7. AIDS FOR STRENGTH DESIGN

7.1 General

To assist the design of penetrations in accordance with the method given in Section 6, a series of design tables and a spreadsheet program named WEBPEN™ are provided.

Details about WEBPEN™ are given in Section 7.2.

The major calculation effort in the strength design method is spent determining $\phi\overline{M}_b$ and $\phi\overline{V}_u$ at the penetration. The design tables allow these values to be readily determined for a wide range of situations covering both bare steel and composite beams. These design tables are suitable for use during the preliminary design stage as well as for final design. A brief description of the tables is given in Section 7.3 and the tables are given in Appendix C.

7.2 WEBPEN™ Spreadsheet Program

The WEBPEN™ spreadsheet program runs on Microsoft® EXCEL™ Version 7. The design tables given in Appendix C were generated using WEBPEN™.

The input data required for the spreadsheet includes:

(a) Steel section data - chosen from an in-built section library that includes all relevant properties for UB and WB sections.

(b) Concrete flange data: b_{cf}, D_c, f'_c and ρ_c.

(c) Profiled steel sheeting data: rib height; and angle of sheeting ribs to steel beam longitudinal axis.

(d) Geometry of the penetration: rectangular/circular, height/diameter, length and eccentricity.

(e) Shear connector data: nominal shear capacity; number of shear connectors from the *HME* to the nearer end of beam; and number of shear connectors within the length of the penetration.

(f) Penetration reinforcement data: plate width; thickness; and nominal yield stress.

(g) Design action effect data: M^* and V^* values at the mid-length of the penetration.

For a given configuration with M^* and V^* values at the penetration having been determined, the spreadsheet will calculate the design capacities $\phi\overline{M}_b$ and $\phi\overline{V}_u$ and plot the design point in relation to the moment-shear interaction curve, allowing the designer to determine whether or not the trial geometry and location of the penetration is satisfactory. The designer can easily trial several combinations of penetration geometries and locations until a satisfactory solution is obtained. A typical moment-shear interaction curve output from WEBPEN™ is shown in Fig. 7.1.

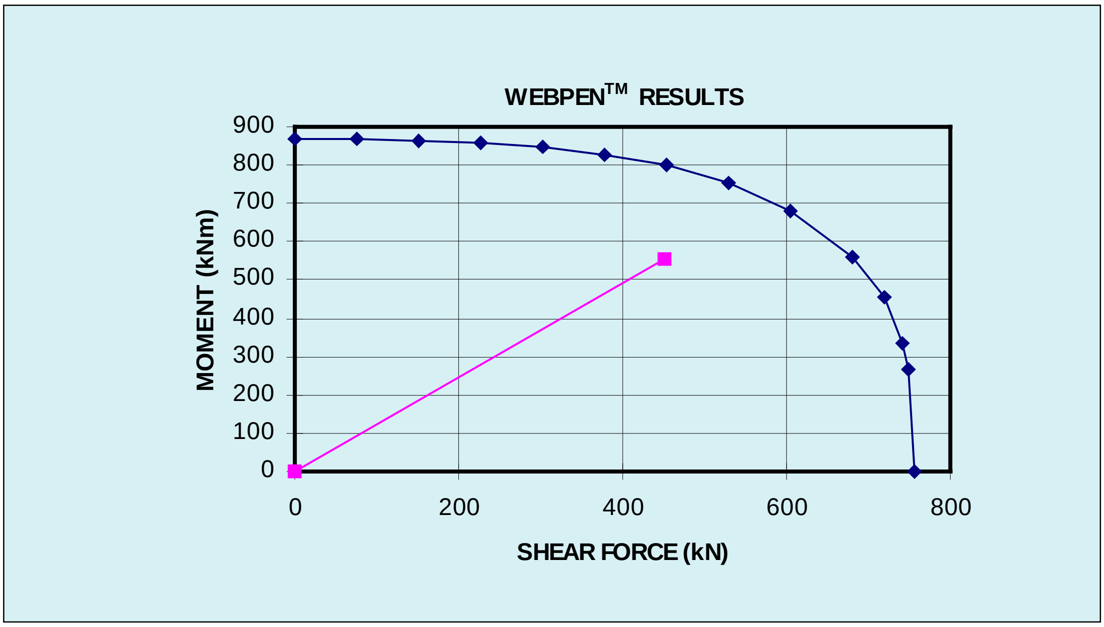

Figure 7.1 Typical Output of WEBPEN™

7.3 Design Capacity Tables

The design capacity tables given in Appendix C can be used to calculate $\phi\overline{M}_b$ and $\phi\overline{V}_u$ for a wide range of bare steel and composite beams with concentric web penetrations of various proportions. The parametric range covered in the tables is given in Table 7.1. A detailed description of the parameters given in the tables and how the tables may be used to calculate $\phi\overline{M}_b$ and $\phi\overline{V}_u$ is given in Paragraphs C2 and C3, respectively.

Table 7.1 Parametric Range of Design Capacity Tables

Parameter	Range
Flange width (b_{cf})	1200, 1600 and 2100 mm
Concrete strength (f'_c)	Grades 25 and 32
Slab thickness (D_c)	120 mm
Direction of sheeting ribs	Perpendicular ($\lambda = 10$) and parallel ($\lambda = 0$) to steel beam
Steel beam (300PLUS®)	700WB115 to 800WB192 310UB32 to 610UB125
Penetration size	Circular: $h_0 / D_s =$ 0.3, 0.5 & 0.7 Rectangular: $h_0 / D_s =$ 0.3, 0.5 & 0.7; and $L_0 / h_0 =$ 1.0, 1.5 and 2.0

8. WORKED EXAMPLES

8.1 General

Two worked examples are presented to demonstrate the strength and deflection design methods given in Section 6. They involve the design of a web penetration in an unpropped simply-supported composite beam for Construction Stage 3 (i.e. bare steel beam), and the in-service condition, as defined in AS 2327.1:

Example 1

Strength design of a bare steel beam with a rectangular web penetration for Construction Stage 3, using hand calculations and the design capacity tables given in Appendix C.

