP Bars
2.4.3 Size Effect of Basalt FRP in The Context of Weibull’s Distribution

It was discussed in the literature review section that Weibull’s weakest link theory can be successfully applied to account for the size-effect associated with brittle and quasibrittle materials. It has been reported by Uwe et al. [14] that the strength of a brittle spherical particle (glass, clinker cement, limestone) decreases with increasing particle size and can be accounted with Weibull’s distribution. The volume-dependent strength characteristics is also exhibited by ceramic material which can be attributed to increased probability of finding critical crack size with increasing magnitude of the control volume [15].

From the experimental work as previously presented and discussed, basalt FRP bars exhibit volume dependent strength property, hence lack a deterministic strength index. The fact can be qualitatively explained based on the same formalism of ceramics material, however, it cannot be ignored that basalt FRP bar is a material with remarkably different micro structure. Basalt FRP bar is a poly-phase composite material with intricate micro-structure. This is further complicated by the variability induced by manufacturing techniques and conditions, material properties of fiber and matrix and other stochastic factors such as curing history [16]. For instance, for an advanced multi-phase composite material (like basalt FRP bar) the critical crack length is generally within the fiber, however, it can be also be present within the matrix phase in the form of micro cracks [16].

Weibull’s weakest link theory has been used to account for the size-effect associated with wood, which is a natural fibrous composite. Sutherland et al. were of the
opinion that man-made fiber reinforced plastics (such as basalt FRP bar in our case) can be viewed structurally analogous to natural composites, hence, amenable to Weibull’s weakest link theory. Carbon/epoxy laminates has been studied in aerospace field using Weibull’s theory [16]. Similarly Weibull’s statistics was used to quantify the size effect exhibited by glass fiber reinforced polymer (GFRP) during fatigue test [17]. Since fatigue is more volume-critical phenomena [17], where Weibull’s modulus can be as small as 1, applicability of Weibull’s statistics is more justified in the case of basalt FRP bars subjected to uniform uniaxial stress field.

Table 2.5 shows the statistical parameters of interest for basalt FRP bars of different sizes.

Table 2.5: Statistical Parameters for Basalt FRP Bars of Different Sizes

<table>
<thead>
<tr>
<th>Test #</th>
<th>Max Load</th>
<th>Strain (measured)</th>
<th>Stress</th>
<th>Modulus</th>
<th>Strain Calculated</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>kN</td>
<td>mm/mm</td>
<td>Mpa</td>
<td>Gpa</td>
<td>mm/mm</td>
</tr>
<tr>
<td><strong>R4 Bars</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mean</td>
<td>16.65</td>
<td>0.0295</td>
<td>1148</td>
<td>44.73</td>
<td>0.0260</td>
</tr>
<tr>
<td>Std. Dev</td>
<td>0.95</td>
<td>0.00320</td>
<td>66</td>
<td>4.99</td>
<td>0.00311</td>
</tr>
<tr>
<td>COV</td>
<td>0.057</td>
<td>0.108</td>
<td>0.057</td>
<td>0.11</td>
<td>0.12</td>
</tr>
<tr>
<td><strong>R7 Bars</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mean</td>
<td>43.44</td>
<td>0.0272</td>
<td>1094</td>
<td>42.70</td>
<td>0.0261</td>
</tr>
<tr>
<td>Std. Dev</td>
<td>2.85</td>
<td>0.00252</td>
<td>73</td>
<td>3.07</td>
<td>0.00235</td>
</tr>
<tr>
<td>COV</td>
<td>0.066</td>
<td>0.093</td>
<td>0.066</td>
<td>0.072</td>
<td>0.083</td>
</tr>
<tr>
<td><strong>R10 Bars</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mean</td>
<td>81.61</td>
<td>0.0250</td>
<td>1080</td>
<td>44.73</td>
<td>0.0242</td>
</tr>
<tr>
<td>Std. Dev</td>
<td>6.09</td>
<td>0.00309</td>
<td>81</td>
<td>1.52</td>
<td>0.00200</td>
</tr>
<tr>
<td>COV</td>
<td>0.075</td>
<td>0.126</td>
<td>0.075</td>
<td>0.034</td>
<td>0.083</td>
</tr>
</tbody>
</table>
Based on the literature review discussed above, the Weibull’s modulus was calculated and is shown in Table 2.6.

Table 2.6: Weibull’s Modulus for Different Basalt FRP Bar Sizes

<table>
<thead>
<tr>
<th>Bar Designation</th>
<th>Nominal Diameter (mm)</th>
<th>COV of Uniaxial Strength</th>
<th>Weibull's Modulus</th>
</tr>
</thead>
<tbody>
<tr>
<td>R4</td>
<td>4.7</td>
<td>0.057</td>
<td>21</td>
</tr>
<tr>
<td>R7</td>
<td>7.1</td>
<td>0.066</td>
<td>18</td>
</tr>
<tr>
<td>R10</td>
<td>9.8</td>
<td>0.075</td>
<td>16</td>
</tr>
</tbody>
</table>

*Average COV from the Sample Population* 0.066 18

Weibull’s theory is based on the assumption that Weibull’s modulus is a material parameter, hence independent of shape and size of the loaded object. Weibull moduli for the longitudinal strength of composites were found to vary between 5 and 20, which is similar to that of ceramics material [17]. Calculated Weibull’s moduli for basalt FRP bar is found to fall within the range, however, found to be a variable.

It can be observed that Weibull’s moduli is decreasing with increasing specimen size. It was reported by Uwe et al. that Weibull’s moduli of nearly spherical brittle materials (glass, clinker cement, limestone) seems to decrease with increasing particle diameter. Kellas and Mortan reported the variation of Weibull’s moduli for carbon/epoxy composite system (7.22 to 156 for tensile test, and 8.5 to 18.3 for the flexural) [16]. It can be stated that variation of Weibull’s moduli in case of Basalt FRP bars is not significant and an average value of 18 may considered for analysis. The following factors can be deemed responsible for the observed variation:
- Weibull’s statistical theory is based on the fundamental assumption that Weibull’s modulus is a material property, hence a constant. Since each BFRP bar of different sizes can have a considerably different micro-structure (due to inhomogeneity in manufacturing process, curing history), basalt FRP bars of different sizes can be viewed to be different material in themselves.

- With the increasing specimen size, there can be invariable changes in material properties between fabrication routes. This can be responsible for the actuation of differing stress field during the test.

- Specimen with larger size has longer free length as compared to smaller size specimens. The possible eccentricity during the fabrication of specimen can induce bending and shear on the specimen during the test. This may account for the actuation of different stress fields on different sizes, which may be responsible for the observed variation in Weibull’s modulus.

- Smaller structures tend to fail in more ductile manner which is accompanied by distributed cracking with strain softening (termed as energetic size-effect), whereas in case of larger size structures (larger than fracture process zone), the failure is brittle (statistical size-effect) [9]. This may also account for the observed variation in Weibull’s modulus.

- The functional basis of a structural FRP polymer bar during loading is the gradual transfer of force to the individual fibers, facilitated by bond with polymer matrix. With the increasing size, the force transfer tends to be less uniform thus introducing different forces (such as inter-laminar shear and micro buckling of the fibers) in the polymer bar. This may introduce adverse force distribution.
mechanisms in the system, thus actuating mechanistic size-effect which is an inevitable departure with the response of smaller sized bars, hence, can be a factor affecting the observed variability in Weibull’s modulus.

Based on a detailed analysis, the following equation is being proposed to correlate uniaxial tensile strength of basalt FRP bars of different sizes.

\[ \sigma_2 = \left( \frac{L_2^3 - d_2^2 + L_2}{L_1^3 - d_1^2 + L_1} \right)^{(1/m)^2} \sigma_1 \]  

(15)

Where

\[ m \approx 1.2 \frac{COV}{10} \]  

(16)

\[ COV = 0.066 \]  

(17)

The number of uniaxial tensile strength tests conducted for R4, R7 and R10 were 11, 10 and 14 respectively. Since for the bigger sized basalt bars (12.6 and 15 mm), the number of tests conducted were less than 10, only test data pertaining to R4, R7 and R10 were considered for the calculation of Weibull’s Modulus to develop a better confidence level.

The proposed equation preserves the fundamental principle of Weibull’s statistical theory that for a uniform stress field, strength characteristics varies inversely with a function of control-volumes. Weibull’s theory has been adequately applied to account for the size effect associated with brittle materials such as glass, cement clinkers and rock. Even though both basalt FRP bar is a brittle material, it should be noted that its internal micro-structure definitely varies with all the previously mentioned materials. Basalt FRP bar is a two phase material system, consisting of basalt fibers and polymer
The micro-material defects of basalt fiber and matrix compounded with uncertainties associated with interphase bond strength, its manufacturing process, curing history and unforeseeable micro-structural flaws (such as micro-buckling of fibers and unaccounted inter laminar shear) render basalt FRP bar statistically a very complicated system.

To account for all the unforeseeable uncertainties, it was inferred that the effective control volume of the basalt FRP bar that can encompass all the probable uncertainties is greater than its geometrical control volume, hence, can also be viewed as virtual cloud-volume which can incorporate all the probable uncertainties. For basalt FRP bar, the virtual control-volume is assumed to be a cube of dimension equal to the free length of the specimen.

The uniaxial tensile strength of basalt FRP of sizes 7.1, 9.8, 12.6 and 15 mm were calculated based on the volumetric strength formulation Equation. Calculations were done with reference to the smallest basalt FRP bar of 4.7 mm diameter. The results are tabulated in Table 2.7.

**Table 2.7: Summary of Uniaxial Tensile Strength Prediction**

<table>
<thead>
<tr>
<th>Specimen Size (mm)</th>
<th>Exp (MPa)</th>
<th>Predicted (MPa)</th>
<th>% difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>7.1</td>
<td>1094</td>
<td>1054</td>
<td>-3.7</td>
</tr>
<tr>
<td>9.8</td>
<td>1080</td>
<td>1028</td>
<td>-4.8</td>
</tr>
<tr>
<td>12.6</td>
<td>945</td>
<td>932</td>
<td>-1.4</td>
</tr>
<tr>
<td>15</td>
<td>920</td>
<td>916</td>
<td>-0.4</td>
</tr>
</tbody>
</table>
It can be observed from Table 2.7 that the provided volumetric formulation based on Weibull’s weakest link theory is able to provide an accurate prediction of the strength of basalt FRP bar. The graphical representation is shown in Figure 2.11.

It can be stated that basalt FRP bars comply with Weibull’s weakest link theory within the range of the sizes tested. However, an adjustment was needed to be made to account for the multifarious variability associated with structural and material heterogeneity of FRP materials, hence the concept of virtual volume was introduced. A virtual volume is the effective control volume that can encompass various unaccounted and unforeseeable factors that can ultimately be instrumental for the actuation of size effect.

It should also be noted that proposed equation is not a mechanistic formulation, hence cannot be considered as an absolute method for the prediction of the uniaxial tensile strength of the basalt FRP polymer bars. The proposed method is a statistical approach which can facilitate engineers to have a sound interpolative technique for the estimation of uniaxial tensile strength of bigger basalt FRP bars based on the available data from the tests performed on smaller bars.
Figure 2.11: Graphical Representation of the Proposed Equation for BFRP Bars of Different Diameters

Proposed Equation:

\[ \sigma_2 = \left( \frac{L_1 - d_2^2 + L_1}{L_2 - d_2^2 + L_2} \right)^{1/m} \sigma_1 \]

\[ m \approx \frac{1.2}{COV} \]

\[ COV = 0.066 \]

<table>
<thead>
<tr>
<th>Specimen Size (mm)</th>
<th>Exp (MPa)</th>
<th>Predicted (MPa)</th>
<th>% difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>7.1</td>
<td>1076</td>
<td>1054</td>
<td>-2.0</td>
</tr>
<tr>
<td>9.8</td>
<td>1017</td>
<td>1005</td>
<td>-1.2</td>
</tr>
<tr>
<td>12.6</td>
<td>960</td>
<td>912</td>
<td>-2.9</td>
</tr>
<tr>
<td>15</td>
<td>912</td>
<td>904</td>
<td>-0.9</td>
</tr>
</tbody>
</table>
2.5 Conclusions and Recommendations

From the uniaxial tensile strength test on basalt FRP bars of five different sizes, valuable information regarding their mechanical properties and structural behavior were obtained. Their detailed discussions are tabulated below.

1. Uniaxial tensile strength and modulus of elasticity of basalt fiber reinforced polymer bars (BFRP) of varying sizes were identified. These properties will be very instrumental in characterizing the material for the structural design purposes and to study the long term behavior of the material.

2. BFRP bars exhibit size-effect on its uniaxial tensile strength, that is, strength varies inversely with its diameter. Based on the experimental results, an algebraic equation was proposed to correlate uniaxial tensile strength as the function of basalt FRP bar diameter.

The equation is given as below:

\[ Y = 0.23 X^2 - 26 X + 1250 \]

Where

Y-Uniaxial average tensile strength of basalt FRP bar (MPa)

X- Diameter of basalt FRP bar (mm)

This observation is particularly significant since the primary motive behind the research is the identification of size-effect. The identification of size-effect associated with uniaxial tensile strength of BFRP bars and its subsequent quantification will allow the engineers to have a qualitative idea of the structural
behavior of basalt FRP bar. The quantification will allow the manufacturers and
the designers to have a pre-estimate of the strength of the bar with different sizes.

3. Within the range of sizes tested, longitudinal modulus of elasticity of BFRP bars
   was found to be independent of their sizes.

4. It is recommended to conduct more tests on basalt FRP bars of different diameters
to have a theoretical basis for size-effect explanation.

5. Statistical study of the size effect exhibited by BFRP bars on uniaxial tension is
   studied in the light of Weibull’s weakest link theory. A modification was
   proposed and it was found to predict the strength of larger sized BFRP bars with
good accuracy. Hence it was concluded that Weibull’s weakest link theory with
some modification can account for the volume-dependent strength property of
BFRP bars within the range of the sizes considered. The modification was
considered based on the heterogeneous nature and unaccountable randomness on
the microstructure of BFRP bar.
CHAPTER III

MECHANICAL CHARACTERIZATION OF BASALT MINI-BAR AND MINI-BAR REINFORCED CONCRETE (MRC)

This section provides a detailed description of the literature review, lab procedures and experimental program that was carried out for the mechanical characterization of new type of material termed as basalt minibar and minibar reinforced concrete (MRC). The primary objective of this part of research-program was the identification of the mechanical properties of new kind of fiber, hereafter termed as basalt minibar (the detailed description will be covered in material sub-section). Once the mechanical characterization of basalt minibar was done, the next step was to identify the mechanical properties of the concrete reinforced with randomly distributed basalt minibar, hereafter termed as minibar reinforced concrete (MRC). The section provides the literature review, description of the materials, description of the lab-procedures and experimental program to meet the objective, finally followed by results and analysis with conclusions and recommendations.
3.1 Literature Review

Material characterization of the constituent materials comprises the most important part for the prediction of global structural response of any kind of fiber reinforced concrete (FRC). To understand the specimen behavior under particular loading scheme, it is a prerequisite to understand their local behavior under uniaxial tension and compression and at the same time, other mechanical properties such as Poisson’s ratio, split-tensile strength and modulus of elasticity. Since global structural response of the specimen, such as flexure and shear, is a convoluted translation of the intrinsic mechanical properties of the constituents, material characterization is the most fundamental step towards the study of any new type of material. The flexural and shear response of Mini-bar reinforced concrete is the symbiotic response of mini-bar and concrete, and their interaction properties. This requires an eminent need to determine the mechanical properties such as longitudinal modulus of elasticity and tensile strength of mini-bar fiber. Mechanical properties of interest for MRC are compressive strength, modulus of elasticity, Poisson’s ratio, direct tensile strength and split tensile strength for mini-bar reinforced concrete. Average residual strength (ARS) test of mini-bar reinforced beams is also included in this study and the possible relation between direct tensile strength, split tensile strength and the ARS values are investigated. The interaction property which is primarily exhibited due to bond between mini-bar and concrete, was not include within the scope of the project. However, based the results available from pull-out tests performed by the sponsors independently, the analysis can be facilitated as desired.
FRC has established itself as a state-of-the-art civil engineering technology in recent times. For any construction material, strength and stiffness are of utmost importance in most of structural applications. Depending upon the purpose of the structure, there is always a need for reciprocal trade-off between these two parameters. Composite materials such as FRP bar, concrete and FRC were introduced to serve that particular purpose. Composite materials allow the designers to tailor certain features of the constituents such that novel features can be extracted, which can’t be served by the parent elements alone. FRC was primarily introduced to enhance the toughness characteristics of regular concrete. Toughness of a material is the physical measure of the energy that is required to deform the material to a particular strain.

A tough material can undergo larger mechanical deformation as compared to a material that lacks toughness when subjected to equal amount of load. Concrete is a composite material which is very strong in compression, but possesses weak tensile strength and intrinsic brittleness. Reinforced concrete is another composite material in which reinforcement is primarily introduced to improve the tensile characteristics of concrete matrix, and finally it’s flexural and shear capacities. Reinforced concrete improves the tensile characteristics of concrete matrix by the actualization of interaction-mechanisms, aided by bond and tension-stiffening. However, in many real situations, a different kind of material is deemed necessary; material that is easy to make cast-in-situ concrete and materials with enhanced toughness characteristics. For instance, if a beam-column joint is considered in a reinforced concrete frame structure, toughness is the most pivotal structural characteristics to seek under impact or seismic loading. Also if the case
of load reversals is considered during seismic excitation, the member must be sufficiently strong to carry the tensile stress reversals across the section. Reinforced concrete, which is based on the provision of reinforcing steel bars at discrete location, is not the best option in such cases. FRC here finds tremendous beneficial applicability in such instances. This inspires the investigation of the tensile and toughness characterization of basalt mini-bar reinforced concrete as a structural concrete.

3.1.2 Mechanics and Structural Response of Fiber Reinforced Concrete

Fiber is primarily used in concrete to inhibit tensile crack growth thus significantly increasing the post-crack tensile strength of the concrete. Addition of fiber to concrete significantly alters the mechanical properties of the member such as increase in toughness (energy absorption), ductility, tensile strength and flexural strength [18]. Fibers may be metallic, organic or synthetic and may be available in various geometries. The philosophy underlying the applicability of fiber reinforced concrete in relation to civil-engineering applications is depicted in Figure 3.1.
The fiber mechanisms have also been viewed as constituting two different levels of interactions. Spacing mechanism constitutes the micro-level arrangement of well distributed fibers that will arrest the micro-cracks and inhibits further propagation. Crack bridging mechanism constitutes the macro-level which comprises bridging the flexural cracks thus facilitating improved post-cracking tensile strength, increased energy absorption, and improved ductility [19]. Figure 3.2 depicts the general response of fiber reinforced concrete in comparison with plain reinforced concrete. Fiber reinforced concrete provides better energy absorption (toughness) and the capacity to undergo large plastic deformation thereby assuring relatively ductile failure. In other words, cracked fiber reinforced concrete in Figure 3.2 exhibits much better energy absorption capacity by
undergoing large plastic deformation before failure while the plain concrete fails suddenly in a brittle manner once the peak load is reached.

Figure 3.2: Structural Response of Fiber Reinforced Concrete

Fiber reinforced concrete is generally characterized by its volume percentage in the concrete, also known as volume fraction. Volume fraction is often coupled with aspect ratio of the fiber to give a generalized characterization, $V_dL_f/d_f$, called reinforcing index [18]. The concepts and mechanics of the enhanced structural response of fiber reinforced concrete can be summarized as Shown in Figure 3.3.
3.2 Materials

This sub-section includes the detailed description of the materials which were used for the investigation of the mechanical properties of minibar and minibar reinforced concrete (MRC). This section is intended to give a comprehensive overview of the various materials that were used, their relevant characteristics, specifications and their associated properties.
3.2.1 Basalt Minibar

Basalt minibar is a discrete non-metallic fiber, provided by the sponsor Refrotech, AS (Norway). The Basalt minibar provided for the research was of nominal diameter of 0.66 to 2 mm. The method for manufacturing basalt minibar bar is an automated Wet-Lay-up process, which is the same method for the manufacture of basalt fiber reinforced polymer bar (BFRP), as discussed in chapter I. Wet Lay-up process is a very simple method of producing FRP composite materials and is performed manually. This process essentially consists of drawing the fibers through polymer filled baths such that it yields usable composite material when hardened and cured. Fibers in the case of basalt FRP bars were extracted from the igneous rock called basalt. The primary composition of basalt rock is generally constituted with various forms of oxides, silica-oxide being the most abundant one. Percentage of silica oxide is generally between 51.6 to 57.5 percent and generally the basalt with the silica-oxide content above 46 percent (acid-basalt) is considered good for fiber-production.

Minerologically, basalt is primarily constituted of minerals Plagioclase, pyroxene and olivine. When heated at high temperature, basalt is capable of producing a natural nucleating agent which plays a major role for the thermal stability of the material. This explains the apparent increased volumetric integrity of basalt as compared to the other materials. The polymeric resin used as the matrix for the basalt minibar is Vinyl ester. Vinyl ester resin is the combination of an epoxy and an unsaturated polyester resin [2].

The advantage of vinyl ester is that it has the meritorious physical properties of the epoxy and the beneficial processing properties of a polyester resin. The basalt mini-
bar are produced in long lengths which are later cut to the required length to facilitate testing. The average volume fraction of fiber in the minibar worked out to be 46%. The outer surface of the basalt mini-bar is provided with helical winding along its length to enhance its bond-properties. It was observed that basalt mini-bars are effectively similar to basalt FRP bars, scaled down to reduced diameter as shown in Figure 3.4.

Figure 3.4: Basalt Minibar

3.2.2 Grouting Material-Epoxy

Epoxy used for the purpose of grouting steel tubes with Basalt FRP bar is a commercial structural epoxy, designated as AKA-Epoxy system. The mix ratio for the particular epoxy as recommended by the manufacturer is 1 part resin to 1 part hardener by volume, with the gel time as specified as 180 minutes. The viscosity of the epoxy, as reported by the manufacturer was around 2300 centipoises.
3.2.3 Anchorage Tubes

Since basalt is an anisotropic material, the strength of the basalt minibar in the transverse direction is low compared to its very high tensile strength in longitudinal direction. Hence methods similar to the preparation of FRP tensile specimens were followed. This necessitated the provision of some kind of anchor at the end of the specimen to provide proper grip during applying the monotonic uniaxial load. To meet this purpose, steel tubes were provide at the ends. The steel tube used for the anchor are black-welded steel tubes each 10 inch long with threads at each end to facilitate the positioning of the cap.

3.2.4 Steel Plates

Since the steel tube anchorages were unstable leading to premature breaking of specimens and were also not compatible with the grips of 500lb Instron machine, a different design for the tensile test specimen was conceived. For making the grips, quarter inch thick steel plates were used. The steel plates used for the purpose were 2 inches long and one inch wide.

3.2.5 Materials Used for Making Concrete

Coarse Aggregate: The coarse aggregate used for making concrete for the cylinders was crushed limestone aggregates with the maximum size of ¾ inch (19mm). The aggregates were angular and free from clay and other impurities.
Fine Aggregate: The fine aggregate was river sand purchased from the local supplier. The sand was free from clay and other inert impurities.

Water: The water used for the concrete mixes was the normal tap water supplied by the city of Akron.

3.3 Test-Procedures

This sub-section comprises the entire laboratory tasks which were performed for the mechanical characterization of basalt minibar and minibar reinforced concrete (MRC). Relevant ASTM standards, ASTM D7205, ASTM C39, ASTM C469 and ASTM C496 were referred for the purpose. For the tensile characterization of basalt minibar, ASTM D7205 was referred with some modifications in the design of tensile specimen. For the mechanical characterization of minibar reinforced concrete (MRC), following ASTM standards for the determination of hardened concrete properties were referred.

1. ASTM C39-Standard Test Method for Compressive Strength of Cylinder Concrete Specimens
2. ASTM C469-Standard Test Method for Static Modulus of Elasticity and Poisson’s Ratio of Concrete in Compression
3. ASTM C469- Standard Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens
3.3.1 Preparation of Specimens

3.3.1.1 Preparation of Basalt minbar Tensile Specimen

Basalt mini-bar was provided in the form of circular coils. The coil was carefully unwound and cut to the length of 20 inches. During unwinding, it was ensured that each mini-bar specimens were straight to avoid any kind of eccentricity during the test. Three glass mini-bar specimens were also prepared for a relative comparison. The specimens were provided with steel tube anchorages using a special wooden stand prepared for the purpose. One end of the specimen was grouted and was let to set for 24 hours. Other ends of the specimens were grouted similarly on the following day.

It was later noted that due to the very small diameter of mini-bar, the design of specimen was not very efficient. The epoxy-grouted anchorages were relatively heavy causing it to bend the mini-bar and break it. The slenderness ratio was rendered high due to the small size of the bar, hence, making it very hard to handle the specimen in the laboratory.

The tensile test specimens prepared for the determination of uniaxial tensile strength and longitudinal modulus of elasticity from the above mentioned method is shown in Figure 3.5.
The specimens were carefully handled and tried to be tested in universal Baldwin machine. However, higher load cell capacity restricted us to retrieve stress-strain data for load level below 500lb. It was also noted afterwards that the grips of Instron machine with 500lb load-cell capacity were only compatible with flat-grips. Hence the specimens with tube-anchorage were not tested and a different test-specimen was devised. The new design was simple and stable, and it was tested on a different Instron testing machine with load cell capacity of 500lb.

To make the specimen compatible with 500lb capacity Instron machine, mini-bars were cut to required sizes and were taped straight onto the surface of the plastic board. The steel plates were coated with AKA structural epoxy system, and carefully applied at both ends. The prepared specimens with new design are shown in Figure 3.6.
Figure 3.6: Mini-Bar Samples using the Steel-plates

To avoid edge-effects at the vicinity leading to any kind of local failure caused by pinching or intrinsic eccentricity of the specimen itself, one side of the grip was provided shorter than the other. This is shown in Figure 3.7.
3.3.1.2 Preparation of Minibar Reinforced Concrete (MRC) Specimens

Preparation of specimens comprised of preparing 6 inches by 12 inches cylinders for Poisson’s ratio, static modulus of elasticity and modulus of rupture tests, preparation of tensile coupon for the direct tensile strength test, and preparation of 4 inches by 8 inches concrete cylinders for the compressive strength tests.

3.3.1.2.1 Preparation of Tensile Test Coupon for the Direct Tensile Strength Test

Since there are no standard test method for the determination of direct tensile strength of fiber reinforced concrete, based on past research, a direct tensile test coupon was designed. The scaled schematic of the tensile specimen was shown in Figure 3.8. The thickness of the specimen was 4 inches such that the cross-sectional dimension at the
neck was 4 inches by 4 inches. Bars of 0.75 in diameter were provided at the end sections to facilitate the provision of grips during testing in the BALDWIN UTM machine.

![Schematic of Direct Tensile Test Specimen](image)

**Figure 3.8: Schematic of Direct Tensile Test Specimen**

Formwork for the tensile test coupon was prepared by cutting Styrofoam conforming to the geometry of specimen as shown in Figure 3.8. The Styrofoam form was carefully protected from all sides such that adequate support was provided during horizontal casting and vibration of the concrete. A wooden box that could accommodate the form and provide support from outside was provided for the purpose. The formwork is shown in Figure 3.9.
It is a well observed phenomenon in homogenous materials that sharp geometries and abrupt change in dimensions can cause stress concentration effects around the critical vicinities. Since concrete is diversely heterogeneous and a brittle material, this issue can be more pivotal. The provision of hole at the end sections can attenuate the effective area at the particular cross-section, which in conjunction with stress-concentrating effects, can lead to local failure at the vicinity.

Considering the fact, the end sections of the specimens were uniformly reinforced with # 2 steel bars. Reinforcing meshes was created and placed at the end-sections to avoid any geometrical effects leading to local failure at critical locations. The reinforcing mesh before placement in the forms is shown in Figure 3.10:
Figure 3.10: Reinforcing Mesh for the Critical Regions of Direct Tensile Strength Test Specimen

In Figure 3.11, we can see that the critical regions were abundantly reinforced to avoid any kind of local failure.
3.3.2 Casting of Specimens

All the required components for mixing of concrete were weighed and stored in a dry place in the laboratory before the day of mixing. The cement was covered with double layer of plastic to avoid moisture absorption. Concrete was mixed in a mechanical mixer in the civil engineering materials laboratory. Ten percent of the mix was prepared as the butter mix to prevent any kind of loss of cement and water from the actual mix. The mixer was first fed with the butter mix, followed by the actual mix. First the coarse aggregate, sand, cement and one third of the quantity of water was fed to the mixer. Materials were mixed for three minutes and the mixer was stopped for one minute to allow for the hydration of the cement. The remaining water was added and the mixer was run for three more minutes. Since basalt minibar can absorb some moisture, to avoid any
moisture absorption, minibar was added before the last two minutes of mixing. This is shown in Figure 3.12.

![Figure 3.12: Preparation of Minibar Reinforced Concrete](image)

The prepared concrete was carefully placed on a wheelbarrow to facilitate the transportation of concrete. All the cylinders were vibrated on a vibrating table to assure good consolidation of the concrete cylinders. During vibration, it was taken care that the specimens were not overly vibrated, leading to preferential alignment of minibars.
Figure 3.13: Casting of Direct Tensile Strength Test Specimen

Figure 3.13 shows the casting of direct tensile test coupon. A needle vibrator was used to facilitate good compaction and distribution of minibars in the concrete mass. Figure 3.14 shows the minibar reinforced cylinder specimens prepared for static modulus of elasticity, Poisson’s ratio, compressive strength and splitting tensile strength test.
3.3.3 Experimental Program

This section includes the comprehensive account of the experimental program pertaining to direct tensile strength, Poisson’s ratio, static modulus of elasticity, compressive strength and split-tensile strength of basalt mini-bar reinforced concrete. All the tests were performed conforming to the pertinent ASTM standards (as listed above), wherever possible.

3.3.3.1 Direct Tensile Strength Test of Basalt Minibar Reinforced Concrete

There is no standard test method for the determination of direct tensile strength of fiber reinforced concrete (FRC). Based on the past research and investigations done in the particular field by other researchers, a tensile test specimen was designed, cast and
tested under monotonic uniaxial tension. The principle underlying the method is similar to tensile strength test for any kind of homogenous materials, such as steel and aluminum. Tensile coupon, as described above, was mounted on the universal testing machine, facilitated by the grips hooked to the steel-tubes, provided at the end sections beforehand. This is shown in Figure 3.15.

![Direct Tensile Strength Test Setup in UTM](image)

**Figure 3.15: Direct Tensile Strength Test Setup in UTM**

The tensile coupon was provided with two strain-gages on either side of mid-section that were connected to a data acquisition system for continuous load-strain data. It is to be noted that the strain-gages deteriorate as the concrete cracks, thus leading to their dysfunctionality on the post-crack regime. Hence to capture the post-crack response, a two inch extensometer was also hooked up at the mid-section (where the failure was
expected). Extensometer used for the experiment was large-gage length model extensometer. The model is 3543-SR-0300-200T-ST and the gage length used for the purpose was two inches. The extensometer was hooked up with the data acquisition system, ADMAT. Specimen was wrapped with few layers of Cellophane to prevent any possible damage to the extensometer which might be caused by sudden and brittle concrete failure during the test.

Figure 3.16: Extensometer and Strain Gage Positioning

3.3.3.2 Mechanical Properties of Basalt Minibar Reinforced Concrete

Poisson’s ratio, Modulus of elasticity and Split Tensile Strength Tests of Minibar Reinforced concrete were tested. The results are presented in the coming sections.
3.3.3.3 Direct Tensile Strength Test of Basalt Minibar

The specimens were tested on Instron material testing machine, which is a 500lb-capacity, testing machine. Tensile test specimens were mounted on the testing machine in such a way that the anchorage plates would be positioned inside the flat-grip of the testing machine. Extensometer is positioned approximately at the center of the free-length which is large-gage length model extensometer. This is shown in Figure 3.17.

Figure 3.17: Experimental set-up of Uniaxial Tensile Strength Test of Basalt Minibar

Extensometer model was 3543-SR-0300-200T-ST and the gage length used for the purpose is 2”. Extensometer is hooked up with the data acquisition system. Figure 3.17 below shows the positioning of the strain-gage on the tensile test specimen. Specimen was loaded at a constant rate of 10 pounds per second until it is stressed to the
complete rupture of the specimen. Extracted data by the connected data-acquisition system was used for the subsequent analysis of the tested specimens.

3.4 Results and Analysis

This sub-section includes the test results obtained from the tensile test performed on Basalt mini-bars of 0.66 mm diameter and the mechanical properties of minibar reinforced concrete for a minibar dosage of 0.5% by volume. Tensile strength test provides information on the tensile strength of the material, its failure modes, its stress-strain characteristics, rupture strain and the modulus of elasticity. Strain data obtained from the strain-gage was used to establish stress-strain curve of the material which further helped to acquire other pertinent characteristics like modulus of elasticity and rupture strain. One of the important aspects of this test is also the investigation of the failure modes of the basalt mini-bar material. Glass mini-bars were also tested in uniaxial tension for comparison. Those tests provide the qualitative comparison between the failure modes of Basalt mini-bar as compared with other existing FRP materials such as glass. Mechanical characterization of basalt minibar reinforced concrete (MRC), on the other hand, helped understand qualitative and quantitative structural performance of basalt minibar when used as a discrete fiber reinforcement to the concrete.

3.4.1 Tensile Characterization of Basalt Mini-Bar

Summary of uniaxial tensile strength test performed on basalt mini-bar is tabulated in Table 3.1. It comprises the mechanical properties of interest, which are,
uniaxial tensile strength, rupture strain and longitudinal modulus of elasticity of each minibar samples tested in this program.

Table 3.1: Summary of Uniaxial Tensile Strength Test on Basalt Minibar

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Max Load, lb</th>
<th>Max load, kN</th>
<th>Rupture Strain</th>
<th>Max Stress, ksi</th>
<th>Max Stress, MPa</th>
<th>Modulus, ksi</th>
<th>Modulus, GPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>178</td>
<td>0.80</td>
<td>0.017</td>
<td>132</td>
<td>910</td>
<td>5775</td>
<td>40</td>
</tr>
<tr>
<td>2</td>
<td>242</td>
<td>1.07</td>
<td>0.026</td>
<td>180</td>
<td>1241</td>
<td>6785</td>
<td>47</td>
</tr>
<tr>
<td>3</td>
<td>200</td>
<td>0.90</td>
<td>0.022</td>
<td>150</td>
<td>1034</td>
<td>7100</td>
<td>49</td>
</tr>
<tr>
<td>4</td>
<td>221</td>
<td>0.98</td>
<td>0.026</td>
<td>164</td>
<td>1131</td>
<td>5700</td>
<td>39</td>
</tr>
<tr>
<td>Mean</td>
<td>210</td>
<td>0.94</td>
<td>0.023</td>
<td>157</td>
<td>1079</td>
<td>6340</td>
<td>44</td>
</tr>
<tr>
<td>Std dev</td>
<td>28</td>
<td>0.12</td>
<td>0.004</td>
<td>20</td>
<td>141</td>
<td>708</td>
<td>5</td>
</tr>
</tbody>
</table>

It can be observed that basalt minibar is a high strength synthetic fiber with an average tensile strength of 157 ksi. Average longitudinal modulus of elasticity was observed to be 6340 ksi, which is almost a similar value relative to its basalt fiber reinforced polymer (BFRP) counterpart.

Stress-strain curves for basalt minibar obtained from the uniaxial tests are shown in Figure 3.18. It can be observed that the stress-strain characteristics of basalt minibar are similar to their basalt polymer (BFRP) bar counterparts; stress varies linearly with strain till failure and the failure mode is brittle and sudden, without any observable plastic deformation.
For the relative comparison, three glass minibar were also tested. The summary of the uniaxial tensile strength test on glass minibar is shown in Table 3.2.
It can be observed from the tables that uniaxial tensile strength of glass minibar is lesser than that of basalt minibar, however, longitudinal modulus of elasticity for both materials are similar.

A typical stress-strain curve for a glass minibar is shown in Figure 3.19. The stress-strain curve is a typical of a composite uniaxial element, as the stress is linearly varying with strain till rupture without undergoing any plastic deformation.
3.4.2 Mechanical Characterization of Basalt Minibar Reinforced Concrete (MRC)

This sub-section comprises the results obtained from the uniaxial tensile strength test, compressive strength test, splitting tensile strength test, Poisson’s ratio and static modulus of elasticity tests performed on basalt mini-bar reinforced concrete.

The summary of all the tests performed for mechanical characterization of basalt minibar reinforced concrete (MRC) conforming to the pertinent ASTM standards are tabulated in Table 3.3.
Table 3.3: Mechanical Properties of Basalt Minibar Reinforced Concrete (0.5%)

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Compressive Strength (psi)</th>
<th>Modulus of Elasticity for Plain Concrete (psi)</th>
<th>Modulus of Elasticity (psi)</th>
<th>Percentage Increase (%)</th>
<th>Poisson's Ratio</th>
<th>Maximum Load (lb)</th>
<th>Split Tensile Strength (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>5127</td>
<td>4.E+06</td>
<td>4.57E+06</td>
<td>14.25</td>
<td>0.23</td>
<td>60949</td>
<td>539</td>
</tr>
<tr>
<td>2</td>
<td>5127</td>
<td>6.00</td>
<td>4.24E+06</td>
<td>6.00</td>
<td>0.22</td>
<td>64171</td>
<td>567</td>
</tr>
<tr>
<td>3</td>
<td>5127</td>
<td>14.25</td>
<td>4.57E+06</td>
<td>14.25</td>
<td>0.22</td>
<td>62231</td>
<td>550</td>
</tr>
<tr>
<td>Mean</td>
<td>5127</td>
<td>11.50</td>
<td>4.46E+06</td>
<td>11.50</td>
<td>0.22</td>
<td>62450</td>
<td>552</td>
</tr>
</tbody>
</table>

It can be observed that there is not an appreciable increase in split tensile strength of the concrete due to the addition of basalt minibar with 0.5% fiber dosage. The average Poisson’s ratio as determined from the tests was found to be 0.22, which is slightly higher compared to that of plain concrete. This can be attribute to the actuation of crack-spacing mechanism at lower load level which is responsible to arrest micro-cracks, thus enhancing the Poisson’s ratio. It is a well-accepted fact that concrete compressive strength and modulus of elasticity are the mechanical properties which do not vary due to the addition of fibers at low dosages. Modulus of elasticity was found to increase marginally compared to the theoretical value as calculated based on ACI equation for the given compressive strength of concrete.

