Application of GFRP Rebar in Slabs on Grade and CRCP

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Abstract:

Steel reinforcement of concrete has been used for over 150 years to create lighter, more cost effective concrete structures. While engineers are familiar with the design of structures using steel reinforcement from its many years of usage and extensive research into the composite action of steel and concrete, it is far from a perfect solution. Steel's primary shortcomings are that it is easily corroded, is heavy, and its use in concrete has a relatively high, negative environmental impact. Recently, there has been a significant amount of interest in using non-corroding, lightweight GFRP rebar to overcome the issue of corrosion and thus lengthen the life of structures using it, which also reduces the environmental impact.

Most civil engineers are with the properties of steel reinforced concrete, but not with the properties of GFRP reinforcement. One of the main differences in properties between steel and GFRP is the relatively low modulus of elasticity of GFRP rebar. This difference is often misunderstood as a detrimental property.

This paper addresses the issue of whether the lower modulus of elasticity of GFRP rebar is a benefit as it relates to its use in slabs on grade, such as floors, parking lots, roads and highways. Other research is reviewed and synthesized to conclude that the lower modulus of elasticity of GFRP is positive as it relates to its use in slabs on grade.

Design guidance from published documents is summarized for both slabs on grade and CRCP.

Keywords: Modulus of elasticity, reinforced concrete, FRP, GFRP, BFRP, CRCP, slabs on grade, temperature and shrinkage reinforcement.

OVERVIEW

The use of fiber reinforced polymer (FRP) rebar, particularly those using glass and basalt fiber, is rapidly becoming more common. These rebar are cost effective, corrosion resistant, lightweight, very strong and environmentally beneficial, as well as having other positive attributes.

This paper is a general discussion of the benefits of the use of glass or basalt fiber reinforced polymer (GFRP or BFRP) rebar in non-structural slabs on grade such as roadways, parking lots, and floors. Since most practicing civil engineers do not have experience with the use of FRP rebar, and its use is rapidly increasing, this paper also addresses the design considerations for their use in these applications. A particular emphasis is placed on the benefits of their having a lower modulus of elasticity than steel, which to many may seem counterintuitive.

Slabs on grade can take many forms such as plain concrete slabs with temperature and shrinkage reinforcement, jointed reinforced concrete pavement, continuously reinforced concrete pavement, and structurally reinforced concrete slabs.

FRP rebar consists of bundles of continuous, longitudinal fibers with high tensile strength bonded together by a resin into a bar shape analogous to a steel reinforcing bar. The fibers are typically glass, aramid, basalt or carbon. Most commonly glass fiber is used, to form GFRP rebar. The most common resins are epoxy and vinyl ester. Dicyclopentadiene (DCPD) is a less commonly used, but is a high performing, resin. Polyester has also been used;

but due to its lower resistance to chemical attack, lower bond strength and hazardous styrene off-gassing (a problem it shares with vinyl ester), it is not commonly used. GFRP is a standardized product for which ASTM 7957 and ACI 440.6-08 provide specifications defining minimum acceptable physical and chemical characteristics for use in structural and non-structural applications. This paper primarily addresses the use of GFRP rebar meeting these standards.

The use of fiber reinforced polymer (FRP) rebar has been widely studied over the last 40 years. The impetus for this has been the expensive, frequent repairs needed for steel reinforced concrete structures, particularly those exposed to corrosion and cyclic loadings. Steel corrosion products, mainly iron oxides, occupy up to seven times the volume of the steel, expanding within the concrete and resulting in tensile failure of the concrete. Also the effective cross section may be significantly reduced by corrosion. Both of these conditions will lead to the need to rehabilitate or replace the structure. The studies of non-corrodable FRP rebar, mainly of GFRP rebar, have shown that their use can increase structure life by up to four times in aggressive environments and reduce maintenance (Feldman et al, (2008)). This increase in structure life leads to reduced life-cycle costs. Eamon, etal, (2012) reported that for bridge superstructures, even the more expensive carbon fiber reinforced polymer bar (CFRP), is usually lower cost than steel on a life cycle basis.

