A carbon-based FRP complete wrap is selected to retrofit the columns.The properties of the FRP system,as reported by the manufacturer,are shown in Table 16.8b.The design calculations to arrive at the number of required complete wraps follow in Table 16.8c.

Table 16.8a—Column cross section details and material properties

f	6.5 ksi	45 MPa
f	60ksi	400 MPa
re	1 in.	25 mm
Bars	12 No.10	12632
Ag	576 in.²	3716 cm²
Aμ	15.24in.?	98 cm²
Pg%	2.65	2.65
ΦP.without FRP	2087 kip	9281kN
中Paic)	2504 kip	11,138kN

Note:The column features stel ties for transverse reinforcement.

Table 16.8b—Manufacturer's reported FRP system properties

Thickness per plyt	0.013 in.	0.33 mm
Ultimate tensile strengthf	550 ksi	3792 MPa
Rupture strain gn	0.0167 in./in.	0.0167mm/mm
Modulus of elasticity E_1	33,000 ksi	227,527 MPa

Table 16.8c—Procedure for strengthening of a noncircular concrete column for axial load increase

Procedure	Calculation in in.-1b units	Calculation in SI units
Step 1—Compute the design FRPsystem properties. The column is located in an interior environment and a CFRPsystem will be used. Therefore, per Table 9.4, an environmental reduction factor of 0.95 is used. Fm=Cef m=Ceen	fm=(0.95)(550ksi)=522.5ksi Gu=(0.95)(0.0167)=0.0159 in./in	fm=(0.95)(3792 MPa)=3603 MPa m=(0.95)(0.0167)=0.0159 mm/mm
Step 2—Determine the required maximum compressive strength of confined concretefa'. fa'can be obtained by reordering Eq(12.1b): $f'_{cc}=\frac{1}{0.85(A_g-A_{st})}\left(\frac{\phi P_{n,req}}{0.80\phi}-f_yA_{st}\right)$	$f'_{cc}=\frac{1}{0.85\times(576\text{ in.}^2-15.24\text{ in.}^2)}\times\left(\frac{2504\text{ kip}}{0.80\times0.65}-60\text{ ksi}\times15.24\text{ in.}^2\right)$ fa=8.18ksi	$f'_{cc}=\frac{1}{0.85\times(371{,}612\text{ mm}^2-9832\text{ mm}^2)}\times\left(\frac{11{,}138\text{ kN}}{0.80\times0.65}-414\text{ MPa}\times9832\text{ mm}^2\right)$ $f_{cc}'=56.4$ MPa
Step 3—Determine the maximum confining pressure due to the FRP jacketf. fi can be obtained by reordering Eq. (12.1g): $f_l=\frac{f'_{cc}-f'_c}{3.3\psi_f\kappa_a}$ where $\kappa_a=\frac{A_e}{A_c}\left(\frac{b}{h}\right)^2$ $\frac{A_e}{A_c}=\frac{1-\frac{\left[\left(\frac{b}{h}\right)(h-2r_c)^2+\left(\frac{h}{b}\right)(b-2r_c)^2\right]}{3A_g}-\rho_g}{1-\rho_g}$	$f_l=\frac{8.18\text{ ksi}-6.5\text{ ksi}}{0.95\times3.3\times0.425}=1.26\text{ ksi}$ Ka=0.425(1)²=0.425 $\frac{A_e}{A_c}=\frac{1-\frac{[2\times(1)(24\text{ in.}-2\times1\text{ in.})^2]}{3\times576\text{ in.}^2}-0.0265}{1-0.0265}$ $\frac{A_e}{A_c}=0.425$	$f_l=\frac{56.4\text{ MPa}-44.8\text{ MPa}}{0.95\times3.3\times0.425}=8.7\text{ MPa}$ Ka=0.425(1)²=0.425 $\frac{A_e}{A_c}=\frac{1-\frac{[2\times(1)(610\text{ mm}-2\times25\text{ mm})^2]}{3\times371{,}612\text{ mm}^2}-0.0265}{1-0.0265}$ $\frac{A_e}{A_c}=0.425$

Table 16.8c, cont.—Procedure for strengthening of a noncircular concrete column for axial load increase

Step4—Determine the number of plies N. N can be obtained by reordering Eq. (12.1h): $N = \frac{f_l \sqrt{b^2 + h^2}}{2E_f t_f \varepsilon_{fe}}$ e=KEm Checking the minimum confinement ratio: $\frac{f_l}{f'_c} \geq 0.08$	$N = \frac{1.26 \text{ ksi} \sqrt{(24 \text{ in.})^2 + (24 \text{ in.})^2}}{2(33{,}000 \text{ ksi})(0.013 \text{ in.})(8.8 \times 10^{-3})}$ N=5.7:6 plies Ee=0.55×0.0159in./in.=8.8×10³ in./in. $\frac{f_l}{f'_c} = \frac{1.26 \text{ ksi}}{6.5 \text{ ksi}} = 0.19 > 0.08$ **OK**	$N = \frac{8.7 \text{ MPa} \sqrt{(610 \text{ mm})^2 + (610 \text{ mm})^2}}{2(227{,}527 \text{ MPa})(0.33 \text{ mm})(8.8 \times 10^{-3})}$ N=5.726plies B%=0.55×0.0159 mm/mm=8.8×10³ mm/mm $\frac{}{} = \frac{8.7 \text{ MPa}}{44.8 \text{ MPa}} = 0.19 > 0.08$ **OK**
Step5—Verify that the ultimate axial strain of the confined concrete Ecu≤0.01. can be obtained using Eq. (12.1j): $\varepsilon_{ccu} = \varepsilon'_c \left(1.5 + 12\kappa_b \frac{f_l}{f'_c}\left(\frac{\varepsilon_{fe}}{\varepsilon'_c}\right)^{0.45}\right)$ where $\kappa_b = \frac{A_e}{A_c}\left(\frac{h}{b}\right)^{0.5}$ If the case that Ecmwas to be greater than 0.01, thenf' should be recalculated from the stress-strain model using Eq. (12.Ic).	E=(0.002 in./in.) $\times\left(1.5 + 12 \times 0.425 \times \frac{1.2 \text{ ksi}}{6.5 \text{ ksi}}\left(\frac{8.8 \times 10^{-3} \text{ in./in.}}{0.002 \text{ in./in.}}\right)^{0.45}\right)$ Ec=0.0067 in./in.<0.01 **OK** K_6=0.425(1)⁵=0.425	=(0.002 mm/mm) $\times\left(1.5 + 12 \times 0.425 \times \frac{8.3 \text{ MPa}}{44.8 \text{ MPa}}\left(\frac{8.8 \times 10^{-3} \text{ mm/mm}}{0.002 \text{ mm/mm}}\right)^{0.45}\right)$ Ec=0.0067mm/mm<0.01 **OK** Kb=0.425(1)⁵=0.425

Table 16.9—Procedure for strengthening of a noncircular concrete column for increase in axial and bending forces

Procedure	Calculation in in.-Ib units	Calculation in SI units
Step1—Determine the simplified curve for the unstrengthened column(n=0plies). Points A,B,and C can be obtained by well-known procedures, and by using Eq(C-1)to(C-5) considering y=1, fa'=f', E₂=0, and cu=8u= 0.003.	oPn)=2087kip;oMm(c)=0kip-ft 中PacB)=1858 kip;ΦMaG=644kip-ft ΦPao=928 kip;ΦMmc=884 kip-ft	中Pmcu=9283kN;ΦMm=0kN-m 中Pacs)=8265kN;ΦMa(B)=873kN ΦPao=4128kN;ΦMmo=1199kN-m

16.9—Strengthening of a noncircular concrete column for increase in axial and bending forces

The column described in 16.8 is subjected to an ultimate axial compressive load P=1900 kip(8451 kN)and an ultimate bending moment M=380 kip-ft (515 kN-m) (e=0.1h).It is sought to increase load demands by 30%at constant eccentricity(Pu=2470 kip,M=494 kip-ft).The calculations to determine moment-axial interaction for the FRP confined column follow in Table 16.9.

Table 16.9,cont.—Procedure for strengthening of a noncircular concrete column for increase in axial and bending forces

Step 2—Determine the simplified curve for a strengthened column. A wrapping system composed of six plies will be the starting point to construct the bilinear Curve A-B-C and then be compared with the position of the required P, and M. Points A, B, and C of the curve can be computed using Eq. (12.1b), (C-1), and (C-2): 中Pma△) = ϕ0.8(0.85/(4g−Am)+fA) 中PaBc) = ϕ[A(y)³+Bv.]²+Cv] +D+ΣAJa 中Ma(BC) = ϕ(E(V)⁴+F(v)³+G(v)² +H(v.)+1)+ΣAfd	Point A: Nominal axial capacity: 中Pm(a)=0.65×0.8(0.85×8.26 ksi×(576 in.²−15.24in.)+60ksi×15.24 in.?) ϕPMA)=2523 kip where fa'=6.5 ksi+3.3(0.425)(1.26 ksi) fa'=8.26 ksi $f_l = \frac{0.95 \times 2 \times 33{,}000 \text{ ksi} \times 6 \times 0.013 \text{ in.} \times \left(0.55 \times 0.0159 \frac{\text{in.}}{\text{in.}}\right)}{\sqrt{(24 \text{ in.})^2 + (24 \text{ in.})^2}}$ $f_l = 1.26$ ksi Point B: Nominal axial capacity: ϕPB)=0.65[−0.22 kip/in.³ (15.33 in.³+10.17 ksi(15.33 in.²−156kip/in. (15.33in.)+3645.2 kip+5.08 in.² (60 ksi)+2.54 in.?(60 ksi)+2.54 in.?(37.21 ksi)] ϕPa(a)=2210 kip where $A = \frac{-24 \text{ in.}(4595 \text{ ksi} - 190.7 \text{ ksi})^2}{12 \times 6.5 \text{ ksi}}\left(\frac{0.0042 \text{ in./in.}}{22 \text{ in.}}\right)^2$ =−0.22 kip/in. $B = \frac{24 \text{ in.}(4595 \text{ ksi} - 190.7 \text{ ksi})}{2}\left(\frac{0.0042 \text{ in./in.}}{22 \text{ in.}}\right)$ =10.17ksi C=−24 in. ×6.5ksi=−156 kip/in. D=24in. ×22 in. ×6.5 ksi $+\frac{24 \text{ in.} \times 22 \text{ in.} \times 190.7 \text{ ksi}}{2}$ ×(0.0042 in./in.) D=3645.2kip	Point A: Nominal axial capacity: ϕPa(4)=0.65×0.8(0.85×56.96 MPa×(371,612 mm²−9832 mm²)+414MPa+9232mm² ϕPM△=11,223kN where a'=44.8 MPa+3.3(0.425) (8.7MPa) $f_{cc}' = 56.96$ MPa $\frac{0 \text{ MPa} \times 6 \times 0.33 \text{ mm} \times \left(0.55 \times 0.0159 \frac{\text{mm}}{\text{mm}}\right)}{\sqrt{(610 \text{ mm})^2 + (610 \text{ mm})^2}}$ f=8.67 MPa Point B: Nominal axial capacity: 中Pms=0.65[−6.003×10⁵kN/mm³ (389 mm)³+70.14×10⁻³kN/mm² (389mm)²−27.32kN/mm(389mm)+16,215kN+3277 mm²(414 MPa)+1639mm² (414 MPa)+1639mm²(257 MPa)] ϕPaa=9892kN where $A = \frac{-610 \text{ mm}(31{,}685 \text{ MPa} - 1315 \text{ MPa})^2}{12 \times 44.8 \text{ MPa}}\left(\frac{0.0042 \text{ mm/mm}}{559 \text{ mm}}\right)^2$ −6.003×10³ kN/mm³ $B = \frac{600 \text{ mm}(31{,}685 \text{ MPa} - 1315 \text{ MPa})}{2}\left(\frac{0.0042 \text{ mm/mm}}{559 \text{ mm}}\right)$ −70.14×10⁻³ kNmm² C=−610mm×44.84 MPa=−27.32kN/mm D=610mm×559mm×44.8 MPa $+\frac{610 \text{ mm} \times 559 \text{ mm} \times 1315 \text{ MPa}}{2}$ ×(0.0042 mm/mm) D=16,215kN

Table 16.9,cont.—Procedure for strengthening of a noncircular concrete column for increase in axial and bending forces