Direct tensile strength specimen for minibar reinforced concrete (MRC) failed at a smaller than expected, peak load being 4462 lb. The corresponding stress-strain curve is shown in Figure 3.20.
The direct tensile strength test was successfully carried out, with the specimen failing on the neck without any local failures in the vicinity of critical regions. However, the direct tensile strength was found to be lesser than the material’s splitting tensile strength. Since there are no standard test methods for the determination of direct tensile strength of fiber reinforced concrete, the test specimen was designed and prepared at University of Akron. There are many uncertainties that might have arisen due to the lack of historic standards, preferential alignment of minibars at the necking region during the vibration of concrete, eccentricities that might have induced undesirable bending of the specimen and also size-effect. The variable that are required to be accounted for in future testing are minibar dosage, geometry and the size of specimen. However, it can be observed from Figure 3.20 that the tensile-specimen exhibited a tensile hardening effect,
hence, it is capable of reaching higher maximum tensile strain for the same load as compared to its plain concrete counterpart.

Figure 3.21: Minibars on the Tensile Crack-Plane

The tensile efficiency of the fiber-reinforced concrete hinges on the bridge-cracking mechanisms at micro and macro level as the load progresses. Bridging mechanism further depends on the availability of abundant number of fibers on the failure plane to carry the post-cracking tensile load by developing adequate interaction mechanisms with the concrete matrix. As it can be seen in Figure 3.21, there were inadequate number of minibar at the failure plane as the minibar dosage is relatively low.
Figure 3.22: Tensile Test Specimen after the Test

Figure 3.22 shows the tensile test specimen after conducting direct tensile strength test on minbar reinforced concrete specimen. It can be observed from Figure that the failure happened near the center of the gage length.

Figure 3.23 shows a typical compressive stress-strain curve that was obtained during the tests. The strain data was obtained from the strain gages that were attached to the cylinders prepared beforehand.
Figure 3.23: Compressive Stress-Strain Curve for Minibar Reinforced Concrete

From Figure 3.23, it can be observed that minibar reinforced concrete exhibits a strain-softening behavior under compression. Even though the maximum strength under compression will not increase due to the addition of basalt minibar, it is capable of undergoing larger deformation as compared to its plain counterpart, hence, exhibits better deformation properties. As it can be seen from the Figure 3.23, concrete with higher volume fraction is capable of undergoing larger deformation and exhibits strain-hardening behavior under compression. At lower fiber dosage, fiber reinforced concrete exhibits strain-softening behavior under compression and exhibits better ductility and energy absorption capability. This is significant from the structural point of view, as will
be discussion in relation to toughness characterization of basalt minibar reinforced concrete in the following chapters.

3.5 Conclusions and Recommendations

The conclusions from the detailed investigations done regarding the determination of mechanical characteristics of basalt minibar and minibar reinforced concrete are described in detail in this sub-section.

1. The average uniaxial tensile strength of basalt minibar was determined to be 157 ksi. Ultimate rupture strain was determined to be 0.023 with longitudinal modulus of elasticity of 6300 ksi.

2. The average uniaxial tensile strength of contemporary glass minibar was determined to be 138 ksi. Ultimate rupture strain was determined to be 0.019 with longitudinal modulus of elasticity of 6600 ksi.

3. Basalt minibar was found to be stronger than glass minibar in uniaxial tension for the same size. Basalt and glass minibar exhibited almost same tensile modulus of elasticity within the scope of the experiments performed. It was also observed that Basalt minibar is capable of undergoing larger tensile rupture strain than its glass minibar counterpart of the same size.


5. Longitudinal modulus of elasticity of basalt minibar is practically the same as its basalt polymer bar counterpart, the tensile characterization of which was described in
chapter 2. From the uniaxial tensile strength test performed on basalt fiber reinforced polymer (BFRP) bar of size 15 mm to basalt minibar of size 0.66 mm, longitudinal modulus of elasticity was found to be a constant material property of the composite system.

6. The modulus of elasticity for 0.5\% volume fraction minibar reinforced concrete (MRC) was determined to be $4.46 \times 10^6$ psi, which is 11.5\% increment on the ACI specified value for plain concrete of same compressive strength.

7. Average Poisson’s ratio for minibar reinforced concrete was determined to be 0.23. Poisson’s ratio of plain concrete is generally in the range of 0.15 to 0.2. The apparent increment can be attributed to the actuation of crack-spacing mechanism at lower load level due to the presence of minibar. Presence of minibar is expected to be responsible for arresting micro-crack which improves the overall ductility of the material, thus, improving its Poisson’s ratio.

8. Minibar reinforced concrete was observed to exhibit strain-hardening effect under direct uniaxial tension. However, more test with varying minibar dosage and specimen-geometry are recommended. The direct tensile strength was determined to be 295 psi, which is significantly lower than the splitting tensile strength of 552 psi; the test specimen and the test method used need to be revised and revised to obtain more meaningful results.

10. There is an immediate necessity to have a standard test method for the direct tension test of fiber reinforced concrete (FRC), which can account for the effect of fiber
dosage, specimen-geometry (size-effect) and any possible correlation between direct
tensile strength, splitting tensile strength and flexural tensile strength.

11. Minibar reinforced concrete was observed to exhibit strain-softening effect
under compression, hence better deformation characteristics compared to its plain
counterpart. However, more test with varying minibar dosage are recommended.
CHAPTER IV
TOUGHNESS CHARACTERIZATION OF MINI-BAR REINFORCED CONCRETE
BEAMS AND ITS APPLICATIONS

This section provides a detailed description of the literature review, lab procedures and experimental program that was carried out for the toughness characterization of minibar reinforced concrete (MRC). The primary objective of this part of research-program is the identification of flexural tensile strength and post-cracking flexural tensile strength of minibar reinforced concrete, hereafter called as average residual strength (ARS). ARS properties were studied for varying minibar dosage to understand the correlation between the above mentioned mechanical properties and minibar dosage. These properties provide an insight into the toughness characteristics, which is the measure of its energy absorption capacity. If strength characterization can be considered very important for quantification of ultimate limit states, toughness characterization can be similarly considered very significant for the quantification of ultimate, as well as serviceability limit states. Toughness characterization is also very important since it deals with the deformation characteristics of the material, which can be instrumental to study the behavior of structures under dynamic loadings, such as an earthquake. The section includes the literature review, description of the materials, description of the lab-procedures and experimental program to meet the objective, finally followed by results and analysis with conclusions and recommendations.
4.1 Literature Review

Literature review section covers a comprehensive study done in regard to the applicability of fiber reinforced concrete in various civil engineering structural applications, especially where toughness and deformation capability are of primary importance. Since toughness characteristics fundamentally relate to energy absorption capacity which is further related to deformation properties of the material, the literature review pertains to the study done in relation to its application primarily under seismic excitation.

4.1.1 Introduction

Owing to its various advantages, reinforced concrete is one of the most widely used materials in the construction industry. The basic premise of steel reinforced concrete is that the steel reinforcement is provided to reinforce the concrete matrix to enhance its inherent weakness in tension. Composite action is facilitated due to the bond between concrete and steel. Applications of reinforced concrete span a broad spectrum of engineering applications such as bridges, dams, tall buildings to low-rise residential construction. The widespread popularity of urban reinforced concrete structures for residential purposes especially in developing countries, can be attributed to low cost and ease of construction associated with it [20]. For example, the unit cost associated with reinforced concrete structures is approximately US $100 per square meter in India, US $250 per square meter in Turkey and US $500 per square meter in Italy [20]. This explains the obvious choice of reinforced concrete for residential buildings in many countries. However, the widespread use of reinforced concrete for residential buildings
also has some associated risk particularly in highly active seismic areas of the Far East and South East Asian countries. Degree of severity of an earthquake can be more devastating in these countries than sparsely populated countries considering the larger concentration of populations. The massive loss of human lives and properties in Pakistan, India and Turkey due to earthquakes in the last decade are compelling examples of the criticality of failures of reinforced concrete structures. Seriousness and the aftermath of such collapses are easy to comprehend (e.g., see Figure 4.1). Therefore, researchers worldwide are finding ways to minimize such failures and the consequential loss of lives. One such research effort is the use of fiber reinforced concrete for improving the overall energy absorption capacity of the structure (quantified by the toughness of the materials), hence enhancing their overall seismic performance.

4.1.2 Limitations of Reinforced Concrete Frame Structures

A rigid jointed cast-in-place building frame resists lateral and vertical loads by different deformation mechanisms. During an earthquake, the frame will be subjected to cyclic loading which creates reversal of various loading effects such as moments, shear forces, and axial loads. The response of the structure during such loading is qualitatively different from its response under monotonic static loading. Figure 4.1 depicts the failure of Nuevo Leon, a fifteen story reinforced concrete structure during the Mexico City earthquake, September 19, 1985. As seen in the Figure, the brittleness of such failures is a very serious problem.
The elements in reinforced concrete frame structures that are important under seismic loading are the joints of columns and beams or floor slabs, structural walls, spliced regions, shear walls, and coupling beams [21]. In general, special care should be taken for cyclic loadings wherever structural discontinuity exists. Critical regions for such cases can also be the region where there is a possibility of plastic hinge formation [22].

The load reversal during cyclic loadings can result in bond and stiffness degradation, thus affecting the overall structural integrity of a member. Under cyclic excitation, critical regions of a reinforced concrete structure are subjected to severe stiffness degradation [23]. Stiffness degradation results in an increase in the flexibility and the period of vibration of the structure, reduction in energy dissipation capacity and unpredictable redistribution of loading and resisting mechanisms. All of these parameters affect the induced dynamic forces. Stiffness degradation also affects the overall structural response [23]. For example, structural joints subjected to cyclic loadings
during an earthquake can experience multiple cracks in critical zones [24, 25] which in turn cause stiffness degradation. The nature and magnitude of energy dissipation are altered leading to a modification of the overall structural response.

### 4.1.2.1 Reinforced Concrete Coupled Wall Systems under Seismic Loading

Another structural system that has been thoroughly investigated in past is the coupled wall system. Coupled wall systems constitute structural walls coupled by coupling beams to resist lateral loads generated during an earthquake in high-rise multistory buildings [26]. The efficacy of such systems depends on its deformation capacity and strength. Coupling beams and structural walls are subjected to high rotation and shear demands during inelastic displacement reversals which are limited by concrete crushing strain limits [27]. Strong wall and weak beam systems are used such that the whole system will not behave as a single cantilever. Coupling beams are designed to be strong enough to withstand the external deformations and must be sufficiently equipped with mechanisms to dissipate energy after the yielding of walls [26]. Provision of confinement reinforcement in the critical regions of coupling beams and structural walls is well established as the current state-of-the-art methodology to tackle such demands. For example, reinforcement confinement can be provided in the boundary region of structural walls to improve ductility and the rotation capacity during cyclic load reversals [28].

Response of a reinforced concrete structure to cyclic loading is a complex subject which may include the synergism of various parameters involved, requiring an optimum balance between strength and deformation capacity. The response is a function of
member geometry, loading history and reinforcement details [29]. Undesirable increase in reinforcement to guarantee sufficient strength of the member can jeopardize its deformation capacity which is pivotal in energy dissipation during earthquakes. This also leads to the congestion of steel reinforcement in critical joints thus severely affecting constructability and ductility. Such pseudo-elastic joints can ultimately form undesirable collapse mechanisms due to strength and stiffness degradation [20]. Figure 4.2 depicts a typical beam-column joint failure during an earthquake due to inadequate confinement, demonstrating the criticality of appropriate detailing.

![Figure 4.2. A Beam-Column Joint Failure](Reprinted with Permission from EERI: Photographer - J. Zhao)

An optimum balance between the strength and deformation capacity should be sought with desirable collapse mechanisms (such as hinge formation in beams) to guarantee satisfactory seismic performance [29]. Reinforced concrete beams need to
exhibit good cyclic response with a decrease in maximum shear stress and increase in transverse steel capacity.

4.1.3 Response of Fiber Reinforced Concrete Structures under Cyclic Loading

Fiber is primarily used in concrete to inhibit tensile crack growth thus significantly increasing the post-crack tensile strength of the concrete. Addition of fiber to concrete significantly alters the mechanical properties of the member such as increase in toughness (energy absorption), ductility, tensile strength and flexural strength [30].

Figure 4.3 and Figure 4.4 show the response of fiber reinforced concrete beams tested to flexure conforming to ASTM C1399 [31] and ASTM C1609 [32]. The tests are designed to find the average residual strength (ARS) and the flexural performance of fiber reinforced concrete. These properties are measures of the post-cracking tensile strength, and overall flexural response of fiber reinforced concrete giving a quantitative indirect measure of toughness, which is the amount of energy absorbed during flexural deformation (also the area under the load-deformations curve in case of ASTM C1609 test).
Figure 4.3. Typical Load-Deflection Curves for Initial Loading and Reloading [12]

Figure 4.4. Typical Load-Deflection Curve for Fiber-reinforced Concrete Beams [13]


The applicability of fiber reinforced concrete to improve the seismic response of a structure is rapidly attracting interest. Seismic performance of a structure is a function of energy dissipation capacity of the structure. The critical regions such as beam-column joint or structural wall coupling system must be adequately equipped for better energy absorption. Fiber reinforced concrete in the recent times has been increasingly recognized to be helpful for that purpose. Figure 4.5 shows the critical regions in a reinforced
concrete frame that are subjected to high ductility and rotational demand during seismic excitation [33]. The critical regions in reinforced concrete frame where fiber reinforced concrete may be used to yield highly improved structural behavior are shown in the figure. Recent research on the topic has revealed that fiber reinforced concrete confinement is one of the efficient ways to enhance the seismic response of critical regions such as beam-column joints, structural walls, and shear walls [22].

![Critical regions in a RC frame under Seismic Excitation](image)

Figure 4.5: Critical regions in a RC frame under Seismic Excitation [33]

4.1.3.1 Bond Degradation of Critical Regions under Seismic Excitation and Its Possible Mitigation

During earthquakes, when the critical regions are subjected to cyclic loading, they are found to be prone to bond degradation. Seismic response of critical regions is highly dependent on the bond interaction mechanisms at the joint [21]. Functional and structural significance of fiber reinforced concrete for such cases cannot be ignored. The presence of confinement in the form of fiber is instrumental in reducing crack-widths, thereby
increasing the post-cracking tensile strength resulting in larger post-splitting bond strength. Reduction in crack width and subsequent improvement in post-splitting tensile and bond strength lead to better energy dissipating mechanisms in the critical regions of interest than the corresponding conventional reinforced concrete. For example, Haraji proposed a local bond-slip constitutive model for steel fiber reinforced concrete with steel longitudinal reinforcement under cyclic loading which is useful for analytical research on fiber reinforced concrete including finite element modeling [21].

Even though bond failure of steel bars and concrete may be either in pull-out or splitting mode, in general cases, splitting mode is more dominant [22]. Since pull-out failure takes place under significant amount of confinement, such situation is not generally encountered in the elements like beams under flexure (which in most cases will have smaller concrete cover). Haraji observed that for the unconfined specimens, the spliced region (one of the critical regions from seismic standpoint) underwent severe bond degradation after the splitting of concrete resulting in significant loss in strength and stiffness. With the addition of steel fibers in the spliced zones, the cyclic local bond-slip response is significantly improved along with the overall ductility. Bond failure for high strength concrete is more catastrophic compared to normal strength concrete, however, the gain in bond strength in fiber reinforced concrete is achieved faster compared to normal strength concrete. The following equation for increased bond strength under the confinement of transverse steel reinforcement and steel fiber was proposed [34]:

$$\frac{U_c}{\sqrt{f'_c}} = 0.23 + 0.46 \frac{c}{d_b} + 14.1 \frac{d_b}{k_s} + \sqrt{f'_c} \left( \frac{A_{tr}f_{yt}}{41.65n_s d_b} + 0.25 \frac{c}{d_b} \frac{V_{rL_f}}{d_f} \right)$$

(18)
Where,

\[ U_c = \text{Total average bond stress at bond failure} \]

\[ f_{c'} = \text{Compressive strength of concrete} \]

\[ c = \text{minimum concrete cover} \]

\[ d_b = \text{diameter of dowel, spliced or starter bars} \]

\[ L_s = \text{lap splice length} \]

\[ A_{tr} = \text{area of transverse steel ties} \]

\[ f_{yt} = \text{yield strength of transverse steel reinforcement} \]

\[ s_t = \text{spacing of transverse steel ties} \]

\[ n_s = \text{number of doweled or spliced bars in tension} \]

\[ V_f = \text{volume fraction of steel fibers} \]

\[ L_f = \text{fiber length} \]

\[ d_f = \text{fiber diameter} \]

Bond deterioration in the critical regions (such as splices and joints) is the main reason for the weak energy dissipation mechanisms. From Equation 18, the increased contribution to the bond strength due to fiber is remarkable. This increased bond strength in symbiosis with better crack resisting mechanisms and increased ductility render the critical regions with better energy dissipating mechanisms. By using fiber for confinement, ductility and energy dissipation mechanisms of concrete can be significantly enhanced [30].
4.1.3.2 Fiber Reinforced Concrete and Its Response under Flexural Load

Haraji [35] also performed flexural tests on confined and unconfined concrete beams and found that there is a significant reduction in bond deterioration and improvement of flexural capacity with the fiber confinement. For the deflection level at which all unconfined members lose 100% of their load carrying capacity due to cyclic load induced bond degradation, the confined members showed significant load carrying capacity depending on the volume fraction of the fiber. With the increase in cyclic load beyond bond splitting, the fiber confined concrete members exhibited higher energy absorption capacity. The increase in energy absorption capacity is proportional to the increase in volume fraction, which results in increase in the amount of confinement, and concrete compressive strength.

Confined concrete joints with steel fiber exhibit higher moment strength and improved ductility [24]. The transverse steel hoop reinforcement in such joints can be replaced by fiber reinforcement. For a given fiber reinforcement index, increasing the bar spacing or the clear cover results in the availability of large number of fibers crossing the cracks that virtually work as transverse shear reinforcement [30]. It is imperative to add shear confinement in the bending zone of high-strength concrete flexural members to achieve the required strength and ductility [36]. Confinement in the case of high strength concrete can also be achieved by using fiber specifically in the critical regions (such as beam-column joints and the region of plastic hinge formation) where the placement of shear hoops can create serious constructional challenges.
4.1.3.3 Longitudinal Elongation of Reinforcing Bars under Cyclic Excitation

Some interesting observations can also be made from the works of Eom [37] who observed that reinforced concrete members under cyclic moments, when subjected to low or moderate compression force, show significant longitudinal elongation, which was not present during the conventional monotonic loading. Deformation and force redistribution caused by the elongation was suggested to result in the formation of a plastic hinge in the columns rather than in the beams. Additional moment and shear force further cause substantial local damage to the columns. The longitudinal deformation caused by repeated cyclic loading results in untimely crushing of concrete in webs thus affecting the overall energy dissipating mechanisms. Crack widths due to flexure also vary with the longitudinal elongation [37]. Fiber confinement in such cases can be a pertinent solution. Since the addition of fiber significantly alters the nature of concrete crushing in webs (making it more ductile) and at the same time bridges the flexural cracks. Addition of fiber confinement in the critical regions of the structure can substantially improve the overall energy absorption capacity of the structure (quantified by its toughness) and consequently their seismic characteristics in such cases.

4.1.3.4 Reinforced Concrete Columns under Biaxial Bending

When the columns under biaxial bending are subjected to cyclic loading, the stiffness deterioration is accelerated as compared to the uniaxial case [34]. Despite the frame being designed based on weak-beam-strong-column approach, there is a likelihood of the formation of plastic hinges in columns due to inelastic biaxial action. With the addition of fibers in the critical regions in columns, the stiffness deterioration reduces
with a considerable increase in energy dissipation capabilities. The overall structural response is more ductile.

4.1.4 Fiber Reinforced Concrete as a Mitigation Measure

From the insight developed into the various failure modes that are common for reinforced concrete frame structures under different forms of loading, the use of fiber reinforced concrete is considered one of the viable mitigation measures to introduce toughness in the structure. Fiber confinement tends to reduce bond and stiffness degradation, increase ductility, improve energy absorption, and provide multiple dissipation mechanisms. These features of fiber reinforced concrete can be judiciously utilized wherever there is a possibility of brittle and catastrophic concrete failure. Whether the failure mode is bond, shear or flexure, the failure mechanisms can be significantly softened with better energy absorption and deformation capabilities by providing fiber confinement.

In the case of coupled wall systems, the coupling beams should be strong enough to withstand the external deformations and must be sufficiently equipped with mechanisms to dissipate energy after the yielding of walls. Application of fiber confinement seems to be a viable option to serve this particular purpose as well.

4.1.4.1 Fiber Reinforced Concrete to Enhance Shear Behavior

Another factor that is critical during earthquake resistant designs is the mobilization of reinforced concrete joints and other critical regions to higher magnitude of shear forces. Earthquakes of large magnitudes give rise to large shear forces at the
beam ends and within the beam-column joints. The bond deterioration along with the shear effects can account for the slippage of reinforcements anchored in beam-column joints [23]. Therefore, the consequent failure of such joints can be viewed as a compound effect of intrinsically brittle natured bond and shear failure mechanisms [38].

Under the influence of seismic loading, beam-column joints are subjected to vertical and horizontal shear forces of magnitude much greater than the adjacent flexural members. The flow of loads may not follow the desired load path if the joint is unable to sustain the imposed resultant shear forces [39]. This is the main reason for the increased transverse reinforcement (reduced spacing) within the joint. However, it will lead to congestion of the joint giving rise to poor detailing effects. The poor detailing is further magnified by the excess seismic energy that is transferred by the adjoining flexural members during the event of an earthquake [38]. Overall loading history ultimately leads the joint to more catastrophic failure.

Shear failures are inherently brittle and catastrophic, and are recommended to be avoided in reinforced concrete structures in seismic zones. From the documentation of several destructive earthquakes of last two decades that happened in Greece, Turkey, and Taiwan (1999), India (2001) and Algeria (2003), shear failure and concrete crushing failure are seen to be some of the dominant collapse mechanisms [20]. Figure 4.6 depicts a reinforced concrete beam with shear failure during an earthquake in Wenchuan (China) in 2008.
With these compelling statistics, it is important to look for options that will counteract these dominant structural deficiencies associated with reinforced concrete subjected to seismic loading. Fiber reinforced concrete appears to be one of the viable countermeasures to minimize such deficiencies.

4.1.4.2 Possible Alteration of Failure Mode with the Addition of Fiber to Reinforced Concrete

Addition of fibers to concrete matrix is very effective in changing the failure mode from brittle to ductile which is generally associated with diagonal shear failure of concrete beams [40]. The failure mode changes from catastrophic to ductile with the
increase in fiber content [24]. Moreover, tests on FRC beams as a substitute for shear stirrups have been reported by some researchers [41]. Plizzari [34] found that fiber reinforcement is a viable replacement for shear reinforcement when the spatial congestion of joints is the primary concern. This provides an answer to the problem of congestion of reinforced concrete joints where the shear hoops can possibly be replaced (at least, partially) by fiber reinforcement. Steel fiber reinforced concrete has been suggested with larger stirrup spacing within reinforced concrete joints [24]. Steel fiber reinforced concrete joints exhibit higher shear strength than the conventional joints designed to conform to the current seismic detailing. Fiber-confined joints were found to dissipate 85% of the energy of the detailed joint whereas undetailed joints dissipated 70% [24]. Others concluded that the addition of steel fiber confinement imparts improved ductility and dimensional stability to the joint accounting for its reduced rate of stiffness degradation [39].

With proper mix design and proper fiber selection, the ductility and shear strength of reinforced concrete beams can be considerably enhanced [42]. The addition of fibers can mitigate excessive diagonal cracking and localization of the tensile crack damage [43]. Addition of small fraction of fibers significantly enhances the structural behavior of concrete members without shear reinforcement and ensuring that the material has adequate toughness. Collapse mechanisms can be changed from shear to flexure. Use of fibers is particularly effective in the case of high strength concrete which is considerably brittle without minimum transverse reinforcement [44]. The increased shear strength of fiber reinforced concrete can be attributed to post-cracking tensile strength of the concrete. Fibers across the cracked surface will carry larger stress, called crack-bridging.
stress, owing to its better tensile properties compared to concrete. The vertical component of the post-cracking tensile load provides the additional shear strength [43]. Another way the fibers contribute to the overall shear robustness of the structure is by enhancing the dowel action, which is one of the components of shear strength. The dowel action is increased by the addition of fibers in concrete which is attributed to the increased post-splitting tensile strength [24].

In summary, to resist the higher magnitude of shear forces during an earthquake, fiber reinforced concrete is a viable option. To make it cost effective, fiber reinforced concrete can be selectively used in only the critical regions of reinforced concrete structures located in seismic zones. Shear failure mode which is inherently catastrophic and brittle can be changed to a ductile failure mode with the addition of fiber. Apparent increase in bond strength can also contribute for the favorable symbiosis with shear resistance.

Recognizing the contribution of fiber in resisting shear in a steel reinforced concrete beam, “ACI 318: Building Code Requirements for Structural Concrete” recently introduced a new provision (ACI 318 - Section 5.6.6) that accepts steel fiber-reinforced concrete in lieu of minimum shear reinforcement in steel reinforced flexural members subject to the following conditions [45]:

(a) Dosage of deformed steel fibers per cubic yard of concrete is at least 100 lb (60 kg/m³)

(b) Residual strength based on ASTM C1609 [13] at midspan deflection of span over 300 is greater than or equal to 90% of measured first peak strength obtained from flexural test or 90% of ACI defined modulus of rupture (flexural tensile strength).
(c) Residual strength based on ASTM C1609 [13] at midspan deflection of span over 150 is greater than or equal to 75% of measured first peak strength obtained from flexural test or 75% of ACI defined modulus of rupture which is the flexural tensile strength [26].

4.1.4.3 Fiber Reinforced Concrete to Improve Punching Shear Strength

Punching shear failure is also a brittle mode of failure. Therefore, fiber reinforced concrete can be used to address this issue as well. Naaman [46] found that the punching shear resistance, energy absorption capacity and the resistance to spalling of HPFRCC (high performance fiber-reinforced cementitious composites) slabs with two bottom layers of reinforcing bars were significantly better than the specimen with four layers of reinforcing bars and regular concrete. Punching shear failure is a localized failure that happens at the slab and column connection and is a brittle and catastrophic failure generally resulting in a smaller failure load than the corresponding flexural failure load of a two way slab.

4.1.4.3.1 Punching Shear Failure in the Context of Reinforced Concrete Structure

Punching shear is prominent in flat slabs. The slab thickness for flat slabs is normally governed by punching shear and serviceability requirements [47]. Punching shear failure in most cases occurs in a brittle manner with the failure cone extending from the column edge to the tensile face of the slab. It physically develops from the circumferential cracks in the slab [48]. A punching shear failure can be characterized by vertical deflection and a truncated conic failure surface [49]. The mean angle of the
failure cone is about 30 degrees for steel reinforced concrete beams. This angle however, decreases for fiber reinforced concrete slabs. With the addition of fiber, the diagonal cracking can be delayed, allowing the structure to undergo large deformation before failure, and changing the failure mode from brittle to more ductile mode [50].

4.1.4.3.2 Punching Shear Failure Response under Seismic Excitation

Punching shear phenomenon is a serious issue while considering the element response against seismic excitation. There has not been significant research on the punching shear strength of slabs when subjected to cyclic load reversals. Most of the earlier work on punching shear is confined to monotonic static loading. One of the major problems associated with flat slabs is well recognized to be their susceptibility against earthquake loadings [49]. The results from monotonic tests are extrapolated to study the dynamic response of flat slabs under cyclic excitation to bridge the gap created by the lack of research in the area.

During the investigation performed by Theodorakopoulos and Swamy [48], test slab specimens were reinforced with concentric rings placed at the boundary of the slab element. Such a layout mitigates the formation of circumferential or circular cracks in the critical region thus favoring the development of only radial cracks. A slab with similar configuration but with an additional ring at the critical region was also tested in that study. The punching shear strength in that case was increased significantly with the mitigation of critical shear cracks. Instead of concentric steel rings, fiber reinforced concrete can be used in the circumferential region to arrest the propagation of critical
shear cracks. Application of fiber reinforced concrete instead of steel rings will promote workability, reduce spatial congestion and the effects of poor detailing.

Slab thickness also plays an important role to define the punching shear strength of slabs. Even with a lower reinforcement ratio, thicker slabs have smaller rotation capacity with a tendency to fail in a brittle manner relative to their thinner counterparts. Muttoni [47] found that the mode of failure is brittle for thinner slabs with large reinforcement ratios. Fiber confinement in such cases can be identified as an excellent countermeasure. Shear strength of reinforced concrete slabs can be increased by the addition of fiber, and thus the size effect associated with punching shear can be partially addressed with fiber confinement.

4.1.4.3.3 Fiber Confinement to Address Punching Shear Failure

Regan and Braestrup [51] report that the dowel action increases the punching shear strength of an orthogonally reinforced concrete slab by 34%. Fiber reinforcement has been found to increase the effective dowel action. The column size was also found to be influential on the failure load by increasing the critical effective shear perimeter. Teng et al. [52] observed that high flexural stress gradients exist around the shorter side of the column sections thus signifying that the critical location for punching shear should be around the shorter side of the rectangular column. The flexural stresses close to the column face are higher than those occurring farther away. Fiber confinement can be judiciously used in the critical regions to increase the effective shear perimeter for such cases by forcing the critical perimeter to move further away from the faces of the column.
An increase of 30% in the punching resistance of slab-column connection as well as improved ductility of the connection was recorded by Alexander and Simmonds [53] due to the addition of corrugated steel fibers. Namaan et al. [46] reported a 40% increase in ultimate punching strength by adding 2% hooked steel fibers by volume. Increase in shear-capacity is indicated by the shallower angle of the punching shear surface as compared to the corresponding plain concrete. Addition of fiber also helps to provide the transition of failure mode from brittle to ductile. By providing fiber by approximately 2%, the punching shear failure of flat slabs changed from brittle to ductile [46]. From load-displacement plots, Naaman showed that the maximum load reached was significantly higher in the case of HPFRCC slabs compared to the control steel reinforced conventional slabs. The peak load is much higher with increased energy absorption capacity such that a ductile failure is ensued.

Alexander and Simmonds [53] conducted experimental work on punching shear strength of concrete slab-column joints containing fiber reinforcement. For the specimens without fiber confinement, the punching failure surface closely follows the contour of the column geometry during punching whereas in the case of fiber reinforced specimen, there was no distinguishable pattern. With the addition of fiber, stress concentrations in the vicinity of column are reduced thus ensuring more ductile and uniform failure. Fibers enhance the force gradient that can be developed in the reinforcing bar, thus effectively improving the shear transfer strength.
4.1.5 ACI 318-11 Ductile Detailing Requirements for Earthquake Resistant Structures

ACI 318-11 specifications [45] on the detailing requirements for earthquake resistant structures are based on the philosophy of providing adequate confinement to the critical regions which allows the structure to sustain repetitive reversal of loading with the response going into the inelastic range without undergoing substantial degradation of strength and stiffness. Confinement provided to critical regions should be sufficient to guarantee the structural integrity of the unit when the seismic loading tends to force the structure into inelastic response.

To achieve sufficient ductility of the critical regions during seismic excitation, ACI 318-11 has recommended important detailing requirements. The basis for some of these requirements is described in detail with reference to regional needs by Patnaik and Jain [54 and 55]. For example, Figure 4.7 schematically depicts the detailing requirements as specified by ACI 318-11 for intermediate moment frames which form a part of seismic-load resisting system. The objective of intermediate moment frames is to reduce the risk of failure in shear in beams and columns. This is achieved through detailing requirements to provide hoop reinforcement at the specified close spacing over a certain length at beam ends and column ends. For beams, the first hoop shall be located not more than 2" (50 mm) from the face of the supporting member. At both ends of a beam, closely spaced hoops are required to be provided for a distance not less than twice the overall depth (h) of the beam. Spacing shall not exceed the smallest of following:

1. $d/4$ (which is one fourth of the effective depth)
2. Eight times the diameter of the smallest longitudinal bar diameter being confined
3. $24 \times$ the diameter of hoop bar
4. 12 inches

Also, stirrups are to be provided at spacing not more than half the effective depth of the section throughout the length of the member.

Similarly, hoops are required to be provided in the column on both sides of the joint, the first hoop being located at $s_0/2$ where $s_0$ is the spacing of hoops, not exceeding the minimum of the following:

1. Eight times the diameter of the smallest longitudinal bar diameter being confined
2. 24 times the diameter of hoop bar
3. One half of the smallest cross-sectional dimension
4. 12 inches

The hoops are required to be distributed over the length ($l_0$) at a spacing which is minimum of the following:

1. $\frac{1}{6}$th of the clear span of the column
2. Maximum cross-sectional dimension of the cross-section
3. 18 inches

Based on these additional ductility detailing requirements in the vicinity of the joints, along with the joint detailing itself, the constructability of the structure can be seriously compromised. Due to the spatial congestion and its consequent effects on the constructability of structure, the possibility of practicing maximum reinforcement ratios when arduous strength requirements are to be met can also be alleviated if steel fiber can be used in conjunction with reduced hoop reinforcement. From a construction point of view, reduced congestion of shear reinforcement helps in better flow of wet concrete around the steel reinforcement during the placement and better compaction thereby
resulting in a better performing joint compared to an ill-compacted joint. The reduced congestion also helps in increased speed of construction which would normally result in cost savings.

Figure 4.7: Detailing Requirements in the Vicinity of a Joint in Intermediate Moment Frames

ACI 318-11 identifies transverse reinforcement to be a type of confinement that can provide lateral support for the longitudinal bars in the regions where yielding expectancy is high. Figure 4.8 shows one typical arrangement of overlapping closed hoops for seismic resistant design.
ACI 318-11 incorporates many detailing constraints that will assure adequate confinement of the longitudinal steel reinforcement in structural members to lead to ductile response under seismic loading. In the case of coupling beams used as a primary part of the lateral load resisting system, ACI 318-11 recommends additional provisions for diagonal reinforcement to provide the required confinement as shown in Figure 4.9.
Figure 4.9. Confinement Requirements of RC and FRC Structural Walls [37]

The increased quantity of hoop reinforcement per unit volume of structure assures ductile response when the structure is forced into nonlinear region under seismic loading. However, at the same time, it seriously affects the constructability and the cost of the
structure thus jeopardizing the overall effectiveness of confinement it is supposed to provide. Fiber reinforced concrete can be a beneficial substitute to hoop reinforcement while considering the overall solution to the problem of congestion [56] and the overall cost of the structure.

Facilitating of strain-hardening in high-performance fiber reinforced concrete coupling beam has been actualized for the design of a high-rise core-wall structure in Seattle [57]. Construction was successfully implemented replacing the detailing requirements of ACI 318-08 (the 2008 version of ACI 318-11) with simplified detailing pertaining to fiber reinforced concrete as shown in Figure 4.9. Efficacy of the diagonal reinforcement is adversely affected when the coupling beams are relatively slender giving rise to shallow angle between diagonal bars and beam longitudinal axis (less than 15 degrees). Fiber reinforced concrete was used to meet the performance standard required under such loading conditions. Constructability challenges related to spatial congestion and diagonal bar placement were minimized by using fiber reinforced concrete in this project [57]. The freedom to bend the diagonal bars and extend them horizontally into the shear walls (under the fiber reinforced concrete detailing) was reported by the contractors to be a remarkable improvement in the constructability of the coupling beams in this project. Application of high performance fiber reinforced concrete allowed approximately 70% reduction in diagonal reinforcement compared to what was required based on ACI 318-08
4.1.5.1 Potential Application of Fractal Analysis for the Toughness Characterization of Minibar Reinforced Concrete

It has been observed that most of the objects we come across in nature can be dealt with Euclidian geometry which relies on the fundamental assumptions of continuity, linearity and smoothness of space. The underlying physics and mathematics of any engineering problem is generally governed by this basic assumption that any object can be spatially characterized using the basic geometrical shapes such as lines and planes. These regular geometrical objects have an integer valued topological dimensions, for instance, 0, 1 and 3 for point, line, surface and a volume respectively [58]. However, this may not hold true for every natural objects and phenomenon. There are various objects and their properties in nature which cannot be usefully accounted for using the conventional Euclidian geometry. If we consider the Koch flake as shown below, it is not hard to surmise that it cannot be totally characterized as a one-dimensional curvilinear line.

![Figure 4.10: A Euclidian Curvilinear Geometrical Object](image)
Figure 4.10 shows a typical Euclidian curve which spatial dimension is one. Figure 4.11 shows a typical fractal object which cannot be dealt with regular Euclidian categories, hence needs some other methods of investigations. A fractal object is characterized by self-similarity such that they depict scale-invariance. No matter what level of magnification we use, a Von Koch curve will look exactly the same for every scale. The concept of fractal for the geometrical characterization of irregular surface was firstly introduced by Mandelbrot [59]. Fractals are also classified as self-similar or self-affine. Self-similar fractals are scale-invariant under any levels of magnification in all directions whereas self-affine fractals exhibit statistical invariance only when scaled by different factors in different directions [59]. Comparing Figures 4.10 and 4.11, it can be stated that here are many natural objects and phenomenon whose properties can’t be judiciously accounted using the regular geometrical objects, such as shown in Figure 4.10. A typical example is the cracked concrete surface. It has been observed by various scientists that a cracked concrete surface tends to be fractal (as shown in Figure 4.11) such that a totally different approach is eminent for their complete characterization. Concrete fracture surface was found to exhibit fractal characteristics over the scale considered [59]. It has been observed for a considerable amount of time that many processes of optimum significance take place at the vicinity of disorderly interfaces (for instance, concrete-crack) which can’t be sufficiently dealt within the conventional
Euclidian paradigm [58]. Fractal geometry allows us to describe and quantify the irregularities associated with natural landscapes and fracture surfaces [59].