The primary reasons for its limited use have been the historically higher initial cost and the lack of engineering design standards. A Michigan company, Neuvokas Corporation, has made advances in high speed manufacturing allowing GFRP rebar to be made at costs competitive with plain steel and at a lower cost than epoxy coated or corrosion resistant steel. Industry standard setting organizations, such as the American Concrete Institute (ACI), the American Association of State Highway and Transportation Officials (AASHTO), the Canadian Standards Association (CSA), have all developed design guidelines for the use of FRP rebar.

With this reduction in price and the availability of design standards, the industry is at an inflection point regarding wider adoption of this beneficial product.

BENEFITS

Comparing GFRP to steel the following attributes are noted.

1. Competitive with steel on first cost basis
2. Lightweight, approximately 30% of steel, resulting in increased worker safety, lower transportation cost
3. Lower installation cost, with contractors reporting 20% less labor and equipment cost due to lower weight
4. Corrosion resistant, resulting in increased structure life (2 to 4 times)
5. Higher tensile strength, approximately 2.5 X steel
6. Electromagnetic transparency, MRI rooms, toll plazas, substation foundations, cattle barns, (stray currents)
7. Low overall embodied energy of concrete reinforced with GFRP, (approximately 25% to 50% of embodied energy in steel reinforced concrete exposed to corrosive elements) (Ozcoban (2017))
8. Lower CO2 emissions, approximately 50% (Ozcoban (2017), Inmana (2016))
9. Lower life cycle cost for structures exposed to corrosive elements
10. Lower modulus of elasticity, reducing internal stress on concrete
11. Coefficient of thermal expansion nearly the same as concrete, reducing internal stress on concrete.

REVIEW, ANALYSIS, AND SYNTHESIS OF RESEARCH AND DESIGN GUIDANCE BY OTHERS

GFRP rebar has unique physical and mechanical properties that must be taken into consideration during design. It is not a direct, one for one, replacement for steel reinforcement. The American Concrete Institute Committee 440 has developed and published design guidelines for the use of GFRP reinforcement that reflects GFRP's unique properties (ACI 440.1R (2015)). AASHTO has done the same for bridges (AASHTO (2018)).

Since GFRP is anisotropic, designs incorporating its use must consider its properties in both the longitudinal and transverse directions. It also does not yield like steel, but is elastic to failure. From a longitudinal tensile strength perspective GFRP rebar is much stronger than steel at approximately 2.5 times that of 60 ksi steel. However, its modulus of elasticity (Young's modulus) is much lower (6,000 ksi v. 29,000 ksi), as is its transverse shear strength (22 ksi v. 45 ksi) than steel.

Slabs on grade are generally reinforced for serviceability reasons and sometimes for structural reasons.

The primary serviceability concern for slabs on grade is controlling the size and spacing of cracks. As concrete cures it shrinks. Cracks result when the tensile stress from drying exceeds the tensile strength of the concrete. Tensile stresses may also occur as a result of temperature changes. In both cases when the negligible tensile strength of the concrete is exceeded, the tensile stress is then transferred to the reinforcement and cracking occurs. The frequency/spacing of cracks, the internal concrete stresses (thermal and contraction) and the modulus of elasticity of the reinforcement determine the width of those cracks. Maintaining those cracks at a small enough width to allow aggregate interlock to be preserved, and thus mechanical load to be transferred across the crack, as well as to minimize water intrusion and spalling, is the primary design consideration for slabs on grade. Crack spacing is important to prevent punchout failure resulting from closely spaced cracks in slabs subjected to loads capable of causing this type of failure.

As an example, the AASHTO guideline for crack width in continuously reinforced concrete pavements is less than 0.04 inches (1 mm) and the minimum crack spacing is 3.5 feet (1.07 m). The maximum crack width is largely established to minimize water intrusion and spalling.

Vetter (1933) recognized the importance of internal stresses in concrete resulting from volume changes, quantified them, and discussed the amount of steel reinforcement needed to resist them to minimize or prevent cracking. The amount of steel required to prevent cracking was "relatively high, and in many cases, …prohibitive." Choi and Chen (2005) evaluated the concrete stresses resulting from temperature and shrinkage in continuously reinforced concrete pavements using GFRP reinforcement. They found that the lower Young's modulus of GFRP when compared to steel led to lower stress levels in the concrete and greater crack spacing.

