PERFORMANCE OF REINFORCED CONCRETE BEAMS RETROFITTED WITH FIBER-REINFORCED SHOTCRETE AT EARLY AGE

By

REID HOLLAND

A thesis submitted to the

School of Graduate Studies

Rutgers, the State University of New Jersey

In partial fulfillment to the requirements

For the degree of

Master of Science

Graduate Program in Civil and Environmental Engineering

Written under the direction of

Hani H. Nassif

And approved by

____

____

____

New Brunswick, New Jersey

October 2019

ABSTRACT OF THE THESIS

Performance of Reinforced Concrete Beams Retrofitted with Fiber-Reinforced Shotcrete at Early Age

By REID HOLLAND

Thesis Director:
Hani H. Nassif

Similar to how a lot of our country's land has already been developed, a lot of our necessary structures have already been built. Our infrastructure relies on old bridges, buildings, dams, etc. which produce a massive economic strain in the form of repair and maintenance. Tearing down an old structure to create a new one is not nearly as economical as its improvement or repair. These economic factors force our hands as engineers to produce new technologies to increase economic efficiency, one of them being shotcrete. In recent years, shotcrete has gained a lot of traction in its usefulness for repairing and retrofitting due to its ease of application and bond strength to the applied substrate.

In this study, the effects of shotcrete on the repair of reinforced concrete beams is examined using fiber reinforced shotcrete (FRS) with and without a steel or basalt mesh for a total of 3 different shotcrete layer types. Each mix is tested at two different curing times after the shotcrete is applied, 3 days and 7 days. The effectiveness of fiber reinforced shotcrete is monitored using synthetic macro fibers (1.5”) in a shotcrete laminate versus the same fibers in

self-consolidating concrete (FR-SCC). The shotcrete proved to have a higher ultimate strength, lower crack width, and higher deflection compared to a concrete beam compared to the FR-SCC laminates and Full Class A beams.

In order to simulate the already deteriorated reinforced concrete beam in a controlled lab environment the beams, stirrups were exposed at the bottom where the shotcrete is applied. Included in the shotcrete layer is a steel mesh or basalt mesh with 1 square inch openings to produce a shotcrete laminate, while also studying an FR-SH layer with no mesh. Of the two meshes used, steel proved to be the most beneficial for ultimate load, while basalt was more beneficial for deflection. Having no mesh also proved to be a viable design option, however, provides no benefit over the two except for early age ductility.

Curing methodology is also studied in order to better gage optimal curing regimes. The three methods examined were wet burlap, curing compound, and dry curing where dry curing involves only wrapping the concrete member in plastic. Of the three, the most consistent performer was wet burlap. Curing compound proved to be more erratic, though overall similar to wet burlap and dry curing proved to have good early properties, however is not considered sufficient for later stages.

In addition to beam testing, a comparison between the hardened properties of shotcrete were compared under different methods. Cylinders were shot directly by the nozzleman during casting and cast by hand using the same mix before being shot in both 4”x8” and 6”x12” cylinders. These were compared with cored samples to see which results more closely relate to the cylinders to help produce an easier method of testing the shotcrete’s properties. Overall, only compression and tensile strengths were relatable due to constraints of how the cylinders can be tested. Cored samples cannot be tested using the same methodology and would have to be developed. For compression strength, the 4”x8” cylinders cast by hand produced results very similar to the cored samples. For tensile strength, the 4”x8” cylinders shot by the nozzleman were very similar. Both have potential to be used as a substitute to cores.

Acknowledgments

I would like to thank Dr. Hani Nassif for his role as my professor and advisor on this project and providing me with an opportunity to learn from him. His experience and leadership was necessary and important since day one and his guidance provided me with the support necessary to take on this project.

This project would not have been completed without the help of Dr. Adi Abu-Obeidah who acted as a member of my graduate committee and a mentor during my time as a graduate assistant at Rutgers. His role has been unmatched in the progression of my research experience and has provided me with clarity and stability in the pursuit of my degree.

Other students have provided me with support in various tasks. Mina Habib, Jonathan Rodriguez, John El-Kouri, Andrew Shehata, Emily Pooley, Albert and Alain Hajjmoussa, Wassim Nasreddine, and many undergraduates helped me in advice on procedures, production of testing materials and testing, which proved to be invaluable in completing the project.

Frank Townsend of Superior Gunite was instrumental in the progress of this work. He has provided insightful comments on the use of shotcrete and its application process, being the necessary expert opinion needed on projects such as this. I would like to thank him and his team for applying the shotcrete expertly as well as supplying various shotcrete technology needed to our site.

I would also like to thank the unwavering support of my parents. My father has provided me with support on how to accomplish what I want while showing me the importance of having an inquisitive mind. My mother helped show me that I can do whatever I want as long as I truly

want it, and provided me with the self-confidence required to push myself by believing in me and my pursuits.

Table of Contents

List of Tables

List of Figures

Introduction

In order to highlight the necessity of this work, along with generalizing the problems that can be attacked using the results, the problem statement and the objective is presented.

1.1 Problem Statement

The deterioration of concrete structures is a guarantee, and therefore must be maintained in order to continuously produce the originally intended strength that the structure or member requires. Specifically in the case of bridges, concrete is exposed to many external factors that attack and degrade the material such as salts, water, excessive loads, and car accidents. All can cause severe damage to the bridge potentially resulting in critical failure, and possibly a human life. Current practices to ensure the structural integrity of a bridge include the regular maintenance of the bridge by regulated bridge repair manuals. In the case of a deteriorated beam, the most economical approach is to repair or retrofit the member in question before the damage is severe enough to produce a critical failure. Usually, when beams are extremely damaged to the point of concern, they are replaced.

New methods are in high demand to prevent the complete renewal of deteriorated beams. Shotcrete is well renowned for its retrofitting capabilities due to the simplicity of the application, the speed of its application, and the strength of its bond, however there is very little research done on its behavior with reinforced concrete beams.

1.2 Objective

Along with the effect of various curing techniques, the effect of macro synthetic fibers in shotcrete is monitored through the mix design's mechanical properties and compared to a control class A concrete mix and an FR-SCC mix design.

The effect of different laminate reinforcement was also studied during this period. Both basalt and steel meshes were used to produce a ferrocement laminate and compared to a control laminate with no mesh. Studies show that thin laminate layers such as the one used in this study are particularly weak in tension compared to their substrate. The steel mesh consisted of a 16 gage galvanized steel wire mesh and both the steel and basalt mesh included square inch spacing. The objective of this study is to see the differences in flexural strength of the shotcrete laminate using the various laminate setups to provide information on optimal laminate properties.

Another comparison made in this study is the difference in curing regimes on the laminate properties. Curing compound, wet burlap, and dry curing were conducted on each set of laminate setups. Curing compound is exceptionally popular in large pours, and in the case of shotcreteing, is commonly used. Curing regimes greatly vary in terms of effectiveness and change the maturity of the concrete member, thus the differences in the three are desired to better understand how shotcrete cures with regards to its strength.

Chapter Synopsis

Chapter 1 presents the problem statement and present current data revolving around the necessity of this study for the advancement of our infrastructure.

Chapter 2 details the literature review that went into this study in order to prepare methodology and various hypotheses. It includes and introduction, review of current repair methods in retrofitting and repairing concrete members, and an overview of shotcrete and fiber reinforced materials.

Chapter 3 details the experimental program and procedures. An explanation of all testing parameters and ASTM regulations are included. It also provides the detailed process of how the beams were created and tested.

Chapter 4 Details the results of the tests described in chapter 3. It also includes an analysis of each data point, making sure to explain the meaning of the results. Major points in chapter 4 are comparisons of the ultimate load, ultimate deformation, crack propagation, differences in materials used for the laminates, differences in curing methods, and differences in full beam materials.

Chapter 5 concludes and summarizes the major points in chapter 4 and provides insight on what should be done in the future to further the study of fiber reinforced shotcrete in beam retrofitting.

Literature Review

A large portion of the future of our highway infrastructure is the assessment of current bridges. Ultimately, all bridges will exceed their designed lifespan, and thus must be repaired. The definition of a structurally deficient bridge has changed recently, and is defined by the Federal Highway Administration as:

> Structurally deficient will be defined in accordance with the Pavement and Bridge Condition Performance Measures final rule as a classification given to a bridge which any component (deck, superstructure, substructure, or culvert) is in poor or worse condition (code 4 or less).

As of 2017, the FHWA has reported that 9% of our bridges fall under the definition stated above.

According to ASCE's 2017 Infrastructure Report Card, 9.1% of our bridges are structurally deficient. Out of the 614,387 bridges built by the end of 2016, that means 56,001 bridges are structurally deficient, meaning that bridge rehabilitation would need approximately $123 billion. A major part, as reported, is the age of bridges. 39% of all bridges in the United States are 50 years or older in age.

Another observed major cause to the deterioration of bridges is the increase in truck traffic and truck weight. According to a report by the transportation research group TRIP, in the state of New Jersey, $816 billion worth of product is shipped to and from it where 73% of the goods are shipped with trucks. Overloading cause's major structural cracks and reduces the overall service life of the bridge. Other major issues include deicing salts, shrinkage, and freeze/thaw, play a role. It is apparent that the determination of our bridges is inevitable, at least by the aforementioned reasons, stressing the need for repair methods.

On the topic of repair, reinforced concrete beams can be difficult and expensive to approach. Many of the retrofit and repair techniques such as ferrocement, fiber-reinforced polymer, grouting, and epoxy injections have major disadvantages that could lead to the lack of their consideration.

2.1 Current Repair Methods

Fiber reinforced polymer (FRP) has grown in popularity of the recent decade because of its use of durable fibers that can produce as-needed qualities in relation to strength, corrosion resistance, and weight. FRP is an epoxy resin plate that is typically reinforced with carbon, glass or aramid fibers. The plate is applied externally to a structural member in order to increase its strength. It has been noted to work particularly well with reinforced concrete beams with respect to flexural and shear strength (Jabr 2017).

Also noted are the major drawbacks of using FRP. Most importantly, its application in harsh environments, brittle behavior, and cost. FRP cannot flourish if subjected to harsh environmental factors (Frigione and Lettieri 2018). Most notably, epoxy resins can absorb a substantial amount of water, reducing both the strength and the stiffness of the FRP. De-icing salts are also extremely harmful to the epoxy resin. An excess of heat or direct sunlight degrade the adhesion strength and degrades the FRP's ability to transfer stresses appropriately between the member it is applied to such as reinforced concrete beams.

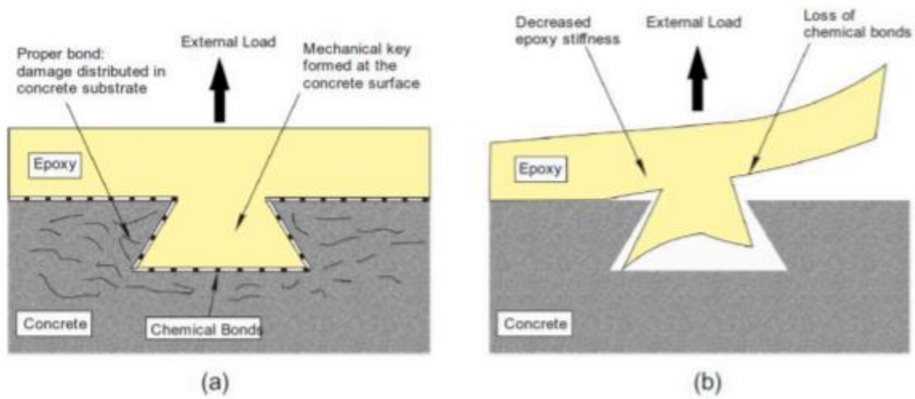


Figure 2.1 – Failure mode of epoxy-concrete bond: (a) in dry ambient conditions; (b) following exposure to moisture. (Blackburn et al. 2015)

Combined with lack of capabilities in extreme weather conditions, FRP is naturally brittle, causing sudden failure.

Similar to FRP, however more common, is steel plating. Steel plating is low cost with easy to obtain materials and improves shear and flexural resistance (Alam et al. 2014) while also restoring load bearing capacity to damaged or degraded beams (Aykac et al. 2013). It involves anchoring steel plates to the failure point in question using bolts or an epoxy. Like FRP, there are issues with the adhesive strength of the epoxy that connects the steel plate to the concrete member. The epoxy can degrade and decay due to high temperatures and excessive moisture. Different from FRP is the use of exposed steel, which can corrode easily, as it is applied externally.

Another popular repair method of reinforced concrete members is the use of ferrocement jackets surround the degraded location. Ferrocement jackets are durable and cheap and produce a large amount of deformation before cracking. It is applied using a steel wire mesh as tensile reinforcement and cast using concrete with high workability using super plasticizers or a mortar.

It is observed to increase load carrying capacity and increase stiffness of reinforced concrete beams (Dhanoa et al. 2016) and columns (Mourad and Shannag 2012). It is also observed to increase cracking capacity (Nassif and Najm 2004). The materials that are in use can be easily obtained and for relatively cheap prices.

Ferrocement laminates have versatility in the material used, making it a desirable repair method. SCC has been used to great effect as a laminate material over a steel mesh, increasing the cracking load performance as well as its deflection and crack width behavior (Sholy 2018).

The major issues with ferrocement jackets lie in the bond between the ferrocement and the substrate along with the large amount of people needed to complete its application. Tying the mesh in place takes a long time, and can be detrimental when used in larger projects due to labor costs. Also, the ferrocement layer must fully cover the steel mesh to avoid heavy corrosion, and also implement shear studs to ensure composite action (Nassif and Najm 2004).

2.2 Shotcrete

Gunite, or sprayed concrete, was created and used in 1907 by blowing dry aggregate and cementitious material out of a hose with compressed air and applying water at the nozzle by Carl Akeley. Gunite would later be renamed to dry-mix shotcrete as wet-mix shotcrete became popular in the 1970s by use of a concrete pump and more improved techniques. Shotcrete is continuously used around the world whenever the production of formwork becomes an issue. Shotcrete is now defined in ACI 506r-16 as “A method of applying concrete projected at high velocity primarily on to a vertical or overhead surface.”

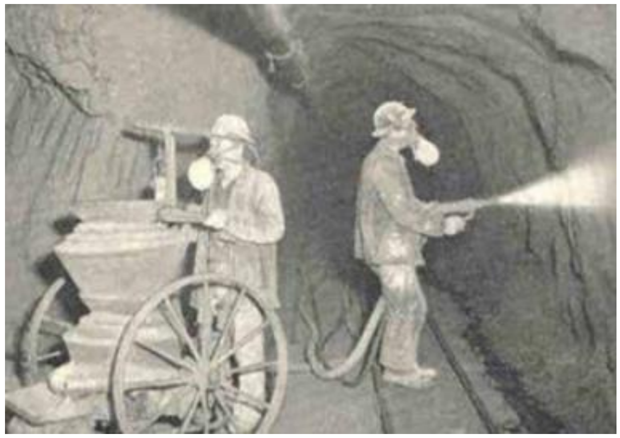


Figure 2.2 – Lining a tunnel with gunite using a double-chamber gun during the 1920s (Morgan and Bernard 2017)

Shotcrete has greatly advanced in its machinery use, producing better and more efficient guns, pumps and nozzles for the job.

The first double-chambered cement gun was created in 1910 which would spearhead shotcrete into mainstream construction. At the time, shotcrete had rapidly gained in popularity due to having better strengths than conventional placed concrete due to the lack of information on the consolidation of general concrete and poor techniques. It was not until the 1950's that wet-mix shotcrete was produced, however still required improvement. ACI committee 506 was also created as demand for shotcrete became more pronounced, so did the demand for research and regulation on the product. The 1970's allowed for greater advancements in the material and machine, as silica fume was introduced to concrete mix designs, reducing rebound and increasing bond strength of shotcrete, and a wet-mix shotcrete pump was created to more effectively push the heavier aggregate due to the addition of water (American Shotcrete Association).

When applied, concrete requires formwork so that it may produce the appropriate shape necessary to fit the structure's needs. The cost of producing formwork is high, can be exceptionally time consuming, and can require specialists depending on the required shape. This also means that reapplying concrete to structurally damaged members generally requires the removal of that member, rendering the structure unusable until its repair. Shotcrete requires no formwork and is self-consolidating by product of the high velocity shooting. It produces a similar if not exact properties to conventional cast-in-place concrete save for the bond which is proven to be exceptionally strong (American Shotcrete Association).

Because shotcrete is a matter of machinery rather than material, as it is exactly like conventional concrete, it has gained great popularity around the world, reported to be an $8.3 billion dollar market by 2021 with underground construction as its greatest application. Europe currently holds the highest market share for shotcrete where there is a large demand for underground transportation.

As stated before, shotcrete is concrete sprayed at a high velocity. There are currently two major types of mixes:

Dry-Mix: Aggregate is batched and pumped to the nozzle dry where water is introduced only at the nozzle.

Wet-Mix: Aggregate is batched similarly to conventional concrete, then pumped using a wet-mix pump to the nozzle.

Generally, dry-mix shotcrete is used for smaller projects and provides more control of the water-cement ratio in the concrete mix to better suit the location being sprayed. Because dry-mix does not have any fresh concrete properties that can be appropriately measured due to the variable water content at the nozzle, conventional concrete testing methods cannot be used.

Wet-mix shotcrete is used for a much higher volume requirement and projects that allow for transit mixers to be transferred on site and constantly mixed. Although the water-cement ratio cannot be changed on the spot to better match the variable conditions, the mix is more consistent with its water content. Because the wet-mix is batched and mixed the same way as conventional concrete, it can be tested in a similar fashion with regards to fresh properties.

2.3 Nozzleman and Applications

The most important aspect of the shotcrete is its application, and therefore the person manning the nozzle, or the nozzleman, must be experienced. ACI states that no nozzleman should be used who isn't certified for the specific mix, wet or dry. Shotcrete nozzleman-in-training requires 25 hours of shotcrete work experience and the pass of a performance examination. The nozzleman-in-training must then complete 475 more hours of shotcrete work experience to qualify to take the ACI written examination and another performance examination. Recertification is required every five years and requires a verbal interview and performance examination. If a person wishes to become certified for both wet and dry mixes, they must pass for both mix types.

Things that a nozzleman must think about are rebound, overspray, angle of application, and encasement. Rebound is an unavoidable by product of shooting aggregate that has not been fully encased by cementitious material at a high velocity onto a hard surface. The aggregate will reflect and possible cause a buildup at a point, which will have an excess of aggregate causing a weak point. Rebound can be contained based on the nozzle angle, amount of accelerator, distance of the nozzle to the applied substrate, and area of application.

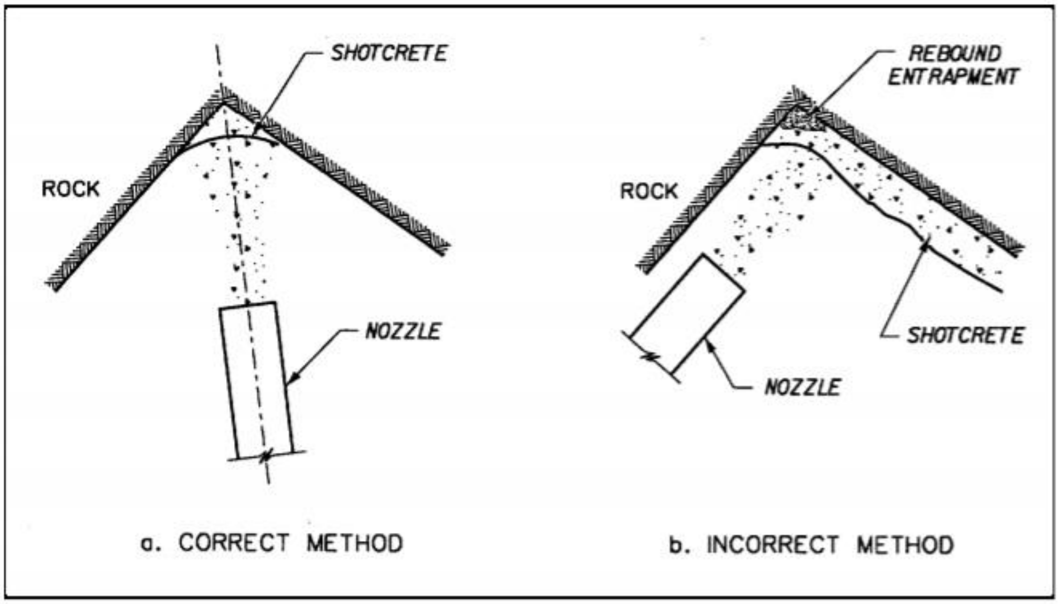


Figure 2.3 – Shotcreteing interior corners (Mahar et al. 1975)

Overspray is similar to rebound, however is used to describe when small or fine materials bounce off a surface and stick to adjacent areas that the nozzle is not directly spraying. This causes a layer with too little coarse aggregate and affects the overall effectiveness of a shotcrete layer.

The angle of application is important when attempting to reduce overspray and rebound, and make sure there is a strong bond between the shotcrete and the substrate. It is defined by the posture and position of the nozzleman, and where he points the nozzle.

