Feasibility Of Fiber Reinforced Composite Materials
Used In Highway Bridge Superstructures
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(ABSTRACT)

Composite materials are considered here as structural materials of highway bridge superstructures. Bridge deck designs can be done according to AASHTO\textsuperscript{1} specification and elastic design concepts.

In order to evaluate the feasibility of composites as structural materials of highway bridge superstructures, composite materials are compared not only to composite materials themselves but also to the most popular bridge structural materials, which are reinforced concrete and structural steel.

The AASHTO\textsuperscript{1} HS20-44 truck load is selected as the standard loading condition of all designs. Loads other than dead load and live load are not considered. Configurations of the bridges are different. Appropriate cross-section of girders are selected according to the material types. For fiber re-
inforced composite materials, box girder is used, for reinforced concrete, T-beam is selected; in addition, steel concrete composite section is another case.

Design methods are different from material to material. Reinforced concrete T-beam design is based on the 'Ultimate Strength Design' method. Steel concrete composite sections are designed according to the 'Load & Resistance Factor Design'. For composite materials, 'Elastic Design' is selected.

The results derived are as expected. Substantial weight saving is achieved by simply replacing concrete or steel with composite materials. This also results in many other advantages such as construction time, cost, foundation settlement and support requirements.
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NOTATIONS

a = depth of equivalent rectangular stress block
A = cross sectional area
Ac = concrete area
As = area of nonprestressed tension reinforcement
Asf = area of reinforcement to develop compressive strength of overhanging flanges of T-sections
Aw = web area of steel beam
B = width of girder
b = unit width in strip design of concrete slab
be = effective compressive flange width of T-beam
bf = flange width of steel beam
bw = web width
c = distance from extreme fiber to neutral axis
Cb = distance from extreme bottom fiber to neutral axis
Ct = distance from extreme top fiber to neutral axis
d = effective depth
D = dead load
Dg = depth of composite material girder
Ec = modulus of elasticity of concrete
Es = modulus of elasticity of reinforcement
El = elastic modulus of composite materials in fiber direction
\(E_2\) = elastic modulus of composite materials perpendicular to fiber direction

\(f'_c\) = specified compressive strength of concrete

\(F_D\) = dead load factor

\(F_L\) = live load factor

\(f_r\) = modulus of rupture of concrete

\(f_y\) = specified yield strength of reinforcement

\(h\) = overall height

\(h_f\) = compressive flange thickness of T-sections

\(I\) = impact effect

\(I_{cr}\) = area moment of inertia of cracked section transformed to concrete

\(I_e\) = effective area moment of inertia for computation of deflection

\(I_g\) = gross area moment of inertia of concrete section

\(I_x\) = area moment of inertia of section

\(L\) = live load

\(M_a\) = maximum moment in member at the stage for which deflection are being computed

\(M_{cr}\) = cracking moment

\(M_D\) = dead load moment

\(M_L\) = live load moment

\(M_n\) = nominal moment strength

\(M_R\) = required moment strength

\(M_u\) = moment due to factored loads
\[ n = \text{modulus ratio} \]
\[ Q = \text{static moment for a point in the cross section} \]
\[ Q_{\text{max}} = \text{maximum static moment} \]
\[ \Sigma Q_n = \text{nominal shear strength of shear connectors} \]
\[ R = \text{design load carrying capacity} \]
\[ S = \text{clear span of girder} \]
\[ S = \text{shear strength of composite material} \]
\[ s = \text{clear span of floor beam or concrete slab} \]
\[ T = \text{thickness of composite material girder} \]
\[ t_s = \text{slab thickness} \]
\[ t_w = \text{web thickness} \]
\[ U = \text{required strength to resist factored loads} \]
\[ V_c = \text{nominal shear force carried by concrete} \]
\[ V_D = \text{dead load shear force} \]
\[ V_L = \text{live load shear force} \]
\[ V_n = \text{nominal shear strength} \]
\[ V_R = \text{required shear strength} \]
\[ V_s = \text{nominal shear strength provided by shear reinforcement} \]
\[ V_u = \text{factored shear force at section} \]
\[ W = \text{wind load} \]
\[ W_D = \text{distributed dead load} \]
\[ W_s = \text{weight of steel beam} \]
\[ WT = \text{estimate weight in steel-concrete composite section design} \]

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\( x \) = depth of neutral axis
\( X_C \) = compression strength in x-direction of composite materials
\( X_T \) = tension strength in x-direction of composite materials
\( x' \) = centroid location of concrete cross section
\( Y_C \) = compression strength in y-direction of composite materials
\( Y_{con} \) = distance from top of steel beam to top of concrete
\( Y_T \) = tension strength in y-direction of composite materials
\( Y_{top} \) = distance from centroid of cross section to extreme fiber in tension, neglecting reinforcement
\( Y_2 \) = distance from concrete flange force to beam top flange
\( \beta_1 \) = ratio of depth of rectangular stress block, \( a \), to depth of neutral axis.
\( \delta_L \) = deflection due to live load
\( \delta_{\text{max}} \) = maximum allowable deflection
\( \phi \) = strength reduction factor
\( \psi \) = analysis factor
\( \rho \) = steel ratio
\( \rho_b \) = balanced steel ratio
\( \rho_f = \frac{A_{sf}}{b_w d} \)
\( \rho_{\text{max}} \) = allowable maximum steel ratio
\( \rho_{\text{min}} \) = allowable minimum steel ratio
\( \sigma_b \) = stress at bottom fiber
\( \sigma_{\text{max}} \) = maximum stress
\( \sigma_t \) = stress at top fiber
\( \tau \) = shear stress
\( \tau_{\text{max}} \) = maximum shear stress
CHAPTER 1 INTRODUCTION

The Voyager aircraft accomplished a non-stop, unrefueled trip around the world in 1987. This had been a dream of workers in the aeronautical field for decades. One of the most important reasons that made this happen is breakthroughs in materials; that is, fiber-reinforced composite resins. Two of the most important characteristics of fiber-reinforced composite resins are the ratio of stiffness-to-weight and the ratio of strength-to-weight. For weight sensitive structures such as airplanes, these advantages will overcome the relatively high raw material cost.

Composite material is usually used as a name for fiber-reinforced composite resin. Its mechanical properties are a function of both fiber orientation and stacking sequence. Composite materials have been used in many application areas. As a result of mass production, the cost of composite material structural components is decreasing so that these materials may be applied in many other areas which were thought not feasible, for instance, in highway bridge deck structures.

Highway bridges are not so weight sensitive as are aircraft. However, as the span becomes longer, the weight of
load carrying structures such as beams and supports increases dramatically. Foundation requirements are also increased with the increasing span, and foundation settlement may become a problem. If the total structural weight can be reduced, a highway bridge built on a weaker foundation may be appropriate.

Three different bridge deck configurations considered in this study are concrete T-beam, steel-concrete composite section, and composite material bridge deck. The first and second configurations are the most commonly used in highway bridge superstructures. The third will be investigated, in several aspects, to evaluate the feasibility of using composite materials to replace concrete and steel.

The design model (Figure 1) analyzed in this study is an interior traffic lane of highway bridge deck, simply supported on both sides, span ranging from 40 to 100 feet, lane width 12 feet, girder spacing 6 feet, future wearing surface of $25lb/foot^2$, sustaining an HS20-44 truck load$^1$. Design methods are different for different configurations because of different material properties. For concrete T-beam, Ultimate Strength Design is used. For steel-concrete composite section, Load & Resistance Factor Design is used. For composite material bridge deck, Elastic Design is used.
Figure 1. Design Model
The aspects evaluated for the three configurations are total weight of structure, maintenance, construction method, fatigue behavior, shrinkage and thermal effects, deflection, and cost.
2-1 Current Highway Bridge Deck Structure

Currently, most bridge deck structures are reinforced concrete or steel-concrete composite. For reinforced concrete, cross sections as shown in Figure 2 are typical. It is apparent that the T-beam configuration will save some weight because there is no cross-sectional area under the flange. All the tensile stress is considered to be taken by the steel reinforcements. In spite of this, reinforced concrete structure suffers from large dead load, especially as the span increases. Prestressed concrete may be considered as an alternative. It is good for longer span bridges, but costs more in construction. Steel-concrete composite structures are also widely used in bridge deck systems, but constant maintenance is required to prevent corrosion of the steel beams.

2-2 AASHTO Specification

The American Association of State Highway and Transportation Officials (AASHTO) published Standard Specification
The Ultimate Strength Design (USD) method is established by introducing plastic design concepts into the design criterion. The basic concept of the USD is the assumption that the stress distribution in the compressive zone of the beams is rectangular as shown in Figure 3. For safety reasons, under-reinforced beam (steel reinforcement yields before concrete crushes) is used in the USD since steel provides large ductility that allows large deflection before failure.
2-4 Load & Resistance Factor Design

American Institute of Steel Construction (AISC³) published the Manual of Steel Construction - Load & Resistance Factor Design, which provides an alternative method for the design of beam and girder structures. It is the most recent design criterion for the steel construction. In the current AASHTO¹ specification, Load Factor Design is also an alternative approach to the elastic design method. The design method is based on the requirement that ultimate strength
must be greater than factored loads. The general form of Load Factor Design is

$$\phi R \geq \psi (F_D \times D + F_L \times L)$$

(2.4.1)

Essentially, it is a limit state design method. The limiting conditions such as overloading and failure are specified.
For reinforced concrete bridges, T-beam design is the most popular structural configuration that has been used. In most cases, T-beam bridge design is as easy as rectangular beam design because the compression area provided by T-beam is so large that it is usually capable of resisting the applied bending moments even though the cross-sectional areas under the flanges are omitted. In addition, due to reduction in weight, requirements on load carrying capacity of beams and supports are reduced. Construction costs are generally lower than for rectangular beams.

In this chapter, a design sequence will be followed to derive the approximate weight for a concrete T-beam with span from 40 feet to 100 feet. AASHTO\textsuperscript{1} specification is used. The design model considered here is a simply supported bridge span, T-beam spacing (center to center) of 6 feet and continuous in the transverse direction. Future wearing surface of 25lb/foot\textsuperscript{2} is added on top of beams as shown in Figure 4.
Figure 4. T-beam Design Model
3-1 Ultimate Strength Design (USD)

Considering the uncertainty of material strength, a strength reduction factor $\phi$ is normally included. AASHTO\(^1\) (8.16.1.2.2) suggests that the $\phi$ value for moment is 0.9 and for shear is 0.85. In highway bridge superstructure design, the impact effect should also be included. In this work, any load except live load, dead load, and impact effect is ignored. The factored loading according to AASHTO\(^1\) (3.22) is determined by

$$\phi R = 1.3 \times (D + 1.67 \times L(1 + I))$$ \hspace{1cm} (3.1.1)

where $R$ is the design load carrying capacity of members.

3-2 Thickness of Concrete Slab

The slab thickness will be dominated by two factors, the transverse load resistance requirement and longitudinal load resistance requirement. The larger slab thickness from the two calculations will be taken as the minimum requirement.

In the transverse direction, the slab is designed by taking a unit width (one foot) of the slab as a rectangular beam. This is called the Strip Design Method (Figure 5). In
Continuity factor of 0.8 is applicable to loads.

Figure 5. Strip Design Method

the longitudinal direction, the required slab thickness is designed together with moment resistance of T-beam.

The slab designed for bending moment in accordance with AASHTO\(^1\) (3.24.3), distribution of wheel loads is considered satisfactory in shear (AASHTO\(^1\) 3.24.4). Therefore, shear capacity was not checked in this effort.
### Dimension of T-beam

The effective flange width of T-beams is determined, according to AASHTO\(^1\) (8.10.1), by

\[
b_E \leq \frac{\text{longitudinal span}}{4} \tag{3.3.1}
\]

\[
b_E \leq \text{spacing of T-beam} \tag{3.3.2}
\]

\[
b_E \leq 12 \times \text{slab thickness} \tag{3.3.3}
\]

The web width must be at least capable of (1). embedding an appropriate number of steel reinforcements used in the longitudinal direction, (2). including two times the diameter of shear reinforcements and (3). adding 1.5 inches on both sides for protection of steel reinforcements.

According to AASHTO\(^1\) (8.16.6.2.1), the shear capacity of concrete is \(V_c = 2\sqrt{f_{c'}b_wd}\), and from AASHTO\(^1\) (8.16.6.3.9), the shear capacity of shear reinforcements is limited as \(V_s \leq 8\sqrt{f_{c'}b_wd}\). Shear design is omitted here because the shear capacity normally does not affect the weight of T-beam design except that the shear force needed to be taken by shear reinforcements exceeds the value discussed above. However, shear resistance of concrete is checked instead and must be...
greater than 20% of the shear load to avoid the situation described.

3-4 Deflection

AASHTO\textsuperscript{1} does not give a definite value of deflection limit for reinforced concrete bridge designs, but does give a general rule to follow. For structural steel bridge design (AASHTO\textsuperscript{1} 10.6.2), the deflection induced by live load must be less than 1/800 of the span. For purpose of comparison, this limit is followed for all the design cases considered.

According to AASHTO\textsuperscript{1} (8.13.3), deflection caused by live load is computed by formulas of elastic deflection. However, the effective area moment of inertia is calculated by

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 I_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr} \leq I_g \quad (3.4.1)$$

For each cross-section selected, the cracking moment is determined by $M_{cr} = \frac{f_r I_g}{Y_{top}}$. The maximum moment for deflection calculation is $M_a = \frac{M_{n}}{2}$ for each beam.