Example 2

Strength and deflection design of a composite beam with a rectangular web penetration for the in-service condition, using hand calculations and the design capacity tables given in Appendix C.

It is assumed that the composite beam without the penetration has been designed in accordance with AS 2327.1.

8.2 Beam and Penetration Data

The beam data and the preferred size and location of the penetration are given in Table 8.1:

Table 8.1 Worked Example Geometry and Properties

Span (L)	=	10.5 m
Spacing of secondary beams	=	2.6 m
Steel beam	=	410UB53.7, 300PLUS® ($D_s = 403$ mm, $b_f = 178$ mm, $t_f = 10.9$ mm, $t_w = 7.6$ mm, $d_1 = 381$ mm, $f_{yw} = 320$ MPa, $f_{yf} = 320$ MPa)
Slab depth (D_c)	=	120 mm (composite slab, $h_r = 55$ mm)
Orientation of sheeting	=	Sheeting ribs perpendicular to the steel beam
Concrete strength (f'_c)	=	25 MPa
Density of concrete	=	25 kN/m^3 (including allowance for reinforcement)
Superimposed dead load	=	0.3 kPa (services and ceilings)
Reducible Live load	=	4.0 kPa
Shear connectors	=	19 mm diameter
Penetration	=	Penetration centreline located at 3300 mm from the support $h_0 = 225\,\text{mm}\,(= 0.56D_s)$, $L_0 = 425\,\text{mm}\,(= 1.9h_0)$ Concentric with no reinforcement (see Fig. 8.1), therefore $s_t = s_b = (D_s - h_0)/2 = 89$ mm

The assumptions made in the calculations are as follows:

(a) the steel and composite beams are assumed to be simply-supported during construction and the in-service condition; and

(b) the maximum ponding deflection of the profiled steel sheeting equals 10.4 mm.

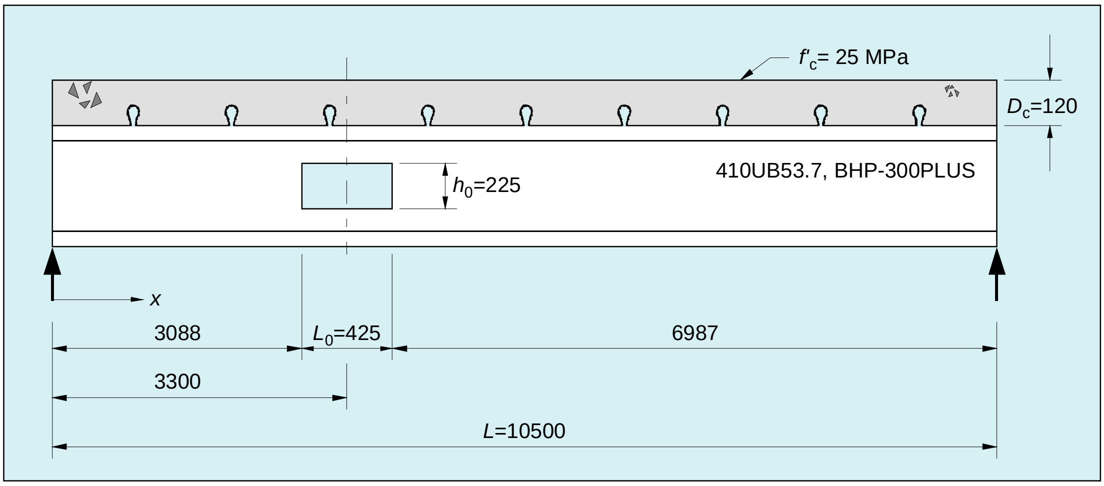

Figure 8.1 Beam Geometry

8.3 Example 1

This worked example demonstrates the strength design procedure for a web penetration in a bare steel beam for Construction Stage 3, using hand calculations and design capacity tables given in Appendix C.

Calculation of Design Action Effects at Mid-Length of Web Penetration

The minimum nominal loads for construction given in Paragraph F2 of AS 2327.1 are used.

The effective span and tributary area are calculated in accordance with the requirements of AS 2327.1.

Span (L) = 10.5 m
Tributary area = 27.3 m^2

Steel beam self-weight = 0.5 kN/m
Weight of slab (including ponding) = (0.12 m + 0.7x2.6/250) x 25 kN/m^3 x 2.6 m = 8.3 kN/m
Dead load (G) = 0.5+8.3 = **8.8 kN/m**
Reducible Construction live load = $[10-(27.3-23)/(46-23)\times(10-0.6)]\times 2.6$ = 2.4 kN/m
$\therefore$ Live load (Q) = **2.4 kN/m**
Design load (W) = 1.25 x 8.81 + 1.5 x 2.4 = **14.6 kN/m**

Support reaction = **76.6 kN**

$\therefore$ At mid-length of web penetration:

M^* = **173 kNm**
V^* = **28.4 kN**

Preliminary Check

Ensure the conditions given in Section 6.2 are satisfied:

- $h_0 = 225\,\text{mm}\,(= 0.56 D_s) \leq 0.7 D_s$ Satisfactory
- for bare steel beams: $(L_0/h_0 + 6h_0/D_s) = 5.2 \leq 5.6$ Satisfactory in Construction Stage 3
- $s_t = 89\,\text{mm} \geq 0.15 D_s\,(= 60.5\,\text{mm})$ Satisfactory
- for bare steel beams: $s_b = 89$ mm $\geq 0.15 D_s$ Satisfactory in Construction Stage 3
- no concentrated loads are applied to the beam, thus loading does not restrict the position of the web penetration Satisfactory

- distance to nearest end support = $3088\,\text{mm} > D_s$ Satisfactory
- the ratios L_0/s_t, L_0/s_b = 425/89 = 4.8 < 12 Satisfactory
- the slenderness of the web will also influence the maximum allowable length-to-height ratio of a penetration and the maximum nominal shear capacity of the member (see Section 6.6).

$$\text{For 410UB53.7, } \left|\frac{d_1}{t_w}\right|\sqrt{\frac{f_{yw}}{250}} = \mathbf{56.7} < 70$$

$$\text{Therefore, } \frac{L_0}{h_0} \le 3.0\text{; and}$$

$$\text{for bare steel beams, } \bar{V}_u \le 0.4 f_{yw} t_w D_s$$

Strength Design by Hand Calculation

Design Moment Capacity

Determine the depth of the compressive stress zone, d_h, and nominal moment capacity, $\bar{M}_b$, of the steel cross-section at the *HME* of the web penetration, using the formulae given in Paragraph B2 of Appendix B.