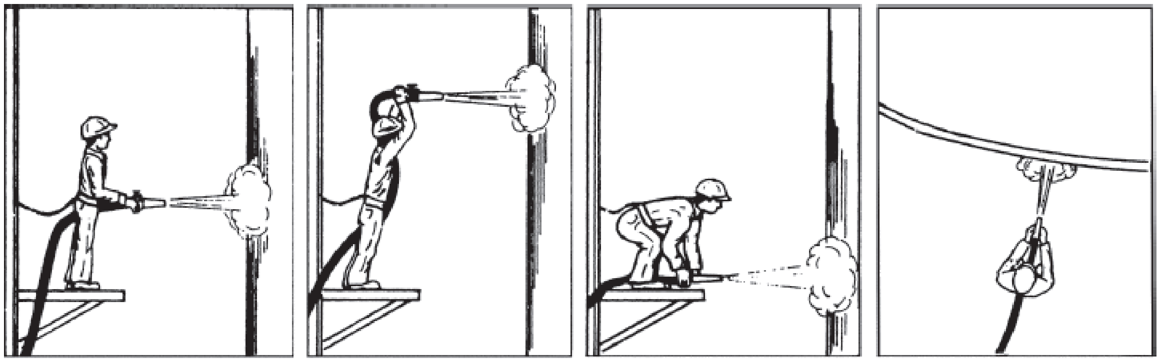


Figure 2.4 – Correct shooting positions (ACI 506r-16)

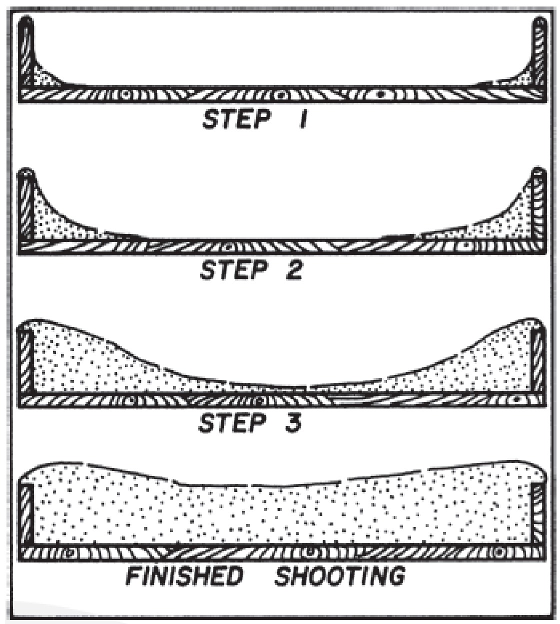


Figure 2.5 – Proper procedure for shooting horizontal surface (ACI 506r-16)

Encasement is what is used to describe the degree at which various protruding elements, such as rebar and a steel mesh, are covered by the shotcrete. Good encasement means that there are no voids along the item, bad encasement implies many or large air voids that usually show up directly behind the element.

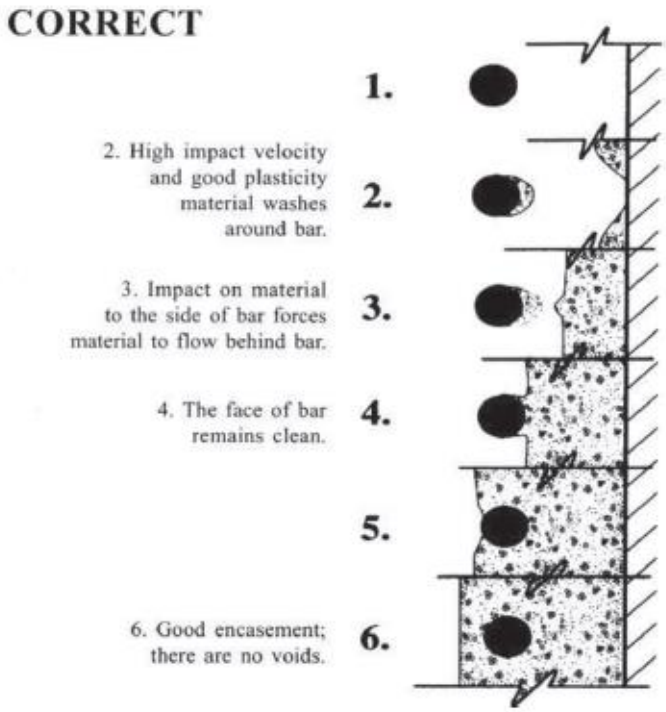


Figure 2.6 – Illustration of correct steps of steel encasement (ACI 506r-16)

2.4 Shotcrete Equipment

There is a variety of equipment that is used for a general shotcreteing job. The main pieces include an air compressor, gun or pump, hose, nozzle, and blowpipes. There are extra considerations depending on the job which may include remote shotcrete gun, fiber feeders, admixture dispensers, and air movers.

2.4.1 Dry-mix Equipment

If the nozzleman is the brains of shotcreteing, the equipment is the heart. Behind every successful job using shotcrete are well maintained guns, air compressors, hoses, manifolds, etc. For dry-mix shotcrete, the main equipment used is the gun and the air compressor. Batch and double-chamber guns are used effectively by using a rotary feed wheel to meter the flow of the batched material being expelled from its pressurized lower chamber. This allows for a constant flow by way of the material being supplied to the top chamber which is then moved to the pressurized chamber. Dry-mix guns are continuous-feed guns or Rotary guns and are by far the most popular gun to use for dry-mix shotcrete. Rotary guns use a rotating airlock that allows for the material to be pressurized while continuously fed through the chamber.

Figure 2.7 – Rotary gun (ACI 506r-16)

Compressed air is placed in the lower chamber in order to push out the material at the necessary velocity. The air come from an air compressor which is to meet the specified requirements of the shotcrete mix type, and the inside diameter of the hose being used. Dry-mix shotcrete requires a more powerful air compressor.

Interior Diameter of Hose (in)	Compressor Capacity (CFM)
1	350
1.25	450
1.5	600
2	750
2.5	1000

Table 1.1 – Dry-mix compressor capacity based on inside diameter of hose (ACI 506r-16)

The flow rate is considered at a pressure of 100 psi, however it is important to note that the operating air pressure may change depending on the length of the hose in use. This is because the outlet of the hose must obtain a certain pressure in order to achieve the velocity needed to push out the shotcrete material. The longer the hose, the more material that must be pushed through in order to reach the outlet, thus the pressure may need to be increased.

Shotcrete nozzle are specialized for each mix. Dry-mix shotcrete nozzles usually contain a nozzle tip, control valve, water ring, and water body and is generally a hydro-mix nozzle which mixes the water through the nozzle body. The nozzle body is separate from the nozzle tip, unlike other nozzles. It acts as a presetting system to wet the dry-mix shotcrete that is being pumped into it so that the shotcrete is wet before leaving the nozzle rather than mixing the water as it is leaving. This provides as even a material property as possible. That being said, this is noted to not remove the need for pre-dampening the shotcrete mix before being pumped, as the nozzle body obviously doesn’t help pump the mix through the hose.

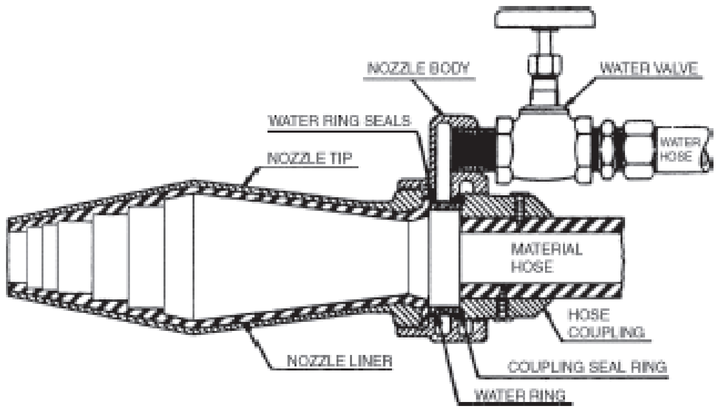


Figure 2.8 – Dry-mix nozzle (ACI 506r-16)

2.4.2 Wet-Mix Equipment

Unlike dry, wet-mix pumps concrete already mixed with water, meaning the concrete being pumped weighs more and has higher workability. A wet-mix pump injects the concrete through a tube into the delivery hose hydraulically, usually using piston pumps. Larger jobs require stronger pumps, as the time needed to be shot is consistent from job to job, however the amount is not. Therefore, large jobs can require shotcreteing rates of 8 yd^3/hr to as much as 30 yd^3/hr in order to finish the project. These pumps have larger large outlet diameters and pistons.

In the case of retrofitting, a generally smaller job category, smaller pistons, outlet diameter, and is applied at a slower rate. This is usually defined for a total mix size of 1.5 to 3 yd^3 of total concrete and are shot at a rate of around 2 yd^3/hr. It is noted that large pumps can be used for smaller jobs, but is generally not recommended, as it required a particularly skilled nozzleman to be able to assure the quality of the application.

With regards to compressed air, instead of applying the compressed air before the inlet of the delivery hose, it is applied at the nozzle. The sole purpose of the compressed air in wet-mix shotcreteing is to break up the clumped concrete that has been pumped by the wet-mix pump and increase exit velocity so that it evens out the application and increases the bond. Because the air compressor is not used to pump the shotcrete, the flow rate capacity of the air compressor does not have to be as high as the dry-mix, and is typically only 200 cfm to 400 cfm at 100 psi (ACI 506r-16).

A wet-mix nozzle includes a rubber nozzle tip, housing, air injection ring, and a control valve. There is no need for extra manifolds for potential admixtures, as the mix already includes both liquid and powdered admixtures. Wet-mix nozzles are much easier to use in that the hose easier to maneuver and rotate while shooting.

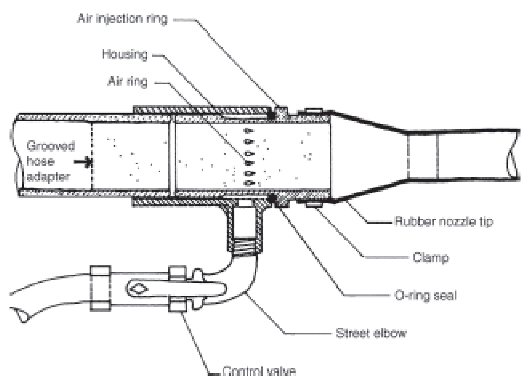


Figure 2.9 – Wet-mix nozzle cut-away (ACI 506r-16)

2.5 Fiber Reinforced Shotcrete

In the 1960s, fiber reinforced concrete (FRC) introduced new ways of increasing the structural dependency of concrete. In terms of mechanical performance, fibers have been observed to increase structural strength, reduce permeability, reduce shrinkage and expansion, and increase overall durability (Kakooei et al. 2011). The two most common fibers used for enhancing concrete are polypropylene and steel fibers.

Fibers have been used to enhance shotcrete since the early 1970s. Because conventional concrete is so close to shotcrete save for application, fiber reinforced shotcrete caught on relatively quickly. Fiber reinforced shotcrete (FRS) is “mortar or concrete containing discontinuous discrete fibers that’s is pneumatically projected at high velocity onto a surface.” Steel, glass, and synthetic fibers are used in shotcrete with steel being the most popular.

2.5.1 Polypropylene Fibers

Polypropylene fibers (PPF) are a type of synthetic fiber commonly used on concrete mix designs. PPF is known to increase tensile strength of concrete members and is therefore widely used to make up for concrete’s weakness to tensile stresses (Wang and Ju 2019). They are also used to reduce overall cracking in a concrete member and reduce overall crack width and area (Banthia and Gupta 2006). In shotcrete, Polypropylene fibers have a role comparable to its role in conventional concrete. It adds tensile strength and flexibility to the member in question.

2.6 Shotcrete Retrofitting

Using shotcrete to repair specific structural members has been in use for some time, however only recently has research really kicked off, leaving experience the main resource in its application. It has been, and still is, used to repair bridge piers, parking garages, dams, sewers, seismic damage, and walls.

For bridge repair, shotcrete has been used effectively for both the superstructure and substructure. It has been noted as an acceptable repair method for patching holes and cracks, and is also capable in repairing the underside of a bridge joint that has degraded. (Wenzlick 2007).

Figure 2.10 – Br. A2175, shotcrete patches around drains now 8 years old (Wenzlick 2007)

Figure 2.11 – Repairs on underside joint of roof of a tunnel bridge (Wenzlick 2007)

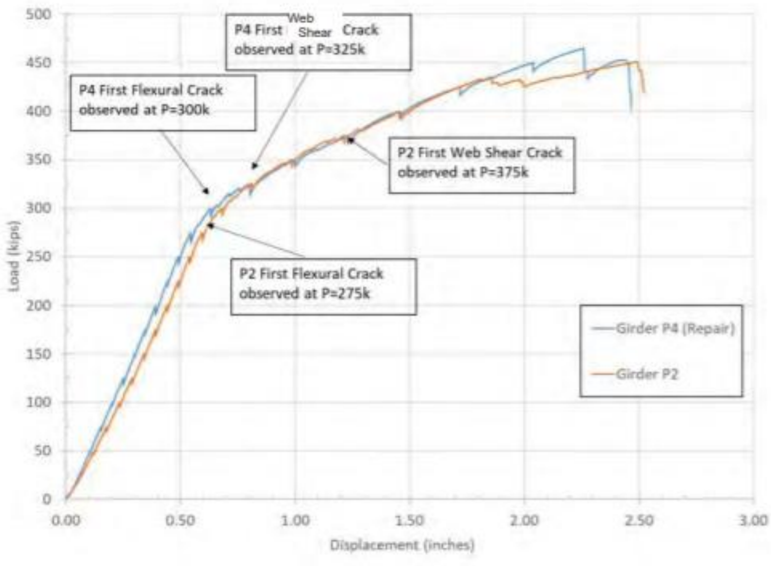
Work has also been done on pier repair where it has been used to repair badly deteriorated pier columns to great effect, such as in repairing the Noblestown Road Bridge in Noblestown Pennsylvania.

(a) (b)

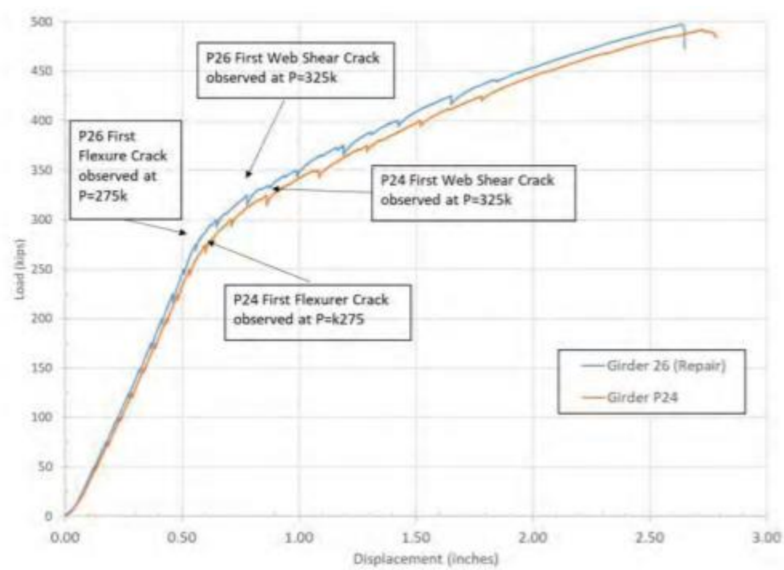
Figure 2.12 – (a) Bridge support ready for repair material; (b) shotcrete being applied to Noblestown Road Bridge (Page 2011)

Cases like these are used are primary reasoning for its use, making the importance of experience in shotcrete the make or break in its application. In order to advance the practice, experimental studies are taken, however as of now, there are very few sources on the performance of shotcrete in repair.

Recently, using shotcrete to repair bridge girders has been looked at and practiced using real world bridge girders. The Minnesota Department of Transportation did a study on the repair of damaged prestressed bridge girder ends using shotcrete and compared the shear capacity of the repaired girder to the shear capacity of the unrepaired girder. The report states that the shotcrete section kept its bond, and in no way, separated from the substrate during testing. The repaired girders had a slightly higher ultimate load, meaning that for shear, shotcrete has the capabilities of strengthening girders to the point of its originally intended load capacity and beyond.



(a)



(b)

Figure 2.13 – Applied load "P" versus displacement graphs (Shield and Bergson 2018)

For flexure, shotcrete has been seen to improve the flexural capacity for small scale beams at the experimental level, however there are very little sources to cite. A study in 1997 by Regina Helena F. Souza and Julio Appleton concludes that using shotcrete can improve the flexural capacity of small scale beams, stating that the beams all failed with higher deformations at midspan. Much more experimental work such as these studies would greatly improve shotcrete's capabilities, and most likely lead to simplification of certain application headaches.

Experimental Program

All experimental data was collected through tests in the Rutgers University Civil Engineering Laboratory under a controlled environment. Before shotcrete was applied and tested, the mechanical properties of the class A layer and SCC were examined and tested for compressive strength, tensile strength, modulus of elasticity, free shrinkage, surface resistivity, and rapid chloride permeability. Fresh properties for the class A layer and the SCC laminate include the slump and air content. For the shotcrete laminate, the fresh properties were tested in slump. The class A layer was poured into the wooden molds and wet cured for 14 days by burlap. After the beams reached their 28 day strength, the shotcrete and SCC mixes were poured into separate beams over 2 layers of steel wire mesh. After the SCC and shotcrete layers were applied, the beams were tested in third point bending for flexure and compared. Full class A beams with no shotcrete or SCC layer along with Full shotcrete beams, one for each mix, were also tested in third point bending as the control group.

3.1 Material Properties

The materials used for the class A mix, SCC mix, and shotcrete mixes are described in Table 1. All materials comply with ASTM standards.

Table 3.1 – Material properties per mix

Material	Class A (Supplier)	Shotcrete (Supplier)	SCC (Supplier)	Standard
Cement	Portland Type I (Clayton Concrete)	Portland Type I (Clayton Concrete)	Portland Type I (Clayton Concrete)	ASTM C150
Fine Aggregate	Concrete Sand (Clayton Concrete)	Concrete Sand (Clayton Concrete)	Concrete Sand (Clayton Concrete)	ASTM C33
Coarse Aggregate	#57 (3/4) (Clayton Concrete)	#8 (3/8th) (Clayton Concrete)	#8 (3/8th) (Clayton Concrete)	ASTM C33
Silica Fume	N/A	Densified (Norchem)	N/A	ASTM C1240
Slag	N/A	N/A	Grade 100 (LaFarge)	ASTM C989
Air Entraining Admixture (AEA)	MasterAir VR 10 (BASF)	N/A	AE92S (Euclid Chemical)	ASTM C260
High-Range Water Reducing Admixture (HRWR)	MasterGlenium 7620 (BASF)	N/A	Plastol 5000 (Euclid Chemical)	ASTM C494 Type F
Fibers	N/A	Macro-Synthetic Fibers (Euclid Chemical)	Macro-Synthetic Fibers (Euclid Chemical)	ASTM C1116, ACI 506.1R-08

Fine and Coarse Aggregate are mimic the practices of ACI 506r-16 which states grading limitations for the combined aggregate (coarse and fine).

Table 3.2 – Grading limitations for combined aggregates (ACI 506r-16)

Sieve size, U.S. standard square mesh	Percent by weight passing individual sieves	
	Grading No. 1	Grading No. 2
3/4 in. (19 mm)	—	—
1/2 in. (12 mm)	—	100
3/8 in. (10 mm)	100	90 to 100
No. 4 (4.75 mm)	95 to 100	70 to 85
No. 8 (2.4 mm)	80 to 98	50 to 70
No. 16 (1.2 mm)	50 to 85	35 to 55
No. 30 (600 µm)	25 to 60	20 to 35
No. 50 (300 µm)	10 to 30	8 to 20
No. 100 (150 µm)	2 to 10	2 to 10

3.2 Mix Designs

For the shotcrete mix design, the mix design was based off of ACI 506r-08, Guide to Fiber-Reinforced Shotcrete. The mix design included macro-synthetic fibers made of polypropylene. Table 3.3 describes the proportions for the shotcrete.

Table 3.3 – Fiber reinforced shotcrete mix design

Fiber Reinforced Shotcrete Mix	
Constituents	Quantity (lb/yd3)
Water	300
Cement	605
Silica Fume	70
Sand	2100
Coarse Aggregate (3/8)	800
Air entraining admixtures	1 (fl. oz/cwt)
Superplasticizer	1 (fl. oz/cwt)
Fibers (1.5”)	5

The SCC mix was proportioned based on previous research conducted by Chris Sholy on the performance of SCC as a reinforcement laminate for concrete beams. This was to be able to compare results and is presented in table 3.4. The Class A mix design is an approved NJTA design and is summarized in table 3.5.

Table 3.4 – Fiber reinforced SCC mix design.

Fiber Reinforced SCC Mix	
Constituents	Quantity (lb/yd3)
Water	287
Cement	439
Slag	236
Sand	1436
Coarse Aggregate (3/8)	1436
Air entraining admixtures	2 (fl. oz/cwt)
Superplasticizer	10 (fl. oz/cwt)
Fibers (1.5”)	5

Table 3.5 – Class A mix design.

Class A Mix	
Constituents	Quantity (lb/yd3)
Water	260
Cement	658
Sand	1205
Coarse Aggregate (3/4)	1800
Air Entraining Admixture	1 (fl. oz/cwt)
Superplasticizer	4 (fl. oz/cwt)

3.3 Procedure

Each mix was conducted in a controlled in-lab environment using a rotating drum mixer. All materials prior to mixing were batched in 5 gallon buckets and measured to the 0.01 pound on a scale. All liquid admixtures used were batched in graduated cylinders and all fibers were batched using a scale measuring to the 0.1 gram. AEA was added and mixed properly to the water used for mixing to ensure proper air entrainment. All dry aggregate was added to the drum mixer before all other components and mixed with approximately a third of the total water content thoroughly. The rest of the water is then added and mixed thoroughly. After all water and dry aggregate are mixed, HRWR is added making sure to make and even spread in the mix. The HRWR is left for 3 minutes to react with the cementitious material and then mixed thoroughly. The mixer is typically left in a horizontal position while mixing to ensure it is mixed properly and angled upwards when adding components. All mixing procedures were in accordance to ASTM standards.