Simplified lane loads of HS20-44 truck (AASHTO\textsuperscript{1} 3.7.7) are used for the calculation of deflection as shown in Figure 6. Therefore, the deflection can be calculated by
18 kips for moment and deflection
16 kips for shear

Figure 6. Simplified HS20-44 Truck Load (AASHTO 3.7.7)

\[ \delta_L = \frac{18 \text{kips} \times S^3}{48 E_c I_e} + \frac{5 \times 0.64 \text{kips/foot} \times S^4}{384 E_c I_e} \]  

(3.4.2)

3-5 Design Procedures

The design of T-beams is accomplished in the following steps.

1. Determine the minimum slab thickness by

   \[ t_s \text{ (inch)} = \frac{s \text{ (foot)} + 10}{30} \times 12 \]  

   (3.5.1)

   or 6.5 inches rounded to next larger integer (AASHTO 8.9.2)
2. Calculate moments per foot of slab width. According to AASHTO\(^1\) (3.24.3.1), a continuity factor of 0.8 is applicable. Dead load moment is derived from equation (3.5.2). Live load moment, according to AASHTO\(^1\) equation (3-15), is determined by (3.5.3).

\[
M_D = 0.8 \times \frac{1}{8} \times W_D \times \frac{s}{2} \quad (3.5.2)
\]

\[
M_L = 0.8 \times \frac{(s + 2)}{32} \times \text{wheel load} \quad (3.5.3)
\]

3. Check whether the minimum slab thickness fulfills the flexure strength requirement.

4. Determine the effective flange width by equations (3.3.1-3)

5. According to AASHTO\(^1\) (8.9.2), let effective beam depth equals to \(\frac{s + 9}{18}\) rounded to next larger inch integer. Assume appropriate web width of beam.

6. Calculate dead load by
   a. Adding 25lb/foot\(^2\) future wearing surface on the top.
   b. If two layers of bars are used, add 3.9 inches to \(d\) as height of girder, as shown in Figure 7-a.
   c. If three layers of bars are used, add 5.1 inches to \(d\) as height of girder, as shown in Figure 7-b.
   d. If four layers of bars are used, add 6.4 inches to \(d\) as height of girder, as shown in Figure 7-c.
Figure 7. Determination of Girder Height
7. Calculate dead load moment and shear, and find live load moment and shear from AASHTO Appendix A (also listed in Appendix A of this thesis).

8. Calculate the impact factor from

$$I = \frac{50}{125 + S}$$  \hspace{1cm} (3.5.4)

but \(I \leq 0.3\) according to AASHTO (3.8.2.1).

9. Calculate required moment and shear resistances from equation (3.1.1)

10. Check that shear resistance of concrete is greater than 20% of the minimal shear strength required as discussed in section 3-3; otherwise, the cross-sectional area must be increased (go back to step 5).

11. Examine the dimension provided is a T-beam design or a rectangular beam design by assuming the depth of equivalent rectangular stress block equals to the slab thickness of the beam. That is, if equation (3.5.5) is satisfied, it is a rectangular beam design; otherwise, it is a T-beam design.

$$0.85f_c't_s b_E(d - \frac{t_s}{2}) > \frac{M_n}{2}$$  \hspace{1cm} (3.5.5)

12. If it is a rectangular beam design, use equation (3.5.6) to determine \(a\). If it is a T-beam design, use equation (3.5.7) instead.
0.85 \( f_{c'} a b_{E} (d - \frac{a}{2}) = \frac{M_{u}}{2 \phi} \)  

(3.5.6)

0.85 \( f_{c'} (t_{s}(b_{E} - b_{w})(d - \frac{t_{s}}{2}) + a b_{w}(d - \frac{a}{2})) = \frac{M_{u}}{2 \phi} \)  

(3.5.7)

13. Use either equation (3.5.8) for rectangular beam design or (3.5.9) for T-beam design to calculate required steel area.

\[ A_{s} f_{y} = 0.85 f_{c'} a b_{E} \]  

(3.5.8)

\[ A_{s} f_{y} = 0.85 f_{c'}(t_{s} \times (b_{E} - b_{w}) + a b_{w}) \]  

(3.5.9)

14. Check the web width by arranging reinforcements. If it is not satisfied, go back to step 5.

15. According to AASHTO (8.16.3.3.3), check that the steel area fulfills the maximum and the minimum steel area requirement by

\[ A_{sf} = \frac{0.85 f_{c'}(b_{E} - b_{w})h_{f}}{f_{y}} \]  

(3.5.10)

\[ \rho_{f} = \frac{A_{sf}}{b_{w}d} \]  

(3.5.11)

\[ \rho_{b} = (\frac{b_{w}}{b_{E}})[0.85 \beta_{1} \times \frac{f_{c'}}{f_{y}} \times (\frac{87000}{87000 + f_{y}}) + \rho_{f}] \]  

(3.5.12)

\[ \text{max. } A_{s} = 0.75 \times \rho_{b} \times A_{c} \]  

(3.5.13)
According to AASHTO\textsuperscript{1} (8.17), the minimum steel area required is that $M_u$ provided by the minimum $A_s$ should be able to resist $1.2 \times M_{cr}$ where cracking moment is calculated by

$$M_{cr} = \frac{f_r \times I_g}{y_{top}}$$

If the maximum or the minimum $A_s$ requirements is not fulfilled, cross section must be changed (go back to step 5).

16. Check that the deflection fulfills the requirement, $\delta_L \leq \frac{\text{span}}{800}$ (AASHTO\textsuperscript{1} 10.6.2), as discussed in section 3-4; otherwise, cross-sectional area must be increased (go back to step 5).

3-6 Design Example

By following the design procedures discussed in the previous section, an example illustrating the design of a T-beam bridge is presented in this section. The span of this example is 40 feet.

1. Assume the web width of T-beams is 12 inches; thus, the effective span is $6 \text{feet} - \frac{1}{2} \times \frac{b_w}{12} = 5.5 \text{feet}$. From equation (3.5.1),
\[ \frac{s + 10}{30} = 0.517 \text{ foot} < 0.542 \text{ foot} \quad (3.5.1) \]

Rounded to next larger inch integer, the minimum slab thickness is 7 inches.

2. The density of reinforced concrete is 0.15 kips/foot^3, therefore the distributed dead load per foot width becomes

\[ \hat{W}_D = 0.15 \times \frac{t_s}{12} + 0.025 = 0.113 \text{ kips/foot/foot} \quad (3.6.1) \]

While continuous span is considered in the transverse direction, a continuity factor of 0.8 (AASHTO 3.24.3.1) can be applied to calculation of moments, which results in

\[ M_D = 0.8 \times \frac{1}{8} \times \hat{W}_D \times s^2 = 0.340 \text{ foot} - \text{kips/foot} \quad (3.5.2) \]

\[ M_L = 0.8 \times \frac{(s + 2)}{32} \times 16\text{kips} = 3.00 \text{ foot} - \text{kips/foot} \quad (3.5.3) \]

From equation (3.5.4), the impact factor 
\[ I = \frac{50}{125 + s} = 0.384. \] Therefore, the value of I is 0.3 according to AASHTO (3.8.2.1). From equation (3.1.1), the per foot ultimate moment is

\[ M_U = 1.3 \times (M_D + 1.67 \times (1 + I) \times M_L) = 8.91 \text{ foot} - \text{kips/foot} \quad (3.1.1) \]
3. Use specified concrete strength $f_{c'} = 4,000$ psi, yield strength of reinforcements $f_y = 60,000$ psi, and ratio of depth of rectangular stress block $\beta_1 = 0.85$ (AASHTO\textsuperscript{1} 8.16.2.7), the balanced steel ratio can be calculated by equation (3.6.2) according to AASHTO\textsuperscript{1} (8.16.3.2.2).

$$\rho_b = 0.85 \beta_1 \times \frac{f_{c'}}{f_y} \times \left( \frac{87000}{87000 + f_y} \right) = 0.0285$$  \hspace{1cm} (3.6.2)

The maximum allowable steel ratio is 75 percent of the balanced steel ratio specified by AASHTO\textsuperscript{1} (8.16.3.1.1), that is, $\rho_{max} = 0.0214$. The effective slab depth is equal to the slab thickness minus 2 inch cover for top reinforcements (AASHTO\textsuperscript{1} 8.22.1) and one half the reinforcement diameter (no.6 bar with 0.75 inch diameter). Thus, effective slab depth is 4.62 inches. The depth of equivalent rectangular stress block of concrete can be solved by equating compression of concrete and tension of reinforcements as

$$0.85 f_{c'}'ab = A_s f_y$$  \hspace{1cm} (3.6.3)

Rearrange the equation (3.6.3) and substitute the magnitudes of variables, resulting in $\frac{a}{2} = 0.7353 A_s$. Next, by equating the required moment resistance to provided moment resistance as given by

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\[ \frac{M_u \times 12}{\phi} = A_s f_y (d - \frac{a}{2}) \]  

(3.6.4)

The required tension steel area can be determined by solving

\[ 44.12A_s^2 - 277.2A_s + 118.8 = 0 \]  

(3.6.5)

As \( A_s = 0.463 \text{inch}^2/\text{foot} \) is determined, steel ratio thus equals to \( \frac{A_s}{bd} = 0.0083 \), and is less than \( \rho_{\text{max}} \). In order to check minimum \( A_s \) requirement, several calculations must be made first.

\[ f_r = 7.5 \times \sqrt{f_c'} = 474 \text{psi} \]  

(3.6.6)

\[ I_g = \frac{1}{12} \times 12 \times 7^3 = 343 \text{inch}^4 \]  

(3.6.7)

\[ y_{\text{top}} = \frac{1}{2} \times 7 = 3.5 \text{inches} \]  

(3.6.8)

The cracking moment, \( M_{cr} \), is checked by

\[ M_{cr} = \frac{f_r I_g}{y_{\text{top}}} = 3.87 \text{foot-kips/foot} \leq \frac{M_u}{1.2} \]  

(3.6.9)

Therefore, the minimum steel area requirement is satisfied. That is, \( t_s = 7.0 \text{inches} \) is adequate.

4. From equations (3.3.1-3) and span of 40 feet,
\[
\frac{1}{4} \times S \times 12 = 120 \text{ inches} \tag{3.3.1}
\]

\[
6 \times 12 = 72 \text{ inches} \tag{3.3.2}
\]

\[
12 \times 7 = 84 \text{ inches} \tag{3.3.3}
\]

the effective flange width is 72 inches, which is dominated by equation (3.3.2).

5. Let \( d = 34 \text{ inches} \) and \( b_w = 12 \text{ inches} \).

6. The distributed weight of future wearing surface is 
\[25 \text{ lb/foot}^2 \times 12 \text{ feet} = 0.30 \text{ kips/foot}.\] Use two layers of bars and let beam height equals to 38 inches. Therefore, the distributed dead load can be calculated by

\[
W_D = \left[ \frac{12t_s}{12} + \frac{2(h - 7)b_w}{144} \right] 0.15 + 0.3 = 2.13 \text{kips/foot} \tag{3.6.10}
\]

7. Calculate the dead load moment and shear from

\[
M_D = \frac{1}{8} \times W_D \times S^2 = 425.0 \text{ foot} - \text{kips/lane} \tag{3.6.11}
\]

\[
V_D = W_D \times S \times \frac{1}{2} = 42.5 \text{kips/lane} \tag{3.6.12}
\]

From Appendix A, find live load moment and shear corresponding to a span of 40 feet. These values are 
\[M_L = 449.8 \text{ foot} - \text{kips/lane} \] and \[V_L = 55.2 \text{kips/lane}.\]
8. For a span of 40 feet, the impact effect calculated from equation (3.5.4) is 0.303. Since this value is larger than 0.3, I = 0.3 is used instead, according to AASHTO\(^1\) (3.8.2.1).

9. From equation (3.1.1), calculate the moment and shear due to factored loads by using

\[
M_u = 1.3(M_D + 1.67 \times (1 + I) M_L) = 1822 \text{ foot-kips/lane} \quad (3.6.13)
\]

\[
V_u = 1.3(V_D + 1.67 \times (1 + I) V_L) = 211.0 \text{ kips/lane} \quad (3.6.14)
\]

Nominal moment strength and shear strength are calculated by dividing strength reduction factors using

\[
M_n = \frac{M_u}{\phi} = \frac{1822}{0.9} = 2024 \text{ foot-kips/lane} \quad (3.6.15)
\]

\[
V_n = \frac{V_u}{\phi} = \frac{211.0}{0.85} = 248.3 \text{ kips/lane} \quad (3.6.16)
\]

10. The nominal shear force carried by concrete is calculated from

\[
V_c = 2 \times 2 \sqrt{f_{c'} b_m d} = 103.2 \text{ kips/lane} \quad (3.6.17)
\]

The \(V_c\) must be larger than 20% of the shear force applied on the member; otherwise, an increase in the cross-sectional dimensions of the member is required. This
point is checked by equation (3.6.18). The fraction left of the shear force can be designed to be taken by shear reinforcements.

$$0.2 V_n = 49.7 \text{kips} < 103.2 \text{kips} \quad OK! \quad (3.6.18)$$

11. Use (3.5.5) to check if it is a T-beam design or a rectangular beam design.

$$0.85 f_{c'} t_s b_E (d - \frac{t_s}{2}) = 4355.4 \text{foot-kips} \quad (3.5.5)$$

It is greater than the design moment of each T-beam section, $\frac{M_u}{2\phi}$; thus, it is a rectangular beam design.

12. Use equation (3.5.6) to determine the location of the depth of equivalent rectangular stress block.

$$0.85 f_{c'} a b_E (d - \frac{a}{2}) = \frac{M_n}{2} \text{ (for each beam)} \quad (3.5.6)$$

Therefore, $a = 1.49 \text{ inches} \quad (3.6.19)$

13. From equation (3.5.8), the steel area is determined to be $6.09 \text{inch}^2$.

$$A_s f_y = 0.85 \times f_{c'} \times a \times b_E \quad (3.5.8)$$
14. Use two layer of #8 steel bars, 4 bars each layer, \( A_s = 6.28 \text{inch}^4 \), the spacing between bars is one inch and .75 inch as diameter of shear reinforcements, the required web width is 11.5 inches and is less than 12 inches.