The coeficients A, B, C, D, E, F, G, H, and Iof the previous expressions are given by Eq. (C-3): $A = \frac{-b(E_c - E_2)^2}{12f_c'}\left(\frac{\varepsilon_{ccu}}{c}\right)^2$ $B = \frac{b(E_c - E_2)}{2}\left(\frac{\varepsilon_{ccu}}{c}\right)$ C=bf! $D = bf_c' + \frac{bcE_2}{2}(\varepsilon_{ccu})$ $E = \frac{-b(E_c - E_2)^2}{16f_c'}\left(\frac{\varepsilon_{ccu}}{c}\right)^2$ $F = b\left(c - \frac{h}{2}\right)\frac{(E_c - E_2)^2}{12f_c'}\left(\frac{\varepsilon_{ccu}}{c}\right)^2 + \frac{b(E_c - E_2)}{3}\left(\frac{\varepsilon_{ccu}}{c}\right)$ $G = -\left(\frac{b}{2}f_c' + b\left(c - \frac{h}{2}\right)\frac{(E_c - E_2)}{2}\left(\frac{\varepsilon_{ccu}}{c}\right)\right)$ $H = bf_c'\left(c - \frac{h}{2}\right)$ $I = \frac{bc^2}{2}f_c' - bcf_c'\left(c - \frac{h}{2}\right) + \frac{bc^2E_2}{3}(\varepsilon_{ccu})$ $- \frac{bcE_2}{2}\left(c - \frac{h}{2}\right)(\varepsilon_{ccu})$ $f_l = \frac{\psi_f 2E_f N t_f \varepsilon_{fe}}{\sqrt{b^2 + h^2}}$	For the calculation of the cocfficients, it is necessary to compute key parameters from the stress-strain model: $y_t = 22\text{ in.} \times \frac{0.003\text{ in./in.}}{0.0042\text{ in./in.}} = 15.33\text{ in.}$ c=22 in. $\varepsilon_t' = \frac{2 \times 6.5\text{ ksi}}{4595\text{ ksi} - 190.7\text{ ksi}} = 0.003\text{ in./in.}$ $E_2 = \frac{7.31\text{ ksi} - 6.5\text{ ksi}}{0.0042\text{ in./in.}} = 190.7\text{ ksi}$ f&'=6.5ksi+3.3(0.425)(0.58 ksi)=7.31ksi Bm=0.002 in./in. $\times\left(1.5 + 12 \times 0.425\left(\frac{0.58\text{ ksi}}{6.5\text{ ksi}}\right)\left(\frac{0.004\text{ in./in.}}{0.002\text{ in./in.}}\right)^{0.45}\right)$ &cm=0.0042 in. in. Ko=K_5=0.425 $f_l = \frac{0.95 \times 2 \times 33{,}000\text{ ksi} \times 6 \times 0.013\text{ in.} \times (0.004\text{ in./in.})}{\sqrt{(24\text{ in.})^2 + (24\text{ in.})^2}}$ Checking the minimum confinement ratio: fU'=0.58ksi/6.5 ksi=0.09:0.08 **OK** The strains in each layer of steel are determined by similar triangles in the strain distribution. The corresponding stresses are then given by: JA=E_1E_5=0.0038 in/in. ×29,000 ksi∴60ksi Ja=&$_2E_5$=0.0026 in./in. ×29,000 ksi260ksi fs=e_3E,=0.0013 in./in. ×29,000 ksi=37.2 ksi Ja=E_4E_4=0 in./in. ×29,000ksi=0ksi	For the calculation of the coefficients, it is necessary to compute key parameters from the stress-strain model: $y_t = 559\text{ mm} \times \frac{0.003\text{ mm/mm}}{0.0042\text{ mm/mm}} = 389\text{ mm}$ c=559mm $\varepsilon_t' = \frac{2 \times 44.8\text{ MPa}}{31{,}685\text{ MPa} - 1315\text{ MPa}} = 0.003\text{ mm/mm}$ $E_2 = \frac{50.4\text{ MPa} - 44.8\text{ MPa}}{0.0042\text{ mm/mm}} = 1315\text{ MPa}$ fa'=44.8 MPa+3.3 $(0.425)(3.97\text{ MPa}) = 50.4$ MPa . =0.002mm/mm $\times\left(1.5 + 12 \times 0.425\left(\frac{3.97\text{ MPa}}{44.8\text{ MPa}}\right)\left(\frac{0.004\text{ mm/mm}}{0.002\text{ mm/mm}}\right)^{0.45}\right)$ Ec=0.0042 mm/mm Ka=KA=0.425 $f_l = \frac{0.95 \times 2 \times 227{,}527\text{ MPa} \times 6 \times 0.33\text{ mm} \times (0.004\text{ mm/mm})}{\sqrt{(610\text{ mm})^2 + (610\text{ mm})^2}}$ Checking the minimum confinement ratio: flfc'=3.97 MPa/44.8 MPa=0.09:0.08 **OK** The strains in each layer of steel are determined by similar triangles in the strain distribution. The corresponding stresses are then given by: f=EE_2=0.0038 mm/mm×200,000 MPa: 414MPa Jz=B_2E_2=0.0026 mm/mm×200,000MPa 414 MPa fs=E_3E_4=0.0013 mm/mm×200,000 MPa= 257 MPa Ja=E_4E_2=0mm/mm×200,000 MPa=0 MPa

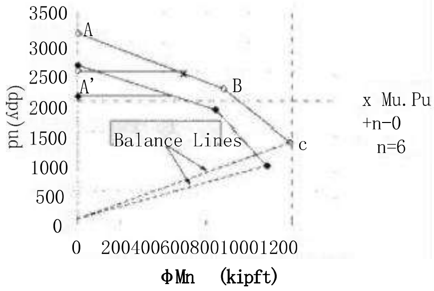
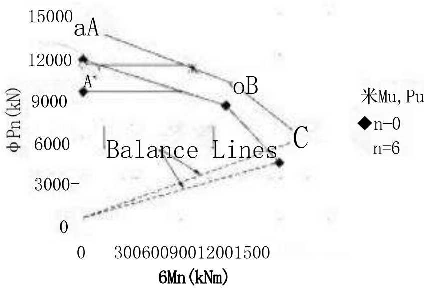
Table 16.9,cont.—Procedure for strengthening of a noncircular concrete column for increase in axial and bending forces

Key parameters of the stress-strain model: $y_t = c\frac{\varepsilon_t'}{\varepsilon_{ccu}}$ $c = \begin{cases} d & \text{for Point B} \\ d\frac{\varepsilon_{ccu}}{\varepsilon_{sy}+\varepsilon_{ccu}} & \text{for Point C} \end{cases}$ $\varepsilon_t' = \frac{2f_c'}{E_c - E_2}$ $E_2 = \frac{f_{cc}' - f_c'}{\varepsilon_{ccu}}$ a’=’+3.3K fi $\varepsilon_{ccu} = \varepsilon_c'\left(1.5+12\kappa_b\frac{f_l}{f_c'}\left(\frac{\varepsilon_{fe}}{\varepsilon_c'}\right)^{0.45}\right)$ Ec=min(0.004,K.en) $\kappa_a = \frac{A_e}{A_c}\left(\frac{b}{h}\right)^2$ $\kappa_b = \frac{A_e}{A_c}\left(\frac{h}{b}\right)^{0.5}$ Notes:The designer should bear in mind that, for the case of pure compression,the effective strain in the FRP,e,is limited by K&n and,in the case of combined axial and bending,by Ee=min(0.004,KEn).	Nominal bending moment: ΦMnB)=0.65[−0.166 kip/in. (15.33 in)^4 +8.99ksi(15.33in.^{3}−179.73 kip/in. (15.33in.)^2 +1560 kip(15.33 in.)+4427 kip-in. +5.08 in.^{2}(60 ksi)(10 in.) +2.54 in. (60ksi)(3.3 in.) −2.54 in?(37.21 ksi)(3.3 in.) ΦMs)=682 kip-ft where $E = \frac{-24\text{ in.}(4595\text{ ksi} - 190.7\text{ ksi})^2}{16 \times 6.5\text{ ksi}}\left(\frac{0.0042\text{ in./in.}}{22\text{ in.}}\right)^2$ =−0.166 kip/in.^{3} F=24 in. (22 in.−12 in.) $\times\frac{(4595\text{ ksi} - 190.7\text{ ksi})^2}{12 \times 6.5\text{ ksi}}\times\left(\frac{0.0042\text{ in./in.}}{22\text{ in.}}\right)^2$ $+\frac{24\text{ in.}(4595\text{ ksi} - 190.7\text{ ksi})}{3}\times\left(\frac{0.0042\text{ in./in.}}{22\text{ in.}}\right)$ =8.99ksi G=−6.5ksi×12 in.+24 in. (22 in.−12 in.) ×0.5×(4595ksi-190.7ksi)×(0.0042 in/in./22 in. =−179.73 kip/in. H=6.5ksi×24in. (22 in.−12 in.)=1560 kip I=6.5 ksi×24in. ×([22 in.]^2/2) −6.5ksi(22 in.−12 in.)×22 in. ×24in. +190.7ksi×24 in. ×([22 in.]^2/3)(0.0042 in./in. −190.7ksi×24 in. ×(22in./3)(22 in.−12 in. (0.0042 in./in.)=4427 kip-in	Nominal bending moment: ΦMniB)=0.65[−4.502×10^{5}kN/mm^3(389mm)^4 +62.01×10^{-3}kN/mm^3(389mm)^3 −31.48kN/mm(389mm)^2+6939kN(389mm) +500,162kN-mm+3277 mm^2(414 MPa)(254 mm) +1639mm^2(414 MPa)(85mm) −1639mm^2(257 MPa)(85mm)] ΦMaB)=924kN-m where $E = \frac{-610\text{ mm}(31{,}685\text{ MPa} - 1315\text{ MPa})^2}{16 \times 44.8\text{ MPa}}$ $\times\left(\frac{0.0042\text{ mm/mm}}{559\text{ mm}}\right)^2 = -0.452 \times 10^{-5}\text{ kN/mm}^3$ F=610mm(559mm−305mm) $\times\frac{(31{,}685\text{ MPa} - 1315\text{ MPa})^2}{12 \times 44.8\text{ MPa}}$ $\times\left(\frac{0.0042\text{ mm/mm}}{559\text{ mm}}\right)^2$ $+\frac{610\text{ mm}(31{,}685\text{ MPa} - 1315\text{ MPa})}{3}$ $\times\left(\frac{0.0042\text{ mm/mm}}{559\text{ mm}}\right) = 62.01 \times 10^{-3}\text{ kN/mm}^2$ G=−44.8 MPa×305mm+610mm(559mm−305 mm)×0.5×(31,685 MPa−1315 MPa)×(0.0042mm/mm/559 mm)=−31.48kN/mm H=44.8 MPa×610mm(559 mm−305mm) =6939kN 1=44.8 MPa×610mm×([559mm]^2/2) −44.8 MPa(559mm−305mm)×(559mm)(610mm) +1315MPa×610mm×([559mm}/3) (0.0042mm/mm)−1315 MPa×610mm ×(559mm/3)(559mm−305mm)(0.0042mm/mm) =500,162kN-mm

Table 16.9,cont.—Procedure for strengthening of a noncircular concrete column for increase in axial and bending forces

	The distances from each layer of steel reinforcement to the geometric centroid of the cross section are: d_1=10 in. d_2=d_3=3.3 in. Point C: Nominal axial capacity: Pno=0.65[-0.49 kip/in. (10.3in.³ +15.14 ksi(10.3in.²-156 kip-in. (10.3 in.) +2448.71 kip+5.08 in. (60 ksi) +2.54 in.?(50.79ksi)+2.54 in.²(-4.61 ksi) +5.08 in.?(-60ksi)] ΦPao=1320kip where $A=\frac{-24\text{ in.}(4595\text{ ksi}-190.7\text{ ksi})^2}{12\times 6.5\text{ ksi}}\left(\frac{0.0042\text{ in./in.}}{14.78\text{ in.}}\right)^2$ =-0.49kip/in. $B=\frac{-24\text{ in.}(4595\text{ ksi}-190.7\text{ ksi})}{2}\left(\frac{0.0042\text{ in./in.}}{14.78\text{ in.}}\right)$ =15.14ksi C=-24 in. ×6.5ksi=-156 kip/in. D=24in. ×14.78 in. ×6.5ksi $+\frac{24\text{ in.}\times 14.78\text{ in.}\times 190.7\text{ ksi}}{2}$ ×(0.0042 in./in.)=2448.71 kip For the calculation of the coefficients,it is necessary to compute key parameters from the stress-strain model: $c=22\text{ in.}\times\left(\frac{0.0042\text{ in./in.}}{0.0021\text{ in./in.}+0.0042\text{ in./in.}}\right)$ =14.67 in. $y_t=14.64\text{ in.}\frac{0.003\text{ in./in.}}{0.0042\text{ in./in.}}=10.5\text{ in.}$	The distances from each layer of steel reinforcement to the geometric centroid of the cross section are: d_1=254 mm d_2=d_3=85mm Point C: Nominal axial capacity: ΦPmc=0.65[-1.33×10⁴kN/mm³(262 mm)³+ 104.41×10³kN/mm²×(262mm)² -27.32kN/mm(262mm) +10,892kN+3277 mm²(414mPa) +1315mm²(350 MPa)+1315mm²(-31.8 MPa) +3277 mm²(-414 MPa)] ΦPmc=5870kN where $A=\frac{-610\text{ mm}(31{,}681\text{ MPa}-1315\text{ MPa})^2}{12\times 44.8\text{ MPa}}\times\left(\frac{0.0042\text{ mm/mm}}{375\text{ mm}}\right)^2$ =1.33×10⁴kN/mm² $B=\frac{-610\text{ mm}(31{,}681\text{ MPa}-1315\text{ MPa})}{2}\times\left(\frac{0.0042\text{ mm/mm}}{375\text{ mm}}\right)$ =-104.41×10³kN/mm² C=-610 mm×44.8 MPa=-27.32kN/mm D=610mm×375mm×44.8 MPa $+\frac{610\text{ mm}\times 375\text{ mm}\times 1315\text{ MPa}}{2}$ ×(0.0042mm/mm)=10,892kN For the calculation of the coefficients, it is necessary to compute key parameters from the stress-strain model: c=560mm $\times\left(\frac{0.0042\text{ mm/mm}}{0.0021\text{ mm/mm}+0.0042\text{ mm/mm}}\right)$ =373mm $y_t=373\text{ mm}\frac{0.003\text{ in./in.}}{0.0042\text{ in./in.}}=266\text{ mm}$
	The strains in each layer of steel are determined by similar triangles in the strain distribution. The corresponding stresses are then given by: fa=E₃E₃=0.0037in./in. ×29,000ksi:60ksi f=EE₄=0.0018 in.in. ×29,000ksi=50.78 ksi fa=EeE₃=-1.59×10⁴in/in. ×29,000ksi=-4.61 ksi fa=e,,E₂=-0.0021 in./in. ×29,000ksi=-60ksi	The strains in each layer of steel are determined by similar triangles in the strain distribution. The corresponding stresses are then given by: fi=E₁E,=0.0037mm/mm×200,000 MPa 414 MPa f2=E₂E₂=0.0018 mm/mm×200,000 MPa= 350 MPa fs=E₃E₄=-1.59×10⁴mm/mm×200,000 MPa =-31.8 MPa f4=ε₄E,=-0.0021 mm/mm×200,000 MPa= 414 MPa

