Investigating the Use of Fiber-Reinforced Polymer Bars in Concrete

By

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Bachelor of Civil Engineering
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Submitted to the Department of Civil and Environmental Engineering in Partial
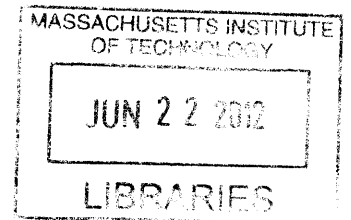Fulfillment of the Requirements for the Degree of

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ABSTRACT:

Fiber Reinforced Polymer bars were introduced to the market over two decades ago. However, their use is still somewhat limited. FRP bars are very corrosion resistant and with much of the US infrastructure degrading, the use of FRP bars over conventional steel has been proposed as a fix to this remedy. This thesis looks into the difference between the properties of FRP and steel such as: Elastic modulus, strength, time dependent behaviors, temperature effects and so on. The thesis terminates with an investigation of life cycle cost analysis performed by several authors looking through the positive and negative aspects. The Life Cycle Cost analysis concentrates on the use of FRP bars in a bridge decks. The findings suggest that FRP reinforcement is highly recommended in corrosive environments and with time will gradually be the favored material for reinforcing bridge decks as the technology slowly proves itself.

Thesis supervisor: Jerome J. Connor
Title: Professor of Civil and Environmental Engineering

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-Mounir Najm

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1.0 Introduction

Most structural engineers today have only designed structures in four materials: timber, masonry, concrete and steel. Timber has been used since prehistoric times. The Romans excelled in the use of concrete and masonry and more recently circa the industrial revolution the use of steel has boomed. All four materials are still very commonly used to this day.

Timber has constantly been used throughout all sectors of the industry, from small residence to mid-rise buildings, to small pedestrian bridges. Most of the timber structures however have an issue with their durability due to their natural decay. It is rare to find a timber structure older than 300 years-old.

Concrete and masonry have been proven materials for millennia and their durability has been proven in structures like the Pantheon in Rome(concrete) that has stood for almost 2000 years, and Brunelleschi's Dome in Florence(masonry). Both these two domes are the largest unreinforced domes in the world in each category.

More recently, during the industrial revolution pig iron was commonly used due to the ease in forming the steel and its higher strength has been refined from carbon to make a great structural material in steel. This has motivated the construction of steel buildings that may be erected faster as you do not have to wait for concrete to set with structural steel before you can continue construction as you typically do with steel.

In certain corrosive environments however, concrete has proven to be a more suitable material. It however needs to be reinforced with steel rebar due to its low tensile strength. This steel is also susceptible to corrosion due to the natural porosity of concrete.

Recently plastics with high tensile strengths have been proposed as a possible replacement to steel reinforcement and steel cables. These materials are known as fiber reinforced polymers/plastics (FRP). This thesis will look into the progress of these materials and current code guidelines for the use of these materials in current structures.

2.0 History of FRP

Most naturally occurring materials generally have low tensile strengths Historically, horsehair was mixed with mortar in order to reduce the cracking of the mortar due to both temperature changes and small tensile forces as a result of the natural creep of mortar. Similarly, straw was used in clay in order to increase its tensile strength of the clay and to reduce the cracking.
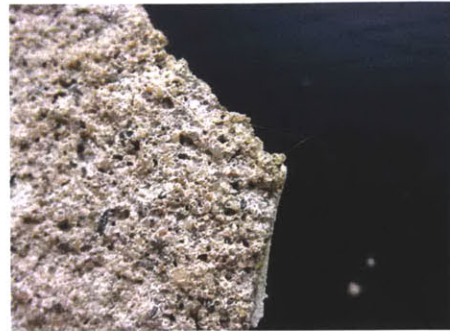
Figure 1: Horse hair in Mortar (St. Louis Brick)

Concrete like clay and mortar, creeps over its lifetime and needs reinforcement to prevent this cracking. In the early 1960s small fibers were placed in a concrete mix to avoid this phenomena. Concrete also has a high coefficient of thermal expansion and the expansion and contraction of concrete results in cracking. The small fibers in the concrete, like the horsehair in the mortar and the straw in the clay, bond to the concrete. When the concrete wants to crack and separate these fibers hold the concrete together resulting in much smaller cracks and a more consistent concrete.

This type of fiber reinforcement however is used primarily for the issues associated with cracking and increase local tensile strengths in the concrete mix. To increase the flexural strength of concrete, continuous reinforcement must be placed in the tension side of the concrete. Concrete is very good in compression, but very poor in tension. In 1863, Joseph Monier, a French gardner realized the power of placing reinforcement in concrete to take advantage of its compression and released a patent in the Paris Exposition of 1867 of iron reinforcement for troughs for horticulture. This patent, originally intended for pots. was followed by several patents until finally, in 1878, he released a patent for reinforced concrete beams.

Since the blast furnace process had still not been refined, and steel was still expensive, iron was the initial material of choice. This quickly was replaced by steel due to the improved cost to reduce the carbon that was found in the pig iron. Since the beginning of the 20th century, steel reinforced concrete has proven to be a very popular

building material and in many regions in the world, the preferred material. The durability of concrete has proven to be superior of that of structural steel.

Although the concrete encompasses the steel when it is pored and covers the steel, concrete is still porous and in corrosive environments, the steel under the concrete deteriorates. Although the deterioration is slow, it incurred significant costs in replacements and maintenance. Presetressed concrete somewhat reduced this deterioration since the concrete voids were pressed closer together.

In the 1980s much research was performed on Fiber Reinforced Polymer as a structural material in civil engineering. With much progress made, in 1987 the Canadian and Japanese societies of Civil Engineering formed committees on FRP independently. In 1991 the American Concrete Institute similarly formed ACI committee 440 ("FRP reinforcement in RC structures" 2). Following the formation of these committees, codes and design guides were released. These guides promoted the use of FRP in the industry now that there was a standard.

The first application of FRP in the structural engineering industry came in cables/tendons. Tendons primarily work in tension. Therefore the axial properties were the only ones of concern. This meant that less testing was required to ensure the material used was safe. The first large scale use of FRP was in cable stayed bridge in Winterthur, Switzerland in a vehicular traffic 124m span bridge named Stork Bridge (Keller 132). Since then there have been about a dozen similar applications of FRP in bridges all of which are being monitored and improved.

Due to some of the disadvantages of FRP, a new hybrid system has been developed that incorporated a mix of steel and FRP fibers within a resin. This new hybrid system has incorporated some of the advantages seen in both steel and FRP to make a new improved system (Cheung 81). This new design is still very novel and much testing will need done before they can be incorporated in design and construction.

3.0 What is FRP?

Fiber Reinforced Polymer/Plastic consists of fibers held together with a typically with resin matrix as shown in Figure 2. The most common fibers used are either carbon, aramid or glass fibers. All of these fibers are typically held together by a resin matrix with the exception of some aramid FRP that uses aramid filaments to encase the aramid fibers . Each of the three fibers have advantages and disadvantages of their own as they will be discussed later in this thesis.

Cross-section of a CRP or GRP rod.

● Carbon or Glass Fibre   ▓ Resin MATRIX
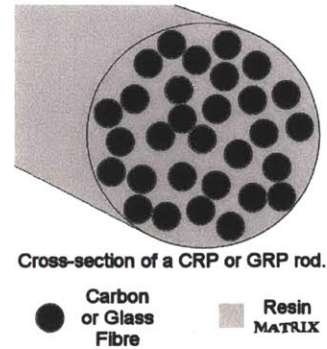
Figure 2: FRP Composition

From this we can clearly see that the transverse strength of the FRP is very different from its axial strength. For this reason, it is typically preferable to use FRP axially and in tension. ACI 440.1R-06 states that "FRP should not be relied on to resist compression". The modulus of elasticity in compression has appeared to be smaller in studies and for this reason they do not recommend FRP be used as reinforcement in any compression applications. This thesis will look into FRP used in reinforcement as bars or tendons in reinforced concrete or as cables in various applications.

FRP may be found in the form of a rod, cable, tendon, grids and even sheets all with their own unique purpose. The FRP found in concrete is typically a rod or a prestressing tendon. ACI committee 440 released two separate codes to aid in the design of these members ACI 440.1R for the rods(rebar) and ACI440.4R for the tendons (prestressing strands).

FRP may also be found as sheets that are used to externally reinforce concrete. These applications become useful when a retrofit is needed for an existing structure that does not satisfy code or when a building is being renovated. The sheets may be used to increase the shear strength of a concrete beams, by incasing the web of the beam with FRP sheets. The sheets may also be attached on the tension side of beams or floor systems to increase their flexural strength. Finally FRP sheets may be used to incase columns as to resist the outward strain when they are under compression. This has proven to significantly increase the compressive strength of columns. (Bisby 8)

3.1 Material properties of FRP

There are three types of FRP: Carbon fiber reinforced polymer (CFRP), Aramid fiber reinforced polymer (AFRP) and Glass fiber reinforced polymer (GFRP) all of which have their own properties and several "grades". The properties also depend on the manufacturer providing the FRP as the composition of the FRP dictates its properties. The manufacturer will typically provide these properties (see Appendix C).