15. Check the maximum and the minimum steel ratios by

\[
A_{sf} = 23.8 \text{inch}^2
\]  
(3.5.10)

\[
\rho_f = 0.0583
\]  
(3.5.11)

\[
\rho_b = 0.0145
\]  
(3.5.12)

max. \( A_s = 0.75\rho_b A_c = 9.52 \text{inch}^4 \geq 6.28 \text{inch}^4 \)  
(3.5.13)

The concrete fracture modulus is 474 psi from (3.6.6). The location of the centroid of concrete cross section is derived as shown in Figure 8, from

\[
b_w \times h \times \frac{h}{2} + (b_e - b_w) \times t_s \times \left( h - \frac{t_s}{2} \right) = 23154 \text{ in}^3
\]  
(3.6.20)

\[
A_c = 876 \text{ inch}^2 \text{(concrete area)}
\]  
(3.6.21)

\[
x' = \frac{23154}{A_c} = 26.43 \text{ inches}
\]  
(3.6.22)

\[
y_{top} = h - 26.43 = 11.57 \text{ inches}
\]  
(3.6.23)
the area moment of inertia of gross concrete section is

\[ I_g = 109112 \text{ inch}^4 \]  

(3.6.24)

For the purpose the deflection calculation, the cracking moment and the maximum moment of deflection computation are derived from

\[ M_{cr} = \frac{f_r I_g}{Y_{top}} = 372.5 \text{kip-foot} \leq \frac{M_u}{1.2} \]  

(3.5.14)

\[ M_a = \frac{M_n}{2} = 1012 \text{kip-foot} \]  

(3.6.25)

Therefore the minimum \( A_s \) requirement is fulfilled.
16. Calculate deflection by determining the location of the neutral axis from equations (3.6.26) to (3.6.28) first. The cracking area moment of inertia is derived from equation (3.6.29).

\[ A_s \times n \times (d - x) = bE \times x \times \frac{x}{2} \quad (3.6.26) \]
\[ 36x^2 + 50.27x - 1709 = 0 \quad (3.6.27) \]
\[ x = 6.23 \text{ inches} \quad (3.6.28) \]
\[ I_{cr} = A_s n (d - x)^2 + \frac{1}{3} \frac{bE x^3}{2} = 44566 \text{ inch}^4 \quad (3.6.29) \]

The effective area moment of inertia is determined from

\[ I_e = \left( \frac{M_{cr}}{M_a} \right)^3 I_g + \left[ 1 - \left( \frac{M_{cr}}{M_a} \right)^3 \right] I_{cr} = 47785 \text{ inch}^4 \leq I_g \quad (3.4.1) \]

Therefore, \( I_e = 47785 \), the deflection is

\[ \delta_L = 0.228 \text{ inch} < \frac{\text{span}}{800} \quad (3.6.30) \]

3-7 Weight of Structure

The design results of spans from 40 feet to 100 feet are listed in Table 1, and the weight curve is shown in Figure 9. By looking at the nominal moments of the T-beams in Table
1, they increase faster as the span increases. Since the moment resistance is the design driver of T-beams of moderate span, a rapid increase in structure weight is expected as shown in Figure 9.

Table 1. Results Of T-beam Design

<table>
<thead>
<tr>
<th>SPAN (feet)</th>
<th>40</th>
<th>60</th>
<th>80</th>
<th>100</th>
</tr>
</thead>
<tbody>
<tr>
<td>t_s (inches)</td>
<td>7</td>
<td>7</td>
<td>7</td>
<td>7</td>
</tr>
<tr>
<td>b_y (inches)</td>
<td>12</td>
<td>14</td>
<td>16</td>
<td>18</td>
</tr>
<tr>
<td>d (inches)</td>
<td>34</td>
<td>46</td>
<td>60</td>
<td>72</td>
</tr>
<tr>
<td>h (inches)</td>
<td>38</td>
<td>51</td>
<td>65</td>
<td>78</td>
</tr>
<tr>
<td>M_u/φ (foot-kips/lane)</td>
<td>2024</td>
<td>4182</td>
<td>7290</td>
<td>11738</td>
</tr>
<tr>
<td>M_n (foot-kips/lane)</td>
<td>2024</td>
<td>4182</td>
<td>7290</td>
<td>11738</td>
</tr>
<tr>
<td>V_u/φ (kips/lane)</td>
<td>248.3</td>
<td>318.0</td>
<td>402.9</td>
<td>510.8</td>
</tr>
<tr>
<td>V_c (kips/lane)</td>
<td>103.2</td>
<td>162.9</td>
<td>242.9</td>
<td>328.0</td>
</tr>
<tr>
<td>a (inches)</td>
<td>1.49</td>
<td>2.29</td>
<td>3.06</td>
<td>4.11</td>
</tr>
<tr>
<td>A_s (inches)</td>
<td>6.28</td>
<td>9.4</td>
<td>12.57</td>
<td>17.28</td>
</tr>
<tr>
<td>WT. of beam (kips/foot)</td>
<td>1.825</td>
<td>2.330</td>
<td>2.983</td>
<td>3.713</td>
</tr>
<tr>
<td>M_cr (foot-kips/lane)</td>
<td>372.5</td>
<td>635.0</td>
<td>976.6</td>
<td>1366.5</td>
</tr>
<tr>
<td>M_l (foot-kips/lane)</td>
<td>449.8</td>
<td>806.5</td>
<td>1164.9</td>
<td>1524.0</td>
</tr>
<tr>
<td>I_c (× 10^3 inch^4)</td>
<td>47.8</td>
<td>125.2</td>
<td>274.5</td>
<td>537.2</td>
</tr>
<tr>
<td>δ_l (inches)</td>
<td>0.228</td>
<td>0.360</td>
<td>0.452</td>
<td>0.537</td>
</tr>
<tr>
<td>span/800 (inch)</td>
<td>0.6</td>
<td>0.9</td>
<td>1.2</td>
<td>1.5</td>
</tr>
</tbody>
</table>
Figure 9. Approximate Weight of T-beam
The strength of a steel-concrete composite section depends on the mechanical interaction between the steel beam and the concrete slab. The steel beam is securely bonded to the concrete slab by carefully designed shear connectors so that the steel beam and the concrete slab act together as a T-beam. A typical cross section is shown in Figure 10.

![Figure 10. Typical Cross Section of Steel-Concrete Composite Section](image)

There are two basic construction methods for steel-concrete composite section and their design procedures are different.
1. **Shored construction**: Temporary shores are used during construction. The concrete slab is supported by shores until the concrete attains 75% of its specified final strength.

2. **Unshored construction**: Temporary shores are not used during construction. The concrete slab is supported by the steel beam.

The purpose of the following design is to derive an approximate weight curve of a composite beam with spans from 40 feet to 100 feet. Shored construction is assumed. Cover plates shown in Figure 10 are not used. Design of shear connectors is ignored. Steel beams considered will be standard rolled W-shapes (AISC\(^3\)). Lateral support of the beams may be necessary for some of the cases, but the design of this portion is omitted and the weight of lateral supports is not included in the final total weight of the structure. The cross section considered is shown in Figure 11.
4-1 Load & Resistance Factor Design (LRFD)

Load & Resistance Factor Design is an alternative method for the design of beam and girder structures of moderate span. In the current AASHTO\textsuperscript{1} specification, Load Factor Design is also an alternative approach to the elastic design method. The general form of Load & Resistance Factor Design is

\[ \phi R \geq \psi (F_D \times D + F_L \times L) \]  

(4.1.1)
According to AASHTO specification, strength reduction factor considered is 0.9 for moment and 0.85 for shear. Analysis factor is 1.3. Dead load factor is 1.0. Live load factor is 1.67. Live load should include appropriate impact effect up to 30% of the live load. Load & Resistance Factor Design equation in the design of steel-concrete composite sections, therefore, has the form of

$$\phi R \geq 1.3[1.0 \times D + 1.67 \times (L + I)]$$  \hspace{1cm} (4.1.2)

4-2 Thickness of Concrete Slab

The requirements for the minimum slab thickness of steel-concrete composite beams are similar to those of reinforced concrete T-beam design. The principal difference is that the web width of T-beam shall be replaced by flange width of the steel beams.

In the design approaches here, the steel-concrete composite section will have the minimum slab thickness required in the transverse direction.

4-3 Dimension of W-section

Three factors are considered in the selection of steel beams. These are moment, shear, and deflection requirements.
Because the steel beam and the concrete slab are taken as a unit, the combined strength and stiffness should be higher than these requirements.

Referring to AISC$^3$, partial design of steel-concrete composite sections will be completed and the corresponding weight evaluated.

4-4 Deflection

According to AASHTO$^1$ (10.6.2), deflection induced by live loads must not exceed $1/800$ of the bridge deck span. For $f_{c'} = 4000$ psi, modular ratio of elasticity, $\frac{E_s}{E_c}$, equals 8, by converting concrete slab area into equivalent steel cross sectional area as shown in Figure 12, the area moment of inertia can be determined. With the simplified live load specified by AASHTO$^1$ (3.7.7), the deflection induced by live load is calculated by elastic formulas.
As discussed in sections 4-1 to 4-4, the design of steel-concrete composite sections is accomplished in the following steps.

1. Similar to the first three steps in section 3-5, determine the minimum slab thickness requirement in transverse direction.
2. Assume appropriate steel beam weight in the longitudinal direction.
3. Calculate dead load from

Figure 12. Equivalent Steel Cross Section
4. Calculate dead load moment and shear from equations (4.5.2) and (4.5.3), get live load moment and shear from Appendix A, then determine the required load resistances from equation (4.1.1).

\[ M_D = \frac{1}{6} \times W_D \times S^2 \]  

(4.5.2)

\[ V_D = W_D \times S \times \frac{1}{2} \]  

(4.5.3)

5. Determine effective concrete flange width from equation (3.3.1-3).

6. Assume depth of equivalent rectangular stress block and depth of steel beam.

7. Using properties of F36 steel, calculate estimated weight of steel beams from equation (4.5.4) \([AISC^3 \text{ (LRFD pp 4-10 c)}]\).

\[ WT = \frac{M_u \times 12}{(-d/2 + Y_{con} - a/2) \phi F_Y} \times 3.4 \]  

(4.5.4)

and check it with the assumed steel beam weight. If large difference is found, go back to step 2.

8. According to WT derived, try appropriate W-sections.
9. Find the maximum $\sum Q_n$ where plastic neutral axis located at the top of steel beam.

10. Calculate required $a$ from equation (4.5.5) (AISC3 4-10 c), and compare with the value assumed in step 6.

\[
a = \frac{\sum Q_n}{0.85f_c' b_E}
\]  

(4.5.5)

If large difference is found, go back to step 6.

11. As shown in Figure 13, determine $Y_2$ from

\[
Y_2 = Y_{\text{con}} - \frac{a}{2}
\]  

(4.5.6)

12. Find factored nominal moment strength, $\phi M_n$, of the corresponding $Y_2$ from AISC3 (c.4) and check with

\[
\phi M_n \geq M_u
\]  

(4.5.7)

If it is not satisfied or the design is too conservative, go back to step 8 and select another W-section.

13. Check shear load resistance by

\[
\phi \times 0.6 \times F_y \times A_w \geq V_u
\]  

(4.5.8)

If it is not satisfied, go back to step 8 and select another W-section.

14. Check deflection induced by live load and impact as discussed in section 4-4. If the deflection requirement is
not satisfied, either go back to step 1 for thicker concrete slab or go back to step 8 for another W-section.

4-6 Design Example

By following the design procedures discussed in the previous section, an example illustrating the design of steel-concrete composite beams is performed in this section. The span of this example is taken as 40 feet.
1. Assume flange width of steel beam is 10 inches, thus the effective span of concrete slab, from equation (4.6.1), is 5.17 feet.

\[ s = 6 - \frac{1}{2} \times \frac{b_f}{12} = 5.17 \text{ feet} \quad (4.6.1) \]

From equation (3.5.1) and AASHTO (8.9.2), determine the minimum slab thickness.

\[ \frac{s + 10}{30} = 0.506 \text{ foot} < 0.542 \text{ foot} \quad (3.5.1) \]

Thus, the minimum slab thickness is 7 inches. Similar to that derived in design example step 2 and step 3 of T-beam, the 7 inch slab thickness fulfills the USD design requirements.

2. Assume weight of steel beams to be 0.15 kips/foot for each traffic lane.

3. The distributed dead load is calculated by

\[ W_D = 0.15 \times \frac{7}{12} \times 12 + 0.30 + W_g = 1.5 \text{ kips/foot} \quad (4.5.1) \]

4. The impact effect for a span of 40 feet is 0.3 as discussed in step 8 of section 3-5. Calculate dead load moment and dead load shear from.

CHAPTER 4 Steel-Concrete Composite Section
From Appendix A, live load moment and shear are

\[ M_L = 449.8 \text{ foot-kip/lane} \]  \hspace{1cm} (4.6.2)

\[ V_L = 55.2 \text{ kips/lane} \]  \hspace{1cm} (4.6.3)

From equation (4.1.1), the factored moment and factored shear are calculated from

\[ M_u = 1.3 \left[ 300 + 1.67 \times 1.3 \times 449.8 \right] = 1660 \text{ foot-kip/lane} \]  \hspace{1cm} (4.6.4)

\[ V_u = 1.3 \left[ 30 + 1.67 \times 1.3 \times 55.2 \right] = 194.8 \text{ kips/lane} \]  \hspace{1cm} (4.6.5)

5. From equations (3.3.1-3), effective flange width is determined.

\[ \frac{40 \times 12}{4} = 120 \text{ inches} \]  \hspace{1cm} (3.3.1)

\[ 6 \times 12 = 72 \text{ inches} \]  \hspace{1cm} (3.3.2)

\[ 12 \times 7 = 84 \text{ inches} \]  \hspace{1cm} (3.3.3)

\[ b_E = 72 \text{ inches} \]  \hspace{1cm} (4.6.6)
6. Assume depth of equivalent rectangular stress block is 3 inches and depth of steel beam is 24 inches.

