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Guide Examples for Design of Concrete Reinforced with FRP Bars

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Synopsis: This paper presents design examples that illustrate the interaction of design parameters, to examine some of the more critical issues and challenges that arise when designing FRP-RC to satisfy ultimate strength and serviceability criteria using the ACI 440.1R-03 *Guide*. A spreadsheet program was written for flexural and serviceability analysis and design of concrete sections reinforced with a single layer of glass or carbon FRP bars. Analysis and design examples were developed and design aids were constructed to assist in economically and efficiently sizing FRP-reinforced concrete members. Potential difficulties that arise from the inherent nature of FRP-reinforced concrete failure modes have been identified and explored.

Keywords: codes and standards; concrete design; FRP reinforcement

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INTRODUCTION

The widespread successful use of composite materials in the automotive, naval and sporting goods industries has resulted in a technology transfer to civil infrastructure applications. Fiber-reinforced polymers (FRP) are impacting the international concrete industry. FRP combines high-strength glass, carbon, and aramid fibers with polymer resins, and can be used as both internal and external reinforcement for concrete members. FRP provides an alternative to steel reinforcement in areas where seawater, deicing salts, and corrosives can destroy the structural integrity of a concrete member due to deterioration of the steel reinforcement.

While there is a growing body of published research on FRP-reinforced concrete (Bakis et al 2002) and while there are a limited number of standards and codes that address design of FRP-reinforced concrete in Canada (CSA 1996 and CSA 2002), Europe (*fib* 2001), and Japan (JSCE 1997), the lack of accepted design guidelines and code language limits the use of FRP reinforcement in commercial concrete structures in the United States. The American Concrete Institute's *Guide for the Design and Construction of Concrete Reinforced with FRP Bars* (ACI 2003) is not intended as a code standard at this time, although it is anticipated that code language may evolve from this document as the research and design community develop experience and confidence in design methods and recommended practices. In order to facilitate code development, research equations and analytical techniques must be successfully translated to the design office.

The objective of the work presented in this paper is to provide insight on designing single layer internal glass and carbon FRP reinforced concrete members in accordance with the ACI *Guide*. A spreadsheet program was written for flexural and serviceability analysis of concrete sections reinforced with a single layer of glass or carbon FRP bars. Analysis and design examples were developed and design aids were constructed to assist in economically and efficiently sizing FRP-reinforced concrete members. Potential difficulties arising from FRP-reinforced concrete failure modes have been identified and explored. This paper also addresses potential problems and misconceptions due to the differences in designing concrete with FRP reinforcement rather than with steel. The spreadsheets and all design examples are based on recommendations in the American Concrete Institute's *Guide for the Design and Construction of Concrete Reinforced with FRP Bars* (ACI 2003). The design and analysis examples and design aids are intended to cover typical concrete construction. The design examples also serve to illustrate application of the recommended design procedures. All design aids and examples were developed using minimum concrete cover requirements from the ACI *Guide*.

FRP-REINFORCED CONCRETE DESIGN PHILOSPHY

General considerations

The design philosophy for FRP-reinforced concrete is based on principles of equilibrium, compatibility of strains, and the stress-strain characteristics of the materials involved. The brittle behavior of both FRP reinforcement and concrete must be considered. Crushing of the concrete or FRP rupture are the mechanisms that control the failure of the section. For concrete crushing, the Whitney rectangular stress block is used to approximate the concrete stress distribution at ultimate strength conditions. For FRP reinforcement, the linear-to-failure stress-strain relationship must be used. If the FRP reinforcement ruptures, sudden and catastrophic failure can occur. Therefore, concrete crushing has typically been considered to be the more desirable failure mode, although there are opposing viewpoints on this issue. Using limit states principles, FRP reinforced sections are designed based on required strength considerations and then checked for fatigue endurance, creep rupture stress limits, and serviceability criteria. In many cases, serviceability limit states are expected to control the design.

Flexural failure of FRP-reinforced concrete sections can only be brittle. However, when FRP reinforcement ruptures in tension (i.e. tension-controlled failures), there is significant elongation. Warning of impending failure is given by extensive cracking and large deflections. The compression-controlled failure mode of concrete crushing also has good deformability, with sections exhibiting pseudo-plastic behavior before failure. Both failure modes are acceptable in the design of FRP-reinforced flexural members provided that strength and serviceability criteria are satisfied.

Differences between FRP and steel reinforcement

A direct substitution between FRP and steel reinforcement is not possible due to differences in the mechanical properties of the two materials. The modulus of elasticity of FRP is much lower than that of steel; thus, larger strains are needed to develop comparable tensile stresses in the reinforcement. If a direct substitution of FRP for steel reinforcement was used, FRP reinforced sections would have larger deflections and crack widths than comparable steel reinforced sections.

In traditional steel reinforced sections, the section is under-reinforced. Specifying a maximum amount of steel reinforcement ensures that the steel yields first, giving a ductile failure mode with good deformability. Through plastic straining in the steel reinforcement, there is adequate warning of impending failure and the failed section is able to absorb large amounts of energy. FRP reinforced concrete sections do not exhibit the ductility that is commonly observed for under-reinforced concrete sections reinforced with steel. Therefore, the suggested margin of safety against failure for FRP reinforced concrete is higher than that used for traditional steel-reinforced sections.

In FRP reinforced sections, the concrete may be over-reinforced by specifying a minimum amount of FRP reinforcement. If the section fails, the concrete will crush in compression (compression-controlled failure). The crushing of the concrete serves as warning of failure and reserves tensile capacity in the FRP reinforcement. Under-reinforced (tension-controlled) failures are also permitted, but the failure mode is much different than that of steel-reinforced concrete. Tension-controlled failures result in FRP rupture, and the failure may be catastrophic due to the linear to elastic failure of the FRP material; however, deformability comparable to steel-reinforced sections (and to compression-controlled FRP sections) is achievable. In steel reinforced concrete members, the ultimate limit states usually control design of the section, with the required strength dictating the geometry. In FRP reinforced concrete, serviceability limit states (deflections, crack widths, or creep rupture stress limits) will in all likelihood control the design of the member.

The general design approach to using internal FRP reinforcement has been to set a lower limit on FRP reinforcement to achieve a compression-controlled failure in the concrete. This is a change in design methodology when compared to steel. Due to the increased tensile strength of the FRP and the compression-controlled failure mode, researchers have suggested using higher concrete compressive strengths to make more efficient use of the FRP tensile strength (Yost and Gross 2002). However, serviceability limits present challenges to this approach in practical design.