3.1.1 Modulus of Elasticity

Typically the highest modulus of elasticity is found within CFRP, followed by AFRP and GFRP. These moduli are all however lower than that of steel but within the same order of magnitude. Shown below in figure 3 is Table3.1 from ACl440.1R-06 which displays the typical tensile properties of reinforcing bars and figure 4 which shows some of the bars used in the FRP reinforcing industry compared to steel

Table 3.3—Usual tensile properties of reinforcing bars*

	Steel	GFRP	CFRP	AFRP
Nominal yield stress, ksi (MPa)	40 to 75 (276 to 517)	N/A	N/A	N/A
Tensile strength, ksi (MPa)	70 to 100 (483 to 690)	70 to 230 (483 to 1600)	87 to 535 (600 to 3690)	250 to 368 (1720 to 2540)
Elastic modulus, $\times 10^3$ ksi (GPa)	29.0 (200.0)	5.1 to 7.4 (35.0 to 51.0)	15.9 to 84.0 (120.0 to 580.0)	6.0 to 18.2 (41.0 to 125.0)
Yield strain, %	0.14 to 0.25	N/A	N/A	N/A
Rupture strain, %	6.0 to 12.0	1.2 to 3.1	0.5 to 1.7	1.9 to 4.4

*Typical values for fiber volume fractions ranging from 0.5 to 0.7.

Figure 3: Table 3.3 from ACl.1R-06
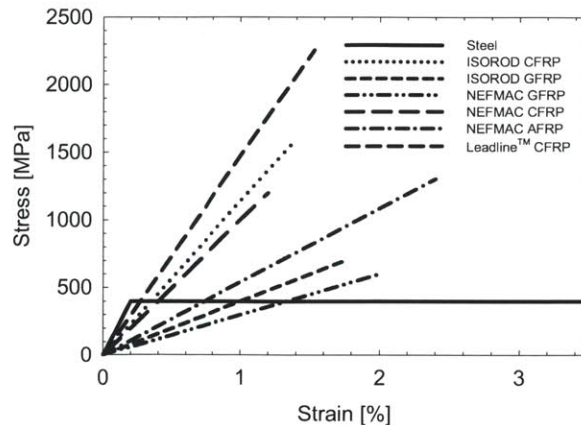
Figure 4: Examples of Industry FRP Stress-Strain curves (Bisby 6)

We can clearly see from figure 3 that the modulus of CFRP has the largest range and in some compositions can reach moduli 3 times that of steel. The most common CFRP however has a modulus around 80-90% of that of steel. Next AFRP have a modulus about 50% of that of steel followed by the lowest modulus of elasticity with that of FRP at about 20-30% of that of steel. Although this may seem low, it simply has to be compatible with the modulus of concrete so that they may have compatible strains within the elastic regions of both materials.

3.1.2 Serviceability

The problem with this lower modulus however becomes an issue with the serviceability in both cracking allowances and more importantly deflections. Since the modulus of FRP is lower that that of steel, for the same diameter bar, the FRP deflects more even though it may be able to take more load.

3.1.3 Cracking

The cracking width requirement in FRP reinforced concrete can be made less conservative when the primary concern of the cracking control width is corrosion. For this reason the Japanese Society of Civil Engineers are only concerned with the aesthetics when it comes to the crack width according to their 1997 code. The Canadian code however allows a lower requirement for exterior exposure and another larger one for interior exposure. The reason for this is to reduce the damages due to freeze-thaw cycles which are common in the cold environments in Canada. ACI adopts the requirements set by the Canadian code. Most FRP utilized is in exterior environments where corrosion is an issue and thus aesthetics are less of a concern. However, the crack width cannot be too large as they may be aggravated by the freeze-thaw cycles of the harsh environments.

3.1.4 Deflection

Another serviceability concern with FRP is deflection. The deflection experienced by the same diameter FRP are much greater that that of steel due to the lower nature of modulus of the FRP (even though it may endure a much higher failure stress). As a result, FRP reinforced members are typically controlled by deflection. Unlike ACI 318,

ACI440.1R controls deflection only by the direct method and not mandating a minimum thickness to the beam or the slab.

3.1.5 Strain failure

From figure 4 we can also clearly see that the FRP fails at much lower strain than steel. Unlike the strain hardening properties of steel, FRP exhibits a linear-elastic behavior and all the materials fail with a brittle fracture. While mild steel fails at strains reaching 15%, all FRP bars fail at a maximum of 3% yield and do not strain harden prior to failure (Bisby 6). The CFRP although appear to have the highest modulus and the highest strength of the three materials showed a much lower failure strain. AFRP showed the highest strain failure at about 2.5% and a high stress failure. The modulus of AFRP was between that observed of CFRP and GFRP. Finally the GFRP showed a very low elastic modulus, and a lower strength than the rest of the FRP bars, although it was still significantly higher that that of steel. The strain failures however were between those observed for CFRP and AFRP (Bisby 6).

As a result of this lower strain failure, moment redistribution cannot be performed as it is for steel rebar according to ACI 318. ACI 440.1R-06 does not allow for moment redistribution in continuous beams or statically indeterminate systems when FRP bars are used (8.2.5.2 ACI 440.1R-06).

3.1.6 Effects of Temperature

The resistance to high temperatures must be taken into account for FRP. Since the concrete cover protects the tendons and rods, the FRP does not burn. It however softens at higher temperatures, which may be experienced in a fire. The temperature at which the FRP's modulus begins to decrease is known as the glass-transition temperature. This is a result of the softening of the resin matrix holding the fibers together and added excessive heat alters its molecular structure. Normally this value is in the region of 65-120°C (ACI 440.1R-06).

Recent studies have presented many results that demonstrate this phenomenon. Below in figure 5 we can see the glass transition temperature of several types of FRP compared to the change in modulus of steel reinforcement specimens. The steel

specimen with the higher diameter shows a constant decrease of modulus with temperature increases while the thinner diameter steel suddenly drops after 400°C. The GFRP and the CFRP have a relatively constant modulus for several of the high temperatures and appear to have a glass transition temperature of approximately 400°C where the modulus declines rapidly (Wang 1).

Figure 5: Normalized Modulus vs Temperature (Wang 7)
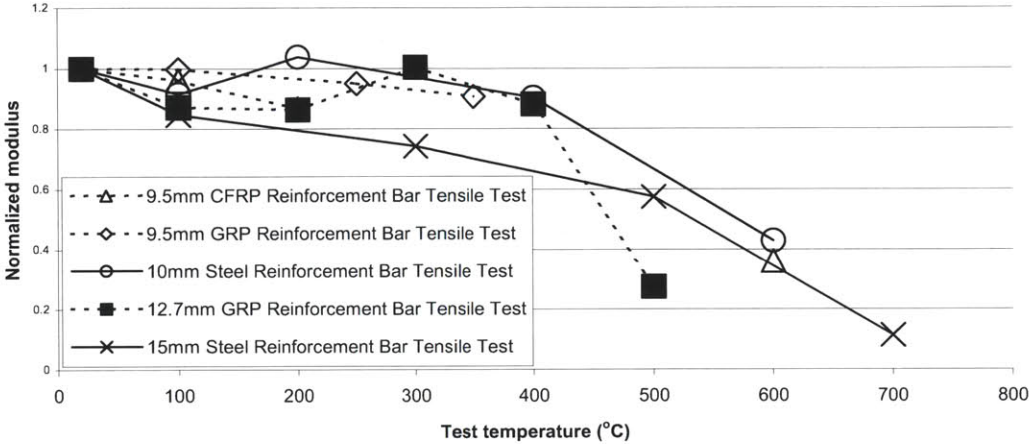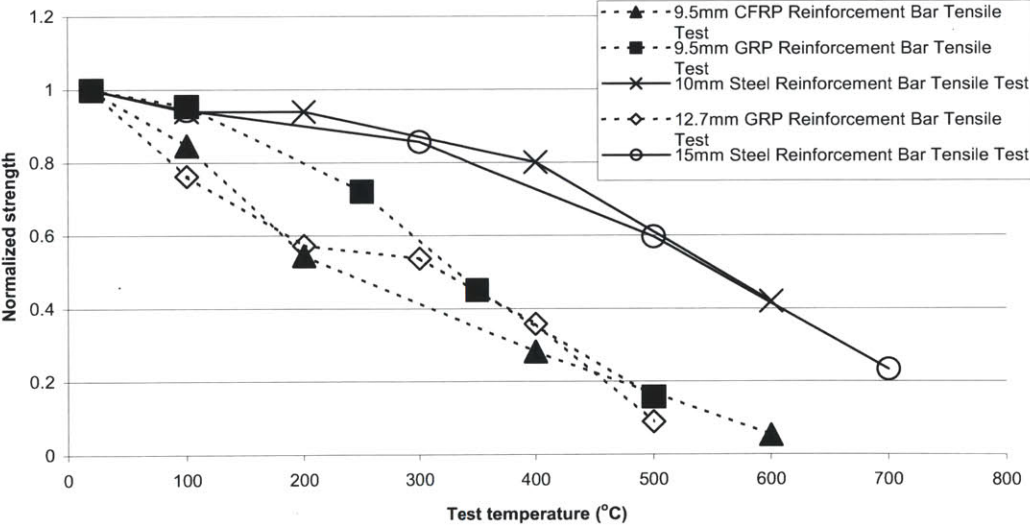

Figure 6: Normalized Strength vs Temperature (Wang 7)

From figure 6, we can see that the strength of the FRP cables decreases rapidly with an increase in temperature while the steel strength remains relatively constant until about 400°C. This makes the strength more of an issue rather than the modulus as a member will more likely fail before it sags significantly.