7. From equation (4.5.4), calculate approximate weight of steel beams.

\[
WT = \frac{1695.5/2 \times 12}{(24/2 + 7 - 3/2) \times 0.85 \times 36} \times 3.4 = 65 \text{ lb/foot} 
\]

(4.5.4)

8. Try W24\times68 steel beams, \( b_f = 9 \text{ inches} \) is very close to the value assumed in step 1.

\[
W_s = 0.136 \text{ kips/foot} < 0.15 \text{ kips/foot} 
\]

(4.6.7)

9. From AISC\textsuperscript{3} 4-20, nominal shear strength of shear connectors is 724 kips.

10. Required \( a \) is determined by

\[
a = \frac{724}{0.85 \times 4 \times 72} = 2.96 \text{ inches} \approx 3 \text{ inches}
\]

(4.5.5)

and the value derived is very close to the assumed value; therefore, no correction is needed.

11. From equation (4.5.6), \( Y_2 \) is determined.

\[
Y_2 = 7 - \frac{a}{2} = 5.5 \text{ inches}
\]

(4.5.6)
12. From AISC$^3$ 4-20, $\phi M_n$ equals 890 foot-kips, which is greater than and is very close to one half of the moment due to factored loads for each traffic lane.

13. From equation (4.5.8), the shear strength of the steel beams is proved to be satisfactory.

$$\phi V_n = 0.85 \times 0.6 \times F_y \times A_w = 181 \text{kips} > \frac{V_u}{2}$$  \hspace{1cm} (4.5.8)

14. As shown in Figure 12, the transformed steel block is a square of 9\text{inch} \times 7\text{inch}. Cross sectional area of a steel beam is 20.1\text{inch}^2. Thus the equivalent area moment of inertia is calculated to be 5689 \text{inch}^4. By substituting the steel elastic modulus equals to $29 \times 10^6$psi, the deflection is calculated from equation (4.6.8) and is less than $\frac{40\text{feet}}{800} = 0.6\text{inch}$.

$$\delta_L = \frac{18\text{kips} \times S^3}{48 E_s I_x} + \frac{5 \times 0.64\text{kips/foot} \times S^4}{384 E_s I_x} = 0.24 \text{inch}$$  \hspace{1cm} (4.6.8)

Same procedures are applied to the cases of different spans and the results are discussed next.
4-7 Weight of Structure

The design results of spans from 40 feet to 100 feet are listed in Table 2, and the weight curve is shown in Figure 14. By looking at the nominal moments of the steel-concrete composite beams in Table 2, they increase faster but not as fast as T-beams as the span increases. Since the concrete slab thickness remains the same and most of the weight comes from concrete, a slower increase in structure weight, compared to T-beam, is expected.

Table 2. Results Of Steel-Concrete Composite Beam Design

<table>
<thead>
<tr>
<th>span (feet)</th>
<th>40</th>
<th>60</th>
<th>80</th>
<th>100</th>
</tr>
</thead>
<tbody>
<tr>
<td>$b_E$ (inches)</td>
<td>72</td>
<td>72</td>
<td>72</td>
<td>72</td>
</tr>
<tr>
<td>$t_s$ (inches)</td>
<td>7</td>
<td>7</td>
<td>7</td>
<td>7</td>
</tr>
<tr>
<td>W-section</td>
<td>W24 ×68</td>
<td>W30 ×108</td>
<td>W36 ×150</td>
<td>W36 ×232</td>
</tr>
<tr>
<td>Wt.of beams(kips/foot)</td>
<td>1.186</td>
<td>1.266</td>
<td>1.350</td>
<td>1.514</td>
</tr>
<tr>
<td>$M_u$ (foot-kips/lane)</td>
<td>1660</td>
<td>3142</td>
<td>4862</td>
<td>6888</td>
</tr>
<tr>
<td>$\phi M_n$ (foot-kips/lane)</td>
<td>1780</td>
<td>3096</td>
<td>4880</td>
<td>7628</td>
</tr>
<tr>
<td>$V_u$ (kips/lane)</td>
<td>194.8</td>
<td>228.9</td>
<td>257.6</td>
<td>287.0</td>
</tr>
<tr>
<td>$\phi V_n$ (kips/lane)</td>
<td>362.0</td>
<td>597.0</td>
<td>822.9</td>
<td>1186.0</td>
</tr>
<tr>
<td>$A_w$ (inch$^2$)</td>
<td>9.85</td>
<td>16.26</td>
<td>22.41</td>
<td>32.29</td>
</tr>
<tr>
<td>$I_x$ (inch$^4$/lane)</td>
<td>11378</td>
<td>23616</td>
<td>41936</td>
<td>65998</td>
</tr>
<tr>
<td>$\delta_L$ (inch)</td>
<td>0.240</td>
<td>0.477</td>
<td>0.758</td>
<td>1.091</td>
</tr>
<tr>
<td>span/800 (inch)</td>
<td>0.6</td>
<td>0.9</td>
<td>1.2</td>
<td>1.5</td>
</tr>
</tbody>
</table>
Figure 14. Weight Curve of Steel-Concrete Composite Beams
The specific moduli (stiffness/weight) and the specific strength (strength/weight) of composite materials are higher than those of concrete and steel. A considerable amount of weight saving is expected. As an example of design, the configuration of composite material bridge deck is shown in Figure 15, simply supported in the longitudinal direction, uniform thickness girder spacing (center to center) of 6 feet, continuous I-shaped floor beams simply supported by the girders. Bolt connections between girders and floor beams are suggested but not actually designed. No reduction on the load resistance caused by holes is considered.

5-1 Elastic Design

Unlike structural steel, fiber reinforced composite materials do not have sufficient ductility to undergo plastic deformation prior to failure. Therefore, design methods involving plastic concepts are not appropriate. Since there is no specific method for composite material bridge design, Elastic Design seems to be the best choice. Maximum strength criterion is widely used in composite design and is adopted.
Figure 15. Composite Material Bridge Deck
for this thesis. The load and resistance are related by considering safety factor equals to 1.67, which results

\[ R \geq 1.67 \times (D + L(1 + I)) \]  

(5.1.1)

**5-2 Material Properties**

All the composite materials considered are continuous fiber reinforced resins. Properties of unidirectional composite materials are listed in Table 3.

<table>
<thead>
<tr>
<th>Material</th>
<th>( E_1 ) (msi)</th>
<th>( E_2 ) (msi)</th>
<th>( X_t ) (ksi)</th>
<th>( Y_t ) (ksi)</th>
<th>( X_c ) (ksi)</th>
<th>( Y_c ) (ksi)</th>
<th>Shear (ksi)</th>
<th>Density (lb/ft³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>T300/5208 Graphite/Epoxy</td>
<td>26.25</td>
<td>1.49</td>
<td>217.5</td>
<td>5.8</td>
<td>217.5</td>
<td>35.7</td>
<td>9.86</td>
<td>99.7</td>
</tr>
<tr>
<td>B(4)/5505 Boron/Epoxy</td>
<td>29.59</td>
<td>2.68</td>
<td>182.7</td>
<td>8.9</td>
<td>362.6</td>
<td>29.3</td>
<td>9.72</td>
<td>124.6</td>
</tr>
<tr>
<td>AS 4/3501 Graphite/Epoxy</td>
<td>20.01</td>
<td>1.3</td>
<td>209.9</td>
<td>7.5</td>
<td>209.9</td>
<td>29.9</td>
<td>13.49</td>
<td>99.7</td>
</tr>
<tr>
<td>Scotchply 1002 Glass/Epoxy</td>
<td>5.6</td>
<td>1.2</td>
<td>154.0</td>
<td>4.5</td>
<td>88.5</td>
<td>17.1</td>
<td>10.44</td>
<td>112.1</td>
</tr>
<tr>
<td>Kevlar 49 Kevlar/Epoxy</td>
<td>11.02</td>
<td>0.8</td>
<td>203.0</td>
<td>1.7</td>
<td>34.1</td>
<td>7.7</td>
<td>4.93</td>
<td>90.96</td>
</tr>
</tbody>
</table>

For bridge deck structures, strength and stiffness requirements differ significantly in different directions. This characteristic matches one of the advantages of composite materials, which is the mechanical properties of the
composite materials can be designed as a function of orientation. Therefore, a member with most of the fibers oriented parallel to the direction in which the largest strength and stiffness are required should be used.

5-3 Deflection

The same deflection requirement as that of concrete T-beam and steel-concrete composite section is applied to the design of composite material bridge deck. Calculation of deflection is based on elastic formulas.

5-4 Floor Beam Design

The cross section of the floor beam is considered as shown in Figure 16. Each floor beam will be connected side by side to form a continuous slab lying between girders. Appropriate connection between girder and floor beam, and between floor beams are assumed.

According to AASHTO\(^1\) (3.30), the tire contact area is considered equal to 0.01P, where wheel load P is 16 kips. The width and length ratio is 1/2.5. That is, the tire contact area is considered a rectangular of 8 inches x 20 inches. Contact pressure is 100 psi. This type of loading as shown
in Figure 17 will be applied on several critical positions of floor beam and design floor beam by considering moment and shear on both directions and punch shear resistance.

Since the top flange has to resist loads acting in both x and y directions as shown in Figure 17, top flange of floor beam is considered having half of the fibers oriented in each direction. The laminate strength will be considered as one half of that of unidirectional laminae in both directions because the transverse strength of a unidirectional lamina is very weak compared to the longitudinal strength. In addi-

CHAPTER 5 Composite Material Bridge Deck

Figure 16. Dimension of Floor Beam
tion, $E_2$ is also much lower than $E_1$. Thus, with the uniform strain assumption, a 90° layer in a laminate has a minor contribution to the laminate strength.

For web and bottom flange, most of the fiber will be considered oriented in the direction perpendicular to the traffic direction. Since specific stacking sequence is not a concern and a unidirectional laminate is not suggested, the strength and stiffness are taken as 85 percent of the properties of a unidirectional laminate. It should be noted that

Figure 17. Live Load on Floor Beams
transverse strength and stiffness will thus be greater than that of the unidirectional laminate.

The following floor beam design procedures are used to determine the floor beam weight as a function of girder width. The results, together with girder design, will be investigated to derive the best design.

Floor Beam Design Procedures:

1. Assume girder width, then determine the clear span of the floor beams as effective flange width minus girder width.
2. According to the simplified design model 1 in Figure 18, design top flange according to moment resistance requirement.
3. According to simplified design model 2, design top flange that fulfills shear requirement.
4. Design top flange according to punch shear requirement and simplified design model 3.
5. From steps 2, 3, and 4, select the maximum value of top flange thickness.
6. According to simplified design model 4, design the cross section of floor beam.
7. Calculate the deflection induced by live load based on simplified design model 4.
Figure 18. Simplified Design Models
5-5 Girder Design

The following girder design procedures are used to determine the girder weight as a function of girder width. The results, together with floor beam design, will be investigated to derive the best design.

1. For a specific girder width, find floor beam weight along the traffic direction from the results of floor beam design in section 5-4.
2. Calculate the weight of future wearing surface in the traffic direction.
3. Calculate dead load with assumed girder weight.
4. Find the maximum live load moment and shear force corresponding to the span considered from Appendix A.
5. Find required strength of girders from equation (5.1.1)
6. Select material.
7. With the constraint equations (5.5.1-3), determine the required depth and thickness of girder.

\[ \sigma_{\text{max}} = \frac{M \times c}{I_x} \leq \text{flexure strength of material} \]  \hspace{1cm} (5.5.1)

\[ \tau_{\text{max}} = \frac{VQ}{It} \leq \text{shear strength of material} \]  \hspace{1cm} (5.5.2)

\[ \delta_{\text{max}} \leq \frac{\text{span}}{800} \quad \text{(due to live load and impact)} \]  \hspace{1cm} (5.5.3)
8. Recalculate the weight of girders and compare with the value assumed in step 3. If the difference is large, go back to step 3.

FORTRAN computer program, DECK (listed in Appendix C), is used to perform these routine procedures. Depth/thickness and width/thickness ratios are restricted to 60 or less to avoid possible local buckling of the structure. Buckling strength of members is not discussed in this effort. The maximum girder width considered is 30 inches.

5-6 Best Design

Since many variables are involved, structural optimization is very complicated. Instead, cases of different conditions are designed. From the results derived, curves will be plotted in order to find the best design of those considered.

Besides the strength and stiffness requirements, two more constraints are added. First, to prevent possible buckling, \( \frac{B}{T} \) and \( \frac{D}{T} \) must be less than 60 as discussed in previous section. Second, to avoid over-deep or over-wide beams, \( \frac{D}{B} \) or \( \frac{B}{D} \) are restricted less than 3.
5-7 Design Example

The case considered in this example is a T300/5208 graphite/epoxy composite bridge deck with a span of 40 feet. The floor beam is designed first, according to design procedures described.

1. Assume width of girder is 22 inches. Clear span of floor beam equals to effective flange width minus B, which is 50 inches.