4.1.5.2 Fractal Dimension

Fractal dimension is an index which characterizes the irregularity and the roughness of a geometrical object and is the Euclidian counterpart of the real world. Fractal geometry characterizes the scaling structures of irregular curves, planes and surfaces by a fractional number, called as Fractal dimension [60]. For the particular examples as shown in figure 1 and 2, object shown in figure 1 has a Euclidian dimension of 1 whereas object shown in figure 2 has the fractal dimension (Df) between 1 and 2, and is fractional. Concept of fractional dimension can be regarded to be overly counter-intuitive but its physical significance cannot be underestimated. Fractal dimension of a curve varies between its Euclidian extremums of 1 and 2 and the fractal dimension of a surface varies between 2 and 3. Fractal dimension will increase as the degree of irregularity increases. Hence for a completely linear curve (as shown in Figure 4.10), the fractal dimension coincides with its Euclidian dimension, which is unity, and it varies between 1 and 2 as the degree of irregularity increases. The concept can be physically realized by observing that a curve with a fractal dimension between 1 and 2 covers more space than a line, but less space than a plane. Usually, the fractal dimension of an object exceeds its topological dimension [58]. A rough surface, such as he cracked concrete face, can be given a number between two ( topological dimension ) and three (Euclidian dimension) and is a mathematical measure of how the surface fills the space it occupies [59].
Fracture mechanics deals with the phenomenon of concrete cracking based on the assumption that the cracks are smooth and straight, however, it is a well known fact that cracks in concrete are highly irregular, hence amenable to fractal analysis [61]. It has been found that surface of hydrated cement paste is highly fractal [58] and it varies with the respective water-cement ratios. Janusz et al. [62] performed the fractal analysis of the macro-pores in concrete and concluded that the fractal dimension if the macro-pores relates with some of the physical and mechanical properties of concrete [62]. Concrete is a multi-phase system consisting of coarse aggregate, fine aggregate, cement and water. It is, therefore, very likely to assume that the resulting geometry of cracks on concrete surface tends to be fractal as the hydrated cement mortar and the aggregate itself exhibit fractal properties. Issa et al. investigated the relationship between fractal dimension of the concrete cracked surface and the maximum aggregate size. It was observed that the fractal dimension of the cracked surface is directly proportional to maximum aggregate size. The observations were well justified as the crack-bridging mechanisms were endowed with more degrees of freedom as the aggregate size increases. With the larger sized aggregates, the response of the material tends to be tougher thus initializing more tortuous path for the crack to propagate [58]. The roughness and turtuosity of the cracked surface is a mathematical measure of the toughness characteristics of the aggregates and the mortar at the interfaces [59].

With the addition of fiber in concrete, this crack-dynamics can be further modified. The crack-bridging mechanisms that are actuated in micro and macro level with the addition of fiber can significantly enhance the toughness of the material, which may be visually depicted with the more tortuous and fractal cracked surface geometry.
Cracks in concrete propagates though kinking along the interfaces between aggregate and cement paste, kinking into the cement paste and finally into the aggregate [61]. Considering the fact that aggregates in themselves are fractal object, the crack path tends to be highly irregular and fractal. It is a good subject to consider what will be the probable changes in the crack-pattern and its geometry with the addition of fiber in the concrete. In the case of high strength concrete, which generally depicts brittle failure, cracks propagates directly through the aggregate. In the case of fiber reinforced concrete which can exhibit softening behavior, the crack propagates through the inter-phase gaps and kinks (between aggregate and fiber, between fiber and fiber and so on), hence is more liable to be geometrically fractal [58].

Figure 4.12: Relationship between the Fractal Dimension of the Crack and Maximum Aggregate Size [58]
4.1.5.3 Significance of Fractal Analysis in FRC

Benoit Mandelbrot was the mathematician who developed the concept of fractal Analysis. The concept of fractal finds in immense significance in material science from the observation that fractal dimension of the cracked line or plane is the physical manifestation of the intrinsic energy dissipation mechanisms, hence can be correlated with the mechanical properties of the material itself. One of the key mechanical properties that have been studied to be correlated with fractal dimension of the cracked surface is the toughness property of the material. Benoit Mandelbrot (1984) was the first scientist to observe that toughness of the material is correlated with the fractal dimension of the cracked surface. It was observed that increased toughness of the material is depicted as the decreasing trend in the fractal dimension of the cracked surface [63]. These types of observations can be very valuable if it can be extended for the study of FRC. The increasing popularity of FRC can be attributed to increased energy absorption, leading to improved toughness characteristics, facilitated by the addition of fibers. If the increasing toughness of the fiber reinforced concrete can be correlated with the changes in the fractal geometry of the cracked surface, we can be equipped with a strong mathematical formalism to characterize the mechanical properties of fiber reinforced concrete.

Thomson and Ashby (1984) proposed the following equation that relates the toughness of the material with the fractal dimension of the cracked surface:

\[
J_{IC} = \frac{\sigma_0}{3} \ln \left[ \frac{4^{(1-D_f)}}{12 f_p} \right] l_0^* \tag{19}
\]
Where $\sigma_0$ is the tensile strength of the material, $f_p$ is the volume fraction of voids and $l_0^*$ is the characteristic distance. The above equation is in contradistinction with the observations of Mandelbrot in the sense that it depicts the increasing trend in toughness with the increment in fractal dimension of the cracked surface. It shows that the increased toughness of the material is geometrically depicted by the more rougher or tortuous crack-path, thus increased fractility of the dimension. The response may vary depending on the nature of the material and the mathematical method employed for the calculation of fractal dimension. However, it can be safely concluded that there exists a definite correlation between the geometry of a cracked surface and the toughness characteristics of the material. This observation is more particularly significant in the case of FRC, where toughness is the governing material property. Based on his works on concrete with different aggregate size, M.A. Issa et al. inferred that fractal geometry of the cracked surface not only quantifies the magnitude of surface roughness but also correlates with the toughness of the material [58]. The Fractal nature of the crack surface is an indicative of the irregular path of the load transfer through the medium. This can be further correlated with the increased amount of energy that must be expended to create the fracture surface. It was found by Erdem et. al [64] that toughness of the material is proportional to the surface roughness of the aggregate. With the addition of fibers, multi-phase sub-systems will be created within the concrete which can trigger different toughening mechanisms. The fibers in themselves can act to increase the effective roughness of the aggregates and the interfacial joints or they can qualitatively affect the load-transfer path, thus rendering the crack-surface more tortuous. Hence it is an important point to consider that increased toughness of the FRC material might very
possibly be the global macroscopic response to the increased fractality of the cracked surface. Based on the past research on concrete response under various types of loading, it has been observed that the material global response in macro-scale is primarily governed by micro-structural heterogeneities [64]. Aggregate interlocking, micro-cracking and crack-bridging are the important toughening mechanisms which are actuated when the crack tends to follow a tortuous path, thus rendering the crack-geometry, fractal [64]. It was inferred by Arasan et al. [65] that Marshall stability of asphalt concrete is closely related with the fractal dimension of the aggregate surface such that a successful quantification of aggregate geometric irregularities is an essential field of study. It was further observed that asphalt mix stability is increased with the increase in fractality of the aggregate geometry such that it can help to determine various mechanical properties of the asphalt concrete [65].

4.1.6 Minibar Reinforced Concrete as a New Reinforcement Measure

It was discussed in detail in the previous section that how metallic and synthetic fibers can be used as a reinforcement measures to enhance the toughness of concrete. There is always an attempt in the scientific community to come up with new kinds of fibrous material that can provide better reinforcing and toughening mechanisms. The optimization of sectional, geometrical and material characteristics can lead to new kind of material that can provide better durability features, actuation of new toughening mechanisms and better performance against corrosion. Conventional longitudinal steel bar has been considered as reactive reinforcement which is functional only after the concrete cracks. Fibrous reinforcement can be considered as a proactive reinforcement
which can carry load at varying strain levels [66], hence, is capable of carrying load at micro and macro crack levels. Minibar can be considered to be a new kind of synthetic proactive reinforcement which provides toughness to the concrete through fiber rupture and pullout. The primary motive behind this part of research program is to assess the toughness characteristics of minibar reinforced concrete and to understand its properties in a line similar to steel reinforced concrete. Steel fibers generally provide the toughening mechanism through fiber pull-out and is susceptible to corrosion. High tensile strength and lesser stiffness of minibar may lead to the actuation of toughening mechanisms that can be achieved by the addition of hybrid fibers only. Flexural tensile strength, post-cracking tensile strength and energy absorption capacity of minibar reinforced concrete was studied conforming to relevant ASTM standards. This is described in detail in coming sections.

4.2 Materials

4.2.1 Basalt Minibar

The basalt minibar that was used for the investigation has been described in Chapter III. Basalt minibar is a macro-fiber made of basalt fiber reinforced polymer material.

4.2.2 Coarse Aggregate

The coarse aggregate used in the tests was crushed limestone with a maximum size of 20 mm (~0.75 inch) for normal concrete mix. Coarse aggregate was obtained
from a local Ohio supplier. Approximate absorption was 0.3 to 0.4% as suggested by the supplier. Smaller size (pea gravel) of limestone of a maximum size of 8 mm (~0.33 inch) was used for dry mix concrete as specified and also, for mix with small aggregate.

4.2.3 Fine Aggregate

Fine aggregate that was used was river sand purchased from the local supplier. The sand was free from clay and other inert impurities.

4.2.4 Cement

Cement used in this test program was Type I/II normal Portland cement. The strength properties were conforming to ASTM C150.

4.2.5 Fly Ash

Class C fly ash was also supplied by a local ready mix plant. Fly ash was used in dry mix as needed.

4.2.6 Lime

Lime was supplied by a local ready mix plant. Lime was used in dry mix concrete as required in the dry mix specifications.
4.2.7 High Range Water Reducer

High range water reducer used in the project was Axim ALLEGRO 122. The admixture was supposed to increase the workability of the concrete.

4.2.8 Water

The water used for the concrete mixes was the normal tap water supplied by the city of Akron.

4.3 Test Procedures

This sub-section comprises the entire laboratory tasks which were performed for the toughness characterization of basalt minibar reinforced concrete (MRC). Following ASTM standards were referred to meet the objective:

1. ASTM C39-Standard Test Method for Compressive Strength of Cylinder Concrete Specimens

2. ASTM C 78-Standard Test Method for Flexural Tensile Strength of Concrete (Using Simple Beam with Simple Third-Point Loading)
3. ASTM C 1399- Standard Test Method for Obtaining Average Residual-Strength of Fiber-Reinforced Concrete

4. ASTM C 1609- Standard Test Method for Flexural Performance of Fiber-Reinforced Concrete (Using Beam with Third-Point Loading)

4.3.1 Preparation of Specimens

Preparation of specimen comprises preparing minibar reinforced concrete beams with the dimensions conforming to relevant ASTM standard for flexural strength and post-cracking tensile strength (ARS) test and concrete cylinders for the hardened concrete properties of interest, that is, compressive strength comprising the following:

- 9 specimens of 4 inch x 4 inch x 14 inch (101 mm x 101 mm x 356 mm) beams for static flexural tests (modulus of rupture) and for ARS Tests
- Five or six specimens of 4 inch diameter x 8 inch long (101 mm x 202 mm) cylinders for compression tests

The variables which were accounted for the toughness characterization of minibar reinforced concrete (MRC) are:

1. Maximum aggregate size

2. Water-Cement Ratio

3. Minibar Dosage

Nine series of beams were prepared with varying maximum aggregate size, minibar dosage and water-cement ratio. The details of the mix-proportions for different series of beams are given in Table 4.1, 4.2 and 4.3.
Table 4.1: Concrete Mix Proportions of Normal Concrete with 20 mm (0.75") Aggregate

<table>
<thead>
<tr>
<th>Series #</th>
<th>Cement (lb (kg))</th>
<th>Water (lb (kg))</th>
<th>Fine Aggregate (lb (kg))</th>
<th>Coarse Aggregate (Max 20 mm) lb (kg)</th>
<th>Water Cement Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Series 1-3, 9</td>
<td>675 (400)</td>
<td>351 (208)</td>
<td>1424 (845)</td>
<td>1424 (845)</td>
<td>0.52</td>
</tr>
</tbody>
</table>

Table 4.2: Concrete Mix Proportions of Dry Mix Concrete

<table>
<thead>
<tr>
<th>Series #</th>
<th>Cement (lb (kg))</th>
<th>Fly Ash (lb (kg))</th>
<th>Lime (lb (kg))</th>
<th>Water (lb (kg))</th>
<th>Fine Aggregate (lb (kg))</th>
<th>Coarse Aggregate (max 8 mm) lb (kg)</th>
<th>Water Cement Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Series 4</td>
<td>405 (240)</td>
<td>101 (60)</td>
<td>34 (20)</td>
<td>162 (96)</td>
<td>1643 (975)</td>
<td>1643 (975)</td>
<td>0.4</td>
</tr>
<tr>
<td>DM Series 1</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1315 (780)</td>
<td>1972 (1170)</td>
<td></td>
</tr>
<tr>
<td>DM Series 2</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1096 (650)</td>
<td>2191 (1300)</td>
<td></td>
</tr>
</tbody>
</table>

Table 4.3: Concrete Mix Proportions of Mix with 8 mm (0.33") Aggregate

<table>
<thead>
<tr>
<th>Series #</th>
<th>Cement (lb (kg))</th>
<th>Water (lb (kg))</th>
<th>Fine Aggregate (lb (kg))</th>
<th>Coarse Aggregate (Max 20 mm) lb (kg)</th>
<th>Water Cement Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Series 7, 8</td>
<td>675 (400)</td>
<td>275 (163)</td>
<td>1315 (779)</td>
<td>1972 (1169)</td>
<td>0.41</td>
</tr>
<tr>
<td>Series 10</td>
<td>675 (400)</td>
<td>275 (163)</td>
<td>1150 (681)</td>
<td>2136 (1266)</td>
<td>0.41</td>
</tr>
</tbody>
</table>

One set of concrete mix proportions was used for normal concrete with maximum size of 20 mm (0.75") aggregate. The mix proportions are given in Table 4.1 for 1 cubic yard (and m³) of concrete for the first type of mix. Similarly, the mix proportions for dry mix proportions for dry mix used in this project are given in Table 4.2. Three batches (Series 4, DM Series 1 and DM Series 2) of dry mix concrete were made because the mix...
proportions were not previously tried at the University of Akron. Series 4 mix resulted in a very dry concrete devoid of any workability (slump less than 1 inch – 25 mm). Therefore, two other mixes (DM Series 1 and DM Series 2) were made with some minor adjustment and with commercially available high range water reducer. The two mixes had adequate workability in order for the test specimens to be cast comfortably with good workability with slump of about 2 to 3 inches (50 mm to 75 mm). Three other mixes were made with concrete with maximum aggregates size of 8 mm (0.33”).

Four Minibar dosages were used for the normal concrete mixes with 20 mm (0.75”) maximum size aggregate. These dosages were 2%, 6%, 8% and 10% by volume which translates to 61 lb/yd$^3$, 182 lb/yd$^3$, and 242 lb/yd$^3$ (36 kg/m$^3$, 108 kg/m$^3$, 144 kg/m$^3$ and 179 kg/m$^3$) as shown in Table 4.4. Minibar dosage for all three series of dry mix concrete was 1.89% by volume. The corresponding dosage by weight was 57 lb/yd$^3$ (34 kg/m$^3$). Two Minibar dosages were used for the normal concrete mixes with 8 mm (0.33 inch) aggregate. These dosages were 2% and 8%.

Table 4.4: Minibar Dosages Used

<table>
<thead>
<tr>
<th>Minibar Dosage % by Volume ($V_f$)</th>
<th>2</th>
<th>6</th>
<th>8</th>
<th>10</th>
<th>1.89</th>
<th>1.89</th>
<th>1.89</th>
<th>1.89</th>
<th>2</th>
<th>8</th>
<th>8</th>
</tr>
</thead>
<tbody>
<tr>
<td>Minibar Dosage (kg/m$^3$)</td>
<td>36</td>
<td>108</td>
<td>144</td>
<td>179</td>
<td>34</td>
<td>34</td>
<td>34</td>
<td>34</td>
<td>36</td>
<td>144</td>
<td>144</td>
</tr>
<tr>
<td>Minibar Dosage (lb/yd$^3$)</td>
<td>61</td>
<td>182</td>
<td>242</td>
<td>303</td>
<td>57</td>
<td>57</td>
<td>57</td>
<td>57</td>
<td>61</td>
<td>242</td>
<td>242</td>
</tr>
</tbody>
</table>

4.3.1.1 Mixing of Concrete

Mixing of concrete was done in a 9 cubic feet (0.25 m$^3$) capacity drum mixer (shown in Figure 4.13) in the concrete laboratory of the University of Akron. The
standard mixing procedure was used with mixing time of 1-3-3-2 minutes so that the Minibar was dispersed evenly. First trial of dry mix worked out very dry and was difficult to work with. However, with minor adjustment to the mix and with the use of superplasticizer, the subsequent mixes were more workable. In general, all the mixes (excepting dry mix #4) had good workability and about 2 to 3 inches (50 mm to 75 mm) slump. There is no particular concern regarding slump because that can be increased with suitable dosage of superplasticizers.

Figure 4.13: Concrete Mixer Used for Mixing

By physical inspection, the Minibar was found to be mixed very well within the concrete uniformly, without balling or segregation for all the mixes (Figure 4.14). There was a generous coating of binder paste on the Minibar as seen in the close up view in Figure 4.14.
Figure 4.14: Uniform Minibar Mixing in Fresh Concrete (Good Coating of Minibar with Cement Paste on the Bottom)

Once the tests specimens were cast and consolidated on a vibrating table, the specimens were covered with plastic sheets for 24 hours to prevent evaporation of moisture from the test specimens. Specimens were demolded after 24 hours and placed in a temperature and humidity controlled fog room until testing. Test specimens were kept exposed at room temperature for at least one day so as to keep the specimens dry at the time of testing.
4.4 Experimental Program

This section includes the comprehensive account of the experimental program pertaining to determination of flexural tensile strength (modulus of rupture), post-cracking tensile strength (average residual strength) and compressive strength of basalt mini-bar reinforced concrete. All the tests were performed conforming to the pertinent ASTM standards (as listed in sub-section 4.3).

4.4.1 Compressive Strength

The compressive strength of Minibar reinforced concrete (MRC) was determined using 4 inch (101.6 mm) diameter x 8 inch (203.2 mm) long cylinders. The compressive
strengths obtained from the tests are shown in Table 4.5. Each strength value is based on the average of three cylinder tests that were tested on the same day as the beam tests.

4.4.2 Modulus of Rupture (Flexural Tensile Strength) Tests

Modulus of rupture (MOR) tests were conducted using ASTM C78-07. MOR is the flexural tensile strength, that is, the tensile strength of concrete which is expected to increase marginally by the addition of fiber for small to moderate fiber dosages for synthetic fiber. Since concrete is intrinsically a heterogeneous composite material system, determination of its direct tensile strength is a very sensitive issue and is dependent on the experimental set-up employed and geometry of the test specimen. Modulus of rupture, determined based on ASTM C 78 is accepted to be an accurate index to quantify the tensile strength of concrete among the engineering community. The MOR is determined from the maximum load that small beam specimens 4 inch (101 mm) x 4 inch (101 mm) can carry over a span of 12 inches (305 mm) in a four point loading (third point loading) configuration as shown in Figure 4.16. The test set-up for MOR tests is similar to the test set-up used for the determination of Average Residual Strength (ARS), and the set-up used for C1609 toughness tests.

The rate of loading is specified in ASTM C78-07 to be between 125 psi/min and 175 psi/min (0.86 and 1.21 MPa/min). The rate of loading used in this project was 150 psi/min (1 MPa/min). Fracture occurred within the middle third of the span for all specimens.
Figure 4.16: Schematic of Flexural Tensile Strength Test Conforming to ASTM C 78

Figure 4.17 shows the actual four-point bending test set up for the determination of flexural tensile strength of minibar reinforced concrete.
4.4.3 Determination of ARS per ASTM C1399

ASTM Standard C1399 specifies the standard method for the determination of ARS of fiber reinforced concrete (FRC). By this method, FRC beams with dimensions of 100 x 100 x 350 mm (14” x 4” x 4”) are tested over a simple span of 300 mm (12”) in two stages under third-point loading. Initial loading is applied to the test beam with a 12 mm (½ inch) thick stainless steel plate placed under the beam so as to induce a flexural crack within the middle third of the span at the bottom of the beam. Initial loading is stopped as soon as the mid-span deflection is 0.02” (0.5 mm) or as soon as first crack appears whichever happens first. In the second stage, reloading of test beams is done without the steel plate placed under the beam. The average residual strength is computed using the
loads recorded for each test beam at four specified deflections during the reloading as shown in Figure 4.3. The average residual strength for loads at reloading deflections of 0.50, 0.75, 1.00 and 1.25 mm (0.02”, 0.03”, 0.04”, and 0.05”) are calculated using the following equation:

\[ ARS = \left( \frac{PA+PB+PC+PD}{4} \right) \times k \]  

(20)

Where:

\[ k = \frac{L}{bd^2}, \text{ mm}^2 (\text{in}^2) \]

\[ ARS = \text{Average Residual Strength, MPa (psi)} \]

\[ PA+PB+PC+PD = \text{recorded loads at specified deflections, N (lbf)} \]

\[ L = \text{span length (distance between supports), mm (in.)} \]

\[ b = \text{average width of specimen, mm (in.) and} \]

\[ d = \text{average depth of specimen, mm (in.)} \]

This test provides data needed to develop the load-deflection curve beyond the point where significant amount of cracking has occurred. ARS is an indirect measure of post-cracking strength of fiber reinforced concrete. It is not a real or true stress. ARS is calculated based on the assumption that the test beam section is solid (uncracked) even after cracking because gross (uncracked) section properties are used. Alternatively, ASTM C1609 test may be used for the determination of flexural toughness of fiber reinforced concrete. The beam dimensions used in the current study are 4” x 4” cross-section (b x d) with 12” span (l) as shown in Figure 4.16. Typical test setup for ARS beam tests is shown in Figure 4.18. The corresponding dimensions in metric units are 101 mm x 101 mm x 305 mm span. With these dimensions, factor k for ARS
calculations is 0.1875 in$^2$ and 1/3378 mm$^2$. Average residual strength for each series of beams was determined based on the average of three beam tests.

4.4.4 Flexural Toughness of Minibar Reinforced Concrete (MRC)

ASTM C1609 tests for the determination of flexural fracture toughness of Minibar reinforced concrete were also conducted in this study. Specimen toughness, which is the area under the load-deflection curve, is an indication of the energy absorption capability of the particular test specimen. One specimen from DM Series 1 and another beam from DM Series 2 were tested.
4.5 Results and Analysis

This sub-section covers the results obtained from experimental works performed on minibar reinforced concrete to identify the flexural tensile strength, post-cracking tensile strength and toughness, conforming to pertinent ASTM standards as discussed in previous sections. First part will cover the detailed elaboration of experimental results, followed by discussions on the topics. Second part will cover the comprehensive analytical study done for the strength characterization of minibar reinforced concrete beams based on material characterization, as described in section II.

4.5.1 Experimental Results

4.5.1.1 Modulus of Rupture (Flexural Tensile Strength) Tests

Modulus of rupture of plain concrete can be predicted with reasonable confidence. It normally ranges from $8 \sqrt{f'_c}$ to $10 \sqrt{f'_c}$, where $f'_c$ is the compressive strength of concrete in psi that is determined using standard cylinder tests. This range is $0.66 \sqrt{f'_c}$ to $0.83 \sqrt{f'_c}$ for normal weight concrete with $f'_c$ in Mpa.

ACI 318-08 recommends the use of the following equation for modulus of rupture:

$$f_r = 7.5 \sqrt{f'_c} \quad \text{for normal weight concrete with } f'_c \text{ in psi} \quad (21)$$

The above equation can be expressed in SI units as follows:

$$f_r = 0.62 \sqrt{f'_c} \quad \text{for normal weight concrete with } f'_c \text{ in MPa} \quad (22)$$
The predicted modulus of rupture (flexural tensile strength) values for all test beam series (six series to date) are also given in Table 4.5.

Table 4.5: Summary of Test Results

<table>
<thead>
<tr>
<th>Minibar Dosage % by Volume (Vf)</th>
<th>Normal Mix (Series 1)</th>
<th>Normal Mix (Series 2)</th>
<th>Normal Mix (Series 3)</th>
<th>Normal Mix (Series 9)</th>
<th>Dry Mix (Series 4)</th>
<th>Dry Mix (Series 1)</th>
<th>Dry Mix (Series 2)</th>
<th>S Agg (Series 7)</th>
<th>S Agg (Series 10)</th>
<th>S Agg (Series 8)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>6</td>
<td>8</td>
<td>10</td>
<td>1.89</td>
<td>1.89</td>
<td>1.89</td>
<td>2</td>
<td>8</td>
<td>8</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>108</td>
<td>144</td>
<td>179</td>
<td>34</td>
<td>34</td>
<td>34</td>
<td>36</td>
<td>144</td>
<td>144</td>
<td></td>
</tr>
<tr>
<td>Minibar Dosage (kg/m³)</td>
<td>61</td>
<td>182</td>
<td>242</td>
<td>303</td>
<td>57</td>
<td>57</td>
<td>57</td>
<td>61</td>
<td>242</td>
<td>242</td>
</tr>
<tr>
<td>Concrete Compressive Strength, MPA</td>
<td>68.4</td>
<td>64.9</td>
<td>69.9</td>
<td>47.0</td>
<td>96.8</td>
<td>64.2</td>
<td>88.1</td>
<td>70.1</td>
<td>26.7</td>
<td>32.3</td>
</tr>
<tr>
<td>Concrete Compressive Strength, psi</td>
<td>9,914</td>
<td>9,418</td>
<td>10,142</td>
<td>6819</td>
<td>14,013</td>
<td>9,529</td>
<td>12,781</td>
<td>10,711</td>
<td>3870</td>
<td>4691</td>
</tr>
<tr>
<td>Flexural Tensile Strength (MPa)</td>
<td>6.00</td>
<td>8.74</td>
<td>10.51</td>
<td>10.67</td>
<td>7.48</td>
<td>7.06</td>
<td>7.33</td>
<td>7.84</td>
<td>9.53</td>
<td>9.85</td>
</tr>
<tr>
<td>Predicted Modulus of Rupture (ACI Equation), MPA</td>
<td>5.15</td>
<td>5.02</td>
<td>5.21</td>
<td>4.27</td>
<td>6.13</td>
<td>5.05</td>
<td>5.85</td>
<td>5.22</td>
<td>3.22</td>
<td>3.54</td>
</tr>
<tr>
<td>Flexural Tensile Strength (psi)</td>
<td>872</td>
<td>1,269</td>
<td>1,527</td>
<td>1,550</td>
<td>1,087</td>
<td>1,025</td>
<td>1,065</td>
<td>1,138</td>
<td>1,384</td>
<td>1,430</td>
</tr>
<tr>
<td>Predicted Modulus of Rupture (ACI Equation), psi</td>
<td>747</td>
<td>728</td>
<td>755</td>
<td>619</td>
<td>888</td>
<td>732</td>
<td>848</td>
<td>796</td>
<td>467</td>
<td>514</td>
</tr>
<tr>
<td>Average Residual Strength (MPa)</td>
<td>3.26</td>
<td>6.68</td>
<td>9.33</td>
<td>10.40</td>
<td>5.84</td>
<td>5.67</td>
<td>6.11</td>
<td>5.64</td>
<td>8.29</td>
<td>8.47</td>
</tr>
<tr>
<td>Average Residual Strength (psi)</td>
<td>474</td>
<td>970</td>
<td>1,335</td>
<td>1,510</td>
<td>848</td>
<td>823</td>
<td>888</td>
<td>819</td>
<td>1,204</td>
<td>1,230</td>
</tr>
</tbody>
</table>

As can be seen in the table, the modulus of rupture values obtained from the failure loads recorded in the tests are much greater than those predicted using ACI equation for plain concrete. This clearly demonstrates that the addition of Minibar has improved the flexural tensile strength of concrete significantly. The test results are plotted in Figure 4.19 for each series for comparison of flexural tensile strengths. From Figure 4.19, it is clear that the flexural tensile strength increases with the increase in Minibar dosage. The ACI equation was rearranged in order to derive a relationship between the Minibar dosage and the extent of increase in flexural tensile strength of MRC.
Figure 4.19: Comparison of Flexural Tensile Strength Obtained from Tests with ACI 318 Predictions for Plain Concrete without Minibar

These values are also plotted in Figure 4.20 with varying Minibar dosage as a percent by volume ($V_r$).
The ACI equation captures the dependence of modulus of rupture on compressive strength of plain concrete by using $\sqrt{f_c}$ term in the equation.

Assuming that the implied relationship holds for Minibar reinforced concrete, a new factor called “M” will be introduced in the equation to derive the effect of Minibar dosage ($V_f$) on flexural tensile strength as follows:

$$M = \frac{f_c}{\sqrt{f_c}}$$  \hspace{1cm} \text{for normal weight concrete with $f_c$ in psi} \hspace{1cm} (23)

$$M = \frac{f_c}{\sqrt{f'_c}}$$  \hspace{1cm} \text{for normal weight concrete with $f'_c$ in MPa} \hspace{1cm} (24)

From ACI 381-08 equation for normal weight concrete, M equals 7.5 for psi units, and 0.62 for MPa units (see Eqs. 1 and 2) for plain concrete. Factor M was calculated for

Figure 4.20: Variation of Flexural Tensile Strength with Minibar Dosage
each series of tests based on the actual failure loads from tests, and is summarized in Table 4.6, and also plotted in Figure 4.21. It seems to increase with Minibar dosage indicating that there is linear increase in flexural tensile strength due to the increase Minibar dosage.

Table 4.6: Factor M based on Test Results and Curve Fitting

<table>
<thead>
<tr>
<th>Minibar Dosage % by Volume (Vf)</th>
<th>Normal Mix (Series 1)</th>
<th>Normal Mix (Series 2)</th>
<th>Normal Mix (Series 3)</th>
<th>Normal Mix (Series 9)</th>
<th>Dry Mix (Series 4)</th>
<th>Dry Mix (Series 1)</th>
<th>Dry Mix (Series 2)</th>
<th>S Agg (Series 7)</th>
<th>S Agg (Series 10)</th>
<th>S Agg (Series 8)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>2</td>
<td>6</td>
<td>8</td>
<td>10</td>
<td>1.89</td>
<td>1.89</td>
<td>1.89</td>
<td>2</td>
<td>8</td>
<td>8</td>
</tr>
<tr>
<td>Factor &quot;M&quot; based on test results (for MPa units)</td>
<td>0.727</td>
<td>1.086</td>
<td>1.259</td>
<td>1.559</td>
<td>0.762</td>
<td>0.872</td>
<td>0.782</td>
<td>0.937</td>
<td>1.847</td>
<td>1.734</td>
</tr>
<tr>
<td>&quot;M&quot; based on Suggested Equation (for MPa units)</td>
<td>0.772</td>
<td>1.076</td>
<td>1.228</td>
<td>1.380</td>
<td>0.770</td>
<td>0.770</td>
<td>0.770</td>
<td>0.900</td>
<td>1.740</td>
<td>1.740</td>
</tr>
<tr>
<td>Factor &quot;M&quot; based on test results (for psi units)</td>
<td>8.76</td>
<td>13.08</td>
<td>15.16</td>
<td>18.77</td>
<td>9.18</td>
<td>10.50</td>
<td>9.42</td>
<td>11.28</td>
<td>22.25</td>
<td>20.88</td>
</tr>
</tbody>
</table>
Figure 4.21: Variation of Factor M with Minibar Dosage

By linear curve fitting with the line forced through an intercept of 7.5 on the y-axis for psi units (forced through 0.62 for MPa units) the following equations were developed. Tentatively, the following equations are suggested for flexural tensile strength of Minibar reinforced concrete:

1. For concrete with a maximum aggregate size of 20 mm (0.75 inch)

\[ f_r = (0.62 + 0.076 V_f) \sqrt{f_c} \quad \text{for } f_c \text{ in MPa and} \]

\[ f_r = (7.5 + 0.915 V_f) \sqrt{f_c} \quad \text{for } f_c \text{ in psi} \]

Where, \( V_f \) is Minibar dosage in percent by volume within the range of 2% to 10%.

2. For concrete with a maximum aggregate size of 8 mm (0.33 inch)
\[ f_r = (0.62 + 0.14V_t) \sqrt{f_c'} \leq 1.74 \sqrt{f_c'} \quad \text{for} \quad f_c' \text{ in MPa} \quad \text{and} \quad (26) \]

\[ f_r = (7.5 + 1.686V_t) \sqrt{f_c'} \leq 21 \sqrt{f_c'} \quad \text{for} \quad f_c' \text{ in psi} \]

where, \( V_t \) is Minibar dosage in percent by volume within the range of 2% to 8%.

3. For dry mix concrete with a maximum aggregate size of 8 mm (0.33 inch) and with 1.89%

\[ f_r = 0.77 \sqrt{f_c'} \quad \text{for} \quad f_c' \text{ in MPa} \quad \text{and} \quad (27) \]

\[ f_r = 9.3 \sqrt{f_c'} \quad \text{for} \quad f_c' \text{ in psi} \]

for a Minibar dosage in percent by volume of 1.89%. For other dosages, additional tests are required.

From Fig. 4.21, it is evident that MRC made with smaller maximum size of coarse aggregate (8 mm or 0.33 inch in this study) provides better flexural tensile strength than that of concrete made with larger size aggregate (20 mm or 0.75 inch maximum size aggregate). Suggested equations for the two cases are also shown in the figure and were found to define somewhat lower bound strengths. A maximum limit of 1.74\( \sqrt{f_c'} \) for \( f_c' \) in MPa (21\( \sqrt{f_c'} \) for \( f_c' \) in psi) is placed for MRC with smaller maximum size aggregate because of the limited range of tests conducted in this study. However, it may be possible to refine these equations with further testing.

A typical load-deflection curve obtained from flexural tensile strength test is shown in Figure 4.22.
Figure 4.22: Typical Load-Deflection Curve Obtained from Flexural Tests (Based on Cross-Head Movement)

It was observed that the minibar reinforced concrete beams displayed improved ductile behavior, capable of undergoing a larger deflection before the failure. This implies better toughness characteristics of minibar reinforced concrete, which can be graphically represented as the area under the load-deflection curve.

The fracture plane of the cracked beam was carefully examined with naked eyes to see if there was any dominant mode of failure of basalt minibar and concrete matrix. For a discrete synthetic fiber like basalt minibar, the following modes of failures can be expected:

1. Minibar Rupture- Minibar of the given aspect ratio is capable of developing its maximum tensile strength, thus rendering the failure mode to be by minibar rupture. Energy dissipation at failure is then primarily comprised of the summation of fracture energy of concrete matrix and basalt minibar.
2. Pull-out Failure: If the effective embedment length of minibar in concrete is not sufficient for minibar to develop its full tensile strength, the governing failure mode will be by pull-out. This comprises of the failure of interfacial bond strength between minibar and concrete matrix. The energy dissipation at failure is primarily comprised of fracture energy associated with cracking of concrete and energy dissipated through friction between minibar and concrete.

From the visual analysis, it is hard to identify the governing failure mode with absolute certainty. There was evidence of minibar rupture, however, many cases of pull-out failure was also observed suggesting that optimization of Minibar length and diameter might be beneficial in improving flexural strength. Figure 4.23 and Figure 4.24 show the typical specimens after failure under flexural tensile strength test conforming to ASTM C 78.
Addition of any type of fiber is not known to significantly influence the compressive strength of the corresponding fiber reinforced concrete. However, the mode of crushing failure of concrete cylinders is changed from brittle mode to a more ductile mode when fiber is added to concrete. MRC cylinders failed in a ductile manner with large deformations. The load was sustained by the cylinders even after significant cracking and reduction in length resulting in large deformations of the test cylinders in the direction of load application. Figure 4.25 shows typical failure of cylinders made
from MRC. The figure shows that the test cylinders are still intact in single piece after the completion of the test which indicates that the addition of Minibar caused ductile failure of MRC compared to brittle failure of plain concrete cylinders which normally shatter upon testing.

![Typical Failure Mode of Cylinder Specimens](image)

**Figure 4.25: Typical Failure Mode of Cylinder Specimens**

4.5.1.2 Average Residual Strength (ARS) Tests

As already discussed, the primary reason to use fiber to reinforce concrete in civil engineering structures is to increase its flexural tensile strength and toughness. Once plain concrete cracks, it immediately loses its load carrying capacity due to its inherent brittle nature. One of the ways to increase the toughness of concrete is to reinforce it with fibers such that different energy distribution mechanisms can be introduced which increases the ductility of the member, ensures its functional and geometrical integrity for
a longer time, increases the value of tensile strain at failure and hence the overall toughness of the member. The toughening mechanisms is found to have actuated through multiple cracking, fiber pullout, fiber deformation fracture and others [67]. Kevin et al. with their work on steel fiber reinforced concrete (SFRC) found out that with a nominal fiber dosage varying between 3.5 and 4%, a hundredfold increase in network of load can be achieved. Through analysis of tomographic images, they quantified the different toughening mechanisms and confirmed that roughly half of the internal energy dissipation is achieved through multiple cracking in concrete matrix facilitated by the presence of fibers, while the remaining is the result of fiber pullout, fiber rupture and others.

There is a qualitative difference between high strength discrete polymer fiber such as basalt minibar and conventional steel fibers, which can account for actuation of other toughening mechanisms. At the crack plane, the minibars protruding from both sides seem to develop a good mechanical interlock, aided further by the better interfacial frictional dissipation among the minibars.

ASTM C 1399 recommends a method which is used to calculate a particular index termed as “Average Residual Strength” as an indirect measure of the post-cracking tensile strength of fiber reinforced concrete.