Since FRP is more resistant to corrosion the cover is typically reduced, however, keeping this issue in mind, the cover has to be increased in order to increase the fire rating of a structure, thus losing this advantage of FRP. In the present industry however, the use of FRP has been primarily directed toward bridges, seawalls and other highly corrosive environments. Their use in building reinforcement where fires are much more likely to break out and where a fire rating is required however, has been less valued and not financially suitable to implement. Part of the reason for this is that building reinforcement is typically within controlled environments.

The lower temperatures that a bridge is more likely to encounter over the long winters in North America in fact help both the strength and the modulus of FRP reinforcing members (Roberts 357). The lower temperatures help increase the strength by about 20% and make the FRP generally stiffer. These properties are all based on the resin matrix holding the fibers together.

3.2 Time dependent behavior

Over time with different loading patterns or sustained loading, FRP bars react slightly different than in the short run. Two considerations that have to be taken into account in the long run for the endurance and safety of FRP structures: creep rupture and fatigue.

3.2.1 Creep Rupture

Creep rupture is the sudden failure of an FRP bar that is experiencing a constant load over a long period of time. Creep rupture is also know as static fatigue as the failure is sudden and under static loads. This property is not seen for steel reinforcement. The time it takes for an FRP member to fail under a static load is known as the endurance time. There is a negative correlation between the ratio of the applied load to the actual strength of the FRP bar and the endurance time. Other factors that effect the endurance time negatively are the environmental conditions such as "high temperatures, ultraviolet radiation, high alkalinity, wet and dry cycles or freezing-and-thawing cycles" (ACI 440.1R).

ACI 440.1R-06 limits stresses due to sustained loads using table 8.3 also displayed below as figure 7 where f_{fu} is the tensile strength of the FRP bar after reductions due to the service environment.

Table 8.3—Creep rupture stress limits in FRP reinforcement

Fiber type	GFRP	AFRP	CFRP
Creep rupture stress limit $f_{f,s}$	$0.20f_{fu}$	$0.30f_{fu}$	$0.55f_{fu}$

Figure 7

Carbon fibers are the least susceptible creep rupture thus relaxing the requirements as it can be seen in figure 7. The CFRP stress rupture limit is almost double AFRP and GFRP's stress limit. In cases where the sustained load is expected to be large, it would be preferable to use a CFRP bar.

3.2.2 Fatigue

Another Time dependent behavior that is important to investigate is that of fatigue. In cases where the bars will experience cyclic loading such as cars running over a bridge deck, it is important to investigate the fatigue properties of the different ranges of FRP rebar. The fatigue tests performed for FRP are typically tension-tension sinusoidal in the direction of the principle fibers with a large period as not to cause self-heating of the specimen (ACI 440.1R-06). All these procedures are to mimic the actual loads that an FRP bar may experience in its lifetime within concrete.

The environment also plays a small effect on the fatigue resistance of FRP. The environmental effects play their largest role in the fatigue of glass fibers which are susceptible to damage from alkaline, moisture and acids. Carbon and Aramid fibers are much less prone to damage from the environment that might affect their fatigue life. The damage greatly depends on the resin used, as the environmental conditions only appear to become an issue when the resin matrix is heavily degraded by the environmental effects (Iqbal 41). Generally the least affected of the three to fatigue is AFRP followed by CFRP and GFRP. GFRP generally has poor fatigue resistance; however it is still superior to that of steel as it can be seen in figure 8 below.

Material systems	Fatigue Strength (ksi)				R	Reference
	10^6 cycles		10^7 cycles			
	ksi	MPa	Ksi	MPa		
E-glass / Phenolic	30	207			0	Branco et al, 1994
E-Glass / Epoxy	30	207			0.1	Agarwal, 1990
Graphite / Epoxy	140	965	30	207	0.1	Agarwal, 1990
Kevlar 49 / Epoxy	150	1034	140	965	0.1	Agarwal, 1990
E-Glass / vinyl ester	9	62			0.1	Hayes et al, 1998
S2 Glass / Epoxy	50	345			0.1	Hayes et al, 1998
Boron / Epoxy	150	1034	140	965	0.1	Agarwal, 1990
2024-T351 Aluminum	21 (threshold)	145			0.1	Chatterjee et al, 1997
Steel	24 (threshold)	166			0.1	AASHTO, 1994

Figure 8: Fatigue Properties (Iqbal 78)

We can clearly see from the figure above that fatigue characteristics of all FRP appear to be superior of those of mild steel. As mentioned earlier, Aramid appears to have the greatest resistance to fatigue. One of the reasons for this may lie within the fact that it can take the larger strain before failure yet still has a high strength. The ribs and surface deformations induced in the rebar to increase the bond strength have proven to increase local stresses thus reduce fatigue life of rebar within concrete (Katz 121). The extra strain that a material can take can significantly help this issue and increase fatigue resistance.

3.3 Corrosion

The primary reason FRP was proposed to replace rebar is due to the corrosion resistance in harsh environments such as marine retaining walls or highway bridges where high concentrations of deicing salts may be found. Salts are know to make steel rust and accelerate the degradation of iron as it can be seen with the increased rusting of cars near the ocean or in cold climates where the use of deicing salts are very common.

Although concrete may appear to be impermeable, it is actually porous and cracks in concrete are very common and typically expected. This porosity means that water can still reach the rebar inlaid within the concrete. When salts are dissolved in the water as you would get in marine environment the corrosion is accelerated.

BEFORE CORROSION.

BUILD-UP OF CORROSION PRODUCTS.

FURTHER CORROSION. SURFACE CRACKS. STAINS.

EVENTUAL SPALLING. CORRODED BAR. EXPOSED.
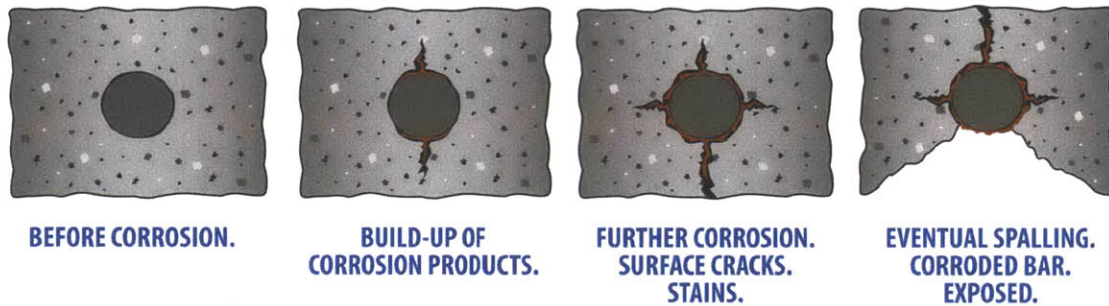
Figure 9: Rust induced cracking (FRP Distributors Inc.)

Since rust has a much greater volume than steel, it needs room to expand and this pushes out on the concrete and encourages cracking and later spalling. The corrosion of the bars results in both the corrosion of the steel rebar and the significant cracking and spalling of the concrete cover which in turn could accelerate the corrosion. The spalling could also make a bridge deck much bumpier for passengers driving over it and accelerate the damage to a bridge.

FRP does not degrade under these conditions and as a result can be more sustainable for all structures, in particular ones that are exposed to harsh environments. This in turn can reduce maintenance and prolong the life of a structure thus saving the owner a significant cost. The life cycle cost analysis of FRP will be looked at in more detail in the next section of this report.

4.0 Miscellaneous Advantages

There many additional advantages of FRP over steel in the structural engineering industry that might make them preferable. Two that will be discussed in this section are their electro magnetic neutrality and their generally low weight.

For many technologies in the modern day, electro magnetic neutrality is essential. With many advancements in the magnetic levitation trains in Japan and with the technology expecting to grow in the next several years, electromagnetic neutrality is required in that application. Very high magnetic forces are induced to levitate the train thus the surrounding reinforcement should not be attracted to these coils. The attraction may result in the failure of these structures by pulling them out of the concrete if the cover is too small. Similarly and more commonly, electromagnetic neutrality is needed in hospitals where the Magnetic Resonance Imaging (MRI) machines are used. These machines create magnetic fields of several Tesla and thus must be surrounded by a neutral environment as to both avoid damage to the building or the MRI machines (Erki 48).

Figure 10: Carrying a bundle of FRP Rebar (Bisby 12)

It is also very important to note that a bar of FRP is between one seventh and one fifth the weight of a bar of steel of the same diameter. The strength to weight ratio of FRP is between 10 and 15 times greater than that of steel. This has many advantages in construction. As it can be seen in figure 10 the construction worker is carrying a bundle of FRP rebar on his shoulder with ease. This can greatly accelerate the construction time and reduce the costs. Rebar are commonly hoisted in using a crane and this saving the cost of a crane in this case too. The high strength to weight ratio can also significantly cut down on the number of rebar used in reinforcement. It is very

common in construction to have a very crowded layout of rebar in order to achieve the required strength in high loading scenarios. By substituting the steel rebar with FRP rebar, the number of bars used can be significantly decreased. This could make the construction much easier for the contractors who sometimes spend valuable time trying to make the various meshes of rebar fit within a certain area. With less rebar the contractor is more confident that the concrete encases all the rebar with minimal voids.