SPREADSHEET ANALYSIS PROGRAM

A spreadsheet program (Feeser 2005) was written for flexural and serviceability analysis of concrete sections reinforced with a single layer of glass or carbon FRP bars. The spreadsheet provides a convenient means to analyze or design a concrete section in accordance with ACI 440.1R-03. The spreadsheet analyzes a reinforced concrete section for flexural strength, serviceability, creep rupture and fatigue based on values for ultimate moment, service moments, section geometry, concrete compressive strength, FRP type and material properties, and environmental conditions. The program compares the analysis results to the design criteria requirements, and indicates whether the section meets applicable criteria.

The major objective in developing the spreadsheet was to investigate design parameters. The spreadsheet was adapted to study how various design parameters interact with each other, thus identifying the challenges in designing a reinforced concrete section to satisfy ultimate strength and crack width requirements for a desired failure mode. Results from that study were plotted as figures and design aids.

GLASS FIBER-REINFORCED POLYMER ANALYSIS

Section efficiency

In practical design of concrete reinforced with glass FRP bars, there are several issues that challenge what has emerged as preferred design concept. An interesting trend becomes apparent from examination of Figure 1, which is referred to as "Section Efficiency" because it displays the ability of a section to achieve larger nominal moment capacity based on reinforcement ratio. The figure shows the variation in design moment capacity (normalized with size of cross section and amount of reinforcement as $\varphi M_n/A_f bd^2$) as a function of reinforcement ratio (normalized to the balanced reinforcement ratio as ρ_f/ρ_{fb}).

Figure 1 displays a direct relationship between the reinforcement ratios to nominal moment capacity for different concrete strengths. This figure shows a variety of height-to-width ratios for rectangular shaped sections with minimum concrete cover, beam width of 457 mm (18") and concrete strengths of 27.5 MPa, 41.5 MPa, and 69 MPa (4 ksi, 6 ksi, and 10 ksi). The analysis was based on a single layer of reinforcement using #25M (#8) bars. Different bar sizes were not considered because bar size has an effect on the tensile strength of the bar, due to the shear lag that develops between fibers in the larger size bars. To simplify the analysis, only #25M bars with a tensile strength of 650 MPa (95 ksi) and modulus of elasticity of 44.8 GPa (6,500 ksi) were considered.

The figure was developed using ρ_f/ρ_{fb} as independent variable to analyze trends as the sections progress through different controlling failure modes. A section with a reinforcement ratio $\rho_f/\rho_{fb} < 1.0$ is tension-controlled with a strength reduction factor ϕ equal to 0.50. A section with a reinforcement ratio $\rho_f/\rho_{fb} > 1.4$ is compression-controlled with a strength reduction factor ϕ equal to 0.70. A section with a reinforcement ratio ρ_f/ρ_{fb} between 1.0 and 1.4 is still compression–controlled, but is considered to be a transition section with a strength reduction factor ϕ linearly varying between 0.5 and 0.7.

Figure 1 shows that the tensile strength of GFRP is not large enough to force the section into the compression-controlled region for larger depth sections or for higher strength concrete. The compression-controlled sections had smaller height-to-width ratios and concrete compressive strengths. There are a limited number of concrete sections capable of achieving a compression-controlled failure, even when the section is reinforced with the maximum amount of GFRP that can be contained in a single layer. It is apparent that multiple layers of reinforcement are necessary to achieve compression-controlled failures, but the current design guide does not address such sections.

The effect of varying the amount of reinforcement can best be understood in terms of the type of failure mode encountered. In the compression-controlled region, an additional amount of reinforcement does not result in an equivalent increase in moment capacity. For example, increasing A_f by 10% would result in an increase of less than 10% in ϕM_n. In the linear transition region, the $\varphi M_n/A_f bd^2$ ratio increases, resulting in a more efficient use of FRP materials because of the increasing strength reduction factor. In the tension-controlled region, an increase in the amount of reinforcement provides an equal increase in moment capacity for a given section. The dramatic decrease in moment capacity as the section failure mode regresses from transition to tension-controlled, as seen in Figure 1, is the direct result of a 0.80 reduction coefficient used in calculating the nominal moment capacity for tension-controlled sections in the ACI 440.1R-03 *Guide*. The *Guide* rationale for the 0.80 coefficient is that it provides a conservative and yet meaningful approximation of the nominal moment capacity. The validity of using the recommended 0.80 coefficient for tension-controlled sections will be discussed later in the paper.

Reinforcement efficiency

Other trends become apparent from examination of Figures 2 and 3. These figures display a direct relationship between the area of reinforcement and nominal moment capacity for different concrete compressive strengths. However, these figures do not directly display the different modes of failure; rather the failure mode can be identified by the change in the slope of each curve. The initial portion of each curve corresponds to a tensioned-controlled section. The first change in the slope represents the point at which the section moves into the linear-transition range. The second change in the slope is the point at which the section becomes completely compression-controlled.

Generally accepted opinion is that using a higher concrete strength results in a more efficient use of FRP (Yost and Gross, 2002). However, for GFRP, Figures 2 and 3 indicate it may actually be more efficient to use lower concrete strengths, especially if a compression-controlled failure mode is preferred. The GFRP's tensile strength is not large enough to take advantage of the higher concrete compressive strength. For example, in Figure 2, the 27.5 MPa (4 ksi) concrete actually achieves a larger moment capacity than do the higher strength concretes for area of reinforcement less than approximately 2870 mm^2 (4.45 in^2), due to the type of failure mode encountered. When the area of reinforcement is increased beyond 2870mm^2, the 41.5 MPa (6 ksi) concrete achieves the largest moment capacity. The 69 MPa (10 ksi) concrete never exceeds the other concrete compressive strengths' moment capacities; it is not possible to provide a sufficient amount of FRP reinforcement in a single layer to take advantage of the full strength of the 69 MPa concrete. Note, however, that sections with the smaller height-to-width ratios will in all likelihood be controlled by deflections. As the height-to-width ratio is increased in Figure 3, the 27.5 MPa reinforced concrete sections achieve moment capacities comparable to the higher strength concrete in the tension-controlled region. The 27.5 MPa reinforced sections become compression-controlled at smaller areas of reinforcement than when higher strength concrete is used; therefore, it is more efficient to use lower concrete compressive strengths with GFRP. Equally reinforced sections with higher concrete compressive strengths can force the failure mode of the section from compression-controlled through transition into tension-controlled sections because of the limiting tensile strength of the glass FRP. Tension-controlled sections do not appear to result in efficient designs, due to the penalties from the much lower ϕ factor (0.5 versus 0.7 for compression-controlled sections) and the 0.8 reduction coefficient in the equation for nominal moment capacity of tension-controlled sections in the ACI 440.1R *Guide*.