Table 16.9,cont.—Procedure for strengthening of a noncircular concrete column for increase in axial and bending forces

	Nominal bending moment: $\phi M_c)=0.65[-0.37\ \text{kip/in.}^3(10.3\text{in.})^4$ $+11.46\ \text{ksi}(10.3\text{in.}^3-120.08\ \text{kip/in.}\ (10.3\text{in.})^2$ $+433.5\text{kip}(10.3\ \text{in.})+11{,}643\ \text{kip-in.}$ $+5.08\ \text{in.?}(60\text{ksi})(10\text{in.})$ $+2.54\ \text{in.?}(50.79\ \text{ksi})(3.33\ \text{in.})$ $-2.54\ \text{in.}^2(4.61\ \text{ksi})(3.33\text{in.})$ $-5.08\ \text{in.}^2(-60\text{ksi})(10\ \text{in.})]$ $\phi M_{nc}=992\ \text{kip-ft}$ where $E=\frac{-24\ \text{in.}(4595\ \text{ksi}-190.7\ \text{ksi})^2}{16\times 6.5\ \text{ksi}}\left(\frac{0.0042\ \text{in./in.}}{14.78\ \text{in.}}\right)^2$ $=-0.37\text{kip/in.}^3$ $F=24\ \text{in.}\ (14.78\ \text{in.}-12\ \text{in.})$ $\times\frac{(4595\ \text{ksi}-190.7\ \text{ksi})^2}{12\times 6.5\ \text{ksi}}\left(\frac{0.0042\ \text{in./in.}}{14.78\ \text{in.}}\right)^2$ $+\frac{24\ \text{in.}(4595\ \text{ksi}-190.7\ \text{ksi})}{3}$ $\times\left(\frac{0.0042\ \text{in./in.}}{14.78\ \text{in.}}\right)=11.46\ \text{ksi}$ $G=-6.5\ \text{ksi}\times 12\ \text{in.}+24\ \text{in.}\ (14.78\text{in.}-12\ \text{in.})$ $\times\left(\frac{4595\ \text{ksi}-190.7\ \text{ksi}}{2}\right)\left(\frac{0.0042\ \text{in./in.}}{14.78\ \text{in.}}\right)$ $G=-120.08\ \text{kip/in}$ $H=6.5\ \text{ksi}\times 24\text{in.}\ (14.78\ \text{in.}-12\ \text{in.})=433.5\ \text{kip}$ $I=6.5\ \text{ksi}\times 24\text{in.}\ \times([14.78\ \text{in.}]\}/2)-$ $6.5\text{ksi}(14.78\ \text{in.}-12\ \text{in.})(14.78\ \text{in.})$ $(24\ \text{in.})+190.7\text{ksi}\times 24\text{in}\times([14.78\ \text{in.}]^2/3)$ $(0.0042\ \text{in./in.})-190.7\text{ksi}\times 24\text{in.}\ \times(14.78\ \text{in.}/3)$ $(14.78\ \text{in.}-12\ \text{in.})$ $(0.0042\ \text{in./in.})=11{,}643\text{kip-in.}$	Nominal bending moment: $\phi M_{nc}=0.65[-9.98\times 10^5\text{kN/mm}^3(262\text{mm})^4+$ $79\times 10^{-3}\ \text{kN/mm}^2(262\text{mm})^3-21.03\text{kN/mm}$ $(262\text{mm}^2+1928\text{kN}(262\text{mm})+1{,}315{,}453$ $\text{kN-mm}+3277\ \text{mm}^2(414\ \text{MPa})(254\ \text{mm})+1639$ $\text{mm}^2(350\text{MPa})(85\text{mm})-1639\text{mm}^2$ $(-31.8\ \text{MPa})(85\text{mm})-3277\ \text{mm}^2$ $(414\ \text{MPa})(254\ \text{mm})]$ $\phi M_n(o=1345\text{kN-m}$ where $E=\frac{-610\ \text{mm}(31.681\ \text{MPa}-1315\ \text{MPa})^2}{16\times 44.8\ \text{MPa}}\left(\frac{0.0042\ \text{mm/mm}}{375\ \text{mm}}\right)^2$ $--9.98\times 10^{-3}\ \text{kN/mm}^3$ $F=610\text{mm}(375\text{mm}-305\text{mm})$ $\times\frac{(31.681\ \text{MPa}-1315\ \text{MPa})^2}{12\times 44.8\ \text{MPa}}\left(\frac{0.0042\ \text{mm/mm}}{375\ \text{mm}}\right)^2$ $+\frac{610\ \text{mm}(31.681\ \text{MPa}-1315\ \text{MPa})}{3}$ $\times\left(\frac{0.0042\ \text{mm/mm}}{375\ \text{mm}}\right)=79\times 10^{-3}\ \text{kN/mm}^2$ $G=-44.8\ \text{MPa}\times 305\text{mm}+610\ \text{mm}(375\text{mm}-305\text{mm})$ $\times\left(\frac{31.681\ \text{MPa}-1315\ \text{MPa}}{2}\right)\left(\frac{0.0042\ \text{mm/mm}}{375\ \text{mm}}\right)$ $G=-21.03\text{kN/mm}$ $H=44.8\ \text{MPa}\times 610\text{mm}(375\text{mm}-305\text{mm})=1928\text{kN}$ $I=44.8\ \text{MPa}\times 610\text{mm}\times(1375\text{mm})^2/2)-44.8$ $\text{MPa}(375\text{mm}-305\text{mm})\times(375\ \text{mm})(610$ $\text{mm})+1315\ \text{MPa}\times 610\text{mm}\times([375\text{mm}]^2/3)$ $(0.0042\text{mm/mm})-1315\ \text{MPa}\times 610\ \text{mm}\times(375$ $\text{mm}/2)(375\text{mm}-305\text{mm})(0.0042\text{mm/mm})=$ $1{,}315{,}453\text{kN-mm}$
Step 3—Comparison of simplified partial interaction diagram with required P, and M.	The following table summarizes the axial and bending nominal capacities (unstrengthened and strengthened) for Points A, B, and C. These points are plotted in the following figure:  	The following table summarizes the axial and bending nominal capacities (unstrengthened and strengthened) for Points A, B, and C. These points are plotted in the following figure. 

16.10—Plastic hinge confinement for seismic strengthening

This example illustrates the design of an FRP retrofit to enhance the plastic rotation capacity of a nonductile reinforced concrete column.In this example,the column cannot efficiently resist seismic loads for two reasons:1)the tie spacing does not conform to current seismic design codes; and 2)the ties do not project into the core and under seismic loads may open once the cover concrete begins to spall.The aforementioned deficiencies indicate that the column may have inadequate shear strength,inadequate confinement of the plastic hinge region,and inadequate clamping of lap splices.In addition,the designer should ensure adequate resistance against buckling of the main longitudinal reinforcement.In this example,the deficiency under consideration is inadequate plastic rotation capacity,which can be enhanced by confinement of the plastic hinge region with FRP.A seismic analysis has already determined that the column is capable ofresisting the required seismic moments. Thus,there is no need to increase the flexural capacity of the section.This example is limited in scope to the FRP design requirements and does not cover the seismic analysis. ASCE/SEI41 is used as the base standard for this example.

The column,which is to be part of a lateral-load-resisting system,is illustrated in Fig.16.10a.Expected material properties andother relevant information are listed in Table 16.10a. The axial load on the column,including gravity plus seismic loads,is Pμ=75 kip(334kN).From a seismic analysis,the column should be capable of developing a plastic rotation θ,=0.025 rad.This plastic rotation demand exceeds the limiting value of 0.015 stipulated in ASCE/SEI41 numerical acceptance criteria for reinforced concrete columns that do not conform to current seismic design codes.The concrete and reinforcing steel strain limitations of ASCE/SEI 41 as listed in Table 16.10b should not be exceeded.

The column is strengthened with CFRP laminates having the composite properties listed in Table 16.10c and bonded around the column using the wet layup technique.Glass FRP(GFRP),however,can similarly be used if desired. The design process was initiated by considering a wrapping system composed of three plies.After two iterations, the final wrapping system was found to require five plies. Only the calculations used to verify the final configuration are provided.These calculations are shown in Table 16.10d.

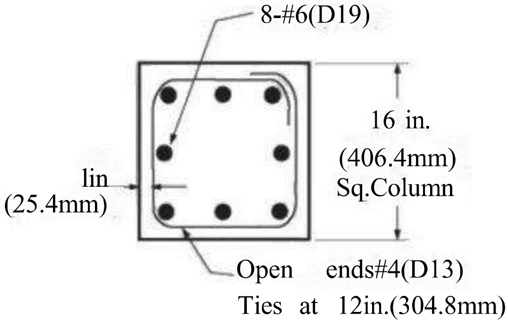


Fig.16.10a—As-built column details.

Table 16.10a—Column material properties

Concrete strengthf'	4000 psi	27.6 MPa
Concrete clastic modulus E.=57,000 √f. (psi) E=4700 √f. (MPa)	3605ksi	25 GPa
Longitudinal reinforcing stel:yield strengthf,	44,000 psi	303 MPa
Modulus of elasticity of steel E,	29,000 ksi	200 GPa
Longitudinal reinforcing steel:yield straine,	0.0015	0.0015
Column height between plastic hinges L	10ft	3.05m
Distance to extreme tension steel d	14.625 in.	371 mm

Table 16.10b—Maximum usable strain levels (ASCE/SEI 41)

	Strain limits	
	Unconfined sections	Confined sections per 13.3.1
Compressive (concrete)	0.003	Ew
Tensile(steel)	0.005	0.05
Compressive(steel)	Limited by the concrete	0.02

Table 16.10c—Manufacturer's reported FRP System properties

Thickness per plyt	0.023 in.	0.584 mm
Ultimate tensile strengthj	155 ksi	1072 MPa
Rupture strain en	0.015	0.015
Modulus of elasticity,E	9326 ksi	64.3 GPa

Table 16.10d—Procedure for designing plastic hinge confinement for seismic strengthening