4.1 Miscellaneous Disadvantages

FRP also comes with many disadvantages. Firstly FRP cannot be bent into hooks on site like steel. In appendix C the price to bend a FRP is given per hook. The FRP is fabricated using thermosetting resin and thus the FRP bars have to be bent while being fabricated. Codes typically require the minimum radius of a bend is larger than that of steel and the tensile strengths are reduced by 50%. (Bisby 14). Since the bars are required to be bent when they are fabricated, if there is a wrong purchase order during construction, construction time may be delayed and thus hike construction costs. With steel, the bars may be bent on site and save significant time in construction.

Since FRP is a much more brittle material, when it is laid in layers prior to pouring a slab, it is typically recommended that the construction workers should not walk on the rebar. Small damages to the rebar can significantly reduce the strength of the rebar. Unlike steel, FRP does not have the ductility and the strain hardening that steel does thus small damages to the FRP can be very dangerous. Construction workers with knowledge of the delicacy of FRP damage can walk on the FRP in such a way as not to damage it (Bisby 14). Construction workers must be educated for this and must be very careful while working which again could possibly slow construction and cost the project slightly more.

The delicacy of FRP also means that it must be handled with care. When FRP rebar is brought on sight, they must be placed in certain conditions in order to avoid degredation and possible damage to the FRP. FRP bars degrade under UV radiation and as a result need to me covered by an opaque plastic to ensure the quality and

performance (Bisby 13). This does not necessarily reduce construction costs since steel rebar must also be protected from moisture on site to avoid corrosion. Epoxy coated steel rebar must also be very carefully handled as flaws on the surface can result in accelerated corrosion. This property does not increase construction costs but must be taken into consideration.

Unlike steel rebar, FRP bars cannot be cut by shearing as that incurs significant damage in the resin and fibers in the rod. The rods must be cut by a fine grinding disc to ensure the quality and performance of the rebar (Bisby 12). This may incur additional labor. In addition the splicing of FRP bar cannot be mechanically connected or welded like steel. This however is not common in rebar and is typically expensive and uncommon in the field. An overlapping splice is typically more common and cost saving.

Finally, in a hybrid system of both FRP and steel, the CFRP may not be spliced with the steel or even be in contact with steel. CFRP interacts with the steel to cause galvanic corrosion which may significantly increase the corrosion of the steel (Tavakkolizadeh et al. 200). This means that the CFRP must be held up with composite chairs. The ties that hold the rebar together must also be nylon or another plastic as to avoid corrosion in any part of the concrete. This can make a hybrid system very inefficient as the contractors will have to pay special attention as to make sure no steel is in contact with the CFRP. This galvanic corrosion is not seen with AFRP and GFRP rods.

5.0 Cost of FRP

As it can be seen in the previous sections, there are a lot of extra costs that come with the use of FRP. First and foremost the price of a bar of FRP is significantly greater than that of steel.

	Steel	GFRP	CFRP
#4 bars	$0.25/ft	$0.68/ft	$8.82lb/ft
Source	Discount Construction Rental & Supplies, Inc.	Appendix	Appendix

Figure 11: Price Compasriosn

In figure 11 above we can clearly see that the price of steel rebar is by far the least expensive followed by GFRP which is almost three times the price and finally CFRP which is significantly more expensive than the rest of the rebar used. AFRP has very similar properties to CFRP; however it is significantly more expensive thus, less popular as a rebar. The cost of the steel rebar shown above is simply the bare steel bar. It is very popular in corrosive environments to use epoxy-coated bars significantly hiking the price of the steel reinforcement by about 35% of the cost of conventional rebar.

In appendix C, it can also be seen that there is a significant price of $1.5 incurred on every required bend in a FRP bar. Steel can be bent onsite at close to no additional cost. These bends can significantly increase the price tag associated with the use FRP in a reinforced concrete structure.

In the construction of a bridge, the cost of the reinforcement is just a very small part of the price of the bridge. There are many other material costs such as the concrete scaffolding etc.. The use of FRP will clearly cost more since special ties and special care in construction must be taken, however the initial cost of a bridge is a very small fraction of the cost of the bridge over its lifetime. To make the use of FRP justifiable a life cycle cost analysis must be performed over the desired lifetime of the bridge. The next section of the paper will look at several papers that have performed life cycle cost analysis.

5.1 Life Cycle Cost Analysis

According to ISIS Canada Educational Module No. 7 Life Cycle Cost Analysis (LCCA) is "a philosophy and methodology of the iterative engineering design process in which engineering systems and artifacts are designed for the life cycle to meet specified needs and levels of service at a minimum life cycle cost." Life cycle cost analysis can be applied to a very broad range of disciplines from products to structures.

When considering the price of a bridge, more than just the initial price should be taken into consideration. Through the lifetime of a bridge there are many maintenance costs such as resurfacing the deck, along with the cycle of resurfacing and concrete repair costs along with the cycle regarding the concrete repairs all the way up to the decommission costs of the bridge. Below in figure 12 we can see the life cycle costing of a bridge deck.
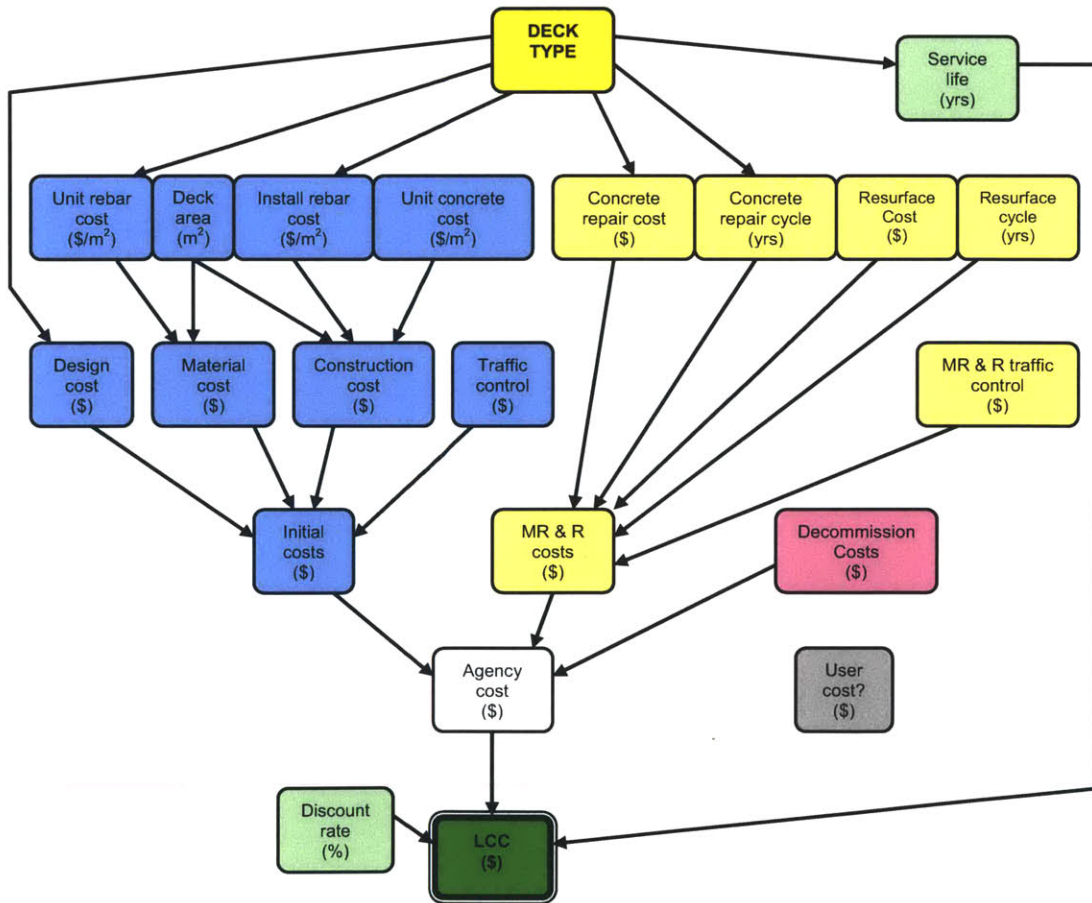
Figure 12: Simplified Life Cycle Costing (Sparks et. Al)

In figure 12, the blue boxes show what goes into the initial costs. "Unit Rebar cost" will be one that is significantly increased with FRP rods. The installation of the rebar for GFRP may also be slightly more expensive as more care should be taken when handling the bars and special ties and chairs must be used which could also increase the costs. However as it can be seen that these two costs are just a fraction of the total cost and the yellow life cycle analysis costs. Figure 12 does not include user costs or externality costs.