Average residual strengths obtained from tests are summarized in Table 4.5 for the ten batches of concrete with different Minibar dosages tested so far. ARS values for different series of tests are also shown in Figure 4.26.
Figure 4.26: ARS Values Obtained from Tests

These values are also plotted with Minibar dosage in Figure 4.27 that shows distinct increase in ARS values with increased dosage of Minibar at an approximate rate of 1 MPa per 1% increase in Minibar dosage by volume ($V_f$). Dry mix concrete outperformed normal concrete, probably because of smaller size aggregate used in dry mix concrete.
Figure 4.27: ARS Values Plotted with Minibar Dosage

Typical load deflections curves obtained from ARS tests are shown in Figure 4.28. Figure shows initial loading curve with steel plate below the specimen, and also shows the reloading curve after the required amount of cracking and the removal of steel plates. Reloading curve shows very good post-crack load carrying capacity of Minibar reinforced concrete which is a measure of toughness.
Since average residual strength is an indirect measure of post-cracking tensile strength of concrete, it is a reasonable assumption that it is purely a function of fiber dosage, interfacial bond strength between fiber and concrete matrix and the aspect ratio of fiber (fiber geometrical parameter). It will be analytically shown in the coming chapter that post cracking tensile strength of minibar reinforced concrete is independent of concrete compressive strength and is the function minibar geometry, interfacial bond strength between minibar and concrete matrix and minibar dosage.

Based on above observations, the following approximate equations for average residual strength (ARS) of Minibar reinforced concrete (MRC) as a function of $V_f$ are suggested for concrete with maximum size of aggregate of 20 mm (¾ inch) and for
concrete with maximum size of aggregate of 8 mm (0.33 inch) until further research is performed:

\[ ARS = 1.05 \ V_f \text{ for MPa units} \quad (10) \]

\[ ARS = 152 \ V_f \text{ for psi units} \quad (28) \]

Where, \( V_f \) is Minibar dosage in percent by volume within the range of \( 1.89\% \leq V_f \leq 8.0\% \).

4.5.1.3 Flexural Toughness of Minibar Reinforced Concrete (MRC)

ASTM C1609 tests for the determination of flexural fracture toughness of Minibar reinforced concrete were also conducted in this study. Specimen toughness, which is the area under the load-deflection curve, is an indication of the energy absorption capability of the particular test specimen. One specimen from DM Series 1 and another beam from DM Series 2 were tested. For the dry mix concrete with 1.89% Minibar dosage, load-deflection curves developed from the tests are shown in Figures 4.29 and 4.30.

The area under the load-deflection curve for the beam tested from DM Series 1 is approximately 39.0 Joules (Nm). This is the specimen toughness as obtained from the test per ASTM C1609. Similarly, the specimen toughness for the test specimen from DM Series 2 is 44.7 Joules (Nm). The average specimen toughness for the two beams tested was about 42 Joules (Nm).
Figure 4.29: Typical Load-Deflection Curves from C1609 Beam Test

(DM Series 1)

Figure 4.30: Typical Load-Deflection Curves from C1609 Beam Test

(DM Series 2)
4.5.1.3.1 Verification of Suitability of Minibar to Provide Shear Resistance in Structural Members

The requirements that must be satisfied to consider steel fiber for shear resistance of structural members made from steel fiber reinforced concrete (SFRC) were introduced in ACI 318-08 (Building Code Requirements for Structural Concrete) for the first time. Following three requirements must be satisfied for the purpose:

(a) Dosage of deformed steel fibers per cubic yard of concrete is at least 100 lb (60 kg/m³)

(b) Residual strength \(f_{100, 1.0}\) based on ASTM C1609 at midspan deflection of span over 300 is greater than or equal to 90% of measured first peak strength obtained from flexural test or 90% of ACI defined modulus of rupture (0.9\(f_r\)).

(c) Residual strength \(f_{100, 2.0}\) based on ASTM C1609 at midspan deflection of span over 150 is greater than or equal to 75% of measured first peak strength obtained from flexural test or 75% of ACI defined modulus of rupture (0.75\(f_r\)).

The above requirements are applicable to steel fiber reinforced concrete only. However, these requirements can form a basis for Minibar reinforced concrete. Therefore, the residual strengths, and the corresponding minimum strengths required to satisfy these requirements were determined and are summarized in Table 4.7 and Table 4.8.
Table 4.7: C1609 Test Results in SI Units

<table>
<thead>
<tr>
<th></th>
<th>$f_r$</th>
<th>$0.9 f_r$</th>
<th>$f_{100, 1.0}$</th>
<th>$0.75 f_r$</th>
<th>$f_{100, 2.0}$</th>
<th>$T_{100, 2.0}$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>MPa</td>
<td>MPa</td>
<td>MPa</td>
<td>MPa</td>
<td>MPa</td>
<td>Joules (Nm)</td>
</tr>
<tr>
<td>DM Series 1</td>
<td>5.05*</td>
<td>4.55</td>
<td>6.21</td>
<td>3.79</td>
<td>3.75</td>
<td>39.0</td>
</tr>
<tr>
<td>DM Series 2</td>
<td>5.85*</td>
<td>5.27</td>
<td>6.76</td>
<td>4.39</td>
<td>4.66</td>
<td>44.7</td>
</tr>
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</table>

Table 4.8: C1609 Test Results in US Customary Units

<table>
<thead>
<tr>
<th></th>
<th>$f_r$</th>
<th>$0.9 f_r$</th>
<th>$f_{100, 1.0}$</th>
<th>$0.75 f_r$</th>
<th>$f_{100, 2.0}$</th>
<th>$T_{100, 2.0}$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>psi</td>
<td>psi</td>
<td>psi</td>
<td>psi</td>
<td>psi</td>
<td>lb-ft</td>
</tr>
<tr>
<td>DM Series 1</td>
<td>732*</td>
<td>660</td>
<td>901</td>
<td>550</td>
<td>544</td>
<td>28.8</td>
</tr>
<tr>
<td>DM Series 2</td>
<td>848*</td>
<td>764</td>
<td>980</td>
<td>637</td>
<td>676</td>
<td>33.0</td>
</tr>
</tbody>
</table>

Where $f_r = \text{Modulus of Rupture (flexural tensile strength)}$ from Table 4.5

From the values given in the tables, Minibar reinforced concrete is seen to satisfy the ACI 318-08 requirements (b) and (c) so as to be use the fiber (Minibar) for shear resistance as verified in the following paragraphs even though the fiber dosage was less than 100 lb/yd$^3$.

**Requirement (a)**

Dosage for DM Series 1 and 2 = 34 kg/m$^3$ or 57 lb/yd$^3$. Steel fiber dosage needed is 60 kg/m$^3$ or 100 lb/yd$^3$. (This requirement is not satisfied).
Requirement (b)

DM Series 1:

\[ f_{100, 1.0} = 6.21 \text{ MPa} > 0.9 f_r = 4.55 \text{ MPa} \quad \text{(or } f_{100, 1.0} = 901 \text{ psi} > 0.9 f_r = 660 \text{ psi)} \]

DM Series 2:

\[ f_{100, 1.0} = 6.76 \text{ MPa} > 0.9 f_r = 5.27 \text{ MPa} \quad \text{(or } f_{100, 1.0} = 980 \text{ psi} > 0.9 f_r = 764 \text{ psi)} \]

Requirement (c)

DM Series 1:

\[ f_{100, 2.0} = 3.75 \text{ MPa} \approx 0.75 f_r = 3.79 \text{ MPa} \quad \text{(or } f_{100, 2.0} = 544 \text{ psi} \approx 0.75 f_r = 550 \text{ psi)} \]

DM Series 2:

\[ f_{100, 2.0} = 4.66 \text{ MPa} > 0.75 f_r = 4.39 \text{ MPa} \quad \text{(or } f_{100, 2.0} = 676 \text{ psi} > 0.75 f_r = 637 \text{ psi)} \]

The above comparisons demonstrate that Minibar satisfies the ACI 318-08 requirements for steel fiber reinforced concrete for steel fiber to be used for shear resistance in structural members. While specific beam tests to determine the extent of shear resistance provided by Minibar are yet to be conducted, there is good evidence from recent research that there can be an increase in shear strength of members without shear reinforcement by 25 to 33% due to the addition of synthetic fiber with a small dosage of 0.5% by volume. The preceding is believed to be a sound basis for considering
the use of Minibar in structural members as shear reinforcement or to reduce the amount of shear reinforcement particularly for slabs and walls.

4.5.2 Analytical Results

To account for the ultimate load carrying capacity of minibar reinforced concrete (MRC) beams, an analytical model was prepared based on the material characterization of basalt minibar and minibar reinforced concrete. The underlying philosophy behind the model is to treat the randomly distributed minibar fibers in the concrete matrix as a flexural reinforcement. It is based on the observation that basalt minibar, unlike other synthetic fibers, is a high strength material which is not only capable of introducing novel stress-distribution mechanisms leading to enhanced toughness characteristics of the concrete, but also can be instrumental on enhancing its strength characteristics.

4.5.2.1 Number of Minibar fibers across the Crack as a Function of Minibar Volume-Fraction

The increased tensile strength and flexural properties of mini-bar reinforced concrete is the result of high tensile strength characteristics of basalt mini-bar fibers and their interaction with concrete matrix, such as actuation of crack-bridging mechanism at the cracked surface. Since these strength and interaction properties are actuated at the crack surface of the specimen, it is safe to assume that they are proportional to the number of fibers available at the cracked surface, hence, to the volume fraction of the fibers in the concrete matrix. Therefore, it is always advantageous to have a theoretical model which can account for the numbers of fibers at the crack-plane of the specimen.
which may be further helpful to have a qualitative assessment of the probable toughening mechanisms on one hand, and quantitative assessment of the strength characteristics (such as flexure and shear) on the other. It has been shown that numbers of fibers per unit area of the cracked concrete surface is given as:

\[
n = \alpha \frac{V_f}{A_f}
\]

(29)

Where \( n \) = number of fiber per unit surface

\( \alpha \) = Orientation coefficient

\( V_f \) = Fiber volume fraction

\( A_f \) = Cross – section of a fiber

Orientation factor accounts for the random distribution of the fibers in the concrete matrix and their effective availability at the cracked surface. Fibers added to the concrete during mixing tend to align randomly at varying angle to the longitudinal axis of the beam. For all practical purpose, it can be assumed that all the fibers added to concrete are proportionally available at the cracked surface. However, it should be noted that effective numbers of fibers will be reduced due to their random orientation which can’t be controlled during mixing. Based on their works on steel fiber reinforced concrete, Dupot et al. [68] proposed an analytical model to account for the random distribution of the fibers in concrete matrix.

Similar kind of works for a different kind of ring type steel fiber reinforced concrete was reported by Lee et al. [69]. The authors made the following simplifying assumptions:
1. Orientation of factor is independent of the geometry of the fiber

2. Orientation effect depends significantly on the vibration time and the workability of the concrete. For instance, in case of highly workable concrete such as self-compacting, fibers tend to align in horizontal direction. However, these effects are fundamentally extrinsic and not dependent on the physical properties of the materials, which render their quantification extremely difficult.

3. Each point of the cross-section of the fiber can be considered to have an equal probability of being the point of gravity of the fiber.

4. The orientation of the fibers is affected by the boundary conditions imposed by the formwork. For instance, the orientation factors for the regions close and parallel to the boundary and for the regions at the intersection of two boundaries are different and must be accounted for.

The effective orientation-factor can then be expressed as:

\[
\alpha = \frac{\alpha_1(b-l_f)(h-l_f) + \alpha_2(b-l_f)(l_f+h-l_f) + \alpha_3 l_f^2}{bh} \tag{30}
\]

*Where* \(\alpha_1 = \text{Orientation factor for the zone unaffected by boundaries}\)

\(\alpha_2 = \text{Orientation factor for the zone parallel to the boundaries}\)

\(\alpha_3 = \text{Orientation factor for the intersection of two boundaries}\)

\(l_f = \text{length of the fiber}; (b, h) = \text{cross-sectional dimensions of the beam}\)

It was analytically found that \(\alpha_1\) is equal to 0.5, \(\alpha_2\) is equal to 0.6 and \(\alpha_3\) is equal to 0.84. It was assumed during the analysis that the embedment length of the fiber is half
of the fiber length. The orientation factors may vary with the embedment length, however, they were not considered during the above mentioned analysis.

4.5.2.1.1 Orientation Factors in Case of Mini-Bar Reinforced Concrete

Even though the orientation factor can be considered to be fairly independent of the geometry of the fiber, what may quantitatively affect the orientation factor for basalt mini-bar fiber compared to regular steel fiber is their considerably lower specific weight. Specific weight of steel-fiber is over 3 times that of the specific weight of the concrete, whereas specific weight of basalt mini-bar is approximately 72% of the specific weight of concrete. Due to the significant difference in their specific weights, the distribution of fibers in concrete can be quantitatively different across the cracked surface. Since steel is heavier than concrete, steel fibers tend to settle down aided with sufficient vibration. In the case of basalt mini-bar which is lighter than the concrete-matrix, buoyant force can have some effect, thus affecting the overall distribution across the control volume. To account for it, a modified relation is proposed for the number of mini-bar fibers across the cracked surface. The fibers across the cracked surface of mini-bar reinforced beams were carefully counted for various volume fractions. Based on the actual data, following relationship was proposed, incorporating the weight effect. It is also to be noted that counting the fiber manually is also prone to human error due to different lengths of protrusion from the fracture surface. To account for the loss of fibers during the mix and during counting, a reduction factor is also incorporated in the proposed equation.

The relation is given as follows:
\[ n = \phi \cdot \alpha \cdot \sqrt[8]{\frac{\gamma_{basalt}}{\gamma_{Steel}}} \cdot \frac{V_f}{A_f} \]  

(31)

Where \( n \) = number of basalt mini-bar fibers per unit surface

\( \alpha \) = Orientation coefficient

\( \phi \) = Correction factor associated with manual fiber counting = 0.85

\( V_f \) = Fiber volume fraction

\( A_f \) = Cross-section of a fiber

\( \gamma_{basalt} \) = Specific weight of basalt mini-bar fiber

\( \gamma_{Steel} \) = Specific weight of steel

Hence, the relation for number of mini-bar fibers per unit area of the cracked surface reduces to:

\[ n = 0.65 \cdot \alpha \cdot \frac{V_f}{A_f} \]  

(32)

The minibars fibers across the crack plane of the beams from the flexural and average residual strength tests were carefully counted and are tabulated in table 4.9.
Table 4.9: Manual counts of Minibar for Different Volume Fraction

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Minibar Volume Fraction</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.35%</td>
</tr>
<tr>
<td>sp-1</td>
<td>4</td>
</tr>
<tr>
<td>sp-2</td>
<td>4</td>
</tr>
<tr>
<td>sp-3</td>
<td>6</td>
</tr>
<tr>
<td>sp-4</td>
<td>6</td>
</tr>
<tr>
<td>sp-5</td>
<td>5</td>
</tr>
<tr>
<td>sp-6</td>
<td>7</td>
</tr>
<tr>
<td>sp-7</td>
<td>10</td>
</tr>
<tr>
<td>sp-8</td>
<td></td>
</tr>
<tr>
<td>sp-9</td>
<td></td>
</tr>
<tr>
<td>sp-10</td>
<td></td>
</tr>
<tr>
<td>sp-11</td>
<td></td>
</tr>
<tr>
<td>sp-12</td>
<td></td>
</tr>
<tr>
<td>sp-13</td>
<td></td>
</tr>
<tr>
<td>sp-14</td>
<td></td>
</tr>
<tr>
<td>sp-15</td>
<td></td>
</tr>
<tr>
<td>sp-16</td>
<td></td>
</tr>
<tr>
<td>sp-17</td>
<td></td>
</tr>
<tr>
<td>sp-18</td>
<td></td>
</tr>
<tr>
<td>sp-19</td>
<td></td>
</tr>
<tr>
<td>sp-20</td>
<td></td>
</tr>
</tbody>
</table>

| Average | 6     | 17    | 17    | 65  | 113 |
| Std Dev | 2     | 5     | 5     | 17  | 15  |

All the fibers were counted manually across the fracture-plane of minibar reinforced concrete beams of different volume fractions. Beams had been tested for the determination of flexural tensile strength and average residual strength.
The number of minibars across the fracture planes were also analytically calculated with the Equation 32 as proposed above. Average summary for the different minibar dosage are tabulated in Table 4.10 with graphical presentation (Figure 4.31).

Table 4.10: Number of Fibers across the Crack-Surface

<table>
<thead>
<tr>
<th>$vf$ (%)</th>
<th>No of Fibers (Exp)</th>
<th>No of Fibers (Proposed)</th>
<th>% Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.35</td>
<td>7</td>
<td>10</td>
<td>-30</td>
</tr>
<tr>
<td>0.5</td>
<td>17</td>
<td>14</td>
<td>21</td>
</tr>
<tr>
<td>0.75</td>
<td>17</td>
<td>21</td>
<td>-19</td>
</tr>
<tr>
<td>2</td>
<td>65</td>
<td>58</td>
<td>12</td>
</tr>
<tr>
<td>4</td>
<td>113</td>
<td>115</td>
<td>-2</td>
</tr>
</tbody>
</table>
It can be observed that the theoretical model which expresses number of minibars across the crack plane as the function of orientation factor, minibar dosage and specific weight of basalt minibar with respect to steel, is able to predict the manual counts of minibar with good accuracy. The prediction is getting more accurate with increasing minibar dosage, which is expected as errors during manual count, weighing and mixing of minibar for lower minibar dosage is more probable than for higher dosages.

4.5.2.2 Flexural Strength of Basalt Mini-Bar Reinforced Concrete Beams

From the standard flexural and average residual strength tests on basalt mini-bar reinforced concrete beams, it was observed that not only the toughness, but the maximum load capacity was also increased due to the addition of minibar. With the increasing fiber content, there was a significant increase in flexural tensile strength, post cracking tensile...
strength (average residual strength) and the maximum load carrying capacity. This implies that the discrete increment in flexural capacity of the mini-bar reinforced concrete beams can be achieved with increasing fiber-content. For all the specimens of different volume fractions for May-July 2012 series, the results are tabulated in Table 4.11.

It can be observed from the plot between normalized average residual strength (with respect to flexural tensile strength) with minibar volume fraction that the there is a linear increase in the normalized post cracking tensile strength of mini-bar reinforced concrete with increasing fiber dosage. At higher volume-fraction of 4%, it can be seen from Figure 4.32 that basalt mini-bar reinforced concrete can provide residual tensile strength of magnitude almost equal to its flexural modulus of rupture.

Table 4.11: Summary of ARS and Modulus of Rupture for Different Volume Fraction

<table>
<thead>
<tr>
<th></th>
<th>$f'_c$</th>
<th>$ARS, psi$</th>
<th>$f_r, psi$</th>
<th>$ARS/fr$</th>
<th>$Pu, lb$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gen 2.1-0.75%</td>
<td>9277</td>
<td>666</td>
<td>1289</td>
<td>0.52</td>
<td>6874</td>
</tr>
<tr>
<td>Gen 2.1-2%</td>
<td>9789</td>
<td>1489</td>
<td>1491</td>
<td>1.00</td>
<td>7953</td>
</tr>
<tr>
<td>Gen 2.1-4%</td>
<td>9475</td>
<td>1981</td>
<td>2113</td>
<td>0.94</td>
<td>11271</td>
</tr>
<tr>
<td>Gen 3- 0.5%</td>
<td>9231</td>
<td>513</td>
<td>1334</td>
<td>0.38</td>
<td>7113</td>
</tr>
<tr>
<td>Gen 3- 2%</td>
<td>10002</td>
<td>1209</td>
<td>1728</td>
<td>0.70</td>
<td>9219</td>
</tr>
<tr>
<td>Gen 3- 4%</td>
<td>10668</td>
<td>2488</td>
<td>2282</td>
<td>1.09</td>
<td>12173</td>
</tr>
<tr>
<td>Glass MB - 0.75%</td>
<td>9779</td>
<td>917</td>
<td>1495</td>
<td>0.61</td>
<td>7971</td>
</tr>
<tr>
<td>Glass MB - 2%</td>
<td>9687</td>
<td>1629</td>
<td>1600</td>
<td>1.02</td>
<td>8534</td>
</tr>
<tr>
<td>Gen 3- 0.35%</td>
<td>9497</td>
<td>327</td>
<td>1345</td>
<td>0.24</td>
<td>7172</td>
</tr>
<tr>
<td>Dry concrete 1.89% Gen 1</td>
<td>8690</td>
<td>1051</td>
<td>1592</td>
<td>0.66</td>
<td>8492</td>
</tr>
</tbody>
</table>
4.5.2.2.1 Flexural Analysis of Basalt Mini-Bar Reinforced Concrete Beams (Discrete Fiber-Properties Approach)

Since minibar is a relatively stiff discrete polymer fiber with high tensile strength, a different approach was deemed necessary to develop an analytical model to account for the flexural analysis of fiber reinforced concrete beams. Conventional ACI approach for the flexural analysis of fiber reinforced concrete beam relies on the assumption that fiber reinforced concrete is a semi-homogenous material which tensile strength is a global mechanical property of the composite system. This can also be termed as “smeared properties approach”.

In case of MRC, where concrete is reinforced with discrete and stiff high strength synthetic fiber, the resultant material system is a discrete two-phase material system (for instance, reinforced concrete beam). For such a system, “Smeared Properties Approach” which relies on the assumption of unitary global material parameter characterizing the
system, may be not provide a correct model. Hence a new approach called “Discrete Fiber Properties Approach” was introduced. This approach is based on the assumption that the fiber at a particular distance from the neutral axis develops corresponding tensile strength similar to longitudinal reinforcement in conventional reinforced concrete. Fibers are treated as the flexural reinforcement which is responsible for the force transfer and resulting equilibrium of the section through perfect bond between minibar and the concrete matrix. Internal moment generated by tensile strengths provided by minibars at different depth from the neutral axis sum up to account for the internal moment of resistance of the section. The following assumptions are made for the analysis:

1. Plane section before bending remains plane after bending, that is, strain is distributed linearly across the section.

2. The concrete reaches the crushing strain of 0.003 at failure and fiber-contribution above the neutral axis is negligible.

3. Rectangular stress-strain block for concrete is assumed at ultimate failure.

4. Actual stress-strain curve for basalt mini-bar is considered for the analysis, that is, stress is proportional to strain until rupture.

5. Tensile force at a particular level below neutral axis is equally divided between all the fibers at that level.

6. Only mode of failure for the minibar is tensile rupture, that is, possible pull-out of the minibars from the matrix is not considered for analysis.

The minibars on the crack-plane were carefully counted and their positions were located with respect to the bottom of the beam. Since we have the stress-strain characteristics of minibar from the material characterization, neutral axial depth was
calculated based on the force equilibrium. Once the neutral axis depth is determined, the theoretical moment strength of the section can be established by taking moments of the tensile forces about the neutral axis. The mathematical methodology is described hereafter.

Figure 4.33: Stress Distribution in Basalt Mini-Bar Reinforced Concrete

Figure 4.33 shows the assumed strain and stress distribution across the depth of a minibar reinforced concrete section. From the linear strain assumption, stress at the different levels (where the number of fibers and their positions were located) can be calculated based on the actual stress-strain curve of basalt minibar.

Tensile stress at a particular level $y$ from the neutral axis is given by

$$\sigma_y = E_f \times \varepsilon_y \leq \sigma_f$$  \hspace{1cm} (33)

where $E_f$ = Modulus of elasticity of Basalt mini – bar

where $\varepsilon_y$ = Tensile strain on the fibers at distance $y$

where $\sigma_f$ = Uniaxial tensile strength of basalt mini – bar

$$T_y = \sum A_{fy} \times \sigma_y$$  \hspace{1cm} (34)
For equilibrium

\[ C = T \]

i.e. \[ 0.85 \cdot f_c \cdot b \cdot a = T_1 + T_2 + \ldots + T_y + T_{n-1} + T_n \] \hspace{1cm} (35)

Neutral axis depth can be determined from the force-equilibrium condition as stated above. Once the neutral axis depth is determined, internal moment of resistance of the section can be obtained from moment equilibrium.

Once the equilibrium is established, the moment of resistance of the section is calculated as

\[ M = \left\{ \left[ \sum A_{f_1} \right] \cdot \sigma_1 \cdot \left( d_1 - \frac{a}{2} \right) + \left[ \sum A_{f_2} \right] \cdot \sigma_2 \cdot \left( d_2 - \frac{a}{2} \right) + \left[ \sum A_{f_3} \right] \cdot \sigma_1 \cdot \left( d_3 - \frac{a}{2} \right) + \ldots + \left[ \sum A_{f_y} \right] \cdot \sigma_y \cdot \left( d_y - \frac{a}{2} \right) + \left[ \sum A_{f_{n-1}} \right] \cdot \sigma_{n-1} \cdot \left( d_{n-1} - \frac{a}{2} \right) + \left[ \sum A_{f_n} \right] \cdot \sigma_n \cdot \left( d_n - \frac{a}{2} \right) \right\} \]

\hspace{1cm} (36)

Or \[ M = \sum_{i=1}^{i=n} \left[ \sum A_{f_n} \right] \cdot \sigma_n \cdot \left( d_n - \frac{a}{2} \right) \]

\hspace{1cm} (37)

Typical spreadsheet calculations for minbar reinforced concrete for varying minbar dosage are presented below:
Table 4.12: Typical Spreadsheet Calculation for 4% Minibar Dosage

<table>
<thead>
<tr>
<th>y</th>
<th>Depth from top</th>
<th>DIS FROM NA</th>
<th>a/2</th>
<th>Moment arm</th>
<th>Strain</th>
<th>No of fibers</th>
<th>Stress</th>
<th>Force</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.8</td>
<td>0.2</td>
<td>-0.31</td>
<td>0.14</td>
<td>0.06</td>
<td>0.00</td>
<td>7.00</td>
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<td>0.00</td>
</tr>
<tr>
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<td>0.00</td>
<td>7.00</td>
<td>-4.14</td>
<td>0.00</td>
</tr>
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<td>3.4</td>
<td>0.6</td>
<td>0.09</td>
<td>0.14</td>
<td>0.46</td>
<td>0.00</td>
<td>5.00</td>
<td>25.02</td>
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<tr>
<td>3</td>
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<td>0.29</td>
<td>0.14</td>
<td>0.66</td>
<td>0.00</td>
<td>6.00</td>
<td>11.61</td>
<td>93.33</td>
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<td>0.14</td>
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<td>0.00</td>
<td>10.00</td>
<td>19.48</td>
<td>261.07</td>
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<td>0.14</td>
<td>1.06</td>
<td>0.00</td>
<td>6.00</td>
<td>27.36</td>
<td>219.95</td>
</tr>
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<td>2.8</td>
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<td>0.89</td>
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<td>1.26</td>
<td>0.01</td>
<td>5.00</td>
<td>35.23</td>
<td>236.05</td>
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<td>2.6</td>
<td>1.6</td>
<td>1.09</td>
<td>0.14</td>
<td>1.46</td>
<td>0.01</td>
<td>10.00</td>
<td>43.11</td>
<td>577.63</td>
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<td>50.98</td>
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<td>5.00</td>
<td>58.86</td>
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<td>1.69</td>
<td>0.14</td>
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<td>0.01</td>
<td>5.00</td>
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<td>447.09</td>
</tr>
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<td>1.89</td>
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<td>0.14</td>
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<td>5.00</td>
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<td>552.61</td>
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<td>5.00</td>
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<td>3</td>
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<td>0.14</td>
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<td>0.02</td>
<td>5.00</td>
<td>113.98</td>
<td>763.65</td>
</tr>
<tr>
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<td>3.09</td>
<td>0.14</td>
<td>3.46</td>
<td>0.02</td>
<td>4.00</td>
<td>121.85</td>
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<td>0</td>
<td>4</td>
<td>3.49</td>
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<td>3.86</td>
<td>0.02</td>
<td>8.00</td>
<td>137.60</td>
<td>1475.09</td>
</tr>
</tbody>
</table>

| T     | 9446.6979 lb   |
|Fc     | 10000 psi      |
| beta  | 0.55           |
| a     | 0.28 in        |
| C     | 9446.70 lb     |

Force equilibrium:

\[ T - C = -1.45E-05 \]

Moment-Strength:

\[ 25589.60 \text{ lb-in} \]

LOAD:

\[ 12794.80 \text{ lb} \]
Table 4.13: Typical Spreadsheet Calculation for 2% Minibar Dosage

Gen 3 2% BF 16 may, 2012-FLEXURE

<table>
<thead>
<tr>
<th>y</th>
<th>Depth from top</th>
<th>DIS FROM NA</th>
<th>a/2</th>
<th>Moment arm</th>
<th>Strain</th>
<th>No of fibers</th>
<th>Stress</th>
<th>Force</th>
</tr>
</thead>
<tbody>
<tr>
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<td>-0.04</td>
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<td>4.00</td>
<td>-1.60</td>
<td>0.00</td>
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<td>1.5</td>
<td>1.06</td>
<td>0.12</td>
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<td>0.01</td>
<td>28.00</td>
<td>48.69</td>
<td>1826.68</td>
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<td>1.5</td>
<td>2.5</td>
<td>2.06</td>
<td>0.12</td>
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<td>2</td>
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<td>0.12</td>
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<td>0.02</td>
<td>18.00</td>
<td>140.12</td>
<td>3379.69</td>
</tr>
</tbody>
</table>

92 T 8135.76 lb

FC 10000 psi
beta 0.55
a 0.24 in
C 8135.76 lb

Force Equilibrium

T-C -9.537E-06

Moment-Strength 19960.31 lb-in

Load 9980.16 lb
Table 4.14: Typical Spreadsheet Calculation for 0.35 % Minibar Dosage

<table>
<thead>
<tr>
<th>y</th>
<th>Depth from top</th>
<th>Dis from NA</th>
<th>a/2</th>
<th>Moment arm</th>
<th>Strain</th>
<th>No of fibers</th>
<th>Stress</th>
<th>Force</th>
</tr>
</thead>
<tbody>
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<td>0.95</td>
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<td>170.87</td>
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<td>1.30</td>
<td>0.03</td>
<td>1.00</td>
<td>138.00</td>
<td>184.92</td>
</tr>
<tr>
<td>2.5</td>
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<td>1.37</td>
<td>0.05</td>
<td>1.45</td>
<td>0.03</td>
<td>1.00</td>
<td>138.00</td>
<td>184.92</td>
</tr>
<tr>
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<td>1.90</td>
<td>1.77</td>
<td>0.05</td>
<td>1.85</td>
<td>0.04</td>
<td>1.00</td>
<td>138.00</td>
<td>184.92</td>
</tr>
<tr>
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<td>2.37</td>
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<td>0.05</td>
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<td>0.06</td>
<td>1.00</td>
<td>138.00</td>
<td>184.92</td>
</tr>
<tr>
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<td>2.97</td>
<td>0.05</td>
<td>3.05</td>
<td>0.07</td>
<td>1.00</td>
<td>138.00</td>
<td>184.92</td>
</tr>
<tr>
<td>0.7</td>
<td>3.30</td>
<td>3.17</td>
<td>0.05</td>
<td>3.25</td>
<td>0.07</td>
<td>1.00</td>
<td>138.00</td>
<td>184.92</td>
</tr>
<tr>
<td>0.35</td>
<td>3.65</td>
<td>3.52</td>
<td>0.05</td>
<td>3.60</td>
<td>0.08</td>
<td>1.00</td>
<td>138.00</td>
<td>184.92</td>
</tr>
</tbody>
</table>

10   T  1835.15  lb

F'c  5000  psi
beta 0.8
a  0.11  in
C  1835.15  lb

Force Equilibrium

T-C  0.000738

Moment- Strength 4260.252  lb-in

Load  2130.126  lb
Table 4.15: Typical Spreadsheet Calculation for 0.5 % VF

<table>
<thead>
<tr>
<th>y</th>
<th>Depth from top</th>
<th>Dis from NA</th>
<th>a/2</th>
<th>Moment arm</th>
<th>Strain</th>
<th>No of fibers</th>
<th>Stress</th>
<th>Force</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.8</td>
<td>0.2</td>
<td>0.00</td>
<td>0.08</td>
<td>0.12</td>
<td>0.00</td>
<td>1.00</td>
<td>0.02</td>
<td>0.02</td>
</tr>
<tr>
<td>3.6</td>
<td>0.4</td>
<td>0.20</td>
<td>0.08</td>
<td>0.32</td>
<td>0.00</td>
<td>1.00</td>
<td>19.93</td>
<td>26.70</td>
</tr>
<tr>
<td>3.6</td>
<td>0.4</td>
<td>0.20</td>
<td>0.08</td>
<td>0.32</td>
<td>0.00</td>
<td>1.00</td>
<td>79.65</td>
<td>106.73</td>
</tr>
<tr>
<td>3.4</td>
<td>0.6</td>
<td>0.40</td>
<td>0.08</td>
<td>0.52</td>
<td>0.00</td>
<td>1.00</td>
<td>19.93</td>
<td>26.70</td>
</tr>
<tr>
<td>1.5</td>
<td>2.5</td>
<td>2.30</td>
<td>0.08</td>
<td>2.42</td>
<td>0.03</td>
<td>1.00</td>
<td>156.00</td>
<td>209.04</td>
</tr>
<tr>
<td>1.6</td>
<td>2.4</td>
<td>2.20</td>
<td>0.08</td>
<td>2.32</td>
<td>0.03</td>
<td>1.00</td>
<td>156.00</td>
<td>209.04</td>
</tr>
<tr>
<td>2.25</td>
<td>1.75</td>
<td>1.55</td>
<td>0.08</td>
<td>1.67</td>
<td>0.02</td>
<td>1.00</td>
<td>154.31</td>
<td>206.77</td>
</tr>
<tr>
<td>2.25</td>
<td>1.75</td>
<td>1.55</td>
<td>0.08</td>
<td>1.67</td>
<td>0.02</td>
<td>1.00</td>
<td>154.31</td>
<td>206.77</td>
</tr>
<tr>
<td>1</td>
<td>3</td>
<td>2.80</td>
<td>0.08</td>
<td>2.92</td>
<td>0.04</td>
<td>1.00</td>
<td>156.00</td>
<td>209.04</td>
</tr>
<tr>
<td>0.5</td>
<td>3.5</td>
<td>3.30</td>
<td>0.08</td>
<td>3.42</td>
<td>0.05</td>
<td>1.00</td>
<td>156.00</td>
<td>209.04</td>
</tr>
<tr>
<td>0</td>
<td>4</td>
<td>3.80</td>
<td>0.08</td>
<td>3.92</td>
<td>0.06</td>
<td>5.00</td>
<td>156.00</td>
<td>1045.20</td>
</tr>
<tr>
<td>0.5</td>
<td>3.5</td>
<td>3.30</td>
<td>0.08</td>
<td>3.42</td>
<td>0.05</td>
<td>1.00</td>
<td>156.00</td>
<td>209.04</td>
</tr>
</tbody>
</table>

Gen 3 0.5% BF 18 may, 2012

<table>
<thead>
<tr>
<th>NA Depth</th>
<th>0.20 in</th>
<th>Modulus 6630 ksi</th>
</tr>
</thead>
<tbody>
<tr>
<td>Area</td>
<td>0.00134 sq.in</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Force equilibrium</th>
<th>T-C</th>
<th>-1.45E-05 lb</th>
</tr>
</thead>
</table>

<table>
<thead>
<tr>
<th>Load</th>
<th>3981.07 lb</th>
</tr>
</thead>
</table>

170
Table 4.16: Typical Spreadsheet Calculation for 0.75 % Minibar Dosage

<table>
<thead>
<tr>
<th>y</th>
<th>Depth from top</th>
<th>Dis From NA</th>
<th>a/2</th>
<th>Moment arm</th>
<th>Strain</th>
<th>No of fibers</th>
<th>Stress</th>
<th>Force</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.8</td>
<td>0.2</td>
<td>0.05</td>
<td>0.04</td>
<td>0.16</td>
<td>0.00</td>
<td>4</td>
<td>6.17</td>
<td>33.07</td>
</tr>
<tr>
<td>2.8</td>
<td>1.2</td>
<td>1.05</td>
<td>0.04</td>
<td>1.16</td>
<td>0.02</td>
<td>1</td>
<td>136.47</td>
<td>182.87</td>
</tr>
<tr>
<td>3.3</td>
<td>0.7</td>
<td>0.55</td>
<td>0.04</td>
<td>0.66</td>
<td>0.01</td>
<td>2</td>
<td>71.32</td>
<td>191.14</td>
</tr>
<tr>
<td>3</td>
<td>1</td>
<td>0.85</td>
<td>0.04</td>
<td>0.96</td>
<td>0.02</td>
<td>1</td>
<td>110.41</td>
<td>147.95</td>
</tr>
<tr>
<td>2.1</td>
<td>1.9</td>
<td>1.75</td>
<td>0.04</td>
<td>1.86</td>
<td>0.03</td>
<td>1</td>
<td>156.00</td>
<td>209.04</td>
</tr>
<tr>
<td>2.2</td>
<td>1.8</td>
<td>1.65</td>
<td>0.04</td>
<td>1.76</td>
<td>0.03</td>
<td>1</td>
<td>156.00</td>
<td>209.04</td>
</tr>
<tr>
<td>1.2</td>
<td>2.8</td>
<td>2.65</td>
<td>0.04</td>
<td>2.76</td>
<td>0.05</td>
<td>4</td>
<td>156.00</td>
<td>836.16</td>
</tr>
<tr>
<td>0.9</td>
<td>3.1</td>
<td>2.95</td>
<td>0.04</td>
<td>3.06</td>
<td>0.06</td>
<td>1</td>
<td>156.00</td>
<td>209.04</td>
</tr>
<tr>
<td>0.8</td>
<td>3.2</td>
<td>3.05</td>
<td>0.04</td>
<td>3.16</td>
<td>0.06</td>
<td>4</td>
<td>156.00</td>
<td>836.16</td>
</tr>
</tbody>
</table>

\[ T = 2854.48 \text{ lb} \]

\[ f_c = 10000 \]

\[ \beta = 0.55 \]

\[ a = 0.08 \text{ in} \]

\[ C = 2854.48 \text{ lb} \]

Force Equilibrium: \[ T-C = 0.00 \]

Moment-Strength: \[ 6826.42 \text{ lb-in} \]

Load: \[ 3413.21 \text{ lb} \]
Table 4.17: Calculated Ultimate Load from the Proposed Approach for Different Volume Fractions

<table>
<thead>
<tr>
<th>$f'c$</th>
<th>NA Depth, in</th>
<th>No of Fibers</th>
<th>Calculated Load, lb</th>
</tr>
</thead>
<tbody>
<tr>
<td>Glass MB - 2%</td>
<td>9687</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.33</td>
<td>53</td>
<td>7760</td>
<td></td>
</tr>
<tr>
<td>0.32</td>
<td>55</td>
<td>8010</td>
<td></td>
</tr>
<tr>
<td>0.35</td>
<td>58</td>
<td>8313</td>
<td></td>
</tr>
<tr>
<td>Average</td>
<td>0.33</td>
<td>55</td>
<td>8027.67</td>
</tr>
<tr>
<td>Gen 3- 2%</td>
<td>10002</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.31</td>
<td>48</td>
<td>7830</td>
<td></td>
</tr>
<tr>
<td>0.36</td>
<td>47</td>
<td>10000</td>
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</tr>
<tr>
<td>0.30</td>
<td>67</td>
<td>6146</td>
<td></td>
</tr>
<tr>
<td>0.44</td>
<td>92</td>
<td>9980</td>
<td></td>
</tr>
<tr>
<td>0.30</td>
<td>42</td>
<td>7594</td>
<td></td>
</tr>
<tr>
<td>0.37</td>
<td>84</td>
<td>7142</td>
<td></td>
</tr>
<tr>
<td>Average</td>
<td>0.35</td>
<td>63</td>
<td>8115.33</td>
</tr>
<tr>
<td>Gen 3- 4%</td>
<td>10668</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.49</td>
<td>100</td>
<td>12277</td>
<td></td>
</tr>
<tr>
<td>0.55</td>
<td>123</td>
<td>14490</td>
<td></td>
</tr>
<tr>
<td>0.51</td>
<td>106</td>
<td>13442</td>
<td></td>
</tr>
<tr>
<td>0.52</td>
<td>114</td>
<td>13696</td>
<td></td>
</tr>
<tr>
<td>0.49</td>
<td>131</td>
<td>11509</td>
<td></td>
</tr>
<tr>
<td>0.51</td>
<td>120</td>
<td>12795</td>
<td></td>
</tr>
<tr>
<td>Average</td>
<td>0.51</td>
<td>116</td>
<td>13034.83</td>
</tr>
<tr>
<td>Gen 3- 0.35%</td>
<td>9497</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.13</td>
<td>10</td>
<td>2130</td>
<td></td>
</tr>
<tr>
<td>0.08</td>
<td>6</td>
<td>986</td>
<td></td>
</tr>
<tr>
<td>Average</td>
<td>0.11</td>
<td>8.00</td>
<td>1558.00</td>
</tr>
<tr>
<td>Gen 3- 0.5%</td>
<td>9231</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.20</td>
<td>17</td>
<td>3980</td>
<td></td>
</tr>
<tr>
<td>0.22</td>
<td>18</td>
<td>3050</td>
<td></td>
</tr>
<tr>
<td>0.16</td>
<td>13</td>
<td>2230</td>
<td></td>
</tr>
<tr>
<td>Average</td>
<td>0.19</td>
<td>16.00</td>
<td>3086.67</td>
</tr>
</tbody>
</table>
Table 4.17 presents the load carrying capacity of minbar reinforced concrete beams calculated based on “Discrete Fiber Properties Approach” as elaborated above. The average summary of the results are tabulated in Table 4.18.