As the penetration is concentric and unreinforced, $F_{tw} = F_{bw}$ (= $A_{tw} f_{yw}$ = 190 kN) and $F_r = 0$. In addition $F_{tf} = F_{bf}$ (= 621 kN) (as the 410UB53.7 section is compact in accordance with AS 2327.1).

Therefore, using Eqs B1 and B2 for Case1;

$$d_h = \frac{t_w f_{yw}(s_t + t_f) + 2F_r + F_{bw} + F_{bf} - F_{tf}}{2 t_w f_{yw}}$$

$$= \frac{7.6 \times 320 \times (89.0 + 10.9) + 2 \times 0 + 190E3 + 621E3 - 621E3}{2 \times 7.6 \times 320}$$

$$= \mathbf{89.0\ mm}$$

and

$$\begin{aligned}\bar{M}_b &= F_{tf}(d_h - t_f/2) + t_w f_{yw}\left|(d_h - t_f)^2 + (s_t - d_h)^2\right|/2 \\ &\quad + F_r(D_s + s_t - s_b - 2d_h) + F_{bw}(D_s - d_h - (s_b + t_f)/2) \\ &\quad + F_{bf}(D_s - d_h - t_f/2) \\ &= 621E3 \times (89 - 10.9/2) + 7.6 \times 320\left[(89 - 10.9)^2 + 0^2\right]/2 \\ &\quad + 0 + 190E3(403 - 89 - (89 + 10.9)/2) \\ &\quad + 621E3(403 - 89 - 10.9/2) \\ &= \mathbf{303\ kNm}\end{aligned}$$

Therefore, the design moment capacity of the cross-section at the penetration is;

$$\phi\bar{M}_b = \mathbf{273\ kNm}$$

Design Shear Capacity

From Eq. 6.4,

$$\phi\bar{V}_u = \phi(V_t + V_b)$$

Nominal shear capacity of the bottom T-section:

Using Eqs 6.27 to 6.33,

$$V_b = \frac{\sqrt{6} + \mu_b}{\nu_b + \sqrt{3}} V_{pb} \le V_{pb}$$

where, $\mu_b = \dfrac{2F_r d_r}{V_{pb} s_b}$ = **0** (since unreinforced)

$$\nu_b = \frac{L_0}{s_b} = \frac{425}{89} = \mathbf{4.78}$$

and $V_{pb} = 0.6 f_{yw} s_b t_w = 0.6 \times 320 \times 89 \times 7.6 = \mathbf{130\ kN}$

Therefore, $V_b = \dfrac{\sqrt{6} + 0}{4.78 + \sqrt{3}} \times 130 \le 130\,\text{kN}$

$= \mathbf{48.9\ kN}$

Nominal shear capacity of the top T-section:

Using the same equations as for the bottom T-section calculations,

$$V_t = \mathbf{48.9\ kN}$$

The design shear capacity is,

$$\phi\overline{V}_u = \phi(V_t + V_b) = 0.9 \times (48.9 + 48.9) = \mathbf{88\ kN}$$

To satisfy the web buckling considerations given in Section 6.6,

$$\overline{V}_u\ (=97.8\text{ kN}) \le 0.4 f_{yw} t_w D_s$$

$$\le 0.4 \times 320 \times 7.6 \times 403 = \mathbf{392\ kN} \quad \text{O.K}$$

To satisfy the considerations for buckling of the top T-section given in Section 6.6,

$$M^* / (V^* D_s) = 15.1 < 20 \qquad \text{criterion satisfied}$$

Moment-Shear Interaction

Applying the moment-shear interaction relationship given in Eq. 6.3,

$$\left(\frac{173}{273}\right)^3 + \left(\frac{28.4}{88.0}\right)^3 = 0.29 \le 1.0 \qquad \text{strength criterion satisfied}$$

Strength Design using Tables

Since $h_0 / D_s = 0.56$ for the penetration, determine $\phi\overline{M}_b$ and $\phi\overline{V}_u$ values by interpolating between the values obtained from Table C26 for $h_0 / D_s = 0.5$ and Table C27 for $h_0 / D_s = 0.7$.

Design Moment Capacity

Values of the design moment capacity of a bare steel beam with a web penetration, $\phi\overline{M}_{b.0}$, are given in the fifth column of the tables. By interpolation, the design moment capacity are calculated as,

$$\phi\overline{M}_b = 269 - \left(\frac{0.56 - 0.5}{0.7 - 0.5}\right) \times (269 - 255) = \mathbf{265\ kNm}$$

Design Shear Capacity

Values of the design shear capacity of a bare steel beam with a web penetration are given in the columns with headings $\phi\overline{V}_{u.0}$ for different web penetration aspect ratios.

Since $L_0 / h_0 = 1.9$ for the penetration, first interpolate between the columns corresponding to $L_0 / h_0 = 1.5$ and $L_0 / h_0 = 2.0$ in each table, and then interpolate between those values to determine $\phi\overline{V}_{u.0}$ corresponding to $h_0 / D_s = 0.56$.

$$\text{Value of } \phi\overline{V}_{u.0} \text{ from Table C26} = 137 - \left(\frac{1.9 - 1.5}{2.0 - 1.5}\right) \times (137 - 113) = 118\text{ kN}$$

$$\text{Value of } \phi\overline{V}_{u.0} \text{ from Table C27} = 45 - \left(\frac{1.9 - 1.5}{2.0 - 1.5}\right) \times (45 - 35) = 37.0\text{ kN}$$

$$\phi \bar{V}_u = 118 + \left| \frac{0.7 - 0.56}{0.7 - 0.5} \right| \times (118 - 37) = \mathbf{93.6\ kN}$$

Moment-Shear Interaction

Applying the moment-shear interaction relationship given in Eq. 6.3,

$$\left(\frac{173}{265}\right)^3 + \left(\frac{28.4}{93.6}\right)^3 = 0.31 \le 10 \qquad \text{strength criterion satisfied}$$

8.4 Example 2

This worked example demonstrates the strength design procedure for a web penetration in a composite beam (described in Section 8.2) for the in-service condition, using hand calculations and design capacity tables given in Appendix C.