There are however many constraints to this model that need to be taken into account when doing the LCCA. Firstly, the costs of maintenance and work on the structure may vary with time. Although it is typically easy to make a good estimate of the initial cost, the long-term costs are much more difficult to estimate. It is highly beneficial to companies to log useful information from previous projects to help perform the LCCA. The required information is also typically hard to find within one company. For example, a concrete repair firm is very likely to be different from the initial contractor of a bridge. The gathering of information may be a troublesome and inaccurate task. When obtaining this data it is essential to pay attention to where this data is being retrieved as the distributors may hold biases to promote their product (Bisby 7-14).

Secondly, there is a lot of uncertainty and risk in the LCCA that may arise through the lifetime of the structure. There are many requirements that the model may have been based off when the LCCA was performed that might change in the real time life of the structure. These unforeseeable requirements may be attributed to environmental, legal, or regulatory concerns as well as any unexpected changes that change the total LCC cost (Bisby 7-15).

In the next section we will look at life cycle cost analysis performed by several institutions and consider the pros and cons of each set of analysis.

5.2 Case Studies of LCC analysis performed

5.2.1 Bisby – Module 7

The first model investigated by Bisby et al Module 7 uses Sparks et al's simplified life cycle engineering methodology to obtain an LCC that compares conventional steel rebar deck and the use of FRP rebar. The model includes the initial costs, maintenance, repair and rehabilitation, decommissioning costs and new technology costs.

The positive aspect of this model is that it includes the price of implementing the new technology. Since most designers and contractors have much less experience with designing and installing FRP bars as reinforcements in bridge decks, it will typically take more time to both design in the office and implement in the field.

In addition, the values used are very conveniently acquired from experienced bridge engineers. Experienced bridge engineers can give very accurate costs estimates for the initial costs making the LCC more accurate. With many assumptions and estimates in the model, it is very important to use more precise estimates where possible.

The model also employs a valuable economic tool in the discount rate and adjustment for inflation that computes the present value of an investment as to more accurately represent the risk and inflation and to display a life cycle cost in terms of real present values. The model uses

$$AWLCC_n = AWIC_n + AWMRC_n + AWDC_n$$

$AWLCC_n$: Annual worth of life cycle costs
$AWIC_n$: Annual worth equivalent of all initial costs
$AWMRC_n$: Annual worth of all maintenance and repair costs
$AWDCn$: Annual worth of all decommissioning costs.

Figure 13, shows the LCCA of a bridge deck using conventional steel rebar and GFRP rebar (Bisby 7-19). The analysis displays that the annual worth of life cycle costs (AWLCC) for each deck option. It can clearly be seen that the GFRP outperforms the steel deck option and costs 30% less in annual terms. The GFRP bridge deck is also expected to have a life expectancy of 75 years which is 25 years longer than that of the steel deck.

STEEL REINFORCED DECK		GFRP REINFORCED DECK	
Discount rate:	6.0%	Discount rate:	6.0%
Service life (years):	50	Service life (years):	75
Initial Cost		**Initial Cost**	
Design ($):	25,000	Design ($):	35,000
Traffic control ($):	150,000	Traffic control ($):	150,000
Deck area (m2):	6,000	Deck area (m2):	6,000
Unit rebar cost ($/m2):	25	Unit rebar cost ($/m2):	94
Unit concrete cost ($/m2):	300	Unit concrete cost ($/m2):	300
Install rebar cost ($/m2):	25	Install rebar cost ($/m2):	20
TOTAL PWIC:	$ 2,275,000	**TOTAL PWIC:**	$ 2,669,000
AWIC:	$ *144,336*	**AWIC:**	$ *162,192*
M&R Costs		**M&R Costs**	
M&R traffic control ($):	75,000	M&R traffic control ($):	75,000
Concrete repair ($):	5,000,000	Concrete repair ($):	2,500,000
Concrete cycle (yrs):	25	Concrete cycle (yrs):	50
Resurface ($):	150,000	Resurface ($):	150,000
Resurface cycle (yrs):	25	Resurface cycle (yrs):	25
PW concrete repair:	$ 1,182,468	**PW concrete repair:**	$ 139,793
AW concrete repair:	$ 92,501	**AW concrete repair:**	$ 8,869
PW resurface:	$ 52,425	**PW resurface:**	$ 52,425
AW resurface:	$ 4,101	**AW resurface:**	$ 4,101
AWMRC:	$ *96,602*	**AWMRC:**	$ *12,970*
Decommission Costs ($):	3,000,000	**Decommission Costs ($):**	3,000,000
AWDC:	$ *10,333*	**AWDC:**	$ *2,306*
TOTAL AWLCC:	$ 251,270	**TOTAL AWLCC:**	$ 177,468

Figure 13: Details of Analysis (Bisby 7-19)

As mentioned in the previous sections, there are many limitations to this simulation. The LCC assumes that the FRP rebar will last a certain amount of time prior to resurfacing or that repairs are less frequent with FRP reinforced concrete decks. This however has not been proven directly. The concrete in itself regardless of the steel or FRP can crack and accumulate damaged and the exact damage done by the steel rebar is hard to predict. The damage can be a result of heavy snow plows during a harsh winter or simply just a result of the salts for applied for deicing that degrade the concrete (Hayes 63). Since there have not been any bridges installed with FRP for longer than 10 years a probabilistic analysis was carried out when estimating the concrete repairs using ranges specified by experienced engineers. The numbers they provided have not been backed empirically hence much uncertainty arises.

In addition, this paper proves a stronger point as it does not incorporate the operational or the externality costs. When repairs need to be made, there is a cost to commuters that are using the bridge that is incurred. This cost is very hard to predict but the convenience to users is a very important aspect to include. Since the FRP deck

requires repairs less often, these user costs can also be cut down. This paper however provides a stronger point by proving the benefits without including the user costs.

5.2.2 Kostuk et al.

The previous example provides a simplified LCCA of a very specific part of a bridge project. Kostuk et al's paper investigates four different bridge deck options and also specifically look at the slab designs of the deck. The specifications of the bridge are provided in appendix E. The 4 design considerations can be seen visually in figure 14 and tabulated in figure 15.

- The membrane in some of the options between the asphalt and the concrete serves as a water proofing layer.
- The MMFX-2 Steel is a type of steel that is corrosion resistant.
- HPC stands for high performance concrete. It is designed to reduce chloride seepage therefore no longer requiring the membrane.
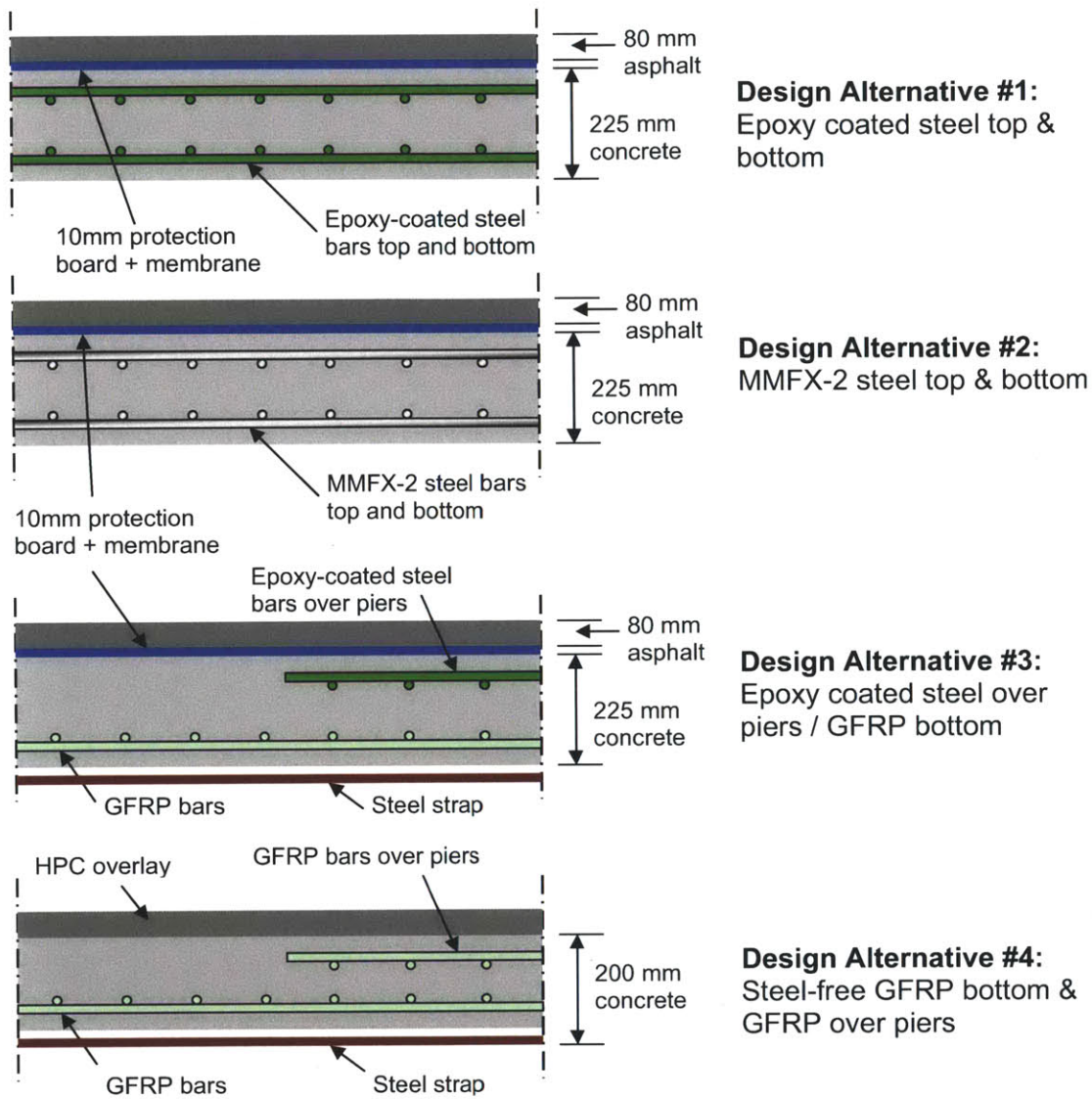
Figure 14: Four bridge deck design alternatives (Kostuk et al.)

