COMPARATIVE STRUCTURAL DESIGN EXAMPLE: WWR (ACI 318) VS. GFRP REBAR (ACI 440.11)

May 2026

DESIGN EXAMPLE INPUTS:

- SUSPENDED ONE-WAY REINFORCED CONCRETE FLOOR SLAB
- MULTI-SPAN WITH MAX SPAN = 22.5 ft (SLAB IS INTEGRAL WITH SUPPORTS)
- SLAB THICKNESS, h =14 inches
- SLAB EFFECTIVE DEPTH TO TENSILE REINFORCEMENT, d = 12 inches
- NORMALWEIGHT CONCRETE = 145 pcf, λ = 1.0
- MAXIMUM CONCRETE COMPRESSIVE STRAIN AT CRUSHING, ε_{cu} = 0.003 in/in
- CONCRETE COMPRESSIVE STRENGTH, f'_c = 4 ksi, β_1 = 0.85
- SUPERIMPOSED UNIFORM DEAD LOAD = 15 psf
- SUPERIMPOSED UNIFORM LIVE LOAD = 80 psf

FLEXURAL DEMAND M_u
= [1.2(.145 X 14/12) + 1.2(0.015) + 1.6(0.080)] X 22.52/ 8 = ***22.1 kip-ft per linear foot of slab width***
(Flexural demand is conservatively calculated based on a simply-supported slab condition.)

1. TENSILE REINFORCEMENT PROPERTIES

DEFORMED WELDED WIRE REINFORCEMENT	GFRP REBAR
YIELD STRENGTH, f_y = 70,000 psi MODULUS OF ELASTICITY OF STEEL REINFORCEMENT, E_s = 29,000 ksi	MANUFACTURER GUARANTEED TENSILE STRENGTH, f_{fu^*} = 145,000 psi MODULUS OF ELASTICITY OF GFRP REINFORCEMENT, E_f = 8,700 ksi ENVIRONMENTAL REDUCTION FACTOR, C_E = 0.85

2. PRESCRIPTIVE MINIMUM TENSILE REINFORCEMENT

DEFORMED WELDED WIRE REINFORCEMENT	GFRP REBAR
ACI 318 SECTION 7.6.1.1 FOR STEEL IN ONE-WAY SLAB 0.0018 X A_g = 0.0018 X 12" X 14" = 0.303 in^2 per linear foot of slab width $A_{WWR,min}$ = 0.303 in^2 per linear foot	ACI 440.11 SECTION 7.6.1.1 FOR GFRP IN ONE-WAY SLAB Minimum GFRP area per linear foot of slab width is the greater of: $\frac{300}{f_{fu}} \times A_g = \frac{300}{f_{fu}^* \times C_E} \times A_g = \frac{300}{145{,}000 \times 0.85} \times 12 \times 14 = 0.409\ in^2$ And $\frac{20{,}000}{E_f} \times A_g = \frac{20{,}000}{8{,}700{,}000} \times 12 \times 14 = 0.386\ in^2$ $A_{GFRP,min}$ = 0.409 in^2 per linear foot

3. REINFORCED CONCRETE STRENGTH DESIGN

For the sake of direct comparison, we will utilize equal areas of WWR and GFRP to start. For this example, we will begin with the previously calculated value for minimum area of GFRP tensile reinforcement.
So, $A_s = A_{WWR} = 0.409$ in^2 per linear foot › $A_{WWR,min}$, and $A_f = A_{GFRP} = 0.409$ in^2 per linear foot.

DEFORMED WELDED WIRE REINFORCEMENT

$\rightarrow Need\ T = C$
$T = A_s f_y = 0.409 \times 70 = 28.63\ kips$
$C = 0.85 \times f'_c \times b \times a = 0.85 \times f'_c \times b \times \beta_1 \times c$

$where\ c\ is\ the\ neutral\ axis\ location.$

$28.63 = 0.85 \times 4 \times 12 \times .85 \times c$
$c = 0.8255\ inches$

Nominal Flexural Strength (per foot width of slab)

$$M_n = A_s f_y \left(d - \frac{a}{2}\right) = 0.409 \times 70 \times \left(12 - \frac{0.85 \times 0.8255}{2}\right) = 27.79\ kip \cdot ft$$

Tensile strain in steel:

$$\varepsilon_s = \frac{0.003d}{c} - 0.003 = \frac{0.003 \times 12}{.8255} - 0.003 = 0.0406\ in/in$$

Per ACI 318 Table 21.2.2, tension-controlled sections are those with net tensile strain in reinforcement ≥ 0.0054 in/in for 70 ksi reinforcement. The corresponding phi factor is 0.90.