It is assumed that the composite beam without the penetration has already been designed in accordance with AS 2327.1. The resulting shear connector distribution given in Fig. 8.2 shows that the number of 19 mm shear connectors from *HME* to the nearer end of the beam is 9 (i.e. $n_H = 9$).

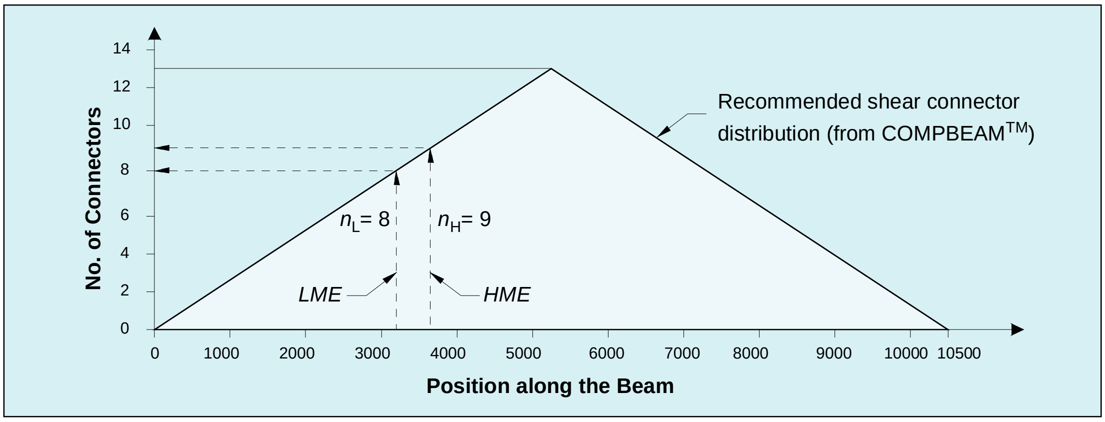

Figure 8.2 Shear Connector Distribution

Design Action Effects at Mid-Length of Web Penetration

The effective span and tributary area are calculated in accordance with AS 2327.1.

$$\text{Span } (L) = \mathbf{10.5\ m}$$
$$\text{Tributary area} = \mathbf{27.3\ m^2}$$

$$\text{Total dead load of beam (G)} = 8.8 + 0.3 \times 2.6 = \mathbf{9.6\ kN/m}$$

From Section 4 of AS 1170.1, the permissible live load reduction equals, $75 - \dfrac{350}{\sqrt{27.3}} = 8\%$

$$\therefore \text{ Reducible live load (Q)} = 4 \times 0.92 \times 2.6 = \mathbf{9.6\ kN/m}$$
$$\text{Design load (W)} = 1.25 \times 9.6 + 1.5 \times 9.6 = \mathbf{26.4\ kN/m}$$

$$\text{Support reaction} = \mathbf{138.6\ kN}$$

Therefore, at mid-length of the web penetration:

$$M^* = 138.6 \times 3.3 - 26.4 \times 3.3^2 / 2 = \mathbf{314\ kNm}$$
$$V^* = 138.6 - 26.4 \times 3.3 = \mathbf{51.4\ kN}$$

Preliminary Check

In addition to the preliminary checks performed in Example 1 for the bare steel beam, the following additional checks must be made for the composite beam.

Effective Width of Concrete Compression Flange

Using Clause 5.2.2 of AS 2327.1,

$$b_{cf} = 2 \times \min.\left(10500/8, 2600/2, 178/2 + 8 \times 120\right)$$
$$= \mathbf{2098\ mm}$$

Web Buckling

Check for web buckling conditions given in Section 6.6:

For 410UB53.7, $|\frac{d_1}{t_w}|\sqrt{\frac{f_{yw}}{250}} = \mathbf{56.7} < 70$

Therefore, $\frac{L_0}{h_0} \leq 3.0$; and

$$\bar{V}_u \leq 0.4 f_{yw} t_w D_s + \min.(V_{pt}(\mu_t / \upsilon_t - 1) \geq 0,\ 0.29\sqrt{f'_c} A_{vc})$$

Compressive Force in Concrete Flange at HME

The compressive force in the concrete at the *HME* of the web penetration, F_{cH}, is calculated using the equations presented in Paragraph A2 of Appendix A.

The compressive capacity of the concrete flange, F_c, is calculated as follows:

$$F_c = F_{c1} + F_{c2}$$
$$F_{c1} = 0.85 f'_c b_{cf} (D_c - h_r)$$
$$= 0.85 \times 25 \times 2098 \times (120 - 55) = \mathbf{2898\ kN}$$
$$F_{c2} = 0.85 f'_c b_{cf} \lambda h_r$$
$$= 0.85 \times 25 \times 2098 \times 0.0 \times 55 = \mathbf{0\ kN}$$

Therefore, $F_c = \mathbf{2898\ kN}$

Since $n_H = 9$,

$$f_{ds} = \phi k_n f_{vs}$$
$$= 0.85 \times \left(118 - |\,0.18/\sqrt{9}\,|\right) \times 89 = \mathbf{84.7\ kN}$$

Therefore, $n_H f_{ds} = 9 \times 84.7 = \mathbf{762\ kN}$

From Eq. A7,

$$\bar{F}_s = \sum_{i=1}^{n_e} A_i f_{yi}$$
$$= 2 \times 178 \times 10.9 \times 320 + 2 \times (89 - 10.9) \times 7.6 \times 320 = \mathbf{1622\ kN}$$

From Eq. A8,

$$F_{cH} = \min.(F_c, n_H f_{ds}, \bar{F}_s)$$
$$= \min.(2898, 762, 1622) = \mathbf{762\ kN}$$

From Eqs A10 and A9,

$$\bar{F}_{cc} = \min.(F_c, \bar{F}_s)$$
$$= \min.(2898, 1622) = \mathbf{1622\ kN}$$
$$\bar{\beta} = n_H f_{ds} / \bar{F}_{cc} \leq 1$$
$$= 762 / 1622 = \mathbf{0.47}$$

Strength Design by Hand Calculation

Design Moment Capacity

Use the formulae given in Para. A3.2.