| Alternative | Reinforcement | | | | Concrete | | | |
	Over Piers	Top	Bottom	External	Type	Slab Thickness (mm)	Surface Thickness (mm)	Membrane
1	Epoxy	Epoxy	Epoxy	None	HPC	225	Asphalt, 80	Yes
2	MMFX-2	MMFX-2	MMFX-2	None	HPC	225	Asphalt, 80	Yes
3	Epoxy	--	GFRP	Strap	HPC	225	Asphalt, 80	Yes
4*	GFRP	--	GFRP	Strap	HPC	200	HDC, 50	No

* A thinner deck with no membrane requirement since corrosion is not an issue with this configuration.

Figure 15: Four bridge deck design alternatives (Kostuk et al.)

This model looks at significantly more of the probabilistic side of the analysis at each step. However, for the overall LCC, the nominal/mean of the probabilistic results are used with the option of including some of the less likely probabilities. Many of these probabilities are found through Monte-Carlo simulations with inputs from highly experienced structural engineers (Ehlen 224) (Hong 140).

Design Alternative #1

	Low	Nominal	High
Overlay ($/m^2)	15	20	25
Membrane ($/m^2)	35	45	55
Concrete for deck ($/m^3)	700	800	900
Bottom mat ($/m^2)	24.92	27	29.08
Top mat ($/m^2)	24.92	27	29.08
Mat over piers ($/m^2)	48	52	56
Straps ($/m^2)	n/a	n/a	n/a

Figure 16: Unit Costs with 95% confidence intervals

In figure 16 we can see what the probability may look like for the first design alternative and the probabilities of the 95% interval provided by experienced structural engineers. These do not require a Monte Carlo simulation as they are typically well known. It is the predictions with the repairs that perform Monte Carlo simulations. Some of these simulations may be seen in Hong et al.'s paper which is a "Simulation-Based determination of optimal life-cycle cost of FRP bridge deck panels".
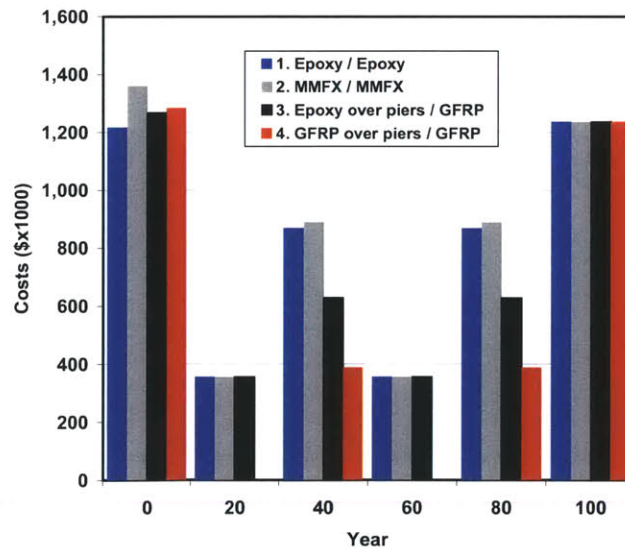

Figure 17: Cash Flows (Kostuk et al.)

28

In Figure 17, we can see the Cash flows of the four different designs and it is clear that the use of GFRP on the top and bottom through the entire life-cycle costing is the most suitable of the alternatives. Again, however the disadvantage of this model is that no user costs are included. Although the authors acknowledge their importance they did not include the operational costs.

5.2.3 Ehlen

Ehlen's paper looks at the "Life cycle of Fiber-Reinforced-Polymer Bridge Decks". These decks are quiet different from FRP reinforced concrete decks and are typically fully composite materials. This paper however follows the same LCCA set up for all previous examples with a new very valuable addition of user costs. The user/operational costs are the costs incurred by the downtime of a bridge and the additional costs the passengers may pay. The costing for the user can be seen below in figure

	Traffic over Bridge		Traffic under Bridge	
Item	Year 1	Last Year	Year 1	Last Year
(1)	(2)	(3)	(4)	(5)
Length of affected roadway (mi) L	1	1	1	1
Average daily traffic (number of cars/day) ADT	30,000	50,000	50,000	80,000
Normal driving speed (mi/h) S_n	45	45	55	55
Driving speed during roadwork (mi/h) S_a	35	35	35	35
Normal accident rate (per million vehicle mi) A_n	1.9	1.9	1.9	1.9
Roadwork accident rate (per million vehicle mi) A_a	2.2	2.2	2.2	2.2
Hourly driver cost (dollars) w	10.73	10.73	10.73	10.73
Hourly vehicle operating cost (dollars) r	8.85	8.85	8.85	8.85
Cost per accident (dollars) c_a	100,000	100,000	100,000	100,000

Figure 18: User Costing

By placing a price on the user costs, a stronger argument can be made for the use of FRP when performing a life-cycle analysis. These costs however are very arbitrary and it can be argued that they are inaccurate as it is very hard to value the costs to the users into a monetary value. It may be best to represent the inconvenience of the user costs using a rating system rather than a monetary value and providing two outputs in the LCCA rather than one monetary output.

6.0 Why FRP is not a common material yet?

Professor Chris Burgoyne from the University of Cambridge and Professor Ionnis Balafas from the University of Cyprus released a paper in 2007 during a conference which provided explanations as to possible reasons for the lack of FRP use in the structural engineering industry. Some of these explanations are displayed below.

The repair and strengthening market has proven to be a financial success. The reason for this is that FRP is lightweight, and will not increase the deadweight significantly when applied making it much lighter than most repairs. It is also easier to handle than many repairs and affects the architecture of a structure much less than its alternatives. This market was unforeseen when FRP first came to the structural engineering applications.

Initially, the manufacturers assumed that the FRP bar industry had not developed and was not a popular option because there were no design codes to support the technology. However even after ACI 440 developed the required codes there was no increase in their use. Burgoyne argues that "codes follow technology, they do not lead it"

A good explanation for this problem was the fact that the FRP industry had not been standardized. This can be seen in the difference of the FRP properties from manufacturer to manufacturer, ie modulus, ultimate strength etc.. Manufacturers are also constantly changing resin, fibers and techniques so there is less trust from the engineers to use this constantly changing product.

Even though codes have been developed and the FRP has been tested long-term properties still maintain quiet a bit of uncertainty. Before the technology is proven, bridge engineers and structural engineers will refrain from the use of FRP for their peace of mind.

Burgoyne et al. also indicate that GFRP which is the cheaper of the FRP choices suffers from a low modulus and will not have a market where deflections are an issue. This suggests that in structures under loading, the concrete is expected to crack prematurely and the cover would have to be increased to prevent this. This would ruin

the purpose of using FRP. Bridge decks and retaining/marine walls where low deflection and high corrosion environment are the markets of target.

Under harsh environmental conditions, concrete itself deteriorates as a result of loading. Many of the papers assume that the reason for the deterioration is the cracks induced by rusting, which accelerates deterioration, however, the concrete may in itself degrade regardless of the materials used in the reinforcement.

Finally, although LCCA is a good reason to argue for the use of FRP, however Burgoyne argues that when a designer designs a bridge, the issues it may inquire with corrosion and spalling may arise 35 years later, when the engineer who designed the bridge may have retired. The increase in the price and arguing for a bid that the owner may not see the benefits for is a very difficult task for the designer and as a result is another reason why the use of FRP is not yet popular.