Chen, Choi, GangaRao and Kopac (2008) reported on a CRCP test in West Virginia where a section of highway was paved with steel reinforcement at mid-depth of a 10 inch roadway and a section was paved with GFRP also at mid-depth. The result was that the GFRP reinforced section had significantly greater spacing between cracks, theorized to be the result of the low modulus of elasticity of the reinforcement. The crack widths were found to be somewhat greater in the GFRP section than the steel section, but the crack widths were within the AASHTO guidelines. The longitudinal reinforcement ratios were 0.7 percent for steel and 1.12 percent for GFRP. Thebeau, Eisa and Benmokrane (2008) conducted a similar study in Quebec and found that crack widths for GFRP sections reinforced with the same 0.77 percent used for steel had crack widths within AASHTO guidelines.

CRACK CONTROL

Cracks can be allowed to develop naturally/passively or actively controlled. In CRCP applications, it has been found that if allowed to propagate passively the crack pattern is commonly one in which there are widely spaced clusters of tightly spaced cracks (Benmokrane et al (2020), and Ren et al (2014)). These tightly spaced clusters as well as random, non-uniform cracks can lead to failures.

In order to actively control cracking in CRCP, active measures may be taken. In jointed plain concrete pavement as well as jointed reinforced concrete pavement active crack control has been considered good practice for many years (McCullough and Dossey (1999)). Kohler and Roesler (2004), discussed active crack control for CRCP applications and demonstrated the ability to reduce clustered cracks as well as non-uniform cracks using active crack control. Active crack control has taken the form of transverse, partial depth, vertical tape, or metal strip, insertion to create a stress concentration or partial depth sawcutting. However, these methods make construction more difficult, reduce the load transfer efficiency by decreasing the contact area, and reduce ride quality through spalling.

Ren et al (2014) noted these shortcomings and observed that CRCP sections without active crack control developed controllable cracks at points of increased stress induced by adjacent pavement. From this observation, it was deduced that a small sawcut at the pavement edge could achieve controlled cracking. A portion of a CRCP highway in Belgium was constructed in 2012 with 400 mm (15 ¾ inch) long, transverse, sawcuts at the pavement edge with some being 30 mm (approximately 1 1/8 inch) deep and others 60 mm (approximately 2 ¼ inch) deep. They were spaced at 1.2 meters (4 feet). The conclusion was that the deeper cuts, made early in the curing of the pavement, resulted in significantly reduced crack clusters and non-uniform cracking and substantially increased the occurrence of uniformly spaced, straight cracks. The resulting cracks were also smaller than those in a passively cracked section.

Cyclic fatigue is not often a consideration in well supported slabs on grade, but it is worth noting that the use of GFRP can also be a benefit in this regard. Mufti and Neale (2007) looked at a bridge deck slab reinforced with GFRP and found that it could withstand over 20 times the cyclic fatigue of steel as a result of its lower modulus of elasticity.

Katz (2004) found that an additional benefit of FRP rebar usage is its much lower environmental impact load. This is a result of the lower environmental impact in the reinforcement manufacturing and transportation process, reduced maintenance activities, and the reduced impact of disposal. The reduced maintenance is in part the result of increased life because of the lower modulus of elasticity that increases the number of cycles resulting in cyclic fatigue failure, as well as the elimination of corrosion. Hammond and Jones (2011) quantified the embodied energy and CO2 of common construction materials.

APPLICATIONS

SLABS ON GRADE

Many jurisdictions have empirical standards for reinforcement of jointed concrete pavement or slabs used for parking, etc. that are simple prescriptions of a certain size rebar placed at a prescribed depth at a certain spacing in a pavement of a given thickness.