Design efficiency

The spreadsheet analysis program was used to determine the maximum allowable service moment that would satisfy crack width criteria. The service moment from crack width limits was compared with the maximum allowable service moment to satisfy ultimate strength limits, calculated by dividing the design strength φM_n by an average overall load factor of 1.512 (from a 2:1 dead load to live load ratio). Figure 4 displays the trends of the maximum allowable service moment to satisfy crack width criteria and maximum allowable service moment to satisfy ultimate strength limits. When the maximum service moment to satisfy ultimate strength limits is larger than the maximum service moment to satisfy crack widths, crack width criteria control resulting in an inefficient design that cannot develop full moment capacity. Conversely, for members in which the maximum service moment to satisfy crack width criteria is greater than the maximum service moment to satisfy ultimate strength limits, strength criteria control and the full moment capacity can be developed, resulting in a much more efficient use of FRP reinforcement.

Smaller bar size and greater H/W ratios result in sections in which efficient designs are more easily achievable. As Figure 4 illustrates, tension-controlled designs will typically be controlled by strength rather than crack width limitations. As these sections approach the transition limit, however, the situation changes and crack width limitations become the more critical factor. This trend continues through much of the compression-controlled region as well. However, if the GFRP reinforced sections could actually be reinforced with larger reinforcement ratios in the compression-controlled region, efficient designs in which ultimate strength was the limiting factor would again be achievable. Such reinforcement ratios are usually not possible when reinforcement is placed in a single layer. Therefore, sections reinforced with GFRP will usually be designed for tension-controlled failures. In these situations, ultimate strength limits rather than crack width criteria (or creep rupture stress limits) will in all likelihood control the design and section geometry. In addition, sections designed using minimum amounts of reinforcement should be more cost effective than designing for a compression-controlled failure. In the tension-controlled region, using higher concrete compressive strengths is not necessarily cost effective because the section will achieve the same moment capacity for an equally sized and reinforced section using lower concrete compressive strength.

CARBON FIBER-REINFORCED POLYMER ANALYSIS

Although the design of CFRP reinforced concrete sections experiences some of the same tradeoffs as with GFRP reinforced sections, there are differences. For example, as Figure 5 shows, the CFRP tensile strength is large enough to force the section into the compression-controlled region ($\rho_f/\rho_{fb} > 1.4$), even with deeper sections and higher concrete strengths. Such is not the case with lower-strength GFRP reinforcement. The negative slope of the CFRP curves in the compression-controlled region indicates that an additional amount of reinforcement does not result in an equivalent increase in moment capacity. This development is more prevalent for CFRP sections because a majority of the sections are compression controlled as compared to GFRP sections. In the tension-controlled region ($\rho_f/\rho_{fb} < 1$), an increase in the amount of reinforcement provides an equal increase in moment capacity for both GFRP and CFRP sections.

The effect of varying the amount of reinforcement for CFRP and GFRP reinforced sections is very similar as Figure 6 indicates. For GFRP, it is more efficient to use lower concrete compressive strengths, but for CFRP higher concrete compressive strength can be used to take advantage of the higher tensile strength of the CFRP. In Figure 6, as the area of reinforcement increases for the CFRP sections, the 27.5 MPa (4 ksi) concrete achieves the greatest moment capacity up until approximately 1550 mm^2 (2.40 in^2) of reinforcement. The 41.5 MPa (6 ksi) concrete then provides larger design moments up until approximately 2160 mm^2 (3.35 in^2) of CFRP; thereafter, the 69 MPa (10 ksi) designs achieve the greatest moment capacity. However, for GFRP, the 69 MPa concrete never exceeds the other concrete compressive strengths' moment capacities. As the height-to-width ratio increases, the relationship is similar, but the change-over points occur at larger areas of reinforcement. All of the sections have equal moment capacities in the tension-controlled region, regardless of reinforcement type or concrete strength. Also note that CFRP sections can be reinforced with a smaller minimum area of reinforcement because of the higher CFRP tensile strength. Thus, unlike GFRP sections, use of higher concrete compressive strengths is more efficient for CFRP sections.

Figure 7 displays the ratio of maximum service moment to satisfy crack width criteria to maximum service moment to satisfy ultimate strength limits, and thus provides a means to evaluate the efficiency of the design. To take full advantage of the FRP reinforcement, sections should preferably be limited by either ultimate strength or creep rupture stress considerations, which mean the ratio of crack width service moment to service moment limited by ultimate strength should be greater than 1. The shape of the design efficiency curves for CFRP is similar to what is seen with GFRP sections, except that the CFRP curves extend more fully into the compression-controlled range. Figure 7 indicates that sections with larger concrete compressive strengths achieve their largest design efficiency ratios when approaching the transition limit in the tension-controlled region. Sections with smaller concrete compressive strengths achieve the greatest design efficiency ratios in the compression-controlled region as the section approaches the maximum possible area of reinforcement that can be fitted into a single layer. Extrapolating the curves for the higher strength concrete, a reasonable assumption is that design efficiencies would also increase in compression-controlled regions, if it were possible to fit the required area of reinforcement into the section. Such an approach, although efficient, would not be particularly economical. Such designs are not likely to satisfy deflection limitations.

In summary, the analysis of CFRP and GFRP reinforced sections show significant similarities and differences. For both CFRP and GFRP sections, tension-controlled sections have equal moment capacities per unit area of FRP for varying amounts of reinforcement and designs are rarely controlled by crack width limitations. In the linear transition region, increasing the amount of reinforcement results in proportionally greater increase in moment capacity from ultimate strength limitations, but these sections will usually have service moment controlled by crack widths. In the compression-controlled region, the $\varphi M_n/A_f bd^2$ ratio decreases with increasing reinforcement ratio. For CFRP compression-controlled sections, the greatest reinforcement efficiencies are achieved using higher concrete compressive strength with practical areas of reinforcement. However, for GFRP sections, the greatest reinforcement efficiencies are achieved using lower concrete compressive strength for tension-controlled sections with practical areas of reinforcement. The difference in section and reinforcement efficiencies between CFRP and GFRP is a direct result of the higher tensile strength of the CFRP.