Table 4.18: Comparative tabulation of experimental and theoretical results

<table>
<thead>
<tr>
<th></th>
<th>$f'_c$ (psi)</th>
<th>No of fibers, exp</th>
<th>No of Fibers, Theoretical</th>
<th>Maximum Load, Exp (lb)</th>
<th>Maximum Load, Theoretical (lb)</th>
</tr>
</thead>
<tbody>
<tr>
<td><em>Gen 3- 0.35%</em></td>
<td>9497</td>
<td>8</td>
<td>10</td>
<td>7172</td>
<td>2130</td>
</tr>
<tr>
<td><em>Gen 3- 0.5%</em></td>
<td>9231</td>
<td>16</td>
<td>14</td>
<td>7113</td>
<td>3086</td>
</tr>
<tr>
<td><em>Gen 3- 2%</em></td>
<td>10002</td>
<td>63</td>
<td>58</td>
<td>9219</td>
<td>8115</td>
</tr>
<tr>
<td><em>Gen 3- 4%</em></td>
<td>10668</td>
<td>116</td>
<td>115</td>
<td>12173</td>
<td>13035</td>
</tr>
<tr>
<td><em>Glass MB - 2%</em></td>
<td>9687</td>
<td>55</td>
<td>58</td>
<td>8534</td>
<td>8028</td>
</tr>
</tbody>
</table>

It was observed that the proposed model was underestimating the ultimate load capacity of the beams for the lower volume fraction of 0.35%, 0.5 and 0.75 % by a considerable amount. However, with the increased volume fraction of 2 and 4%, the average ultimate load capacity was predicted very accurately. Summary of the calculation is shown in Table 4.18. Results are reasonable as it was expected that for lower volume fraction, there is greater probability of less number of fibers being available on the failure plane. The analysis was also able to provide insight into the structural behavior of basalt mini-bar reinforced concrete beams as a function of fiber volume fraction. At lower volume fraction, availability of smaller number of fibers on the failure plane may inhibit the actuation of fiber-bridging effect, thus reducing the chances of gradual energy absorption leading to flexural failure. As the crack-width increases with deflection, each
half of the beam on either side of the crack tends to rotate more freely about the support point due to inadequate fiber-bridging effect across the crack due to the absence of sufficient number of fibers. This may lead to the premature rupture of the mini-bar before reaching their ultimate tensile strength. Considering these facts, it was concluded that a unified approach is necessary which can account for the varying volume-fractions of minibar, shear span to depth ratio and compressive strength of concrete.

Based on a trial and error, corrections are provided that can account for the variation of:

1. Concrete Compressive Strength
2. Minibar Dosage
3. Shear Span to Depth Ratio

For \( f_c' \geq 7000 \text{ psi} \), \textit{Ultimate load carrying capacity} =

\[
(Predicted \text{ load carrying capacity}) * 1.8 * \left( V_f * \frac{a_s}{d} \right)^{-0.5}
\]  

\text{(38)}

For \( f_c' = (4000 \text{ to } 6000) \text{ psi} \), \textit{Ultimate load carrying capacity} =

\[
(Predicted \text{ load carrying capacity}) * 0.6 * \left( V_f * \frac{a_s}{d} \right)^{-0.5}
\]  

\text{(39)}

Where

\[
\frac{a_s}{d} = \text{Shear Span to Depth Ratio}
\]

\[
V_f = \text{Shear Span to Depth Ratio}
\]

\[
f_c' = \text{Concrete Compressive Strength}
\]
Proposed modification was applied to the minibar reinforced concrete beams (as tabulated in Table 4.18) and the results are tabulated in Table 4.19. The graphical representation can be seen on Figure 4.34.

Table 4.19: Predicted Load Carrying Capacity of Beams of varying volume fraction ratio with span depth to ratio of 1

<table>
<thead>
<tr>
<th>Volume-Fraction (%)</th>
<th>Predicted (lb)</th>
<th>Experimental (lb)</th>
<th>Correction</th>
<th>Corrected Prediction (lb)</th>
<th>% difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.35</td>
<td>2130</td>
<td>7172</td>
<td>3.04</td>
<td>6480.64</td>
<td>-9.64</td>
</tr>
<tr>
<td>0.5</td>
<td>3086</td>
<td>7113</td>
<td>2.55</td>
<td>7855.67</td>
<td>10.44</td>
</tr>
<tr>
<td>0.75</td>
<td>3950</td>
<td>6874</td>
<td>2.08</td>
<td>8209.92</td>
<td>19.43</td>
</tr>
<tr>
<td>1.89</td>
<td>4560</td>
<td>8492</td>
<td>1.31</td>
<td>5970.44</td>
<td>-29.69</td>
</tr>
<tr>
<td>2</td>
<td>8115</td>
<td>9219</td>
<td>1.27</td>
<td>10328.71</td>
<td>12.04</td>
</tr>
<tr>
<td>4</td>
<td>13035</td>
<td>12173</td>
<td>0.90</td>
<td>11731.50</td>
<td>-3.63</td>
</tr>
</tbody>
</table>
Figure 4.34: Graphical Comparison of Experimental vs. Theoretical Load Carrying Capacity

From Table 4.18 we can see that “Discrete Fiber Properties Approach” is able to predict the ultimate load carrying capacity of minibar reinforced concrete beams with the proposed modifications accounting for varying minibar dosage, concrete compressive strength and shear span to depth ratio. 30 small beams (Oct, 2012) with shear span to depth ratio of 1 were tested. The compressive strength varied between 4500 to 5600 psi. Two volume fraction values, 0.2% and 0.5% were considered for study. The summary of the average results are tabulated in Table 4.20. The graphical presentation of the comparison is shown in Figure 4.35.
Table 4.20: Tabulated Results for Maximum Load-Carrying Capacity of Minibar Reinforced Concrete Beams from Theoretical Model

<table>
<thead>
<tr>
<th>Fiber-Type</th>
<th>Volume-Fraction (%)</th>
<th>Exp (lb)</th>
<th>Predicted (lb)</th>
<th>Corrected (lb)</th>
<th>% Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>G-3 (old)</td>
<td>0.2</td>
<td>4125</td>
<td>2196</td>
<td>2946</td>
<td>28.58</td>
</tr>
<tr>
<td>G-3 (new)</td>
<td>0.2</td>
<td>3657</td>
<td>3211</td>
<td>4307</td>
<td>-17.79</td>
</tr>
<tr>
<td>G-3/1</td>
<td>0.2</td>
<td>3541</td>
<td>3118</td>
<td>4183</td>
<td>-18.13</td>
</tr>
<tr>
<td>G-3/2</td>
<td>0.2</td>
<td>4380</td>
<td>3301</td>
<td>4428</td>
<td>-1.11</td>
</tr>
<tr>
<td>G-3/3</td>
<td>0.2</td>
<td>4199</td>
<td>3099</td>
<td>4157</td>
<td>0.99</td>
</tr>
<tr>
<td>G-3 (old)</td>
<td>0.5</td>
<td>3895</td>
<td>4835</td>
<td>4216</td>
<td>-8.23</td>
</tr>
<tr>
<td>G-3 (new)</td>
<td>0.5</td>
<td>3850</td>
<td>4386</td>
<td>3722</td>
<td>3.32</td>
</tr>
<tr>
<td>G-3/1</td>
<td>0.5</td>
<td>3711</td>
<td>5607</td>
<td>4758</td>
<td>-28.22</td>
</tr>
<tr>
<td>G-3/2</td>
<td>0.5</td>
<td>4321</td>
<td>5405</td>
<td>4586</td>
<td>-6.14</td>
</tr>
<tr>
<td>G-3/3</td>
<td>0.5</td>
<td>4376</td>
<td>5040</td>
<td>4276</td>
<td>2.27</td>
</tr>
</tbody>
</table>
4.6 Conclusion and Recommendations

This subsection comprises conclusions that were based on the experimental and analytical works done in regard to the toughness characterization of minibar reinforced concrete.

1. The failure mode of test cylinders made from Minibar reinforced concrete (MRC) for the determination of compressive strength changed from brittle mode to ductile mode due to the addition of Minibar as with other types of fibers.

2. The following linear equations for flexural tensile strength of MRC were developed from the test results:

For concrete with a maximum aggregate size of 20 mm (0.75 inch):

\[ f'_{r} = (0.62 + 0.076 V_{f}) \sqrt{f'_{c}} \quad \text{for} \quad f'_{c} \quad \text{in MPa} \quad \text{and} \]

\[ f'_{r} = (7.5 + 0.915 V_{f}) \sqrt{f_{c}} \quad \text{for} \quad f_{c} \quad \text{in psi} \]
where, $V_f$ is Minibar dosage in percent by volume within the range of 2% to 10%.

For concrete with a maximum aggregate size of 8 mm (0.33 inch):

\[
fr = (0.62 + 0.14 V_f) \sqrt{fc'} \quad \text{for } fc' \text{ in MPa} \quad \text{and}
\]

\[
fr = (7.5 + 1.686 V_f) \sqrt{fc'} \quad \text{for } fc' \text{ in psi}
\]

where, $V_f$ is Minibar dosage in percent by volume within the range of 2% to 8%.

For dry mix concrete with a maximum aggregate size of 8 mm (0.33 inch) and with 1.89%:

\[
fr = 0.77 \sqrt{fc'} \quad \text{for } fc' \text{ in MPa} \quad \text{and}
\]

\[
fr = 9.3 \sqrt{fc'} \quad \text{for } fc' \text{ in psi}
\]

for a Minibar dosage in percent by volume of 1.89%. For other dosages, additional tests are required.

3. The average residual strengths (ARS) obtained for Minibar reinforced concrete were much greater than expected, suggesting that Minibar has significantly helped in the post-cracking performance of concrete in the current test program. The following equation is suggested:

\[
ARS = 1.05 V_f \quad \text{for MPa unit and } ARS = 152 V_f \quad \text{for psi units}
\]

where, $V_f$ is Minibar dosage in percent by volume within the range of 2% to 10%.
The proposed equation is based on the observation that average residual strength, which is an indirect measure of post-cracking tensile strength, is independent of concrete compressive strength and is purely a function of minibar volume fraction and minibar aspect-ratio.

One test per ASTM C1609 was performed for each of dry mix series 1 and 2 with 1.89% dosage. The test results demonstrate that MRC satisfies ACI 318-08 requirements that are specified for steel fiber reinforced concrete for using fiber as shear reinforcement.

4. An analytical expression was proposed for the number of mini-bars across a unit area of cracked surface as given below:

\[ n = \phi \times \alpha \times \frac{\sqrt{Y_{\text{basalt}}}}{\sqrt{Y_{\text{Steel}}}} \times \frac{V_f}{A_f} \]

5. An analytical model was proposed to predict the ultimate load carrying capacity of MRC beams for different volume fractions. More tests are being conducted to correlate the model with more data and the results are very positive.
CHAPTER V
FLEXURAL AND SHEAR CHARACTERIZATION OF MRC BEAMS OF VARYING SHEAR-SPAN TO DEPTH RATIOS

This section provides a detailed description of the literature review, laboratory procedures and experimental program that was carried out for the strength characterization (flexure and shear) of minibar reinforced concrete (MRC) beams of varying span to depth ratios. Primary objective of this part of the research program is the identification of flexural capacity and shear strength of minibar reinforced concrete beams, with a development of an analytical model. Overall research was conducted in two sets: the first set comprises minibar reinforced concrete beams of laboratory scale with a fixed minibar dosage (0.5%) and varying shear span to depth ratios, the second set comprises of large scale beams with minibar dosage and shear span to depth ratios as variables. The second set was important from the viewpoint that it was able to provide an insight into the structural behavior of minibar reinforced concrete beams at structural scale. Investigations were also important in that it was able to provide insight into the size-effect associated with minibar reinforced concrete. If toughness is very important for quantification of the serviceability limit states, strength characterization can be similarly considered very significant for the quantification at strength limit states, as well as serviceability limit states. The section will orderly follow the literature review, description of the materials, description of the laboratory procedures and experimental
program to meet the objective, finally followed by results and analysis with conclusions and recommendations.

5.1 Literature Review

Shear strength of reinforced concrete has always been a challenging subject of study for civil engineers for a prolonged period of time. Even though the history of its study almost dates back to 100 years, there is still a lack of general consensus among scientific community about its rational elaboration [70]. The current codes are found be conservative in predicting the shear-strength of reinforced concrete beams. However, the flexural behavior has been adequately addressed. This can be attributed to difficulty associated with creating a simple experimental model which can produce a pure shear in the specimen.

For the conventional four point loading case with two equal point loads applied at the top of the beam and two supports provided at the bottom of the beam, the region between the point loads is subjected to pure flexure whereas the shear span is subjected to constant shear and uniformly varying moment [70]. This unavoidable coupling of normal stress and shear stress in the shear-span of the beam may lead to further interaction mechanisms. The non-linear and heterogeneous nature of material and the variation of shear force along the length and depth of the beam may add more variability in the model.

It should be noted that shear strength in most of the beams is smaller than the direct shear strength of the concrete [71]. The governing factor is supposed to be the diagonal tensile stress that develops on the concrete beam at different sections from the
coupling of shear and longitudinal normal stress. If a small concrete element at a particular length of the beam is considered, it is subjected to direct shear and normal bending stress which give rise to principal tensile and compressive stress at the section at certain orientation depending on the stress intensities. Since tensile strength of concrete is significantly lower, it governs the failure criteria at a stress level much lower than the actual shear strength of the concrete.

It can be observed that large bending moment can reduce the shear force at which diagonal cracks form to roughly one half of the value at which they would form if the moments were zero or nearly so. This may be attributed to the larger cracks at the region with higher moment thus reducing aggregate interlock and other shear resisting mechanisms.

5.1.1 Development of Traditional U.S. Code Provisions for shear

The basic model for shear transfer through reinforced concrete beams can be regarded to be the parallel truss model as proposed by Ritter in 1899. In this model, the resisting mechanism was analogized with a truss and shear force is transferred by the corresponding development of internal stresses in various members. The top member is the compression chord and the stirrups act as vertical tension ties. The concrete was modeled to be the compression diagonal member inclined at 45 degree with the horizontal. When the 45 degree parallel chord truss model was first introduced it was found by various researchers that the experimental shear strength values was greater than that predicted by truss model by nearly a constant amount [72]. This led to the introduction of concept of concrete contribution to shear strength. It is stated in the
NHCRP report 549 that there is no physical reason to support that the concrete contribution to shear resistance at ultimate is equal to the diagonal cracking load and is a convenience justified by the simplicity of the result [72]. The 45 degree truss model followed in the ACI code to account for the shear resistance provided by shear stirrups in addition to the diagonal cracking strength of concrete is provides a reasonably conservative estimate of shear capacity.

5.1.2 Some Discussion on ACI Provision on Shear Strength of Reinforced Concrete (RC) Beams

On the basis of the discussion above, it can be surmised that current ACI provision for the calculation of shear strength of RC beams is overly conservative. It can be attributed to the following factors.

1. 45 degree truss model

ACI recommendation for the adoption of 45 degree truss model may not be truly representative of the actual shear resisting mechanisms. It was found that angle of inclination of concrete is generally not 45 degree but may vary between 25 to 65 degrees [71]. In European practice, the angle of diagonal compression is taken as low as 18 degrees [72]. Modified compression field theory (MCFT) has also gained a lot of attention in recent times as a consistent mechanistic model to describe shear behavior of RC beams and has already gone through code-based implementation [73]. In this model this angle is determined by considering the calculated longitudinal strain at mid depth of the member. This model entails as the longitudinal strain becomes larger (as the moment increases) the value of the
angle of inclination increases. As per the model, this results in the subsequent reduction in shear resistance contributions from both concrete and shear stirrups.

2. Diagonal Cracking Strength of Concrete

ACI 318 addresses the concrete contribution to shear strength by assuming that the shear resistance at ultimate is equal to the diagonal cracking load. Experimental data sufficiently supported this argument even though there is no rational theoretical basis to back this assumption [72]. AASHTO standard specifications are also based on similar arguments. It also suggests that shear resistance provided by concrete is taken to be the lesser of the force required for web-shear cracking or flexure-shear cracking.

5.1.3 Shear strength of RC beams without Shear Reinforcement

There has been considerable development in formulating a rational mechanistic model for shear strength of transversely reinforced beams (compression-field based theories, fixed angle softened truss model, MCFT). In the case of concrete beams which are not provided with shear stirrups, this is still a very turbulent issue [74]. Currently there is no generally recognized physical model to explain the structural response of such type of beams [47].

As per the ACI building code, the cracking shear strength of RC beams without transverse reinforcement is dependent on longitudinal reinforcement ratio, concrete’s compressive strength and shear span to depth ratio. It should be noted that ACI code is only valid for cracking shear strength whereas well recognized Zsutty’s equation is valid
only for ultimate cracking strength [47]. As in the general case, it assumes shear strength to be proportional to square root of concrete’s compressive strength.

5.1.4 Size-Effect in RC Beams without Shear Reinforcement

The size effect in RC beams is a sufficiently well recorded phenomenon. As per the study done by Shioya [72] in which they tested RC beams without transverse reinforcement with depths ranging from 4 to 118 inches, it was observed that shear stress at failure decreases as the depth of the member increases. In the given test, all the members were lightly reinforced in longitudinal direction and were subjected to uniformly distributed load. For the conventional isotropic material, however, it can be said that shear strength is proportional to the depth of the member. This entails that due to inherent heterogeneity in its material properties, concrete member without shear reinforcement may act in a way that may run counter to our intuition. It further demands the need of more general rational theories that may account for such observations. In the conventional four point bending test, the size effect may be visible in the form of shear span to depth ratio.

It is a well-recognized phenomenon that shear strength of RC beams without transverse reinforcement is strongly dependent on shear span to depth ratio. This can also be regarded as an indirect measure of size-effect. An interesting phenomenon called Kani’s valley was observed during the study conducted by Leonhardt and Walther [47]. During the study, several tests were performed on RC beams with a/d ratio varying from 1.5 to 8. For the smaller a/d values, the beams reached their flexural strength as
expected. It was recognized by Kani that shear span to depth ratio of 2.5 is most critical in terms of shear strength [40].

For larger values of shear span to depth ratio (a/d) diagonal tension cracks developed resulting in shear failure. However, for very large a/d value, flexural strength is reached before the critical crack develops. This observation again runs counter to our intuition of shear failure to be the governing parameter as the a/d ratio indefinitely increases. This may be due to the effectuation of different resisting mechanisms with different a/d ratio which can’t be explained in terms of classical mechanics of materials.

Bazant’s equation considers the size effect on shear strength based on non-linear fracture mechanics [75]. Beforehand the size effect phenomenon was generally investigated based on the light of Weibull theory which assumes that failure occurs right at the initiation of a microscopic crack and size effect is assumed to be caused by randomness of local material strength. As per the study conducted by Bazant, the Weibull-type theories does not capture the essential cause of the size effect for quasibrittle material like concrete but was attributed to the global release of stored energy from the structure as a result of large fracture and the associated redistribution of stresses [76].

Based on the non-linear fracture mechanics and fractal geometry, Bazant came up with some interesting observations. For brittle failure of geometrically similar quasibrittle materials such as concrete, it was observed that the crack lengths at maximum load are approximately geometrically similar. In case of failure like diagonal shear of beams (which is of particular interest here) it was found out that they are geometrically similar to other failures like punching of slabs, torsion and anchor pull-out. Based on
these observations, Bazant tried to correlate the size-effect associated with brittle failure of materials like concrete with the fractal aspect of the morphology of cracked surfaces. However it was found out that size effect can’t be explained based on the fractality of cracked surfaces.

However, it should be noted that with the addition of minimum amount of transverse reinforcement, the size-effect seems to disappear [40].

5.1.5 Different Models for Shear Strength of RC Beams without Web Reinforcement

1. ACI Code Equation [77]

\[ V_c = \left( 0.16 \sqrt{f'_c} + 17 \frac{V_{ud}d}{M_U} \right) b_w d \text{ (in Newtons) for } \frac{a_s}{d} > 2.5 \]  
\[ \text{for } \frac{a_s}{d} \leq 2.5 \]  

2. Canadian Code [77]

\[ V_c = 0.2 \sqrt{f'_c} b_w d \text{ (In Newtons)} \]  

3. CEP-FIP Model [77]

\[ V_c = \left[ 0.15 \left( \frac{3d}{a_s} \right) \left( 1 + \sqrt{\frac{200}{d}} \right) \left( 100 \rho f'_c \right)^{1/3} \right] b_w d \text{ (In Newtons)} \]  

4. Zsutty Equation [77]

\[ V_c = 2.2 \left( f'_c \rho \frac{d}{a} \right)^{1/3} b_w d \text{ (In Newtons) for } \frac{a}{d} \geq 2.5 \]  
\[ \text{for } \frac{a}{d} < 2.5 \]  

5. Bazant Equation [77]
\[ V_c = \left[ 0.54 \sqrt[3]{\rho} \left( \sqrt{f_c'} + 249 \frac{\rho}{(a/d)^3} \right) * \frac{1 + \sqrt{\frac{5.08}{d_0}}}{\sqrt{1 + \frac{d}{(25d_0)}}} \right] b_w d \text{ (In Newtons)} \quad (45) \]

**Notations:**

\[ f_c' = \text{Compressive strength of concrete at 28 days (MPa)} \]

\[ b_w d = \text{Width and depth of cross-section (mm)} \]

\[ \frac{a}{d} = \text{Shear span to depth ratio} \]

\[ \rho = \text{Longitudinal reinforcement ratio} \]

### 5.1.6 Shear Strength of Fiber Reinforced Concrete Beams

It is a generally accepted fact that shear failure of a RC beam without shear reinforcement initiates when the principal tensile stress in the shear span exceeds the tensile strength of the concrete thus resulting in diagonal crack propagation [78]. Though the diagonal crack initiation is hard to define and depends on the subjective judgment of the observer’s itself, diagonal crack is defined as the major inclined crack extending from the level of longitudinal reinforcement towards the point of load application [74]. The code based equations for RC beams without shear stirrups are based on the assumption that useful shear strength is exhausted with the development of inclined crack [74]. It is a well observed fact that shear failure without stirrups is a non-ductile, brittle failure [41].

Regarding the above stated facts, the concept of fiber reinforced concrete was introduced. Addition of fibers to the concrete matrix is very effective in changing the failure mode from brittle to ductile which is generally associated with diagonal shear
failure of concrete beams [40]. Moreover, tests on FRC beams as a substitute for stirrups have been reported [41]. With the proper mix design and proper fiber selection, the ductility and shear strength of a RC beams can be considerably enhanced [42]. It was reported that addition of steel fibers can mitigate excessive diagonal cracking and localization of the tensile crack damage [79]. As per Minnelli [40] addition of small fraction of fibers significantly enhances the structural behavior of concrete members without shear reinforcement and ensuring that the material is tough enough, the collapse mechanism can be changed from shear to flexure.

The use of fibers can also be particularly effective in the case of high strength concrete which can be considerably brittle without minimum transverse reinforcement [44]. Increased shear strength of the FRC concrete members can be attributed to the post cracking tensile strength of the concrete. The fibers across the cracked surface will carry more stress called crack-bridging stress owing to its better tensile properties than the concrete. Vertical component of the post-cracking tensile load is likely to contribute to shear strength [79].

5.1.7 Some Prevalent Empirical Equations for the Prediction of Shear Strength of FRC beams without Web Reinforcement

1. Sharma [44]

\[ v_u = k f_t \left( \frac{d}{a} \right)^{0.25} \]  \hspace{1cm} (46)

\[ v_u = \text{average shear stress at shear failure} \]

\[ k = \frac{2}{3} \]
\[ \frac{a}{d} = \text{shear span to depth ratio} \]

\[ f_t^f = \text{split cylinder strength if known} \]

\[ = 0.79 \left( f'_c \right)^{0.5} \text{MPa} \]

2. Narayan and Darwish [44]

\[ v_u = e \left[ 0.24 f_{spfc} + 80 \rho \frac{d}{a} \right] + 0.41 \tau F \] (47)

\[ e = 1 \text{ for } \left( \frac{a}{d} \right) > 2.8 \]

\[ e = 2.8 \left( \frac{d}{a} \right) \]

3. Khuntia et al.[44]

\[ v_u = (0.167 + 0.25F) \sqrt{f'_c} \] (48)

4. Ashour et al.[44]

\[ v_u = \left( 0.7 \sqrt{f'_c} + 7F \right) \frac{d}{a} + 17.2 \rho \frac{d}{a} \] (49)

5. Kwak et al.[44]

\[ v_u = 2.1e \rho \left( f_{spfc} \right)^{0.7} \left( \rho \frac{d}{a} \right)^{0.22} + 0.8(0.41 \tau F)^{0.97} \] (50)

\[ e = 1 \text{ for } \left( \frac{a}{d} \right) > 3.5 \]

\[ e = 3.5 \left( \frac{d}{a} \right) \]

Notations:
\( f'_c = \text{Concrete compressive strength (MPa)}, \)

\( a/d = \text{Shear span to depth ratio} \)

\( F = \text{fiber factor} \left( \frac{L_f}{D_f} \right) V_f \)

\( f_{spfc} = \text{computed value of split cylinder strength of fiber concrete (MPa)} \)

\( \tau = \text{interfacial bond strength between fiber and concrete (MPa)} \)

### 5.1.8 Shear Strength of FRP Reinforced Beams without Transverse Reinforcement

ACI Committee 440 equation for the shear strength of FRP reinforced beams is based on the assumption that shear strength of FRP reinforced concrete beams varies proportionally with the shear strength of identical steel reinforced beams as the function of the ratio of axial stiffness of FRP bar to that of corresponding steel bar.

\[
V_{c,f} = \left( \frac{\rho_f E_f}{\rho_s E_s} \right) V_c
\]  

(51)

The value of \( \rho_s \) was taken as the half of maximum reinforcement allowed by ACI 318 or 0.375 \( \rho_{sb} \), considering a typical steel yield strength of 420 MPa for flexural reinforcement, the ACI Committee 440 recommended shear strength provided by concrete is as follows:

\[
V_{c,f} = \frac{\rho_f E_f}{90 \beta_1 f_c} \left( \frac{\sqrt{f'_c b_w d}}{6} \right) \leq \frac{\sqrt{f'_c b_w d}}{6}
\]

(52)

ISIS-M03-01 design manual
The shear strength of FRP reinforced beams is calculated based on the same principle as for steel reinforced beams with a reduction factor of the square root of the ratio of stiffness of FRP bar to steel bar to address the reduced stiffness of FRP reinforced beams.

\[ V_{c,f} = 0.2\lambda \phi_c f'_c b_w d \frac{E_f}{E_s} \]  

(53)

For sections with an effective depth greater than 300mm, the concrete shear resistance is taken as

\[ V_{c,f} = \frac{260}{1000+d} \lambda \phi_c f'_c b_w d \frac{E_f}{E_s} \geq 0.1\lambda \phi_c f'_c b_w d \frac{E_f}{E_s} \]  

(54)

Tureyen and Frosch provided the following simplified model that calculates the concrete contribution to shear strength of reinforced concrete beams. It is simplified to provide a general equation applicable to both steel and FRP reinforced beams as follows

\[ V_c = 5 f'_c b_w c (in - lb \ units) \]  

(55)

where \( c = kd \) such that \( k = \sqrt{(2\rho n + (\rho n)^2)} - \rho n \)

Based on the studies conducted on GFRP and CFRP reinforced beams by A.K.EL-Sayed [81], it was concluded that concrete shear strength of concrete beams reinforced with FRP bars to that of beams reinforced with steel is proportional to the cube root of axial stiffness ratio between FRP and steel reinforcing bars i.e
\[ V_{c,f} = \frac{3}{\sqrt{\rho_s E_s}} V_c \]  
(56)

Such that the following model was proposed

\[ V_{c,f} = \frac{3}{\sqrt{90 \beta_1 f_c}} \left( \frac{f_c' b_w d}{6} \right) \leq \frac{f_c' b_w d}{6} \]  
(57)

Works by A. Muttoni [47] on the shear strength of members without transverse reinforcement as function of critical shear crack width is also worth mentioning. The shear strength of members without stirrups which is a function of square root of the concrete compressive strength is strongly dependent on critical shear crack width and on its roughness. Critical shear crack theory represents this relationship as

\[ \frac{V_R}{bd} = \sqrt{f_c'} f (w, d_g) \]  
(58)

Where \( w \) is the critical shear crack width. Critical crack width is proportional to the product of longitudinal strain in the control depth (taken as 0.6d from the compression face) times the effective depth of the element. Shear strength is checked at the section which may be critically representative for given loading and geometry. And \( d_g \) is the maximum aggregate size.

The following analytical expression was proposed to evaluate the shear strength:

\[ \frac{V_R}{bd \sqrt{f_c'}} = \frac{1}{6} \cdot \frac{2}{1 + 120 \left( \frac{e_d}{16 + d_g} \right)} \text{ SI units} \]  
(59)
5.2 Fiber Reinforced Concrete in the Context of Punching Shear Response

Punching shear failure is a local failure that happens at the slab and column connection. It is a brittle and catastrophic failure which is generally smaller than the flexural failure load of the slab or the bridge deck. Punching shear failure of reinforced concrete bridge decks supported by girders occurs when the flexural strength of the deck exceeds the shear strength of the concrete due to higher reinforcement ratio leading to brittle failure. It is more prominent in the case of flat slabs. The appropriate choice of slab thickness for flat slabs is governed by serviceability conditions and punching shear [47]. As per the study, the transition of failure modes from flexure to punching shear is governed by the effective reinforcement ratio. They performed the load-rotation analysis for their study.

For lower reinforcement ratio (0.5 %), the test slab showed ductile behavior followed by the overall yielding of the flexural reinforcement. The slab reached its flexural strength and punching shear occurred after large plastic deformation leading to sudden brittle failure. For moderate reinforcement ratio (0.5 to 1%) some yielding of the flexural reinforcement in the vicinity of the column. The punching of the slab was observed before the yielding of overall reinforcement such that the strength of the slab is lower than its flexural strength.

For large reinforcement ratios (2.1%) punching failure occurs before any yielding of the reinforcement leading to a brittle failure. It was observed that strength of the slab is significantly lower than its flexural capacity. This response is similar to that of reinforced concrete beams which tend to fail in a brittle manner followed by concrete reaching
maximum strain if it is over reinforced. For the test designed to investigate the punching failure, it is advisable to have a high reinforcement ratio to make sure that the failure is not ductile or by flexure. The role of shear reinforcement is also an important factor affecting the failure mode. With lesser reinforcement ratio, the failure may be completely ductile owing to the flexural failure, however, the shear reinforcement may be erroneously judged responsible for the observed ductility.

Punching shear failure in most of the cases generally occurs in a brittle manner as previously discussed with the failure cone extending from the column edge to the tensile face of the slab. It appears as the circumferential crack in the slab [48]. It was reported by Matthew and Taerwe that the mean angle of the failure cone was 30.5 degrees for steel reinforced concrete beams. This angle however, was found to be decreasing with FRP reinforced concrete slabs.

Most of the design codes and formulae for punching shear are based on the experiments on thin slabs, however, are also applicable for thick slabs and footings. However, it should be noted that the reinforcement ratio and size-effect factors must be properly addressed to characterize the punching shear failure of the flat slabs and bridge decks. Many of the European codes and Canadian codes are found to have incorporated these factors.

5.2.1 Critical Shear Crack Theory

Based on the work done by A.Muttoni [47], a so called critical shear crack theory was proposed by him to account for the punching shear failure of RC slabs. Based on the load-rotation analysis performed on the slabs, it was observed that the punching shear
strength decreases with increasing rotation of the slab. This was accounted by the reduction on shear strength by the presence of a critical shear crack that propagates through the slab into the inclined compression strut carrying shear force to column. During the investigation, the tested slabs were reinforced with concentric rings placed at the boundary of the slab element only. The particular layout was designed to mitigate the formation of circumferential or circular cracks in the critical region thus only favoring the development of radial cracks. Similarly a slab with similar configuration but with an additional ring at the critical region was also tested. It was observed that the punching shear strength was increased significantly with the mitigation of critical shear cracks.

Based on the kinematics of slab rotation, it was inferred by Muttoni that the opening of the critical shear crack reduces the strength of the inclined compression strut which is responsible for transferring the shear force to the column. It was also assumed that the width of the critical crack is proportional to the product of slab rotation and the effective depth. The following relationship was proposed for the failure criterion.

\[
\frac{V_R}{b_0 d_f f_c} = \frac{3/4}{1 + 15 \frac{\Psi d}{d_{g0} + d_g}} \quad (\text{Psi, in}) \quad (60)
\]

Where \(d_g\) is the maximum aggregate size and \(d_{g0}\) is the reference size equivalent to 16mm (0.63in). \(b_0\) is the critical shear perimeter and \(d\) is the effective depth of the slab. The slab rotation can be obtained from direct measurement or calculated from the measured deflection assuming a conical deformation case. Considering the maximum aggregate size, it can be observed that the size-effect factor has also been addressed. The following analytical expressions were proposed for load-rotation relationships.
\[ V = \frac{2\pi}{r_q - r_c} EI \Psi \left( 1 + \ln \frac{r_s}{r_0} \right) \] for elastic regime (when the radius of yielded zone is less than or equal to radius of critical crack) \hspace{1cm} (61)

\[ V = \frac{2\pi}{r_q - r_c} EI \Psi \left( 1 + \ln \frac{r_s}{r_y} \right) \] For elasto-plastic region (radius of yielded zone greater than radius of critical crack but lesser than the length of the slab). Where \( r_s \) defined by the point of contra flexure of the radial moments and is given as (as per the linear-elastic estimate) \hspace{1cm} (62)

\[ r_s = 0.22L \] Where \( L \) is the axis-to-axis spacing of the column. \hspace{1cm} (63)

The flexural strength of the slab specimen is reached when the radius of the yielded zone equals the radius of the slab \( r_s \) and is given by

\[ V_{flex} = 2\pi m_R \frac{r_s}{r_q - r_c} \] Where \( m_R \) is the negative of radial moment per unit length. \hspace{1cm} (64)

5.2.2 Influence of Slab-Thickness and Size Effect

It has been observed by different researchers that the slab-thickness also plays an important role to define the punching shear strength of the slabs. Even with a lower reinforcement ratio, the thicker slabs have lesser rotation capacity such that they tend to fail in a brittle manner compared to their thinner counterparts. As reported by Muttoni [47], it was observed that for thinner slabs with high reinforcement ratios, the mode of failure is brittle and the ACI was found to be under predicting.
For the case of thicker slabs at lower reinforcement ratios, ACI was found to be over predicting. It was found out that for slabs thicker than 16 inches ACI overestimates the punching shear strength and does not ensure ductile behavior. This may be particularly significant for the cases of thick slabs and foundations mats which generally exceed this limit.

ACI design equation is based on Moe’s equation which postulates the punching shear strength to be a function of the ratio of punching shear strength to flexural strength of the slab. The relationship was found to be linear. This relationship also helps to quantify the effect of bending reinforcement. Increasing the reinforcement ratio increases punching shear capacity but decreases the ratio of punching shear strength to the flexural strength at the same time which leads to much stiffer response thus ensuing brittle failure. This also states the importance of shear reinforcement on such cases.