3.3.1 Fresh Concrete Properties Testing

Class A

The Class A mix fresh properties tested were the slump and air content tests. The SCC mix fresh properties tested were the J-Ring, Visual Stability Index (VSI), Air Content, and T50 tests. The shotcrete mix included no fresh properties testing due to being a dry-mix shotcrete, and thus not being able to be tested at that stage.

SCC

For SCC, the T50, VSI, and J-Ring are all done directly after the slump test, and are conducted in accordance to ASTM C1611. This highlights the importance of slump in that it tests the workability of the SCC mix, which is the main feature of SCC. It is specifically described to be tested with an inverted slump cone in contrast with the normal slump test.

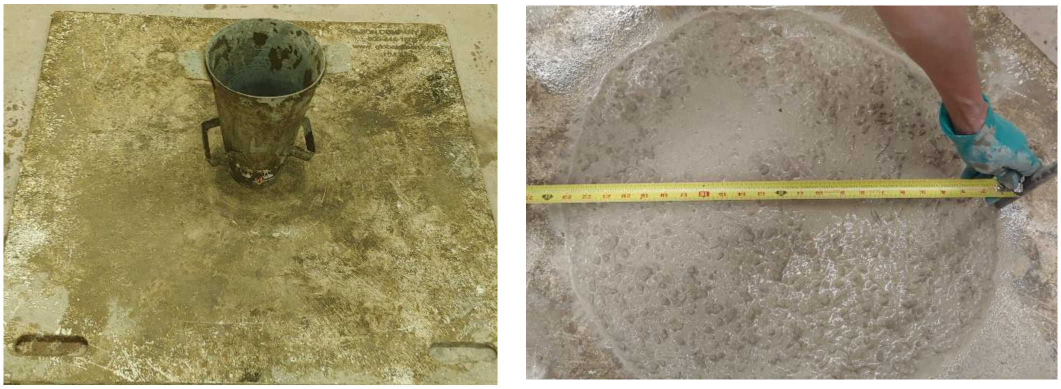


Figure 3.1 – Inverted cone test setup and result

The procedure included filling the inverted slump cone without tampering or vibrating before removing excess material on the top using a sawing motion and lifting the cone slowly allowing the SCC mix to flow freely. The slump is measured as the average diameter between the horizontal and vertical measurements. If the horizontal measurements are more than 2” different, the test was discarded and redone.

The J-Ring test was done for the SCC mix as described in ASTM C1621. The procedure includes similar directions to the procedures used for slump with the only difference being that you place the cone in the center of a J-Ring in order to determine the SCC’s ability to flow through tight reinforcement.

Figure 3.2 – J-Ring testing for SCC

The Air Content test was done using a Type-B as described by the procedure of ASTM C231. The bowl was filled in 1/3rd increments and rodded using a tamping rod 25 times and subsequently hit with a wooden mallet 10-15 times. The lid is then clamped tightly in place and water was pumped into the petcock with a squirt bottle until all the air voids were filled as described by the meter. The valves were then closed and the meter is pumped to the initial pressure before being released. The value shown after was recorded and taken as the percent air content.

Figure 3.3 – Air content type A meter

Shotcrete

For shotcrete fresh properties testing, only the slump was measured and recorded. The slump was taken from the material at the end of the hose without the nozzle to make sure that the correct properties were produced. If taken before the shotcrete was run through the line, the shotcrete properties would have a different air content and produce different results.

Figure 3.4 – Shotcrete run through the line

3.3.2 Curing

Mixing for the SCC and Class A mix included enough concrete for 40-4”x8” cylinders and 3-3”x3”x11” free shrinkage molds and 10% excess concrete for contingency. For the SCC mix, the concrete was not vibrated or tamped while the Class A concrete included addition tamping and vibration. After the concrete cylinders were cast, they were placed into a controlled environmental chamber that is consistently maintained at 50% ± 2% relative humidity and 24℃. The cylinders were demolded 24 hours after casting time and placed into a wet curing chamber for 7 days while 5 extra cylinders are kept in the wet curing chamber until rapid chloride permeability and surface resistivity tests are conducted at 28 days. After wet curing, all samples save for the 5 mentioned previously were placed into the environmental chamber for the remainder of the curing period.

Figure 3.5 – Environmental chamber

Shotcrete curing was conducted based on two different shotcrete types, divided up by shotcrete pumped through the line without a nozzle (PS) and sprayed shotcrete (S). Both categories included 36-4”x8” cylinders, 2-3”x3”x11” free shrinkage molds and 2-3”x3”x11” flexure molds. All 4”x8” and 6”x12” cylinders were cured the same way as the SCC and Class A concrete, however the free shrinkage molds differ. 2 free shrinkage molds were tested with 7 day wet curing and 2 were kept at 14 day curing.

3.3.3 Hardened Properties Testing

As defined by ASTM procedures, the compressive strength, splitting tensile strength, modulus of elasticity, free shrinkage, and rapid chloride permeability (RCPT) were tested and recorded for the Class A, FR-SCC, and FRS mixes.

3.3.3.1 Compressive Strength

Compressive strength was done as specified by ASTM C39. The compressive strength was recorded at 1, 3, 7, 14, 28, and 56 days using 4”x8” cylinders by being loaded axially until failure using a one-million pound Forney Compression Testing Machine. Failure was defined as the maximum load the specimen was capable of holding, or when the recorded reactive load dropped significantly, and the measured strength is defined as the load divided by the contact area. Two cylinders were tested minimum at each day, only testing an additional cylinder should complications arise due to inconsistent capacity results. For shotcrete, 6”x12” cylinders and 3”x6” cores were also used and tested the same way as the 4”x8” cylinders. In accordance with ASTM C617, all cylinders were capped using a sulfur compound to smoothen the contact surface with the machine to distribute the load evenly across the specimen.

Figure 3.6 – Compression test setup

3.3.3.2 Splitting Tensile Strength

Splitting tensile strength was tested using the same machine as the compressive strength tests, the Forney Compression Testing Machine and follows the procedure described by ASTM C496. It is also done for ages of 1, 3, 7, 14, 28, and 56 using 4”x8” cylinders. The cylinders are placed to receive load on its side on top of a flat rigid steel plate. Similar to the compressive strength testing for shotcrete, 6”x12” cylinders and 3”x6” cores were also used for tensile strength, and were testing the same way as the 4”x8” cylinders.

Figure 3.7 – Splitting tension test setup

3.3.3.3 Modulus of Elasticity

Modulus of elasticity was tested in accordance with ASTM C469. Similar to compressive and splitting tensile strength, the modulus of elasticity was tested at concrete ages of 1, 3, 7, 14, 28, and 56 days. A minimum of 2 cylinders were tested and were capped using the sulfur compound. Elastic modulus is always done after compressive testing due to the procedure requiring the modulus to be preloaded to 40% of the age's compressive strength. After capped, the cylinder is placed into a steel cage assembly capable of measuring its displacement at certain load intervals with a length comparator. The cylinder is then loaded to 40% capacity before the length between the front and back screws of the cage is measured and recorded. The cylinders is then loaded again with the length comparator reading recorded at the noted load intervals. The load intervals are dependent on the compressive strength of the specimen and evenly describe the displacement of the cage. This is repeated for each cylinder tested to include a set of results.

Figure 3.8 – Modulus of elasticity setup

3.3.3.4 Free Shrinkage

Free shrinkage testing is done as described by ASTM C157. Free shrinkage samples were measured using 3”x3”x11” molds that were cast along with the cylinders. The molds include studs that were attached to the interior of the molds at both ends so that when the molds was removed, the studs can be used to measure the change in length of the member using a length comparator. After curing for 24 hours, the members were measured and recorded to provide an initial length. They are then placed into the curing chamber for the remainder of their curing period were they are not to be measured. This is because during the wet curing stage, the amount of shrinkage is minimal as drying shrinkage is considered to be exceptional small during this stage. After the curing period, drying shrinkage is a major issue, therefore the member is measure every day for the first 14 days after the curing period. During this time, the specimens are placed in the environmental chamber. After 14 days, the samples are tested twice a week, or every 3 days, as the rate of shrinkage becomes much lower.

Figure 3.9 – Free shrinkage micrometer

3.3.3.5 Rapid Chloride Permeability

Rapid chloride permeability testing (RCPT) was conducted at 28 and 56 days in accordance with ASTM C1202. The goal of the test is to showcase the permeability of the concrete sample by measuring the amount of chloride ions that penetrate the concrete. Cylinders are prepared for RCPT testing by being wet cured in the curing chamber until the day before testing when 1.875 inch thick pieces of the 4"x8" cylinder are cut and placed into a vacuumed filled with distilled water. The pieces used were made sure to have smooth surfaces with little to no holes on the surface. The vacuum pump was turned on for 6 hours then turned off leaving the samples in the distilled water for approximately 20 hours. The samples were then taken out of the vacuum and placed into cells, making sure to tighten the screws holding the cells together the ensure there are no leaks. The cells are then filled with sodium hydroxide on one side and sodium chloride on the other and attached to the ProoveIt RCPT testing machine. Immediately the ProoveIt software was turned on and began recording the output over a 6 hour period. After the 6 hour period, the cells are detached to the sample and cleaned.

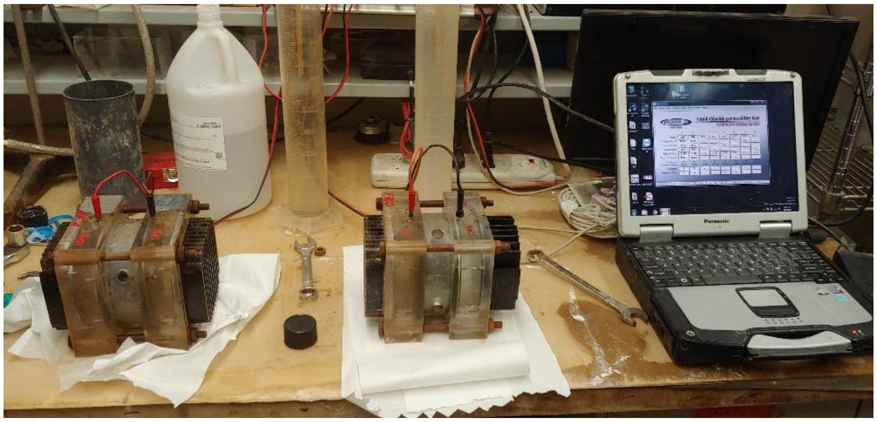


Figure 3.10 – RCPT setup

3.4 Beam Mold Preparation

Small-scale beams were created using wooden molds to test the effectiveness of the varying laminates. The beams were cast to recreate deformed concrete beams by leaving exposed rebar and a roughened substrate surface for the laminate to be applied to. The layout for the stud preparation and beam formwork is shown below.

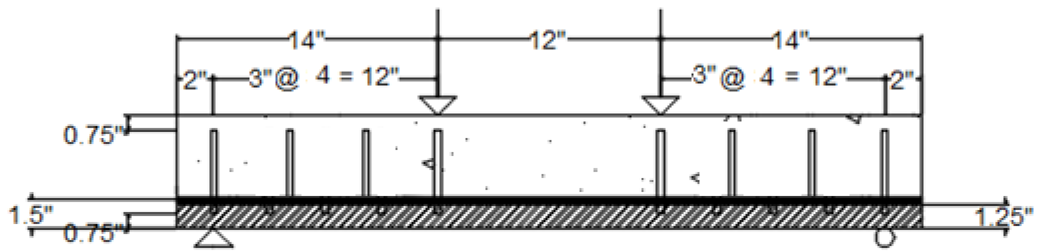


Figure 3.11 – Beam dimensions

The beams were designed to only fail in flexure, as the research focuses on the flexural performance of shotcrete. Wood and steel rebar was purchased and cut to size. Rebar were prepared for strain gage installation at the midspan of the beam. Molds were sealed using wood filler.

All pieces of wood were purchased to be approximately 1 inch thick. The base piece for the mold was cut to be 10” by 40”. The side pieces were cut to be 6” by 40”. The end pieces were cut to be 10” by 6”. The support blocks were cut to be 3” by 6”. The support pieces were drilled into the side pieces from the side and drilled from the bottom into the base to reduce the amount of warping the wood may undergo during casting. End pieces were drilled into the side pieces. Any areas that included a gap were filled using the wood filler. Rebar locations were drilled into the end pieces along with holes necessary to suspend the stirrups in place.

Figure 3.12 – Beam mold arrangement, before and after substrate curing

3.4.1 Stirrups and Reinforcement

All reinforcing steel used were #3. Rebar was purchased at 20’ increments and cut to their specific length. Each flexural reinforcement steel member was cut to be approximately 44” to provide room for grip at the end of the beam. The stirrups were cut to be 4”x4” with a double leg stirrup shape. In order to suspend the stirrups in place for casting to assure an even cover on all sides, 16-gage steel wire was used and drilled into the molds for both lateral and longitudinal support along the beam. The stirrups were tied to the 16 gage steel wires at the corner of the stirrups to prevent it from sliding along the beam as it was being cast. For Full shotcrete beams, the stirrups were also reinforced on the underside of the stirrups to resist the force of the shotcrete spray, and prevent it from being pushed to the bottom of the mold.

3.4.2 Substrate and Mesh Preparation

For beams with both a substrate and a laminate, class A concrete was cast and cured for 28 days. When cast, the beams were vibrated using a mechanical vibrator to consolidate the class A concrete, then after an hour of open air curing, the beams were covered with wet burlap and plastic wrap for each day to be wet cured for 14 days. For each beam, 2 layers of mesh was provided to provide the laminate with more tensile capacity. The mesh was applied after the substrate casting as to not have a buildup of the substrate mix on the mesh, requiring it to be cleaned. Both basalt mesh and 16 gage galvanized steel mesh were used. The 2 layers of steel mesh were tied together using steel ties before application to prevent them from producing smaller, uneven openings caused by movement. The 2 layers were then tied using steel ties to the stirrups to prevent the mesh layer from sliding and moving from the center of the laminate layer. The basalt mesh layers were combined using electrical tape due to the steel ties being too large and damaging to the basalt mesh weave. Steel ties were only used to attach the layers to the stirrups.

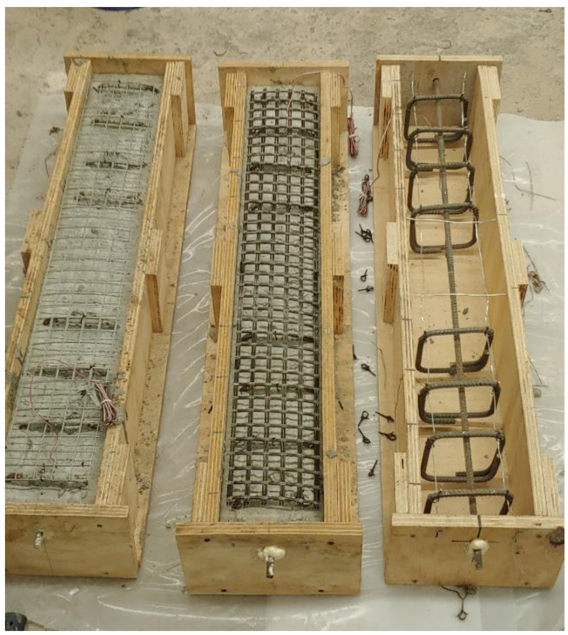


Figure 3.13 – Mesh preparation before casting

Due to the limitations of the concrete round drum mixer in use, the beam substrates had to be cast using different mixes with the same mix design, making sure that each mix met the requirements of the Class A classification as well as being as similar to each other as possible. It was split up into 4 different mixes for the beam substrates, each being 4 cubic yards.

In order to increase the bond between the substrate and the laminate, the surface of contact was roughened. This also doubled as a way for the stirrups to have an increase in gap underneath it to provide ample room for the shotcrete to flow and encapsulate. ACI recommends the room for the shotcrete to flow appropriately be at least three times the size of the largest aggregate size.

3.5 Shotcrete Laminate Casting

All beams not specified as a control beam included a mesh and laminate layer. Each of the beams' class A substrate was cured properly before applying the specified laminate. All meshes used were comprised of only 2 layers of the specific mesh material, whether galvanized steel or basalt. The size of the openings for both types of meshes were 1 square inch. Beams to be shotcrete were propped up at an angle. The nozzleman shot the beams perpendicular to the beam to reduce rebound and improve encapsulation of the exposed steel.

Figure 3.14 – Shotcrete laminate casting

Although the testing methods between wet mix shotcrete and conventional concrete are similar, there are some major differences with how it is set up. Typically, shotcrete is tested by coring samples from a slab that has been shot by the nozzleman using the same mix being used on site. There have been recent studies that state that there is little to no difference between the process of shooting directly into cylinders as there is to coring. This is examined by shooting normal 4”x8” cylinders and 6”x12” cylinders with the shotcrete and comparing to a shot slab with the same mix. These are compared to other 4”x8” cylinders and 6”x12” cylinders that were filled with the shotcrete mix without being shot.

Figure 3.15 – Shotcrete casting for hardened properties

The nozzleman was ACI certified for vertical wet mix shooting, using the equipment necessary to implement the project. This included a wet-mix nozzle, 350 CFM air compressor at 100 PSI, and wet mix shotcrete pump. The shotcrete mix was delivered using a transit vehicle and delivered to the site of casting. The transit vehicle poured directly into the wet-mix shotcrete pump and was run through the hose line in order to fill it with material before shooting as a method of cleaning out the line and make sure that there is an appropriate amount of pressure at the outlet to match the inlet pressure.

(a)

(b)

(c)

(d)

Figure 3.16 – Shotcrete equipment used for casting: (a) air compressor, (b) wet-mix shotcrete pump, (c) wet-mix shotcrete nozzle, (d) transit mixer

3.6 Shotcrete Laminate Curing

The use of wet burlap curing, curing compound, and dry curing was examined and tested for each beam layer type. For steel mesh FRS, basalt mesh FRS, and no mesh FRS, 1 beam for the curing periods of 3 and 7 days were cured using each curing method. This is in contrast to the class A substrate, which was cured using wet burlap only.

Wet burlap curing included the use of wetted burlap and placing it on top of the laminate approximately 3 hours after shooting for the curing period described. The burlap was rewetted every day during the curing period until testing. Curing compound was sprayed on top of the selected beam laminates approximately 3 hours after shooting as described by the curing compound manufacturer. The curing compound was left and not reapplied at any given time. Dry cured beam laminates had no methodology applied to them after being cast. Each of the beams were placed under a plastic wrap, divided by each mesh layer. All basalt laminates were placed under the same plastic wrap, all the steel layers were placed under the same plastic wrap, etc. All beams were checked daily during the process to make sure that they were curing appropriately.

3.7 FR-SCC Laminate Casting and Curing

The FR-SCC mix was batched and prepare the day before casting to give time for the aggregate water content to be calculated. The FR-SCC layer was cast directly onto the substrate once the substrate was properly cured. The exposed portion of the class A substrate was cleaned using compressed air and the excess of ties that were exposed were removed. Cured concrete on the stirrups were hammered off to ensure the bond strength of the laminate to the stirrup. All SCC laminate beams were cured using wet burlap which was not applied until approximately 3 hours after the laminate was cast, or when the laminate had hardened. The wet burlap was checked and rewetted every day until the curing period of 7 days was over.

3.8 Beam Testing

All beams were tested using the third point bending test to remove any excess shear values that come with three point bending and guarantee only flexural testing. A Test Resources 600 series Static Hydraulic Universal Test Machine was used to load all beams. Each beam was loaded at a rate of 900 pounds per minute as described by ASTM C78 testing parameters for the specified beam dimensions. 2 rollers were placed 2” from the edge of the each side of the beam and two point loads were 12” from the supports leaving a 12” gap from one point load to the other.

Figure 3.17 – Beam testing setup

The beam prior to loading was painted and gridded using pencil and diluted white paint. The white paint was only done at the bottom of the beam on the side that was crack mapped. The white paint aids whoever is crack mapping during the loading procedure to see cracks more easily due to the added contrast. The paint is diluted using water as to not put too thick of a coat on the beam, which could ultimately hide cracks from view. The grid was 2” longitudinally and transversely across the face being mapped. This helps identify the location of the cracks that propagate and enable a more accurate placement of the beam with regards to the loading setup.

The setup on the machine uses a red I-beam to stabilized and lift the beam up, as the beam is 40” long but the machine does not innately support the length. 2 steel plates under 2 roller supports are placed on top of the I-beam. On top of the beam are 2 roller plates which act as loading points for third point bending. The load is evenly dispersed using a steel form which is under a load cell. The load from the testing machine is directly applied to the load cell.

3.9 Loading and Data Collection

The load cell placed at the top of the apparatus and under the machine’s loading plate calculates the load applied from the machine. The strain gage applied to the rebar at the midspan along with two linear variable differential transformers (LVDT) and the loading cell are all attached to the CR3000 data collecting system.

The system collected the displacement of the beam at midspan from the LVDTs, the applied load from the load cell, and the strain of the reinforcing steel until failure. Failure was defined as the point the rebar breaks, or enters plastic deformation. The data is extracted from the data logger and placed into an excel file for developing the necessary Figures and tables.

3.10 Crack Mapping

Crack mapping is the process at which one records the cracking behavior of the beam during testing until failure, and begins when the first crack appears. The crack is then marked with a sharpie and a picture is taken using the software DinoCapture and a microscope camera. The load at which the crack is first seen is also recorded. This is repeated for when new cracks propagate. At certain intervals, pictures are retaken so that the increase in the crack’s propagation can be measured as the load increases, thus measuring crack width growth. Using the

DinoCapture software, the crack width is measured and recorded using the photoFigures taken during the loading period.