From previous calculations,

$$F_{cH} = \mathbf{762\ kN}$$
$$F_c = \mathbf{2898\ kN}$$
$$\overline{F}_s = \mathbf{1622\ kN}$$
$$F_{c1} = \mathbf{2898\ kN}$$
$$F_r = \mathbf{0\ kN} \text{ (penetration is unreinforced)}$$

Since $F_{cH} \le F_c$,

$$d_c = F_{cH} \Big/ \left(0.85 b_{cf} f'_c\right) \quad \text{[Eq. A11]}$$
$$= 762E3 \Big/ \left(0.85 \times 2098 \times 25\right) = \mathbf{17.1\ mm}$$

Since top flange is compact,

$$F_{tf} = b_{eff} t_f f_{yf} = 178 \times 10.9 \times 320 = \mathbf{621\ kN} \quad \text{[Eq. A12]}$$

Since $\left(\overline{F}_s - 2F_{tf}\right) < F_{cH} \le \overline{F}_s$, Case 2 in Para. A3.2 is applicable.

From Eq. A20,

$$d_h = \mathbf{128\ mm}$$

From Eq. A21,

$$\overline{M}_b = \mathbf{408\ kNm}$$

Therefore, the design moment capacity of the composite cross-section at the penetration is:

$$\phi \overline{M}_b = \mathbf{367\ kNm}$$

Design Shear Capacity

From Eq. 6.4,

$$\phi \overline{V}_u = \phi (V_t + V_b)$$

The nominal shear capacity of the bottom T-section, V_b, is the same as that calculated for the bare steel beam in Example 1.

Therefore,

$$V_b = \mathbf{48.9\ kN}$$

The nominal shear capacity of the top T-section, V_t, is calculated as:

$$V_t = \frac{\sqrt{6} + \mu_t}{\nu_t + \sqrt{3}} V_{pt} \le V_{pt} + 0.29\sqrt{f'_c} A_{vc} \quad \text{[Eq. 6.11]}$$

where,

$$V_{pt} = 0.6 f_{yw} s_t t_w \quad \text{[Eq. 6.16]}$$
$$= 0.6 \times 320 \times 89 \times 7.6 = \mathbf{130\ kN}$$
$$\nu_t = \frac{L_0}{s_t} = \frac{425}{89} = \mathbf{4.78} \quad \text{[Eq. 6.14]}$$

and

$$\mu_t = \frac{2F_r d_r + F_{ctH} d_{ctH} - F_{ctL} d_{ctL}}{V_{pt} s_t} \quad \text{[Eq. 6.12]}$$
$$F_{ctH} = \min.(F_c, n_H f_{ds}, (F_{tf} + F_{tw} + F_r)) \quad \text{[Eq. 6.18]}$$
$$= \min.(2898, 762, (621 + 190 + 0)) = \mathbf{762\ kN}$$

where,

$$d_{ctH} = D_c - \frac{F_{ctH}}{1.7 f'_c b_{cf}} = 120 - \frac{762 \times 10^3}{1.7 \times 25 \times 2098} \quad \text{[Eq. 6.19]}$$
$$= \mathbf{111.5\ mm}$$

$$F_{ctL} = F_{ctH} - (n_H - n_L) f_{ds} = 762 \times 10^3 - 1 \times 84.7 \times 10^3 \quad \text{[Eq. 6.20]}$$
$$= \mathbf{677.3\ kN}$$

$$d_{ctL} = (1-\lambda)h_r + \frac{F_{ctL}}{1.7 f_c^{'} b_{cf}} = 55 + \frac{677.3 \times 10^3}{1.7 \times 25 \times 2098} \quad \text{[Eq. 6.21]}$$
$$= \mathbf{62.6\ mm}$$

Therefore,
$$\mu_t = \frac{0 + 762 \times 10^3 \times 1115 - 677.3 \times 10^3 \times 62.6}{130 \times 89} = \mathbf{3.68}$$

and
$$V_t = \frac{\sqrt{6} + \mu_t}{\nu_t + \sqrt{3}} V_{pt} = \frac{\sqrt{6} + 3.68}{4.78 + \sqrt{3}} \times 130 = \mathbf{122\ kN}$$

Check,
$$V_t \le V_{pt} + 0.29\sqrt{f_c^{'}} A_{vc} = 130 + 0.29\sqrt{25} \times 23.4$$
$$= 164\ \text{kN} \quad \text{O.K}$$

Therefore, $V_t = \mathbf{122\ kN}$

Substituting the values of V_b and V_t in Eq. 6.4,

$$\phi\overline{V}_u = \phi(V_t + V_b) = 0.9 \times (122 + 48.9) = \mathbf{154\ kN}$$

To satisfy the web buckling conditions given by Eqs 6.37 and 6.38 in Section 6.6,

$$\overline{V}_u\ (=171\ \text{kN}) \le 0.4 f_{yw} t_w D_s + \min.(V_{pt}(\mu / \nu - 1) \ge 0,\ 0.29\sqrt{f_c^{'}} A_{vc})$$
$$\le 0.4 \times 320 \times 7.6 \times 403 + \min.(130(3.68 / 4.78 - 1) \ge 0,\ 0.29\sqrt{25} \times 23.4)$$
$$\le \mathbf{392\ kN} \quad \text{O.K}$$

To satisfy the condition for buckling of the top T-section given in Section 6.6,

$$M^* / (V^* D_s) = 15 < 20 \quad \text{criterion satisfied} \quad \text{O.K}$$

Moment-Shear Interaction

Substituting the calculated values of M^*, V^*, $\phi\overline{M}_b$ and $\phi\overline{V}_u$ in Eq. 6.3,

$$\left|\frac{314}{367}\right|^3 + \left(\frac{514}{154}\right)^3 = \mathbf{0.66} \le 1.0 \quad \text{strength criterion satisfied}$$

Strength Design using Tables

Since $h_0 / D_s = 0.56$ for the penetration, determine $\phi\overline{M}_b$ and $\phi\overline{V}_u$ values by interpolating the values obtained from Tables C26 and C27 that correspond to $h_0 / D_s = 0.5$ and $h_0 / D_s = 0.7$, respectively.