Design Flexural Strength with A_s = 0.409 in^2

$\varphi M_n = 0.90 \times 27.79 = 25\ kip \cdot ft, per\ foot\ width\ of\ slab$

→ NOTE: Due in part to the ductile inelastic response of steel, 90% of the calculated nominal strength is usable per ACI 318.

Demand-to-Capacity Ratio (DCR)

$$\frac{M_u}{\varphi M_n} = \frac{22.1}{25} = 0.884 < 1.0\ \therefore strength\ is\ adequate$$

GFRP REBAR

$\rightarrow Need\ T = C$

$T = A_f E_f \varepsilon_{ft}$
$where\ \varepsilon_{ft}\ is\ the\ tensile\ strain\ in\ reinforcement.$

$$\varepsilon_{ft} = lesser\ of\ \left(\varepsilon_{cu} \times \frac{d - c}{c}\right)\ and\ \varepsilon_{fu}$$

$where\ \varepsilon_{fu}\ is\ the\ reinforcement\ design\ rupture\ strain.$

$$\varepsilon_{fu} = \frac{f_{fu}}{E_f} = \frac{145 \times 0.85}{8{,}700} = 0.014167\ in/in$$

$C = 0.85 \times f'_c \times b \times \beta_1 \times c$

$where\ c\ is\ the\ neutral\ axis\ location.$

$$Neutral\ axis\ at\ balanced\ condition\ c_{bal} = \frac{\varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_{fu}} \times d = 2.097\ inches$$

$We\ will\ iterate\ c\ until\ we\ arrive\ at\ C = T.$

$Start\ with\ c = 2\ inches.$
$T = 50.41\ kips$
$C = 69.36\ kips > 50.41\ kips$
$T \neq C$

$\therefore try\ c = 1\ inch.$
$T = 50.41\ kips$
$C = 34.68\ kips < 50.41\ kips$
$T \neq C$

$$\therefore try\ c = \frac{50.41 - 34.68}{69.36 - 34.68} \times (2 - 1) + 1 = 1.4536\ inches$$

$T = 50.41\ kips$
$C = 50.41\ kips$
$T = C$
$\therefore neutral\ axis\ c = 1.4526\ inches$

Nominal Flexural Strength (per foot width of slab)

$$M_n = T \times arm = 50.41 \times \left(d - \frac{a}{2}\right) = 50.41 \times \left(12 - \frac{.85 \times 1.4526}{2}\right)$$

$M_n = 47.82\ kip \cdot ft$

Tensile strain ε_{ft} in GFRP in this case equals rupture strain = 0.014167 in/in. Per ACI 440.11 Table 21.2.2, this is a tension/rupture-controlled section. The corresponding phi factor is 0.55.

Design Flexural Strength with A_f = 0.409 in^2

$\varphi M_n = 0.55 \times 47.82 = 26.3\ kip \cdot ft, per\ foot\ width\ of\ slab$

→ NOTE: Due in part to the non-ductile fracture response of GFRP, only 55% of the calculated nominal strength is usable per ACI 440.11.

Demand-to-Capacity Ratio (DCR)

$$\frac{M_u}{\varphi M_n} = \frac{22.1}{26.3} = 0.84 < 1.0\ \therefore strength\ is\ adequate$$

4. REINFORCED CONCRETE SERVICEABILITY (DEFLECTION) CHECK – PRESCRIPTIVE MINIMUM SLAB THICKNESS

Because the stiffness of GFRP is only about 30% that of steel, GFRP-reinforced concrete members exhibit significantly larger service-level deflections (and corresponding cracking) when compared to a steel-reinforced concrete member of identical proportioning and reinforcement area.