Figure 3.18 – Beam crack mapping

Results

As part of the experimental program, trial mixes were used to establish the mechanical properties of the mix using 4”x8” cylinders, 6”x12” cylinders, 3”x3”x11” free shrinkage and flexure molds, and restrained shrinkage molds. Fresh properties were tested alongside the casting of each mix and are presented with the hardened properties in this chapter.

The beams were prepared firstly by preparing the wooden molds, stirrups, strain gages, and rebar before casting the Class A concrete. All Class A layers or Full beams were consolidated using a vibrator alongside the cylinders for each mix. The class A subjects were cured using wet burlap for 14 days and left to dry cure for another 14 days until it reached 28 day strength. The Full beams include shotcrete, SCC, and class A beams. The flexural and crack performance of the three mixes were compared and are presented in this chapter.

SCC and Shotcrete laminates were casted after the 28 curing period for the Class A concrete. Each shotcrete and SCC mix was fiber reinforced with 1.5” macro synthetic polypropylene fibers of the same content (5 lb/cuy). The SCC and shotcrete mixes included a mesh layer, involving a basalt mesh, steel mesh, and a layer without any mesh. The results on the differences between the different laminates in terms of flexural and cracking properties are presented in this chapter.

The shotcrete beams included three different curing regimes. Of each laminate type, 2 were dry cured, 2 were cured with wet burlap, and 2 were cured using curing compound. The differences between the 3 curing regimes were recorded and presented in this chapter.

Hardened properties of shotcrete were recorded using shotcrete that has been sprayed directly into the cylinders and shotcrete that has only run through the line. This was done for compression, tension, modulus of elasticity, free shrinkage, RCPT, and flexure. These results are presented and compared to the results of cored samples in this chapter.

For all beams tested in this study and presented in this chapter, the differences between the laminate strength at 3 days and 7 days is recorded and presented in this chapter.

4.1 Fresh Concrete Properties

The fresh concrete properties for all mix designs were measured and recorded at the time of mixing, before the mix was cast. The shotcrete must obtain a specific slump to ensure compaction and pumpability, while the SCC must be able to flow through congested reinforcement. In the event the requirements were not met for slump, HRWR was added in order to increase flow and slump, making sure not too add too much, resulting in a soupy mix. All properties reached satisfactory results.

4.2 Hardened Concrete Properties

In order to relate the shotcrete mix, SCC mix, and Class A mix, the hardened properties of each mix were tested using cylinders. Strength testing included compressive, tensile, and elastic modulus. Stability was gaged through free shrinkage testing, and Endurance was tested through RCPT testing. Each test was done in accordance with the ASTM standard for their respective category.

For the shotcrete, cores were not tested at 1 day strength. This is because the breaking age of the slab is noted to be approximately 3 days, giving the slabs enough time to reach a point where the cored samples can come out cleanly and not involve an excess of local failure. The 6”x12” cylinders were excluded from the finalized results due to their unreliable results, and thus were not shown in the figures below.

4.2.1 Compressive Strength

The results for the FR-SCC, FRS, and Class A compressive strengths are presented in figure 4.1. It is important to note that the addition of fiber is reported to decrease the compressive strength of high performance concrete (HPC) mixes is 7% - 14% (Abu-Obeidah et al. 2017). It is also reported that for SCC concrete, fibers reduce the compressive strength of concrete by 10% - 14% (Sholy 2018). Shotcrete is somewhere between the two mix types of HPC and SCC but using normal concrete, therefore the percent decrease is similarly associated with the decrease in compressive strength seen in HPC and SCC.

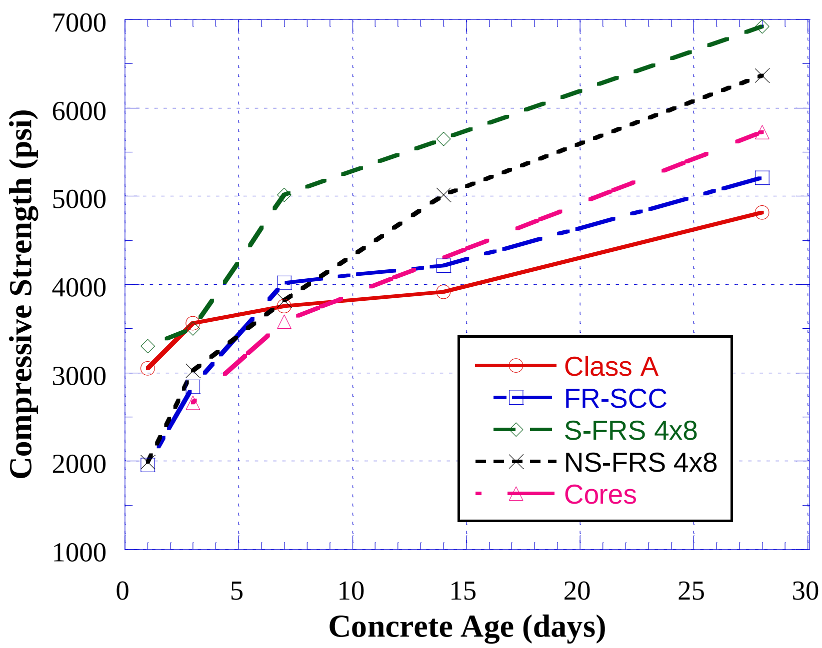


Figure 4.1 – Compressive strength of concrete

When comparing the results between the shotcrete cylinders that were fill by hand and not shot directly, the 4x8 cylinders, the strength for 3, 7, and 28 day results are 0.5%, 3.78%, and 10% higher respectively. The comparison is described in figure 4.2.

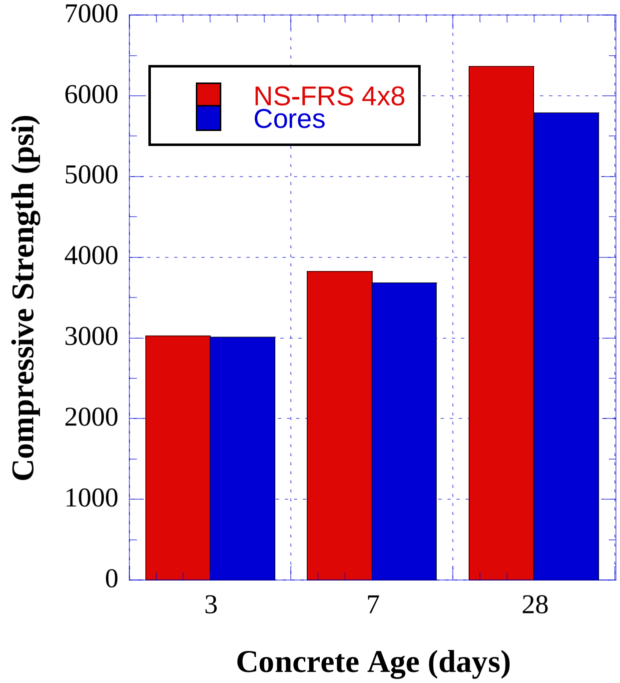


Figure 4.2 – Comparison of hand cast cylinders for compressive strength

The results for the shotcrete cylinders shot directly by the nozzleman during the laminate casting show that the 4x8 cylinders for 3, 7, and 28 day strengths are 16.3%, 36.2% and 19.6% stronger than the cores respectively. The comparison is described in figure 4.3.

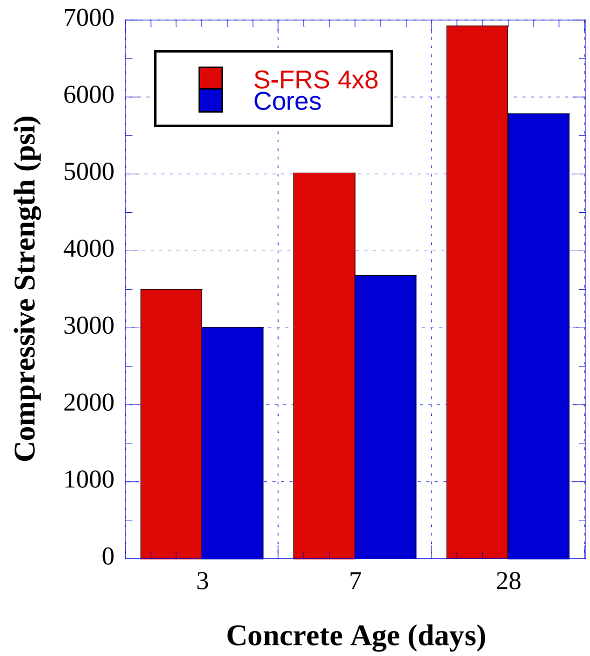


Figure 4.3 – Comparison of shot cylinders for compressive strength

Table 4.1 – Percent differences of FRS from cored samples in compression

	3 Day Difference		7 Day Difference		28 Day Difference	
Type	%	PSI	%	PSI	%	PSI
NS-FRS 4x8	0.48	14.47	3.77	138.90	10.00	578.76
S-FRS 4x8	16.35	491.95	36.20	1332.60	19.63	1135.82

4.2.2 Splitting Tensile Strength

The results for the splitting tensile strength of the materials are presented in figure 4.4. The data shows inconsistencies throughout the ages. Most notably in the NS-FRS 6"x12" cylinders and for Class A cylinders. There is no increase to very little increase in the tensile strength of the 6"x12" cylinders for the cylinders that were cast by hand using shotcrete, and thus is not considered in analysis.

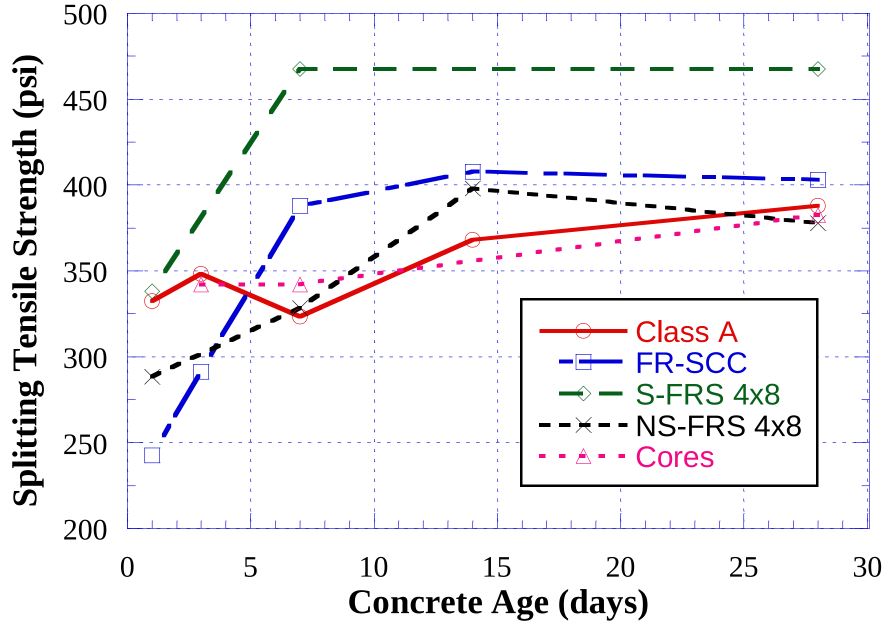


Figure 4.4 – Splitting tensile strength of concrete cylinders

The differences in strength between the cored cylinders and the NS-FRS cylinders are described in figure 4.5. As stated, the NS data, involving the 4"x8" cylinders is comparable. For 3, 7, and 28 day strengths, they are 15.7%, 18.5%, and 14.6% weaker. Although the results are relatively far off, as a whole, the cylinders are consistently approximately 16% weaker.

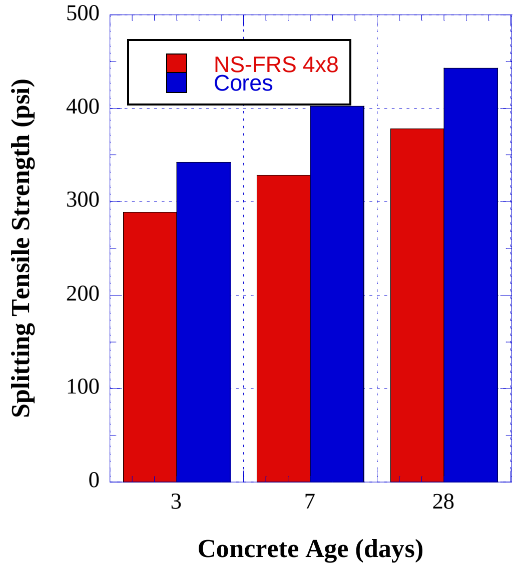


Figure 4.5 – Comparison of cylinders cast by hand for tensile strength

As for the S-FRS data, the S-FRS 4x8 cylinders for 3, 7, and 28 day strengths are 8.7% weaker, 16.1% stronger, and 5.6% stronger respectively. The results are shown in figure 4.6.

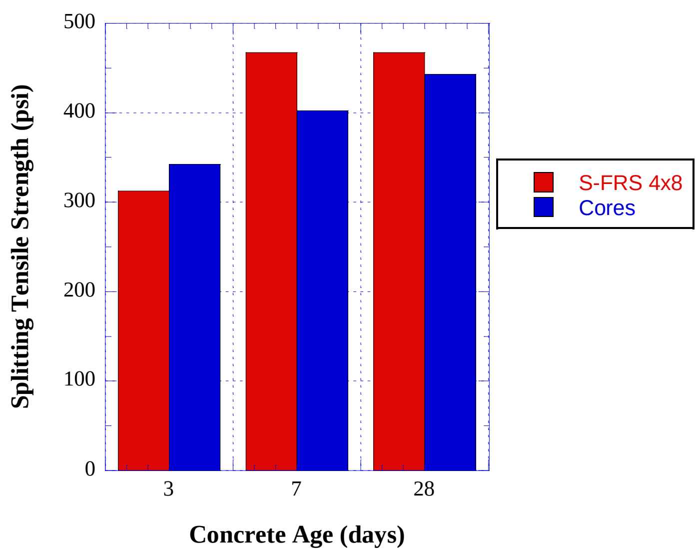


Figure 4.6 – Comparison of cylinders shot for tensile strength

Table 4.2 – Percent differences of FRS from cored samples in tension

	3 Day Difference		7 Day Difference		28 Day Difference	
Type	%	PSI	%	PSI	%	PSI
NS-FRS 4x8	-15.71	-53.75	-18.47	-74.35	-14.65	-64.87
S-FRS 4x8	-8.73	-29.87	16.12	64.91	5.57	24.65

4.2.3 Elastic Modulus

Figure 4.7 shows the elastic modulus results for the 4"x8" cylinders. The elastic modulus is used to help gage the materials stiffness, where the higher the elastic modulus, the more stiff the material.

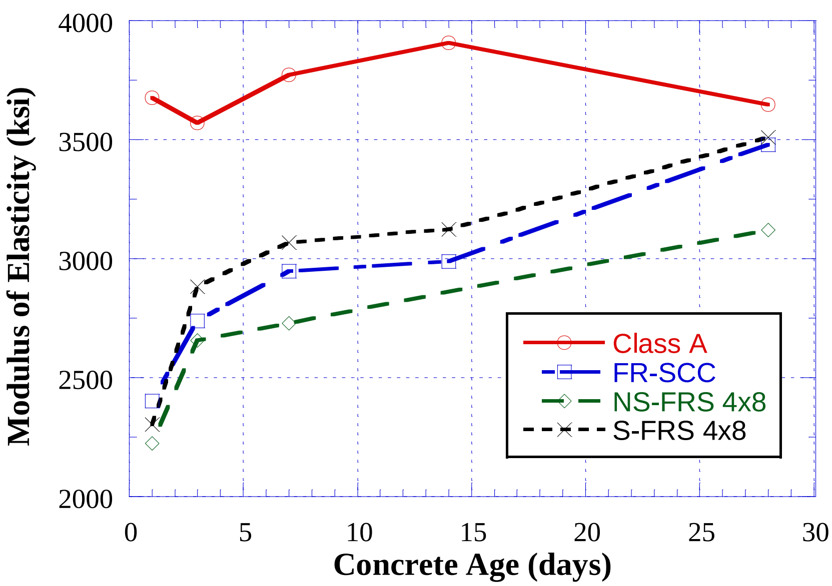


Figure 4.7 – Modulus of elasticity of concrete samples

For all ages, the elastic modulus for the FR-SCC, NS-FRS, and S-FRS samples were lower than that of the Class A. The percent differences are summarized in table 4.3.

Table 4.3 – Percent differences of FRS and FR-SCC from Class A samples for modulus of elasticity

	Concrete Age (Days)	1	3	7	14	28
Modulus of Elasticity (ksi)	FR-SCC	-34.68	-23.28	-21.85	-23.53	-4.63
	NS-FRS 4x8	-39.47	-27.75	-25.79	N/A	-15.13
	S-FRS 4x8	-37.35	-21.6	-16.51	-15.02	-4.52

4.2.4 Free Shrinkage

Figure 4.9 and 4.10 present the results for the 7 and 14 day wet cured shotcrete specimens. The 7 day wet cured free shrinkage samples were done for only the shotcrete mixes in order to see early stage free shrinkage. All sample types, FR-SCC, Class A, S-FRS, and NS-FRS cured for 14 days. Table 4.4 shows all results for the total macrostrain of each specimen type. Figure 4.8 includes the results of all 14 day cured free shrinkage specimens.

Table 4.4 – Microstrain of all free shrinkage specimens

CONCRETE AGE (DAYS)	1	7	14	21	28
CLASS A	0.00	N/A	0.00	107.25	199.98
FR-SCC	0.00	N/A	1.03	147.03	245.05
NS-FRS-7	0.00	1.00	185.67	259.42	289.70
S-FRS 7	0.00	0.98	121.10	150.59	164.28
NS-FRS-14	0.00	1.00	0.37	205.43	231.76
S-FRS-14	0.00	0.98	0.67	116.89	131.63

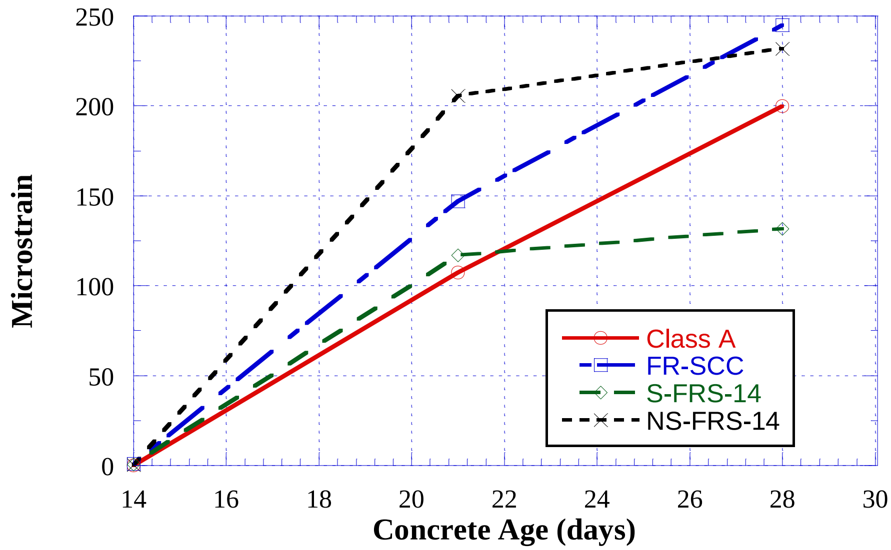


Figure 4.8 – Free shrinkage results of all 14 day cured samples

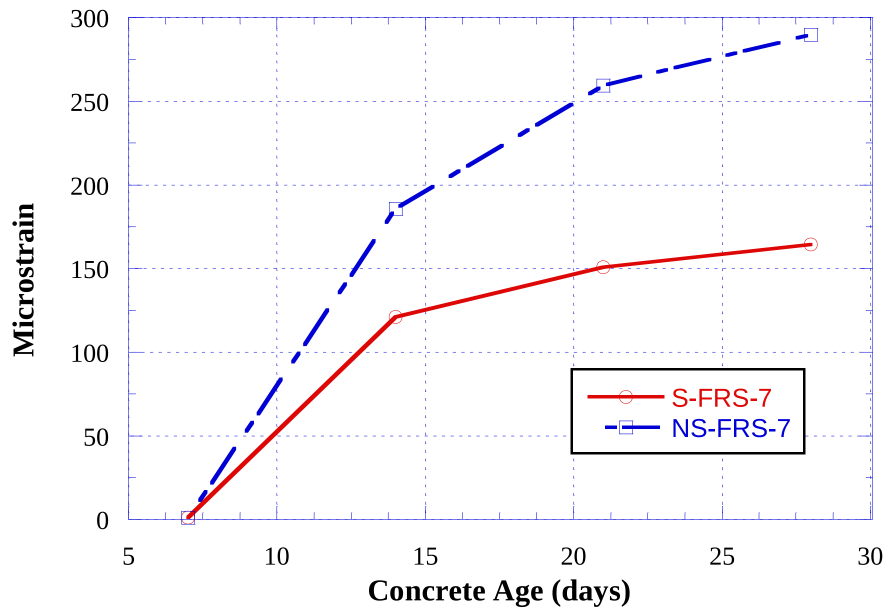


Figure 4.9 – Free shrinkage results of 7 day cured samples

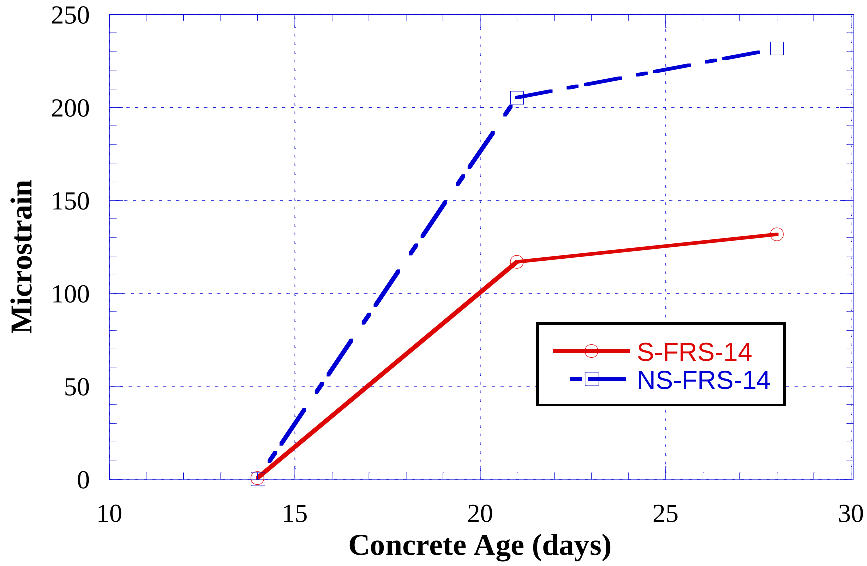


Figure 4.10 – Free shrinkage results of 14 day cured shotcrete samples

For the 7 day wet cured samples, the NS-FRS samples had values 65.22%, 58.05%, and 56.71% higher than the S-FRS results for 14 day, 21 day, and 28 day concrete ages respectively. For the 14 day wet cured samples, the NS-FRS samples had values 56.90% and 56.80% higher values than the S-FRS results for 21 day and 28 day ages respectively.