2. According to simplified design model 1, live load moment is

\[ M_L = 2400\text{lb - inch} \]  \hspace{1cm} (5.7.1)

Dead load moment and shear of simplified design model 1 is negligible compared to live load. Therefore,

\[ M_R = 1.67 \times (1 + 0.3) \times M_L = 5210\text{lb - inch} \]  \hspace{1cm} (5.7.2)

\[ \sigma = \frac{M_R \times t_f}{\frac{1}{12} \times t_f^3} \leq \text{strength of flange} \]  \hspace{1cm} (5.7.3)

\[ \therefore t_f = 0.536\text{inch} \]  \hspace{1cm} (5.7.4)
3. Live load shear according to simplified design model 2 is

\[ V_L = 600lb \]  \hspace{1cm} (5.7.5)

\[ V_R = 1.67 \times (1 + 0.3) \times V_L = 1303lb \]  \hspace{1cm} (5.7.6)

Shear stress is determined by

\[ \tau = \frac{V_R \times Q}{I_x \times t_f} \leq \text{shear strength of flange} \]  \hspace{1cm} (5.7.7)

Thus, the flange thickness required is

\[ t_f = 0.583\text{inch} \]  \hspace{1cm} (5.7.8)

4. Check punch shear requirement according to simplified design model 3.

\[ \tau = \frac{16\text{kips}}{2 \times (20 + 8) \times t_f} \leq \text{shear strength of flange} \]  \hspace{1cm} (5.7.9)

\[ t_f = 0.038\text{inch} \]  \hspace{1cm} (5.7.10)

5. From steps 2, 3, and 4, flange thickness is determined equal to 0.583 inch.

6. Assume weight of floor beam is 25lb/foot in transverse direction. Weight of future wearing surface is 25lb/foot^2 \times 4\text{feet} = 100lb/foot. Thus the distributed dead
load is 10.4 lb/inch. By using simplified design model 4, moment and shear are determined by

\[ M_L = 160 \text{ kip} - \text{inch} \]  \hspace{1cm} (5.7.11)

\[ M_D = 3.3 \text{ kip} - \text{inch} \]  \hspace{1cm} (5.7.12)

\[ M_R = 1.67 \times ((1 + I) \times M_L + M_D) = 352.9 \text{ kip} - \text{inch} \]  \hspace{1cm} (5.7.13)

\[ V_L = 12.8 \text{ kips} \]  \hspace{1cm} (5.7.14)

\[ V_D = 0.26 \text{ kip} \]  \hspace{1cm} (5.7.15)

\[ V_R = 1.67 \times ((1 + I) \times V_L + V_D) = 28.2 \text{ kips} \]  \hspace{1cm} (5.7.16)

Shear is considered taken by webs only and \( \frac{d}{t_w} \) ratio is taken as 12, thus

\[ 2 \times t_w \times d = \frac{V_R}{S} \]  \hspace{1cm} (5.7.17)

\[ t_w = 0.345 \text{ inch} \]  \hspace{1cm} (5.7.18)

\[ d = 4.14 \text{ inches} \]  \hspace{1cm} (5.7.19)

As shown in Figure 19, location of centroid is 0.933 inch from the top and area moment of inertia is 44.76 inch^4. Therefore the flexural stress is checked by

\[ \frac{M_R \times 0.933}{I_x} = 7.35 \text{ksi} \]  \hspace{1cm} (5.7.20)
7. The maximum deflection of simplified design model 4 is 0.050 inch and is small enough compared to 40 feet/800 = 0.6 inch.

Figure 20 shows the results of floor beam design. As the girder width increases, floor beam weight decreases for every composite material considered. Numerical data are listed in Appendix B. These results do not imply that a wider girder will lead to a lighter design because the weight of girders will be increased. As the floor beam design is completed, girder design is performed by following the design procedures discussed in section 5-5.

Girder Design Example:

1. According to the results of floor beam design, weight of floor beams in traffic direction is 71.5 lb/foot.

2. Weight of future wearing surface is 0.3 kip/foot in traffic direction.
3. Assume girder weight is 60 lb/foot per traffic lane, the distributed dead load becomes 0.43kip/foot ( = 0.072 + 0.3 + 0.06 ).

4. From Appendix A, live load moment and shear are 449.8kips-foot and 55.2kips for this case of span equals 40 feet.

5. Impact factor is 0.3 from equation (3.5.4). Dead load moment and shear are derived from

\[ M_D = \frac{1}{8} \times w_D \times 40^2 = 86.8 \text{ foot – kips} \quad (5.7.22) \]
Figure 20. Weight of Floor Beam vs. Girder Width
\[ V_D = \frac{1}{2} \times W_D \times 40 = 8.6 \text{ kips} \]  
\[ (5.7.23) \]

From equation (5.1.1), required moment strength and shear strength are determined.

\[ M_R = 1.67 \times (M_D + (1 + I) \times M_L) = 1120 \text{ foot} - \text{kips} \]  
\[ (5.1.1) \]

\[ V_R = 1.67 \times (V_D + (1 + I) \times V_L) = 134.2 \text{ kips} \]  
\[ (5.1.1) \]

6. T300/5208 Graphite/Epoxy is selected.

7. From equation (5.7.24), required area moment of inertia of girders is determined as 7628 inch\(^4\).

\[ \frac{\text{span}}{800} = (1 + I) \times \left( \frac{18 \times \text{span}^3}{48 E_1 I_x} + \frac{5 \times 0.64 \times \text{span}^4}{384 E_1 I_x} \right) \]  
\[ (5.7.24) \]

For \( \frac{D}{T} = 60 \), D required is 25.1 inches and T is 0.42 inch.

Check flexure stress by

\[ \sigma_t = \frac{M_R \times C_t}{I_x} = 22.1 \text{ ksi} < 0.85 \times 217.5 \text{ ksi} \]  
\[ (5.7.25) \]

\[ \sigma_b = \frac{M_R \times C_b}{I_x} = 22.1 \text{ ksi} < 0.85 \times 217.5 \text{ ksi} \]  
\[ (5.7.26) \]

In order to check shear stress, the maximum static moment is calculated from equation (5.7.27) first. Then, shear stress is checked with equation (5.7.28).
\[ Q_{\text{max}} = B \times T \times \left( \frac{D}{2} - \frac{T}{2} \right) + \left( \frac{D}{2} - T \right)^2 \times \frac{2T}{2} = 175.8 \text{ inch}^3 \]  \hspace{1cm} (5.7.27)

\[ \tau = \frac{VQ}{I_x T} = 3.68 \text{ ksi} < 0.85 \times 9.86 \text{ ksi} \]  \hspace{1cm} (5.7.28)

8. Weight of girders is 54 lb/foot/lane. No correction is needed because the difference between assumed value and calculated value contributes only a small fraction of the total dead load.

The results derived from computer program, DECK, are plotted as Figure 21 shown. The least weight is located at \( B \approx 24\text{ inches} \) which is 123 pound/foot in traffic direction (including only weight of floor beams and girders).

The results of different cases of Composite Material Bridge Decks are shown in Figure 22 and data are listed in Appendix D.

5-8 Weight of Structure

From Figure 22 shown, T300/5208 graphite/epoxy weighs the least and glass/epoxy weighs the most. The weight of glass/epoxy composite bridge decks is about twice as much as those of T300/5208 graphite/epoxy. However, the per pound
Figure 21. Design Results (DEPTH vs. WEIGHT)
Figure 22. Weight Curves of Composite Material Design
price of glass/epoxy is much lower than any of the other composite materials considered here.
CHAPTER 6 RESULTS AND DISCUSSION

According to the results derived in chapters 3, 4, and 5, three different design models are compared and discussed with regard to design characteristics, weight, and deflection. In addition, maintenance, construction method, fatigue behavior, shrinkage and thermal effects, and cost are also discussed in this chapter.

6-1 Characteristics of Design

Essentially, all three designs concentrate on three most important design drivers - moment, shear, and deflection.

The concrete T-beam has shear reinforcements and large cross sectional area to resist shear loads. Except for short span bridges, shear resistance usually does not control concrete T-beam bridge design. This point can be realized simply by looking at the how the required shear and moment strength increases as a function of span in Table 1. Required moment strength increases much faster than required shear strength. That is, flexural resistance will control the design of moderate span bridges before shear stress reaches the allowable value. Since the area moment of inertia of a concrete T-beam
is relatively large, the deflection requirement is usually satisfied.

Steel-concrete composite sections have large concrete slabs as do T-beams, but the concrete slab does not contribute much to shear resistance. In order to simplify the design process, shear load is considered to be taken by the web of the steel beam. As in concrete T-beams, moment resistance controls the design of steel-concrete composite sections. Usually, when a W-section steel beam is selected for moment resistance of bridge design, the web area will be large enough for shear resistance. However, shear stiffners can be used if needed.

Due to the geometry of the cross section selected, composite material bridge decks have much a smaller area moment of inertia. Even though the stiffness for some of the composites is high, deflection control is the most critical point of the design. The design results show that the stress/strength ratios of composites are low. This means that the materials are not fully utilized. To achieve better utilization of material, different cross sectional geometry can be used; for example, uneven thickness girder.

Composite material bridge deck design in this thesis considered only macromechanical design of the bridge deck.
However, micromechanical behavior of composite materials is as important as the macromechanical behavior. In future design consideration, micromechanical behavior should be included.

6-2 Weight of Three Design Models

Figure 23 and Table 4 show that composite material bridges weigh much less than concrete T-beam bridges and steel-concrete composite bridges for those designs considered in this study. By comparing glass/epoxy bridges with other non-composite material bridges, glass/epoxy has a weight approximately 16% of concrete T-beam bridge or 23% of steel-concrete composite bridge. This is a dramatic saving in weight. Several reasons contribute to these results. First, glass/epoxy has a specific density only 75% of that of the reinforced concrete. Second, the strength of glass/epoxy is much higher than those of concrete and steel. Third, the longitudinal stiffness of glass/epoxy is higher than that of concrete. Fourth, the geometric shape of composite materials is flexible.
### Table 4. Weight of Designs (lb/foot/lane)

<table>
<thead>
<tr>
<th>SPAN (FEET)</th>
<th>40</th>
<th>60</th>
<th>80</th>
<th>100</th>
</tr>
</thead>
<tbody>
<tr>
<td>CONCRETE T-BEAM</td>
<td>1644</td>
<td>1900</td>
<td>2440</td>
<td>2946</td>
</tr>
<tr>
<td>STEEL-CONCRETE COMPOSITE BEAM</td>
<td>1186</td>
<td>1266</td>
<td>1650</td>
<td>2070</td>
</tr>
<tr>
<td>T300/5208 Graphite/Epoxy</td>
<td>123</td>
<td>149</td>
<td>185</td>
<td>225</td>
</tr>
<tr>
<td>B(4)/Epoxy Boron/Epoxy</td>
<td>151</td>
<td>180</td>
<td>223</td>
<td>270</td>
</tr>
<tr>
<td>AS/3501 Graphite/Epoxy</td>
<td>122</td>
<td>155</td>
<td>197</td>
<td>242</td>
</tr>
<tr>
<td>Scotchply 1002 Glass/Epoxy</td>
<td>221</td>
<td>300</td>
<td>389</td>
<td>487</td>
</tr>
<tr>
<td>Kevlar 49 Kevlar/Epoxy</td>
<td>199</td>
<td>245</td>
<td>296</td>
<td>353</td>
</tr>
</tbody>
</table>

#### 6-3 Maintenance and Repair

Generally speaking, the expected service life of a concrete structure is about 40 years. Not much maintenance is required unless damage occurs. Corrosion is a problem for steel beams. In some cases, steel beams are covered by concrete for protection, but this method substantially increases dead load. Sometimes the steel beam is designed to have thicker dimensions to compensate for a corroded part. This also increases construction cost, and the appearance is bad. Painting is the most popular method of protection, but regular maintenance is required.
Figure 23. Weight of Designs
When damage occurs or service requirements are changed, replacement or reconstruction of all or part of the bridge often takes as long as (sometimes even longer than) constructing a new bridge. In order to decrease the interruption of traffic flow, a temporary bridge is sometimes required. Thus, the cost may be very high.

Maintenance requirements of composite material bridge decks are not known, but the reliable design can be used to achieve a maintenance free composite design during the life of the structure. When part of the bridge is damaged, composite material bridges may take less time and money to replace or repair since they are light weight and easy to handle.

6-4 Construction Methods

Concrete is used in both T-beam bridges and Steel-concrete composite bridges. Appropriate forms and shores are required to provide shape and support of concrete paste. After the concrete paste has been put into forms, it is necessary to cure the concrete paste for about seven days before the forms and shores can be taken away. Precast concrete members can be used to shorten construction time, but high weight capaci-
ity lifting equipment will be needed to locate the precast members.

Composite material bridge decks can be made close to the site of the bridge if appropriate equipment is provided. This will reduce the work involved in shipping and handling. Even though the initial cost of delivering the equipment is needed, the requirements on the weight lifting equipment will be much less than that of precasted concrete members because the composite materials are light weight. In addition, floor beams and girders can be handled separately.

6-5 Fatigue

Fatigue failure is not a problem for concrete. In fact, strength of concrete increases as a function of time. Yet when steel is used as reinforcement, fatigue behavior of the bridge structures involving steel become very important since the loading on the bridge is cyclic.

Fatigue behavior of composite materials is generally much better than those of metals because the internal damping of composite materials is high. In addition, initial imperfections such as cracks can be larger than those of metals. Fatigue failure of composite materials requires consideration of the fatigue sensitivity of matrix, interfacial
effects, and fiber type. Experimental results (Tsai\textsuperscript{9} chapter 19) show that graphite/epoxy composites have relatively flatter S-N curves than those of metals, especially in the case of unidirectional laminates. After one million cycles, they still have 80% of the initial static strength. Since the stress level in composite material designs is well below the static strength of the materials, reduction in strength due to cyclic load is not as important as reduction in stiffness. The modulus reduction of composite materials is mainly a result of ply cracks and delamination. Test results (Tsai\textsuperscript{9} pp19-12) show that graphite/epoxy with [0/45/90/ – 45]\textsubscript{s} stacking sequence will degrade about 25% on its modulus after 10\textsuperscript{5} cycles. The results of unidirectional laminates tend to be better because graphite fiber is insensitive to fatigue. For other fiber systems, fatigue behavior may vary. However, as long as the internal damping remains high, fatigue behavior of composite materials will be better than metals.

