Appendix II

Post-Tensioned Shear Calculations (**sample calculations at 9')**

Steps taken from the PCI Design Handbook Precast and Prestressed Concrete 6th Edition

Material Properties

f_c' = 5000 psi, normal – weight concrete ⇒ $l = 1$
f_{ci}' = 3500 psi
f_{pu} = 270 ksi, (low-relaxation steel)
f_{ps} = 240 ksi
f_{pe} = 148 ksi ⇒ $f_{pe} > 0.4\ f_{pu}$
f_{yv} for stirrups = 60 ksi

Sectional Properties

b=12"
A_c = 60 in^2
I_c = 125 in^2
h = 5 in
y_b = 2.5
y_t = 2.5 in
Z_b = 50 in^3
Z_t = 50 in^3
$2b_w$ = 24 in

Use the same value for the effective depth d_p for the midspan as well as other sections.

Tendon Properties

e_e = 0.5 in
e_c = 1.2 in
A_{ps} = 18, ½ in (12.7 mm) dia strands = 18 x 0.153 in^2 = 2.754 in^2
F = f_{pe} . A_{ps} = 148 ksi x 2.754 in^2 = 407.6 kips

$k_b = Z_t/A_c$ = .833 in

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Assuming • = 0.8
h = $f_{pe}/f_{pi} = F/F_i = 0.8$

Factored Loads

Factored dead load, superimposed dead load and live load:

w_u = 1.2 (w_D + w_{SDL}) +1.6(w_L) = 1.2(100) + 1.6(140) = 2615 plf = 2.615 kip/ft

Factored superimposed dead load and live load:

• w_u = 1.2 (w_{SDL}) +1.6(w_L) = 1.2 x (80) + 1.6 x (100) = 1392 plf = 1.392 kip/ft

ACI EQUATIONS

$e_{o@9ft}$ = 0.9"
$d_{p@9ft}$ = $e_{o@9ft}$ + y_t = 3.4
For equation used in elaborate approach, d_p is limited by 0.8h = 0.8 x 5 = 4 in
Taking $d_{p@mid}$ as mentioned in the question to be $d_{p@9ft}$, we have
$d_{p@9ft}$ = 4 in

<u>Computation of the Flexure – Shear Resistance:</u>

$$\begin{cases} \text{Flexure - shear stress resistance} \\ v_{ci} = 0.6\lambda\sqrt{f_c'} + \dfrac{V_G}{b_w d_p} + \left(\dfrac{\Delta V_u \times \Delta M_{cr}}{\Delta M_u}\right)\dfrac{1}{b_w d_p} \geq 1.7\lambda\sqrt{f_c'} \\ \text{Flexure - shear force resistance} \\ V_{ci} = 0.6\lambda\sqrt{f_c'}b_w d_p + V_G + \left(\dfrac{\Delta V_u \times \Delta M_{cr}}{\Delta M_u}\right) \geq 1.7\lambda\sqrt{f_c'}b_w d \end{cases}$$

where V_G = shear force due to self – weight of member at section considered

$$= w_G\left(\frac{l}{2} - x\right) = 1.019 \text{ kips/ft } (12\text{-}9)\text{ft} = 26.5 \text{ kip}$$

• V_u = factored shear force due to superimposed dead load plus live load at section considered under same loading as • M_u

$$= \Delta w_u\left(\frac{l}{2} - x\right) = 1.392 \text{ kips/ft } (12\text{-}9) = 36.2 \text{ kip}$$

• M_u = factored bending moment due to superimposed dead load plus live load at section considered = $\Delta w_u \dfrac{x(l-x)}{2}$ = 1.392 x 9 (24-9)/2 = 382.104 kip-ft = 4582 kip-in

M_G = moment due to self weight of member $= w_G \dfrac{x(l-x)}{2}$ = 1.019 x 9 (24-9)/2
= 279.72 kip-ft = 3357 kip-in