4.2.5 Rapid Chloride Permeability Test

Figure 4.11 presents the RCPT data. It is important to note that the shotcrete cylinders used for the RCPT results were the 4”x8” cylinders due to the size of the apparatus, only that size could be tested.

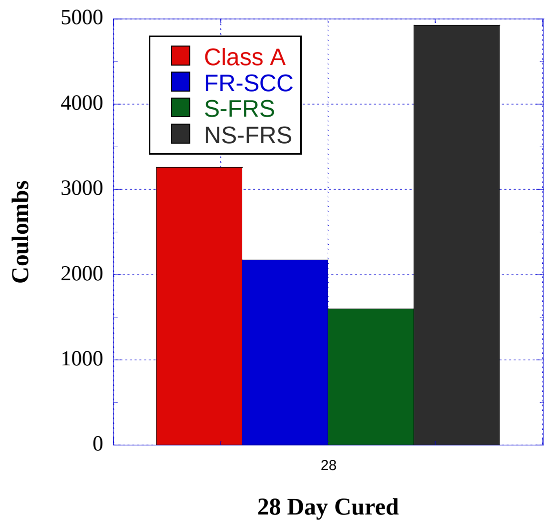


Figure 4.11 – RCPT results at 28 day age

The most important difference in the data is the difference between the two shotcrete mixes. S-FRS is much lower, resulting in a relatively low value comparatively to other materials while the NS-FRS is much higher.

4.2.6 Analysis of Mechanical Properties of Shotcrete

The importance of analyzing the mechanical properties lies in the difference between using the shotcrete cylinders versus using the cores. Coring is time consuming for creating different mix designs while the cylinders are exceptionally faster, especially in a lab setting.

The compressive strength of the NS-FRS 4"x8" most closely represents the strength of the cores from 3 day to 28 day strengths. The percent differences and standard deviations are presented in table 4.5.

Table 4.5 – Average percent differences of FRS compressive strengths from cored samples

	3 Day	7 Day	28 Day	Average (%)	Std. Dev. (%)
NS-FRS 4x8	0.48	3.77	10.00	4.75	3.95
S-FRS 4x8	16.35	36.20	19.63	24.06	8.69

The NS-FRS 4”x8” cylinders have a low average and a low standard deviation, making them the most consistent with regards to the compressive strength.

For tensile strength, table 4.6 describes the percent differences from shotcrete to the cores. The S-FRS 4”x8” cylinder averages most closely simulate the cores, however it has the largest standard deviation, making it unreliable.

Table 4.6 – Average percent differences of FRS tensile strengths from cored samples

	3 Day	7 Day	28 Day	Average (%)	Std. Dev. (%)
NS-FRS 4x8	-15.71	-18.47	-14.65	-16.27	1.61
S-FRS 4x8	-8.73	16.12	5.57	4.32	10.18

If one had to be chosen to test for cores, the chosen method would be with the lowest standard deviation combined with the lowest average. Because the lowest overall average could not be considered due to its high standard deviation, the best would be the S-FRS 6”x12” which has a very low standard deviation, though not the lowest, and the second lowest average.

For the modulus of elasticity, the most noteworthy set of results are the FR-SCC and the S-FRS. The results are strikingly similar on all days. This must come from the fact that both use compaction methods to produce their properties, and thus are highly comparable. Table 4.7 and figure 4.12 showcase the two sets.

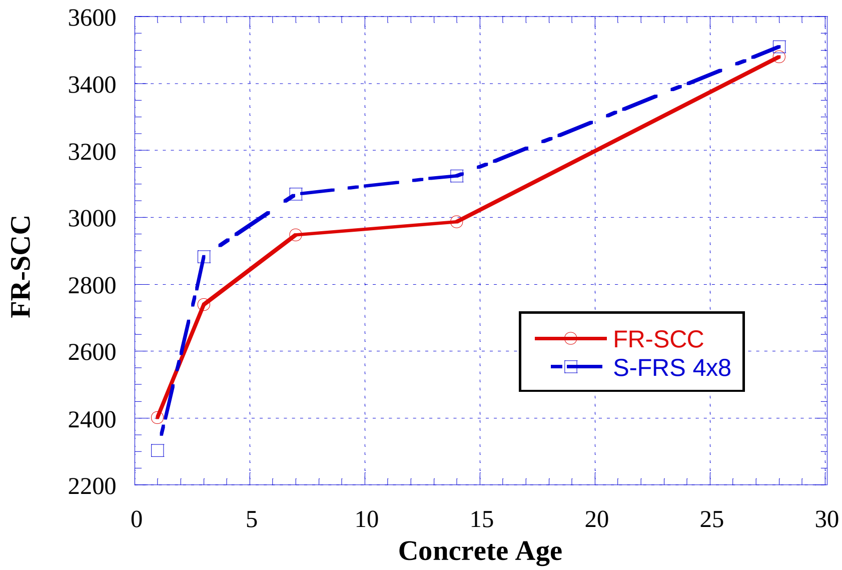


Figure 4.12 – Modulus of elasticity results for FR-SCC and S-FRS 4”x8” cylinders

Table 4.7 – Modulus of elasticity results for FR-SCC and S-FRS 4”x8” cylinders

	Concrete Age (Days)	**1**	**3**	**7**	**14**	**28**
Modulus of Elasticity (ksi)	FR-SCC	2401	2739	2947	2987	3479
	S-FRS 4x8	2303	2882	3069	3124	3510

The modulus of elasticity could not be closely examined with relation to the cores due to the cores having a non-conforming size to the modulus testing rig in the lab. However, if one was to be chosen in order to progress, the S-FRS method would be chosen based on its ability to produce results more closely related to the compaction of the actual shotcrete that would be applied.

There is very little that can be compared with regards to the free shrinkage and shotcrete slabs. As it stands, the slab would have to be cut out in a similar shape to the molds and compared, however this is not deemed to be reliable considering the impact of cutting the slab.

The RCPT data cannot be closely examined with relations to the cores because the cores are not transferrable. That being said, the RCPT shows that the S-FRS more closely produced results comparable to what was expected being that more compacted cylinders would have lower values.

4.3 Beam Testing Results

A total of 25 beams were tested in third-point bending. All beam substrates were made with Class A concrete, laminates were applied using SCC or fiber reinforced shotcrete. Each beam included a laminate depth of 1.5”, starting approximately 1 inch below the exposed stirrup. All reinforcement, both shear and flexural consist of #3 rebar. The load, deflection, and strain data was recorded with each test and presented in this chapter. Failure was defined at the point when deflection greatly increased without any increase in load, implying failure of the flexural reinforcement. This is confirmed at the point at which the flexural reinforcement strain increases rapidly to the point where no data could be recorded, hence the failure in the reinforcement. Table 4.8 presents the orientation and amounts of the beams created and tested.

Table 4.8 – Beam testing regime

	Beam Type	Curing Method	Age of Testing (Days)	Amount
Control	Full Class A	Wet Burlap	Testing Days	2
	Full FR-SCC	Wet Burlap	7	1
	Full FRS	Wet Burlap	3, 7	2
	FR-SCC-Basalt	Wet Burlap	7	1
	FR-SCC-Steel	Wet Burlap	7	1
Retrofitted Beams	FRS-Basalt	Dry Cured	3, 7	2
		Curing Compound	3, 7	2
		Wet Burlap	3, 7	2
	FRS-Steel	Dry Cured	3, 7	2
		Curing Compound	3, 7	2
		Wet Burlap	3, 7	2
	FRS-NoMesh	Dry Cured	3, 7	2
		Curing Compound	3, 7	2
		Wet Burlap	3, 7	2

Beams are loaded in third point flexure. It is important that the beams are load in third point bending, where the load is placed on two different locations on-top of the beam at a third of the beam's length, rather than three point bending, which places one load at midspan. Three point bending induces extra shear stresses but cannot be accurately measured during testing and is recorded as an addition to the flexural stresses. This increases the recorded stress, and due to not being accurately measured, cannot be realistically subtracted from the required flexural stress results, and thus was not used. Third point bending eliminates the excess shear stresses and tests the beam in pure flexure. All beams are design to fail in flexure, having cracks under the two load points and at midspan, plus any flexure cracks outside of the loading area. Figure 4.13, 4.14, and 4.15 present the FRS beams after loading.

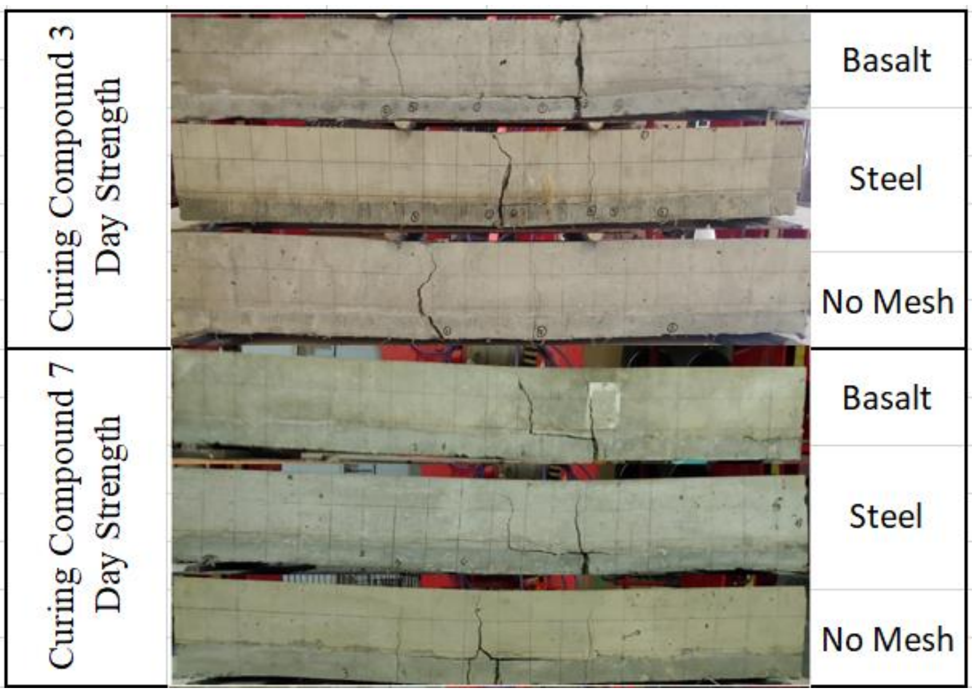


Figure 4.13 – Curing compound FRS beams after loading

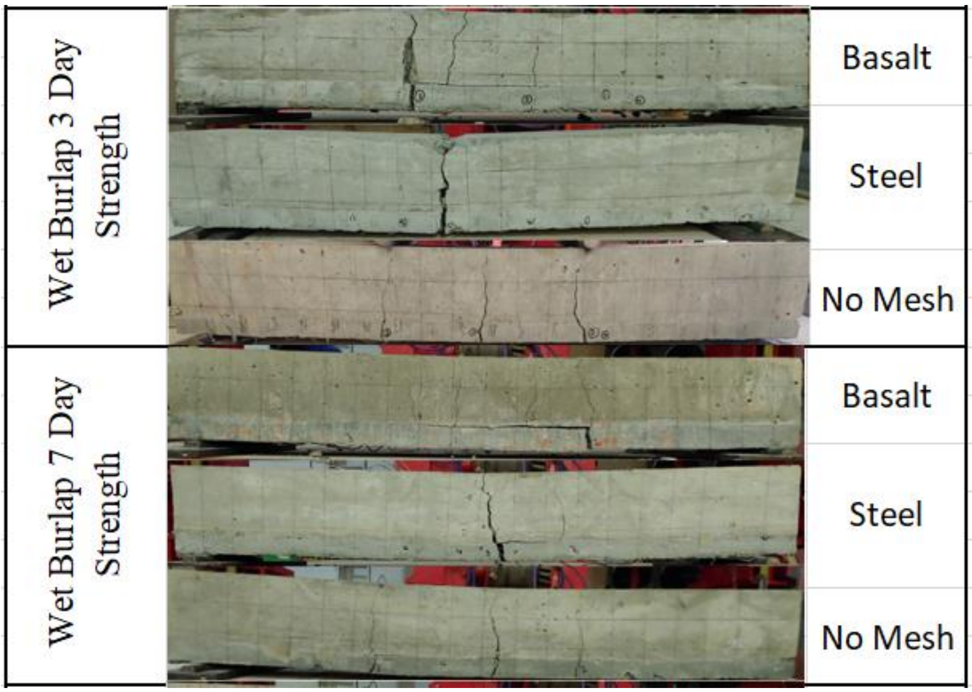


Figure 4.14 – Wet burlap FRS beams after loading

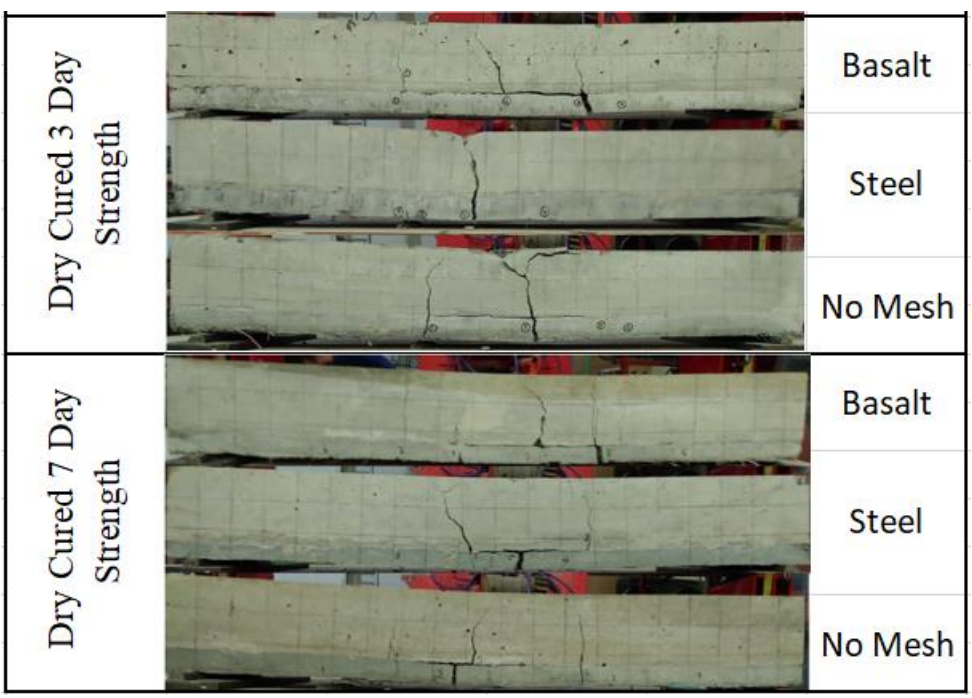


Figure 4.15 – Dry cured FRS beams after loading

4.3.1 Load vs Deflection

For comparing the load deflection results, only the wet burlap data was considered because the control beams were cured using wet burlap, and is considered a standard method. Figure 4.16 presents the 3 day strength data compared to the class A concrete, while figure 4.17 presents the 7 day strength data compared to the class A concrete. Other control beams, such as the SCC beams are not used in 3 day curing comparison due to having all been cured for 7 days, which is why they are only presented in table 4.17.

The location of ultimate load is considered to be at the highest load point shown on the Figure. The ultimate load point is not always considered as the same point of the ultimate deflection. This is the case when a dip in the load is present, and is found at the lowest point of the "drop", which is presented in the Figure as the final point in the data.

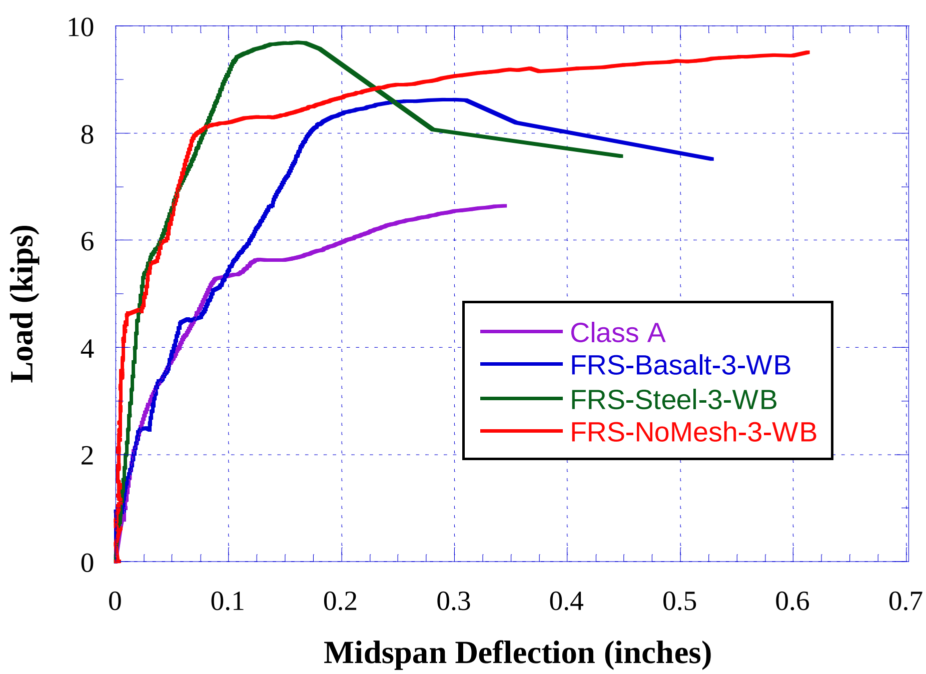


Figure 4.16 – Class A and FRS load vs deflection results at 3 day strength

As shown, Class A concrete had the lowest ultimate load compared to the shotcrete beams. The FRS-Basalt-3, FRS-NoMesh-3, and FRS-Steel-3 were 29.82%, 43.07%, and 45.93% higher ultimate load respectively. As for the difference in ultimate deflection, FRS-Basalt-3 and FRS-NoMesh-3 were 11.86% and 29.87% higher respectively, while the FRS-Steel-3 is 5.09% lower.

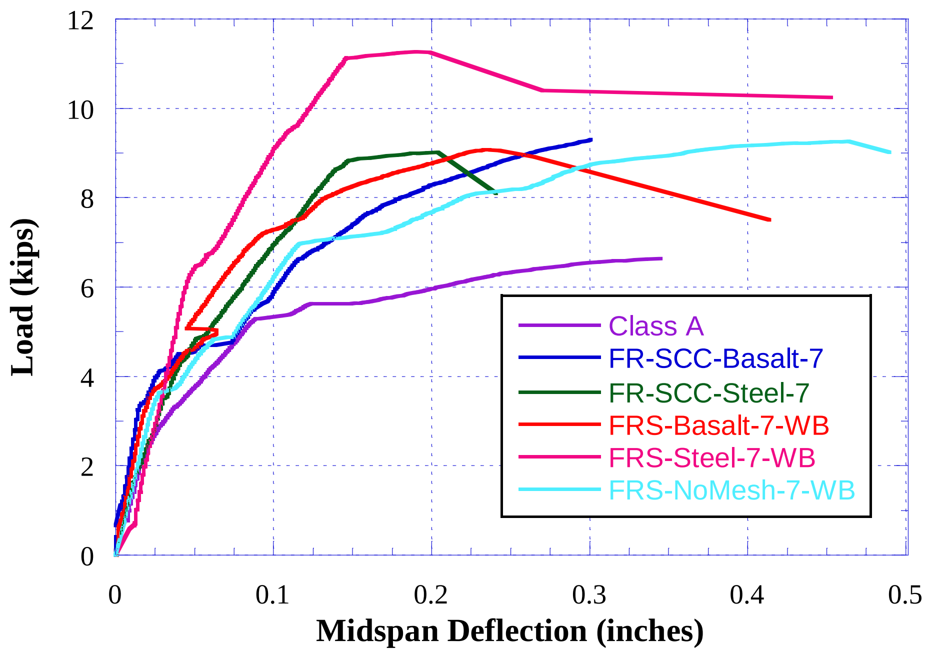


Figure 4.17 – Class A, FRS, and FR-SCC load vs deflection results at 7 day strength

With regards to the 7 day strength data, the class A had the lowest ultimate load, similar to 3 day strength, however now SCC can be considered due to having a 7 day curing time itself. FRS-Steel-7 had a higher ultimate strength, higher deflection, and lower crack width than FR-

SCC-Steel by 25.08%, 88.00%, and 5.48%. As for basalt, FRS-Basalt-7 had a 2.37% lower ultimate strength, 103.32% higher deflection, and the same crack width.