Apart from the slab and column geometry, it has been observed by different researchers that the tensile strength of the concrete which finally governs the shear strength, is influenced by design parameters such as the reinforcement ratio, maximum aggregate size effect and geometrical size effect [51]. For instance Menerey has proposed the following factors to account for the reinforcement, aggregate size and slab-column geometry.

\[
\xi = \begin{cases} 
-0.1\rho + 0.46\rho + 0.35 & \text{for } \rho < 0.02 \\
0.87 & \text{for } \rho \geq 0.02 
\end{cases}
\]

\[
\mu = 1.6\left(1 + \frac{d}{d_1}\right)^{\left(-1/2\right)}
\]  

(65)  

(66)
\[ \eta = 0.1 \left( \frac{rs}{h} \right)^2 - 0.5 \left( \frac{rs}{h} \right) + 1.25 \text{for} \left( \frac{rs}{h} \right) < 2.5 \]

\[ \eta = 0.625 \text{for} \left( \frac{rs}{h} \right) \geq 2.5 \]  

Equation (67)

The failure surface as previously discussed (based on the experimental works) is generally assumed to be a conical section inclined at a particular angle. H.Oh [51] has assumed the failure surface to vary tri-linearly, that is, the failure surface can be assumed to be composed of three different surfaces with varying inclinations. As per the author, the failure surface can be divided into three parts. The first part consists of the top concrete surface with a particular inclination, the second part which basically includes the punching cone is inclined at different inclination and the portion of the bottom concrete is assumed to be inclined at different inclination.

The underestimation of the punching shear strength of a traditional RC slab was attributed to the discrepancy created by neglecting the dowel action of the rebar [51]. Regan and Braestrup have proposed that the dowel action increases the punching shear strength of an orthogonally reinforced concrete slab by 34%. The following Dowel action relationship has been proposed by Millard and Johnson and Menerey [51],

\[ V_{dow} = \frac{1}{2} \sum \phi_h \sqrt{2} f_{y1} f_{y2} \left( 1 - \zeta^2 \right) \sin \theta \]  

Equation (68)

Where \( \zeta \) is the ratio of failure stress of steel to its yield stress. The steel stress at failure is further given as

\[ f_s = \frac{V_{pun} \cot \theta}{\sum As} \]  

Equation (69)
Where the denominator is the sum of the cross sectional areas of the reinforcing steel bars embedded in the punching cone.

5.2.3 Effect of Column Geometry on the Punching Shear-Strength

The column size was also found to be influential on the failure load by apparently increasing the critical effective shear perimeter. It was also observed [52] that high flexural stress gradients exist around the shorter side of the column section thus signifying that the critical location for punching shear should be around the shorter side of the rectangular column. It was observed that the flexural stress close to the column face is higher than it is the farther away.

The assumption of uniform shear stress along the critical shear perimeter is reasonably justified in the case of square column section. However, in the case of rectangular column section, as it was discussed, the shear stress intensity varies along the critical shear perimeter. A factor was introduced by S.Teng to account for the rectangularity of the column section such that the conservative prediction of the shear capacity is avoided.

\[ v_u = \beta \frac{V_u}{b_0 d} \]  

(70)

Where

\[ \beta = \left( \frac{b_2}{b_1} \right)^{(1/4)} \]  

(71)
It was also observed by the author that the presence of an opening in the vicinity of the slab-column section tends to affect the punching shear strength of the system. The research depicted that when an opening is located at a distance less than ten times the effective depth of the slab from the edge of the column or concentrated load, then the effective critical parameter should be reduced. A simple graphical method was also proposed to account for the presence of an opening. As per the author, the part of the perimeter that is enclosed by the radial projections from the centroid of the end portion of the column to the edge of the openings to be ineffective. For the rectangular column-slab system that failed in punching shear, the lowest capacity is found with an opening along the shorter side of the column section.

The literature on punching shear strength can be broadly categorized into two parts. The first category includes the empirical relationships that associate the strength to the geometric and material properties. The second category is the analytical mechanics-based models based on different boundary conditions and material properties. In the analysis of steel reinforced RC slabs, the punching shear strength is considered to be the result of the interaction of the two critical cracking-shear and flexural [48]. Failure is assumed to occur in the concrete’s compression zone above the critical shear and flexural cracks when the shear stress reaches the tensile splitting strength of the concrete. The punching failure load is given by

\[ V_u = m.v \cdot b_p \cdot X \cdot \cot \theta \]  

(72)

Where the limiting shear stress at the compression as the function of the concrete’s cube strength is given as
\[ \nu_c = 0.27 \sqrt{f_{cu}} \]  

(73)

Where \( m \) is the coefficient equal to 1 for normal weight and 0.8 for lightweight concrete and \( \theta \) is the angle of the failure surface of the conical surface. The dowel action is accounted by the factor \( b_p \) which is given as

\[ b_p = 4c + 12d \]  

(74)

Where \( c \) is the diameter or the side width of the column and \( d \) is the effective depth of the slab. The shear-flexure interaction is expressed as the combined effective neutral axis depth taken as the harmonic mean of the shear-section neutral axis depth and neutral axis of the flexural section.

\[ X = \frac{2X_sX_f}{X_s + X_f} \]  

(75)

Where

\[ X_s = 0.25 d \]  

(76)

\[ X_f = \frac{\rho_s f_s - \rho_t f_t}{k_1 f_{cit}} d \]  

(77)

Where \( \rho_s \) and \( f_s \) are the reinforcement ratio and the working stress in the tension zone and the second part is the corresponding values for compression reinforcement. \( k_1 \) is the equivalent rectangular compression stress block as per BS 8110-97. A correction factor for the depth of the beam is also given as
\[ \xi_s = \sqrt[6]{\frac{100}{d}} \]

(78)

5.2.4 Punching Shear Strength of FRC Slabs

FRC concrete is being extensively used these days owing to the enhancement of post-cracking tensile behavior of the concrete, increase in deformation-capacity and various other meritorious features. Addition of fiber has been found to increase the energy absorption such that the post-cracking behavior is significantly modified. Since punching shear failure is also a brittle failure, it is a general consensus among the engineers that the FRC concrete can also be used to address this issue. As per the experimental program conducted by A.E.Naaman [46], it was inferred that the punching shear resistance, energy absorption capacity and their resistance to spalling of HPFRCC (high performance fiber-reinforced cementitious composites) slabs with two bottom layers of reinforcing bars were significantly better than the specimen with four layers of reinforcing bars and regular concrete.

It was reported by Alexander and Simmonds [53] that the an increase in the punching-resistance of slab-column connection by almost 30% was observed by the addition of corrugated steel fibers as well as improved ductility of the connection. Haraji also reported a 40% increase in ultimate punching strength by adding 2% hooked steel fibers by volume. The increase in shear-capacity is indicated by the shallower angle of the punching shear surface as compared to the plain concrete [46]. The addition of fiber also may help to provide the transition of failure mode from brittle to ductile. By
providing fiber by approximately 2%, Shaban and Gesund also concluded that the punching shear failure of flat slabs changed from brittle to ductile.

From the load-displacement plots, Naaman also showed that the maximum load reached was significantly higher in the case of HPFRCC slabs in comparison to the control steel reinforced conventional slabs. From the plots, it was observed that the peak load was much higher with increased energy absorption capacity such that more ductile failure can be anticipated. The critical shear perimeter and $\beta$ factor which is the ratio of average shear stress at failure to the square root of the compressive strength of concrete were found to be different as that predicted by ACI. The actual critical perimeters were found to be higher than that provided by ACI and the $\beta$ factors seem to be overestimated by ACI.

Based on the limited experimental results, it was concluded that for HPFRCC slabs with similar geometric properties, the minimum punching shear resistance using the critical perimeter in accordance with ACI can be safely taken as $2 \times \sqrt{f'_c}$.

5.2.5 Punching Shear Models

ACI 318-02

The punching shear capacity of a non-pretressed steel reinforced slab is given by the smallest of
\[ V_{ACI} = \left( 2 + \frac{4}{\beta_c} \right) \sqrt{f'_c u_{0.5} d} \]

\[ V_{ACI} = \left( \frac{\alpha_s d}{u_{0.5}} + 2 \right) \sqrt{f'_c u_{0.5} d} \]

\[ V_{ACI} = 4 \sqrt{f'_c u_{0.5} d} \tag{79} \]

Where \( \beta_c \) is the aspect ratio of the column, \( \alpha_s \) is equal to 40 for interior columns and \( u_{0.5} \) is the critical shear perimeter taken at 0.5d from the column face.

**ACI 440**

\[ V_{440} = 10K \sqrt{f'_c u_{0.5} d} \]

\[ k = \sqrt{2\rho \eta + (\rho \eta)^2 - (\rho \eta)} \]

\[ \eta = \frac{E_d}{E_c} \]

\[ \rho = \frac{A_f}{b_d} \tag{80} \]

**Eurocode 2**

\[ V_{EC2} = 0.25 \frac{f_{ck}}{\gamma} K_{EC} (1.2 + 40 \rho) u_{1.5} d \]

*with*: \( f_{ck} = 0.7 f_{cem} \)

\[ f_{cem} = 0.33 \sqrt{f'_c} \]

\[ K_{EC} = (1.6 - d) \geq 1 \]

\[ \rho = \sqrt{\rho_x \rho_y} \leq 0.15 \tag{81} \]

Where the strengths are in Mpa and dimensions are in meter.

**BS 8110**
\[ V_{BS} = \frac{0.79 k_1 k_2}{\gamma} \sqrt[4]{100 \rho} \sqrt[4]{400 \frac{d}{u_{1.5}}} (d\text{mm}) \]

\[ K_2 = \frac{f_{ck}}{\gamma} \geq 1 (\text{MPa}) \]

0.0015 ≤ \( \rho \) ≤ 0.03

\[ \frac{400}{d} \geq 1 \]

**CEB-Model Code 1990**

\[ V_{MC90} = 0.12 \xi \sqrt[3]{100 \rho f_{ck} u_2 d} \]

\[ \xi = 1 + \sqrt[4]{\frac{0.2}{d}} \]

or

\[ V_{MC90} = 0.3 \left( 1 - \frac{f_{ck}}{250} f_{cd u_{1.5} d} \right) \]

(83)

Where strengths are in MPa and dimensions are in meters.


\[ V_{MT} = 1.36 \sqrt[3]{100 \rho f_{cm}} \sqrt[4]{u_{1.5} d} \]

(84)

Where \( d \) is in mm and strength in MPa.

All the discussions that follow is intended to develop a contextual idea about the strength characterization of concrete, reinforced concrete, FRP reinforced concrete and Fiber reinforced concrete materials. We had a brief overview of the prevalent ideas on
shear strength of concrete with various types of reinforcement; longitudinal steel bar, polymer bar and fibrous reinforcement.

This part of research program dealt with the strength characterization of minibar reinforced concrete beams. Minibar is a high strength synthetic fiber with moderate stiffness. It was known from the previous section that minibar is able to alter the toughness characteristics of concrete, increases the flexural tensile strength, post-cracking tensile strength and the overall load carrying capacity of the member. Since concrete is a brittle material and exhibit size-effect, it is necessary to study the behavior of minibar reinforced concrete beams at bigger scale. Based on the literature review, an experimental program was designed which comprised of minibar reinforced concrete beams of varying dimensions, shear span to depth ratios and minibar dosage. The program was intended to provide the insight into the flexural and shear behavior of minibar reinforced concrete beams, development of analytical model and its possible correlation with material and toughness characterized.

5.3 Materials

5.3.1 Basalt Minibar

The details of basalt minibar is given in Chapter II. Basalt minibar is a new form of composite fibrous reinforcement.
5.3.2 Coarse Aggregate

The coarse aggregate used in the tests was crushed limestone with a maximum size of 20 mm (~0.75 inch) for normal concrete mix. Coarse aggregate was obtained from a local Ohio supplier. Approximate absorption was 0.3 to 0.4% as suggested by the supplier. Smaller size (pea gravel) of limestone of a maximum size of 8 mm (~0.33 inch) was used for dry mix concrete as specified and also, for mix with small aggregate.

5.3.3 Fine Aggregate

The fine aggregate was river sand purchased from the local supplier. The sand was free from clay and other inert impurities.

5.3.4 Cement

Cement used in this test program was Type I/II normal Portland cement. The properties were conforming to ASTM C150.

5.3.5 Water

The water used for the concrete mixes was the normal tap water supplied by the city of Akron. The water was clean and was usable for preparation of specimens.
5.4 Test Procedures

This sub-section comprises the entire laboratory tasks which were performed for the strength characterization of basalt minibar reinforced concrete (MRC) beams. Variables of interest were minibar dosage and shear span to depth ratios.

5.4.1 Preparation of Specimens

Preparation of specimen comprises preparing minibar reinforced concrete beams with dimensions as tabulated in Table 5.1 and Table 5.2. Table 5.1 presents the dimensional and reinforcement details of small-scale beams (laboratory scale) and Table 5.2 gives the description of large scale minibar reinforced concrete beams. Within the small scale beam series, there are three sets of beams, each series consisting of seven beams with varying shear span to depth ratios. The first set of beams was reinforced with only basalt minibar, the second set was steel-reinforced concrete beams and the third set was steel-reinforced beam reinforced with basalt mini-bar. All the beams were identical and the program was designed to develop a quantitative idea on the probable structural modifications on the structural behavior of steel reinforced concrete beams, aided by the addition of basalt mini-bar.
Table 5.1: Details of Small Scale Test Beams

<table>
<thead>
<tr>
<th>Set</th>
<th>Length, in</th>
<th>Width, in</th>
<th>Depth, in</th>
<th>Fiber Volume Fraction (%)</th>
<th>Steel Area (sq.in)</th>
<th>Shear Span, in</th>
<th>Shear Span to Depth Ratio (a/d)</th>
<th>No of Beams</th>
</tr>
</thead>
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<tr>
<td>I</td>
<td>26</td>
<td>4</td>
<td>0.5</td>
<td>0</td>
<td>9.5</td>
<td>2.25</td>
<td>2</td>
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<tr>
<td></td>
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<td>3</td>
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<tr>
<td></td>
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<td>3</td>
<td></td>
<td></td>
<td>14</td>
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<td>2</td>
<td></td>
</tr>
<tr>
<td></td>
<td>32</td>
<td>4</td>
<td></td>
<td></td>
<td>13</td>
<td>2.75</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>II</td>
<td>26</td>
<td>4</td>
<td>0</td>
<td>0.22</td>
<td>9.5</td>
<td>2.25</td>
<td>2</td>
<td></td>
</tr>
<tr>
<td></td>
<td>50</td>
<td>3</td>
<td></td>
<td></td>
<td>22</td>
<td>7.00</td>
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</tr>
<tr>
<td></td>
<td>34</td>
<td>3</td>
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<td></td>
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<td>4.16</td>
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<td></td>
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<td>4</td>
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<td>13</td>
<td>2.75</td>
<td>1</td>
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</tr>
<tr>
<td>III</td>
<td>26</td>
<td>4</td>
<td>0.5</td>
<td>0.22</td>
<td>9.5</td>
<td>2.25</td>
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<td>7.00</td>
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<tr>
<td></td>
<td>34</td>
<td>3</td>
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<td></td>
<td>14</td>
<td>4.16</td>
<td>2</td>
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<td></td>
<td>32</td>
<td>4</td>
<td></td>
<td></td>
<td>13</td>
<td>2.75</td>
<td>1</td>
<td></td>
</tr>
</tbody>
</table>

Table 5.2 gives the dimensional and reinforcement details of large scale minibar reinforced concrete beams. These tests are intended to study:

1. Flexural and shear response of large-scale mini-bar reinforced concrete beams.
2. Possible alteration on the structural response of plain concrete beam due to addition of mini-bar.
3. To develop insight into possible size-effect on flexural tensile strength of large scale concrete beams.
5.4.1.1 Mixing of Concrete

Mixing of concrete was done in a 9 cubic feet (0.25 m³) capacity drum mixer in the concrete laboratory of the University of Akron. The standard mixing procedure was used with mixing time of 1-3-3-2 minutes so that the Minibar was dispersed evenly. Figure 5.1 and Figure 5.2 shows the mixing of small scale and large scale beams.

Table 5.2: Dimensional Details of Large Scale Beams

<table>
<thead>
<tr>
<th>Length, in</th>
<th>Depth, in</th>
<th>Width, in</th>
<th>Fiber Volume Fraction (%)</th>
<th>Shear Span, in</th>
<th>Shear Span to Depth Ratio</th>
<th>No of Beams</th>
</tr>
</thead>
<tbody>
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<td></td>
<td>0.75</td>
<td>53</td>
<td>6.6</td>
<td>1</td>
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<tr>
<td>118</td>
<td></td>
<td></td>
<td>0.5</td>
<td>53</td>
<td>6.6</td>
<td>1</td>
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<td>0.5</td>
<td>21</td>
<td>2.6</td>
<td>2</td>
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<tr>
<td>59</td>
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<td></td>
<td>0</td>
<td>21</td>
<td>2.6</td>
<td>1</td>
</tr>
<tr>
<td>42</td>
<td></td>
<td></td>
<td></td>
<td>13</td>
<td>1.6</td>
<td>1</td>
</tr>
</tbody>
</table>
Figure 5.1: Placing Concrete for Small Scale Beams

Figure 5.2: Placing Concrete for Large Scale Beams
Figure 5.3: A typical Large Scale Beam

From Figure 5.2 it can see that the complete casting of large scale beams could not be done in one mix. Four mixes were consecutively prepared to complete the casting of a single beam. The concrete cylinders for compressive strength test were stored in the curing-room with complete humidity. However, the large beams were cured outside the curing room because they were too large. Figure 5.3 shows a typical large scale minibar reinforced concrete beam. The beams were provided with steel hooks at specified position to facilitate the lifting and subsequent movement of the beams within the laboratory. Since there was not enough space to store the large scale beams inside the humid rooms, the large scale beams were covered with burlap and were periodically sprayed with water for curing. It can be seen in Figure 5.4 that the beams were carefully
put on the wheels to facilitate the transport and were covered with burlap that was wetted periodically.

![Curing of a Specimen](image)

Figure 5.4: Curing of a Specimen

5.4.2 Experimental Program

All the seven beams from the first set were instrumented with strain-gages. Two strain-gages were provided on top and bottom surfaces. The concrete surfaces were grinded, followed by the application of sand-papers of varying grades. After the surfaces were prepared, strain-gages were fixed on one side of the beam and were clamped in place using C-clamps. The strain-gages were allowed to set for 24 hours. Other sides of the beams were similarly instrumented with strain-gages and were let to set. It was later observed that some of the strain gages were not getting bonded with the concrete
surfaces. After the strain gages were fixed to the concrete surfaces, they were soldered with the electric wires to facilitate their connection to data acquisition system. This is depicted in Figures 5.5:

Figure 5.5: Preparation of Strain Gages on Concrete Beam Surface
The first series of beams were tested on BALDWIN test machine under four pointing. Figure 5.6 depicts the schematic of four point bending performed on small and large scale beams.
Figure 5.6: General Schematic of Four-Point Bending Test Arrangement for Small Scale Beams

Baldwin testing machine has a 300,000lb-capacity. The type of the machine is UTM (model 300HV-300,000LB capacity). Special I-sections were provided to act as a spreader beam as reaction beam to provide the rigid substratum for the roller supports. This arrangement is supposed to ensure the four point bending condition. The beams were loaded at a uniform rate of loading, which generally varied from 10 to 15 lb/s. Deflection data were manually recorded from the dial gage attached to the centre of the beam at the bottom. Dial-gage used for the purpose is a dial-gage of a brand called FOWLER with the least-count of 0.0005 in. Two dial gages were provided at the top surface of the beams. The beams were tested to failure and the continuous load-strain data were acquired through the strain gages connected to ADMAT data acquisition system. The two dial-gage readings were recorded manually at a fixed interval of loadings.

Figure 5.7 depicts a general schematic of the test set-up for four-point bending test on large scale beams. The tests were conducted in civil-engineering materials lab at University of Akron. Load was applied using a manually operated hydraulic jack. Hydraulic jack was manually actuated and rate of loading was maintained as uniform as
possible. Deflection data was manually recorded at regular interval from dial-gage which was provided at the top surface of the beam. A continuous video of the pressure-gage was also made to record maximum load.

Figure 5.7: General Schematic of Four-Point Bending Test Arrangement for Large Beams

The actual test set up is shown in figure 5.8 for small and large scale beams.

Figure 5.8: Actual Test Set Up for Four-Point Bending Test for Large Beams
5.5 Results and Analysis

5.5.1 Small Scale Beams

This section will include the detailed description of the results of experimental and analytical works done in regard to small scale beams. This comprises experimental load-deflection plots, experimental load-strain plots and analytical work done in regard to minibar reinforced flexural and shear characterization.

5.5.1.1 Load-Deflection Plots

Figure 5.9 shows the experimental load-deflection plots obtained from four point bending on small scale beams from first set that is, only reinforced with minibar. It can be
seen from the plots that plain concrete beams when reinforced with minibar, exhibit good load deflection characteristics and energy absorption capacity. It can be observed that the deflection of beams is reducing marginally after cracking, in contrast to the plain beam counterparts where there will be an abrupt and brittle drop in load carrying capacity after cracking. The presence of minibars at the crack plains actuates crack-bridging mechanism, thus making the load-deflection response more elastic and tough. This is significant from the view point that the addition of minibars for even at a small dosage of 0.5 % is capable of modifying the structural response of plain concrete beams and make them less brittle. The presence of minibar is able to develop the new energy-distribution mechanisms after concrete cracking, thus rendering them improved energy absorption capacity.
Figures 5.10, 5.11 and 5.12 show the load-deflection comparison of small scale beams from second and third set. Second set of beams were reinforced with longitudinal steel reinforcement, whereas third set is reinforced with longitudinal steel bars and minibar. From the figures it can be observed that due to the addition of minibar, the beams were capable of undergoing larger deformation as compared to steel reinforced beams. The improved response can be attributed to the development of better deformation capabilities that are achieved due to the addition of minibar.
Figure 5.10: Load-Deflection Comparison for Beams from Sets II and III
Figure 5.11: Load-Deflection Comparison for Beams from Sets II and III
Figure 5.12: Load-Deflection Comparison for Beams from Sets II and III

5.5.1.2 Load-Strain Plots

Figures 5.13 and 5.14 show the experimental load-strain plots obtained from the strain gage data. It was expected that with the addition of minbar, there will be an increase in the maximum tensile strain reached during failure. Load-deflection plot confirms that maximum deflection has increased with the addition of minbar, which further implies the increase in maximum strain capacity. With increasing deflection, there is every possibility of strain-gages malfunctioning. This might have led to the situation that the loads and strains values closer to failure load could not be sufficiently extracted due to inevitable deterioration of strain gages with increasing load.
Figure 5.13: Experimental Load-Strain Plots for first and Second Sets of Small Scale Beams
Figure 5.14: Experimental Load-Strain Plots Third Set of Small Scale Beams

From the load-strain analysis, it was observed that it is always advisable to have a load-deflection data system so that the improvement on deformation behavior of the material is judicially quantified. In Figure 5.15, it was observed that a minibar reinforced concrete is capable of undergoing larger deflection at ultimate failure and was capable to retain its sectional integrity due to the actuation of crack-bridging mechanism at the failure-plane. Keeping these observations on mind, it can be stated that Mini bar reinforced concrete beams have a better energy absorption capacity which can be further harnessed to serve various serviceability limit states under actual loading conditions.
Figure 5.15: Improved Ductile Response of Minibar Reinforced Concrete Beam
5.5.1.3 Flexural and Shear Strength of Minibar reinforced concrete beams

5.5.1.3.1 Flexural Strength of Minibar Reinforced Concrete Beams

The load carrying capacity of first set of small scale beams (only reinforced with minibars) were analytically studied using the approach called “Modified Discrete Fiber Property Approach” as developed in chapter 4. The results are tabulated in Table 5.3.

Table 5.3: Predicted load carrying capacity Using Modified Discrete Fiber Property Approach for first set of 0.5 % vf MRC beams (Compressive Strength ≥ 7000 psi)

<table>
<thead>
<tr>
<th>Shear Span to Depth ratio</th>
<th>Predicted (lb)</th>
<th>Experimental (lb)</th>
<th>correction</th>
<th>Corrected Prediction (lb)</th>
<th>% difference</th>
</tr>
</thead>
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<tr>
<td>2.25</td>
<td>1514</td>
<td>3000</td>
<td>1.70</td>
<td>2569</td>
<td>-14.36</td>
</tr>
<tr>
<td>2.75</td>
<td>1243</td>
<td>1900</td>
<td>1.54</td>
<td>1908</td>
<td>0.42</td>
</tr>
<tr>
<td>4.16</td>
<td>1150</td>
<td>1186</td>
<td>1.25</td>
<td>1435</td>
<td>21.02</td>
</tr>
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<td>7.00</td>
<td>720</td>
<td>800</td>
<td>0.96</td>
<td>693</td>
<td>-13.41</td>
</tr>
</tbody>
</table>

Note: For $f'_c \geq 7000$ psi, Ultimate load carrying capacity

\[
= (Predicted load carrying capacity) \times 1.8 \times \left( V_f \times \frac{a_s}{d} \right)^{-0.5}
\]

“Modified Discrete Fiber Property Approach” which account for the varying shear span to depth ratio, concrete’s compressive strength and minibar dosage is able to predict the load carrying capacity of minibar reinforced concrete beams with reasonable accuracy. The graphical comparison is shown in Figure 5.16.

It can be observed from Figure 5.16 that for a given minibar dosage and concrete compressive strength, the strength of the beams decreased with increasing shear span to
depth ratios. The “Modified Discrete Fiber Property Approach” which applies the mechanical properties of basalt minbar (as obtained from mechanical characterization from chapter 2) to account for the maximum load carrying capacity of minbar reinforced concrete beam, is able to model the flexural response of minbar reinforced concrete small scale beams with good accuracy.

Figure 5.16: Theoretical prediction of Ultimate Load Capacity of Beams from First Set

5.5.1.3.2 Shear Strength of Minibar Reinforced Concrete Beams

A mechanics based analytical shear-strength relationship was proposed for the shear strength characterization of minbar reinforced concrete beams. Once the first diagonal crack appears, reinforced beams loses their shear strength significantly due to
the increased crack-width thus reducing the aggregate interlock and dowel action. Hence
the ultimate shear strength can be assumed to be equal to cracking shear strength.
However, in the case of beams reinforced with minibar, the minibars at the crack plane
develop post-cracking tensile strength for which vertical component will account for the
increased shear strength of the beams. This is depicted in Figure 5.17.

Assumptions:
1. Aggregate interlock is negligible and not dealt analytically.
2. Shear resistance is provided by concrete in compression zone and by the post-
   cracking tensile strength of the minibar
3. Number of fibers crossing a unit area of crack may be taken as

   \[ N = \frac{0.5V_f}{\pi r^2_f} \]  

   (85)

4. Three different failure criteria for fibers are considered.
   a. As in the case of steel fibers, pull-out of the fibers invariably occurs (due to
      high stiffness) and it was assumed that mean fiber pull-out length is \( L_f/4 \).
   b. Considering the smaller stiffness of the non-metallic fibers, for instance,
      minibar, all the fibers fail in pure tension.
   c. It is assumed that half of the fibers fail by debonding and the rest by pure
      tension.

Ultimate shear strength of the concrete without transverse reinforcement can be
calculated based on the following relationships proposed by different researchers.

\[ V_{uc} = \left( 10\rho f'_c \left( \frac{d}{a_s} \right)^\frac{1}{3} \right) f'_{or} \frac{a_s}{d} > 2.5 \]  

(86)
\[ V_{uc} = (160 \rho f_c \left( \frac{d}{a_s} \right)^{\frac{1}{3}} \frac{a_s}{d} < 2.5 \]

\[ V_{uc} = 11.4e \left( \rho f_c \frac{d}{a_s} \right)^{\frac{1}{3}} \text{ where } e = 1 \text{ for } \frac{a_s}{d} > 3.5 \text{ and } \left( 3.5 \frac{d}{a_s} \text{ for } \frac{a_s}{d} \leq 3.5 \right) \] (87)

\[ V_{uc} = 3.7e \frac{2}{s_{spf}} \left( \rho \frac{d}{a_s} \right)^{\frac{1}{3}} \text{ where } e = 1 \text{ for } \frac{a_s}{d} > 3.5 \text{ and } \left( 3.5 \frac{d}{a_s} \text{ for } \frac{a_s}{d} \leq 3.5 \right) \]

\[ V_c = V_s + V_d + V_a + V_{cz} + V_f \]

Figure 5.17: Shear Resisting Mechanism in MRC Beam

Equating external moment to internal moment of resistance

\[ M_u = V_t \times a_s = V_{uc} \times (b \times d) a_s = \phi \times 0.85 \times f_c' \times a \times b \times \left( d - \frac{a}{2} \right) \]

where \( a = \text{ equivalent stress block depth } = \beta_1 C \)

or \( V_{uc} \times b \times d \times a_s = 0.85 \times f_c' \times a \times b \times \left( d - \frac{a}{2} \right) \)

or \( \frac{a}{d} = 1 - \sqrt{1 - 2.353 \left( \frac{V_{uc}}{f_c} \right) \frac{a_s}{d}} \) \text{ then } \( C = \frac{a}{\beta_1} \)

length of the inclined crack = \( \frac{(h-c)}{\sin \alpha} \)
effective area of the fiber contribution = \( b \times \frac{(h-c)}{\sin \alpha} \)

number of fibers crossing a unit area of crack may be taken as 
\[ N = \frac{0.5V_f}{\pi r^2_f} \quad (88) \]

Where,
\[ V_f = \text{volume} - \text{fraction} \]
\[ r_f = \text{radius of each fiber (minibar)} \]

Assumption-1

As in the case of steel fibers, due to higher stiffness, pull-out of the fibers invariably occurs and the mean pull-out length is \( L_f/4 \).

Average pull-out force per fiber (f) \( f = \tau \times \pi \times d_f \times \frac{l_f}{4} \) where

\[ \tau = \text{interfacial bond strength} \]

ultimate stress sustained by unit area of crack at failure (by fiber) \( \sigma_{cu} \)
\[ = N \times f = \frac{0.5V_f}{\pi r^2_f} \times \tau \times \pi \times d_f \times \frac{l_f}{4} = \frac{0.5V_f \times \tau \times l_f}{d_f} \]

total force perpendicular to crack \( F_{cu} = \sigma_{cu} b \times \frac{(h-c)}{\sin \alpha} \)

vertical component of the force is the contribution to shear resistance

provided by fiber \( V_f = F_{cu} \times \sin \alpha = \sigma_{cu} \times b \times (h-c) \)

unit strength provided by fiber, \( V_{uf} = \frac{V_f}{(b \times d))} = \frac{\sigma_{cu} \times (h-c)}{d} \)

Total shear strength of FRC - beam, \( V_U = V_{uc} + V_{uf} \)
Assumption-2

Regarding the lesser stiffness of non-metallic fibers as compared to steel fibers, it is assumed that all the fibers fail in pure tension.

From equilibrium condition \( \sigma_f \frac{\pi d_f^2}{4} = \tau \pi d_f l_f \)

or \( \tau = \frac{\sigma_f d_f}{4 l_f} \) where \( \sigma_f \) is the tensile strength of the fiber - strand

now \( \sigma_{cu}' = N * \tau = \frac{0.5 + 4V_f}{\pi d_f^2} * \frac{\sigma_f d_f}{4 l_f} = \frac{0.5V_f + \sigma_f}{S_f} \)

where \( S_f = \) surface area of individual fiber

then \( F_{cu} = \sigma_{cu}' * b * \frac{(h-c)}{\sin \alpha} \)

and \( V_{uf}' = \sigma_{cu}' * \frac{(h-c)}{d} \)

Total shear strength = \( v_{uc} + v_{uf}' \)

Assumption-3

It is assumed that half of the fibers fail by debonding and the rest by pure tension.

\( f' = \left( \frac{\sigma_f d_f}{4 l_f} + \tau \pi d_f \frac{l_f}{4} \right) * \frac{1}{2} = \frac{S_A}{8} * \left( \tau + \frac{\sigma_f}{\pi l_f^2} \right) \)

where \( S_A = \) surface area of individual fiber.

now \( \sigma_{cu}' = N * f' = \frac{0.5 + 4V_f}{\pi d_f^2} * \frac{\pi l_f d_f}{8} * \left( \tau + \frac{\sigma_f}{\pi l_f^2} \right) \)

if \( \frac{l_f}{d_f} = \) aspect - ratio of fiber = \( \beta \)

then \( \sigma_{cu}' = \left( \frac{1}{4} \right) * V_f * \beta * \left( \tau + \frac{\sigma_f}{\pi l_f^2} \right) \)
then \( F'_{cu} = \sigma'_{cu} \times b \times \frac{(h-c)}{\sin \alpha} \)

\( V'_{uf} = \sigma'_{cu} \times \frac{(h-c)}{d} \)

**Total shear strength** = \( v_{uc} + v'_{uf} \)

From the general observation, assumption 2 (failure due to minibar rupture) was found to apply better with minibar reinforced concrete beams. The analytical model presents the post cracking tensile strength of minibar reinforced concrete to be independent of concrete compressive strength and purely the function of minibar dosage and fiber geometry. This observation comply with an empirical model proposed in chapter 4 where post-cracking tensile strength of minibar reinforced concrete was expressed purely as a function of minibar dosage.

Table 5.4 shows the summary of four point bending test on three sets of small scale beams. It can be observed that for the beams failing in flexure, there is no improvement on the load carrying capacity of steel reinforced beams due to the addition of basalt minibar. However, beams designated as B-3, B-4 and B-5 which were observed to fail in shear, show increased load carry capacity relative to the corresponding beams without minibar addition.
Table 5.4: Summary of the Ultimate Load Carried by the Small Scale Beams

The theoretical shear-strength based on mechanics based model as described above was calculated and compared to experimental results. It is depicted in Table 5.5 and Figure 5.18.

Table 5.5: Theoretical Prediction of Shear Strength of MRC Beams

<table>
<thead>
<tr>
<th>MRC Beams (lb)</th>
<th>Steel Reinforced (lb)</th>
<th>(Mini-Bar + Steel) Reinforced (lb)</th>
<th>Length (in)</th>
<th>Depth (in)</th>
<th>Shear Span (in)</th>
<th>a/d</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-1 800 lb</td>
<td>4062</td>
<td></td>
<td>50</td>
<td>3</td>
<td>21</td>
<td>7.00</td>
</tr>
<tr>
<td>B-2 1020 lb</td>
<td>4325</td>
<td></td>
<td>50</td>
<td>3</td>
<td>21</td>
<td>7.00</td>
</tr>
<tr>
<td>B-3 3150 lb</td>
<td>9985</td>
<td>10762</td>
<td>26</td>
<td>4</td>
<td>9</td>
<td>2.25</td>
</tr>
<tr>
<td>B-4 2800 lb</td>
<td>9954</td>
<td>11407</td>
<td>26</td>
<td>4</td>
<td>9</td>
<td>2.25</td>
</tr>
<tr>
<td>B-5 1900 lb</td>
<td>8780</td>
<td>10719</td>
<td>30</td>
<td>4</td>
<td>11</td>
<td>2.75</td>
</tr>
<tr>
<td>B-6 1100 lb</td>
<td>6800</td>
<td>6324</td>
<td>33</td>
<td>3</td>
<td>12.5</td>
<td>4.17</td>
</tr>
<tr>
<td>B-7 1150 lb</td>
<td>6325</td>
<td>6040</td>
<td>33</td>
<td>3</td>
<td>12.5</td>
<td>4.17</td>
</tr>
</tbody>
</table>

Theoretical (lb) = MRC Beams (lb) * % increase

Table 5.5: Theoretical Prediction of Shear Strength of MRC Beams

<table>
<thead>
<tr>
<th>MRC Beams (lb)</th>
<th>Steel Reinforced (lb)</th>
<th>(Mini-Bar + Steel) Reinforced (lb)</th>
<th>% increase</th>
<th>Theoretical (lb)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-3 3150 lb</td>
<td>9985</td>
<td>10762</td>
<td>7.78</td>
<td>10924</td>
</tr>
<tr>
<td>B-4 2800 lb</td>
<td>9954</td>
<td>11407</td>
<td>14.60</td>
<td>10924</td>
</tr>
<tr>
<td>B-5 1900 lb</td>
<td>8780</td>
<td>10719</td>
<td>22.08</td>
<td>10300</td>
</tr>
</tbody>
</table>
Figure 5.18: Predicted Shear Strength of MRC Beams

It can be observed that the proposed model which presents shear strength provided by minbar as the vertical component of post cracking tensile strength which is purely a function of minbar geometry, its tensile strength and interfacial bond strength between minbar and concrete matrix, is capable to predicting the additional shear strength with reasonable accuracy. However, more tests are deemed necessary for furthur substantiation.
Figure 5.19: Small Scale Beams from Second and Third Sets

Figure 5.19 showed the pictures of small scale beams from second and third set tested under four point bending. It can be observed that beams B-3, B-4 and B-5 which failed in shear in both cases. However, it can be seen that beams from the third set (with steel longitudinal reinforcement and basalt minivar) are exhibiting more flexural cracks, which might be indicative of the fact that the failure mode can be changed to more favorable flexural mode with the addition of basalt minivar. This may have a distinct
advantage in seismic applications (such as beam-column joint) where brittle shear failure is undesirable and the failure mode can be altered with the addition of minibars.

Figure 5.20: Typical Flexural and Shear Failures in Beams from Second and Third Sets

In figure 5.20, typical shear and flexural failure mode from second and third sets are shown. It can be observed that for a typical beam that failed in shear, the failure mode is more dominated by favorable flexural cracks for beams with the addition of minibar.