<table>
<tr><th>Procedure</th><th>Calculation in in.-Ib units</th><th>Calculation in SI units</th></tr>
<tr><td>Step1—FRP-system design material properties:
En=Cen
Cg=0. 95
The column is located in an interior space. Therefore, per Table 9.4, an environmental-reduction factor of 0.95 is used.
Φ=0. 004≤KEm</td><td>Em≤0. 95×0. 015=0. 014
K_o=0. 55
Ee=0. 004≤0. 55×0. 014=0. 0078Ee=0. 04</td><td>%=0. 04</td></tr>
<tr><td rowspan="2">Step 2—Plastic hinge length:
In FRPjacketed columns, the plastic hinge length is:
Lp=g+0. 0003f, dn(psi and in.)
Lp=g+0. 044/, d3 (MPa and mm)</td><td>Lp=2+0. 0003×44, 000×0. 75=12 in.</td><td>L, =50. 8+0. 044×303. 4×19=305 in.</td></tr>
<tr><td colspan="2">Note:Use a maximum gap between the FRPand the column base of 2 in. (50.80 mm)</td></tr>
<tr><td rowspan="2">Step 3—Preliminary calculations:
b=h=16 in. (406mm)
From 12. 1. 2:
$D=\sqrt{h^2+b^2}$
$\kappa_a=\frac{A_e}{A_c}\left(\frac{b}{h}\right)^2$
$\kappa_b=\frac{A_e}{A_c}\sqrt{\frac{h}{b}}$
where AJ/A is calculated as:
$\frac{A_e}{A_c}=\frac{1-\left[\frac{\left(\frac{b}{h}\right)(h-2r_c)^2+\left(\frac{h}{b}\right)(b-2r_c)^2}{3A_g}\right]-\rho_g}{1-\rho_g}$</td><td>$D=\sqrt{16^2+16^2}=22.63$ in.
AJ/A. =0. 62
κ_a $0.62\left(\frac{A_e}{A_c}\right)^2$ 0.62
κ_b $0.62\sqrt{\frac{16}{16}}$ 0.62
8. ' =0. 002</td><td>DN $\sqrt{406.4^2\ 406.4^2}$ 575mm
AJA. =0. 62
Kn=0. 62
K_6=0. 62
B' =0. 002</td></tr>
<tr><td colspan="2">h/b=1. 00
Note:For rectangular members, plastic hinge confinement by jacketing is not recommended for members featuring side aspect ratios, h/b, greater than 1.5, or face dimensions, b or h, exceeding 36 in. (900 mm) unless testing demonstrates their effectiveness(13. 3. 1).</td></tr>
<tr><td rowspan="2">Step4—Confining concrete model variables:
Using the trial design of five plies, compute the confined concrete model parameters listed in the following:
$f_l=\frac{2E_f N t_f \varepsilon_{fe}}{D}$
Ja' =' +y, 3. 3k4fi
$\varepsilon_{ccu}=\varepsilon_c'\left(1.50+12\kappa_b\frac{f_l}{f_c'}\left(\frac{\varepsilon_{fe}}{\varepsilon_c'}\right)^{0.45}\right)$</td><td>$f_l=\frac{2\times 9,326,000\times 0.023\times 5\times 0.004}{22.63}=380$ psi
=0. 95 for fully wrapped sections
fa' =4000+0. 95×3. 3×0. 62×380=4740 psi
$\varepsilon_{ccu}=0.002\left(1.50+12\times 0.62\times\frac{380}{4000}\left(\frac{0.004}{0.002}\right)^{0.45}\right)$
=0. 0049≤0. 01</td><td>$f_l=\frac{2\times 64,300\times 0.584\times 5\times 0.004}{575}=2.61$ MPa
=0. 95
fa' =27. 6+0. 95×3. 3×0. 62×2. 61=32. 66MPa
$\varepsilon_{ccu}=0.002\left(1.50+12\times 0.62\times\frac{2.61}{27.60}\left(\frac{0.004}{0.002}\right)^{0.45}\right)$
=0. 0049≤0. 01</td></tr>
<tr><td colspan="2">Bcu is limited to 0. 01 to prevent excessive cracking and the resulting loss of concrete integrity</td></tr>
<tr><td colspan="3">Note:The expressions presented previously are used in conjunction with a moment-curvature(M-Φ)analysis program to obtain the yield and ultimate curvature. These curvatures are presented in Steps 5 and 6.</td></tr>
<tr><td rowspan="2">Step5—Output from M-Φ analysis program obtain the neutral axis at yielding of longitudinal reinforcement, Gxrp</td><td>Neutral axis:
Csi=5. 34in.</td><td>Neutral axis:
Cscm=136 mm</td></tr>
<tr><td colspan="2">Note:Yield strain of longitudinal reinforcement:e, =0. 00152</td></tr>
</table>

Table 16.10d, cont.—Procedure for designing plastic hinge confinement for seismic strengthening

Step6—Output from M-p analysis program obtain the neutral axis at ultimate,Ca,p	Neutral axis: Cup=1.95 in.	Neutral axis: Ciu=50mm
1.Concrete compression strain: Cc=0.0049	Note:The ultimate concrete compressive strain is computed per Step 4:Ecw=0.0049.	
2.Steel tension strain: $\varepsilon_s = \varepsilon_{ccu}\left(\frac{d}{c_u}-1\right)$	$\varepsilon_s = 0.0049\left(\frac{14.625}{5.34}-1\right) = 0.032 < 0.05$	$\varepsilon_s = 0.0049\left(\frac{371}{136}-1\right) = 0.032 < 0.05$
Step7—Compute the yield curvature: $\phi_{y,frp} = \frac{\varepsilon_y}{d - c_{y,frp}}$	From an M-o analysis the yield curvature is: $\phi_{y,frp} = \frac{0.0015}{14.625-5.34} = 0.000163/\text{in.}$	From an M-p analysis the yield curvature is: $\phi_{y,frp} = \frac{0.0015}{371-136} = 0.0064/\text{m}$
Step8—Compute the ultimate curvature capacity: $\phi_{u,frp} = \frac{\varepsilon_{ccu}}{c_{u,FRP}}$	From an M-analysis the ultimate curvature is: $\phi_{u,frp} = \frac{0.0049}{1.95} = 0.0025/\text{in.}$	From an M-p analysis the ultimate curvature is: $\phi_{u,frp} = \frac{0.0049}{50} = 0.099/\text{m}$
Step9—Compute the ultimate curvature demand: $\phi_D = \frac{\theta_p}{L_p} + \phi_{y,frp}$	$\phi_D = \frac{0.025}{12} + 0.000163 = 0.0022/\text{in.}$	$\phi_D = \frac{0.025}{0.305} + 0.0064 = 0.084/\text{m}$
Step 10—Verify design: ΦD<9u.i	D=0.0022/in.<0.0025/in. **OK**	中p=0 . 084/m<0 . 099/m **OK**
Step 11—Length of confined region,(m Transverse reinforcement as specified per Section 18.7 ofACI 318-19(22)shall be provided over a length l。	$\ell_o \geq \begin{cases} 16\text{ in.} \\ \frac{120/2}{6} = 10\text{ in.} \\ 18\text{ in.} \end{cases}$	$\ell_o \geq \begin{cases} 406.4\text{ mm} \\ \frac{3050/2}{6} = 254\text{ mm} \\ 457\text{ mm} \end{cases}$
	t>Lp=18 in. (457mm)	
Design summary: Completely wrap the section with five transverse plies. Confining jacket should extend at least 18 in. (457mm)beyond the joint interface.		

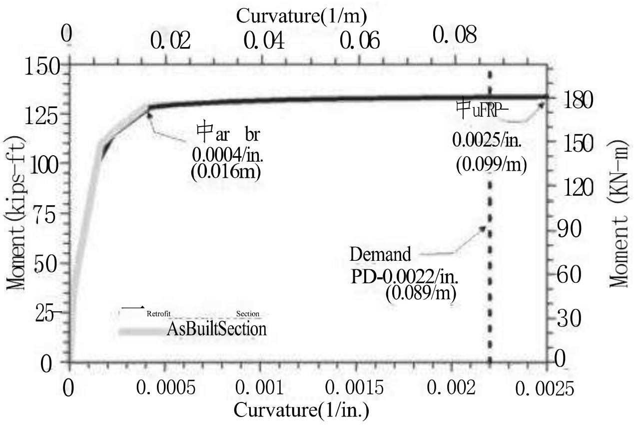


Fig.16.10b—Moment-curvature analysis.

Figure 16.10b shows the moment curvature analysis of the as-built and retrofit rectangular sections.The moment curvature analysis results show the as-built ultimate curvature capacity is significantly lower than the required curvature demand.

Table 16.11a—Column material properties

Concrete strengthf	4000 psi	27.6 MPa
Concrete elastic modulus E.=57,000 √:(psi) E,=4700 √(MPa)	3605ksi	25 GPa
Longtudinal reinforcing steel:yield strengthf	44,000 psi	303.4 MPa
Modulus of elasticity of steel E,	29,000 ksi	200 GPa
Longitudinal reinforcing steel:yield strain e,	0.0015	0.0015
Column height between plastic hinges,L	10 f	3.05m
Distance to extreme tension steel,d	14.625 in.	371mm

16.11—Lap-splice clamping for seismic strengthening

This example illustrates the design of an FRP retrofit to improve the seismic performance of a reinforced concrete column that is constructed with a lap splice in a region of plastic rotations.Material properties,details,and other relevant information are provided in Table 16.11a and Fig.16.11.The column is strengthened with CFRPlaminates having the composite properties listed in Table 16.11b and bonded around the column using the wet layup technique.

The final wrapping system was found to require five plies. Only the calculations used to verify the final configuration are provided.These calculations are shown in Table 16.11c.

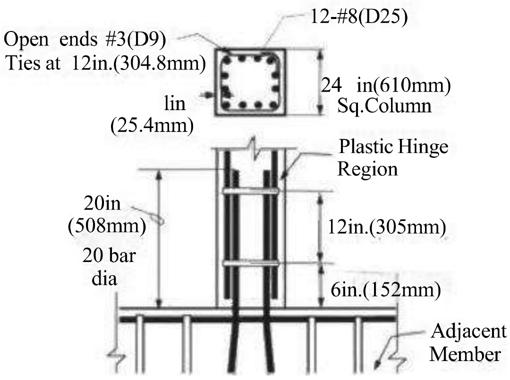


Fig.16.11—Detail of lap splice in a plastic hinge region.

Table 16.11b—Manufacturer's reported FRP system properties

Thickness per ply t	0.08 in.	2mm
Ultimate tensile strength	143ksi	986 MPa
Rupture strain	0.010	0.010
Modulus of elasticity Ey	13,900 ksi	95.8 GPa

Table 16.11c—Procedure for designing lap-splice clamping for seismic strengthening

Procedure	Calculation in in.-Ib units	Calculation in SI units
Step1—Compute the tensile stress that existing splice can develop:(ACI 318-19(22)Section 25.4) $$f_s=\frac{40\ell_d\lambda\sqrt{f'_c}\left(\frac{c_bK_{tr}}{d_b}\right)}{3d_b\psi_t\psi_e\psi_s}\text{ (in. and psi)}$$ $$f_s=\frac{3.33\ell_d\lambda\sqrt{f'_c}\left(\frac{c_bK_{tr}}{d_b}\right)}{3d_b\psi_t\psi_e\psi_s}\text{ (mm and MPa)}$$ $$K_{tr}=\frac{40A_{tr}}{sn}$$ Y,=Ve=V.=1.00 A=1.00(normalweight concrete)	$s = 12$ in. $n = 1$ Longitudinal bars in the potential plane of splitting $$K_{tr}=\frac{40\times1\times0.11}{12\times1}=0.37\text{ in.}$$ $$\frac{c_b+K_{tr}}{d_b}=\frac{(1.00+0.375+0.5)+0.37}{1.00}=2.24\leq2.50$$ Per Eq. (25.4.2.4a)ofACI 318-19(22) the value $\frac{c_b+K_{tr}}{d_b}$ cannot be greater than 2.5. $$f_s=\frac{40\times20.0\times1.0\sqrt{4000}\times2.24}{3\times1.0\times1.0\times1.0\times1.0}$$ =37,779 psi≤44,000 psi	$s = 305$ mm $n = 1$ $$K_{tr}=\frac{40\times1\times71}{305\times1}=9.3\text{ mm}$$ $$\frac{c_b+K_{tr}}{d_b}=\frac{(25.4+9.53+12.7)+9.3}{25.4}$$ =2.24≤2.50 $$f_s=\frac{3.33\times508\times1.0\sqrt{27.6}\times2.24}{3\times25.4\times1.0\times1.0\times1.0}$$ =260 MPa≤303 MPa
	Note:Computed stressf,does not reachf,longitudinal bar yield strength,and as such lap splice must be clamped.	

Table 16.11c, cont. —Procedure for designing lap-splice clamping for seismic strengthening

Step 2—Stress corresponding to pullout eapacity of splice $f_s \le \frac{33\ell_d\lambda\sqrt{f'_c}}{d_{bc}\psi_t\psi_e\psi_s}$ (in. and psi) $f_s \le \frac{2.75\ell_d\lambda\sqrt{f'_c}}{d_{bc}\psi_t\psi_e\psi_s}$ (mm and MPa)	p=20dα=20×1.0=20 in. Pullout capacity of splice: $f_s = \frac{33\times 20.0\times\sqrt{4000}}{1.0\times 1.0\times 1.0\times 1.0} = 41{,}742$ psi	=20du=20×25.4=508mm Pullout capacity of splice: $f_s = \frac{2.75\times 508\times\sqrt{27.6}}{25.4\times 1.0\times 1.0\times 1.0} = 289$ MPa
Step3—Compute the required jacket thickness, $Nt_f = 218\times\frac{D}{E_f}$ (ksi and in.) $Nt_f = 1500\times\frac{D}{E_f}$ (MPa and mm)	D=24in. (maximum dimension in a rectangular member) E=13,900ksi $t_j = 218\times\frac{24}{13{,}900} = 0.38$ in.	d=610mm E,=95,800 MPa $t_j = 1500\times\frac{610}{95{,}800} = 9.6$ mm
Step4—Compute the required number of plies, N N=t/1	t=0.08 in. N=0.38/0.08=five plies	ty=2mm N=9.60/2=five plies
Design summary: Completely wrap the section with five transverse plies. Confining jacket should extend at least full height of lap splice—that is, 20 in. (508mm) Using five plies of the FRP-specified results in a splice capacity of41,742 psi (289 MPa), which is controlled by the maximum pullout capacity calculated in Step 2.		