• M_{cr} = moment in excess of self – weight moment, causing flexural cracking in the precompressed tensile fiber at section considered = M_{cr} - M_G

$$= Z_b\left[6\sqrt{f_c^{'}} + \frac{F}{A_c}\left(1 + \frac{e_o A_c}{Z_b}\right)\right] - M_G$$

= 5179 kip-in

Therefore:

$$\begin{cases} \begin{cases} v_{ci} = 0.6\times 1\times\sqrt{5000}\,psi + \dfrac{26.5kip\times 1000.}{12.5in\times 30in} + \left(\dfrac{36.2kip\times 5179kip-in}{4582kip-in}\right)\dfrac{1000}{12.5in\times 30in} \\ = 42.426\,psi + 179.778\,psi = 222\,psi \geq 1.7\,l\sqrt{f_c^{'}} = 120\,psi \end{cases} \\ \begin{cases} V_{ci} = 0.6\times 1\times\sqrt{5000}\,psi\times 12.5in\times 30in + \left[25.6 + \left(\dfrac{36.2kip\times 5179kip-in}{4582kip-in}\right)\right]1000 \\ = 15910lb + 66517lb = 82427lb = 82.4kip \geq 1.7\,l\sqrt{f_c^{'}}b_w d_p = 45kip \end{cases} \end{cases}$$

V_p = vertical component of prestressing force at section considered

=Fsin• = 10083 lb

Therefore:

$$\begin{cases} v_{cw} = 3.5\times 1\times\sqrt{5000}\,psi + 0.3\times 417\,psi + \dfrac{10083lb}{12.5in\times 30in} = 400\,psi \\ V_{cw} = (3.5\times 1\times\sqrt{5000}\,psi + 0.3\times 417\,psi)\times 12.5in\times 30in + 10083lb = 150kip \end{cases}$$

The shear resistance is the smaller of v_{ci} (V_{ci}) and v_{cw} (V_{cw}) at 9ft.

Therefore nominal shear strength provided by concrete, v_c = 222 psi (or V_c = 82.4 kip)

Computation of Design Shear Strength

V_u = Design shear force resulting from factored loads

$$= w_u\left(\frac{l}{2} - x\right) = 2.615\frac{kip}{ft}(35-9)\,ft = 68kip = 68000lb$$

Therefore:

$$\begin{cases} \dfrac{V_u}{\phi} = \dfrac{68000lb}{0.75} = 90667lb \\ \dfrac{v_u}{\phi} = \dfrac{V_u}{\phi b_w d_p} = \dfrac{68000lb}{0.75\times 12.5in\times 30in} = 242\,psi \end{cases}$$

The value of V_u/ϕ (or v_u/ϕ) is to be compared to $V_c/2$ (or $v_c/2$) and V_c. (or v_c) As V_u/ϕ = 90667 lb (or v_u/ϕ = 242 psi) is more than $V_c/2$ (or $v_c/2$) as well as V_c (or v_c) the nominal shear strength to be provided by the shear reinforcement,

$$\begin{cases} V_s = \dfrac{V_u}{\phi} - V_c = 90667lb - 82427lb = 8240lb < 8\lambda\sqrt{f_c^{'}}b_w d_p = 212132lb \\ v_s = \dfrac{v_u}{\phi} - v_c = 242\,psi - 222\,psi = 20\,psi < 8\lambda\sqrt{f_c^{'}} = 566\,psi \end{cases}.$$

Therefore there is no need to change concrete cross-section (i.e., larger $b_w d_p$)

$$A_v = \frac{(V_u/\phi - V_c)s}{f_{vy}d} \Rightarrow \frac{(V_u/\phi - V_c)}{d} = \frac{A_v f_{vy}}{s} = 275lb/in$$

$$If \quad s = 12in, \quad \begin{cases} > 3in \\ \le 0.75h = 0.75\times 30 = 22.5in \quad HenceOK \\ \le 24in \end{cases}$$

$$A_v = \frac{275lb/in\times 12in}{60000\,psi} = 0.055in^2$$

<u>Hence the amount of excess shear can be provided by using welded wire reinforcement W2.9 (A_v = 0.058 in^2/ft), at a spacing of 12 in.</u>