As an example, Harris County, Texas (Houston area) requires #4 bars at 18 inches on center for a 7 inch concrete pavement section. This is approximately equivalent to 0.13 square inches of reinforcement per foot of concrete or 0.156%. The reinforcement is to be placed at mid-depth and contraction joints installed at 20 foot spacing with expansion joints at 80 foot spacing. Thus, the reinforcement provides negligible structural strength, and the pavement is designed more like a plain jointed concrete pavement with larger than "normal" contraction joint spacing ("normal" is less than twenty four times the thickness of the concrete or 14 feet in this example). The reinforcement's apparent purpose is to restrain cracking and to maintain slab integrity and is considerably less than the commonly used minimum 0.6 percent ratio for CRCP.

When considering what amount of GFRP reinforcement would yield a similar result to the empirically prescribed amount of steel in the above example, the question to be answered is what is the general purpose of the reinforcement and how much GFRP can result in a similar or acceptable design. In this case it is maintaining sufficiently tight cracks that load transfer across the crack is not affected by the lack of aggregate interlock and that corrosive deicing or saltwater does not wash through the slab, corroding the steel. AASHTO standards, based on field studies, indicate that a 0.04 inch crack is acceptable for these purposes.

The Portland Cement Association uses the drag equation to estimate the area of steel in slabs on grade based on the allowable stress in the reinforcement as follows:

$$A_s = \frac{\mu L w_{slab}}{2 f_{s,allow}}$$

where A_s is the cross-sectional area of steel per linear foot in in.^{2}; $f_{s,allow}$ is the allowable stress in steel reinforcement in psi, commonly taken as two thirds to three-fourths of f_y; μ is the coefficient of subgrade friction (1.5 is recommended for slabs on ground (Portland Cement Association (1990)); L is the distance between joints in feet; and w is the dead weight of the slab in lb/ft2, usually assumed to be 12.5 lb/ft^2 per in. of slab thickness. The stress in the steel in psi, f_s, is therefore inversely proportional to the area of steel.

Rearranging this equation to solve for the stress in the reinforcement:

$$f_s = \frac{\mu\, L\, w}{2\, A_s}$$

Solving for the stress in the steel using the area of steel in the Harris County design, the stress in the steel would be 10,100 psi, resulting in a strain of 0.00035.

If the steel is replaced with #4 FRP, with a modulus of elasticity of 6,000 ksi, the strain in the FRP would be 0.0017 at this same stress. Even using a #3 FRP bar in place of the steel would only increase the stress to 17,700 psi and the strain to 0.003.

ACI 440.1R equation 7.3.1a calculates the maximum spacing of GFRP reinforcement resulting in a target maximum allowable crack width:

$$s_{max} = 1.15\,\frac{E_f\, w}{f_{fs} k_b} - 2.5\, c_c \le 0.92\,\frac{E_f\, w}{f_{fs}\, k_b}$$

Where s_{max} is bar spacing in inches, E_f is modulus in psi, w is maximum allowable crack width in inches, k_b is the bond dependent coefficient (1 for steel and 0.8 for Neuvokas GatorBar Glass), c_c is the clear cover of the reinforcement in inches, and f_{fs} is the service level stress in the reinforcement in psi. Rearranging this formula to solve for crack width yields the following:

$$w = \left(\frac{s_{max} + 2.5\, c_c}{1.15\, E_f}\right)\left(f_{fs} k_b\right)$$

Using this formula the crack width may be estimated from the calculated stress in the slab. For this example it is assumed that k_b is equal to 1. This results in a crack width of just under the AASHTO guideline of 0.04 inches of crack width using #4 GFRP. The tension on the bar is approximately 2,000 pounds, and the bond stress at failure of 2,500 psi requires just over ½ inch of embedment.

For #3 bar the crack width is approximately 0.065 inches. Checking this stress, 17,700 psi, against the ultimate bond stress of #3 GFRP bar, the tension on the #3 bar would be just less than 2,000 pounds and the bar has a bond stress at failure of approximately 2,500 psi, or nearly 3,000 pounds per inch of embedment length.