6-6 Shrinkage and Thermal Effects

As concrete shrinks during the curing process, a difference of strain is introduced between concrete and steel. Shrinkage strain results tensile stress to concrete and compressive stress to steel. A strain value of 0.0002 inch/inch is usually assumed for the purpose of shrinkage.
stress calculation. Since concrete is designed to resist compressive stress only, the cracks that are caused by shrinkage have minimal effect on the capacity of concrete.

Because the difference of coefficient of thermal expansion between concrete (0.00055/°F) and steel (0.00065/°F) is small, stresses induced by thermal effects are usually ignored. In some of the cases, however, especially steel-concrete composite beams in hot and sunny weather, the temperature difference between concrete slab and steel beams is large since the steel beams are shaded and the concrete slab is not.

For composite material bridge decks, thermal effects will result in interlaminar stresses. Delamination may be caused by cyclic temperature change. In addition, bearing stresses induced by thermal displacement may cause bearing failure because the bearing strength of composite materials is low. Thus the thermal effects should be carefully considered in future work.

6-7 Deflection

As shown in Figure 24, T-beam bridges have the smallest deflection of those considered. Composite material bridges have the largest deflection. Since the designs of composite
materials are constrained by deflection requirements, if the maximum allowable deflection can be somewhat released, more weight saving in composite material bridges are expected.

6-8 Cost

The costs of bridge construction depend on many factors. By comparing major factors that affect the costs, relative costs can be obtained. Several major factors that affect the costs most are listed and compared in Table 5.
Figure 24. Deflection of Three Design Models

Deflection of Bridges vs. Span

- Composite materials
- Steel concrete composite sections
- T-beam

SPAN (FOOT)

DEFLECTION OF BRIDGES (INCHES)
### Table 5. Factors Affecting The Cost Of Bridges

<table>
<thead>
<tr>
<th></th>
<th>Concrete T-beam Bridge</th>
<th>Steel Concrete Composite Bridge</th>
<th>Composite Material Bridge</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unit price of raw material</td>
<td>very low</td>
<td>low</td>
<td>high</td>
</tr>
<tr>
<td>Normalized Weight Of Structure</td>
<td>1.0</td>
<td>0.69</td>
<td>0.08 to 0.16</td>
</tr>
<tr>
<td>Forms</td>
<td>required</td>
<td>required</td>
<td>not required</td>
</tr>
<tr>
<td>Shores</td>
<td>required</td>
<td>depends</td>
<td>not required</td>
</tr>
<tr>
<td>Maintenance Requirement</td>
<td>low</td>
<td>paint job needed</td>
<td>low</td>
</tr>
<tr>
<td>Time for Repair and Construction</td>
<td>long</td>
<td>long</td>
<td>short</td>
</tr>
<tr>
<td>Service Life</td>
<td>long</td>
<td>long</td>
<td>depends on materials</td>
</tr>
<tr>
<td>Fatigue Behaviors</td>
<td>fair</td>
<td>fair</td>
<td>good</td>
</tr>
<tr>
<td>Normalized Area of Post Required</td>
<td>1.0(based on normal force required)</td>
<td>0.8</td>
<td>0.5</td>
</tr>
<tr>
<td>Transportation and Weight Lifting Requirements</td>
<td>High</td>
<td>High</td>
<td>Low</td>
</tr>
<tr>
<td>Design</td>
<td>Easy</td>
<td>Easy</td>
<td>Complicated and Short of Experience</td>
</tr>
</tbody>
</table>
CHAPTER 7 SUMMARY AND CONCLUSION

With the design results shown, composite material bridge decks appear very competitive to concrete T-beam and steel-concrete composite bridges. The advantages of using composite materials as bridge deck structural materials are following:

1. Weight saving of 84% to 92% on bridge decks and 50% on supports compared to T-beam bridges. To steel-concrete composite bridges, saving is 77% to 88% and 38% separately.
2. Maintenance free design achievable.
3. Easy construction and repair.
5. Flexible cross section.
6. No forms and shores required.

The disadvantages of using composite materials as bridge deck structural materials are following:

1. High material costs.
2. Complicated design due to anisotropic material properties and micromechanical behaviors.
3. Low bearing strength.
4. Larger deflection.
5. Lack of practical experience.

Since the mechanical behavior of the composite materials are very complicated and are not well known, design specifications according to AASHTO\(^1\) should be modified in accordance with experimental results and practical experience. In order to accomplish a complete composite material bridge design, more effort is needed. For instance:

Long term behavior of composite material bridge decks.
Different cross sections and configurations.
Different composite materials.
Connections.
Composite material member manufacture.
Slenderness ratio vs. allowable stresses.
Use of stiffeners.
Modification of specifications.
Composite material column design.
### Table 6. The Maximum Moment and Shear of Highway Bridges, HS20-44 Truck, Simply Supported, One Lane.

<table>
<thead>
<tr>
<th>SPAN (feet)</th>
<th>Moment (foot-kips)</th>
<th>Shear (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>40 feet</td>
<td>449.8</td>
<td>55.2</td>
</tr>
<tr>
<td>50 feet</td>
<td>627.9</td>
<td>58.5</td>
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<td>60 feet</td>
<td>806.5</td>
<td>60.8</td>
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<td>70 feet</td>
<td>985.6</td>
<td>62.4</td>
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<td>80 feet</td>
<td>1164.9</td>
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<tr>
<td>90 feet</td>
<td>1344.4</td>
<td>64.5</td>
</tr>
<tr>
<td>100 feet</td>
<td>1524.0</td>
<td>65.3</td>
</tr>
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</table>
## Table 7. Floor Beams of T300/5208 Graphite/Epoxy

<table>
<thead>
<tr>
<th>Girder width</th>
<th>10&quot;</th>
<th>14&quot;</th>
<th>18&quot;</th>
<th>22&quot;</th>
<th>26&quot;</th>
<th>30&quot;</th>
</tr>
</thead>
<tbody>
<tr>
<td>Span (inch)</td>
<td>62.</td>
<td>58.</td>
<td>54.</td>
<td>50.</td>
<td>46.</td>
<td>42.</td>
</tr>
<tr>
<td>Top flange thickness (inch)</td>
<td>.583</td>
<td>.583</td>
<td>.583</td>
<td>.583</td>
<td>.583</td>
<td>.583</td>
</tr>
<tr>
<td>Web and bottom flange thickness (inch)</td>
<td>.354</td>
<td>.351</td>
<td>.349</td>
<td>.345</td>
<td>.341</td>
<td>.337</td>
</tr>
<tr>
<td>Depth (inch)</td>
<td>4.247</td>
<td>4.217</td>
<td>4.182</td>
<td>4.142</td>
<td>4.095</td>
<td>4.039</td>
</tr>
<tr>
<td>Density (lb/foot$^3$)</td>
<td>99.7</td>
<td>99.7</td>
<td>99.7</td>
<td>99.7</td>
<td>99.7</td>
<td>99.7</td>
</tr>
<tr>
<td>Weight in transverse direction (lb/foot)</td>
<td>23.1</td>
<td>23.0</td>
<td>23.0</td>
<td>22.9</td>
<td>22.8</td>
<td>22.7</td>
</tr>
<tr>
<td>Weight in traffic direction (lb/foot)</td>
<td>89.4</td>
<td>83.4</td>
<td>77.5</td>
<td>71.5</td>
<td>65.6</td>
<td>59.6</td>
</tr>
<tr>
<td>Deflection (inch)</td>
<td>.091</td>
<td>.076</td>
<td>.062</td>
<td>.050</td>
<td>.040</td>
<td>.031</td>
</tr>
</tbody>
</table>
Table 8. Floor Beams of B(4)/5505 Boron/Epoxy

<table>
<thead>
<tr>
<th>Girder width</th>
<th>10&quot;</th>
<th>14&quot;</th>
<th>18&quot;</th>
<th>22&quot;</th>
<th>26&quot;</th>
<th>30&quot;</th>
</tr>
</thead>
<tbody>
<tr>
<td>Span (inch)</td>
<td>62.</td>
<td>58.</td>
<td>54.</td>
<td>50.</td>
<td>46.</td>
<td>42.</td>
</tr>
<tr>
<td>Top flange thickness (inch)</td>
<td>.586</td>
<td>.586</td>
<td>.586</td>
<td>.586</td>
<td>.586</td>
<td>.586</td>
</tr>
<tr>
<td>Web and bottom flange thickness(inch)</td>
<td>.357</td>
<td>.354</td>
<td>.351</td>
<td>.348</td>
<td>.344</td>
<td>.339</td>
</tr>
<tr>
<td>Depth (inch)</td>
<td>4.280</td>
<td>4.249</td>
<td>4.214</td>
<td>4.174</td>
<td>4.126</td>
<td>4.070</td>
</tr>
<tr>
<td>Density(lb/foot³)</td>
<td>124.6</td>
<td>124.6</td>
<td>124.6</td>
<td>124.6</td>
<td>124.6</td>
<td>124.6</td>
</tr>
<tr>
<td>Weight in transverse direction (lb/foot)</td>
<td>29.0</td>
<td>29.0</td>
<td>28.9</td>
<td>28.8</td>
<td>28.7</td>
<td>28.6</td>
</tr>
<tr>
<td>Weight in traffic direction (lb/foot)</td>
<td>112.5</td>
<td>105.0</td>
<td>97.5</td>
<td>90.0</td>
<td>82.5</td>
<td>75.0</td>
</tr>
<tr>
<td>Deflection (inch)</td>
<td>.078</td>
<td>.065</td>
<td>.053</td>
<td>.043</td>
<td>.034</td>
<td>.027</td>
</tr>
</tbody>
</table>

Appendix B. Results Of Composite Material Floor Beam Design
### Table 9. Floor Beams of AS 4/3501 Graphite/Epoxy

<table>
<thead>
<tr>
<th>Girder width</th>
<th>10&quot;</th>
<th>14&quot;</th>
<th>18&quot;</th>
<th>22&quot;</th>
<th>26&quot;</th>
<th>30&quot;</th>
</tr>
</thead>
<tbody>
<tr>
<td>Span (inch)</td>
<td>62.</td>
<td>58.</td>
<td>54.</td>
<td>50.</td>
<td>46.</td>
<td>42.</td>
</tr>
<tr>
<td>Top flange thickness (inch)</td>
<td>.546</td>
<td>.546</td>
<td>.546</td>
<td>.546</td>
<td>.546</td>
<td>.546</td>
</tr>
<tr>
<td>Web and bottom flange thickness (inch)</td>
<td>.303</td>
<td>.301</td>
<td>.298</td>
<td>.295</td>
<td>.292</td>
<td>.288</td>
</tr>
<tr>
<td>Depth (inch)</td>
<td>3.633</td>
<td>3.607</td>
<td>3.577</td>
<td>3.543</td>
<td>3.503</td>
<td>3.454</td>
</tr>
<tr>
<td>Density (lb/foot&lt;sup&gt;3&lt;/sup&gt;)</td>
<td>99.7</td>
<td>99.7</td>
<td>99.7</td>
<td>99.7</td>
<td>99.7</td>
<td>99.7</td>
</tr>
<tr>
<td>Weight in transverse direction (lb/foot)</td>
<td>20.8</td>
<td>20.8</td>
<td>20.7</td>
<td>20.7</td>
<td>20.6</td>
<td>20.6</td>
</tr>
<tr>
<td>Weight in traffic direction (lb/foot)</td>
<td>80.7</td>
<td>75.3</td>
<td>70.0</td>
<td>64.6</td>
<td>59.3</td>
<td>54.0</td>
</tr>
<tr>
<td>Deflection (inch)</td>
<td>.199</td>
<td>.165</td>
<td>.136</td>
<td>.110</td>
<td>.088</td>
<td>.069</td>
</tr>
</tbody>
</table>

Appendix B. Results Of Composite Material Floor Beam Design
### Table 10. Floor Beams of Scotchply 1002 Glass/Epoxy

<table>
<thead>
<tr>
<th>Girder width</th>
<th>10&quot;</th>
<th>14&quot;</th>
<th>18&quot;</th>
<th>22&quot;</th>
<th>26&quot;</th>
<th>30&quot;</th>
</tr>
</thead>
<tbody>
<tr>
<td>Span (inch)</td>
<td>62.</td>
<td>58.</td>
<td>54.</td>
<td>50.</td>
<td>46.</td>
<td>42.</td>
</tr>
<tr>
<td>Top flange thickness (inch)</td>
<td>.840</td>
<td>.840</td>
<td>.840</td>
<td>.840</td>
<td>.840</td>
<td>.840</td>
</tr>
<tr>
<td>Web and bottom flange thickness (inch)</td>
<td>.344</td>
<td>.342</td>
<td>.339</td>
<td>.336</td>
<td>.332</td>
<td>.327</td>
</tr>
<tr>
<td>Depth (inch)</td>
<td>4.130</td>
<td>4.100</td>
<td>4.066</td>
<td>4.027</td>
<td>3.982</td>
<td>3.927</td>
</tr>
<tr>
<td>Density (lb/foot³)</td>
<td>112.1</td>
<td>112.1</td>
<td>112.1</td>
<td>112.1</td>
<td>112.1</td>
<td>112.1</td>
</tr>
<tr>
<td>Weight in transverse direction (lb/foot)</td>
<td>35.2</td>
<td>35.1</td>
<td>35.1</td>
<td>35.0</td>
<td>34.9</td>
<td>34.8</td>
</tr>
<tr>
<td>Weight in traffic direction (lb/foot)</td>
<td>136.3</td>
<td>127.3</td>
<td>118.3</td>
<td>109.3</td>
<td>100.3</td>
<td>91.3</td>
</tr>
<tr>
<td>Deflection (inch)</td>
<td>.415</td>
<td>.346</td>
<td>.285</td>
<td>.231</td>
<td>.185</td>
<td>.145</td>
</tr>
</tbody>
</table>