Sections reinforced with CFRP may be designed for compression-controlled failures. Crack width criteria rather than ultimate strength or creep rupture stress limits will likely control section geometry, the reverse of what occurs in GFRP sections where designs are most likely to be tension-controlled. Higher concrete compressive strengths should be used with CFRP reinforcement to take advantage of CFRP's higher tensile strength.

TENSION-CONTROLLED MOMENT EQUATION

ACI 440.1R (ACI 2003) recommends that moment capacity of tension-controlled sections be calculated as

$$M_n = 0.8 A_f f_{fu}\left(d - \frac{\beta_1 c_b}{2}\right) \qquad (1)$$

The $A_f f_{fu}$ term represents the tensile force in the FRP at rupture, while $d - {}^1/_2\ \beta_1 c_b$ provides a conservative lower-bound approximation to the actual moment arm. The rationale for the 0.8 factor is less clear, as the *Guide* only indicates that it provides a "conservative and yet meaningful approximation of the nominal moment." The implication is that the reduction coefficient adjusts computed values to reflect test data.

If the 0.80 reduction coefficient is removed from the calculation for tension-controlled moment capacities in Equation (1), continuity is achieved in the design strength φM_n values as a section's reinforcement ratio moves from the tension-controlled region into the transition range, for both the GFRP and the CFRP reinforcement. In Figure 8, the normalized design moment makes a smooth transition as the section moves into the linear-transition range, in contrast to the same curves in Figure 5. However, it will not be possible to take complete advantage of the increase in moment capacity that might be expected from the removal of the 0.8 reduction coefficient. Figure 9 indicates that serviceability criteria will now control the design in the tension-controlled region, instead of ultimate strength, as was the case in the tension-controlled range in Figure 7.

DESIGN EXAMPLES AND DESIGN AIDS

Guided design examples can be used to illustrate how theoretical approaches to design can be applied to FRP reinforced concrete sections and to identify unresolved design issues. Two separate design scenarios were considered. The scenarios were created to simulate real and practical design applications using FRP reinforcement. Both design scenarios contained several design element examples: one-way slab, simply supported T-shaped beam, and continuous rectangular shaped girder.

The first design scenario was a hospital floor that supports electromagnetic equipment where magnetic fields are present. The hospital floor plan has a typical bay spacing of 7325 mm by 4875 mm (24' x 16') and floor live loading of 2.875 kN/m^2 (60 psf). Glass FRP reinforcement was used for all of the hospital floor's supporting elements because the GFRP reinforcement can be used more effectively for lighter loads and shorter spans.

The second design scenario was a typical manufacturing plant floor exposed to a highly corrosive environment where the corrosion-resistance of the FRP reinforcement is most beneficial. The manufacturing floor plan has a typical bay spacing of 9750 mm by 7325 mm (32' x 24') and floor live loading of 6 kN/m^2 (125 psf). Carbon FRP reinforcement was used for all of the manufacturing plant floor's supporting elements because the CFRP reinforcement can be used more effectively for heavier loads and longer spans.

Both design scenarios contain similar elements – a floor slab that forms the flanges of simply supported T beams, which are themselves supported on continuous rectangular girders - illustrating the similarities and differences in applications using GFRP and CFRP reinforcement. Design aids were developed as an efficient means to proportion member sizes on the basis of ultimate strength, desired failure mode, and serviceability criteria. The design aids should be used with caution to proportion reinforced sections because dead load to live load ratio, shear reinforcement size and clear cover can greatly affect serviceability criteria. Also note that deflection and creep rupture stress criteria are not included in the design aids but may control section design. This means that member sizes may require adjustment from what is initially chosen based on the design aids, which should only be used to provide a reasonable starting point for sizing members.

The spreadsheet program was used to develop two sets of design aids, one set for glass and carbon FRP reinforced concrete slabs, and the other for GFRP and CFRP reinforced concrete beams. A typical design aid is shown in Table 1. The design aids provide the ultimate strength φM_n and maximum service moments to satisfy crack width criteria for a variety of section dimensions, reinforcement bar sizes, and number of bars in a single layer of reinforcement, assuming minimum concrete cover. The design aid maximum service moment was based only on crack widths for interior exposure conditions because crack width criteria usually limit the service moment. A member geometry that meets crack width criteria will typically be adequate for creep rupture limits as well. Deflection limits depend on performance criteria such as time-dependent factor for sustained loads and tolerance provisions for support or attachment of nonstructural elements, and were not included in the development of the design aids.

Hospital design examples with GFRP reinforcement

The hospital floor supports electromagnet equipment where magnetic fields are present. The hospital floor plan has a typical bay spacing of 7325 mm by 4875 mm (24' x 16'). The floor is designed to support a service live load of 2.875 kN/m^2 (60 psf), 20% sustained load, and superimposed service dead load of 1.44 kN/m^2 (30 psf). Normal weight concrete with a 28 day strength of 27.5 MPa (4 ksi) will be reinforced with GFRP. Deflections shall not exceed L/360 immediate live load deflection and L/480 long-term deflection with a time-dependent factor of 5 years or more, because the floor supports nonstructural elements likely to be damaged by large deflections. Members are designed using minimum clear cover and #12M (#4) stirrups for shear reinforcement. The crack widths are computed using a recommended bond coefficient, k_b, of 1.2. All elements (floor slab, simply supported T beam, and continuous rectangular girder) were designed for flexure and serviceability; the simply supported T beam was also designed for shear and bond and anchorage requirements. The results of the design are shown in Figure 10.

Manufacturing plant design examples with CFRP reinforcement

The manufacturing plant floor is exposed to an environment where the corrosion-resistance of FRP reinforcement is beneficial. The floor plan has a bay spacing of 9750 mm by 7325 mm (32' x 24'). The floor is designed to support a service live load of 6 kN/m^2 (125 psf) with 20% sustained load and a superimposed service dead load of 1.44 kN/m^2 (30 psf). Normal weight concrete with 55 MPa (8 ksi) compressive strength will be reinforced with CFRP reinforcement. Beam deflections must not exceed immediate live load deflection of L/360 and long-term deflection of L/240 because the floor does not support nonstructural elements likely to be damaged by large deflections. Members are designed using minimum clear cover and #12M (#4) stirrups. The crack widths are computed using a recommended bond coefficient, k_b, of 1.2. All elements (floor slab, simply supported T beam, and continuous rectangular girder) were designed for flexure and serviceability; the continuous girder was also designed for shear and bond and anchorage requirements. The results of the design are shown in Figure 11.