ACI 440.1R-15 Appendix A – SLABS-ON-GROUND states, "Because of the lower modulus of the FRP reinforcement, the governing equation should be based on the strain rather than the stress level when designing shrinkage and temperature FRP reinforcement. At the allowable stress, the strain in steel reinforcement is approximately 0.0012; implementing the same strain for FRP will result in a stress of 0.0012 *Ef*, and" the above equation "can be written as

$$A_{f,sh} = \frac{\mu L w}{2\left(0.0012\, E_f\right)}$$

And "…can also be used to determine joint spacing L for a set amount of reinforcement. No experimental data have been reported on FRP slab-on-ground applications; research is required to validate this approach."

Using the allowable stress in GFRP as 0.0012 E_f, the allowable stress is 7,200 psi. In the above example this would reduce the #4 bar spacing from 18 inches to approximately 12 inches with all other factors constant. Contraction joints more closely spaced could also offset the reduction in spacing, as could changing the coefficient of subgrade friction.

In general it may be said that slabs-on-grade using GFRP will require modestly more reinforcement cross sectional area than steel to achieve the same crack width, but that they will have a longer life as a result of corrosion being eliminated.

CONTINUOUSLY REINFORCED CONCRETE PAVEMENT

CRCP is commonly used for high traffic, heavily loaded highways. It is continuous rigid pavement with longitudinal reinforcement to control cracking and is constructed without expansion or contraction joints. It provides a smooth ride quality over a longer period of time (20-30 years) than other rigid pavement types. This extends the life of the pavement, which reduces the life cycle cost and environmental impacts.

Steel reinforcement in CRCP has traditionally been used, with the first use in about 1920. Today CRCP is typically designed in accordance with AASHTO guidelines (AASHTO 1993). Nearly 70 years of experience with CRCP and extensive research and testing has been used to arrive at these guidelines. However, steel corrosion, particularly in areas using deicing salts or subject to saltwater, diminishes the longevity of CRCP and is a major problem for steel reinforced CRCP (Kim et al (2000(, Choi and Chen (2005)).

The use of FRP reinforcement eliminates the corrosion issue. Walton and Bradberry (2005) using finite element analysis theorized that the use of GFRP could result in better performance than steel. This study and others led to the construction of full scale test sections of GFRP reinforced CRCP in 2007 in West Virginia (Choi and Chen 2008) and in 2013 in the Province of Quebec (Benmokrane et al (2020)). The Quebec section was designed after several years of experimentation with smaller sections (Benmokrane (2008)) and the application of steel CRCP design criteria modified for the unique physical characteristics of GFRP (AASHTO (1993), USDOT (1996), Choi and Chen (2003)) to guide the selection of reinforcement ratios and placement in the test sections.

The theoretical longitudinal reinforcement ratio was calculated by both teams (Quebec and West Virginia) to be approximately 1%. Benmokrane (2008) tested short (22 meter) sections with reinforcement ratios varying from 0.77% to 1.57% with varying bar sizes, spacing and layers before deciding to construct the full scale test section with a reinforcement ratio of 0.93%, using #8 bars in a single layer spaced at approximately 6 ¾ inches at mid-depth in a 12 ½ inch slab. In the West Virginia section a 1% reinforcement ratio was achieved in a 10 inch slab using #7 bars spaced at 6 inches at mid-depth.

Both the West Virginia and Quebec test sections were constructed alongside conventionally designed steel reinforced sections with the same subbase, subgrade and concrete.

After 6 years of use, Benmokrane et al (2020) reported on the longer term results of the Quebec test section. They found that both the steel and GFRP sections had similar crack widths and in both cases less than the AASHTO standard. The crack spacing consisted of widely spaced groups of cracks. The test section was deemed to be a successful demonstration of GFRP CRCP. The West Virginia test section yielded similar good results with the crack spacing in the GFRP section approximately twice those of the steel reinforced section. In both sections the average crack widths were less than AASHTO standard. The reduced internal stresses related to the lower modulus of elasticity of GFRP were believed to result in reduced spalling and susceptibility to punchout failure.

Strain gauge test results in the Quebec section showed that the GFRP and concrete had similar strain responses to temperature changes.