<table>
<tr>
<th>DEFORMED WELDED WIRE REINFORCEMENT</th>
<th>GFRP REBAR</th>
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<tr>
<td>ACI 318 allows the designer to bypass detailed deflection calculations for one-way steel-reinforced concrete slab provided the concrete slab proportioning is per Section 7.3.1.1. Here we see that the minimum thickness of the slab, assuming a multiple bay configuration wherein the slab is integral with supports, would be as follows:

$h_{min} = \frac{l}{24} \times \left(0.4 + \frac{f_y}{100{,}000}\right) = \frac{22.5 \times 12}{24} \times \left(0.4 + \frac{70{,}000}{100{,}000}\right)$

$h_{min} = 12.375\ inches < 14\ inches\ \therefore slab\ thickness\ sufficient$

In practice, then, the designer would be able to proceed with the 14-inch slab thickness without carrying out detailed deflection calculations. For this example, however, we will carry out detailed deflection calculations to illustrate the difference between serviceability performance of steel reinforcement and GFRP reinforcement.

We will use ACI 318 Section 24.2 to calculate deflections of the steel-reinforced slab due to service-level gravity loads. Deflections are based on an idealized simply-supported slab.</td>
<td>ACI 440.11 does not contain provisions for prescriptive minimum slab thicknesses.

We must use ACI 440.11 Section 24.2 to calculate deflections of the GFRP-reinforced slab due to service-level gravity loads. Deflections are based on an idealized simply-supported slab.</td>
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5. REINFORCED CONCRETE SERVICEABILITY (DEFLECTION) CHECK – DETAILED CALCULATION

DEFORMED WELDED WIRE REINFORCEMENT

ACI 318 Section 24.2

$Cracking\ moment, M_{cr} = \frac{f_r I_g}{y_t}$

where:

$Modulus\ of\ rupture, f_r = 7.5 \times \lambda \times \sqrt{f'_c} = 474.34\ psi$

$Gross\ moment\ of\ inertia, I_g = \frac{b \times h^3}{12} = 2{,}744\ in^4\ per\ foot\ width$

$Centroidal\ axis\ of\ gross\ section, y_t = \frac{14\ inches}{2} = 7\ inches$

$Cracking\ moment, M_{cr} = \frac{f_r I_g}{y_t} = 185{,}941\ lb \cdot in\ per\ foot\ width\ of\ slab$

$Cracking\ moment, M_{cr} = 15.495\ kip \cdot ft\ per\ foot\ width\ of\ slab$

$Effective\ moment\ of\ inertia, I_e \rightarrow if\ M_a \leq (2/3)M_{cr}$:

$I_e = I_g$

$Effective\ moment\ of\ inertia, I_e \rightarrow if\ M_a > (2/3)M_{cr}$:

$$I_e = \frac{I_{cr}}{1 - \left(\frac{(2/3)M_{cr}}{M_a}\right)^2 \times \left(1 - \frac{I_{cr}}{I_g}\right)}$$

Moment of inertia of cracked section transformed to concrete, I_{cr}:

$$I_{cr} = \frac{b \times (kd)^3}{3} + n_s A_s \times (d - kd)^2$$

$where\ k = \sqrt{2\rho_s n_s + (\rho_s n_s)^2} - \rho_s n_s$

$where\ reinforcement\ ratio, \rho_s = \frac{A_s}{bd} = \frac{0.409}{12 \times 12} = 0.00284$

$where\ modular\ ratio, n_s = \frac{E_s}{E_c} = \frac{29{,}000{,}000}{57{,}000 \times \sqrt{f'_c}} = 8.044$

$\rho_s n_s = 0.02284$

$where\ k = \sqrt{2 \times 0.02284 + (0.02284)^2} - 0.02284 = 0.192$

$kd = neutral\ axis\ depth = 0.192 \times 12 = 2.304\ inches$

$$I_{cr} = \frac{12 \times (2.304)^3}{3} + 8.044 \times 0.409 \times (12 - 2.304)^2 = \mathbf{358.2\ in^4}$$

$(per\ foot\ width\ of\ slab)$

GFRP REBAR

ACI 440.11 Section 24.2

$Cracking\ moment, M_{cr} = \frac{f_r I_g}{y_t}$

where:

$Modulus\ of\ rupture, f_r = 7.5 \times \lambda \times \sqrt{f'_c} = 474.34\ psi$

$Gross\ moment\ of\ inertia, I_g = \frac{b \times h^3}{12} = 2{,}744\ in^4\ per\ foot\ width$

