ERRATA for
Structural Engineering Practice Exam
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Errata posted 11-7-2016

Revisions are shown in red type.

Vertical Forces PM Bridges

Solution 703, pp. 119–124

Solution 703 should read as follows on page 119:

703. (a) Verify the plate size for the flange splice.

Per AASHTO Section 6.13.2.1, the connection must be checked for slip resistance and the shear and bearing resistance checked separately.

Calculate effective flange area—tension flange:

$$A_e = \left(\frac{\phi_u F_u}{\phi_y F_{yt}} \right) A_n$$ Eq. 6.13.6.1.4c-2

Deducted flange width for bolt hole — Art. 6.8.3
Hole diameter = 15/16 in. — Table 6.13.2.4.2-1

$W_n = 16.5 - (4)(5/16) + (2)(3.5)^2/[(4)(3.5)]$
$= 14.5$ in.
$A_n = (14.5)(1.26)$
$= 18.27$ in^2
$A_g = b_f \times t_f$
$= (16.5)(1.26)$
$= 20.79$ in^2
$\phi_u = 0.80$ — Art. 6.5.4.2
$\phi_y = 0.95$
$F_u = 58$ — Table 6.4.1-1
$F_{yt} = 36$

$$A_e = \frac{(0.8)(58)}{(0.95)(36)} \left(18.27 \text{ in}^2 \right)$$

$$= (1.357) \left(18.27 \text{ in}^2 \right) = 24.79 \text{ in}^2 > A_g = 20.79 \text{ in}^2 \text{ Governs.}$$

The gross section properties can be used.

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Determine the controlling flange:

Check each flange for stress at the midpoint of the flange due to tension forces from applied loads.

At the bottom flange (Dead Load + Maximum Positive Live Load):

Tables 3.4.1-1 & 2

M = 0.9 DC + 1.5 DW + 1.75 (LL + I)
= 0.9(–69) + 1.5(26) + 1.75(730)
= 1,254.4 ft-kips

S_X = I_X/C At **center** of beam flange
= 15,600/[(36.5 – 1.26)/2]
= 885 in^3

Solution 703 should read as follows on page 122:

Check splice plate size:

Outside plate:

$W_n = 16.0 - (4)(15/16) + (2)(3.5)^2/[(4)(3.5)]$
$= 14.0$ in.

$A_n = 14.0 \times 5/8" = 8.75\ in^2 > 0.85\ A_g$ Governs.

$0.85\ A_g = 0.85(10) = 8.5\ in^2$ Art. 6.13.5.2

$\therefore A_n = 8.5\ in^2$

Inside plate:

$W_n = 2\left\{6.5 - (2)(15/16) + (3.5)^2/[(4)(3.5)]\right\}$
$= 11.0$ in.

$A_n = 11.0 \times 5/8" = 6.88\ in^2 < 0.85\ A_g$

$0.85\ A_g = 0.85(8.13) = 6.91$ OK

$\therefore A_n = 6.88\ in^2$

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For tension:
For yielding: Eq. 6.8.2.1-1

Outside $P_r = \phi_y F_y A_g = (0.95)(36\text{ ksi})(10\text{ in}^2) = 342\text{ kips} > 309.6\text{ kips}$ OK

Inside $P_r = \phi_y F_y A_g = (0.95)(36\text{ ksi})(8.13\text{ in}^2) = 278.0\text{ kips} > 251.7\text{ kips}$ OK

For fracture: Eq. 6.8.2.1-2

Outside $P_r = \phi_u F_u A_n U = (0.8)(58\text{ ksi})(8.5\text{ in}^2)(1.0) = 394.4\text{ kips} > 309.6\text{ kips}$ OK

Inside $P_r = \phi_u F_u A_n U = (0.8)(58\text{ ksi})(6.88\text{ in}^2)(1.0) = 319.2\text{ kips} > 251.7\text{ kips}$ OK

For compression:
$R_r = \phi_c F_y A_s$ where $A_s = A_g$ Eq. 6.13.6.1.4c-4

Outside $R_r = (0.9)(36\text{ ksi})(10) = 324\text{ kips} > 309.6\text{ kips}$ OK

Inside $R_r = (0.9)(36\text{ ksi})(8.13) = 263.4\text{ kips} > 251.7\text{ kips}$ OK

Solution 703 should read as follows on p. 123.

(b) Verify the number of bolts in the flange splice, and revise the number if required.

Check flange bolts:

Maximum distance between end fasteners = 3.5 in. × 7 spaces = 24.5 in. < 50 in. so 20% decrease in bolt strength per Art. 6.13.2.7 is not required.

By specification, the bolt threads are included in the shear plane; hence the bolt strength from Eq. 6.13.2.7-2 will be used directly.

Bolt shear strength:

$R_n = 0.38\, A_b F_{ub}\, N_s$ Eq. 6.13.2.7-2

$F_{ub} = 120\text{ ksi}$ Art. 6.4.3.1

$R_n = 0.38(0.6)(120)(1) = 27.4\text{ kips}$

$\phi R_n = 0.8(27.4)$ where $\phi_s = 0.80$

$= 21.9\text{ kips per bolt}$ Bolt strength controls over bearing strength.

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Bolt bearing strength:
F_u = 58 ksi M-270, Gr 36 steel
L_c = 2 – (1-in.-dia./2) Clear distance from edge of hole to edge of connected plate
= 1.5 in. < 2d = 1.75 in.
d = 7/8 in. Nominal bolt diameter
t = 5/8 in. or 1.26 in. Thickness of connected material
$R_n = 1.2L_ctF_u$ Eq. 6.13.2.9-2

Bearing on splice plates:
ϕR_n = (0.8)(1.2)(1.5)(5/8)(58) where $\phi_{bb} = 0.8$
= 52.2 kips per bolt Does not control bolt strength

Bearing on beam flange:
ϕR_n = (0.8)(1.2)(1.5)(1.26)(58)
= 105.2 kips per bolt Does not control bolt strength

Required number of bolts:
At outside plate = (21.9 kips per bolt)(16 bolts) Single shear per plate
= 350.4 kips > 309.6 kips OK

At inside plates = (21.9 kips per bolt)(16 bolts) Single shear per plate
= 350.4 kips > 251.7 kips OK

Previously posted errata continued on next page.