Design Moment Capacity

Interpolating between Tables C26 and C27 to calculate $\overline{\beta}$ $\overline{F}_{cc}$ and n_{max} for the penetration with $h_0 / D_s = 0.56$,

$$\overline{\beta} = 0.79 - \left|\frac{0.56 - 0.5}{0.7 - 0.5}\right| \times (0.79 - 0.70) = \mathbf{0.76}$$

$$\overline{F}_{cc} = 1710 - \left|\frac{0.56 - 0.5}{0.7 - 0.5}\right| \times (1716 - 1527) = \mathbf{1653\ kN}$$

$$n_{max} = 19 - \left|\frac{0.56 - 0.5}{0.7 - 0.5}\right| \times (19 - 17) = \mathbf{18}$$

The number of studs to *HME*, $n_H = 9$ OK since $n_H < n_{max}$ (see Fig. 3.3)

From previous calculations,

$$F_{cH} \quad = \quad 762 \text{ kN and } \bar{\beta} = 0.47$$

Interpolating between Tables C26 and C27 to determine the design moment capacities $\phi\overline{M}_{b.0}$ and $\phi\overline{M}_{b.5}$ corresponding to $h_0 \ / \ D_s = 0.56$,

$$\phi\overline{M}_{b.0} \quad = \quad 269 - \left(\frac{0.56 - 0.5}{0.7 - 0.5} \right) \times (269 - 255) \ = \mathbf{265 \ kNm}$$

$$\phi\overline{M}_{b.5} \quad = \quad 385 - \left(\frac{0.56 - 0.5}{0.7 - 0.5} \right) \times (385 - 341) \ = \mathbf{372 \ kNm}$$

Now, interpolate between the values of $\phi\overline{M}_{b.0}$ and $\phi\overline{M}_{b.5}$ to determine $\phi\overline{M}_{b}$ corresponding to $\bar{\beta} = 0.46$,

$$\phi\overline{M}_{b} \quad = \quad 265 + \left(\frac{0.46 - 0.0}{0.5 - 0.0} \right) \times (372 - 265) \ = \mathbf{363 \ kNm}$$

Design Shear Capacity

First interpolate between the columns corresponding to $L_0 / h_0 = 15$ and $L_0 / h_0 = 2.0$ in Tables C26 and C27 to determine $\phi\overline{V}_{u.0}$ and $\phi\overline{V}_{u.5}$ for $L_0 / h_0 = 19$,

$$\text{Value of } \phi\overline{V}_{u.0} \text{ from Table C26} \quad = \quad 137 - \left(\frac{19 - 15}{2.0 - 15} \right) \times (137 - 113) = 118 \text{ kN}$$

$$\text{Value of } \phi\overline{V}_{u.5} \text{ from Table C26} \quad = \quad 201 - \left(\frac{19 - 15}{2.0 - 15} \right) \times (201 - 182) = 185 \text{ kN}$$

$$\text{Value of } \phi\overline{V}_{u.0} \text{ from Table C27} \quad = \quad 45 - \left(\frac{19 - 15}{2.0 - 15} \right) \times (45 - 35) \ = 37.0 \text{ kN}$$

$$\text{Value of } \phi\overline{V}_{u.5} \text{ from Table C27} \quad = \quad 102 - \left(\frac{19 - 15}{2.0 - 15} \right) \times (102 - 90) = 96 \text{ kN}$$

Secondly, interpolate between these values to determine the moment capacities $\phi\overline{V}_{u.0}$ and $\phi\overline{V}_{u.5}$ corresponding to $h_0 \ / \ D_s = 0.56$,

$$\phi\overline{V}_{u.0} \quad = \quad 118 - \left(\frac{0.56 - 0.5}{0.7 - 0.5} \right) \times (118 - 37) \ = \mathbf{94 \ kN}$$

$$\phi\overline{V}_{u.5} \quad = \quad 185 - \left(\frac{0.56 - 0.5}{0.7 - 0.5} \right) \times (185 - 96) \ = \mathbf{158 \ kN}$$

Finally, interpolate between the values of $\phi\overline{V}_{u.0}$ and $\phi\overline{V}_{u.5}$ to determine the value of $\phi\overline{V}_{u}$ corresponding to $\bar{\beta} = 0.46$,

$$\phi\overline{V}_{u} \quad = \quad 158 - \left(\frac{0.5 - 0.46}{0.5 - 0.0} \right) \times (158 - 94) \ = \mathbf{153 \ kN}$$

Moment-Shear Interaction

Substituting the calculated values of M^*, V^*, $\phi\overline{M}_{b}$ and $\phi\overline{V}_{u}$ in Eq. 6.3,

$$\left(\frac{314}{363} \right)^3 + \left(\frac{514}{153} \right)^3 \quad = \quad \mathbf{0.68} \quad \leq 10 \qquad \text{strength criterion satisfied}$$

Deflection Calculations

In this example, the additional deflection component calculated corresponds to short-term loading during the in-service condition. The design load, W , for short-term loading is given as:

$$W \ = G_{sup} + \psi_s Q \quad = \quad 0.78 + 0.7 \times 9.6 \ = \mathbf{7.5 \ kN/m}$$

The design action effects at the web penetration resulting from the in-service loading are:

$$V^* = \textbf{14.6 kN} \text{ (calculated at the mid-length of the penetration)}$$

$$M_H^* = \textbf{92 kNm}$$

$$M_L^* = \textbf{86 kNm}$$

From Eqs 6.56 and 6.57,

$$M_d^* = M_H^* - M_L^*$$

$$= 92\text{-}86 = \textbf{6 kNm}$$

$$M_{se}^* = \frac{-V_t^* L_0}{2}$$

$$= \frac{-14.6 \times 0.425}{2} = \textbf{-3.1 kNm}$$

The second moment of area of the composite cross-section at the web penetration can be calculated as,

$$I_0 = \mathbf{510 \times 10^6 \ mm^4}$$

Similarly, the second moment of area of the top T-section can be calculated as,

$$I_t = \mathbf{30.90 \times 10^6 \ mm^4}$$

The modular ratio used in these calculations was based on the short-term properties of the concrete in accordance with AS 2327.1.