5.5.2 Large Scale Beams

The large scale beams with different minibar dosage and shear span to depth ratios are shown in Figures 5.21, 5.22 and 5.23.
Figure 5.21: Large-Scale Beam with Dosage 0.75%; Shear-Span to Depth Ratio-6.6
Figure 5.22: Large-Scale Beam with Mini-Bar Volume Fraction of 0.5%; Shear-Span to Depth Ratio-6.6
Figure 5.23: Large-Scale Plain Concrete Beam of Shear-Span to Depth Ratio-6.6

Similarly, in Figure 5.24, it can be seen that 48 inch span long beam with 0.75 and 0.5 % minibar dosage subjected to four point bending. Figure 5.25 depicts the test conducted for plain concrete beam of similar size.

Figure 5.24: Large-Scale Beam with Mini-Bar Volume Fraction of 0.75 and 0.5%; Shear-Span to Depth Ratio-2.6
Figure 5.25: Large-Scale Plain Concrete Beam; Shear-Span to Depth Ratio-2.6

It can be observed from the figures that minibar reinforced large scale beams were able to undergo relatively ductile failure followed by smaller crack width, compared to their plain concrete counterpart. In the case of large scale beams this observation is more important as the brittle concrete failure is highly undesirable. The presence of minibars at the failure plane seemed to develop a good crack bridging mechanism thereby actuating more ductile failure response, leading to smaller crack width, higher maximum strain and better energy absorption capacities.
Figure 5.26: Exaggerated Deflection and Crack-Width of Plain Concrete Beam as Compared to Mini-Bar Reinforced Concrete Beam
5.5.2.1 Load-Deflection Analysis

The load-deflection data from the dial-gage reading were manually recorded and plotted to have an idea on the load-deflection behavior of large scale mini-bar reinforced beams. All the respective plots are depicted in Figure 5.27 and Figure 5.28.

Figure 5.27: Load-Deflection Plots for Large Scale Beams (48 and 32 Inches Span)

From Figure 5.27, it can be observed that there is an improvement in ductility and energy absorption capacity of the beams reinforced with mini-bar fiber, with same shear span to depth ratio as compared to plain concrete beams. The improvement on the ductility and toughness of the concrete with the addition of minibar is encouraging, particularly for the large scale beams, due to the impact it can have on the overall serviceability limit states of minibar reinforced concrete structures.
5.5.2.2 Fiber-Distribution across the Depth of Beam on the Crack-Plane

The apparent increase in load-carrying capacity of mini-bar reinforced concrete beams can be accounted for the extra load supported by mini-bars at the crack-plane. Hence it can be said that distribution of fibers across the depth of the beam is pivotal for the improved flexural response of mini-bar reinforced beams. To have a quantitative measure of the fiber distribution, individual fibers were counted along the depth of the beam. Figure 5.29 shows the average trend of fiber distribution across the depth of the beam.

Figure 5.28: Load-Deflection Plots for Beams of Span 112 Inches
From Figure 5.29, we can see that there is no particular trend of fiber distribution across the depth of large scale beams. The number of minibar counted were compared with the theoretical model prepared in chapter 4. This is shown in the inset of Figure 5.29. The model has an excellent match the number of fibers counted with 0.75% minibar dosage, however, there is a discrepancy of 19% with 0.5% minibar dosage. With the increasing specimen size, there is a higher probability of fibers getting distributed in larger control volume, hence can also affect the accuracy of fiber count. More tests are deemed necessary to validate the model with larger specimen size. Figure 5.30 shows the scheme that was adopted for fiber counting. Every inches of the depth of the beam was marked and fibers were counted between adjacent lines.
Figure 5.30: Fibers Counted along the Depth of a Beam

Figure 5.31 depicts the presence of mini-bars across the crack planes which are instrumental for increased post-cracking tensile strength, thus leading to improved ductility and lesser crack width with increasing load.
Figure 5.31: Typical Fiber Distribution across the Crack-Planes
5.5.2.3 Moment-Strength of Large-Scale Beams

The maximum loads carried by the beams were compared to theoretical moment-capacity of plain concrete beam as per ACI 319-11 to develop insight into the apparent increase in moment-capacity due to addition of mini-bar fiber.

Moment Strength of the plain structural concrete beam subjected to flexure (ACI 318-11) is calculated as

$$M_n = 5 \sqrt{f'_c} S_m$$

where $S_m$ is the sectional modulus of the section.

Similarly, design of rectangular cross sections subject to shear shall be based on

$$V_n = \frac{4}{3} \sqrt{f'_c} b_w h$$

As presented in figures in above section, all the beams were observed to fail in flexure. However, one of the 48 inches span beams with 0.75% mini-bar volume fraction was found to reach its shear capacity, however, the mode of final failure was observed as flexural.

The results are tabulated in Table 5.6 and Table 5.7. Table 5.6 shows the comparison of experimental load-carrying capacity of all beams in comparison to ACI provision for calculating flexural strength of plain concrete beams.

Similarly Table 5.7 shows the relative comparison of plain concrete beams with mini-bar reinforced concrete beams with two different volume-fractions.
Table 5.6: Summary of the Ultimate Moment Capacity of Mini-Bar Reinforced Concrete Beams

<table>
<thead>
<tr>
<th>Span (in)</th>
<th>Width (in)</th>
<th>Depth (in)</th>
<th>Fiber Volume Fraction (%)</th>
<th>Concrete Compressive Strength, f'_c (psi)</th>
<th>Experimental Maximum Load (lb)</th>
<th>ACI Maximum Load (lb)</th>
<th>% Increase</th>
<th>Maximum Moment-Strength (kip-ft)</th>
<th>% Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>112</td>
<td>20</td>
<td>8</td>
<td>0.75</td>
<td>8780</td>
<td>4541</td>
<td>2483</td>
<td>83</td>
<td>12.50</td>
<td>7.94</td>
</tr>
<tr>
<td>112</td>
<td>0.50</td>
<td></td>
<td></td>
<td>8630</td>
<td>4251</td>
<td>2715</td>
<td>57</td>
<td>11.33</td>
<td>7.87</td>
</tr>
<tr>
<td>112</td>
<td>0</td>
<td></td>
<td></td>
<td>8200</td>
<td>3597</td>
<td>2576</td>
<td>40</td>
<td>10.00</td>
<td>7.64</td>
</tr>
<tr>
<td>48</td>
<td>0.75</td>
<td></td>
<td></td>
<td>8780</td>
<td>19293</td>
<td>8600</td>
<td>124</td>
<td>17.31</td>
<td>7.94</td>
</tr>
<tr>
<td>48</td>
<td>0.75</td>
<td></td>
<td></td>
<td>8780</td>
<td>14715</td>
<td>8600</td>
<td>71</td>
<td>13.31</td>
<td>7.94</td>
</tr>
<tr>
<td>48</td>
<td>0.50</td>
<td></td>
<td></td>
<td>8630</td>
<td>11772</td>
<td>8524</td>
<td>38</td>
<td>10.71</td>
<td>7.87</td>
</tr>
<tr>
<td>48</td>
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<td></td>
<td>8630</td>
<td>12753</td>
<td>8524</td>
<td>50</td>
<td>11.57</td>
<td>7.87</td>
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<tr>
<td>48</td>
<td>0</td>
<td></td>
<td></td>
<td>8200</td>
<td>10905</td>
<td>8260</td>
<td>32</td>
<td>9.85</td>
<td>7.64</td>
</tr>
<tr>
<td>32</td>
<td>0</td>
<td></td>
<td></td>
<td>8200</td>
<td>18312</td>
<td>11,610</td>
<td>58</td>
<td>10.20</td>
<td>7.64</td>
</tr>
</tbody>
</table>

Table 5.7: Comparative Tabulation of Ultimate Moment Capacity of Plain Concrete Beams with Mini-Bar Reinforced Beams

<table>
<thead>
<tr>
<th>Span (in)</th>
<th>Average Compressive Strength, f'_c (psi)</th>
<th>Average Maximum Moment (kip-ft)</th>
<th>% Experimental Increase in Moment Capacity</th>
<th>Average Maximum Load (lb)</th>
<th>% Increase in Load Capacity with Volume-Fraction (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.75</td>
<td>0.5</td>
<td>0</td>
<td>0.75</td>
<td>0.5</td>
<td>0.75</td>
</tr>
<tr>
<td>8780</td>
<td>8630</td>
<td>8200</td>
<td>12.50</td>
<td>25.00</td>
<td>4541</td>
</tr>
<tr>
<td>112</td>
<td>11.33</td>
<td>10.00</td>
<td>55.43</td>
<td>13.10</td>
<td>17004</td>
</tr>
<tr>
<td>32</td>
<td>10.20</td>
<td></td>
<td>18312</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
5.6 Conclusions and Recommendations

This subsection provides a summary, conclusions and recommendations based on the work done on minibar reinforced concrete beams of different scales. The program was broadly categorized into two groups; small scale and large scale. Small scale beams comprised of three groups, the first group reinforced with minibar, the second group with longitudinal steel rebar, and third group with both longitudinal rebar and basalt minibar. Large scale beams consisted of plain concrete beam, and beams with varying minibar dosage and shear span to depth ratios. The following conclusions were drawn from the study:

- Modified Discrete Fiber Property approach is able to predict the ultimate load carrying capacity of MRC beams of varying shear-span to depth within reasonable accuracy.
- There is no considerable increase in flexural strength of MRC beams with the addition of 0.5% of mini-bar, however, ductility, maximum strain carried and failure modes were improved.
- Shear-Strength is found to increase by almost 15% with the addition of 0.5% mini-bar.
- The mechanics-based analytical model is able to predict the shear-strength of MRC beams with good accuracy. However, more tests with different volume fractions of mini-bar are needed.
- There is no notable trend on the distribution of fibers along the depth of the beam.
- It can be observed that the method of mixing and vibrating can affect the moment strength of mini-bar reinforced concrete beams. More minibar fibers accumulated
near the neutral axis; therefore the effective moment carrying capacity tends to be reduced.

- With the addition of 0.5% fiber, there is an average increase of 42% on the moment-strength as compared to its plain-concrete counterpart as per ACI 318. For 0.75% volume-fraction of mini-bar, there was an average increment of 82%.

- With the addition of 0.5% mini-bar, an average increase in moment strength of 13% relative to the moment strength of the corresponding plain concrete beams was observed. For 0.75% volume fraction, the percent increase of moment strength was 25 and 55% for 118” and 59” long beams respectively.

- The standard beams that are used for determining modulus of rupture of concrete as per ASTM C78 are typically small. For large-scale beams, the flexural tensile strength can be significantly lower due to size-effect. It is advisable to use mini-bar as a reinforcement for full-scale beams such that apparent decrease in flexural tensile strength due to size effect can be properly countered.

- From the load-deflection plots, it can be observed that apparent ductility of beams has improved with the addition of fibers. A post-crack hardening behavior was shown by load-deflection curves of mini-bar reinforced concrete beams.
CHAPTER VI
FIBER REINFORCED CONCRETE IN THE CONTEXT OF ELEVATED TEMPERATURE

This section will provide a detailed description of the literature review, laboratory procedures and the experimental program that was carried out to study elevated temperature performance of minibar reinforced concrete slabs. The primary objective of this part of research-program is the qualitative and quantitative assessment of the deterioration of strength characteristics of minibar reinforced concrete slabs under high temperature. Since in the previous sections, material, toughness and strength characterization of minibar reinforced concrete was reported, research is also deemed necessary to develop insight into structural behavior of the material under fire. If the good toughness properties of minibar reinforced concrete is found to be a viable option by the industries in coming years (for instance in tall structures), the fire-characterization of the material for buildings is a must. The section outlines the literature review, description of the materials, description of the laboratory-procedures and experimental works that was undertaken to achieve the purpose. A fixed minibar dosage of 0.75% was used in this part of the test program.
6.1 Literature Review

To sufficiently characterize the response of reinforced or fiber-reinforced concrete under fire, it is hugely imperative to develop insight into the influence of fire on constituent materials itself. Fiber-reinforced concrete is a two-phase composite material whose structural behavior is dependent on the bond between the concrete matrix and the fibers. Basalt minibar in itself is a two-phase composite material consisting of fiber and matrix. The properties of the composite in longitudinal direction are governed by relatively stiff fiber characteristics whereas polymeric resin governs the properties in transverse direction. Polymeric resin also provides the path to load transfer between the individual fibers which hinges on the adequate bond between resin and fiber, thus assuring the composite action. Under fire, there is every possibility of the change in mechanical behavior of the constituent materials (concrete and fiber) and the bond properties between them (local-effects), which consequently affect the overall global response of the structures. This includes the global bond behavior (between concrete matrix and composite fiber), dimensional variations and probable actualization of different force resisting mechanism with time. It further implies an imminent need of research into the material behavior under elevated temperature.

6.1.1 Behavior of concrete under elevated temperature

Most of the existing fire-codes work on the philosophy of providing the adequate measures that ensure satisfactory performance of the structure under fire. This is achieved by making the structure conforming to some standard fire ratings [81] which automatically allow us to disregard the temperature-dependent response of the materials.
However, considering the fact that new codes tend to be more performance based, study of material properties under high temperature is an absolute necessity. Heating of concrete invariably leads to changes in physiochemical structure in concrete affecting thermodynamic stability field of various sub-phases [82]. This coupled with various mineralogical/chemical changes of the constituent materials can finally lead to an adverse affect on the global structural response of concrete. Based on polarizing and fluorescent microscopy (PFM) techniques, Nijland [82] tabulated the phase changes in hardened cement paste which can provide some very significant insights into the degraded mechanical properties at structural scale.

Table 6.1: Phase Changes in Hardened Cement Paste and Concrete with Increasing Temperature

<table>
<thead>
<tr>
<th>°C</th>
<th>Reaction</th>
<th>Macro/microscopically diagnostic features</th>
</tr>
</thead>
<tbody>
<tr>
<td>55 - 60</td>
<td>14 Å-tobermorite → 11 Å-tobermorite</td>
<td></td>
</tr>
<tr>
<td>70 - 80</td>
<td>Dissociation of ettringite</td>
<td>Absence of ettringite in cement paste</td>
</tr>
<tr>
<td>70 - 90</td>
<td>Jennite → metajennite</td>
<td></td>
</tr>
<tr>
<td>70 - 200</td>
<td>Gypsum → hemihydrate or γ-CaSO₄</td>
<td>Absence of gypsum in cement paste</td>
</tr>
<tr>
<td>100 - 300</td>
<td>11 Å-tobermorite → 9 Å-tobermorite</td>
<td></td>
</tr>
<tr>
<td>180</td>
<td>Dissociation of gel-like C-S-H</td>
<td>Increase in capillary porosity</td>
</tr>
<tr>
<td>180 - 190</td>
<td>Decomposition of monosulfate</td>
<td></td>
</tr>
<tr>
<td>200 - 310</td>
<td>Hydrogarnet → mayenite + portlandite</td>
<td>Increase in capillary porosity</td>
</tr>
<tr>
<td>300</td>
<td>Loss of chemically bound water</td>
<td></td>
</tr>
<tr>
<td>300 - 350</td>
<td>Oxidation of FeOOH to α-Fe₂O₃</td>
<td>Change in color to pink or reddish brown</td>
</tr>
<tr>
<td>450 - 550</td>
<td>Portlandite → CaO + H₂O</td>
<td>Absence of portlandite in cement paste</td>
</tr>
<tr>
<td>573</td>
<td>α-quartz → β-quartz</td>
<td>Radial cracks around aggregate particles</td>
</tr>
<tr>
<td>650 - 700</td>
<td>Decomposition of C₃S-H</td>
<td></td>
</tr>
<tr>
<td>680 - 700</td>
<td>Xonotlite → belite</td>
<td></td>
</tr>
<tr>
<td>800 - 900</td>
<td>Decomposition of carbonates</td>
<td>Desintegration of carbonate grains</td>
</tr>
<tr>
<td>800</td>
<td>C₃S-H → belite + wollastonite</td>
<td>Complete desintegration of cement paste</td>
</tr>
<tr>
<td>900</td>
<td>Jennite → belite + wollastonite</td>
<td>Complete desintegration of cement paste</td>
</tr>
<tr>
<td>1050</td>
<td>β-quartz → cristobalite</td>
<td>Appearance of cristobalite</td>
</tr>
<tr>
<td>1150 - 1200</td>
<td>incipient melting</td>
<td>Presence of quenched melt / melt textures</td>
</tr>
</tbody>
</table>

Some of the pronounced macroscopic effects on concrete due to the above mentioned chemical-mineralogical changes in microscopic scale are: dehydration of
cement paste, increase in the degree of porosity, reduction in moisture content, differential thermal expansions, increase in pore vapor pressure, thermal cracking, transitional thermal creep and spalling [83]. It has been observed that concrete compressive strength tend to decrease with increased temperature with an increase in strain value associated with it [81]. It has also been observed that stress-strain relationship for concrete vary under elevated temperature [81]. One of the immediate effects of elevated temperature on concrete has been identified to be the decrease in concrete compressive strength and increase in strain corresponding to it. Following equation has been proposed to account for the decrease in ultimate concrete strain due to temperature:

\[
\varepsilon_{cu} \text{(unconfined concrete)} = 4.942 \times 10^{-3} - 6.995 \times 10^{-5} f'_c + 3.993 \times 10^{-7} f'_c^2
\]  

(91)

6.1.2 Concrete Compressive Strength at Elevated Temperature

1. Lie et al. [81]

\[
f'_{ct} = f'_c \cdot (1 - 0.001.T) \quad T \leq 500^0C
\]

\[
f'_{ct} = f'_c \cdot (1 - 0.00175.T) \quad 500^0C \leq T \leq 700^0C
\]  

(92)

\[
f'_{ct} = 0 \quad T \geq 700^0C
\]

2. Eurocode Model [81]

\[
f'_{ct} = f'_c \quad T \leq 100^0C
\]

\[
f'_{ct} = f'_c \cdot (1.067 - 0.00067.T) \quad 100^0C \leq T \leq 400^0C
\]  

(93)
\[ f'_{cT} = f'_{c_s} \left( 1.44 - 0.00166. T \right) \quad T \geq 400^\circ C \]

3. Lie and Lin Model [81]

\[ f'_{cT} = f'_{c_s} \left( 2.011 - 2.353. \frac{T-20}{1000} \right) \leq f'_{c_s} \quad (94) \]

4. Li and Purkiss [81]

\[ f'_{cT} = f'_{c_s} \left( 0.00165. \left( \frac{T}{100} \right)^3 - 0.03. \left( \frac{T}{100} \right)^2 + 0.025. \left( \frac{T}{100} \right) + 1.002 \right) \leq f'_{c_s} \quad (95) \]

5. Hertz [81] proposed a model which accounts for different aggregate types, hence can be termed more comprehensive.

\[ f'_{cT} = f'_{c_s} \left[ \frac{1}{1 + \frac{T}{T_1} + \left( \frac{T}{T_2} \right)^2 + \left( \frac{T}{T_8} \right)^8 + \left( \frac{T}{T_{64}} \right)^{64}} \right] \quad (96) \]

Siliceous aggregate: \( T_1 = 15000, T_2 = 800, T_8 = 570, \) and \( T_{64} = 100000 \)

Lightweight aggregate: \( T_1 = 100000, T_2 = 1100, T_8 = 800, \) and \( T_{64} = 940 \)

Other aggregates: \( T_1 = 100000, T_2 = 1100, T_8 = 800, \) and \( T_{64} = 940 \)

6. Li and Guo [81]

\[ f^{T}_{cu} = \frac{f_{cu}}{\left[1 + 2.4 \left(T - 20\right)^6 \times 10^{-17}\right]} \quad (97) \]
6.1.3 Concrete Strain at Peak Stress under Elevated Temperature [81]

Some of the models that consider the effect of temperature on the peak-strain of the concrete are described below.

1. Lie

\[ \varepsilon_{0T} = 0.0025 + (6.0 \cdot T + 0.04 \cdot T^2) \times 10^{-6} \]  

(98)

2. Li and Pukiss

\[ \varepsilon_{0T} = \frac{2f' c}{\varepsilon_{ct}} + 0.21 \times 10^{-4} \cdot (T - 20) - 0.9 \times 10^{-8} \cdot (T - 20)^2 \]  

(99)

3. Lu and Yao

\[ \varepsilon_{0T} = \varepsilon_0 \cdot (0.0019 \cdot T + 0.9615) \]  

(100)

4. Khennane and Baker

\[ \varepsilon_{0T} = 0.003 \quad 20 \leq T \leq 200^\circ C \]  
\[ \varepsilon_{0T} = 0.00001156 \cdot T + 0.000686 \quad 0.0082 \leq T \geq 200^\circ C \]  

(101)

5. Bazant and Chern

\[ \varepsilon_{0T} = 0.0000064 \cdot T + 0.00216 \quad 20 \leq T \leq 600^\circ C \]  
\[ \varepsilon_{0T} = 0.000015 \cdot T - 0.003 \quad 600 \leq T \leq 650^\circ C \]  

(102)

6. All the relationships as described above consider the case when the concrete specimen were not loaded during the heating process, hence can be considered to be an index of residual material properties. Khennane and Baker [81] proposed
the following expression for the peak compressive strain at elevated temperature when the specimen was loaded during heating.

\[ \varepsilon_{0T} = 0.00000167 \cdot T + 0.002666 \geq 0.003 \quad T \leq 800^\circ C \]  

(103)

6.1.4 Modulus of Elasticity at Elevated Temperature

It has been observed that like other mechanical properties of concrete, modulus of elasticity is also adversely affected under high temperature. Under high temperature, the pore water inside concrete is vaporized, creating extra gaseous pressure, thus weakening the bond between the aggregates. This will ultimately affect the compressive strength of the concrete as well as its modulus of elasticity. Similar observations were made that the same factors are pivotal on affecting both the strength and stiffness properties of concrete [81]. It has been observed that reduction in tensile strength is more pronounced as compared to reduction in modulus itself [84].

1. Lu

\[ E_{cl \ T} = (1 - 0.0015 \cdot T) \cdot E_{ci} \quad 20 \leq T \leq 200^\circ C \]  

(104)

\[ E_{cl \ T} = (0.87 - 0.00084 \cdot T) \cdot E_{cl} \geq 0.28 \cdot E_{cl} \]

2. Li and Guo

\[ E_{cl \ T} = E_{ci} \quad 20 \leq T \leq 60^\circ C \]  

(105)

\[ E_{cl \ T} = (0.83 - 0.0011 \cdot T) \cdot E_{ci} \quad 60 \leq T \leq 700^\circ C \]

3. Li and Purkiss

259
\[ E_{cl}T = \frac{800-T}{740}.E_{cl} \leq E_{cl} \quad (106) \]

4. BSI

\[ E_{cl}T = \frac{800-T}{740}.E_{cl} \leq E_{cl} \quad (107) \]

5. Schneider, Normal weight concrete:

\[ E_{cl}T = (-0.001552 \cdot T + 1.03104).g.E_{cl} \quad 20 \leq T \leq 600^\circ C \quad (108) \]

\[ E_{cl}T = (-0.00025 \cdot T + 0.25).g.E_{cl} \quad 600 \leq T \leq 1000^\circ C \]

Schneider, Normal weight concrete:

\[ E_{cl}T = (-0.00102 \cdot T + 1.0204).g.E_{cl} \quad 20 \leq T \leq 1000^\circ C \quad (109) \]

Where:

\[ g = 1 + \frac{f_{cl}}{f_c} \cdot \frac{T-20}{100} \cdot \frac{f_{cl}}{f_c} \leq 0.3 \quad (110) \]

6. Khennane and Baker, Preloaded Concrete:

\[ E_{cl}T = (-0.000634 \cdot T + 1.012673).E_{cl} \quad 20 \leq T \leq 525^\circ C \quad (111) \]

\[ E_{cl}T = (-0.002036 \cdot T + 1.749091).E_{cl} \quad 525 \leq T \leq 800^\circ C \]

Khennane and Baker, Unloaded Concrete:

\[ E_{cl}T = (-0.001282 \cdot T + 1.02641).E_{cl} \quad 20 \leq T \leq 800^\circ C \quad (112) \]

7. Anderberg and Thelandersson:

\[ E_{cl}T = \frac{2f'}{\epsilon_{\sigma T}} \quad (113) \]
Since tensile strength of concrete is the key parameter governing the flexural and shear strength of concrete members, evolution of tensile strength of concrete under elevated temperature is another important subject to consider. It can be more significant if the substantial increase in tensile strength of concrete with the addition of fibers is considered. In the current study, we are studying the effect of fire on structural performance of basalt mini-bar reinforced concrete slabs. The fiber is expected to increase the tensile strength of concrete and also its post-cracking behavior. Since it has been observed that elevated temperature can have detrimental effect on both concrete and fiber characteristics, it is reasonable to assume that it can have a possible deleterious effect on the expected performance of fiber reinforced slab. Hence, it is very desirable to have some idea on how tensile strength characteristics of concrete are affected under elevated temperature [84].

1. Bazant and Chern

\[
f_{crT} = f_{cr} \cdot (-0.000526 \cdot T + 1.01052) \quad 20 \leq T \leq 400^\circ C
\]

\[
f_{crT} = f_{cr} \cdot (-0.0025 \cdot T + 1.8) \quad 400 \leq T \leq 600^\circ C
\]

\[
f_{crT} = f_{cr} \cdot (-0.0005 \cdot T + 0.6) \quad 600 \leq T \leq 1000^\circ C
\]

2. Li and Guo

\[
f_{crT} = f_{cr} \cdot (1 - 0.001 \cdot T) \quad 20 \leq T \leq 1000^\circ C
\]

3. Xie and Quian

\[
f_{crT} = f_{cr} \cdot \left(2.08\left(T/100\right)^2 - 2.666\left(T/10\right) + 104.79\right)
\]

\[
f_{crT} = f_{cr} \cdot [0.58 \left(1.0 - T/300\right) + 0.42] \quad (20 - 300^\circ C)
\]
It has already been discussed above that material characteristics undergo detrimental deterioration under elevated temperature. Another important factor to be considered next is the bond-deterioration under high temperature. In the case of fiber-reinforced concrete, high temperature can cause the weakening of various bond-mechanisms, for instance, hydrostatic pore water pressure and chemical adhesion and surface characteristics. It is worth noting that the effect of temperature on bond between concrete-steel and concrete-fiber can be qualitatively different. In case of concrete-steel interfacial bond, where wedging effect is the prominent bond-mechanism, concrete can be regarded to be more susceptible to increased temperature effects. Since steel can retain its geometrical and volumetric integrity for a wide range of temperature, the governing bond-mechanism that depends on steel-characteristics may not get severely affected as compared to the bond-mechanisms governed by concrete characteristics. In the case of concrete-fiber interface, larger fraction of bond-strength is derived from chemical adhesion, surface characteristics of fiber, hydrostatic pore water pressure and the pressure developed by swollen fiber against hardened concrete. This entails that bond-characteristics in the case of fiber-reinforced concrete is dependent on the thermo-mechanical properties of both concrete and fiber. However, it is important to have some study on effect of fire on bond-characteristics of concrete-steel interface to have a general overview of the topic.

1. Xie and Qian Model

\[
\tau_{u,T} = \tau_{u,0} \cdot \left[ 2.7438 \left( \frac{T}{100} \right)^2 - 3.322 \left( \frac{T}{100} \right) + 105.881 \right] \times 10^{-2} \quad (117)
\]
6.1.5 Quantification of fire-effect on FRP Bars

1. Saafi

If reduction factor is defined as:

$$\frac{f_{fuT}}{f_{fu20°C}} = k_f$$  \hspace{1cm} (118)

GFRP Rebars

$$k_f = (1 - 0.0025T) \quad 0 \leq T \leq 400°C$$  \hspace{1cm} (119)

$$k_f = 0 \quad 400°C \leq T$$

AFRP Rebars

$$k_f = 1 \quad 0 \leq T \leq 100°C$$  \hspace{1cm} (120)

$$k_f = (1.333 - 0.00333T) \quad 100°C \leq T \leq 400°C$$

$$k_f = 0 \quad 400°C \leq T$$

CFRP Rebars

$$k_f = 1 \quad 0 \leq T \leq 100°C$$  \hspace{1cm} (121)

$$k_f = (1.267 - 0.00267T) \quad 100°C \leq T \leq 475°C$$

$$k_f = 0 \quad 475°C \leq T$$

Similarly reduction factor for modulus of elasticity was defined as

$$\frac{E_{fT}}{E_{fu20°C}} = k_E$$  \hspace{1cm} (121)

GFRP and AFRP rebars

263
\[ k_E = 1 \quad 0 \leq T \leq 100^\circ C \]  

\[ k_E = (1.25 - 0.0025T) \quad 100^\circ C \leq T \leq 300^\circ C \]

\[ k_E = (2 - 0.005T) \quad 300^\circ C \leq T \leq 400^\circ C \]

\[ k_E = 0 \quad 400^\circ C \leq T \]

CFRP rebars

\[ k_E = 1 \quad 0 \leq T \leq 100^\circ C \]  

\[ k_E = (1.175 - 0.00175T) \quad 100^\circ C \leq T \leq 300^\circ C \]

\[ k_E = (1.625 - 0.00325T) \quad 300^\circ C \leq T \leq 500^\circ C \]

\[ k_E = 0 \quad 500^\circ C \leq T \]

### 6.1.6 FRP and Fiber in the context of high-temperature effects

FRP bars and fibers are the composite systems comprising of stiff fibers impregnated with polymeric resin. The mechanical properties of the system can be termed as a combination of fiber and matrix properties. Integral response of these systems is guaranteed by the sufficient bond between fiber and the polymeric resins which ensures its structural unity by the transference of stresses from resin to the fiber and its volumetric integrity by their chemical stability. It can be stated that durability issues in case of fiber reinforced concrete is not only the function of constituents characteristics only but also their interaction properties. The fiber-dominated properties such as longitudinal tensile strength and stiffness differ qualitatively from the resin-dependent properties such as transverse strength and stiffness. As already mentioned, transverse
properties govern the transfer of shear stresses from resin to fiber and from fiber to fiber which ultimately account for the overall integrity of the member with the subsequent development of bond stresses at the interface [85]. At high temperature, the polymeric resins are generally affected which may result in the deterioration of the composite’s properties and its interaction mechanisms with concrete. One of the key parameters governing these issues is glass transition temperature of the polymeric resin. When the temperature exceeds the glass transition temperature of the polymeric matrix, it loses its mechanical properties. It was reported that the compressive strength of epoxy was substantially decreased when the temperature exceeds its glass transition temperature of 100°C. S

Similar observations were made regarding its tensile and shear strength [84]. Since composite properties on transverse directions are governed by resin properties, this can be a serious issue when dealing with high temperature events. For instance, shear strength of a low viscosity epoxy was reduced to almost 30% of its strength at ambient temperature when temperature was at 80°C [84]. The degree of deterioration depends on the chemical composition of the material, such as amount of crosslinks and degree of crystallization which define its mechanical stability in micro-scale [86]. Fibers are generally stable under high temperature. However, it was observed that glass fibers can lose almost 50% of its tensile strength when subjected to a temperature of 550°C [84]. However, the transverse properties such as shear strength which is instrumental on the transfer of stresses under flexural loading are dependent on resin properties. Significant decrease in flexural strength has been recorded under high temperature which relies on actualization of matrix-dependent shear properties of the composite [86]. Based on the
pull out test performed on FRP bars embedded in concrete, Kaz et al. observed that there is a reduction in the peak load and post-peak response was governed primarily by physical-friction mechanism. This implies the severe effect of fire on surface and geometrical properties of the FRP rods which can be deemed responsible for the decreased bond-strength and shift in bond-mechanisms. Since mini-bars can be regarded to be a very short FRP bars of smaller diameter, there is every possibility that similar mechanisms are actualized when mini-bar reinforced concrete specimens are subject to different types of loadings. It was observed by Kaz et al. that the development of internal pressure due to massive water vapor at the FRP-concrete interface was responsible for the decreased adhesion between the constituents. This loss in adhesion can be attributed to the deteriorated chemical adhesion between the FRP-concrete interface and the gradual destruction of the surface roughness with time.

At elevated temperature the degradation of bond between FRP bars and concrete can be exclusively attributed to the polymer, irrespective of the effects on concrete [86]. For the temperature of 130°C, it was observed that the reduction in peak load was 20 % for steel bars whereas for FRP bars it was between 40 and 60 % [86]. This clearly signifies the increased susceptibility of FRP and fiber reinforced concrete to elevated temperature phenomena. The crosslinks between the polymer chains which ultimately accounts for the mechanical strength and stiffness of polymer in macro-scale, tends to change from thermoplastic to thermosetting in nature as the temperature increases. This may help for the moderation of the deterioration as the temperature increases. As per the work done by Wang et al., it was reported that GFRP and CFRP bars undergo deterioration in their strength and stiffness at high temperature. The authors conclude that
temperature of about $350^0\text{C}$ is observed to be critical for the case of FRP bars. However, it should be noted that the magnitude of degradation may be the convoluted function including synergistic response of fiber characteristics, polymer properties, curing methods and their interaction properties. This renders durability issues associated with fiber reinforced concrete and FRP reinforced concrete under elevated temperature more rigorous and painstaking as compared to conventional steel reinforced concrete members. The complexity of topic lies in the fact that the durability issues under fire in case of fiber reinforced concrete is the function of resin and fiber characteristics alongwith their interaction properties, as opposed to corrosion issues associated to steel reinforced concrete [85]. Another fact to be considered in case of fiber reinforced concrete is the influence of the variation of thermal properties of fiber in different directions and its possible influence on the interaction mechanisms. Thermal properties of some commercially available FRP bars as compared to steel are listed in Table 6.2 [85]:

Table 6.2: Coefficients of Thermal Expansion for Different Fibers

<table>
<thead>
<tr>
<th>Direction</th>
<th>Coefficient of thermal expansion ($*10^{-6} / ^\circ\text{C}$)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Steel</td>
</tr>
<tr>
<td>Longitudinal, $\alpha_L$</td>
<td>11.7</td>
</tr>
<tr>
<td>Longitudinal, $\alpha_T$</td>
<td>11.7</td>
</tr>
</tbody>
</table>

From Table 6.2, it can be observed that the coefficients of thermal expansion in orthogonal directions for fibrous materials are substantially different. Under high temperature situation, this may lead to an added radial straining at the concrete-rebar interface due to differential expansion, resulting in increased cracking of concrete and subsequent longitudinal splitting [85]. This may allow the heat to flow into the concrete more easily thus affecting fiber’s stiffness and strength, thus inducing more cracks and
degradation thereby. Here we can observe that anisotropic material properties of fiber can
lead to a non-linear symbiosis of events that ultimately leads to severe bond-degradation,
thus affecting the overall integrity of fiber reinforced concrete. The relatively lower
stiffness of the fiber can lead to larger deformation resulting in considerable cracks,
which facilitates the heat to propagate rapidly into the structure, which will further
aggravate the fiber material properties thus resulting in increased deformation. This can
initiate a type of positive feedback loop which can ultimately lead to the failure of the
structure. Y.C. Wang et al. studied the strength-stiffness properties of GFRP and CFRP
bars under elevated temperature. They concluded from their investigations that FRP bars
retain a large fraction of their strength just below a thermal-index, termed as critical-
temperature by the authors. Critical temperature is defined as the temperature at which
material loses 50 % of its strength, thereby not able to withstand applied load. They
Stated 3250°C and 2500°C to be the critical temperature for GFRP and CFRP bars
respectively, whereas 5800°C is for the steel reinforcing bars. Below 3500°C, FRP materials
were found to be capable of retaining almost 90 % of their strength at ambient
temperature [87].

Research works performed by L.T. Phan at NIST also provide some valuable
insight into the performance of concrete under elevated temperature. It is a well-
documented fact that high strength concrete behaves qualitatively different from normal
strength concrete. HSC are susceptible to more brittle failure in compression as well as in
shear and flexure. Irrespective of NSC, where failure occurs across the weakest plane
through the aggregate and cement-sand matrix, failure tends to occur through the
aggregate itself in case of HSC. This renders the HSC more brittle in nature which might
be a very important issue to consider when explosive spalling can be expected under high temperature. The explosive spalling in case of HSC was found to be associated with vaporization and the transport characteristics of free and chemically bound water [88]. It was concluded from the investigation that pore-water pressure build-up may be the primary cause of explosive spalling rather than the thermal stresses developed during heating. In case of HSC, where water-cement ratio is relatively lower, the higher cement content can account for the obstruction of transport mechanisms of vaporized water molecules under elevated temperature, ultimately leading to explosive spalling. It was experimentally confirmed that at lower water-cement ratio, effect of explosive spalling was significantly moderated [88]. Following the above discussion, it can be stated that thermal behavior of HSC can be beneficially tailored by the addition of basalt mini-bars. With the application of fibrous materials, the concrete behavior can be modified in following ways:

1. Bond between fiber and the concrete matrix

   The explosive spalling of concrete can be attributed to the sudden release of thermal energy and the summation of different other effects generated by the kinematics of the vaporized water molecules. With the addition of fiber, the bond between fiber and concrete matrix can provide an efficient means for the gradual dissipation of energy during spalling, thus leading to a more ductile failure. However, it should be noted that the bond between concrete and fiber will also be affected at elevated temperature, thus alleviating the effective magnitude of energy dissipation.

2. Free passage to the vaporized molecules
Apart from providing a more efficient energy dissipating mechanism, fibers have been found to be instrumental for the reduction in pore water pressure. Phan observed that reduction in pore water pressure increases with the increasing dosage of polypropylene fibers. He concluded that addition of 1.5kg/m³ was sufficient to avoid explosive spalling [88]. This can be attributed to the free and void spaces that the fibers will invariably create within the concrete matrix. Free space and voids created by the presence of fibers in concrete matrix will provide an enlarged passage for the transport of energized water vapor molecules thus reducing the effecting pore water pressure. Fibers in themselves will absorb some fraction of water which will virtually increase the water-cement ratio. Considering all the above facts, it can be stated that for a unit volume of unreinforced concrete and FRC, FRC will have smaller number of free molecules with larger mean free path, thus reducing the effective pore water pressure.