$Centroidal\ axis\ of\ gross\ section, y_t = \frac{14\ inches}{2} = 7\ inches$

$Cracking\ moment, M_{cr} = \frac{f_r I_g}{y_t} = 185{,}941\ lb \cdot in\ per\ foot\ width\ of\ slab$

$Cracking\ moment, M_{cr} = 15.495\ kip \cdot ft\ per\ foot\ width\ of\ slab$

$Effective\ moment\ of\ inertia, I_e \rightarrow if\ M_a \leq 0.8M_{cr}$:

$I_e = I_g$

$Effective\ moment\ of\ inertia, I_e \rightarrow if\ M_a > 0.8M_{cr}$:

$$I_e = \frac{I_{cr}}{1 - \gamma\left(\frac{0.8M_{cr}}{M_a}\right)^2 \times \left(1 - \frac{I_{cr}}{I_g}\right)}$$

$where\ \gamma = 1.72 - 0.72 \times \left(\frac{0.8M_{cr}}{M_a}\right)$

Moment of inertia of cracked section transformed to concrete, I_{cr}:

$$I_{cr} = \frac{b \times (kd)^3}{3} + n_f A_f \times (d - kd)^2$$

$where\ k = \sqrt{2\rho_f n_f + \left(\rho_f n_f\right)^2} - \rho_f n_f$

$where\ reinforcement\ ratio, \rho_f = \frac{A_f}{bd} = \frac{0.409}{12 \times 12} = 0.00284$

$where\ modular\ ratio, n_f = \frac{E_f}{E_c} = \frac{8{,}700{,}000}{57{,}000 \times \sqrt{f'_c}} = 2.413$

$\rho_f n_f = 0.00685$

$where\ k = \sqrt{2 \times 0.00685 + (0.00685)^2} - 0.00685 = 0.1104$

$kd = neutral\ axis\ depth = 0.1104 \times 12 = 1.325\ inches$

$$I_{cr} = \frac{12 \times (1.325)^3}{3} + 2.413 \times 0.409 \times (12 - 1.325)^2 = \mathbf{121.8\ in^4}$$

$(per\ foot\ width\ of\ slab)$

5. REINFORCED CONCRETE SERVICEABILITY (DEFLECTION) CHECK – DETAILED CALCULATION (continued)

DEFORMED WELDED WIRE REINFORCEMENT

Service level moments and corresponding effective moments of inertia:

Dead Load

$$M_D = \left[\left(0.145 \times \frac{14}{12}\right) + 0.015\right] \times 22.5^2 \div 8 = 11.654\ kip \cdot ft$$

$$M_a = M_D = 11.654 > (2/3)(M_{cr} = 15.495) = 10.33$$

$$\therefore\ I_e = \frac{I_{cr}}{1 - \left(\frac{(2/3)M_{cr}}{M_a}\right)^2 \times \left(1 - \frac{I_{cr}}{I_g}\right)}$$

$$I_{e,D} = \frac{358.2}{1 - \left(\frac{2/3 \times 15.495}{11.654}\right)^2 \times \left(1 - \frac{358.2}{2{,}744}\right)} = 1{,}130.4\ in^4$$

Dead Load + Live Load

$$M_{D+L} = \left[\left(0.145 \times \frac{14}{12}\right) + 0.015 + 0.08\right] \times 22.5^2 \div 8 = 16.72\ kip \cdot ft$$

$$M_a = M_{D+L} = 16.72 > (2/3)(M_{cr} = 15.495) = 10.33$$

$$\therefore\ I_e = \frac{I_{cr}}{1 - \left(\frac{(2/3)M_{cr}}{M_a}\right)^2 \times \left(1 - \frac{I_{cr}}{I_g}\right)}$$

$$I_{e,D+L} = \frac{358.2}{1 - \left(\frac{2/3 \times 15.495}{16.72}\right)^2 \times \left(1 - \frac{358.2}{2{,}744}\right)} = 536.1\ in^4$$

GFRP REBAR

Service level moments and corresponding effective moments of inertia:

Dead Load

$$M_D = \left[\left(0.145 \times \frac{14}{12}\right) + 0.015\right] \times 22.5^2 \div 8 = 11.654\ kip \cdot ft$$

$$M_a = M_D = 11.654 < (0.8)(M_{cr} = 15.495) = 12.396$$

$$\therefore\ I_e = I_g = 2{,}744\ in^4$$

$$I_{e,D} = 2{,}744\ in^4$$

Dead Load + Live Load

$$M_{D+L} = \left[\left(0.145 \times \frac{14}{12}\right) + 0.015 + 0.08\right] \times 22.5^2 \div 8 = 16.72\ kip \cdot ft$$

$$M_a = M_{D+L} = 16.72 > (0.8)(M_{cr} = 15.495) = 12.396$$

$$\therefore\ I_e = \frac{I_{cr}}{1 - \gamma\left(\frac{0.8M_{cr}}{M_a}\right)^2 \times \left(1 - \frac{I_{cr}}{I_g}\right)}$$

$$where\ \gamma = 1.72 - 0.72 \times \left(\frac{0.8 \times 15.495}{16.72}\right) = 1.1862$$

$$I_{e,D+L} = \frac{121.8}{1 - 1.1862\left(\frac{0.8 \times 15.495}{16.72}\right)^2 \times \left(1 - \frac{121.8}{2{,}744}\right)} = 323.1\ in^4$$

5. REINFORCED CONCRETE SERVICEABILITY (DEFLECTION) CHECK – DETAILED CALCULATION (continued)

DEFORMED WELDED WIRE REINFORCEMENT

Service level moments and corresponding effective moments of inertia:

Sustained Load (Dead plus 20% Live)

$$M_{sus} = \left[\left(0.145 \times \frac{14}{12}\right) + 0.015 + 0.20 \times 0.08\right] \times 22.5^2 \div 8 = 12.67\ kip \cdot ft$$

$$M_a = M_{sus} = 12.67 > (2/3)(M_{cr} = 15.495) = 10.33$$

$$\therefore\ I_e = \frac{I_{cr}}{1 - \left(\frac{(2/3)M_{cr}}{M_a}\right)^2 \times \left(1 - \frac{I_{cr}}{I_g}\right)}$$

$$I_{e,sus} = \frac{358.2}{1 - \left(\frac{2/3 \times 15.495}{12.67}\right)^2 \times \left(1 - \frac{358.2}{2{,}744}\right)} = 848.7\ in^4$$

Calculate immediate (short-term) deflections:

$$\Delta_{dead} = \frac{5Ml^2}{48E_cI_e} = \frac{5 \times 11.654 \times 22.5^2 \times 12^3}{48 \times 3605 \times 1130.4} = 0.261\ inches$$

$$\Delta_{D+L} = \frac{5Ml^2}{48E_cI_e} = \frac{5 \times 16.72 \times 22.5^2 \times 12^3}{48 \times 3605 \times 536.1} = 0.788\ inches$$

$$\Delta_{live} = 0.788 - 0.261 = 0.527\ inches$$

$$\Delta_{sustained} = \frac{5Ml^2}{48E_cI_e} = \frac{5 \times 12.67 \times 22.5^2 \times 12^3}{48 \times 3605 \times 848.7} = 0.377\ inches$$

$$\Delta_{live,unsustained} = \Delta_{D+L} - \Delta_{sustained} = 0.411\ inches$$

GFRP REBAR

Service level moments and corresponding effective moments of inertia:

Sustained Load (Dead plus 20% Live)

$$M_{sus} = \left[\left(0.145 \times \frac{14}{12}\right) + 0.015 + 0.20 \times 0.08\right] \times 22.5^2 \div 8 = 12.67\ kip \cdot ft$$

$$M_a = M_{sus} = 12.67 > (0.8)(M_{cr} = 15.495) = 12.396$$

$$\therefore\ I_e = \frac{I_{cr}}{1 - \gamma\left(\frac{0.8M_{cr}}{M_a}\right)^2 \times \left(1 - \frac{I_{cr}}{I_g}\right)}$$

$$where\ \gamma = 1.72 - 0.72 \times \left(\frac{0.8 \times 15.495}{12.67}\right) = 1.0156$$

$$I_{e,sus} = \frac{121.8}{1 - 1.0156\left(\frac{0.8 \times 15.495}{12.67}\right)^2 \times \left(1 - \frac{121.8}{2{,}744}\right)} = 1715.4\ in^4$$