7.0 Future of FRP

There have been many new developments in the use of FRP. One that is related to FRP bars is the development of a new hybrid FRP bar that contains many different fibers including steel. The composition of this bar can be seen in figure 19 below.
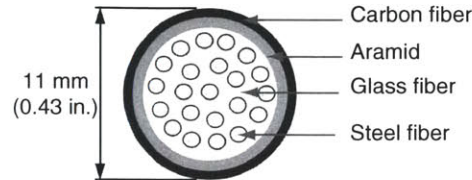
Figure 19: Composition of hybrid FRP bar (Cheung et al. 82)

This Hybrid bar will incorporate the advantages of each of the materials at a cost that is not significantly higher than of GFRP and that will outperform steel rebar in the long run despite the higher initial cost. By employing steel fibers within the resin, the ductility of the steel is incorporated into this new bar. This issue is not addressed for conventional FRP bars. The steel fibers will also be protected from corrosion in the resin and the Carbon and Aramid coatings. The bar also has a high strength close to that of GFRP however still has a modulus that is close to that of steel (Cheung et al.). This hybrid FRP bar appears to incorporate all the advantages of each material into one bar with small trade-offs. There is a good future to this material.
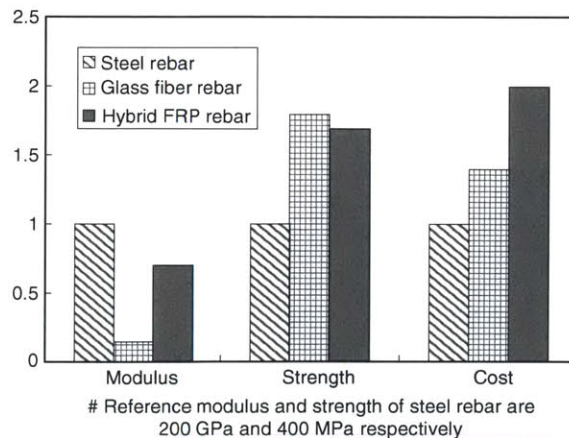
Figure 20: Comparison of Hybrid Bar(Cheung et al. 91)

8.0 Conclusion

It is clear that the material properties and life cycle assessment of FRP has proven to be mostly positive. It is also clear that the most common and the most cost-effective of the three different types of FRP bars to date appears to be GFRP. Although they have not been used extensively in the field, with increasing steel demands worldwide and with booming economies in Asia, it is expected for the price of steel to continue increasing over the years and thus making FRP a more viable option.

The development of the Hybrid FRP bar could also help the FRP bar industry as the bar poses a high modulus making it an advantage where deflection is an issue while at the same time poses a high strength and corrosion resistance. The hybrid FRP also adds ductility to the FRP bars, an issue that engineers feared initially, thus also improving the confidence of the engineer when designing.

With increasing experience in the field and after the current applications of FRP prove to be successful, the use of FRP will increase dramatically. Much of the issue to date has been the uncertainty of the FRP material and while it has proven its properties, it has not proven its long term durability in a real life application.

What many research papers forget to include is the fact that many designers do not like to take the risk to design with a new material. They need to learn to use new codes and are afraid they miss an important aspect in the design. To many designers it is not yet a proven technology and when companies start winning project bids because of their use of FRP is when there will be a push by all companies to become competitive.

The Departments of Transportation across the country have to encourage this change and mind set. DOTs should invest in an experiment to investigate the Life-Cycle Cost of a bridge using a control and an FRP deck funded for their lifetime to evaluate whether FRP should or should not be used. They should also start accepting bids, that although appear to be more expensive, will benefit future generations.

The standardization of FRP bars and other products is also a very important aspect that will help engineers achieve the confidence to design with FRP rather than steel. ASTM standards should be placed on the manufacturing and testing of FRP bars.

The standards should include fixed compositions including choice of resin for different grades of a material. These ASTM standards will make it easier for designer to know more accurately what kind of material they are dealing with.

Finally, a survey with practicing Bridge engineers should be preformed to find out if/why they do/don't design bridge decks with FRP reinforcement. This will create insight to ACI and the manufacturers that will help them make well informed decisions in the future of the FRP market.

With a higher emphasis on sustainability in today's market, especially after observing the infrastructure from the 1950s degrading they way it is, the use of FRP is expected to increase significantly in the years to come.

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Appendix A

(Bisby 9)

Table 2-2. Selected Properties of Typical Currently Available FRP Strengthening Systems*

FRP System	Fibre Type	Weight [g/cm^2]	Thickness [mm]	Tensile Strength [MPa]	Tensile Elastic Modulus [GPa]	Strain at Failure [%]
Fyfe Co. LLC [www.fyfeco.com]						
Tyfo SEH-51	Glass	930	1.3	575	26.1	2.2
Tyfo SCH-35	Carbon	--	0.89	991	78.6	1.3
Mitsubishi [www.mitsubishichemical.com]						
Replark 20	Carbon	200	0.11	3400	230	1.5
Replark 30	Carbon	300	0.17	3400	230	1.5
Replark MM	Carbon	--	0.17	2900	390	0.7
Replark HM	Carbon	200	0.14	1900	640	0.3
Sika [www.sikacanada.com]						
Hex 100G	Glass	913	1.0	600	26.1	2.2
Hex 103C	Carbon	618	1.0	960	73.1	1.3
CarboDur S	Carbon	2240	1.2-1.4	2800	165	1.7
CarboDur M	Carbon	2240	1.2	2400	210	1.2
CarboDur H	Carbon	2240	1.2	1300	300	0.5
Watson Bowman Acme [www.wabocorp.com]						
MBrace EG 900	Glass	900	0.35	1517	72.4	2.1
MBrace CF 530	Carbon	300	0.17	3500	373	0.94
MBrace AK 60	Aramid	600	0.28	2000	120	1.6

* Additional information can be obtained from the specific FRP manufacturers

Appendix B

Temperature effects of FRP (Robert 357)

Table 2. Experimental Results of Mechanical Tests

Temperature (°C)	Number of test samples	Tensile strength (MPa)	COV (%)	Shear strength (MPa)	COV (%)	Flexural strength (MPa)	COV (%)
−100	5	897	3.7	304	1.8	1843	2.4
−80	5	838	3.5	281	5.1	1750	2.2
−60	5	785	2.0	255	2.6	1516	2.9
−40	5	791	2.2	227	3.0	1325	2.3
−20	5	784	2.4	219	5.9	1288	1.4
0	5	754	3.2	211	2.6	1101	0.3
25	5	756	1.8	199	3.8	1093	2.1
50	5	757	3.5	198	2.5	1088	4.5
100	5	674	2.7	177	2.6	922	3.1
150	5	532	6.6	176	2.8	255	5.1
200	5	513	7.4	114	2.1	142	8.1
250	5	464	2.9	56	14.8	106	5.7
300	5	405	2.8	50	7.6	74	6.8
325	5	353	3.2	44	7.0	65	4.2

Appendix C
Example of FRP bar costs in the market

Aslan FRP

Hughes Brothers
210 N. 13th Street Seward, NE 68434
Ph: 402-643-2991 Fax: 402-643-2149
www.hughesbros.com

Distributor Price Schedule ~ Effective January 1, 2011

Aslan 100 Vinyl ester matrix GFRP Rebar
Aslan 101* Polyester matrix GFRP Rebar
* For non-portland cement and temporary use applications. Available only upon request and appropriate lead-time.

Straight bar is priced by the lineal foot as follows:

Bar Diameter	Published List Price/ft	Unit Weight/ft
2(6mm)	$0.33	0.052
3(10mm)	$0.47	0.107
4(13mm)	$0.68	0.189
5(16mm)	$1.10	0.287
6(19mm)	$1.45	0.408
7(22mm)	$1.76	0.544
8(25mm)	$2.46	0.730
9(29mm)	$3.00	0.950
10(32mm)	$3.67	1.150

Bends:
For quantities less than 300 use $1.50/bend plus straight bar price
For quantities in excess of 300 bends, use $1.25/bend plus straight bar price

Aslan 200/201 CFRP Rebar
For use as Rebar, Tendons and NSM Strengthening of existing concrete, masonry and wood. Aslan 200 is textured surface. Aslan 201 is sand coated.

Bar Diameter	Published List Price/ft	Unit Weight/ft-(lbs)
2(6mm)	$4.40	0.035
3(10mm)	$6.25	0.075
4(13mm)	$8.82	0.125

Notes:
#2 & #3 available in any length up to 250ft at the above per foot rate.
#4 bar stocked in 20ft lengths. Selling price based on utilizing the 20ft stick or minimum 10ft length. (For example, 18ft price is same as 20ft stick.)

Typical Data sheet from an FRP rebar provider

Aslan 100 Glass Fiber Reinforced Polymer (GFRP) Rebar
Product Data Sheet

Physical / Mechanical Properties – Tensile, Modulus & Strain

Nominal Diameter			Nominal Area		f^*_{fu} - Guaranteed Tensile Strength		Ultimate Tensile Load		E_f - Tensile Modulus of Elasticity		Ultimate Strain
Size	mm	in	mm^2	in^2	MPa	ksi	kN	kips	GPa	psi 10^6	%
2	6	¼	31.67	0.049	896	130	28.34	6.37	46	6.7	1.94%
3	10	⅜	71.26	0.110	827	120	58.72	13.20	46	6.7	1.79%
4	13	½	126.7	0.196	758	110	95.90	21.56	46	6.7	1.64%
5	16	⅝	197.9	0.307	724	105	143.41	32.24	46	6.7	1.57%
6	19	¾	285.0	0.442	690	100	196.60	44.20	46	6.7	1.49%
7	22	⅞	387.9	0.601	655	95	254.00	57.10	46	6.7	1.42%
8	25	1	506.7	0.785	620	90	314.27	70.65	46	6.7	1.34%
9	29	1-⅛	641.3	0.994	586	85	375.83	84.49	46	6.7	1.27%
10	32	1-¼	791.7	1.227	551	80	436.60	98.16	46	6.7	1.19%
11*	35	1-⅜	958.1	1.485	482	70	462.40	104*	46	6.7	1.04%
12*	38	1-½	1160	1.800	448	65	520.40	117*	46	6.7	0.97%
13*	41	1-⅝	1338	2.074	413	60	553.50	124*	46	6.7	0.90%

* Tensile properties of #11, #12 & #13 bar are NOT guaranteed due to the inability to achieve a valid bar break per ASTM D7205.