Another topic that is particularly important for the analytical study of FRC is the actual distribution of temperature within the element during heating. In the absence of adequate research on the topic, the work done on FRP reinforced concrete can be judiciously extended for the case of FRC. The determination of temperature profile within the element can help us to have an idea of the criticality of the structure at different level of loadings and locations. For instance, the recognition of critical sections based on temperature gradient across the specimen can help us to optimize the locations which are critical from structural point of view. Knowledge of temperature distribution across the cross-section of concrete members is also helpful to analyze their residual bearing capacity after withstanding fire [89]. Theoretical prediction of temperature across
the concrete section can impart us with valuable insight into the structural behavior of concrete under fire, thus saving a considerable amount of time and expenditure associated with the corresponding experimental assemblages. Structural behavior of a concrete structure can only be studied holistically when the symbiotic relationship between the transfer analysis and structural analysis is ascertained properly. Determination of temperature distribution on RC members is evidently a complex phenomenon due to the anisotropic nature of concrete and the associated thermo-mechanical parameters, which are inherently stochastic in nature. Analysis can be more rigorous if we consider the materials with low thermal conductivity retaining moisture within it [89]. Presence of moisture can render the thermal properties of the concrete more indeterminate and can affect the probable dynamics of energy transfer. It can be regarded as the function of energy transfer across the specimen caused by the temperature difference created during the test. The heat energy is transferred by convection and radiation from the fire into the member and further by conduction within the member itself [89]. Following are the prevalent methods in contemporary literature for the determination of temperature profile for concrete specimens.

1. Empirical, semi-empirical charts and tables provided by respective codes and standards
2. Application of simple 1-dimensional heat transfer model
3. 2-dimensional heat transfer models with the application of finite-element or finite-difference methods
4. Simplified algebraic polynomial expressions based on test-data
Based on his study on rectangular concrete beams subjected to British standard fire curve on three sides, Desai proposed a cubic equation to describe the temperature profile [84]. The temperature (°C) inside the beam, along a contour x mm away from these sides can be expressed as:

\[ T = \left( D - Ax + Bx^2 - Cx^3 \right) / r^{0.25} \]  

(124)

Where the constants A, B, C and D are expressed as:

\[ A = 3.33 \left( 3 + 0.0033t + \frac{(100-t)}{b} \right) \]  

(125)

\[ B = 0.086 \]  

(126)

\[ C = 0.000221 \]  

(127)

\[ D = 475r^{7/12} - \left( b - 105t^{1/3} \right) \]  

(128)

Where t is the exposure period in minute, b is the width of the cross-section (mm) and r is the ratio of overall height to the width.

Simplified Method Proposed by Wickstrom (1987)

This is a very simple model proposed to account for the temperature-gradient developed across the concrete specimen during heating and doesn’t account for the spalling of concrete [89].

The fire exposed surface temperature \( T_s \) of a concrete member at a time t is expressed as:

\[ T_s = \eta_w T_f \ with \ \eta_w = 1 - 0.0616t^{-0.88} \]  

(129)
Where $\eta_w$ is the ratio between gas and the surface temperatures of concrete members ($^0\text{C}$) and $T_f$ is the gas atmosphere temperature ($^0\text{C}$).

As the uniaxial heat flow condition was assumed, the temperature $T_c$ at a depth $X$ (m) from the fire-exposed surface is expressed as:

$$T_s = \eta_x T_f \text{ with } \eta_x = 0.18 \ln \left( \frac{t_h}{x^2} \right) - 0.81$$

(130)

6.2 Energy Based Heat Transfer Analysis and Finite Difference Formulation for 1-Dimensional case [90].

The temperature gradient created across the cross-section of the concrete specimen makes the phenomenon of heat transfer possible at elevated temperature. The heat transfer is constituted of three modes:

1. Conduction through the solid specimen
2. Convective heat transfer through the atmosphere
3. Heat transfer by radiation via electromagnetic waves

It was observed by Loah [91] that classical Fourier law can sufficiently deal with many conduction problems except for rapid heating response. It can thus be assumed that Fourier law can be extended to the transfer analysis problems relating to regular concrete and FRC. For the cases of idealized solids where thermal conductivity may be the function of internal stress distribution, the governing equation tends to be of hyperbolic shape, hence not amenable to classical Fourier analysis. However, for a slower process as in the case of heat transfer through concrete slab, Fourier’s law can be applied with
reasonable accuracy. Another important factor to be considered for mass and heat transfer in the cases of porous medium like concrete is the correlation between thermal conductivity and moisture content. As thermal conductivity of water is almost 25 times greater than that of air [92], it is not very difficult to visualize the sensitive dependence of overall transfer analysis on these factors. In the case of porous medium like concrete, it is imperative to have knowledge of the correlation between thermal conductivity and the moisture content. In most semi-transparent cellular materials (for example concrete) the heat transfer is primarily governed by conduction through the gas enclosed in cells and radiation [92]. However, it should be noted that the presence of fiber in concrete can also have quantitative effect on the thermal behavior of the concrete. The presence of fibers in concrete can affect the effective moisture content of the matrix, can fill up the cellular voids, thus significantly altering the governing dynamics of heat transfer. If the conventional Fourier’s law can be modified to account for the presence of fibers, the problem of heat transfer through FRC can be modeled more accurately.
Figure 6.1: Schematic diagram of the heat transfer mechanism inside a cellular moist material

From the Fourier’s heat conduction law for an isotropic and homogenous media, conductive heat flux \( q \) across the depth of the member (here in \( X \) direction) is given by

\[
-k \frac{\partial T}{\partial x} = q
\]  

(131)

Where \( k \) is the thermal conductivity of the material and \( T \) is the temperature. In one dimensional cartesian coordinates, the governing differential equation of the heat conduction is given as

\[
k \frac{\partial^2 T}{\partial x^2} + g = \rho c \frac{\partial T}{\partial t}
\]  

(132)

The above expression was derived based on the conservation of energy principle where net rate of energy entering by conduction to the elemental volume is equal to the rate of internal energy generation and rate of increase of internal energy. For a one dimensional system with constant thermal properties and assuming null heat generation, the equation simplifies to
\[ k \frac{\partial^2 T}{\partial x^2} = \rho c \frac{\partial T}{\partial t} \quad (133) \]

On imposing the convection and radiation boundary conditions, for one dimensional case, the following equation will hold:

\[-k \frac{\partial T}{\partial x} = q = -h(T_f - T_s) - \varepsilon \sigma (T^4_f - T^4_s) \quad (134)\]

Where \( T_f \) is the fire temperature and it is the function of time.

The above equations can be solved using finite element method or finite difference method to characterize the temperature profile across the cross-section of the concrete specimen.

6.3 Structural Response of Reinforced and Fiber Reinforced Concrete Slabs under Fire

Fire resistance of reinforced concrete slabs or composite steel to concrete slabs has been a interesting topic of research recently. Fire resistance may not be a typical structural property of civil-engineering interest, albeit it is of significant importance the way it can dictate the temporal evolution of other mechanical/structural properties. For instance, the temporal evolution of concrete’s compressive strength, steel stiffness, bond-strength between concrete and steel are of paramount importance while considering both serviceability and ultimate limit states. Under fire load, the material and structural properties may evolve with time, thus leading to their undesirable degradation. Hence for the structural engineers, cognizance of global response of the structure under fire may be hugely imperative to ensure safe design within acceptable margin of safety. International Building Code clearly states its objective as “…to establish the minimum requirements.
to safeguard the public health, safety and general welfare through structural strength, means of egress facility, stability and safety to life and property from fire and other hazards attributed to the built environment and to provide safety to fire fighters and emergency responders during emergency operations” [93]. Three standard categories of failure that are to be considered in a fire resistance test are [93]:

1. Structural stability
2. Integrity
3. Insulation

The effect of fire-load on structures can have multifarious effects which can be broadly viewed under following philosophies:

1. Reductionist- This school of thought basically studies the effects of fire on intrinsic (compressive strength of concrete, stiffness of steel) and composite (bond-strength) material properties. The approach is fundamentally reductionist as it studies the evolution of material properties with time under fire and their eventual effect on the overall load-carrying capacity of structure. For instance, rise in temperature may result in reduced strength and modulus of elasticity for concrete and steel [94]. The rise in temperature will inevitably cause the free water in concrete pore to change to a gaseous state thus affecting the overall rate of heat transfer from the surface to the interior of concrete and reinforcing bar [94]. It may consequently induce micro-stress inside the concrete, leading to bigger cracks-widths and increased rate of heat-transfer.

2. Holistic- If the overall structure is viewed as a system of interconnected elements and members (as it actually is), the problem may require us to view it globally.
Despite the degradation of material properties, there is every possibility of actualization of different force-resisting mechanisms with time. Resulting force-redistribution due to interaction of cold and hot regions, development of new resisting-mechanisms (for example, tensile membrane action) and many reciprocal responses can lead to a qualitatively different structural response that can’t be predicted with reductionist approach alone.

These two approaches have inspired different design codes around the globe to incorporate fire related requirements. The reductionist approach has led to prescriptive ratings system which basically is about specifying the correctional measures regarding individual structural elements. For instance, specifying minimum slab thickness or the required concrete cover to the steel reinforcement can be viewed as prescriptive rating. It may also include providing a fire resistant coat on exposed surfaces. Covering the steel components with materials of low thermal conductivity such as concrete is also one of the methods [95]. The ratings are primarily based on experiments performed on isolated structural members such that they may not be representative of the overall global response of the structure [96]. Under prescriptive methods, the structural elements are subjected to standard time-temperature fire curve which will not continue till the cooling stage [97] and it is assumed that the structure will be functional within acceptable level-of-safety when the tested elements form the part of the structure [95]. These generic ratings also don’t account for two-way action or the influence of axial restraint at the supports which render them incapable of simulating the real structure [98].

New design methods are progressively moving to more performance based approaches which are based on holistic philosophy. This will allow designer to evaluate
the response of the structure under realistic fire scenarios. The method typically involves the definition of a most likely fire scenario, resulting temperature distribution across the structures and the eventual structural response [95]. Primary advantage of performance based approach is the application of protection materials only to the required components thereby saving time, labor and resources. For instance it has been shown that if we resort to a composite-floor system, the supporting grillage beams can be left unprotected [95].

6.3.1 ASTM E 119

ASTM E 119 provides the basic guidelines and standards for a systematic fire test of building construction materials. This also includes the end-point criteria which can be defined as the condition which when reached during a fire test, the tested specimen is deemed no more functional as a fire resisting element. For instance, the capacity of the structural element to withstand maximum load for the period of fire exposure is one of the typical end-point criteria. Fire endurance of a test-specimen is defined as the elapsed time during a fire test until an end point is reached [99]. For barrier elements, such as walls and slabs, the unexposed surface reaching a temperature of 250 degree F forms one of the end-point criteria.

ASTM E 119 provides standard time-temperature test curve which is primarily an envelope representative of the potential maximum fire that may possibly occur in the structure. However, some new performance based guidelines, such as EN 1991-1-2, also discuss the parametric temperature-time curve to represent natural fire situation. The temperature of parametric fire increases quickly at early stages, but with times as the combustibles are consumed, temperature decreases rapidly. Whereas in the case of
standard fire, the temperature increases steadily with time [100]. It is shown in Figure 6.2:

![Figure 6.2: Standard Temperature vs. Time Curve](image)

One of the issues of interest for structural engineers is the distribution of temperature gradient within concrete element which may vary with the nature of fire and exposure conditions. As it is a well-known fact that most of the research was based on the conformity to E 119, universal generalization can be definitely questioned. Another thing to be noted is that with the gradual deterioration of material and mechanical properties with fire, the structure may undergo different force-redistributions actualizing new resisting mechanisms.

6.3.2 Structural response of reinforced concrete slabs under fire load

The structural response of reinforced concrete slabs under fire is generally dealt with by classical flexural yield line theory. However, under the fire load, structural
response of a monolithically cast slab can be qualitatively different than the regular isolated slabs under monotonic loading. This can be attributed to the following reasons:

1. Reciprocal responses between interconnected structural elements
2. Restraint conditions
3. Development of different load-resisting mechanisms while undergoing large deformation under fire load

Recent studies on full scale real fire tests have revealed that steel beams and slabs do not fail despite undergoing large deformation [101]. The before-mentioned points can be illuminating here. Redistribution of stresses to the cooler parts of the structure and the development of tensile membrane action (when there is large vertical deflection) has been deemed responsible for the apparent robustness of the structure [93]. It has been observed that in case of composite slabs, the slab acts as a membrane during the fire supported by colder perimeter beams thus resisting the applied load by tensile membrane action [93]. Hence the classical yield line theory which does not account for strain-hardening response of steel reinforcement and membrane effect has been observed to be overly conservative [93]. Six concrete slabs of size 4.3m * 3.3m were used for the fire test in a research conducted at University of Canterbury. The slabs were simply supported on four sides above the furnace and were horizontally unrestrained. Slabs were exposed to fire from below with temperature-profile conforming to AS/NZSI 1530.4. It was observed that after being exposed for 3 hours to the standard fire, the slabs did not collapse and took load almost 3.7 times the load as predicted by yield line theory. However, substantial curling up of the slabs was observed around the support which may be a thing to consider. It can be surmised from the investigation that despite the absence of any
horizontal restraints, the slabs generated substantial residual strength by mobilizing membrane effect. In case of real structures where there is an inevitable presence of adjacent hot and cool regions, different kinds of resisting mechanisms can be present. It has been observed that the adjacent cool region provides substantial constraint and continuity, thus increasing the fire resistance of the heated region [102].

The primary objective behind this research program is to study the behavior of minibar reinforced concrete slab at elevated temperature based on the literature review performed. As discussed in the introduction of this chapter, a fixed minibar dosage of 0.75% was used to prepare the specimen.

6.4 Materials

6.4.1 Basalt Minibar

The basalt minibar used to reinforce the concrete slab was the same material as discussed in chapters discussed beforehand. The details can be found in Chapter II.

6.4.2 Coarse Aggregate

The coarse aggregate used in the tests was crushed limestone with a maximum size of 20 mm (~0.75 inch) for normal concrete mix. Coarse aggregate was obtained from a local Ohio supplier. Approximate absorption was 0.3 to 0.4% as suggested by the supplier. Smaller size (pea gravel) of limestone of a maximum size of 8 mm (~0.33 inch) was used for dry mix concrete as specified and also, for mix with small aggregate.
6.4.3 Fine Aggregate

The fine aggregate was river sand purchased from the local supplier. Sand was free from clay and other inert impurities.

6.4.4 Cement

Cement used in this test program was Type I/II normal Portland cement. The properties were conforming to ASTM C150.

6.4.5 Water

The water used for the concrete mixes was the normal tap water supplied by the city of Akron. The water was clean and free from any impurities.

6.5 Test Procedures

This sub-section comprises the entire laboratory tasks that were performed for the study of basalt minibar reinforced concrete (MRC) under elevated temperature. Following ASTM standards were referred to meet the objective:

1. ASTM C39-Standard Test Method for Compressive Strength of Cylinder Concrete Specimens

2. ASTM C 78-Standard Test Method for Flexural Tensile Strength of Concrete (Using Simple Beam with Simple Third-Point Loading)

6.5.1 Preparation of Specimen

To evaluate the response of fiber reinforced concrete slab, a specimen was prepared and rested to dry before the actual test. Slab was cast in the University of Akron, Materials lab. The dimensional details of the slab are shown in Figure 6.3. Since the available concrete mixer has the capacity of 3.5 cubic foot, three mixes were done to complete the casting. Six cylinders for each mix were prepared to have an idea on the variability in strength and workability of the concrete with each mixes. Basalt mini-bar was used as the fibrous reinforcement with a volume fraction of 0.75%. The approximate aspect-ratio of the fiber used was around 60 and was fairly workable for the particular volume fraction. A needle vibrator was used to ensure enough compaction of concrete but was used with caution to avoid the segregation of fiber from the concrete matrix. Wooden planks were fixed on the formwork to facilitate the engagement of the thermocouples at five different locations on the slab. Prepared specimen was rested and covered with burlap to facilitate hydration. The burlap was soaked every day to ensure the availability of moisture for hydration. Schematic diagrams of the slab are shown below in Figure 6.3:
Figure 6.3 shows the details of the reinforcement that was provided in the vicinity of supports to avoid any kind of local failure in the region. A quarter inch thick steel plate was provided along with # 2 steel bars as the reinforcement. Lifting hooks were engaged to the rebars to facilitate the transportation of the slab around the lab and to the fire test center in Buffalo, NY. The minibar reinforced concrete specimen is shown in Figure 6.4.
As it can be seen in Figure 6.4, two plastic tubes for inserting humidity sensors were also placed during the casting. Five thermal sensors (thermostats) were provided during casting to measure the development of temperature along the slab’s depth during testing.

6.5.2 Experimental Program

The experimental program was undertaken complying with ASTM E 119, under which the specimen is fundamentally pre-loaded to a certain percentage of its ultimate strength and is exposed to a standard fire under one way slab action. Figure 6.5 shows the plot of a standard fire complying with ASTM E 119.
Figure 6.5: Standard Fire-Curve as Per ASTM E-119
The general schematic of the test set up can be seen on Figure 6.6.

Figure 6.6: Schematic of Test Set-Up for Fire-Test (NGC Testing Services, Buffalo, New-York)

From figure 6.6, we see that the specimen was rested along its length and is loaded with a water tank to pre-load the slab before applying standard fire conforming to ASTM E 119. The test was conducted by NGC Testing Services, Buffalo, New York. This required an elaborate process as described below.

6.5.2.1 Transportation of Slab to the Testing Facility

Since the slab was effectively reinforced with minibar only, special care was taken during the transportation of the slab. A special lifting frame was fabricated in the
machine shop at University of Akron. The set up allowed the specimen to be lifted and transported without applying any bending stresses. This is shown in Figures 6.6 and 6.7.

Figure 6.7: Transportation of Slab from the Lab to the Truck
Figure 6.8 shows the loading of minibar reinforced concrete slab on the truck.

The slab was carefully placed on the truck with enough support and cushion. It was taken care that there would not be any premature cracks of the slab during handling and transportation. Once the slab was taken to the research facility, it was carefully unloaded and moved to the testing premises. Figure 6.8 shows the slab being delivered at the research premises at Buffalo, New York.
6.5.2.2 Test Sep Up at the Testing Center

Once the slab was delivered inside the testing center, the slab was carefully set down in the test area. Additional thermal sensors were connected to the surface of the slab to retrieve more thermal history of the slab during fire loading. This is shown in Figure 6.10.
Figure 6.10: Placement of Slab at the Test Facility
The details of all the embedded and non-embedded thermocouples are shown in Figure 6.11.

Figure 6.11: Location Details of Embedded and Non-Embedded Thermocouples

Once the thermocouples were connected, the slab was loaded with a tank filled with water. The loading was designed to preload the specimen to 20 % of its ultimate flexural strength. This is shown in Figure 6.12.
Once the slab was fire loaded to failure, the examination of the exposed surface to fire revealed a significant deterioration of minibars at the vicinity. Minibars seemed to have lost its volumetric integrity (due to the melting of vinyl Easter polymer), bond with concrete matrix and hence the overall strength and stiffness. The minibars on the exposed surface were structurally unable to actuate any kind of force-redistribution at the crack surface, thus leading to a brittle and catastrophic failure. It was observed that the bond deterioration of the individual basalt bars with the polymeric resin on the local scale and resulting debonding of minibar with concrete, resulted in a total loss of strength and stiffness of minibar, and hence the overall deterioration of flexural tensile strength of minibar reinforced concrete.
6.6 Results and Analysis

The failed specimen was transported back to University of Akron to do the forensic analysis. Core and beam specimens were cut from the body of failed minibar reinforced slab to do a quantitative assessment of the effect of fire on the deterioration of compressive and flexural tensile strength of minibar reinforced concrete.

Figure 6.13: Cores and Beams Specimens Extracted for Forensic Analysis

Figure 6.13 shows the detailed layout of beams and core specimens that were cut from the failed slab specimen. While cutting the beams specimens, it was extracted for different orientations to account for any possible fiber-alignment effect. Thirteen beam specimens were obtained in the longitudinal direction, and 4 beams were obtained for transverse direction.
Figure 6.14: Retrieval of Core and Beam Specimens from Tested Slab
Figure 6.14 shows how beam and cores were extracted from the test slab. Once the beam and core specimens were cut from the slab, they were tested conforming to ASTM C 39 and ASTM C 1399.

Figure 6.15: Beam Specimens Tested as Per ASTM C 1399
Figure 6.15 showed the typical beam specimen tested for flexural tensile strength conforming to ASTM C78. Figure 6.15 also shows the exposed side of the slab when loaded sideways. The total deterioration of volumetric stability, stiffness and bond strength between minibar and concrete were observed from these tests.

Similarly Figure 6.16 shows all the tested core and beam specimens.

![Tested Core and Beam Specimens for Forensic Analysis](image1)

![Tested Core and Beam Specimens for Forensic Analysis](image2)

Figure 6.16: Tested Core and Beam Specimens for Forensic Analysis
Table 6.3 and Table 6.4 present the summary of the compressive strength and flexural tensile strength test performed on core and beams specimens. It was observed that there is no any deterioration of the concrete compressive strength due to fire. This demonstrated that compressive strength of minibar reinforced concrete is not affected by fire.

Flexural tensile tests performed on the beam specimens cut from the slab showed a substantial degradation. The functional, geometrical and structural degradation of minibar during fire results in the absence of many force redistribution mechanisms that are generally actuated during the tensile failure. Sudden loss of volumetric integrity and bond between minibar and concrete during fire makes the force distribution mechanism such as crack-spacing (at lower load level) and crack bridging mechanisms ineffective, resulting in sudden loss in flexural tensile strength, followed by sudden and brittle failure of the structure.

Table 6.3: Compressive Strength of the Cores at Different Positions

<table>
<thead>
<tr>
<th>Core-ID</th>
<th>Position</th>
<th>Diameter (in)</th>
<th>Length (in)</th>
<th>Load (lb)</th>
<th>Strength (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C-1</td>
<td>Center</td>
<td>3.75</td>
<td>3.5</td>
<td>84870</td>
<td>7684</td>
</tr>
<tr>
<td>C-2</td>
<td>Center</td>
<td>3.75</td>
<td>3.75</td>
<td>103550</td>
<td>9375</td>
</tr>
<tr>
<td>C-3</td>
<td>Center</td>
<td>3.75</td>
<td>3.75</td>
<td>114620</td>
<td>10377</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td><strong>Average</strong></td>
<td><strong>9145</strong></td>
</tr>
<tr>
<td>C-10</td>
<td>Edge</td>
<td>3.75</td>
<td>3</td>
<td>94210</td>
<td>8529</td>
</tr>
</tbody>
</table>
Table 6.4: Flexural Tensile Strength of Beams at Different Positions and Orientation

<table>
<thead>
<tr>
<th>Beam-ID</th>
<th>Position</th>
<th>Direction</th>
<th>Loading-Plane</th>
<th>Maximum Load ( lb )</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-1</td>
<td>Edge</td>
<td>Longitudinal</td>
<td>Top</td>
<td>1009</td>
</tr>
<tr>
<td>B-2</td>
<td>Edge</td>
<td>Longitudinal</td>
<td>Top</td>
<td>1213</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td><strong>Average</strong></td>
<td><strong>1111</strong></td>
</tr>
<tr>
<td>B-16</td>
<td>Edge</td>
<td>Transverse</td>
<td>Top</td>
<td>2150</td>
</tr>
<tr>
<td>B-17</td>
<td>Edge</td>
<td>Transverse</td>
<td>Top</td>
<td>1910</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td><strong>Average</strong></td>
<td><strong>2030</strong></td>
</tr>
<tr>
<td>B-3</td>
<td>Center</td>
<td>Longitudinal</td>
<td>Side</td>
<td>1917</td>
</tr>
<tr>
<td>B-4</td>
<td>Center</td>
<td>Longitudinal</td>
<td>Side</td>
<td>1214</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td><strong>Average</strong></td>
<td><strong>1566</strong></td>
</tr>
</tbody>
</table>

To have a quantitative idea of strength deterioration due to fire, flexural tensile strength test was conducted conforming to ASTM C78 on beams reinforced with 0.75% minibar dosage. The average summary of the test results with and without fire loading is presented in Table 6.5. From the table it can be seen that there has been a 338% reduction on the maximum load carrying capacity and hence on the flexural tensile strength of minibar reinforced concrete due to fire. It can also be observed that there is a minimal reduction of 5% on concrete compressive strength.
Table 6.5: Summary of the Comparison between Regular Concrete Beams with Specimens with Fire Load

<table>
<thead>
<tr>
<th></th>
<th>W/O Fire</th>
<th>With Fire</th>
<th>% Reduction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum Load (lb)</td>
<td>6874</td>
<td>1569</td>
<td>338</td>
</tr>
<tr>
<td>Flexural Tensile Strength (psi)</td>
<td>1289</td>
<td>294</td>
<td>338</td>
</tr>
<tr>
<td>Concrete Compressive Strength, $f_c$ (psi)</td>
<td>9300</td>
<td>8837</td>
<td>5</td>
</tr>
</tbody>
</table>

6.7 Conclusions and Recommendations

This subsection presents the conclusions and recommendation based on the fire test performed at NCG, New York and the forensic analysis that followed at The University of Akron.

- The thermal performance of minibar reinforced concrete was not found to be satisfactory for structural purposes within the scope of the test performed. More fire tests with varying minibar dosage and member geometry is recommended.
- The tested slab was brought back to University of Akron. A forensic analysis to identify residual mechanical properties is performed.
• Residual compressive strength and modulus of rupture for different locations and orientations have been identified. This was achieved by cutting core and beam specimens from the tested slab pieces.

• Compressive strength of the concrete was found not to have degraded, however, severe reduction in flexural tensile strength was observed. For the different locations and orientations, the beams sawn from the edge along longitudinal direction loaded at top yielded the lowest flexural tensile strength.

• Microscopic study of exposed surface of the slab is recommended in the next phases of the project.
This chapter includes a summary, conclusions and recommendations on the various parts of project that was conducted for the overall structural characterization of basalt minibar reinforced concrete. Minibar reinforced concrete (MRC) is a type of concrete reinforced with randomly distributed new type of material called basalt minibar. Basalt minibar is a new kind of fiber for which mechanical characteristics were the subject of this study. Minibar is a new non-corrosive structural macro fiber made from basalt fiber reinforced polymer (BFRP) material. It is designed to be used as a proactive reinforcing material in concrete. For every new material, strength, stiffness, toughness, durability and workability are the five major factors that need to be understood initially. A structural element must be sufficiently strong and stiff to carry loads, must be tough to absorb energy during different kind of loadings, must be durable against adverse weather and at the same time must be easy to make. Primary motive behind this investigation was the overall characterization of basalt mini bar reinforced concrete in all of those respects. This dissertation consists of six chapters to cover all those topics. It was found out that basalt minibar and minibar reinforced concrete have many excellent properties to serve many good structural purposes. Some of the important factors that were noted are listed below:

- Minibar has high tensile strength.
• Minibar reinforced concrete possesses good toughness characteristics resulting in excellent post-cracking behavior.

• Due to high tensile strength and good bond with concrete matrix, Minibar reinforced concrete can be used as a load carrying agent. Most of the synthetic fibers are generally used as the toughening agent. They actuate different force-distribution mechanisms as the concrete cracks. Minibar helps the member to develop good energy absorption capacity and can also provide significant load carrying capacity after the concrete cracks.

• Minibar reinforced concrete was found to exhibit better flexural tensile strength and post-cracking tensile strength with increasing volume fraction. For higher volume fraction of 4%, it was able to carry stress equal to its flexural tensile strength even after the cracking of concrete. This is particularly encouraging because of its better workability at higher minibar dosage. Minibar at dosages ranging from 0.35% to 4% by volume, which is 6.3kg/m³ to 72 kg/m³ by weight, was mixed with concrete without any difficulty. There was no bleeding, balling or segregation in the concrete at higher minibar dosage mix. This has been possible due to its lower specific weight as compared to concrete, which makes the minibars to evenly distribute in the concrete. Such features may be hard to achieve with the metallic fibers due to their higher specific weight with respect to concrete such that they tend to ball at higher fiber dosage. It can be said that with minibar, an excellent optimization between workability, strength and toughness was obtained.
• Handling, placing, consolidating and finishing of concrete was possible without any special measure.

• At higher minibar dosages, the specimen was able to develop multiple cracking. This can be viewed as the development of better stress-redistribution mechanisms, leading to more ductile and tougher response.

• The behavior of minibar reinforced concrete at elevated temperature was not found satisfactory. Under fire load, as the polymer matrix reached its glass transition temperature, minibar was observed to lose its volumetric integrity and bond with the concrete. This leads to a substantial reduction on the flexural tensile strength.

7.1 Potential Structural Applications of Minibar Reinforced Concrete

Based on the experimental program conducted at The University of Akron, different mechanical properties of minibar reinforced concrete of structural interest were identified. Equipped with light weight, corrosion resistance and better durability properties, 0.66 mm Minibar can develop a direct uniaxial tensile strength of 160 ksi. Flexural tensile strength value was able to reach up to 15 MPa for minibar dosage of 4 % with 62 MPa concrete strength. The average residual strength value was able to reach up to 17 MPa for minibar dosage of 4 % with 62 MPa concrete strength. It was noted that minibar can be used as shear and flexural reinforcement in concrete beam members. The presence of minibar was found to encourage better ductile response, thus was able to alter the brittle shear failure modes. With all the good features, minibar can be viewed as an emerging new material with various practical applications. Several practical applications
have been successfully demonstrated in Europe showing their applicability in construction of pontoons, inner walls and façade walls [5]. Shotcrete, bridge decks, pavements, double-Tee precast beams, roofing tiles, balconies and agricultural products are some of the applications being considered. Other potential structural applications of minibar in civil engineering includes earthquake sensitive locations, corrosion sensitive structures (such as sea wall in coastal areas) and also as an inhibitive measure against brittle and catastrophic explosion of concrete.

7.2 Summary of the Different Chapters

Among seven chapters, the dissertation document comprises of five chapters which deal with the various experimental programs that were conducted pertaining to minibar reinforced concrete. This section comprises brief summary of each chapters followed by conclusion and recommendations.

7.2.1 Chapter II: Size-Effect and Its Significance on FRP Rebar Material Utilization

The chaptered covered the literature review, laboratory procedures, experimental program and analysis that was conducted for the study of size-effect associated with BFRP bars. The following conclusions and recommendations were made:

1. Uniaxial tensile strength and modulus of elasticity of basalt fiber reinforced polymer bars (BFRP) of varying sizes were identified.
2. BFRP bars exhibit size-effect on its uniaxial tensile strength, that is, strength varies inversely with its diameter.
3. Within the range of sizes tested, longitudinal modulus of elasticity of BFRP bars was found to be independent of their sizes.

4. Statistical study of the size effect exhibited by BFRP bars on uniaxial tension is studied in the light of Weibull’s weakest link theory. A modification was proposed and it was found to predict the strength of larger sized BFRP bars with good accuracy.

5. It is recommended to conduct more tests on basalt FRP bars of different diameters to have a sufficient substantiation of Weibull’s hypothesis as amended in this dissertation.

7.2.2 Chapter III: Mechanical Characterization of Minibar and Minibar Reinforced Concrete (MRC)

The chaptered covered the literature review, laboratory procedures and experimental program that was conducted for the identification of material properties of basalt minibar and minibar reinforced concrete. This part will be further useful to correlate with toughness and strength characterization of minibar reinforced concrete. The following conclusions and recommendations were made:

1. The average uniaxial tensile strength, rupture strain and modulus of elasticity of basalt minibar was determined. The tensile strength was found to be 160 ksi.

2. The average uniaxial tensile strength of contemporary glass minibar was determined to be 138 ksi.
3. Basalt minibar was found to be stronger than glass minibar in uniaxial tension for the same size. It was also observed that Basalt minibar is capable of undergoing larger tensile rupture strain than its glass minibar counterpart of the same size.


5. The modulus of elasticity for 0.5% volume fraction minibar reinforced concrete (MRC) was determined to be $4.46 \times 10^6$ psi, which is 11.5% increment on the ACI specified value for plain concrete of same compressive strength.

6. The average Poisson’s ratio for minibar reinforced concrete was determined to be 0.23. The Poisson’s ratio of plain concrete is generally in the range of 0.15 to 0.2.

7. Minibar reinforced concrete was observed to exhibit strain-hardening effect under direct uniaxial tension. However, more tests with varying minibar dosage and specimen-geometry are recommended.

8. Minibar reinforced concrete was observed to exhibit strain-softening effect under compression, hence better deformation characteristics compared to its plain counterpart. However, more test with varying minibar dosage are recommended.

7.2.3 Chapter IV: Toughness Characterization of Minibar Reinforced Concrete (MRC) and its application

This chapter covered the literature review, laboratory procedures, experimental program and theoretical analysis in relation to the identification of flexural tensile strength, post-cracking tensile strength and energy absorption capacity of minibar reinforced concrete. Theoretical model was proposed to predict the number of minibar across the crack plane.
of a fractured beam. A theoretical model was also prepared to predict the load carrying capacity of minibar reinforced concrete beams. The following conclusions and recommendations were drawn:

- The failure mode of test cylinders made from Minibar reinforced concrete (MRC) for the determination of compressive strength changed from brittle mode to ductile mode due to the addition of Minibar as with other types of fibers.
- Linear equations for flexural tensile strength of MRC as a function of concrete compressive strength and minibar dosage were developed from the test results.
- The average residual strengths (ARS) obtained for Minibar reinforced concrete were much greater than expected, suggesting that Minibar has significantly helped in the post-cracking performance of concrete in the current test program. Empirical equations were suggested.
- One test per ASTM C1609 was performed for each of dry mix series 1 and 2 with 1.89% dosage. The test results demonstrate that MRC satisfies ACI 318-08 requirements that are specified for steel fiber reinforced concrete for using fiber as shear reinforcement.
- An analytical expression was proposed for the number of mini-bars across a unit area of cracked surface.
- An analytical model was proposed to predict the ultimate load carrying capacity of MRC beams for different volume fractions. More tests are being conducted to correlate the model with more data and the results have been promising.
7.2.4 Chapter V: Flexural and Shear Characterization of MRC Beams of Varying Shear-Span to Depth Ratios

This chapter covered the literature review, laboratory procedures, experimental program and theoretical analysis in relation to flexural and shear characterization of minibar reinforced concrete beams. Minibar reinforced beams were of laboratory and structural scale. The variables that were considered for analysis were minibar dosage, shear span to depth ratios and cross-sections. An analytical model that was derived in previous chapter was extended for the flexural analysis of minibar reinforced concrete beams. A mechanics based model was developed for the prediction of shear strength. The following conclusions and recommendations were made from the overall project:

- Modified Discrete Fiber Property approach is able to predict the ultimate load carrying capacity of MRC beams of varying shear-span to depth within reasonable accuracy.
- Shear-Strength is found to increase by almost 15% with the addition of 0.5% mini-bar.
- The mechanics-based analytical model is able to predict the shear-strength of MRC beams with good accuracy. However, more tests with different volume fractions of mini-bar are needed.
- There is no notable trend on the distribution of fibers along the depth of the beam.
- With the addition of 0.5% fiber, there is an average increase of 42% on the moment-strength as compared to its plain-concrete counterpart as per ACI 318. For 0.75% volume-fraction of mini-bar, there was an average increment of 82% for large beams.
• With the addition of 0.5% mini-bar, an average increase in moment strength of
13% relative to the moment strength of the corresponding plain concrete beams
was observed. For 0.75% volume fraction, the percent increase of moment
strength was 25 and 55% for 118” and 59” long beams respectively.
• From the load-deflection plots, it can be observed that apparent ductility of beams
improved with the addition of fibers.

7.2.5 Chapter VI: Fiber Reinforced Concrete Beam in the Context of Elevated
Temperature

This chapter covered the literature review, laboratory procedures and
experimental program pertaining to the study of minibar reinforced concrete slab under
elevated temperature. The variables considered for the study was minibar dosage of 0.75
% by volume. One concrete slab was prepared at the University of Akron and was
transported to Buffalo, New York for testing. The tested slab was brought back to the
University of Akron for forensic analysis. Following conclusions and recommendations
were made from the study:

• The thermal performance of minibar reinforced concrete was not found to be
satisfactory for structural purposes within the scope of the test performed. More
fire tests with varying minibar dosage and member geometry is recommended.
• Compressive strength of the concrete was found not to have degraded, however,
severe reduction in flexural tensile strength under fire was observed. A severe
degradation of nearly 350 % flexural tensile strength was observed. Microscopic
study of exposed surface of the slab is recommended in the next phases of the project.

7.3 Concluding Comments

Minibar is a new non-corrosive structural macro fiber made from basalt fiber reinforced polymer material (BFRP) with high tensile strength and with an aspect ratio of 60. It is to be appreciated that the development of this novel material is the result of cumulative research on strength and durability characterizations, leading to subsequent modifications on manufacturing processes and mechanical properties of basalt fiber reinforced polymer (BFRP) bar. BFRP bars were primarily developed as a potential internal reinforcement in concrete structures [9, 13, 15, 24, 28]. One of the primary motives was the mitigation of corrosion of steel reinforcement in alkaline concrete environment [1, 4, 6, 12, 10]. The material was also expected to improve the functional (shrinkage, ductility) and long-term behavior of concrete reinforced with it [2, 3, 5, 23]. BFRP material has also been comprehensively investigated as the external retrofitting material [17, 20, 21, 25, 26, 27]. Based on the detailed research on particular material, basalt mini-bar was finally conceived which is expected to yield various meritorious features, structural and functional [8, 19]. Within the detailed research program conducted in different phases, various properties of minibar were determined. It is a light weight and high strength material which showed great workability with concrete even at higher fiber dosages. The material was easy to handle, place and finish. Minibar concrete was able to reach higher values of flexural tensile strength, higher post cracking tensile strength and exhibited better ductility properties. There seemed to be a good optimization
between the higher tensile strength of minibar and its lightweight property. Its lightweight enabled to make concrete with high minibar dosage that could reach good flexural tensile strength. Minibar was found to have an excellent potential as a flexural and shear reinforcement and was found to improve the toughness and strength characteristics of large scale beams. Competing along with other high strength material like steel fiber and high performance synthetic fibers, minibar showed a very good potential as a novel material that can provide the hybrid response of both types at the same time. The unsatisfactory performance of minibar against fire load inspired the need for more investigations with better polymer material. Study of durability properties of minibar and minibar reinforced concrete, and response against fatigue and creep will be the next promising phase, while study of fatigue behavior of minibar reinforced concrete is already being carried out at the University of Akron.
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