4.3.2 Load vs Deflection Analysis

Table 4.9 presents the percent differences between day 3 and day 7 strengths for the shotcrete beams cured using wet burlap. Table 4.10 presents the percent differences between the 7 day ultimate load values for all shotcrete beams cured using wet burlap and the class A and FR-SCC ultimate load values.

Table 4.9 – Percent differences between 3 and 7 day FRS beam ultimate load and deformation

Ultimate Load			
Mesh	Basalt	No Mesh	Steel
3	8.62	9.50	9.69
7	9.07	9.27	11.27
%(+/-)	5.22	-2.42	16.31
Ultimate Deformation			
3	0.53	0.61	0.45
7	0.61	0.60	0.45
%(+/-)	15.91	-1.63	1.12

The strength and deformation had increased for basalt and steel, while there was a decrease for both for the beams with no mesh. For all the beams, both the deflection and the load capacity should increase with time, however the results for beams without a mesh and beams with a steel mesh, there is little to no differences in the deflection.

The strongest between the three layer types is the steel mesh beam, which produced an ultimate load of 11.27 kips at 7 days. This is consistent with 3 day strength as well, where steel reached a strength of 9.69 kips. Basalt was the weakest. This is noted to be due to the material of the mesh. The basalt mesh is a fabric, rather than a rigid piece similar to the steel, therefore when the mesh was shot by the nozzleman, it would move and become pushed in. This would create an inconsistency in the distribution of aggregate and fibers in the mix as it was being cast due to the mesh pushing it as it was moving, which may result in a decrease in overall strength.

Table 4.10 – Percent differences between 7 day strength FR-SCC and FRS ultimate load and deflection

Ultimate Load		
	Basalt	Steel
FR-SCC-7	9.29	9.01
FRS-7	9.07	11.27
%(+/-)	-2.37	25.08
Ultimate Deflection		
FR-SCC-7	0.30	0.24
FRS-7	0.61	0.45
%(+/-)	103.32	87.97

Compared to the FR-SCC, the FRS is considered to be stronger and more flexible. This contradicts what is seen in the basalt results, however the FRS basalt beams are considered an outlier in terms of strength due to what was described previously with the how the mesh moved when the shotcrete was being cast.

4.3.3 Curing Method

The effects of three different curing regimes were tested and examined by curing each of the FRS beam layer types; basalt, steel and no mesh, using dry curing, wet burlap, and curing compound. Wet burlap curing included the use of wetted burlap and placing it on top of the laminate approximately 3 hours after shooting for the curing period described. The burlap was rewetted every day during the curing period until testing. Curing compound was sprayed on top of the selected beam laminates approximately 3 hours after shooting as described by the curing compound manufacturer. The curing compound was left and not reapplied at any given time.

Figures 4.18 and 4.19 present the effects of the three different curing methods compared to each other for the basalt mesh at both 3 and 7 day strengths respectively. Figures 4.20 and 4.21 present the effects of each curing method for beams without a mesh for 3 and 7 day strengths respectively. Figures 4.22 and 4.23 present the effects of each curing methods for beams with a steel mesh for 3 and 7 day strengths respectively. Class A beams are used to compare the strengths and deflections with regards to the different curing methods.

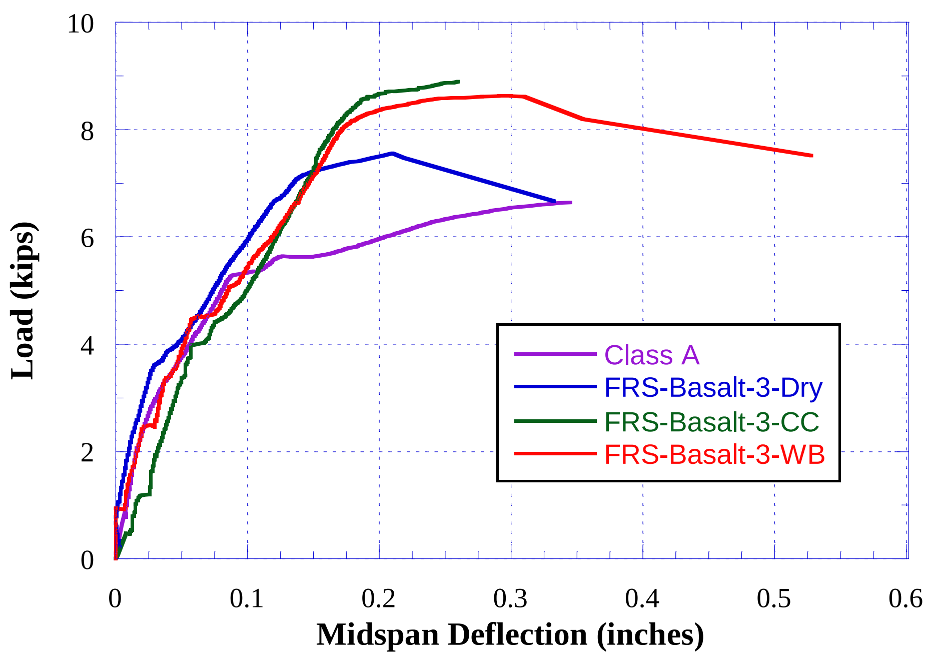


Figure 4.18 – Class A and FRS-Basalt load vs deflection at 3 day strength

For 3 day strengths, all beams had a higher ultimate strength. FRS-Basalt-3-Dry, FRS-Basalt-3-CC, and FRS-Basalt-3-WB had an increase in ultimate load capacity by 13.86%, 33.89% and 29.82%. The test results for wet burlap and curing compound curing methods are very similar with only a 4% difference in their capacity increase. For the ultimate deflection, FRS-Basalt-3-dry and FRS-Basalt-3-WB have increased ultimate deflections by 11.86% for both. The FRS-Basalt-3-CC beam had a deflection 44.92% lower.

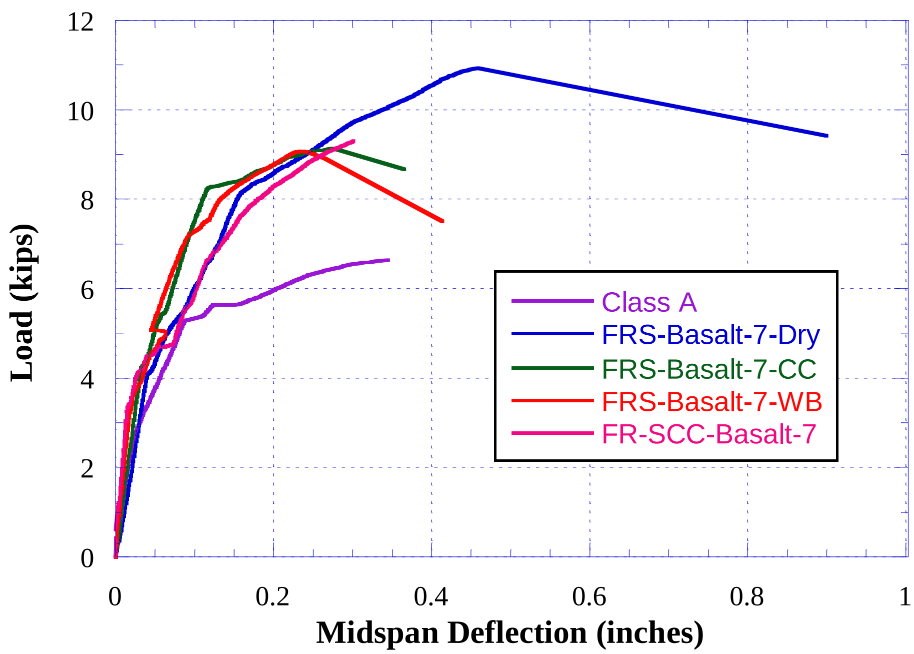


Figure 4.19 – Class A, FRS-Basalt, and FR-SCC load vs deflection at 7 day strength

At 7 day strengths, the ultimate capacity loads for the FRS beams were higher than that of the class A. The dry cured, curing compound, and wet burlap basalt beams have an increased strength by 64.46%, 37.65%, and 36.6% respectively. For the ultimate deflection, dry cured and wet burlap methods provided an increase to the deflection by 90.68% and 29.66% respectively. Similar to the 3 day performance of the FRS-Basalt-3-CC, the ultimate deflection for curing compound was much lower, with a 22.46% shorter deflection.

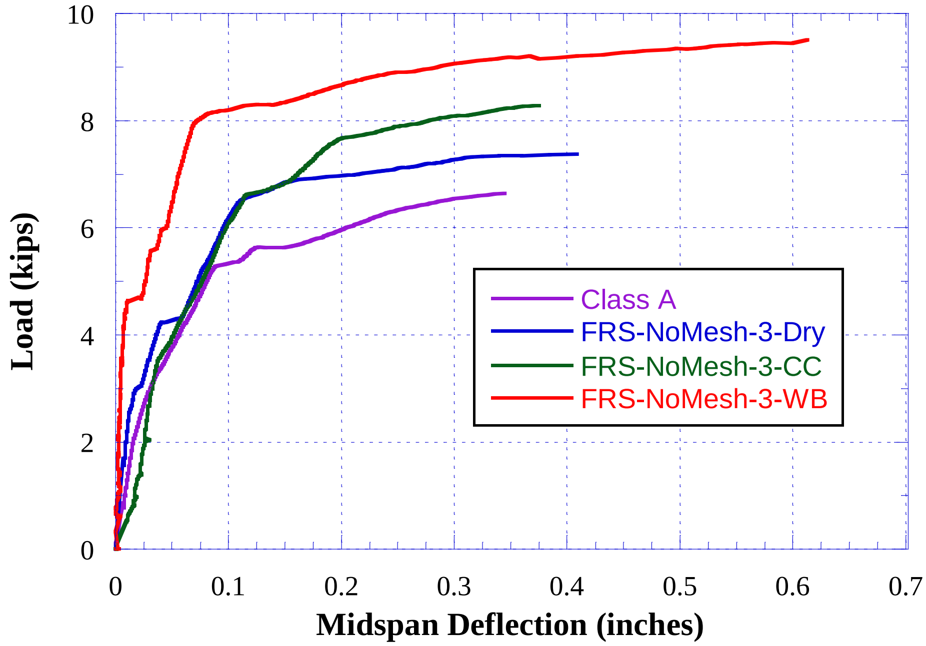


Figure 4.20 – Class A and FRS-NoMesh load vs deflection at 3 day strength

The performance of FRS-NoMesh-3 compared to class A was better in terms of ultimate load capacity. The dry cured, curing compound, and wet burlap had an increased in ultimate load capacity of 10.99%, 24.70%, and 43.07% respectively. As for the ultimate deflection, wet burlap had an increased deflection by 29.87%. Both dry cured and curing compound methods have a decreased deflection by 13.35% and 20.55% respectively.

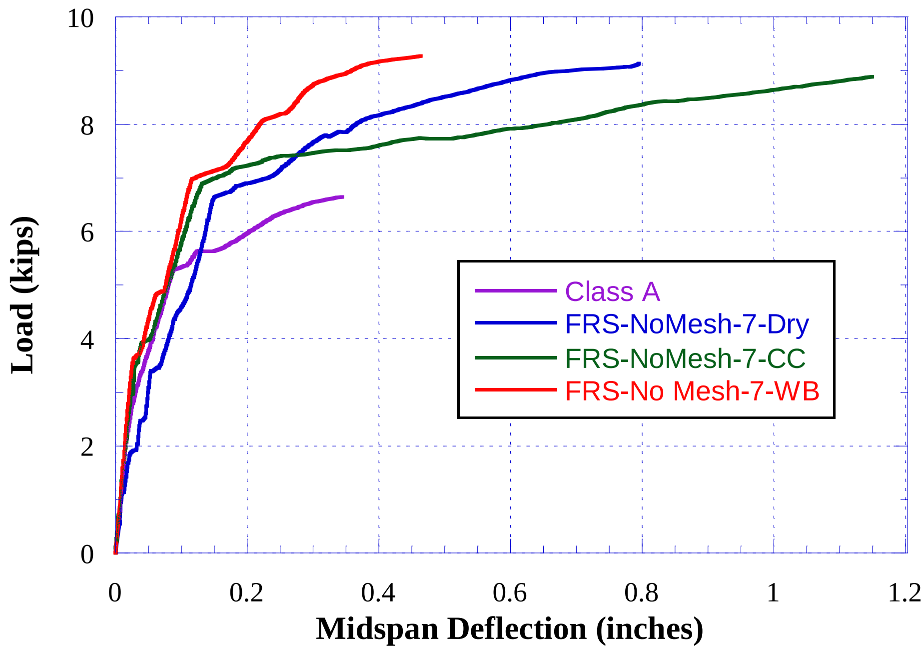


Figure 4.21 – Class A and FRS-NoMesh load vs deflection at 7 day strength

For 7 day strength with regards to the FRS beams without any mesh, the ultimate load capacity increased by 37.50%, 33.73%, and 39.61% for dry cured, curing compound, and wet burlap beams respectively. With regards to ultimate deflection, there was an increase of 68.64%, 77.12%, and 3.81% for dry cured, curing compound, and wet burlap beams respectively.

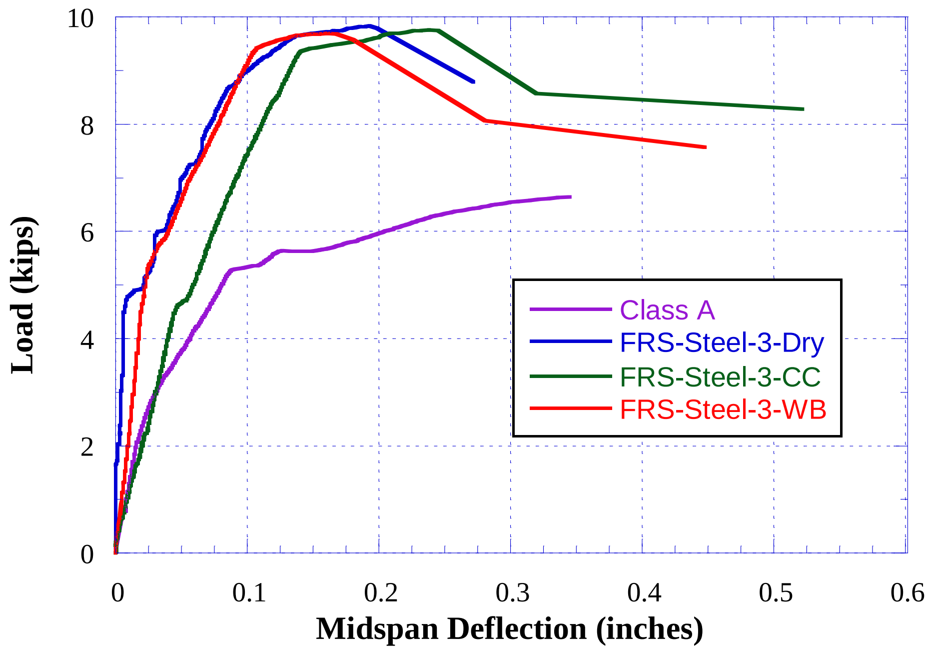


Figure 4.22 – Class A and FRS-Steel load vs deflection at 3 day strength

For steel mesh beams at 3 day strengths, the ultimate load capacity increased by 48.04%, 46.99%, and 45.93% for dry cured, curing compound, and wet burlap beams respectively. For ultimate deflection, both dry cured and wet burlap beams had decreased by 59.11% and 5.08% respectively, while the curing compound beam had increased by 10.59%.

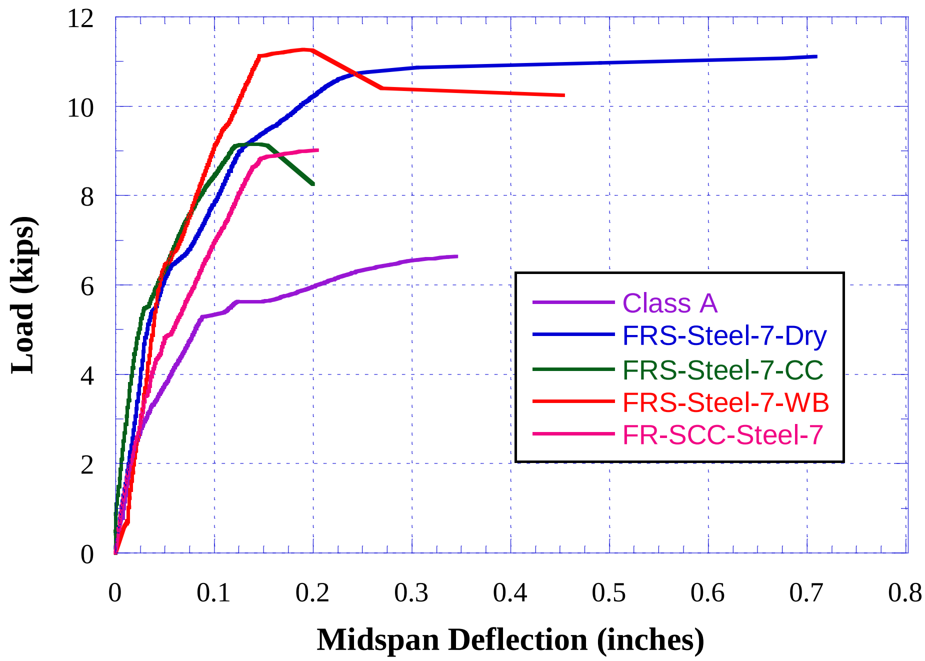


Figure 4.23 – Class A, FRS-Steel, and FR-SCC-Steel load vs deflection at 7 day strength

For 7 day strengths of the steel mesh beams, the ultimate capacity had increase for all beams by 67.17%, 37.80%, and 69.73% for dry cured, curing compound, and wet burlap beams respectively. The ultimate deflection had increased by 50.21% for the dry cured beam while the curing compound and wet burlap beams had a decrease in ultimate deflection by 57.63% and 4.03%.

4.3.4 Curing Method Load Deflection Analysis

Figure 4.24 presents all results for 3 day ultimate load capacity for each beam type, while figure 4.25 presents all results for 7 day ultimate load capacities for each beam type.

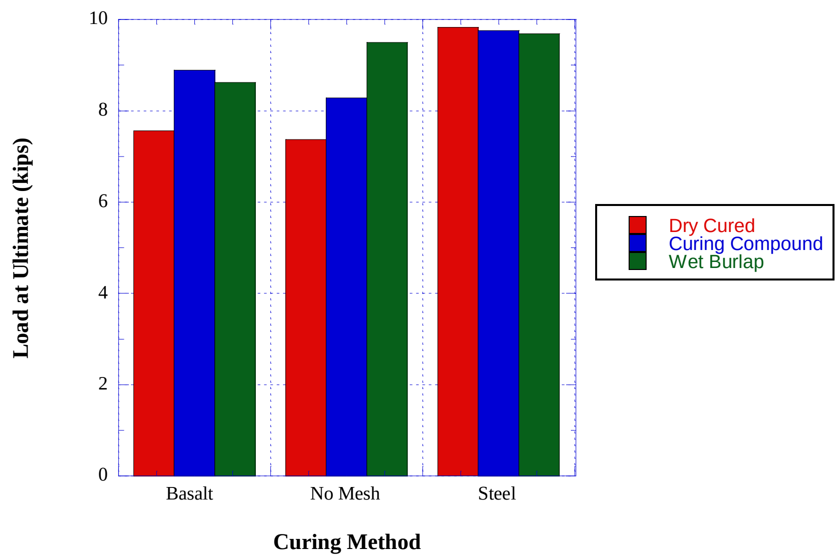


Figure 4.24 – 3 day ultimate load capacity for each FRS beam curing method

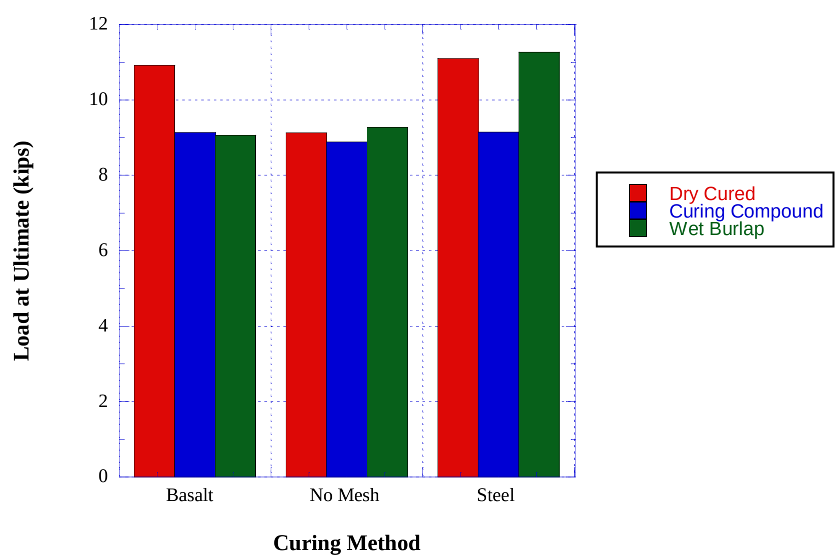


Figure 4.25 – 7 day ultimate load capacity for each FRS beam curing method

Comparing the three beam types, the mesh that stands out the most, at least as far as 3 day strength goes, is steel. The steel mesh results show consistently higher load capacities when compared to the other two mesh types, implying a better strength at early stages. That being said, there is no apparent difference between the curing types for steel at this age.