16.12—Seismic shear strengthening

This example illustrates the main steps in calculating the amount ofFRPrequired for the shear strengthening of a reinforced concrete member.The member used in this example is illustrated in Fig.16.12.Column material properties are shown in Table 16.12a,the configuration of the FRP shear reinforcement is described in Table 16.12b,and the CFRP laminate material properties are listed in Table 16.12c. ASCE/SEI 41 is used as the standard for this example. Calculations for determining the FRP required are shown in Table 16.12d.

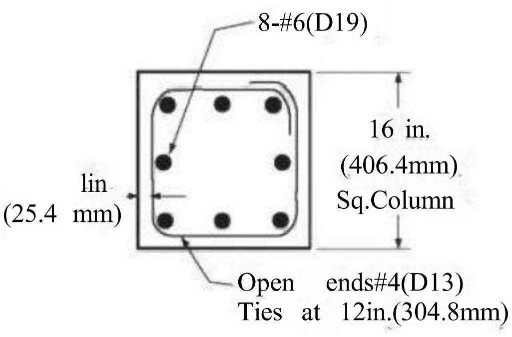


Fig.16.12—As-built column details.

Table 16.12a—Column material properties

Concrete strengthf	4000 psi	27.6 MPa
Concrete elastic modulus E.=57,000 √:(psi) $E_c = 4700\sqrt{f'_c}$ (MPa)	3605ksi	25 GPa
Longtudinal reinforcing steel:yield strengthf,	44,000 psi	303.4 MPa
Modulus of elasticity of steel E,	29,000 ksi	200 GPa
Longitudinal reinforcing steel:yield strain e,	0.0015	0.0015
Column height between plastic hinges, L	10f	3.05m
Distance to extreme tension steel, d	14.625 in.	371 mm
Ultimate axial load P.	75 kip	333.62kN

Table 16.12b—Configuration of supplemental FRP shear reinforcement

Minimum section dimension	16 in.	406 mm
d_3 (governed by minimum section dimension)	16 in.	406 mm
Width w	8 in.	203mm
Spacing s(full coverage)	8 in.	203mm

Table 16.12c—Manufacturer's reported FRP system properties

Thickness per plyt	0.023 in.	0.584 mm
Ultimate tensile strengthf*	155ksi	1072 MPa
Rupture strain em*	0.015	0.015
Modulus of elasticity E	9326 ksi	64.3 GPa

Table 16.12d—Procedure for seismic shear strengthening

<table>
<tr><th>Procedure</th><th>Calculation in in.-1b units</th><th>Calculation in SI units</th></tr>
<tr><td rowspan="2">Step1—Compute the probable moment capacity of the member,Mm,per 13.5.1;in calculating the probable moment Φ=1 and y-=1 are used.</td><td>From a M-analysis the probable moment capacity is:
Mm=322 kip-ft</td><td>From aM-analysis the probable moment capacity:
Mm=437kN-m</td></tr>
<tr><td colspan="2">Notes:1)In seismic applications,the shear design process initiates with the calculation of the probable moment capacity of the section;and 2)moment M,is computed at both top and bottom ends of the member.</td></tr>
<tr><td>Step 2—Obtain the yield and ultimate curvature from an M-panalysis:</td><td>Yield curvature:
9=0.000163/in.

Ultimate curvature:
中usp=0.0019/in.</td><td>Yield curvature:
中up=0.0064/m

Ultimate curvature:
中unp=0.074/m</td></tr>
<tr><td>Step 3—Compute the displacement ductility and reduction factor,k.In FRPjacketed columns,the plastic hinge length is (Eq.(1):

L,=g+0.0003f,d
f,is in ksi and d,is in inches

Lp=g+0.044,dn
f is in MPa and d,is in mm

In this example:
$$L_{eff}=\frac{L}{2}=60\text{ in. }(1524\text{ mm})$$
Yield deflection△:
$$\Delta_{y,frp}=\frac{\phi_{y,frp}L_{eff}^2}{3}$$
Plastice defiection△p
$$\Delta_{p,frp}=(\phi_u-\phi_y)L_p\left(L_{eff}-\frac{L_p}{2}\right)$$
Displacement ductility A:
$$\mu_\Delta=1+\frac{\Delta_p}{\Delta_y}$$
Shear reduction factor per ASCE/SEI41
$$\begin{cases}\mu_\Delta\le 2; & k=1.0\\ 2\le\mu_\Delta\le 6; & k=0.7+\dfrac{0.30\times(6-\mu_\Delta)}{4}\\ \mu_\Delta>6; & k=0.7\end{cases}$$</td><td>Plastic hinge length:

L=2+0.0003×4,000×0.75=12 in.

Note:Use a maximum gap between the FRP and the column base of 2 in.

Yield deflection:
$$\Delta_{y,frp}=\frac{0.000163\times 60^2}{3}=0.20\text{ in.}$$
Plastic deflection:
$$\Delta_{p,frp}=(0.0019-0.000163)\times 12\left(60-\frac{12}{2}\right)=1.13\text{ in.}$$
Displacement ductility:
$$\mu_\Delta=1+\frac{1.13}{0.20}=6.7$$
Reduction factor:
k=0.70</td><td>Plastic hinge length:

Lp=50.8+0.044×303.4×19=305 mm

Note:Use a maximum gap between the FRP and the column base of 50.80 mm.

Yield deflection:
$$\Delta_y=\frac{0.0065\times 1524^2}{3\times 1000}=5.1\text{ mm}$$
Plastic deflection:
$$\Delta_{p,frp}=\frac{(0.074-0.0064)}{1000}305\left(1524-\frac{305}{2}\right)=28.6\text{ mm}$$
Displacement ductility:
$$\mu_\Delta=1+\frac{28.6}{5.1}=6.7$$
Reduction factor:
k=0.70</td></tr>
</table>

Table 16.12d, cont.—Procedure for seismic shear strengthening

Step4—Compute the shear design force, V_4 Per ACI 318-19(22) Section 18.7.6: $$V_u = \frac{M_{pr,top} + M_{pr,bot}}{L} \pm \frac{w_u L}{2}$$	In this example, with the exception of the earthquake forces, there are no applied uniform loads on the member, as such: W=0.00 From Step 1, the probable moment capacity is: Mmop=Mmcbo=322 kip-ft(437kN-m)	
	Design shear force: $$V_u = \frac{322+322}{120/12} \pm 0 = 64.4\ \text{kip}$$	Design shear force: $$V_u = \frac{437+437}{3.05} \pm 0 = 286\ \text{kN}$$
Step 5—Calculate the concrete, V., and steel contribution, V_3 Concrete contribution, V: Per ACI 318-19(22) Section 18.7.6.2.1, and because: $$\frac{M_{pr,r} + M_{pr,l}}{\ell_m} \geq \left(\frac{w_u \ell_m}{2}\right)\frac{1}{2} \Rightarrow$$ V.=0 Steel contribution V, $$V_s = A_{sh} f_y \frac{d}{s}$$	Concrete contribution to shear capacity: V=0 Steel contribution to shear capacity: $$V_s = 2 \times 0.20 \times 44 \times \frac{14.625}{12} = 21.4\ \text{kip}$$ Combined concrete and steel contribution: k(Ve+V.)=0.70(0+21.4)=14.98 kip	Concrete contribution to shear capacity: V_6=0 Steel contribution to shear capacity: $$V_s = 2 \times 129 \times 303.4 \times \frac{371.475}{305} = 95.2\ \text{kN}$$ Combined concrete and steel contribution: k(Ve+V)=0.70(0+95.2)=66.6kN
Step6—Calculate the required FRPforce, V; $$V_{f,R} = \frac{\left[\frac{V_u}{\phi} - k(V_c + V_s)\right]}{\psi_f}$$ Per ASCE/SEI41, Φ=1.00 For completely wrapped members, Table 11.3 recommends y=0.95.	Required FRP force: $$V_{f,R} = \frac{\left[\frac{64.4}{1.00} - 14.98\right]}{0.95} = 45.26\ \text{kip}$$	Required FRP force: $$V_{f,R} = \frac{\left[\frac{286}{1.00} - 66.6\right]}{0.95} = 201.3\ \text{kN}$$
Step 7—Calculate the efective stress, fje For fully wrapped members, the effective strain is computed using Eq. (11.4.1.1); %=0.75C=0.90.004 The column is located in an interior space. Therefore, per Table 9.4, an environmental-reduction factor of0.95 is used The effective FRP stress can be computed from Hooke's law: 一 E	Effective strain; &c=0.75×0.95×0.015=0.11≤0.004 Effective stress: fo=0.004×9326=37.3ksi	Effective strain: Ee=0.75×0.95×0.015=0.11≤0.004 Effective stress: Jo=0.004×64,300=257MPa
Step 8—Calculate the number of plies, N. Area of a single ply for a fully wrapped member (Eq. (11.4a) A,=2tw; The shear contribution of the FRP can be then calculated from Eq. (11.4a). $$V_f = A_{fv} f_{fe} \frac{(\sin\alpha + \cos\alpha) d_{fv}}{s_f}$$	Area per ply: An=2×0.023×8=0.37 in.² Force per ply: $$V_f = 0.37 \times 37.3 \frac{(1)16}{8} = 27.6\ \text{kip}$$ Number of plies: $$N = \frac{V_{f,R}}{V_f} = \frac{45.26}{27.6} = 2(\text{minimum})$$	Area per ply: Ap=2×0.584×203=237mm² Force per ply: $$V_f = 237 \times 257 \frac{(1)406}{203} = 122\ \text{kN}$$ Number of plies: $$N = \frac{V_{f,R}}{V_f} = \frac{201.3}{122} = 2(\text{minimum})$$
Design summary: Completely wrap the section with two transverse plies.		

16.13—Flexural and shear seismic strengthening of shear walls

This example illustrates the use of an FRP retrofit to increase the shear and flexural strength of an existing cantilevered concrete wall. The shear strengthening is achieved by installing horizontally oriented FRP on one face of the wall. The flexural strengthening is achieved by installing multiple layers of vertically oriented (longitudinal) FRP on both faces of the wall near the wall ends. Details for anchorage of the FRP to the foundation are not addressed. Carbon FRP (CFRP) is used in this example.

The example incorporates the following two major phases:

1. Determine the shear and flexural capacity of the existing wall
2. Design the FRP to achieve the required strength

Details of the wall and relevant information are provided in Fig. 16.13a and Table 16.13a. The wall is assumed to be an ordinary shear wall, that is, the wall is at risk of a brittle shear failure. Plastic hinging or inelastic deformations are not expected. The wall is strengthened with FRP having the composite properties listed in Table 16.13b. A factored axial load, P, of 12 kip (53 kN) is assumed in addition to the lateral force. ASCE/SEI 41 is assumed to be the standard used as the basis for the rehabilitation and shear is considered a force-controlled action.

This example illustrates a manual calculation approach for the design of FRP strengthening of a shear wall. A moment curvature analysis of the existing and repaired wall, shown in Fig. 16.13b, is used to assess the accuracy of the manual approach. The reasonably good agreement of the flexural results from the design example with those from a moment-curvature analysis validates the illustrative example. The design calculations are shown in Table 16.13c.

Per 13.7.2.2, the longitudinal FRP should be anchored to the foundations to ensure load path continuity and, where feasible, confined or anchored (refer to 13.7.2.2).

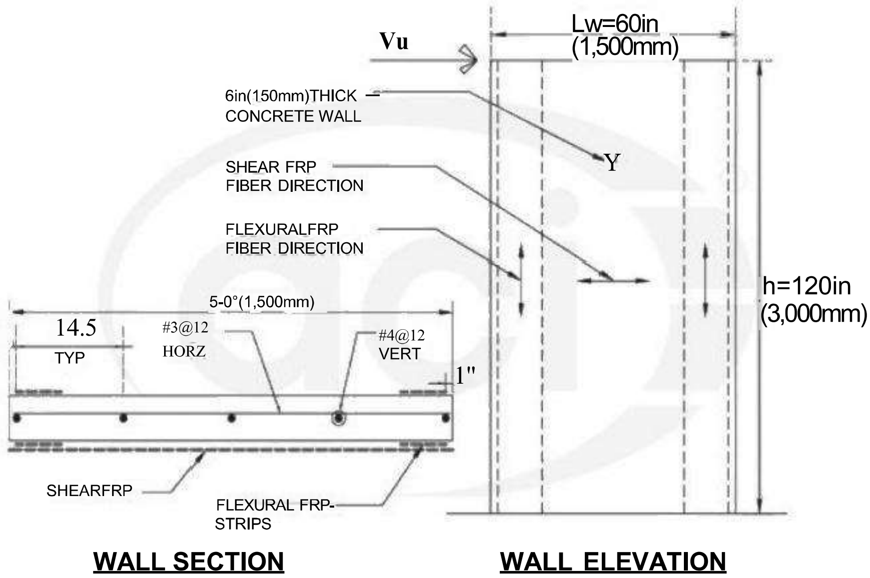


Fig. 16.13a—Concrete wall details.