Calculate immediate (short-term) deflections:

$$\Delta_{dead} = \frac{5Ml^2}{48E_cI_e} = \frac{5 \times 11.654 \times 22.5^2 \times 12^3}{48 \times 3605 \times 2744} = 0.107\ inches$$

$$\Delta_{D+L} = \frac{5Ml^2}{48E_cI_e} = \frac{5 \times 16.72 \times 22.5^2 \times 12^3}{48 \times 3605 \times 323.1} = 1.310\ inches$$

$$\Delta_{live} = 1.310 - 0.107 = 1.203\ inches$$

$$\Delta_{sustained} = \frac{5Ml^2}{48E_cI_e} = \frac{5 \times 12.67 \times 22.5^2 \times 12^3}{48 \times 3605 \times 1715.4} = 0.187\ inches$$

$$\Delta_{live,unsustained} = \Delta_{D+L} - \Delta_{sustained} = 1.123\ inches$$

5. REINFORCED CONCRETE SERVICEABILITY (DEFLECTION) CHECK – DETAILED CALCULATION (continued)

DEFORMED WELDED WIRE REINFORCEMENT

Calculate long-term) deflections:

Deflection @ 12 months

$\Delta_{12\ months} = \Delta_{long\ term,sustained} + \Delta_{live,unsustained}$

$\Delta_{12\ months} = 0.6 \times \xi_{12\ months} \times \Delta_{sustained} + \Delta_{live,unsustained}$

$\Delta_{12\ months} = 0.6 \times 1.4 \times 0.377 + 0.411 = 0.728\ inches$

Deflection @ 5 years

$\Delta_{5\ years} = \Delta_{long\ term,sustained} + \Delta_{live,unsustained}$

$\Delta_{5\ years} = 0.6 \times \xi_{5\ years} \times \Delta_{sustained} + \Delta_{live,unsustained}$

$\Delta_{5\ years} = 0.6 \times 2.0 \times 0.377 + 0.411 = 0.863\ inches$

Check ALLOWABLE deflections:

Immediate live load

$$\Delta_{live} = 0.527" < \frac{l}{360} = \frac{22.5 \times 12}{360} = 0.750"$$

$\therefore \Delta_{live}$ ***is OK***

Long-term deflection @ 5 years

$$\Delta_{5\ years} = 0.863" < \frac{l}{240} = \frac{22.5 \times 12}{240} = 1.125"$$

$\therefore \Delta_{5\ years}$ ***is OK***

GFRP REBAR

Calculate long-term) deflections:

Deflection @ 12 months

$\Delta_{12\ months} = \Delta_{long\ term,sustained} + \Delta_{live,unsustained}$

$\Delta_{12\ months} = 0.6 \times \xi_{12\ months} \times \Delta_{sustained} + \Delta_{live,unsustained}$

$\Delta_{12\ months} = 0.6 \times 1.4 \times 0.187 + 1.123 = 1.280\ inches$

Deflection @ 5 years

$\Delta_{5\ years} = \Delta_{long\ term,sustained} + \Delta_{live,unsustained}$

$\Delta_{5\ years} = 0.6 \times \xi_{5\ years} \times \Delta_{sustained} + \Delta_{live,unsustained}$

$\Delta_{5\ years} = 0.6 \times 2.0 \times 0.187 + 1.123 = 1.347\ inches$

Check ALLOWABLE deflections:

Immediate live load

$$\Delta_{live} = 1.203" > \frac{l}{360} = \frac{22.5 \times 12}{360} = 0.750"$$

$\therefore \Delta_{live}$ ***EXCEEDS ALLOWABLE DEFLECTION***

Long-term deflection @ 5 years

$$\Delta_{5\ years} = 1.347" > \frac{l}{240} = \frac{22.5 \times 12}{240} = 1.125"$$

$\therefore \Delta_{5\ years}$ ***EXCEEDS ALLOWABLE DEFLECTION***

6. REINFORCED CONCRETE DESIGN SUMMARY

DEFORMED WELDED WIRE REINFORCEMENT

STRENGTH CHECK

Design Flexural Strength with A_s = 0.409 in^2
$\varphi M_n = 0.90 \times 27.79 = 25\ kip \cdot ft, per\ foot\ width\ of\ slab$

Demand-to-Capacity Ratio (DCR)
$$\frac{M_u}{\varphi M_n} = \frac{22.1}{25} = 0.884 < 1.0\ \therefore strength\ is\ adequate$$