For the beams without any mesh, there is an increase in strength between the curing types from dry cured to curing compound, to wet burlap. The dry cured method being the lowest is not too much of a surprise considering the nature of leaving concrete out without any curing method. There is an excess of shrinkage at early stages for dry curing that can lead to a weaker state. The difference in curing compound to wet burlap can be associated with how it is placed and its consistency. The shotcrete laminate was not smooth for all beams, there were some minor pits which would break the curing compound film, making it uneven.

At 7 day strengths, steel is once again the most consistent. The only thing that can be taken as an outlier is the curing compound, which is lower than that of the dry cured and wet burlap curing methods for steel. That being said, curing compound is consistent throughout each mesh type at approximately 9 kips. For the beams without mesh, there is very little differences between the curing methods strengths, and for basalt, the dry cured strength is much higher than the other methods.

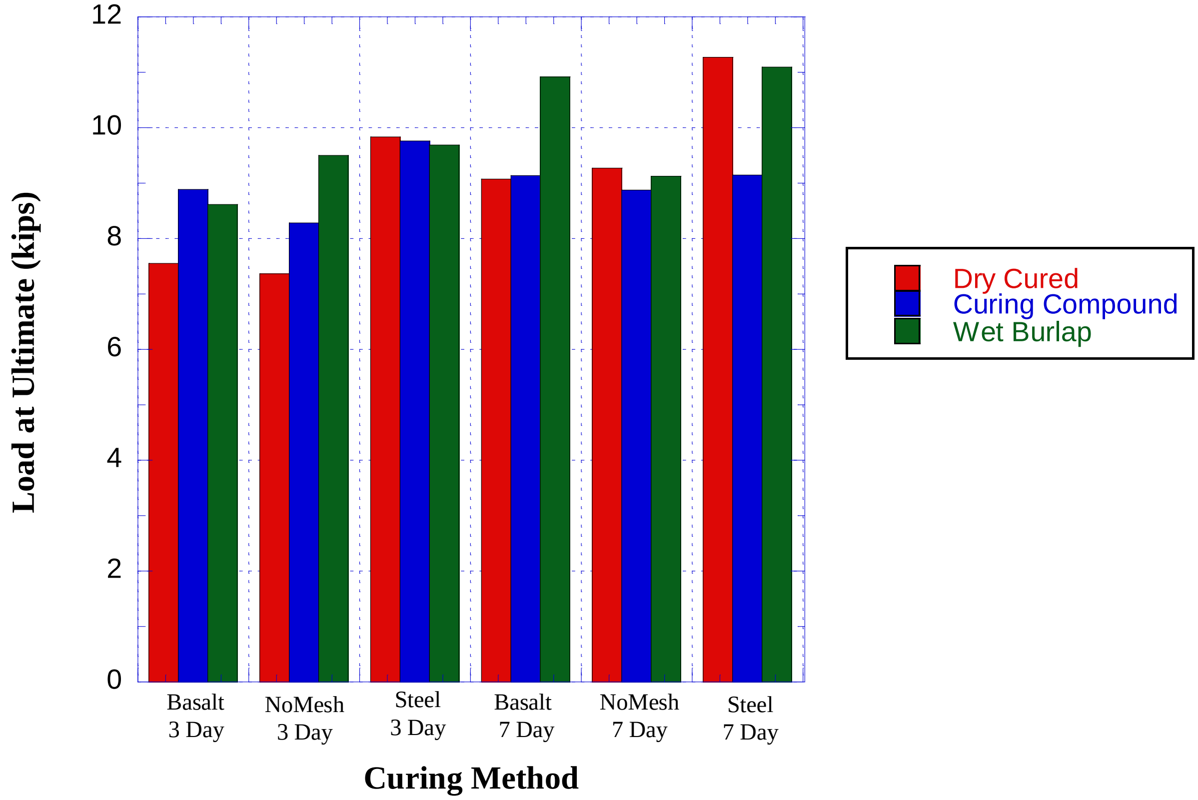


Figure 4.26 – Side by side comparison of 3 day and 7 day age beam ultimate strengths using different curing methods

To better view the changes, the 3 day results are closely compared to the 7 day results as shown in figure 4.26. This showcases a few things, the first being how static curing compound and wet burlap curing is compared to dry curing. Save for the wet burlap results for steel mesh, the basalt and beams without a mesh proved to have very little difference from 3 day strength to 7 day strength in terms of curing compound and wet burlap, however there is a large increase in strength from 3 day to 7 day strengths with regards to dry cured.

Moving onto the ultimate deflection at midspan, Figure 4.27 and 4.28 present the deflection results for each beam under their respective curing methods for 3 day and 7 day strength respectively. Figure 4.29 presents a side by side view of all ultimate deflection results.

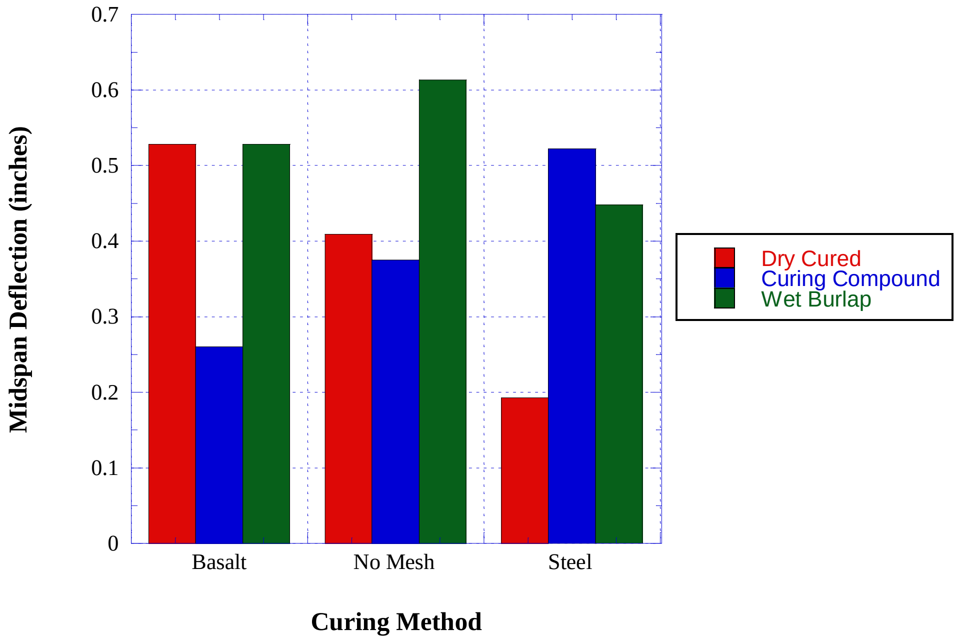


Figure 4.27 – 3 day ultimate deflection results for each curing method

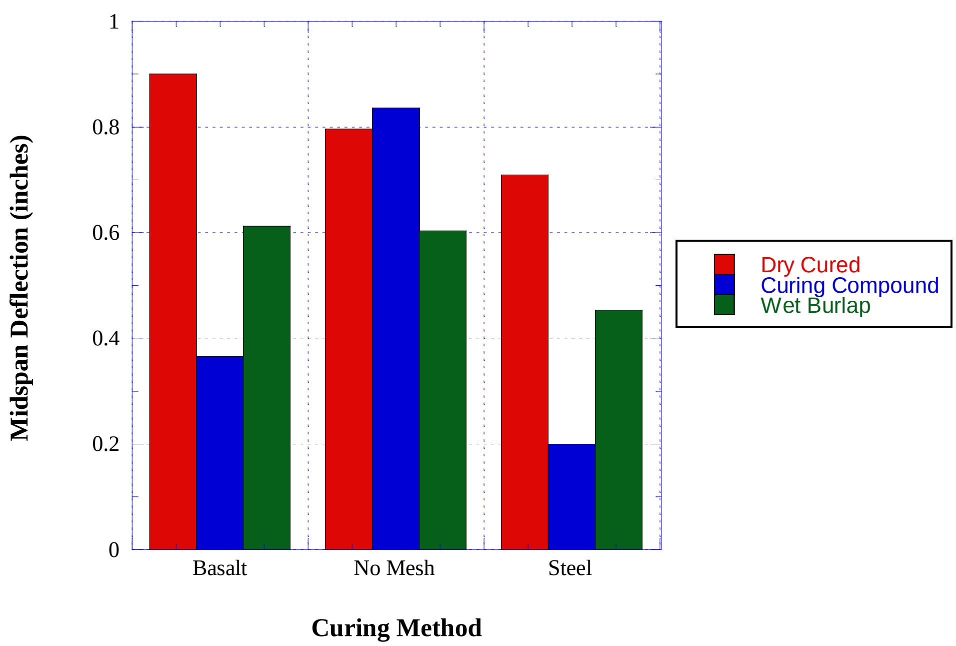


Figure 4.28 – 7 day ultimate deflection results for each curing method

At 3 day strengths, curing regimes drastically changed the deflection. The methods did not produce consistent results throughout each mesh type, implying a large impact on the laminate's part. The same can be said for the 7 day results, where there is little consistency between the curing methods. That being said, the deflection had increased from 3 to 7 day strengths, except for steel using a curing compound and no mesh using wet burlap.

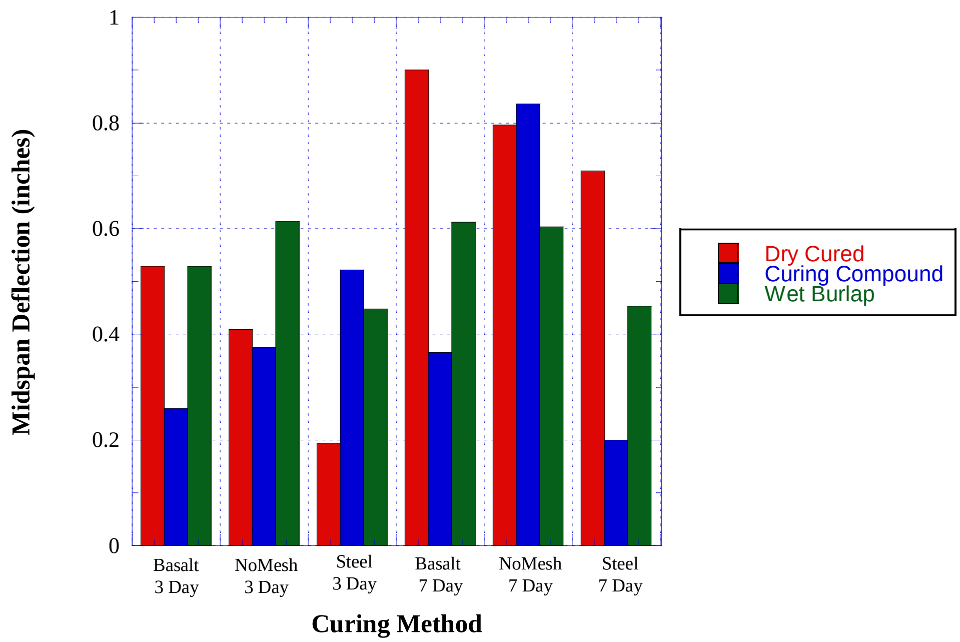


Figure 4.29 – Side by side comparison of 3 day and 7 day age beam ultimate deflections using different curing methods

Similar to the results for ultimate load capacity, the dry cured beams produced the largest difference from 3 day to 7 day, and for each difference, the deflection had increased implying a great increase to the flexibility of the beam, and large gap from elastic to plastic deformation. This is consistent throughout all beam laminate types. Curing compound has a subtle increase in deflection for basalt but massive increase in deflection for the laminate without a mesh. This could be due to the delamination experienced in the basalt mesh beams.

4.3.5 Analysis of the Load Drop

Figure 4.30 and 4.32 shows all beams that had a load drop in the results for the load versus deflection at 3 day strength and 7 day strength respectively. It is considered to be the shift at which the laminate no longer provides any major support, leaving only the substrate so resist the load.

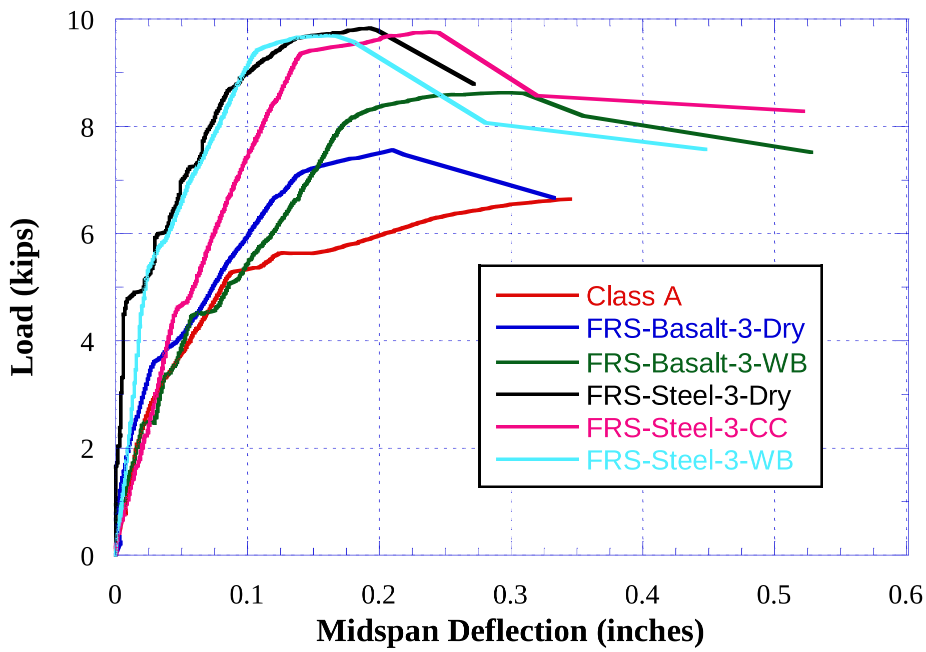


Figure 4.30 – FRS beams with a load drop for 3 day strength

The drop in the load happens after the ultimate load and drops sharply with the slope of the line approaching 0 as shown in the data above. In previous works (Nassif and Najm 2004) there is a similar dip in FRS layers that confirms this behavior as seen.

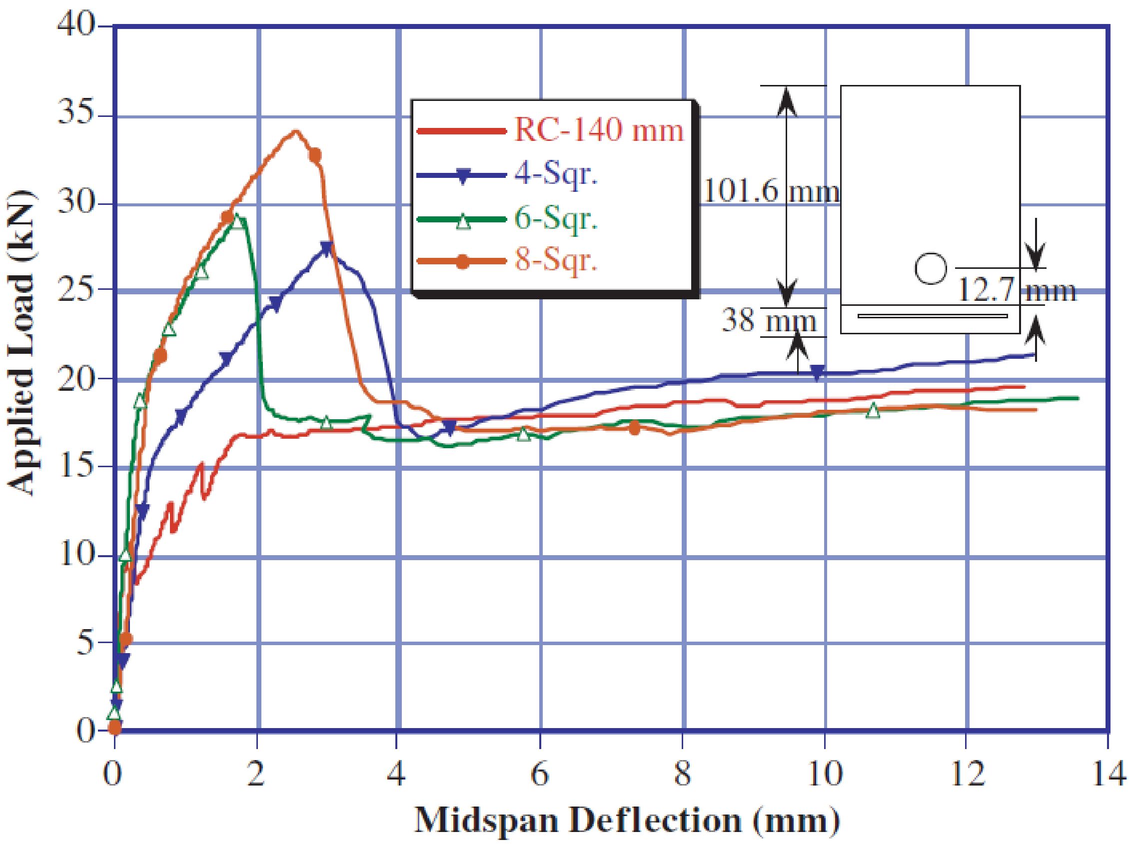


Figure 4.31 – Applied load versus deflection for group (A2) beams using square mesh.

The difference between the results seen in that paper versus the results seen here is the residual strength that the shotcrete provides, and its ability to strengthen the substrate during the curing process. The shotcrete is shot at high velocities which allows the FRS layer to seep directly into the class A layer during the hydration process. Surface saturation is done to prevent the class A layer from sucking too much water from the FRS laminate, however there is still some of the FRS content that is pushed and sucked into the still dryer class A layer. This is believed to be the reason that the drop witnessed does not go all the way to the strength of the class A when the bottom of the drop is in fact only the class A strength. This implies that not only does the FRS layer improve the member over all by adding 2 inches to the overall depth with a sufficient bond, but it enhances the substrate, improving its overall flexural strength.

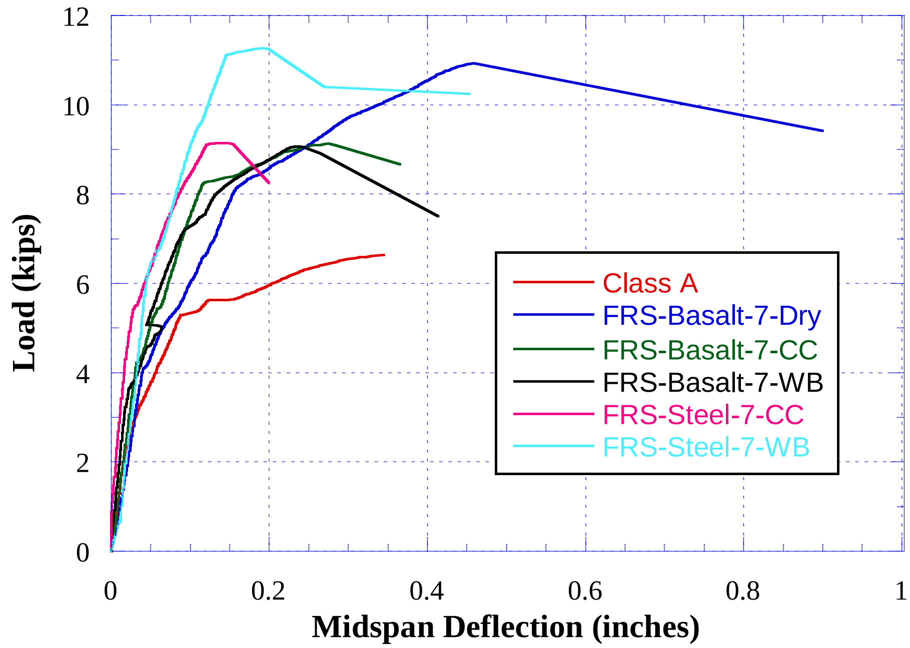


Figure 4.32 - FRS beams with a load drop for 7 day strength

This material phenomena is more noticeable at 7 day strength, in that the difference in strength is much higher. The major difference from age 3 to age 7 is that the load is much higher due to the strength of the FRS layer, however there is much less improvement from that time in the strength of the class A substrate considering it is past 28 day strength. This is why the beam fails a bit more abruptly when the load is only being resisted by the substrate. Regardless, FRS-Basalt-7-Dry and FRS-Steel-7-WB are proof that the class A layer is much stronger, implying that it is possible to drastically improve the strength of the class A layer even if the age of the class A substrate is already high enough for little strength increase.

4.3.6 Crack Propagation

Figures 4.33 and 4.34 present the crack width data for the different curing methods and laminate types for 3 and 7 day strengths respectively. Tables 4.11 and 4.12 provide the percent differences between the crack widths of the FRS beams and the class A crack widths. The crack widths that are included are the largest crack widths at ultimate failure. This helps understand the ductility of the beam by displaying how well it disperses the load throughout the length of the beam. A small amount of cracks and large crack widths imply less ductility while a larger amount of cracks and smaller crack widths imply more ductility.