DESIGN ISSUES AND CONSTRUCTION CONCERNS

The design examples for GFRP and CFRP reinforced sections identified several design issues for FRP reinforcement material properties and concrete compressive strength.

FRP reinforcement material properties

Designers must recognize that FRP material properties vary more from manufacturer to manufacturer than do steel material properties. FRP reinforcement bars are not currently standardized across the FRP manufacturing industry. Unless the designer mandates a specific commercial FRP manufacturer and bar type in the design specifications, FRP bars used in construction will in all likelihood have different properties than those used in design. If the contractor is free to choose the FRP manufacturer, the initial design should be checked in the shop drawing phase for changes in FRP material properties to assure all design requirements are met. As a result, designers should take extra caution to properly specify required properties for their design in the drawings and specifications.

Concrete compressive strength

In typical reinforced concrete design situations, the designer specifies a minimum 28-day concrete compressive strength. In most cases, the concrete supplied exceeds the specified strength. For FRP reinforced sections, actual concrete strength greater than specified strength could (under certain circumstances) result in a change of failure mode where larger safety factors for tension-controlled failures in the current version of the ACI *Guide* could potentially result in smaller than intended design strength φM_n, despite an increase in failure moment M_n. Table 2 illustrates the effect that increasing concrete compressive strength can have on an FRP reinforced design. The continuously spanning rectangular shaped girder was originally designed with a concrete compressive strength of 55 MPa (8 ksi), as part of the manufacturing plant design example. The same design is analyzed to investigate the impact that an actual concrete compressive strength of 64 MPa (9300 psi), 16% higher than specified, would have. The rectangular shaped girder (see Figure 11) positive moment design capacity is φM_n = 1020 kN-m (752.20 kip-ft), which is sufficient for the M_u = 890 kN-m (656.35 kip-ft) moment demand from the factored loads on a 9.75 m (32 ft) span. The resulting design was a 380 mm by 915 mm (15” by 36”) section with five #19M (#6) CFRP bars. However, the section’s moment capacity φM_n decreases to 767 kN-m (565.68 kip-ft) when the concrete compressive strength increases from 55 MPa to 64 MPa and if the nominal moment capacity M_n is calculated by Eq. 1 as recommended in ACI 440.1R. This is because the increase in concrete compressive strength causes an increase in the section’s balanced reinforcement ratio, switching the mode of failure from a compression-controlled failure in the transition zone with 55 MPa concrete to a tension-controlled failure with 64 MPa concrete. Removal of the 0.8 reduction coefficient from Eq. 1, as recently proposed by ACI Committee 440, will mitigate this situation although design strengths φM_n may still be smaller than intended - as is the case for the example illustrated in Table 2.

CONCLUSIONS AND RECOMMENDATIONS

The corrosion resistance, nonconductive, and nonmagnetic properties of FRP reinforcement provide advantages to steel reinforcement in specialized environments. However, where there is an opportunity that FRP could be substituted for steel, FRP reinforcement needs to provide a bottom line cost benefit to the owner. A lifetime project cost-benefit analysis should be employed, where costs over the anticipated service life of the structure can be equally compared for FRP and steel reinforced concrete elements.

The design approach to using internal FRP reinforcement in concrete members has been to set a lower limit on FRP reinforcement to achieve a compression-controlled failure in the concrete. This is a change in design methodology when compared to steel. Due to the increased tensile strength of the FRP, researchers have suggested using higher concrete compressive strengths for efficient use of the FRP tensile strength. Concrete sections reinforced with a single layer of GFRP bars contradict such an approach because the tensile strength of the GFRP is not large enough to achieve the larger reinforcement ratios required to take advantage of the benefits of a compression-controlled section. Rather, GFRP reinforced sections should be designed for tension-controlled failures. With the moment capacity equation currently recommended by the ACI *Guide,* ultimate strength limits rather than crack width criteria will in all likelihood control the design and determine the section’s geometry. Under these conditions, sections designed using minimum amounts of reinforcement should be more cost effective than designing for a compression-controlled failure. For tension-controlled sections where the reinforcement is placed in a single layer, using higher concrete compressive strengths is less cost effective because the section will achieve the same moment capacity for an equally sized and reinforced section using lesser concrete compressive strengths. Sections reinforced with multiple layers of GFRP reinforcement may be more attractive and should be explored. Multiple layers of reinforcement will enable a GFRP reinforced section to achieve larger reinforcement ratios for a compression-controlled failure. Using larger concrete compressive strengths to make more efficient use of the GFRP tensile strength is more applicable to designs with multiple layers of reinforcement.

The 0.80 reduction factor in the moment equation for tension-controlled sections should be revisited. Indeed, ACI Committee 440 has recently chosen to recommend such changes. For GFRP sections, such a change may cause serviceability rather than ultimate strength criteria to control section geometry.

Concrete sections reinforced with a single layer of CFRP bars can reasonably use larger concrete compressive strengths to make better use of FRP tensile strength. The tensile strength of CFRP is large enough to achieve the larger reinforcement ratios that can take advantage of the compression-controlled failure mode; thus, CFRP reinforced sections should be designed for compression-controlled failures. Crack width criteria and deflection limits will probably determine section geometry, unlike what occurs in GFRP sections. CFRP sections should be designed using the minimum reinforcement necessary to achieve compression-controlled failure. Larger concrete compressive strengths may be used with CFRP reinforcement to take advantage of the higher tensile strength of the CFRP. A cost-benefit analysis studying the impact of using larger concrete compressive strengths should be done to identify optimum concrete compressive strength for CFRP reinforcement. Typically, additional costs are involved in the production of higher concrete compressive strengths, which may offset any potential benefits.