Benmokrane et al (2020), using Huang's (2003) design formulae for CRCP, based on the AASHTO (1993) method of calculating reinforcement, modified those formulae to reflect the lower modulus of GFRP. A coefficient of $(200/E_{GFRP})^{0.15}$ was proposed to address the difference in modulus based on regression analysis of test section results (200 gPa being the modulus of steel). The maximum bar stress was limited to 35% of guaranteed maximum tensile stress to avoid large crack widths. The following formulae resulted:

$$P_{max,GFRP} = \left(\frac{200}{E_{GFRP}}\right)^{0.15} \left\{ \frac{1.062\left(1+\frac{f_t}{6.894}\right)^{1.457}\left(1+\frac{\alpha_{GFRP}}{2\alpha_c}^{0.25}\right)(1+0.04\varphi)^{0.476}}{(1.294\,X_{min})^{0.217}\left(1+\frac{\sigma_w}{6.894}\right)^{1.13}(1+1000Z)^{0.398}} - 1 \right\}$$

$$P_{min1,GFRP} = \left(\frac{200}{E_{GFRP}}\right)^{0.15} \left\{ \frac{1.062\left(1+\frac{f_t}{6.894}\right)^{1.457}\left(1+\frac{\alpha_{GFRP}}{2\alpha_c}^{0.25}\right)(1+0.04\varphi)^{0.476}}{(1.294\,X_{max})^{0.217}\left(1+\frac{\sigma_w}{6.894}\right)^{1.13}(1+1000Z)^{0.398}} - 1 \right\}$$

$$P_{min2,GFRP} = \left(\frac{200}{E_{GFRP}}\right)^{0.15} \left\{ \frac{0.358\left(1+\frac{f_t}{6.894}\right)^{1.435}(1+0.04\varphi)^{0.484}}{(0.04\,CW)^{0.22}\left(1+\frac{\sigma_w}{6.894}\right)^{1.079}} - 1 \right\}$$

$$P_{min3,GFRP} = \left\{ \frac{50.834\left(1+\frac{1.8\,DT_D+32}{100}\right)^{0.155}\left(1+\frac{f_t}{6.894}\right)^{1.493}}{(145\,\sigma_{GFRP})^{0.365}\left(1+\frac{\sigma_w}{6.894}\right)^{1.146}(1+1000Z)^{0.18}} - 1 \right\}$$

Where P is the longitudinal reinforcement ratio in %, f_t is concrete tensile strength in mPa, α is the coefficient of thermal expansion °C^{-1}, φ is the bar diameter in mm, σ_w is the wheel load stress in mPa, X is mean crack spacing in m, DT_D is design temperature drop in °C, CW is crack width in mm, σ is the allowable stress in the reinforcement and Z is concrete shrinkage in 28 days in mm/mm. The allowable stress in GFRP rebar was set at 35% of the guaranteed maximum tensile strength to maintain crack width at less than the AASHTO standard of 1 mm (based on field test observations) and the crack width in the formulae was set at 1 mm to reflect this. Using these calculations the reinforcement ratio should be set above the highest minimum ratio calculated and below the maximum.

SUMMARY

The benefits of using GFRP rebar in concrete slabs on grade are significant, and are many, including:

- Lower internal stresses in the concrete during shrinkage and temperature change
- No corrosion, increasing life of structure and no bleed through
- Increase in crack width spacing
- Reduced life cycle costs and in many cases reduced installed cost
- Significantly reduced environmental impacts
- Lower weight, reducing installation cost and increasing worker safety
- Electromagnetic transparency

These benefits combined with the recently lowered cost of GFRP rebar, and the availability of design information based on field testing is resulting in wider adoption of its use.

For engineers to use this product in slabs on grade, they must understand the differences in its properties from the steel that has traditionally been used. For slabs on grade the primary difference is GFRP's lower modulus of elasticity. Properly designed, the lower modulus of elasticity is an advantage over steel, since it is closer to the modulus of concrete. Through field experience and research, engineering design tools have been developed to allow the design of slabs on grade to meet varying conditions using GFRP rebar. This paper provides an introduction to some of these design tools.

It is also important to note that very large environmental benefits can be achieved by wider use of GFRP rebar. The World Business Council for Sustainable Development (2002) estimated that 8% of world CO_2 emissions were the result of cement production. Doubling the life of reinforced concrete structures could significantly reduce worldwide CO_2 emissions.

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