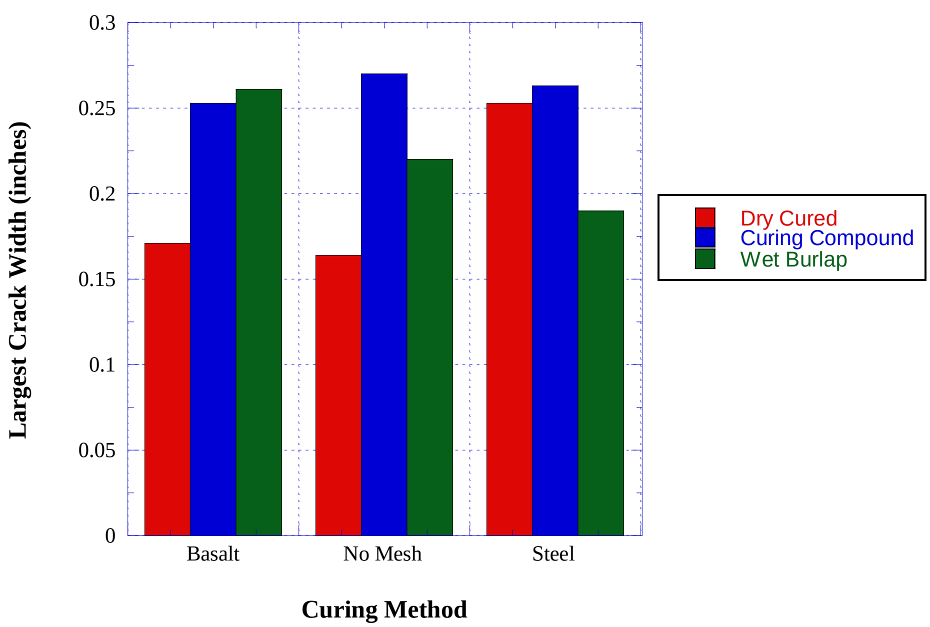


Figure 4.33 – Crack width results at ultimate of FRS samples for each curing method at 3 day strength

Table 4.11 – Percent differences of each crack width for FRS from full Class A beams at 3 day strength

Mesh	Basalt			No Mesh			Steel		
Curing	Dry	CC	WB	Dry	CC	WB	Dry	CC	WB
Width (in)	0.17	0.25	0.26	0.16	0.27	0.22	0.25	0.26	0.19
%(+/-)	8.23	60.13	65.19	3.80	70.89	39.24	60.13	66.46	20.25

The 3 day strength crack widths showcased how indifferent curing compound was with regards to what laminate type is used. That being said, it was also the worst performer at 3 day age, having higher than 0.25 for all three laminate types and getting as high as 0.27 inches in width. Wet burlap is the least consistent, and descends in size from basalt to no mesh to steel. All values were worse than that of class A, with curing compound being as high as 70.89% wider. This is ultimately a comparison to class A concrete that has cured for some time to the FRS laminate that only had 3 days to age, therefore these results are not a surprise.

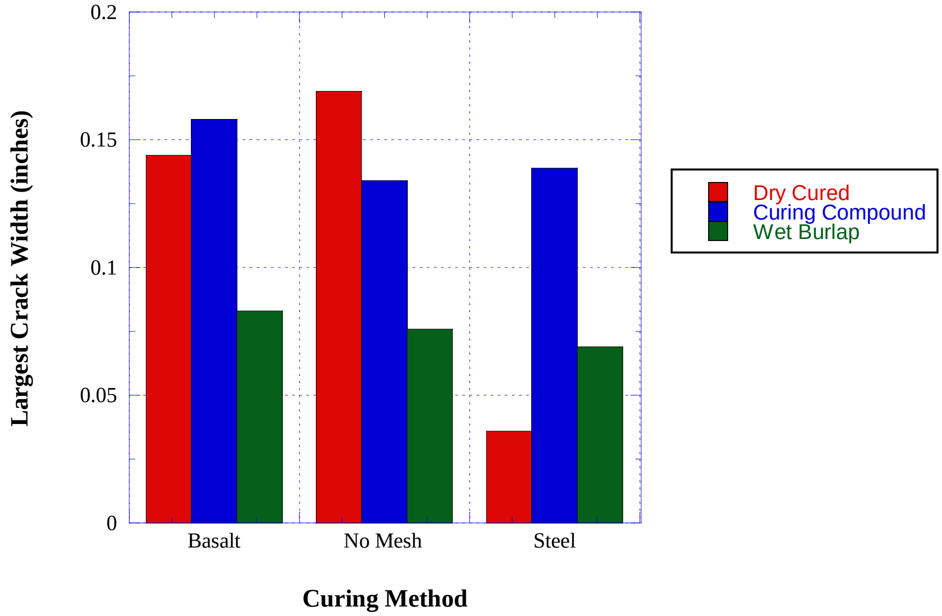


Figure 4.34 – Crack width results at ultimate of FRS samples for each curing method at 7 day strength

Table 4.12 – Percent differences of each crack width for FRS from full Class A beams at 7 day strength.

Mesh	Basalt			No Mesh			Steel		
Curing	Dry	CC	WB	Dry	CC	WB	Dry	CC	WB
Width (in)	0.14	0.16	0.08	0.17	0.13	0.08	0.04	0.14	0.07
%(+/-)	-8.86	0.00	-47.47	6.96	-15.19	-51.90	-77.22	-12.03	-56.33

The 7 day strength crack widths are much smaller than the widths at 3 days of age, with only the dry cured FRS without a mesh being larger than that of the class A. It takes some time for the laminate to mature to reach desirable ductility, however by just 7 days the laminate is proven to have favorable crack propagation over that of the class A.

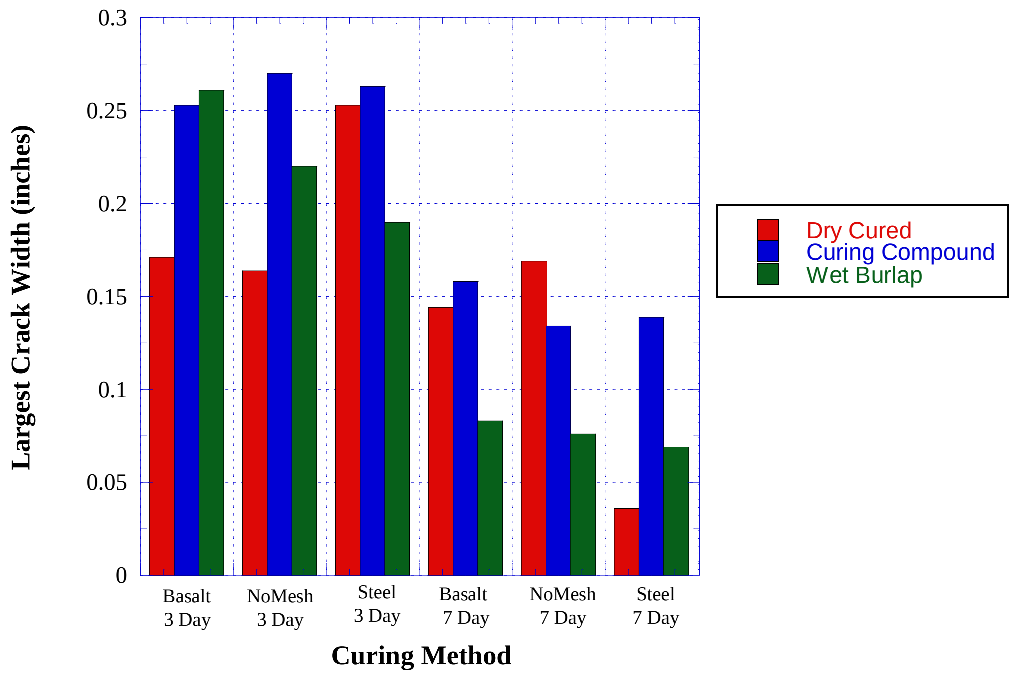


Figure 4.35 - Side by side comparison of 3 day and 7 day age beam midspan crack widths using different curing methods

Comparing the two ages together, as shown in figure 4.35, it is obvious that the crack width is overall reduced from 3 to 7 day strengths. This is because of the necessary maturity needed for peak performance from the FRS layer. The biggest difference can be seen in dry cured steel mesh FRS with a reduction of 0.21 inches, which is an 85.8% reduction. In fact, this follows suit with the performance bump seen ultimate deflection as well. In terms of ductility, dry cured not only improves the greatest amount, but is the best performer. This would most likely change at higher ages, as the curing compound and wet burlap cured beams have a slower hydration process, providing a smoother transition into the member's potential strength and ductility. If later ages such as 14 day and 28 day strengths were observed, the dry cured beams would most likely be out performed on all accounts, however due to the nature of beam retrofitting, the beams must be usable at a reasonable amount of time so the structure can be used or opened for use, therefore only 3 and 7 day beams were considered.

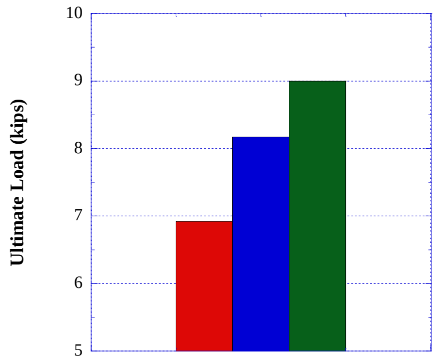
Table 4.13 – Percent differences in crack widths at 7 day strength between FR-SCC and FRS cured using wet burlap

	Basalt	**Steel**
FR-SCC-7 (in)	0.083	0.073
FRS-7 (in)	0.083	0.069
%(+/-)	0.00	-5.48

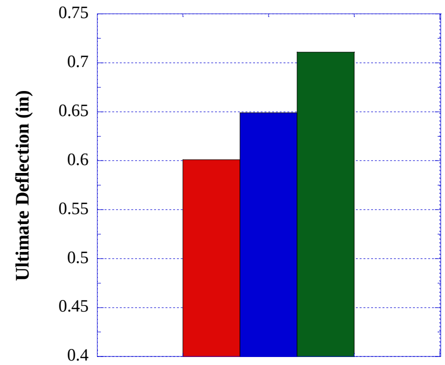
When comparing the crack performance comparison to FR-SCC, as shown in table 4.13, it is apparent that the FRS beam performs better for steel and the same for basalt. Ultimately, FRS is considered better all-around because of the nature of the basalt fabric when being shot by the nozzleman. If the basalt mesh was of a similar rigidness to the steel mesh, which is available for basalt meshes, the performance would most likely be better, and provide smaller crack widths.

4.3.7 Full Beams

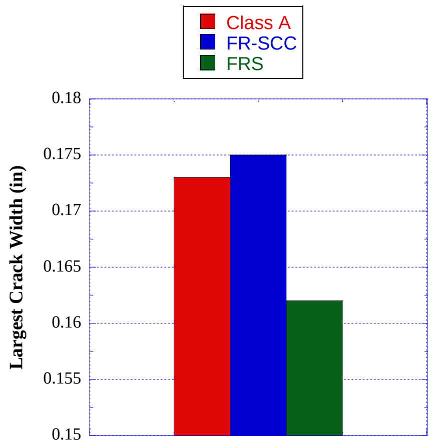
In order to gage the performance of the material as an entire beam, full FRS beams and FR-SCC beams were created and compared to the control class A beam. There is no mesh in any of the beams, only stirrups and flexural steel reinforcement. Results are presented in figures 4.36a, 4.36b, and 4.36c.



(a)



(b)



(b)

Figure 4.36 – Full beam results at 7 day strength for (a) ultimate strength, (b) ultimate deflection at midspan, (c) and ultimate crack width

Table 4.14 – Ultimate load, crack width, and ultimate deformation results for Class A, FR-SCC, and FRS mixes at 7 day strength.

	Class A	**FR-SCC**	**FRS**
Ultimate Load (kips)	6.92	8.17	9
Ultimate Deformation (in)	0.601	0.649	0.711
Largest Crack Width (in)	0.173	0.175	0.162

The FRS full beam performed the best throughout all 3 categories that were recorded. Incrementally in terms of overall performance, the materials got increasingly better from class A to FR-SCC to FRS. FRS had the highest ultimate capacity, highest deformation and lowest crack width, implying it to be the strongest in flexure and most ductile. This is expected due to the natural compaction of shotcrete bringing it the closes to high performance concrete than the others. It should be noted, however, the difference in strength from 3 day strength to 7 day strength for the FRS.

Table 4.15 – Percent differences of full FRS beams between 3 and 7 day age for ultimate load, ultimate deformation, and crack width

	Ultimate Load (kips)	Ultimate Deformation (in)	Largest Crack Width (in)
3	10.17	0.638	0.175
7	9	0.711	0.162
%(+/-)	-11.50	11.44	-7.43

The FRS full beam improved in terms of ductility, however its strength was greatly reduced. It was noticed that each FRS beam had failed in shear rather than flexure like all of the other beams. This can be due to movement in the stirrups as it was being placed, as it is difficult to guarantee that the stirrups would not move at all during casting without adding more reinforcement to hold them in place. That being said, under third-point loading, the beams was still capable to withstand a higher load than that of the other materials. Extra care must be taken to ensure that the stirrups would not move. It is believed that if the stirrups had not moved at all, the 7 day strength FRS would increase greatly.

Summary and Conclusions

The mechanical properties were an important part of this study for two reasons. The first is to compare the mixes to other sources along with their designed parameters to make sure that there are no errors during the mix design phase. The second is to look for compatibility with easier, more commonplace hardened properties testing methods could be used to gage the actual properties of the shotcrete. For this study, the compression tensile strength could be compared to the cored samples. The following conclusions could be made.

1. For compressive strength, the average difference between 4”x8” cylinders cast by hand using the shotcrete mix design is 244.0 psi with a standard deviation of 296.5 psi across the average of ages 3, 7, and 28 days. These numbers are close enough to be reliably considered accurate for gaging the compressive strength of the shotcrete for the specified ages.
2. For tensile strength, the closest comparison to the cored samples are the 4”x8” cylinders shot directly by the nozzleman which had an average difference of 19.9 psi with a standard deviation of 47.6 psi. Also close enough to be considered reliable, that being said, the 4”x8” cylinders that were cast by hand had a much smaller standard deviation of 10.31 psi, making it more reliable when being used in correlation to the strength of the cored samples.
3. 6”x12” cylinders are not recommended for use due to their high variance and unreliable results.

The major point of the project was to conclude on the capabilities of shotcrete in retrofitting concrete beams. The following conclusions can be made:

1. Steel meshes produced the best results for ultimate load for both 3 day and 7 day concrete ages. The FRS laminate without any mesh produced the highest deformation. This is consistent with the average numbers across curing methods.
2. When compared to curing compound, wet burlap yields on average the highest ultimate load, lowest crack width, and highest deflection. Wet burlap is the recommended method of curing, even though the dry curing produced more favorable results due to long term capabilities.
3. FRS with mesh enhances the strength of the substrate after the concrete laminate has failed to an undetermined degree.
4. All basalt mesh beams had exhibited some form of delamination before ultimate, and ultimately produce results that do not meet its potential.
5. Full FRS beams perform better in ultimate load, deflection, and crack width than both Class A and FR-SCC full beams.

Table 5.1 – Results of FRS beams

	Age	Dry	CC	WB
Ultimate Strength (Kips)	3 Day	8.25	8.98	9.27
	7 Day	10.38	9.06	9.87
Ultimate Deflection (inches)	3 Day	0.38	0.39	0.53
	7 Day	0.80	0.47	0.56
Ultimate Crack Width (inches)	3 Day	0.20	0.26	0.22
	7 Day	0.12	0.14	0.08

Various things would need to be verified going forward in order to advance this topic. More beams to produce a larger sample size with longer curing times would be greatly beneficial. A similar study but with different shotcrete enhancements, such as steel fibers, hybrid reinforcement, and no fibers would also enhance certain claims. For full beams, the FRS beams have lower shear strength, therefore the amount of studs would have to be looked at in order to figure out if it has a place in full beam applications.

References

1. Alam, Md Ashraful, et al. "Effective Method of Repairing RC Beam Using Externally Bonded Steel Plate." *Applied Mechanics and Materials*, vol. 567, 2014, pp. 399–404., doi:10.4028/www.scientific.net/amm.567.399.
2. Aykac, Sabahattin, et al. "Strengthening and Repair of Reinforced Concrete Beams Using External Steel Plates." *Journal of Structural Engineering*, vol. 139, no. 6, 2013, pp. 929–939., doi:10.1061/(asce)st.1943-541x.0000714.
3. Banthia, Nemkumar, and Rishi Gupta. "Influence of Polypropylene Fiber Geometry on Plastic Shrinkage Cracking in Concrete." *Cement and Concrete Research*, vol. 36, no. 7, 2006, pp. 1263–1267., doi:10.1016/j.cemconres.2006.01.010.
4. Blackburn, B. Paige, et al. "Effects of Hygrothermal Conditioning on Epoxy Adhesives Used in FRP Composites." *Construction and Building Materials*, vol. 96, 9 Aug. 2015, pp. 679–689., doi:10.1016/j.conbuildmat.2015.08.056.
5. "Certification Programs." *Concrete.org*, American Concrete Institute, www.concrete.org/certification/certificationprograms.aspx?m=details&pgm=Shotcrete Construction&cert=Shotcrete Nozzleman and Nozzleman-in-Training (Dry-Mix Process).
6. "Deficient Bridges by Highway System 2017." *Federal Highway and Safety Administration*, U.S. Department of Transportation/Federal Highway Administration, www.fhwa.dot.gov/bridge/nbi/no10/defbr17.cfm.
7. Dhanoa, Gurpreet Singh, et al. "Retrofitting of Reinforced Concrete Beam by Ferrocement Technique." *Indian Journal of Science and Technology*, vol. 9, no. 15, 2016, doi:10.17485/ijst/2016/v9i15/88243.

8. Frigione, Mariaenrica, and Mariateresa Lettieri. "Durability Issues and Challenges for Material Advancements in FRP Employed in the Construction Industry." *Polymers*, vol. 10, no. 3, 2018, p. 247., doi:10.3390/polym10030247.
9. *Guide to Shotcrete*. American Concrete Institute, 2016.
10. Jabr, Abdulla. "Flexural Strengthening of RC Beams Using Fiber Reinforced Cementitious Matrix, FRCM." *Scholarship at UWindsor*, *University of Windsor*, University of Windsor, 13 Apr. 2017, scholar.uwindsor.ca/cgi/viewcontent.cgi?article=6944&context=etd.
11. Kakooei, Saeid, et al. "The Effects of Polypropylene Fibers on the Properties of Reinforced Concrete Structures." *Construction and Building Materials*, vol. 27, no. 1, 2012, pp. 73–77., doi:10.1016/j.conbuildmat.2011.08.015.
12. Mahar, James William, et al. *Shotcrete Practice in Underground Construction: Final Report*. Dept. of Civil Engineering, University of Illinois at Urbana-Champaign, 1975.
13. Morgan, Dudley R., and E. Steffan Bernard. "A Brief History of Shotcrete in the Underground Industry." Oct. 2017.
14. Mourad, S.m., and M.j. Shannag. "Repair and Strengthening of Reinforced Concrete Square Columns Using Ferrocement Jackets." *Cement and Concrete Composites*, vol. 34, no. 2, 2012, pp. 288–294., doi:10.1016/j.cemconcomp.2011.09.010.
15. Nassif, Hani H, and Husam Najm. "Experimental and Analytical Investigation of Ferrocement–Concrete Composite Beams." *Cement and Concrete Composites*, vol. 26, no. 7, 2004, pp. 787–796., doi:10.1016/j.cemconcomp.2003.08.003.

16. "New Jersey Transportation By The Numbers." *Tripnet*, TRIP, June 2016, www.tripnet.org/docs/NJ_Transportation_by_the_Numbers_TRIP_Report_06-24-2016.pdf.
17. Sholy, Christopher. "Investigation of Fiber Reinforced Self-Consolidation Concrete Laminates in Retrofitting Concrete Beams." *Rutgers, The State University of New Jersey*, 2018.
18. "Shotcrete/Sprayed Concrete Market by Process, Application, System, and Region - Global Forecasts to 2021." *Reportbuyer*, Sept. 2016, www.reportbuyer.com/product/4169027/shotcrete-sprayed-concrete-market-by-process-application-system-and-region-global-forecasts-to-2021.html.
19. Wang, Dehong, et al. "Mechanical Properties of High Performance Concrete Reinforced with Basalt Fiber and Polypropylene Fiber." *Construction and Building Materials*, vol. 197, 2019, pp. 464–473., doi:10.1016/j.conbuildmat.2018.11.181.
20. *ASCE 2017 Infrastructure Report Card*. ASCE, 2017, *ASCE 2017 Infrastructure Report Card*, www.infrastructurereportcard.org/wp-content/uploads/2017/01/Bridges-Final.pdf.
21. Page, Kelly M., editor. *Noblestown Road Bridge Restoration Project*. Pennsylvania Department of Transportation, 2011, *Noblestown Road Bridge Restoration Project*, cdn.ymaws.com/www.icri.org/resource/resmgr/crb/2011marapr/CRBMarApr11_NoblestownRdBrid.pdf.
22. Shield, Carol, and Paul Bergson. *Experimental Shear Capacity Comparison Between Repaired and Unrepaired Girder Ends*. Minnesota Department of Transportation,

2018, *Experimental Shear Capacity Comparison Between Repaired and Unrepaired Girder Ends*, www.dot.state.mn.us/research/reports/2018/201807.pdf.

23. Souza, Regina Helena F., and Julio Appleton. “Flexureal Behavior of Strengthened Reinforced Concrete Beams.” *Materials and Structures*, vol. 30, Apr. 1997, pp. 154–159., link.springer.com/content/pdf/10.1007/BF02486387.pdf.

24. Wenzlick, John. *Viability and Durability of Shotcrete for Repairing Bridges*. Missouri Department of Transportation, 2007, *Viability and Durability of Shotcrete for Repairing Bridges*, library.modot.mo.gov/RDT/reports/Ri03011/or07014.pdf.