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Table 1: DESIGN AID 1 (kN-m)

GFRP Reinforced Slab, 27.5 MPa Concrete

Bar Spacing	102 mm Slab		127 mm Slab		152 mm Slab		203 mm Slab	
#10M Bars @	Mn	Service	Mn	Service	Mn	Service	Mn	Service
203 mm	*2.0*	*1.2*						
152 mm	3.7	1.6	*3.7*	*2.4*				
102 mm	**5.0**	**2.8**	7.6	4.1	8.7	5.4	*9.4*	*8.0*
#12M Bars @	Mn	Service	Mn	Service	Mn	Service	Mn	Service
305 mm	*2.2*	*1.1*	*3.0*	*1.6*				
203 mm	4.7	1.9	5.6	2.7	*5.6*	*3.7*		
152 mm	**5.3**	**2.7**	8.4	3.9	9.6	5.3	*10.3*	*7.9*
102 mm	**6.1**	**4.6**	**9.9**	**6.6**	**14.1**	**8.8**	20.7	13.3
#16M Bars @	Mn	Service	Mn	Service	Mn	Service	Mn	Service
305 mm	4.1	1.6	*4.1*	*2.3*	*5.0*	*3.0*		
203 mm	**5.4**	**2.6**	8.7	3.9	9.9	5.1	*10.7*	*7.7*
152 mm	**6.0**	**3.8**	**9.8**	**5.6**	**14.0**	**7.5**	18.2	11.3
102 mm	**6.9**	**6.2**	**11.3**	**9.4**	**16.3**	**12.6**	**28.3**	**19.1**
#19M Bars @	Mn	Service	Mn	Service	Mn	Service	Mn	Service
305 mm	**5.1**	**2.0**	7.2	3.0	*6.8*	*3.9*	*9.5*	*6.1*
203 mm	**6.0**	**3.2**	**9.6**	**5.0**	**14.1**	**6.8**	18.0	10.3
152 mm	**6.5**	**4.7**	**10.7**	**7.2**	**15.7**	**9.8**	26.8	14.9
102 mm	**7.5**	**7.5**	**12.3**	**11.9**	**18.0**	**16.3**	**31.9**	**25.1**

NOTES : 1. Minimum concrete cover is assumed.

2. All *Italic* values are Tension-Controlled Sections based on ACI 440.1R-03 Equation 8-6b.
3. All Regular values are Linear-Transition Sections based on ACI 440.1R-03 Equation 8-5.
4. All **BOLD** values are Compression-Controlled Sections based on ACI 440.1R-03 Equation 8-5.
5. Maximum service moments satisfy crack width criteria based on ACI 440.1R-03 Equation 8-9c.
6. Exposure Condition - Not exposed to earth and weather
7. Deflection limits are NOT considered in design aids but may control final design.
8. Creep rupture limits are NOT considered in design aids but may control final design.

Table 2: Effect of Higher Concrete Strength on Nominal Moment Capacity

	CONCRETE COMPRESSIVE STRENGTH		
	55 MPa (Design Strength) Transition Section	64 MPa (Actual Strength) 0.8 M_n Reduction (Eq 1)	No 0.8 M_n Reduction
M_n	1758.6 kN-m	1534.1 kN-m	1917.6 kN-m
φ	0.58	0.50	0.5
φM_n	1020.0 kN-m	767.1 kN-m	958.8 kN-m