Additional Deflection due to Bending

From Fig. 8.1,

$$a = 3088 \text{ mm and } b = 6987 \text{ mm}$$

From Eq. 6.58,

$$\theta_L = \left| \frac{M_{se}^* I_0 \left(L_0^2 - 2L_0(3b + 2L_0)\right) - M_d^* I_t L_0 (3b + 2L_0)}{6EI_0 I_t L} \right|$$

$$= \frac{\left| \begin{array}{l} -3.1E6 \times 510E6 \times \left(425^2 - 2 \times 425(3 \times 6987 + 2 \times 425)\right) \\ -6E6 \times 30.9E6 \times 425 \times (3 \times 6987 + 2 \times 425) \end{array} \right|}{6 \times 2E5 \times 510E6 \times 30.9E6 \times 10500}$$

$$= \mathbf{137 \times 10^{-6} \ rad}$$

$$\theta_H = -\left| \frac{\left(M_d^* I_t + 2M_{se}^* I_0\right) L_0}{2EI_0 I_t} \right| - \theta_L$$

$$= -\frac{(6E6 \times 30.9E6 + 2 \times (-3.1E6) \times 510E6) \times 425}{2 \times 2E5 \times 510E6 \times 30.9E6} - 137E^{-}6$$

$$= \mathbf{63.7 \times 10^{-6} \ rad}$$

Therefore, the additional bending deflections at the *HME* and at midspan are:

$$\delta_b \text{ at the } HME = 6987 \times 63.7 \times 10^{-6} = \textbf{0.4 mm}$$

$$\delta_b \text{ at midspan} = 5250 \times 63.7 \times 10^{-6} = \textbf{0.3 mm}$$

Additional Shear Deflection

The additional shear deflection is determined from the rotation at the *HME* resulting from the shear force carried by the top T-section.

From Eq. 6.64,

$$\delta_s' = \frac{kV_t L_0}{G s_t t_w}$$

$$= \frac{12 \times 14.6 \times 10^3 \times 425}{80 \times 10^3 \times (89 \times 7.6)} = \mathbf{0.1\ mm}$$

From Eq. 6.62, $\theta'_L = \frac{2L_0\delta'_s}{3bL}$

$$= \frac{2 \times 425 \times 0.14}{3 \times 6987 \times 10500} = \mathbf{541 \times 10^{-9}\ rad}$$

From Eq. 6.63, $\theta'_H = \left|\frac{\delta'_s}{b}\right| - \theta'_L$

$$= \left|\frac{0.14}{6987}\right| - 541 \times 10^{-9} = \mathbf{19.5 \times 10^{-6}\ rad}$$

Thus, the additional shear deflections at the *HME* and at mid-span are:

δ_v at the *HME* = $6987 \times 19.5 \times 10^{-6}$ = **0.1 mm**

δ_v at midspan = $5250 \times 19.5 \times 10^{-6}$ = **0.1 mm**

Total Additional Deflections

The total additional deflection of the beam at the in-service condition is the sum of the bending and shear components.

$(\delta_b + \delta_v)$ at the *HME* = 0.4+0.1 = **0.5 mm**

$(\delta_b + \delta_v)$ at midspan = 0.3+0.1 = **0.4 mm**

For this particular example, the additional deflections due to the penetration are not significant.