Table 16.13a—As-built shear wall properties and demands

Concrete strength	2500 psi	17.23 MPa
Longitudinal reinforcing steel yield strength	40.0 ksi	275.8 MPa
Modulus of elasticity of steel	29,000 ksi	200 GPa
Longitudinal reinforcing yield strain	0.0014 in./in.	0.0014 mm/mm
Shear wall height h	10f	3000mm
Shear wall length	60.0 in.	1500 mm
Shear wall thickness	6 in.	150 mm
Existing wallreinforcement Horizontal: No.3 at 12 in. (305mm) on center=>p_2=0.0015 Longitudinal: No.4 at 12 in. (305mm) on center=>pe=Pb=0.0027	—	—
Axial factored load P	12 kip	53.4kN
Design shear demand V (force-controlled action)	52 kip	232kN
Design moment demand at wall base M	260 kip-ft	348 kN-m
Asw=area of wall web steel=>area of four No.4 (No.M13) bars (neglecting one bar within the compression zone)	0.8 in.^{2}	500 mm^2

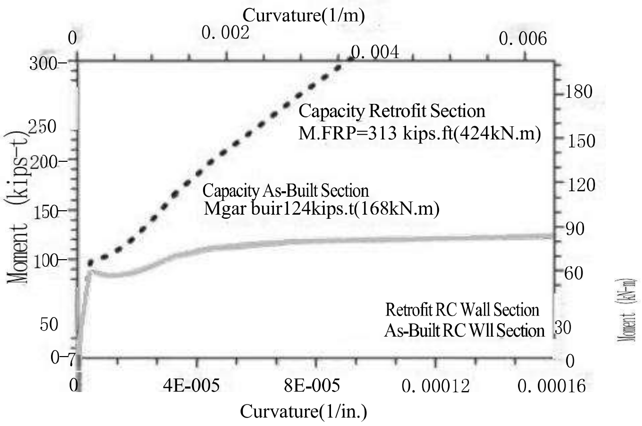


Fig.16.13b—Moment-curvature analysis for shear wall example.

Table 16.13b—Manufacturer's reported FRP system properties

Thickness per plyt	0.023 in.	0.575mm
Ultimate tensile strengthf	140ksi	965.3 MPa
Rupture strain en	0.012 in./in.	0.012mm/mm
Modulus of elasticity E	9600 ksi	66.2GPa

Table 16.13c—Procedure for flexural and shear seismic strengthening of shear walls

Procedure	Caleulation in in.-Ib units	Calculation in SI units
Step1—Compute existing wall capacity Shear capacity: Shear capacity per ACI 318-14:		
V.=2λJ?ad (in-1b)	$V_c = \frac{2(6\text{ in.}\times 0.8\times 60\text{ in.})\sqrt{2500\text{ psi}}}{1000}$	$V_c = \frac{(0.167\times 150\text{mm}\times 0.8\times 1500\text{mm})\sqrt{17.23\text{MPa}}}{1000}$
$V_c = \frac{0.167 b_w d\sqrt{f_c'}}{1000}$ (SI)	V.=28.8kip	V.=124.8 kN
$V_{sw} = \frac{A_v f_y d_{fv}}{s}$	$V_{sw} = 0.11\text{ in.}^2\times 40{,}000\text{psi}\times\frac{0.8\times 60\text{ in.}}{12\text{ in.}}$ $V_{sw} = 17.6$ kip	$V_{sw} = \frac{71\text{mm}^2\times 275.8\text{ MPa}\times 0.8\times\frac{1500\text{mm}}{305\text{mm}}}{1000}$ Vm=77.0kN
V,=V.+V	Va=228.8 kip + 17.6 kip $V_n = 46.4$ kip	Va=124.8kN+77.0kN V.=201.8kip
Using Φ=1.0 for shear (per ASCE/SEI 41):	ΦVn=46.4 kip<V₄=52kip	ΦVa=201.8kN<V=232 kN
Flexural capacity: Aquick method to assess the flexural capacity is shown in the following.		
Assume that all web steel in the wall yields This includes allthe longitudinal reinforcement except for the one bar that is adjacent to the compression face, that is, four No.4 bars can be considered to yield. $a = \frac{A_{sw} f_y + P_u}{0.85 f_c' t_w}$	$a = \frac{0.8\text{ in.}^2\times 40\text{ ksi} + 12\text{ kip}}{0.85\times 2.5\text{ ksi}\times 6\text{ in.}} = 3.45$ in.	$a = \frac{\left(\frac{500\text{ mm}^2\times 275.8\text{ MPa}}{1000}\right) + 53.4\text{ kN}}{\left(\frac{0.85\times 17.23\text{ MPa}\times 150\text{ mm}}{1000}\right)} = 87.1$ mm
M=(Af,+P)(d-a/2) where d=LW/2	d=L./2=30 in. M=104 kip-ft	d=750 mm Mn=135.1 kN-m
Using Φ=1.0: It can be observed that the wall does not have adequate capacity.	ΦM,=104 kip-t<M=260 kip-t	ΦM=135.1 kN-m<M=348kN-m

Table 16.13c,cont.—Procedure for flexural and shear seismic strengthening of shear walls

Step 2—FRP design material properties Use environmental reduction factors from Table 9.4.	For exterior exposure for carbon FRP:Cg=0.85	
Per 9.4: *fu=Cef* *B=CEBm*	fm=199ksi Em=0.0102 in./in.	fm=820.5 MPa E=0.0102 mm/mm
Step 3—Flexural strengthening		
Step 3a—Compute the debonding strain limit,eμu This is the limit for the cffective strain in the FRP. Per Eq.(10.1.1): In in.-1b units: $\varepsilon_{fd} = 0.083\sqrt{\frac{f'_c}{NE_f t_f}} \le 0.9\varepsilon_{fu}$	For one layer of FRP,eu=0.0088 For two layers of FRP,ey=0.0062 For three layers of FRP,a=0.0051	For one layer of FRP,en=0.0088 For two layers of FRP,e=0.0062 For three layers of FRP,ca=0.0051
In SI units: $\varepsilon_{fd} = 0.41\sqrt{\frac{f'_c}{NE_f t_f}} \le 0.9\varepsilon_{fu}$		
Flexural design involves iteration to achieve equilibrium across the section.Usually,the first step is to assume the depth of the neutral axis c.	Use the available information to assist with an assumption for the depth of the neutral axis c.	
Assume that the flexural strengthening will require three layers of 8 in.(200 mm)wide strips on each side of the wall at each end. Assume that the effective strain is at the centroid of the FRP area.Knowing the maximum effective strain in the FRP,compute the force in the FRP:	w=8 N=3 T;=0.023 in. Ay=two sides×3(0.023 in.)(8 in.) Ay=1.104 in.²	w=200 mm N=3 T=0.575 mm A;=690mm²
Ty=C₆AsE	T=0.85(1.104 in.?)(0.0051)(9600 ksi)=45.94kip	$T_f = \frac{0.85(690\ \text{mm}^2)(0.0051)(66{,}200\ \text{MPa})}{1000} = 198.0\ \text{kN}$
Depth of corresponding compression block: $a = \frac{A_{sw}f_y + P_u + T_f}{0.85 f'_c t_w}$	a=7.05 in.	a=177.2mm
Compute an estimate of the depth of the neutral axis,c:	c=a/0.85=8.3 in.	c=a/0.85=208.5mm
Check actual strain at centroid ofFRP area and corresponding force in the FRP:	$\varepsilon_{fe\,CG} = \varepsilon_{fd}\left(1+\frac{W_f/2}{c+1-L_u}\right) = 0.00469$	$\varepsilon_{fe\,CG} = \varepsilon_{fd}\left(1+\frac{W_f/2}{c+25.4-L_u}\right) = 0.00469$
It is observed that the force in the FRP does not agree with that based on theinitial assumption. However,the above steps provide a reasonable starting point for an assumption for c.	Corresponding force in FRP, T;=CAeccE=42.32 kip Assume c=8.0 in.	*Tı=CeAeycGE,=182.1kN* Assume c=200 mm

Table 16.13c, cont.—Procedure for flexural and shear seismic strengthening of shear walls

Compute concrete strain at extreme compression surface per Eq. (13.7.2.1): $\varepsilon_c = \varepsilon_{fd}\left(\frac{1}{(L_u/c)-1}\right) \le \varepsilon_{cu}$	8. =0.0008 &. <Bm=0.003 **OK**	E:=0.0008 B<&u=0.003 **OK**
Compute strain in the bar in the compression zone: $\varepsilon_{sc} = \left(\frac{\varepsilon_c}{c}\right)(c-d')$	Ese=0.007 where d=1in. e<e, =0.0014	Bμ=0.007 where d=25mm En<E, =0.0014
Compute strain in the bars in the tension zone:	$\varepsilon_{st3} = \left(\frac{\varepsilon_c}{c}\right)(14.5\text{ in.}+1\text{ in.}-8.0\text{ in.})$ En=0.0075<2,	$\varepsilon_{st1} = \left(\frac{\varepsilon_c}{c}\right)(362.5\text{ mm}+25\text{ mm}-200\text{ mm})$ En=0.0075<8y
Compute strain at centroid of FRParea:	Similarly, Bxz=0.0022>8,	Similarly, Eu_2=0.0022>8y
Recompute total tensile force components at the above determined strain levels:	Es3=0.0037>, Ex4=0.0051>e Ejecc=0.0047	Eu3=0.0037>8y E4=0.0051>E Becc=0.0047
Recalculate depth of compression block and depth to neutral axis:	T=42.34 kip Tm=0.2 in.^{2}×0.00075×29,000 ksi+3×0.2 in.?	T=182.5kN T_3w=125mm^2×0.00075×200,000 MPa+3×
Iterate as required to reach convergence:	×40ksi=28.35 kip	125mm^2×275.8 MPa=122,175kN Tm=122.2kN
Step 3b—Calculate the strains and force components in concrete, FRP, and reinforcing steel	a=6.48 in. c=7.63 in.	a=163mm c=191.8 mm
Force in the bar in the compression zone:	Final value of the depth of the neutral axis is achieved after iteration. c=7.648 in. a=6.5 in.	c=191 mm a=162.4mm
Strains and forces in bars in tensile zone:	B=0.00076<Cc =0.00066<8, $C_{sc} = 0.2 \times 0.00066 \times 29{,}000 = 3.83$ kip	$_5$=0.00076<Ecu E_6=0.00066<8, C=125×0.00066×200=16.5kN
Strain and force in FRP:	Bnri=0.0078<e, Es2=0.0022>6 Es3=0.0037>8 Es4=0.0051>E	Bai=0.0078<Ey Eua=0.0022>8y Eus=0.0037>8, Em4=0.0051>Ey
From equilibrium, compressive force in concrete:	Tn=4.52kip T_2=T_3=Tn_4=8.0 kip ∴T_3r=28.52kip	Tn=19.5kN Tn_2=Ts=Tn=34.48 kN ∴T, =122.94kN

Table 16.13c, cont.—Procedure for flexural and shear seismic strengthening of shear walls

Step 3c—Calculate the moment capacity of the section Compute lever arm for different force components:	Eeco=0.0047 T=42.37 kip C.=(.Ta+Ty+Pa+Ca)=79.06 kip Bar in compression: d_1=c−1in.=7.648−1=6.65 in. Concrete compression: $\left(c-\frac{a}{2}\right)=7.648\text{ in.}-\frac{6.5\text{ in.}}{2}=4.4\text{ in.}$ First bar in tension: (1 in.+14.5 in.−c)=7.85 in. Second bar in tension:22.35 in. Third bar in tension:36.85 in. Fourth bar in tension:51.35 in. Lm−c−W/2−1 in.=47.35 in. P_u: $L_w/2-c=22.35$ in.	jeco=0.0047 T=182.5kN C.=(.Ta+Ty+Pu+Ca)=342.4kN Bar in compression: d_1=c−25mm=166mm $\left(c-\frac{a}{2}\right)=109.8$ mm First bar in tension: (25mm+362.5mm−c)=196.5mm Second bar in tension:559mm Third bar in tension:921.5mm Fourth bar in tension:1284mm FRP: $L_w-c-W_f/2-25$ mm $=1184$ mm P_u: $L_w/2-c=559$ mm
Nominal moment capacity, M: Per Section 10.2.10, include reduction factor for FRP contribution, ψ=0.85.	Ma=79.06kip×4.4 in.+3.83 kip×6.65in.+ 4.52 kip×7.85 in.+8kip×22.35 in.+8 kip× 36.85 in.+8kip×51.35 in.+0.85×42.37 kip× 47.35 in.+12 kip×22.35 in. Mn=3266.7 kip-in. Mn=272.2 kip-ft ΦMa=272.2 kip-ft>M=260 kip-ft ΦM>M∴ OK	Mn=353kNm ΦM=353kN • m>Mn=348kNm ΦM,>M∴ OK

Table 16.13c,cont.—Procedure for flexural and shear seismic strengthening of shear walls