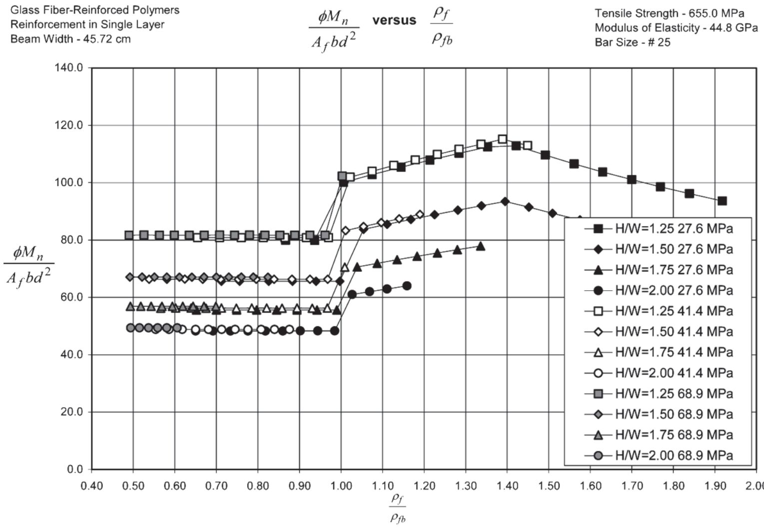

Figure 1: Section Efficiency for GFRP Designs

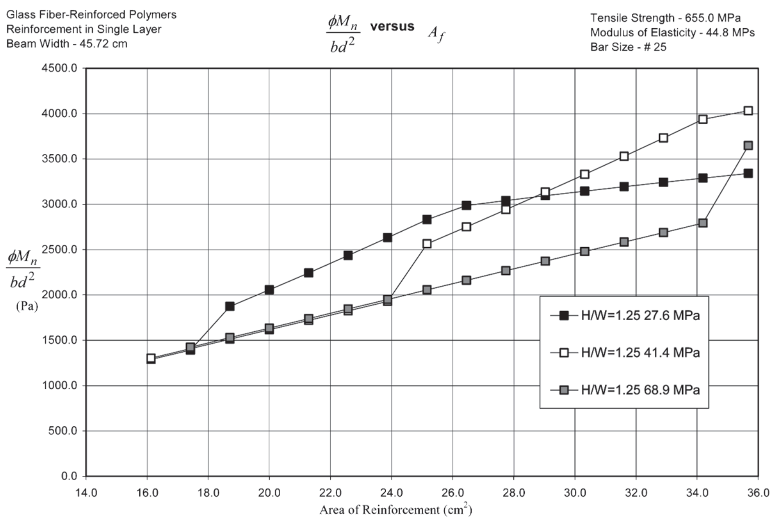

Figure 2: Reinforcement Efficiency for a 1.25 Height-to-Width Ratio

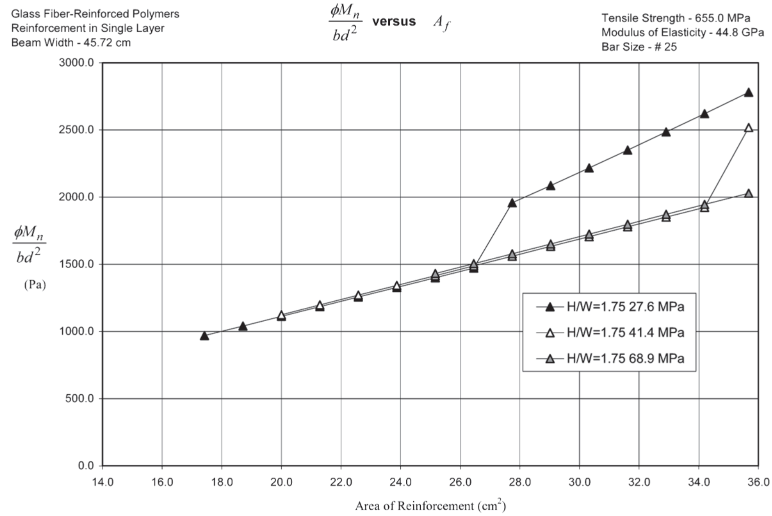

Figure 3: Reinforcement Efficiency for a 1.75 Height-to-Width Ratio

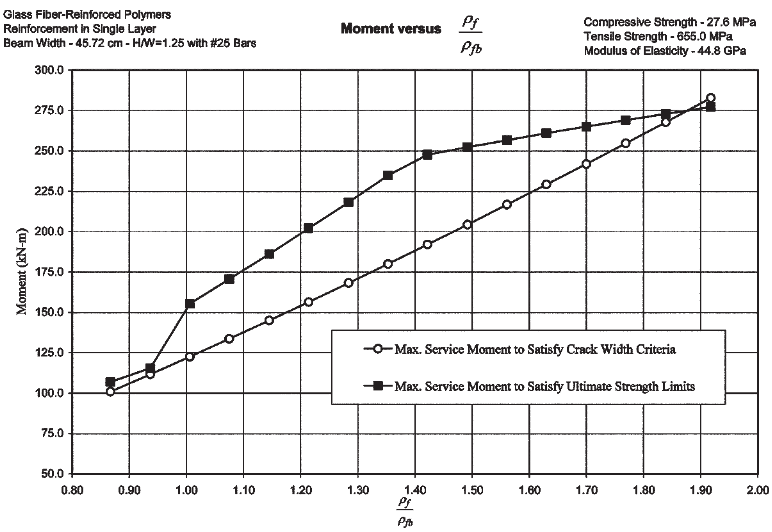

Figure 4: Allowable Service Moment as Controlled by Serviceability & Strength

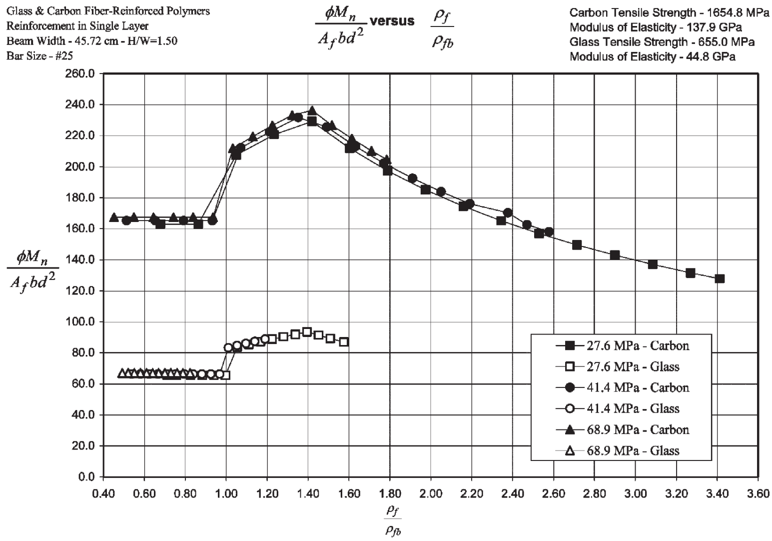

Figure 5: Section Efficiency of CFRP Relative to GFRP

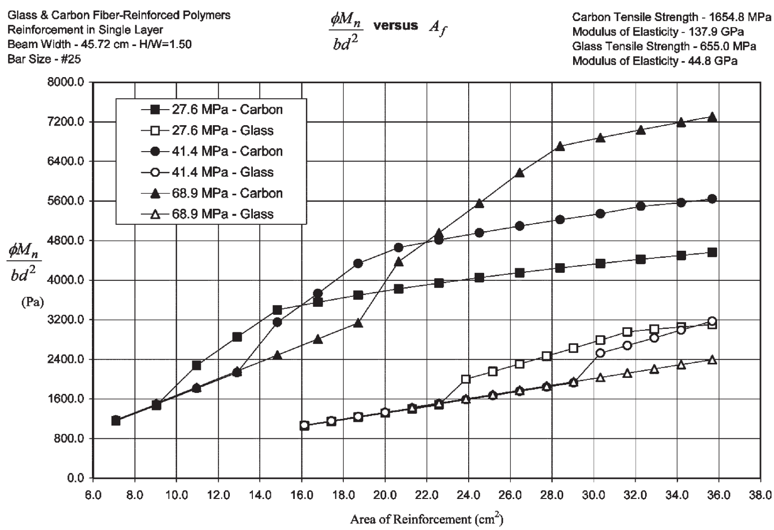

Figure 6: Reinforcement Efficiency of CFRP Relative to GFRP

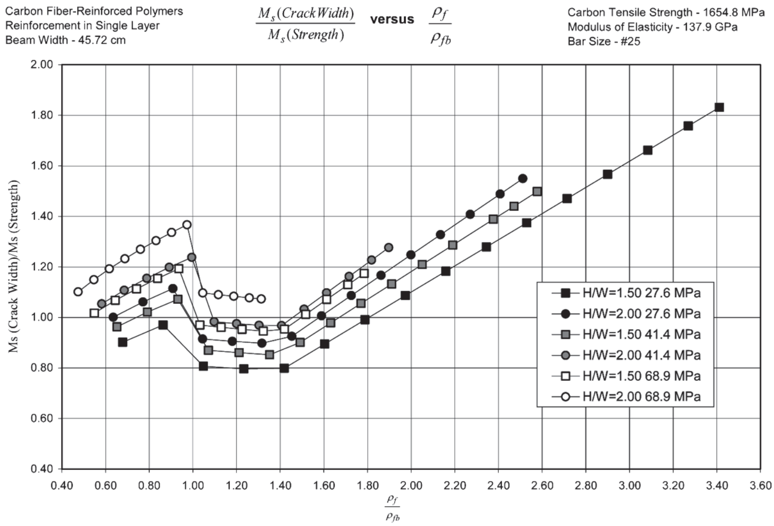