Appendix B. Results Of Composite Material Floor Beam Design
Table 11. Floor Beams of Kevlar 49/Epoxy

<table>
<thead>
<tr>
<th>Girder width</th>
<th>10&quot;</th>
<th>14&quot;</th>
<th>18&quot;</th>
<th>22&quot;</th>
<th>26&quot;</th>
<th>30&quot;</th>
</tr>
</thead>
<tbody>
<tr>
<td>Span (inch)</td>
<td>62</td>
<td>58</td>
<td>54</td>
<td>50</td>
<td>46</td>
<td>42</td>
</tr>
<tr>
<td>Top flange thickness (inch)</td>
<td>1.352</td>
<td>1.352</td>
<td>1.352</td>
<td>1.352</td>
<td>1.352</td>
<td>1.352</td>
</tr>
<tr>
<td>Web and bottom flange thickness (inch)</td>
<td>.501</td>
<td>.497</td>
<td>.493</td>
<td>.488</td>
<td>.483</td>
<td>.476</td>
</tr>
<tr>
<td>Depth (inch)</td>
<td>6.009</td>
<td>5.967</td>
<td>5.918</td>
<td>5.861</td>
<td>5.794</td>
<td>5.714</td>
</tr>
<tr>
<td>Density (lb/foot³)</td>
<td>91.0</td>
<td>91.0</td>
<td>91.0</td>
<td>91.0</td>
<td>91.0</td>
<td>91.0</td>
</tr>
<tr>
<td>Weight in transverse direction (lb/foot)</td>
<td>47.4</td>
<td>47.3</td>
<td>47.2</td>
<td>47.1</td>
<td>46.9</td>
<td>46.8</td>
</tr>
<tr>
<td>Weight in traffic direction (lb/foot)</td>
<td>183.8</td>
<td>171.6</td>
<td>159.4</td>
<td>147.2</td>
<td>135.0</td>
<td>122.8</td>
</tr>
<tr>
<td>Deflection (inch)</td>
<td>.057</td>
<td>.047</td>
<td>.039</td>
<td>.032</td>
<td>.025</td>
<td>.020</td>
</tr>
</tbody>
</table>

Appendix B. Results Of Composite Material Floor Beam Design
APPENDIX C. FORTRAN PROGRAM "DECK"

* DESIGN OF COMPOSITE BRIDGE DECK
* B : WIDTH OF GIRDER ( INCH )
* BOT : RATIO OF B/T
* D : DEPTH OF GIRDER ( INCH )
* DEFL : DEFLECTION ( INCH )
* DOB : RATIO OF D/B
* EIR : EX*IX REQUIRED FOR DEFLECTION
* (KIP*IN**2)
* EX,EY : EALSTIC MODULI (MSI)
* IMP : IMPACT FACTOR
* IX : MOMENT OF INERTIA IN X DIRECTION
* L : CLEAR SPAN ( FEET )
* MATL : NAME OF MATERIALS
* ML, MD, MR : LIVE, DEAD, AND RESISTING MOMENT
* (FEET-KIP, ONE LANE)
* RHO : DENSITY OF MATERIALS (LB/FEET**3)
* S : SHEAR STRENGTH (KSI)
* SIGX : FLEXURAL STRESS (KSI)
* T : THICKNESS OF GIRDER ( INCH )
* TAUX : SHEAR STRESS (KSI)
* VL, VD, VR : LIVE, DEAD, AND RESISTING SHEAR
* (KIP,ONE LANE)
* WD : DISTRIBUTED DEAD LOAD ( KIP/FEET,
* ONE LANE )
* XC,XT,YC,YT: STRENGTHS OF MATERIALS(KSI)
*
REAL L(7),ML(7),IMP(7),MD(7),MR(7),IX
CHARACTER MATL(5)*30
DIMENSION VL(7),VD(7),VR(7),EX(5),EY(5),EIR(7),
XT(5),XC(5),YT(5),YC(5),S(5),RHO(5)

***************+*************************************
* INPUT SPAN—LENGTHES (FEET), LIVE-LOAD-MOMENTS,
* LIVE-LOAD-SHEARS 7 SETS OF DATA, SPAN FROM 40 TO 100 FEET
DO 10 I=1,7
10 READ(5,*)L(I),ML(I),VL(I)

***************+*************************************
* INPUT MATERIAL PROPERTIES
* 5 MATERIALS
DO 20 I=1,5
READ(5,*)MATL(I),EX(I),EY(I),XT(I),XC(I),YT(I),
*YC(I),S(I),RHO(I)
* FOR LAMINATE, CONSIDER ONLY 85% OF THE PROPERTIES IN
* PRINCIPAL DIR.
EX(I)=.85*EX(I)

Appendix C. FORTRAN PROGRAM "DECK"
EY(I) = 0.85 * EY(I)
XT(I) = 0.85 * XT(I)
XC(I) = 0.85 * XC(I)
YT(I) = 0.85 * YT(I)
YC(I) = 0.85 * YC(I)
S(I) = 0.85 * S(I)

******************************************************************************
* INPUT WIDTH/THICKNESS RATIOS, INITIAL WD ASSUMED
READ(5,*) BOT, WD
******************************************************************************
* I : ORDER OF SPAN, 40 FEET IS 1, 50 FEET IS 2, ETC.
* UNTIL I = 7
* M : MATERIAL TYPE, 1 FOR T300/5208, 2 FOR BORON/EP,
* 3 FOR AS3501 GR/EP
* 4 FOR SCOTCHPLY GL/EP, 5 FOR KEVLAR/EP
* B : GIRDER WIDTH (IN INCH)
* IG: 1 FOR ANOTHER CASE FOLLOWING, 0 FOR NONE
* RATIO : ALLOWABLE D/T OR B/T RATIO
READ(5,*) I, M, B, IG, RATIO
* IC : CONTROLLER, PREVENT FROM BAD WD DERIVED
IC = 0
******************************************************************************
* DERIVE IMPACT FACTOR
CALL IMPFAC(L(I), IMP(I))
******************************************************************************
* ELASTIC DESIGN, CALCULATE REQUIRED MOMENT AND SHEAR RESISTANCES
CALL ED(ML(I), VL(I), MD(I), VD(I), MR(I), VR(I), L(I),
* WD, IMP(I))
******************************************************************************
* ELASTIC DESIGN, CALCULATE REQUIRED EI VALUE TO FULFILL DEFLECTION REQUIREMENT
CALL EIREQ(L(I), IMP(I), EIR(I))
WRITE(6,200) MATL(M)
200 FORMAT(5X,'MATERIAL ',A27)
******************************************************************************
* DETERMINE DEPTH OF GIRDER FROM DEFLECTION REQUIREMENT
CALL NEWTON(B, D, T, EIR(I), EX(M), RATIO)
******************************************************************************
* FLOOR BEAM DESIGN
EF = (EX(M) + EY(M)) / 0.85 / 2.
CALL FBW(B, RHO(M), XT(M), XC(M), S(M), WDB, EX(M), EF)
******************************************************************************
* DERIVE NEW WD
CALL NEWWD(RHO(M), WDB, B, D, T, WD1, WDG)
******************************************************************************
* WITH NEW WD DERIVED, CHECK ASSUMED WD AND REDESIGN GIRDER IF NECESSARY
IF (ABS(WD1/WD - 1.) .GT .03) THEN
WD = WD1
IC=IC+1
IF (IC .GT. 10) THEN
WRITE(6,*)' TROUBLE WITH WD'
GOTO 50
ELSE
END IF
GOTO 30
ELSE
WD=WD1
CALL ED(ML(I),VL(I),MD(I),VD(I),MR(I),VR(I),L(I),WD,
*IMP(I))
END IF

***** DESIGN ANALYSIS
CALL ANAL(B,D,T,MR(I),VR(I),SIGX,TAUX,L(I),DEFL,
*EX(M),IMP(I))

***** OUTPUT
WRITE(6,99)
99 FORMAT(/5X,'GIRDER DESIGN',/)
WRITE(6,110)'SPAN (ft)' ,L(I)
WRITE(6,110)'WIDTH (in)' ,B
WRITE(6,130)'DEPTH (in)' ,D
WRITE(6,130)'THICKNESS (in)' ,T
A=B*D-(B-2.*T)*(D-2.*T)
IX=1./12*(B*D**3-(B-2.*T)*(D-2.*T)**3)
WRITE(6,120)'AREA (in**2)' ,A
WRITE(6,120)'I (in**4)' ,IX
WRITE(6,120)'MU (ft-k)' ,MR(I)
WRITE(6,120)'VU (ft-k)' ,VR(I)
WRITE(6,110)'FLEX. STRESS (ksi)' ,SIGX
WRITE(6,110)'SHEAR (ksi)' ,TAUX
WRITE(6,130)'DEFLECTION (in)' ,DEFL
WRITE(6,130)'WC (k/ft)' ,WDG

WRITE(6,140)WDB+WDG
140 FORMAT(/5X,'TOTAL WEIGHT OF STRUCTURE, PAVE. NOT
*INCLUDED (k/ft)' ,F12.3)
IF (IG .EQ. 1) THEN
GOTO 25
ELSE
END IF
STOP
END

***** DESIGN ANALYSIS
SUBROUTINE ANAL(B,D,T,MR,VR,SIGX,TAUX,L,DEFL,EX,IMP)
REAL IX, MR, L, IMP
SIGX = MR * 12. * D / 2. / IX
QX = (B - T) * T * 0.5 * (D - T) + 2. * T * 0.5 * (D - T) ** 2 * 0.25
TAUX = VR * QX / IX / (2. * T)
DEFL = 5. / 384. * 0.64 * L ** 4
RETURN
END

******************************************************************
* DETERMINE EI VALUE REQUIRED TO FULFILL THE DEFLECTION REQUIREMENT
* SUBROUTINE EIREQ(L, IMP, EIR)
REAL L, IMP
EI1 = 5. / 384. * 0.64 * L ** 4
EIR = (1. + IMP) * (EI1 + EI2) * 800. / L * 144.
EIR = EIR / 2.
RETURN
END

******************************************************************
* FLOOR BEAM DESIGN
* GIVEN WIDTH OF GIRDER B IN INCH
* DETERMINE THE DISTRIBUTED WEIGHT OF FLOOR BEAM ALONG THE TRAFFIC DIRECTION
* SUBROUTINE FBW(B, R, XT, XC, S1, WB, EW, EF)
REAL I1, I2, ML, MR
S = S1 / 0.85
TENF = XT / 0.85 * 0.5
COMF = XC / 0.85 * 0.5
SP = 72. - B
ML = 2400.
MR = 1.67 * ML * 1.3
T1 = (MR / 2. * 12. / MIN(TENF, COMF) / 1000.) ** 0.5
VL = 600.
VR = 1.67 * VL * 1.3
T2 = (VR / 8. * 12. / S / 1000.) ** (1. / 3)
T3 = 16. * 1.3 / 40. / S
T = MAX(T1, T2, T3)
VL = 16. * (SP - 10.) / SP
VD = 10.4 / 2. * SP / 1000.
VR = 1.67 * (1.3 * VL + VD)
TW = (VR / S / 24.) ** 0.5
D = 12. * TW
MR = 1.67 * 1.3 * ML
ZO = 48. * EF / EW
Z1 = ZO * T ** 2 / 2.