Hughes Brothers reserves the right to make improvements in the product and/or process which may result in benefits or changes to some physical-mechanical characteristics. The data contained herein is considered representative of current production and is believed to be reliable and to represent the best available characterization of the product as of July 2011. Tensile tests per ASTM D7205.

Design Tensile & Modulus Properties ..... per ASTM D7205-06. The area used in calculating the tensile strength is the nominal cross sectional area. The "Guaranteed Tensile Strength", f^*_{fu} is as defined by ACI 440.1R as the mean tensile strength of a given production lot, minus three times the standard deviation or $f^*_{fu} = f_{u,ave} - 3\sigma$. The "Design or Guaranteed Modulus of Elasticity is as defined by ACI 440.1R as the mean modulus of a production lot or $E_f = E_{f,ave}$.

Material Certs & Traceability ..... Available for any production lot of Aslan 100 bar, traceable by bar marks imprinted on the bar in intervals showing the bar diameter, stock order and production date.

Cross Sectional Area Tolerance .....   – 0% / + 20%

Design properties are determined using "Nominal" diameters and equivalent calculated cross sectional areas. Surface undulations and sand coatings that facilitate bond are accommodated for in ASTM D7205, section 11.2.5, with a tolerance of minus zero, plus 20% as determined by the Archimedes method of volume displacement in a fluid.

Bond Depended Coefficient .... $k_b = 0.9$
per ASTM draft test method. As used in ACI equation 8-9.

Glass Fiber Content .... > 70% by weight per ASTM D2584

Transverse Shear Strength .... > 22,000 psi (150MPa)
per ASTM D7617 & ACI 440.3R method B.4

Void Content .... No Continuous Voids after 15 minutes of capillary action, per ASTM D5117

Moisture Absorption .... 24 hour absorption at 122°F (50°C) ≤ 0.25%, per ASTM D570

Density

Diameter			Unit Weight / length	
Size	mm	in	kg / m	lbs / ft
2	6	¼	0.0774	0.052
3	10	⅜	0.159	0.107
4	13	½	0.2813	0.189
5	16	⅝	0.4271	0.287
6	19	¾	0.6072	0.408
7	22	⅞	0.8096	0.544
8	25	1	1.0462	0.730
9	29	1-⅛	1.4137	0.950
10	32	1-¼	1.7114	1.15
11	35	1-⅜	1.9346	1.30
12	38	1-½	2.4554	1.65
13	41	1-⅝	2.8721	1.93

Aslan FRP

November 10, 2011

Bent Bars & Stirrups

- Must be made at the factory, field bending not permitted.
- Industry standard bent shapes are available, standard shape codes are used.

Some limitations include:
- Max leg length of a stirrup is 60" (152cm)
- Redirection of bends, such as Z-shapes or gull-wings types are not very economical. Bent shapes should continue in the same circular direction.
- Closed square shapes are best furnished as pairs of U-bars or continuous spirals.
- A 90-degree bend with $12d_b$, bar diameter, pigtail used to shorten development length is equally as effective as a J-shape as per ACI 440.1R.
- The radius on all bends is fixed as per the table shown. Some U-shaped stirrups fall in between the range of these two bend radiuses and are not possible.

We advise that you work closely with the factory to implement the most economical detailing of bent bars and stirrups.

Bend Radius

Diameter			Inside Bend Radius	
Size	mm	in	mm	in
2	6	¼	38	1.5
3	10	⅜	54	2.125
4	13	½	54	2.125
5	16	⅝	57	2.25
6	19	¾	57	2.25
7	22	⅞	76	3.0
8	25	1	76	3.0

Field Forming of Large Radius Curves

Permitted when the radius is larger than in the following table. The table gives the minimum allowable radius for induced bending stresses without any consideration for additional sustained structural loads.

Diameter			Interior Use $C_e = 0.8$ Min Radius		Exterior Use $C_e = 0.7$ Min Radius	
Size	mm	in	cm	in	cm	in
2	6	¼	107	42	122	48
3	10	⅜	170	67	196	77
4	13	½	246	97	282	111
5	16	⅝	323	127	368	145
6	19	¾	404	159	462	182
7	22	⅞	495	195	566	223
8	25	1	597	235	678	267
9	29	1-⅛	711	280	813	320
10	32	1-¼	871	343	996	392
11	35	1-⅜	1052	414	1204	474
12	38	1-½	1237	487	1412	556
13	41	1-⅝	1448	570	1656	652

Strength of the Bent Portion of the Bar

.... > 50% strength of the straight length of the bar , per ACI 440.3R method B.5

Characteristic Properties - Characteristic Properties are those that are inherent to the FRP bar and not necessarily measured or quantified from production lot to production lot.

Durability – Alkali Resistance ~ without load

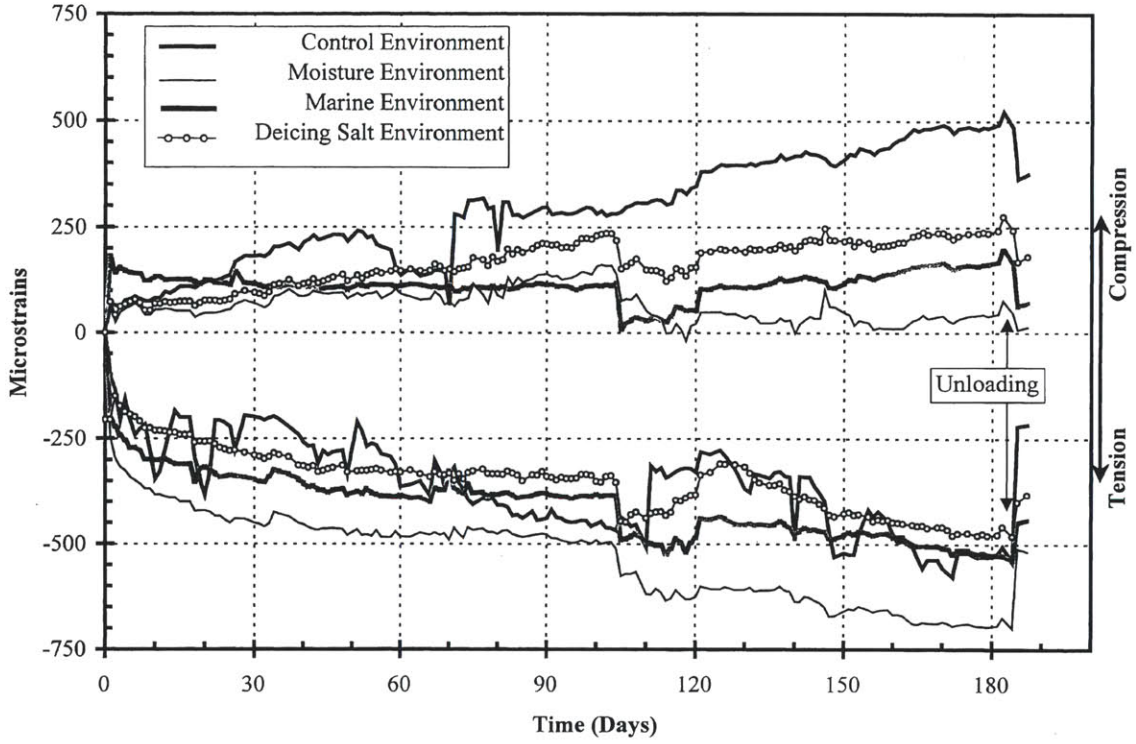.... > 80% strength retention, when exposed to 12.8pH solution for 90 days at 140°F (60°C)

Tensile Strength at Cold Temperature .... < 5% strength reduction from ambient at -40°F (-40°C), per ASTM D7205.

Transition Temperature of Resin - T_g .... > 230°F (110°C) per DSC method

Handling and Placement

- Follow guidelines in ACI440.5-08 "Specification for Construction with FRP Bars".
- In general, field handling and placement is the same as for epoxy or galvanized steel bars.
- Do NOT shear FRP bars. When field cutting of FRP bars is necessary, use a fine blade saw, grinder, carborundum or diamond blade.
- Sealing the ends of FRP bars is not necessary.
- Support chairs are required at two-thirds the spacing of steel rebar.
- Plastic coated tie wire is the preferred option for most projects. When completely non-ferrous reinforcing, i.e., no steel is required in the concrete, nylon zip ties (available from local building materials centers) or plastic bar clips are recommended. (Don't forget to use non-metallic form ties in formwork.)
- It is possible, especially in precast applications, for GFRP bars to "float" during vibrating. Care should be exercised to adequately secure GFRP in the formwork.

Aslan FRP

Appendix D

The effects of different environments on creep of FRP bars (Singhvi et al. 357)

Appendix E
Kostuk et al.: Bridge specifications
- Setting: Urban Area
- Traffic: 20,000 vehicles/day
- Bridge
  - 2 lanes
  - 7 spans @ 43.5m each
  - 6 piers – made continuous over piers
  - Total length = 304.5m
  - Deck Width = 11.64m
  - Roadway Width = 10.4m

Costing assumptions:
- Planning Horizon
- Discount Rate = 5%
- Costing Categories
  - Agency Costs
  - User Costs NOT included