DEFLECTION CHECK

$$\Delta_{live} = 0.527" < \frac{l}{360} = \frac{22.5 \times 12}{360} = 0.750"$$

$\therefore \Delta_{live}$ ***is OK***

$$\Delta_{5\ years} = 0.863" < \frac{l}{240} = \frac{22.5 \times 12}{240} = 1.125"$$

$\therefore \Delta_{5\ years}$ ***is OK***

GFRP REBAR

STRENGTH CHECK

Design Flexural Strength with A_f = 0.409 in^2
$\varphi M_n = 0.55 \times 47.83 = 26.3\ kip \cdot ft, per\ foot\ width\ of\ slab$

Demand-to-Capacity Ratio (DCR)
$$\frac{M_u}{\varphi M_n} = \frac{22.1}{26.3} = 0.84 < 1.0\ \therefore strength\ is\ adequate$$

DEFLECTION CHECK

$$\Delta_{live} = 1.203" > \frac{l}{360} = \frac{22.5 \times 12}{360} = 0.750"$$

$\therefore \Delta_{live}$ ***EXCEEDS ALLOWABLE DEFLECTION***

$$\Delta_{5\ years} = 1.347" > \frac{l}{240} = \frac{22.5 \times 12}{240} = 1.125"$$

$\therefore \Delta_{5\ years}$ ***EXCEEDS ALLOWABLE DEFLECTION***

The above results indicate that both WWR and GFRP are adequate for strength when using a reinforcement area equal to 0.409 in^2 per foot width of slab, but GFRP deflections exceed the allowable.

Assuming the slab thickness is maintained at 14 inches as originally defined in the example problem statement, how much does the GFRP reinforcement area need to be increased to arrive at an acceptable solution from a serviceability standpoint? Additionally, can the WWR steel area be refined even further by using readily-available 80 ksi material?

7. REINFORCED CONCRETE "ALTERNATE DESIGN"

DEFORMED WELDED WIRE REINFORCEMENT

STRENGTH CHECK

Design Flexural Strength with Grade 80 A_s = 0.358 in^2
$\varphi M_n = 0.90 \times 27.79 = 25\ kip \cdot ft, per\ foot\ width\ of\ slab$

Demand-to-Capacity Ratio (DCR)
$$\frac{M_u}{\varphi M_n} = \frac{22.1}{25} = 0.884 < 1.0\ \therefore strength\ is\ adequate$$

DEFLECTION CHECK

$$\Delta_{live} = 0.596" < \frac{l}{360} = \frac{22.5 \times 12}{360} = 0.750"$$

$\therefore \Delta_{live}$ ***is OK***

$$\Delta_{5\ years} = 0.961" < \frac{l}{240} = \frac{22.5 \times 12}{240} = 1.125"$$

$\therefore \Delta_{5\ years}$ ***is OK***

(Note that the steel area selected is simply based on a ratio of yield strength: 70ksi / 80 ksi x 0.409 in^2 = 0.358 in^2)

GFRP REBAR

STRENGTH CHECK

Design Flexural Strength with A_f = 0.690 in^2
$\varphi M_n = 0.591 \times 71.81 = 42.4\ kip \cdot ft, per\ foot\ width\ of\ slab$

Demand-to-Capacity Ratio (DCR)
$$\frac{M_u}{\varphi M_n} = \frac{22.1}{42.4} = 0.52 < 1.0\ \therefore strength\ is\ adequate$$

DEFLECTION CHECK

$$\Delta_{live} = 0.742" < \frac{l}{360} = \frac{22.5 \times 12}{360} = 0.750"$$

$\therefore \Delta_{live}$ ***is OK***

$$\Delta_{5\ years} = 0.881" < \frac{l}{240} = \frac{22.5 \times 12}{240} = 1.125"$$

$\therefore \Delta_{live}$ ***is OK***

(Note that the GFRP area was selected iteratively to arrive at satisfactory deflection results.)

The above "alternate design" results indicate that we can utilize 80 ksi WWR with a reduced steel area and still satisfy both strength and serviceability design requirements for the subject 14-inch slab.

In contrast, the GFRP reinforcement area needs to be increased nearly 70% to 0.69 in^2 for the 14-inch slab to be adequate for serviceability.