Step4—Shear strengthening Per 13.7.4,Eq. (13.7.2.4b): V is the shear contribution of FRPand is computed in accordance with Chapter 11.	Vn=V+w,V From Step 1,V=46.4kip	$V_n = V_n^* + \psi_f V_f$ From Step 1,V*=201.8kN
Step 4a—Calculate V; Because the FRPwill be face-bonded to the wall,the effective FRPstrain will be: Be=K,m≤0.004 Use Eq. (11.4.1.2b)to compute K, $\kappa_v = \frac{k_1 k_2 L_e}{468\varepsilon_{fu}} \leq 0.75$ (in.-1b) $\kappa_v = \frac{k_1 k_2 L_e}{11{,}900\varepsilon_{fu}} \leq 0.75$ (SD) Laky and kzare calculated from Eq.(11.4.1.2c), (11.4.1.2d),and(11.4.1.2e),respectively Note:The small difference in k,is due to soft conversions in the FRP system thickness and in Eq. (11.4.1.2d)for k.	For one layer of the FRP: L.=1.99 in. k_1=0.731 k_2=0.91 K,=0.277 B%=0.277×0.0102=0.00283	For one layer of the FRP: L=51.36mm k_1=0.741 k_2=0.91 K,=0.285 %=0.285×0.0102=0.0029
Corresponding tensile stress in the FRP: Per 13.7.4.1,d,is taken as 0.8L Compute shear capacity of FRP strengthened wallusingΦ=1.0 and Eq. (13.7.2.4c)for one-sided strengthening: V,=0.75AneAEdv/S; FRPis installed over the full height of the wall and not in discrete strips. Am=Nid, N=1 d/sy=1.0	d=0.8L=48 in. V=0.75×0.023 in. ×48 in. ×0.00283×9600ksi =22.49 kip Reduction factor for FRPshear contribution,v=0.85 Vy=19.11 kip Shear capacity of the retrofitted wall: ΦV,=1.0(46.4 kip+19.11 kip) 中V_6=65.51 kip>V=52kip ΦV,>V: OK Ma=272.2 kip-ft	dn=0.8Lw=1200 mm V=0.75×0.575mm×1200mm×0.0029×66.2GPa =99.35kN WV=84.45kN ΦV=1.0(201.8kN+81.5kN) ΦVa=283.3kN>V=232 kN oVn>V∴OK M=353kNm
Step 4b—Compute the shear corresponding to the nominal flexural strength.Refer to Section 13.7.1. Note:The FRP for shear strengthening could be optimized by using horizontal strips rather than ull coverage.	Shear corresponding to the nominal flexural strength: Vumom=272.2 kip-f/10ft=27.2kip The shear strength of the wallis: Vn=65.3 kip V△Vunom.OK	VMnmm=353kNm/3m=117.1 kN The shear strength of the wallis: Va=284.6kN Vn∴VMnoOK

16.14—Flexural strengthening of continuous unbonded prestressed concrete slab with FRP laminates

Figure 16.14a shows a design example ofa 10in.(250 mm) thick one-way continuous unbonded post-tensioned slab.The tendons are 1/2 in.(12.7 mm)seven-wire stress-relieved strands,Grade 270 ksi(1860 MPa)having area/strand of 0.153 in.^{2}(98.7 mm^2)spaced at 10 in.(250 mm)apart (Ap =0.184 in.^{2}/ft [390 mm^2/m]).Ordinary steel reinforcement consisting of No.4(No.M13)bars spaced at 10 in.(250 mm) (A.=0.236 in.?/f [500 mm^2/m])and placed at a depth d= 8.75 in.(222 mm)is provided at the top and bottom of the slab throughout the span length as minimum bonded reinforcement required in members with unbonded tendons by ACI 318.The prestressing steel,reinforcing steel,and concrete material design properties are given in Table 16.14a.This example is intended only to illustrate application offlexural design provisions covered in Section 10.3.2;other criteria,such as serviceability,might govern the overall design of the upgraded slab.

The slab was originally designed for a superimposed dead load of92 Ib/t^2(4.4kN/m^2)and live load of 40.91b/f^2(2.0 kN/m^2) of slab surface area.The effective prestressing force Pe=26.5k/ft(388 kN/m)and the depths dpi,dp_2,and dp_3(or eccentricities) of the prestressing steel at critical sections 1,2,and 3,respectively (Fig.16.14a),were selected such that the prestressing force at service balances the self-weight of the slab.It is required to renovate the floor and raise the load capacity of the slab by using carbon FRP laminates at the critical positive and negative moment regions of the slab for resisting an upgraded live load of 160 Ib/f^2(7.7 kN/m^2).At the same time,to compensate partly for the increased live load,renovation measures were taken to reduce the design superimposed dead load from 921b/f^2(4.4 kN/m^2)to 701b/f^2(3.4 kN/m^2).The manufacturer's reported FRP properties are given in Table 16.14b.

A summary of moments produced by self-weight of the slab,reduced superimposed dead load,and upgraded live-load(for different loading patterns to produce maximum live-load moment M)is provided in Table 16.14c.Included also in Table 16.14c is the secondary moment due to prestressing Mo corresponding to Pe=26.5 kip/ft(388 kN/m).M。is calculated as the difference between the moment produced by the upward equivalent uniform load of the prestressing force(equal to the self-weight of the slab of 0.125 k/f^2 [6.0 kN/m^2])and the internal moment caused by the effective prestressing force of Pe.Without FRP strengthening, the ultimate moment capacity of the slab φM is calculated at 24.8 kip-f/ft(110.0kN-m/m),27.6 kip-ft/t(123.0 kN-m/m),and 26.7 kip-f/ft(119.0kN-m/m)at sections 1,2,and 3,respectively,which are all lower than the applied ultimate moment Mat the respective sections(Table 16.14c).

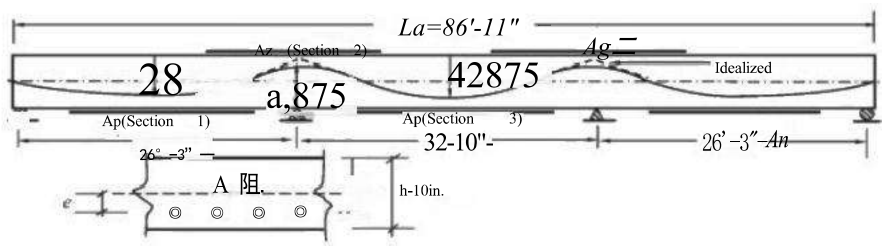
Starting with one FRP layer,a trial and error procedure should be used for estimating the width b,of the FRP sheets, or the area A;=bjy per unit width of the slab such that φM ≥M at every critical section,where φM,is the ultimate load capacity of the FRP-strengthened slab per unit width at the section under consideration.It was decided to use a single layer of 600 mm (23.6 in.)wide strips per meter(3.28 ft) width of the slab at the critical negative moment section (section 2)[(A_2=0.284 in.^{2}/ft [600 mm^2/m]),and 300 mm (11.8 in.)wide strips per meter(3.28 ft)width of slab at each of the critical positive moment sections(sections 1 and 3) [An=Ag=0.142 in.^{2}/ft(300 mm^2/m)].

Table 16.14a—Concrete,prestressing steel,and reinforcing steel properties

	5000 psi	34.5 MPa
A	0.184in.^{2}/ft	390mm^{2}m
fu	270 ksi	1860 MPa
Jeu	230 ksi	1585 MPa
Ep	28,500ksi	196,500 MPa
f	145ksi	1000 MPa
A,	0.236 in.^{2}/ft	500mm^2/m
f,	60ksi	414MPa
E,	29,000 ksi	200,000 MPa

Table 16.14b—Manufacturer's reported FRP system properties

Thickness per ply,t	0.0394in.	1 mm
Ultimate tensile strength,fa	140ksi	965 MPa
Rupture strain,E	0.01 in./in	0.01 mm/mm
Modulus of elasticity of FRPlaminates,E	13,895ksi	95,800 MPa



h	10 in	250 mm
d	8in	203mm
d_2	8.75 in	222 mm
ds	8.75in	222mm
ei	3 in	76mm
e_2	3.75in	95mm
e	3.75 in	95mm

Fig.16.14a—Schematic of idealized multi-span unbonded post-tensioned one-way slab with FRP external reinforcement. (Note:I in.=25.4 mm.)

Table 16.14c—Summary of applied moments per unit width of slab

Loading/moments	Positive moment section in the exterior span (section 1)		Negative moment section at the interior support(section 2)		Positive moment section in the interior span(section 3)	
Slab moment	5.9kip-f/ft	26.3kN-m/m	-11.1 kip-f/ft	49.5kN-m/m	5.8 kip-ft/ft	25.9kN-m/m
Superimposed dead-load moment	3.3 kip-f/f	14.7kN-m/m	-6.2 kip-ft/t	-27.6 kN-m/m	3.2 kip-ft/f	14.3 kN-m/m
Total dead-load moment, Mo	9.2 kip-f/f	41.0kN-m/m	-17.3 kip-ft/t	-77.1 kN-m/m	9.0 kip-ft/ft	40.2kN-m/m
Maximum live-load moment, M	11.5kip-f/t	51.3kN-m/m	-16.0 kip-f/ft	-71.3 kN-m/m	12.2 kip-f/t	54.4kN-m/m
Span(s) loaded to produce maximum live-load moment	Two exterior spans		Two adjacent spans		Interior span	
Secondary moment due to prestressing, M	1.2 kip-f/t	5.4kN-m/m	2.8 kip-f/ft	12.5kN-m/m	2.8 kip-ft/ft	12.5kN-m/m
Ultimate moment, M.*	30.6 kip-ft/ft	137.0kN-m/m	-43.6 kip-f/ft	-194.0kN-m/m	33.1 kip-t/ft	148.0kN-m/m

M_2=1.2Mou+1.6Mu+M。

Table 16.14d—Procedure for flexural strengthening of an interior unbonded prestressed concrete slab with FRP

Procedure	Calculations in in.-1b units	Caleulations in SI units
Step1—Preliminary calculations Concrete modulus of elasticity: $$E_c = 57,000\sqrt{f_c'}$$	E=57,000 √5000 psi=4,030,000 psi	E。=4700 √34.5MPa=27,600 MPa
Effective prestressing force: P。=A, de	P。=(0.184 in.²/f)(145ksi)=26.7 kip/t	P.=(388mm²/m)(1000 MPa/1000=388kN/m
Effective strain in the unbonded prestressing stecl: $$\varepsilon_{pe} = \frac{f_{pe}}{E_p}$$	$\varepsilon_{pe} = \frac{145\text{ ksi}}{28{,}500\text{ ksi}} = 0.0051$ in./in.	$\varepsilon_{pe} = \frac{1000\text{ MPa}}{196{,}500\text{ MPa}} = 0.0051$ mm/mm
Step 2—Calculate the FRPsystem design properties. The slab is located in an interior space(interior exposure) and CFRPmaterial will be used. Therefore, per Table 9.4, an environmental reduction factor of 0.95 is suggested $$f_{fu} = C_E f_{fu}^*$$ $$\varepsilon_{fu} = C_E \varepsilon_{fu}^*$$	fa=(0.95)(140ksi)=133ksi 咖=(0.95)(0.01in./in.)=0.0095 in./in.	J=(0.95)(965 MPa)=917MPa Eu=(0.95)(0.01 mm/mm)=0.0095mm/mm
Step 3—Determine the design strain of the FRP system The design strain of FRPaccounting for debonding failure is calculated using Eq. (10.1.1) $$\varepsilon_{fd} = 0.083\sqrt{\frac{f_c'}{NE_f t_f}} \le 0.9\varepsilon_{fu} \quad \text{(in.-lb)}$$ $$\varepsilon_{fd} = 0.41\sqrt{\frac{f_c'}{NE_f t_f}} \le 0.9\varepsilon_{fu} \quad \text{(SI)}$$	$\varepsilon_{fd} = 0.083\sqrt{\frac{5000\text{ psi}}{(1)(13{,}895{,}000\text{ psi})(0.0394\text{ in.})}}$ =0.0079≤0.9(0.0095)=0.0086	$\varepsilon_{fd} = 0.41\sqrt{\frac{34.5\text{ MPa}}{(1)(95{,}800)(1)}}$ =0.0079≤0.9(0.0095)=0.0086
Step4—Determine the existing state of strain at the different critical sections. The only external load acting on the beam at the time of the FRPapplication is the self-weight of the slab Because the self-weight of the slab is balanced by the effect of the prestressing force, the initial strain in the sofit at all sections is calculated as: $$\varepsilon_{bi} = \frac{-P_e}{E_c A_{cg}}$$	$\varepsilon_{bi} = \frac{-(26.5\text{ kip/ft})(1000)}{(4{,}030{,}000\text{ psi})(120\text{ in.}^2/\text{ft})} = -5.5\times10^{-5}$	$\varepsilon_{bi} = \frac{-(388\text{ kN/m})(1000)}{(27{,}600\text{ MPa})(250{,}000\text{ mm}^2/\text{m})} = -5.5\times10^{-5}$

Table 16.14d,cont.—Procedure for flexural strengthening of an interior unbonded prestressed concrete slab with FRP