9. REFERENCES

1. Standards Australia, *Steel Structures*, AS 4100-1990.
2. Standards Australia, *Composite Structures, Part 1: Simply-Supported Beams*, AS 2327.1-1996.
3. OneSteel Market Mills, *Design of Simply Supported Composite Beams for Strength (To Australian Standard AS 2327.1-1996)*, Composite Structures Design Manual DB1.1, 2nd Ed., February, 2001.
4. American Society of Civil Engineers, Task Committee on Design Criteria for Composite Structures in Steel and Concrete, *Proposed Specification for Structural Steel Beams with Web Openings*, Journal of Structural Engineering, ASCE, 118(12), 1992, pp. 3315-3349.
5. Chick, C.G., Dayawansa, P.H. and Patrick, M., *Design of Composite Beams with Large Steel Web Penetrations*, 15th Aust. Conf. on the Mech. of Struct. and Mat., Melb., Australia, 1997, pp. 159-164.
6. Chick, C.G., Dayawansa, P.H. and Patrick, M., *Strength Design of Simply-Supported Composite Beams with Large Steel Web Penetrations*, Aust. Struct. Engg. Conf., Auck., New Zealand, 1998, pp. 159-166.
7. Tse, D. and Dayawansa, P.H., *Elastic Deflection of Steel and Composite Beams with Web Penetrations*, The Structural Engineer, J. of the Inst. of Struct. Eng., 70(21), 1992, pp. 372-376.
8. Redwood, R.G. and Cho, S.H., *Design of Steel and Composite Beams with Web Openings*, J. Construct. Steel Research, 25, 1993, pp. 23-41.
9. Lawson. R.M., *Design for Openings in the Webs of Composite Beams*, CIRIA Special Publication 51, CIRIA/SCI, London, 1987.
10. Oehlers, D.J. and Bradford, M.A., *Composite Steel and Concrete Structural Members: Fundamental Behaviour*, Pergamon, 1995.
11. Chick, C.G., Dayawansa, P.H. and Goh, C.C., *Reliability Analysis to Determine ϕ-factor for Strength Design of Web Penetrations*, BHP Melbourne Research Laboratories Report.
12. OneSteel Reinforcing, *DECKMESH*™, September, 2000.
13. Patrick, M., *The Application of Structural Steel Decking in Commercial and Residential Buildings,* Malaysian Structural Steel Association Convention, (Paper 6), December, 1998.
14. Watson, K.B., Dallas, S., van der Kreek, N. and Main, T., *Costing of Steelwork from Feasibility through to Completion*, Steel Construction Journal, AISC, 30(2), 1996.
15. Patrick, M., Tse, D. and Wilkie, R., *Combined Shear-Tension Testing of the Hilti HVB Shear Connector*, Structural Steel PSSC '95 4th Pacific Structural Steel Conference, Vol. 3, Steel-Concrete Composite Structures, Pergamon, 1995, pp. 203-210.
16. Patrick, M., Dayawansa, P.H. and Watson, K.B., *A New Reinforcing Component for Preventing Longitudinal Shear Failure of Composite Edge Beams*, Structural Steel PSSC '95 4th Pacific Structural Steel Conference, Vol. 3, Steel-Concrete Composite Structures, Pergamon, 1995, pp. 101-108.
17. Patrick, M., Dayawansa, P.H., Eadie, I., Watson, K.B. and van der Kreek, N., *Australian Composite Structures Standard AS 2327, Part 1: Simply-Supported Beams*, Steel Construction Journal, AISC, 29(4), December, 1995, pp. 2-40.
18. Redwood, R.G. and Shrivastava, S.C., *Design Recommendations for Steel Beams with Web Holes*, Can. J. Civ. Engg., **7**(4), 1980, pp. 642-650.
19. OneSteel Market Mills, *Design of the Shear Connection of Simply Supported Composite Beams (To Australian Standard AS 2327.1-1996)*, Composite Structures Design Manual DB1.2, 1st Ed., February, 2001.

Design of Simply-Supported Composite Beams with Large Web Penetrations

Design Booklet DB1.3

OneSteel Market Mills

Composite Structures Design Manual

February 2001

Published by

OneSteel Manufacturing Pty Limited

ABN 42 004 651 325

Produced by the

Centre for Construction Technology & Research

University of Western Sydney

Contributors

Dr. Mark Patrick *

Centre for Construction Technology & Research

Dr. Cameron Chick *
Dr. Daya Dayawansa *
Dr. Chong Chee Goh *
Mr. Rodney Wilkie *

* Formerly BHP Melbourne Research Laboratories

Reviewed by

Dr. Brian Uy

The University of New South Wales

Edition 1.0 - April 1999

Edition 2.0 - February 2001

Disclaimer

Foreword

OneSteel is a leading manufacturer of steel long products in Australia after its spin-off from BHP Pty Ltd on the 1st November 2000. It manufactures a wide range of steel products, including structural, rail, rod, bar, wire, pipe and tube products and markets welded beams.

OneSteel is committed to providing to design engineers, technical information and design tools to assist with the use, design and specification of its products. This design booklet "Design of Simply-Supported Beams with Large Web Penetrations" was the third design booklet of the Composite Structures Design Manual, which is now being completed and maintained by OneSteel.

The initial development work required to produce the design booklets was carried out at BHP Melbourne Research Laboratories before its closure in May 1998. OneSteel Market Mills is funding the University of Western Sydney's Centre for Construction Technology and Research in continuing the research and development work to publish this and future booklets.

The Composite Structures Design Manual refers specifically to the range of long products that are manufactured by OneSteel and plate products that continue to be manufactured by BHP. It is strongly recommended that OneSteel sections and reinforcement and BHP plate products are specified for construction when any of the design models in the design booklets are used, as the models and design formulae including product tolerances, mechanical properties and chemical composition have been validated by detailed structural testing using only OneSteel and BHP products.

To ensure that the Designer's intent is met, it is recommended that a note to this effect be included in the design documentation.

Contents

Preface

This design booklet forms part of a suite of booklets covering the design of simply-supported and continuous composite beams, composite slabs, composite columns, steel and composite connections and related topics. The booklets are part of the OneSteel Market Mills' Composite Structures Design Manual which has been produced to foster composite steel-frame building construction in Australia to ensure cost-competitive building solutions for specifiers, builders and developers.

The additional design information necessary to allow large web penetrations to be incorporated into simply-supported bare steel and composite beams is presented in this booklet. Design issues with respect to strength and deflection control are addressed. The non-composite bare steel state arises during construction prior to the concrete hardening.

Large rectangular and circular penetrations are often made in the steel web of composite beams for the passage of horizontal building services. This allows the plenum height to be reduced when using economical, standard UB and WB steel sections. However, large penetrations weaken a composite beam locally and reduce its overall flexural stiffness, and therefore their effect must be considered in design.

Neither the Steel Structures Standard AS 4100 nor the Composite Beam Standard AS 2327.1 contains design provisions for large web penetrations. The rules provided in the booklet for designing bare steel beams with large penetrations are compatible with AS 4100. For the composite state, the rules are compatible with AS 2327.1, and have been proposed as an acceptable method of design to be referred to in Amendment No. 1 of this Standard expected to be published this year.

Information is also given to assist design engineers to understand the engineering principles on which the design methods are based. This includes:

(a) explanatory information on important concepts and models;

(b) the limits of application of the methods; and

(c) worked examples.

Design capacity tables are given in Appendix C to simplify the strength design process. The information provided can be used to design for either the bare steel or composite states. The tables cover a range of situations involving 300PLUS® UB and WB steel sections supporting a composite slab and incorporating large web penetrations. A spreadsheet program named WEBPEN™ is available to assist with the strength design calculations.

Although these design aids are intended to make the design process more efficient, it is essential that the user obtain a clear understanding of the basis of the design rules and the design approach by working through this document and the relevant parts of associated design Standards such as AS 4100 and AS 2327.1.