Appendix C. FORTRAN PROGRAM "DECK"
Z2 = 2.0*(TW*(D-T-TW)*.5*(D-TW+T/2.))
Z3 = 2.0*(DTW*(D-.5*TW))
Z4 = Z1 + Z2 + Z3
Z5 = T*Z0 + (D*.2 + (D-T-TW)*2.+TW
XB = Z4/Z5
I1 = Z0*T**3/12.+Z0*T*(XB-T/2.)+2*(D*TW)*(D-XB-TW/2.)**2
I2 = 2.0*(TW*(D-TW-TF))*((D-TW-TF)*.5+T-XB)**2+
*TW/12.0*(D-T-TW)**3
I = I1 + I2
SC = MR*XB/I
ST = MR*(D-XB)/I
IF (ST > XT) THEN
WRITE(6,15) SC, XT
ELSE
END IF
CALL DEFLECTION(SP, DEFL, EW, I)
A = (48.*T+(4.*D-2.*T-2.*TW)*TW)/144.
WDT = A*R
WBG = WDT*SP/48.*2.*1.5
WRITE(6,100) B
WRITE(6,101) SP
WRITE(6,102) T
WRITE(6,103) TW
WRITE(6,104) D
WRITE(6,105) R
WRITE(6,106) WDT
WRITE(6,107) WBG
WBG = WBG/1000.
WRITE(6,108) DEFL
100 FORMAT('Girder width (inch)',F5.0)
101 FORMAT('Clear span (inch)',F5.0)
102 FORMAT('Top flange thickness (inch)',F5.3)
103 FORMAT('Web and bottom flange thickness (inch)',F5.3)
104 FORMAT('Depth (inch)',F5.3)
105 FORMAT('Density (lb/foot**3)',F5.1)
106 FORMAT('Weight in transverse direction (lb/foot)',F5.1)
107 FORMAT('Weight in traffic direction (lb/foot)',F5.1)
108 FORMAT('Deflection (inch)',F5.3)
RETURN
END
***********************************************************************
* DETERMINE FLOOR BEAM DEFLECTION
SUBROUTINE DEFLECTION(B, DEFL, E, I)
REAL I
EI = E*I
Y = (B-20.)/2.
DEFL = 1./6.*B**3+0.8*Y**4/48.*
*0.8*(B**4/256.-Y*B**3/48.+Y*Y*B*B/32.)
DEFL=DEFL*2./EI*1000.*1.3
RETURN
END
*********************************************************
* DETERMINE IMPACT FACTOR
SUBROUTINE IMPFAC(L,IMP)
REAL L, IMP
IMP=50./(L+125.)
IF (IMP .GT. 0.3) THEN
IMP=0.3
ELSE
END IF
RETURN
END
*********************************************************
* ELASTIC DESIGN TO DETERMINE DESIGN LOAD
SUBROUTINE ED(ML,VL,MD,VD,MR,VR,L,WD,IMP)
REAL ML, MD, MR, L, IMP
MD=0.125*WD*L**2
VD=.5*WD*L
MR=1.67*(MD+(1.+IMP)*ML)
VR=1.67*(VD+(1.+IMP)*VL)
MR=MR/2.
VR=VR/2.
RETURN
END
*********************************************************
* NEWTON METHOD IS USED TO DETERMINE DEPTH REQUIRED TO
* FULFILL DEFLECTION REQUIREMENT
SUBROUTINE NEWTON(B,D,T,EIR,E,RATIO)
I=0
D=40.
10 T=MAX(B,D)/RATIO
   C1=T/6.
   C2=T/2.*(B-2.*T)
   C3=-T**2*(B-2.*T)
   C4=+T**3*8./12.*(B-2.*T)-EIR/(E*1000.)
   F=C1*D**3+C2*D*D+C3*D+C4
   FP=3.*C1*D*D+2.*C2*D+C3
   D1=D-F/FP
   I=I+1
   IF (I .GT. 10) THEN
      WRITE(6,*) 'NEWTON'S METHOD FAILED'
      RETURN
   ELSE
   END IF
   IF (ABS(D1-D) .LT. .01) THEN
      D=D1
   RETURN
   END
Appendix C. FORTRAN PROGRAM "DECK" 93
ELSE
D=D1
GOTO 10
END IF
END

*******************
* DETERMINE DISTRIBUTED DEAD LOAD
SUBROUTINE NEWWD(RHO, WDB, B, D, T, WD, WDG)
A=B*D-(B-2.*T)*(D-2.*T)
WDG=RHO*A*2./12.*2/1000.
WD=WDG+WDB+0.15*2.
RETURN
END
APPENDIX D. RESULTS OF GIRDER DESIGN

Table 12. T300/5208 Graphite/Epoxy

<table>
<thead>
<tr>
<th>Girder width (inch)</th>
<th>24.0</th>
<th>30.0</th>
<th>30.0</th>
<th>30.0</th>
</tr>
</thead>
<tbody>
<tr>
<td>Floor beam weight (kips/foot)</td>
<td>.069</td>
<td>.060</td>
<td>.060</td>
<td>.060</td>
</tr>
<tr>
<td>SPAN (feet)</td>
<td>40.0</td>
<td>60.0</td>
<td>80.0</td>
<td>100.0</td>
</tr>
<tr>
<td>Girder depth (inch)</td>
<td>24.610</td>
<td>31.792</td>
<td>39.724</td>
<td>47.295</td>
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<td>Thickness (inch)</td>
<td>.410</td>
<td>.530</td>
<td>.662</td>
<td>.788</td>
</tr>
<tr>
<td>Area (inch²)</td>
<td>39.20</td>
<td>64.35</td>
<td>90.59</td>
<td>119.35</td>
</tr>
<tr>
<td>I (inch⁴)</td>
<td>3803.43</td>
<td>10329.5</td>
<td>21407.7</td>
<td>38125.1</td>
</tr>
<tr>
<td>Mu (foot-kips)</td>
<td>558.90</td>
<td>1024.0</td>
<td>1534.0</td>
<td>2103.2</td>
</tr>
<tr>
<td>Vu (kips)</td>
<td>66.98</td>
<td>75.73</td>
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<td>88.56</td>
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<td>Flex. stress (ksi)</td>
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<td>18.9</td>
<td>17.1</td>
<td>15.7</td>
</tr>
<tr>
<td>Shear stress (ksi)</td>
<td>3.80</td>
<td>2.85</td>
<td>1.83</td>
<td>1.42</td>
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<tr>
<td>Deflection (inch)</td>
<td>.600</td>
<td>0.900</td>
<td>1.200</td>
<td>1.500</td>
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<tr>
<td>Girder weight (kips/foot)</td>
<td>.054</td>
<td>0.089</td>
<td>.125</td>
<td>.165</td>
</tr>
<tr>
<td>Total weight of structure (kips/foot)</td>
<td>.123</td>
<td>0.149</td>
<td>.185</td>
<td>.225</td>
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Table 13. B(4)/5505 Boron/Epoxy

<table>
<thead>
<tr>
<th>Girder width (inch)</th>
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<th>30.0</th>
<th>30.0</th>
<th>30.0</th>
</tr>
</thead>
<tbody>
<tr>
<td>Floor beam weight (kips/foot)</td>
<td>.086</td>
<td>.075</td>
<td>.075</td>
<td>.075</td>
</tr>
<tr>
<td>SPAN (feet)</td>
<td>40.0</td>
<td>60.0</td>
<td>80.0</td>
<td>100.0</td>
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<tr>
<td>Girder depth (inch)</td>
<td>23.600</td>
<td>30.635</td>
<td>38.303</td>
<td>45.622</td>
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<tr>
<td>Thickness (inch)</td>
<td>.400</td>
<td>.511</td>
<td>.639</td>
<td>.760</td>
</tr>
<tr>
<td>Area (inch²)</td>
<td>37.44</td>
<td>60.89</td>
<td>85.60</td>
<td>112.67</td>
</tr>
<tr>
<td>I (inch⁴)</td>
<td>3374.1</td>
<td>9163.6</td>
<td>18991.2</td>
<td>33821.7</td>
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<tr>
<td>Mu (foot-kips)</td>
<td>563.58</td>
<td>1035.9</td>
<td>1559.4</td>
<td>2150.2</td>
</tr>
<tr>
<td>Vu (kips)</td>
<td>67.45</td>
<td>76.52</td>
<td>83.53</td>
<td>90.44</td>
</tr>
<tr>
<td>Flex. stress (ksi)</td>
<td>23.7</td>
<td>20.8</td>
<td>18.9</td>
<td>17.4</td>
</tr>
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<td>Shear stress (ksi)</td>
<td>4.08</td>
<td>2.80</td>
<td>2.00</td>
<td>1.55</td>
</tr>
<tr>
<td>Deflection (inch)</td>
<td>.600</td>
<td>0.900</td>
<td>1.200</td>
<td>1.500</td>
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<tr>
<td>Girder weight (kips/foot)</td>
<td>.065</td>
<td>0.105</td>
<td>.148</td>
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<tr>
<td>Total weight of structure (kips/foot)</td>
<td>.151</td>
<td>0.180</td>
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Appendix D. Results of girder design
### Table 14. AS 4/3501 Graphite/Epoxy

<table>
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<tr>
<th>Girder width (inch)</th>
<th>26.0</th>
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<th>30.0</th>
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<tbody>
<tr>
<td>Floor beam weight (kips/foot)</td>
<td>.059</td>
<td>.054</td>
<td>.054</td>
<td>.054</td>
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<tr>
<td>SPAN (feet)</td>
<td>40.0</td>
<td>60.0</td>
<td>80.0</td>
<td>100.0</td>
</tr>
<tr>
<td>Girder depth (inch)</td>
<td>26.262</td>
<td>34.556</td>
<td>43.130</td>
<td>51.301</td>
</tr>
<tr>
<td>Thickness (inch)</td>
<td>.438</td>
<td>.576</td>
<td>.719</td>
<td>.855</td>
</tr>
<tr>
<td>Area (inch(^2))</td>
<td>44.99</td>
<td>73.02</td>
<td>103.09</td>
<td>136.09</td>
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<tr>
<td>I (inch(^4))</td>
<td>4989.5</td>
<td>13550.8</td>
<td>28083.5</td>
<td>50014.2</td>
</tr>
<tr>
<td>Mu (foot-kips)</td>
<td>558.67</td>
<td>1026.4</td>
<td>1541.7</td>
<td>2121.5</td>
</tr>
<tr>
<td>Vu (kips)</td>
<td>66.96</td>
<td>75.89</td>
<td>82.65</td>
<td>89.29</td>
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<tr>
<td>Flex. stress (ksi)</td>
<td>17.6</td>
<td>15.7</td>
<td>14.2</td>
<td>13.1</td>
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<tr>
<td>Shear stress (ksi)</td>
<td>3.33</td>
<td>2.21</td>
<td>1.58</td>
<td>1.22</td>
</tr>
<tr>
<td>Deflection (inch)</td>
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<td>0.900</td>
<td>1.200</td>
<td>1.500</td>
</tr>
<tr>
<td>Girder weight (kips/foot)</td>
<td>.062</td>
<td>0.101</td>
<td>0.143</td>
<td>.188</td>
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<tr>
<td>Total weight of structure (kips/foot)</td>
<td>.122</td>
<td>0.155</td>
<td>.197</td>
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</table>

Appendix D. Results of girder design 97
Table 15. Scotchply 1002 Glass/Epoxy

<table>
<thead>
<tr>
<th>Girder width (inch)</th>
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<th>30.0</th>
<th>30.0</th>
<th>30.0</th>
</tr>
</thead>
<tbody>
<tr>
<td>Floor beam weight (kips/foot)</td>
<td>.091</td>
<td>.091</td>
<td>.091</td>
<td>.091</td>
</tr>
<tr>
<td>SPAN (feet)</td>
<td>40.0</td>
<td>60.0</td>
<td>80.0</td>
<td>100.0</td>
</tr>
<tr>
<td>Girder depth (inch)</td>
<td>37.577</td>
<td>50.807</td>
<td>63.071</td>
<td>74.675</td>
</tr>
<tr>
<td>Thickness (inch)</td>
<td>.626</td>
<td>.847</td>
<td>1.051</td>
<td>1.245</td>
</tr>
<tr>
<td>Area (inch²)</td>
<td>83.07</td>
<td>133.97</td>
<td>191.23</td>
<td>254.38</td>
</tr>
<tr>
<td>I (inch⁴)</td>
<td>17828.6</td>
<td>48419.7</td>
<td>100348.4</td>
<td>178711.4</td>
</tr>
<tr>
<td>Mu (foot-kips)</td>
<td>572.22</td>
<td>1080.9</td>
<td>1670.3</td>
<td>2377.2</td>
</tr>
<tr>
<td>Vu (kips)</td>
<td>68.62</td>
<td>79.52</td>
<td>89.08</td>
<td>99.52</td>
</tr>
<tr>
<td>Flex. stress (ksi)</td>
<td>7.3</td>
<td>6.8</td>
<td>6.3</td>
<td>6.0</td>
</tr>
<tr>
<td>Shear stress (ksi)</td>
<td>1.70</td>
<td>1.11</td>
<td>.83</td>
<td>.67</td>
</tr>
<tr>
<td>Deflection (inch)</td>
<td>.600</td>
<td>0.900</td>
<td>1.200</td>
<td>1.500</td>
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<tr>
<td>Girder weight (kips/foot)</td>
<td>.129</td>
<td>0.209</td>
<td>.298</td>
<td>.396</td>
</tr>
<tr>
<td>Total weight of structure(kips/foot)</td>
<td>.221</td>
<td>0.300</td>
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Appendix D. Results of girder design
## Table 16. Kevlar 49/Epoxy

<table>
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<th>30.0</th>
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<td>Girder width (inch)</td>
<td>30.0</td>
<td>30.0</td>
<td>30.0</td>
<td>30.0</td>
</tr>
<tr>
<td>Floor beam weight (kips/foot)</td>
<td>.123</td>
<td>.123</td>
<td>.123</td>
<td>.123</td>
</tr>
<tr>
<td>SPAN (feet)</td>
<td>40.0</td>
<td>60.0</td>
<td>80.0</td>
<td>100.0</td>
</tr>
<tr>
<td>Girder depth (inch)</td>
<td>30.528</td>
<td>41.443</td>
<td>51.599</td>
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<td>Thickness (inch)</td>
<td>.509</td>
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<td>Area (inch(^2))</td>
<td>60.58</td>
<td>96.77</td>
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<tr>
<td>I (inch(^4))</td>
<td>9059.9</td>
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</tr>
<tr>
<td>Mu (foot-kips)</td>
<td>571.7</td>
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<td>Vu (kips)</td>
<td>68.26</td>
<td>78.15</td>
<td>85.98</td>
<td>93.90</td>
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<td>Flex. stress (ksi)</td>
<td>11.6</td>
<td>10.7</td>
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<td>9.1</td>
</tr>
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<td>Shear stress (ksi)</td>
<td>2.52</td>
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<td>1.17</td>
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<tr>
<td>Deflection (inch)</td>
<td>.600</td>
<td>0.900</td>
<td>1.200</td>
<td>1.500</td>
</tr>
<tr>
<td>Girder weight (kips/foot)</td>
<td>.077</td>
<td>0.122</td>
<td>.174</td>
<td>.230</td>
</tr>
<tr>
<td>Total weight of structure (kips/foot)</td>
<td>.199</td>
<td>0.245</td>
<td>.296</td>
<td>.353</td>
</tr>
</tbody>
</table>

Appendix D. Results of girder design
REFERENCES

2. ACI, "Building Code Requirements for Reinforced Concrete", ACI 318-83 revised 1986, American Concrete Institute 1986
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