Step 5—Consider the most critical section at the interior support(Section 2) Estimate the depth of the neutral axis at this section.A reasonable initial estimate of c is 0.15h.The value of cis adjusted after checking equilibrium. c=0.15h	c=(0.15)(10.0 in.)=1.5 in.	c=(0.15)(250 mm)=38 mm
Step 6—Determine the effective level of strain in the FRP reinforcement The effective strain in the FRP reinforcement may be found from Eq. (10.3) $\varepsilon_{fe}=\varepsilon_{cu}\left(\frac{d_f-c}{c}\right)-\varepsilon_{bi}\leq\varepsilon_{fd}$ Failure is controlled by FRP debonding,and therefore the concrete strain at debonding failure is calculated from; $\varepsilon_c=(\varepsilon_{fd}+\varepsilon_{bi})\left(\frac{c}{d_f-c}\right)$	$\varepsilon_{fe}=0.003\left(\frac{10\text{ in.}-1.5\text{ in.}}{1.5\text{ in.}}\right)-(-5.5\times10^{-5})\leq0.079$ =0.017>0.0079 Bo=0.0079 $\varepsilon_c=(0.0079-5.5\times10^{-5})\left(\frac{1.5\text{ in.}}{10\text{ in.}-1.5\text{ in.}}\right)=0.0014$	$\varepsilon_{fe}=0.003\left(\frac{250\text{ mm}-38\text{ mm}}{38\text{ mm}}\right)-(-5.5\times10^{-5})\leq0.079$ c=0.017>0.0079 Be=0.0079 $\varepsilon_c=(0.0079-5.5\times10^{-5})\left(\frac{38\text{ mm}}{250\text{ mm}-38\text{ mm}}\right)=0.0014$
Step 7—Caleulate the strain in the existing reinforcing steel $\varepsilon_s=\varepsilon_c\left(\frac{d-c}{c}\right)$	$\varepsilon_s=0.0014\left(\frac{8.75\text{ in.}-1.5\text{ in.}}{1.5\text{ in.}}\right)=0.0068$	$\varepsilon_s=0.0014\left(\frac{222\text{ mm}-38\text{ mm}}{38\text{ mm}}\right)=0.0068$
Step 8—Calculate the strain in the unbonded prestressing steel from Eq. (10.3.2.2b) $\varepsilon_{ps}=\varepsilon_{pe}+\eta\varepsilon_c\left(\frac{d_p-c}{L_a}\right)$ For the section under consideration(negative moment at the interior support), η=43.5	$\varepsilon_{ps}=0.0051+(43.5)(0.0014)\left(\frac{8.75\text{ in.}-1.5\text{ in.}}{1043\text{ in.}}\right)$ =0.0055	$\varepsilon_{ps}=0.0051+(43.5)(0.0014)\left(\frac{222\text{ mm}-38\text{ mm}}{26,490\text{ mm}}\right)$ =0.0055
Step 9—Calculate the stress level in the FRP, prestressing steel and reinforcing steel from Eq. (10.2.6),(10.2.10b),and(10.3.2.2c) f=Eee =E,B,≤f f=Eem≤0.95fm	Je=(13,895ksi)(0.0079)=109.8 ksi f=(29,000 ksi)(0.0068)=197.2 ksi≤60ksi f.=60ksi Jm=(28,500 ksi)(0.0055)=156.8ksi≤(0.95) (230 ksi) =218.5ksi	fo=(95,800 MPa)(0.0079)=757 MPa f=(200,000 MPa)(0.0068)=1360 MPa≤ 414 MPa f_5=414 MPa f=(196,500 MPa)(0.0055)=1081 MPa≤ (0.95)(1585 MPa)=1506 MPa

Table 16.14d,cont.—Procedure for flexural strengthening of an interior unbonded prestressed concrete slab with FRP

Step 10—Calculate the internal force resultants and check equilibrium Concrete stress block factors may be calculated using ACI 318.Approximate stress factors may also be calculated based on the parabolic stress-strain relationship for concrete as follows: $\beta_1 = \frac{4\varepsilon_c' - \varepsilon_c}{6\varepsilon_c' - 2\varepsilon_c}$ $\alpha_1 = \frac{3\varepsilon_c'\varepsilon_c - (\varepsilon_c)^2}{3\beta_1(\varepsilon_c')^2}$ where e,'=1.76/E。 Force equilibrium is verified by checking the initial estimate of c using Eq. (10.3.1.6f): $c = \frac{A_{ps}f_{ps} + A_s f_s + A_f f_{fe}}{\alpha_1 f_c' \beta_1 b}$	$\beta_1 = \frac{4(0.0021) - 0.0014}{6(0.0021) - 2(0.0014)} = 0.714$ $\alpha_1 = \frac{3(0.0021)(0.0014) - (0.0014)^2}{3(0.714)(0.0021)^2} = 0.726$ $\varepsilon_c' = \frac{1.7(5000)}{4030 \times 10^3} = 0.0021$ c=[(0.184 in.²/B)(156.8ksi)+(0.236 in.²/t)(60 ksi)+(0.284 in.²/f)(109.8 ksi)]/[0.726(5 ksi)(0.714)(12 in./f)]=2.39 in.≠1.5 in**NG** Revise estimate ofc and repeat Steps 5 through 10 until force cquilibrium is achieved	$\beta_1 = \frac{4(0.0021) - 0.0014}{6(0.0021) - 2(0.0014)} = 0.714$ $\alpha_1 = \frac{3(0.0021)(0.0014) - (0.0014)^2}{3(0.714)(0.0021)^2} = 0.726$ $\varepsilon_c' = \frac{1.7(34.5)}{27,600} = 0.0021$ c=[(390mm²/m)(1081 MPa)+(500 mm²/m)(414 MPa)+(600 mm²/m)(757 MPa)/[0.726(34.5 MPa)(0.714)(1000)]=61mm≠38 mm **NG** Revise estimate ofc and repeat Steps 5 through 10 until force cquilibrium is achieved
Step 11—Adjust c until force equilibrium is achieved Steps 5 through 10 were repeated with different values of cuntil force equilibrium is achieved.The results of the final iteration are: c=1.96 in.;E=0.0019;em=0.0056;E,=0.0067;f =160.6ksi;β₁=0.74;a₁=0.86;andf₀=109.8ksi	c=[(0.184 in.?/f)(160.6 ksi)+(0.236 in.²/ft)(60 ksi)+(0.284 in.²/f)(109.8 ksi)/[0.86(5 ksi)(0.74)(12 in./t)]=1.96 in.=1.96 in**OK**	c=[(390 mm²/m)(1107 MPa)+(500mm²/m)(414MPa)+(600 mm²/m)(757 MPa)][0.86(34.5 MPa)(0.74)(1000)]=50mm=50 mm **OK**
Step 12—Estimate the moment capacity by considering the contribution of prestressing steel,reinforcing steel,and FRP to the fexural strength(Eq.10.3.1.6g) $M_n = A_{ps}f_{ps}\left(d_p - \frac{\beta_1 c}{2}\right) +$ $A_s f_s\left(d - \frac{\beta_1 c}{2}\right) +$ $\psi_f A_f f_{fe}\left(d_f - \frac{\beta_1 c}{2}\right)$	$M_n = (0.184\text{in.}^2)(160.6\text{ ksi})\left(8.75\text{ in.} - \frac{0.74 \times 1.96\text{ in.}}{2}\right) +$ $(0.236\text{ in.}^2)(60\text{ ksi})\left(8.75\text{ in.} - \frac{0.74 \times 1.96\text{ in.}}{2}\right) +$ $0.85(0.284\text{ in.}^2)(109.8\text{ ksi})\left(10\text{ in.} - \frac{0.74 \times 1.96\text{ in.}}{2}\right)$ =597.0kip-in/t=49.8 kip-ft/t	$M_n = (390\text{ mm}^2)(1107\text{ MPa})\left(222\text{ mm} - \frac{0.74 \times 50\text{ mm}}{2}\right) +$ $(500\text{ mm}^2)(414\text{ MPa})\left(222\text{ mm} - \frac{0.74 \times 50\text{ mm}}{2}\right) +$ $0.85(600\text{ mm}^2)(757\text{ MPa})\left(250\text{ mm} - \frac{0.74 \times 50\text{ mm}}{2}\right)$ =219,357,000 N-mm/m=219.0 kN-m/m
Step 13—Determine the strength reduction factorΦ and,hence,calculate the design flexural strength of the section ΦM The strain in concrete s at the level of prestressing steel is expressed as: $\varepsilon_{ct} = \varepsilon_c\left(\frac{d_p - c}{c}\right)$	$\varepsilon_{ct} = 0.0019\left(\frac{8.75\text{ in.} - 1.96\text{ in.}}{1.96\text{ in.}}\right) = 0.0066$ ≥0.005 Hence, Φ=0.9 ΦMn=0.9(49.8)=44.8 kip-f/ft≥M=43.6 kip-ft/t The strengthened section(Section 2)is capable of sustaining the new required moment strength.	$\varepsilon_{ct} = 0.0019\left(\frac{222\text{ mm} - 50\text{ mm}}{50\text{ mm}}\right) = 0.0066$ =≥0.005 Hence, Φ=0.9 ΦMa=0.9(219.0)=197.0kN-m/m≥M=194.0 kN-m/m The strengthened section (Section 2)is capable of sustaining the new required moment strength.

Table 16.14d,cont.—Procedure for flexural strengthening of an interior unbonded prestressed concrete slab with FRP

Step 14—Carry out similar calculations(Steps 5 through 13)for Section 1,noting that:A_n=0.142 in?/f (300 mm²/m)and N,=19.0 for this section. Following is a summary of the results: c=1.68 in.;=0.0016;eμ=0.0053;e_3=0.0067;fm =150.3 ksi;β_1=0.723;a_1=0.786;f=109.8 ksi;Ea= 0.006;ϕ=0.9	$M_v=(0.184 \text{in.}^2)(150.3 \text{ ksi})\left(8.0 \text{ in.}-\frac{0.723\times 1.68 \text{ in.}}{2}\right)+$ $(0.236 \text{ in.}^2)(60 \text{ ksi})\left(8.75 \text{ in.}-\frac{0.723\times 1.68 \text{ in.}}{2}\right)+$ $0.85(0.142 \text{ in.}^2)(109.8 \text{ ksi})\left(10 \text{ in.}-\frac{0.723\times 1.68 \text{ in.}}{2}\right)$ =444.2 kip-in/ft=37.0 kip-f/f ΦM,=0.9(37.0)=33.3 kip-f/t≥M= 30.6 kip-f/ft The strengthened section(Section 1)is capable of sustaining the new required moment strength.	$M_n=(390 \text{ mm}^2)(1041 \text{ MPa})\left(203 \text{ mm}-\frac{0.723\times 43 \text{ mm}}{2}\right)+$ $(500 \text{ mm}^2)(414 \text{ MPa})\left(222 \text{ mm}-\frac{0.723\times 43 \text{ mm}}{2}\right)+$ $0.85(300 \text{ mm}^2)(757 \text{ MPa})\left(250 \text{ mm}-\frac{0.723\times 43 \text{ mm}}{2}\right)$ =164,099,000 N-m/m=164.0kN-m/m ΦM_n=0.9(164.0)=148.0kN-m/m≥M= 137.0kN-m/m The strengthened section(Section 1)is capable of sustaining the new required moment strength.
Step 15—Carry out similar calculations (Steps 5 through 13)for Section 3,noting that:A_3=0.142 in.2/f(300 mm²/m)and N,=24.5 for this section. Following is a summary of the results: c=1.69 in.;g=0.0016;em=0.0054;6,=0.0067;f =152.6ksi;β_1=0.724;a_1=0.788;f=109.8 ksi;Ea= 0.0067;ϕ=0.9	$M_n=(0.184 \text{in.}^2)(152.6 \text{ ksi})\left(8.75 \text{ in.}-\frac{0.724\times 1.69 \text{ in.}}{2}\right)+$ $(0.236 \text{ in.}^2)(60 \text{ ksi})\left(8.75 \text{ in.}-\frac{0.724\times 1.69 \text{ in.}}{2}\right)+$ $0.85(0.142 \text{ in.}^2)(109.8 \text{ ksi})\left(10 \text{ in.}-\frac{0.724\times 1.69 \text{ in.}}{2}\right)$ =468.0 kip-in/t=39.0kip-ft/ ΦM=0.9(39.0)=35.1 kip-f/A≥M= 33.1 kip-f/ft The strengthened section(Section 3)is capable of sustaining the new required moment strength.	$M_n=(390 \text{ mm}^2)(1053 \text{ MPa})\left(222 \text{ mm}-\frac{0.724\times 43 \text{ mm}}{2}\right)+$ $(500 \text{ mm}^2)(414 \text{ MPa})\left(222 \text{ mm}-\frac{0.724\times 43 \text{ mm}}{2}\right)+$ $0.85(300 \text{ mm}^2)(757 \text{ MPa})\left(250 \text{ mm}-\frac{0.724\times 43 \text{ mm}}{2}\right)$ =172,760,000 Nmmm=173.0kN-mm ΦM=0.9(173.0)=156.0kN-m/m≥M= 148.0kN-m/m The strengthened section(Section 3)is capable of sustaining the new required moment strength.

CHAPTER 17—REFERENCES

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