Figure 7: Crack Width Limitation on Allowable Service Moment

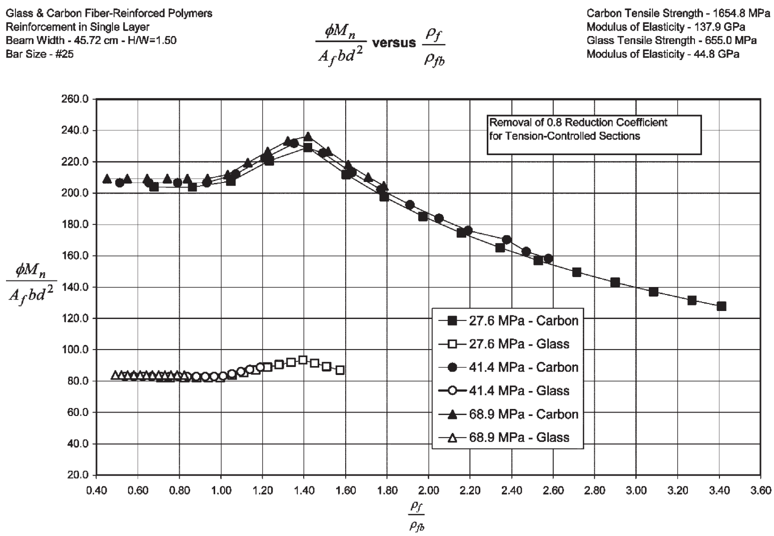

Figure 8: Impact of Tension-Controlled Reduction Coefficient on Section Efficiency

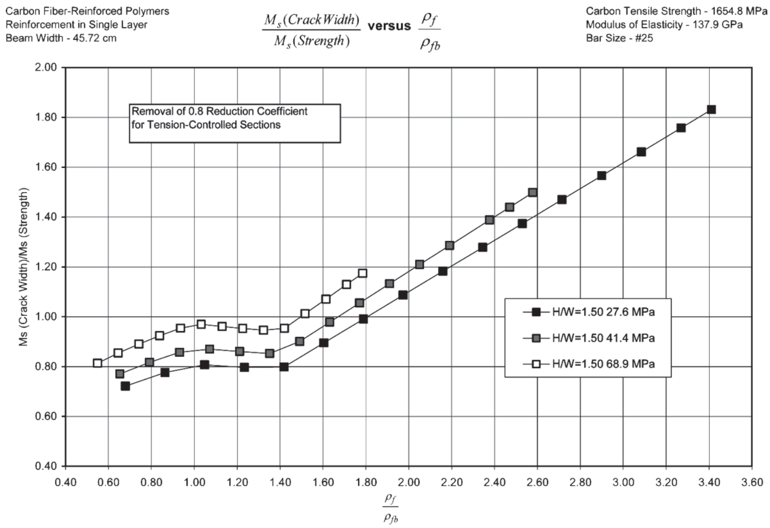

Figure 9: Impact of Tension-Controlled Reduction Coefficient on Design Efficiency

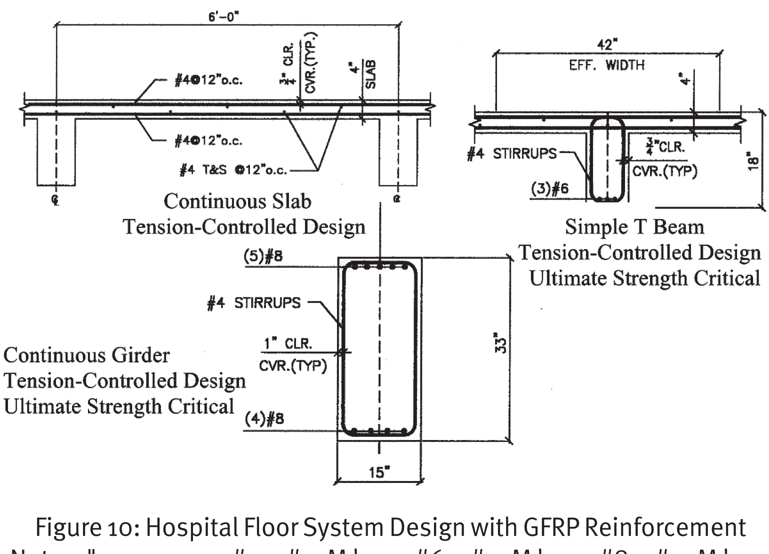

Figure 10: Hospital Floor System Design with GFRP Reinforcement
Note: 1" = 25.4 mm; #4 = #12M bars; #6 = #19M bars; #8 = #25M bars

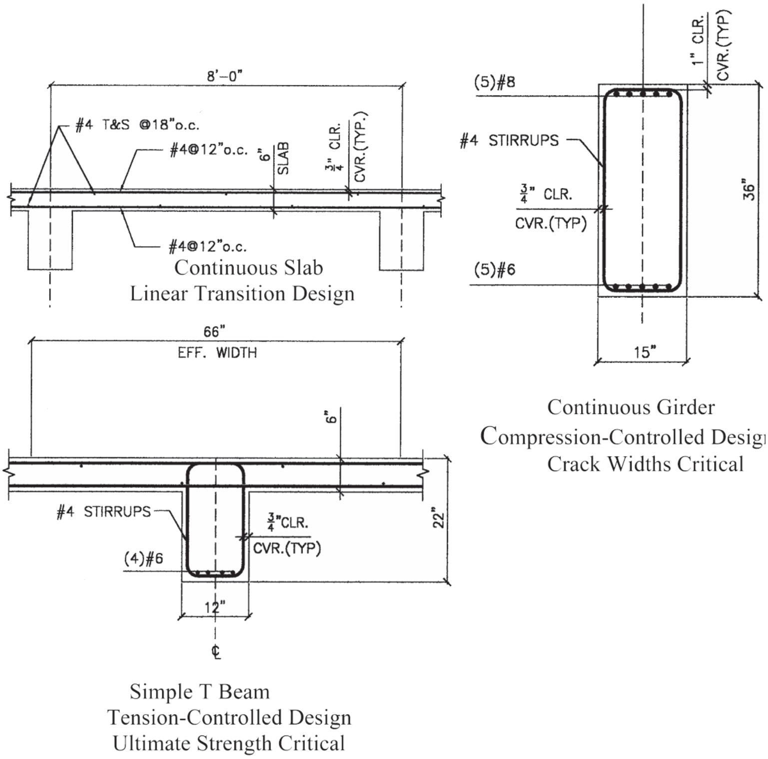

Figure 11: Manufacturing Plant Floor System Design with CFRP Reinforcement
Note: 1" = 25.4 mm; #4 = #12M bars; #6 = #19M bars; #8 = #25M bars