8.6.7.2 Resistance under axial force, shear and out-of-plane loading

(1) The methods in 8.6.6.2 may be used in the ultimate limit state when in-plane compression or shear is less than 10 % of the corresponding resistance. The methods in 8.6.2 to 8.6.5 may be used if the out-of-plane loading is less than 10% of the corresponding resistance.

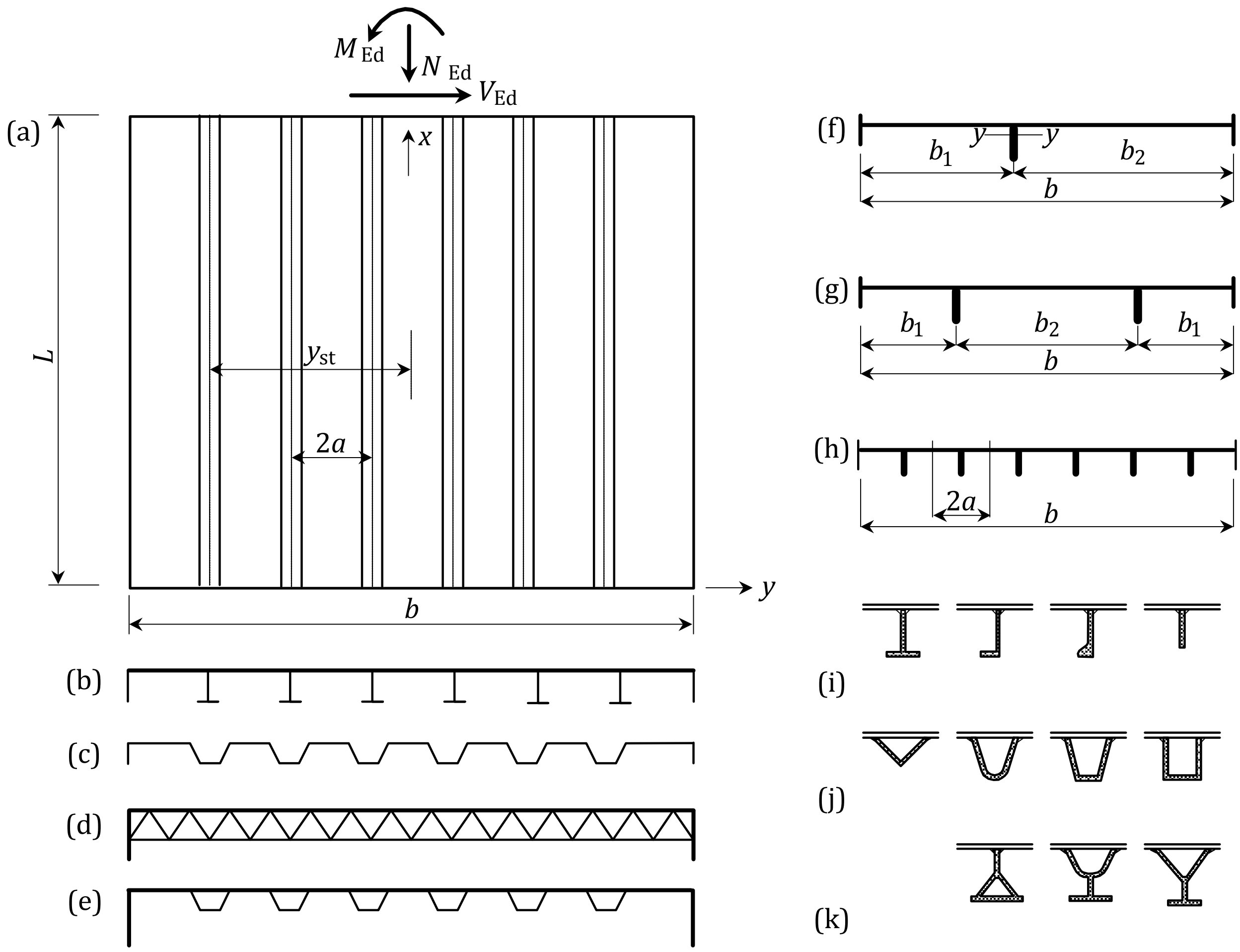
8.7 Stiffened plates under in-plane loading

8.7.1 General

(1) 8.7 should be applied for plates supported on all four edges and stiffened with one or two, central or eccentric longitudinal stiffeners, or three or more uniformly spaced longitudinal stiffeners or corrugations (see Figure 8.25). The clause should also be used for orthotropic plating (see Figure 8.25(c), (d) and (e) and subclause 8.7.6) and for extruded profiles with one or two open stiffeners, see 8.1.4.3.

(2) The stiffeners may be unsupported on their whole length or be continuous over intermediate transverse stiffeners. The dimension, L, should be taken as the spacing between the supports. The longitudinal stiffening, but not the transverse stiffening, may be ”sub-critical”, i.e. it may deform with the plating in an overall buckling mode.

(3) The resistance of stiffened plating to longitudinal normal stress in the direction of the stiffeners should be found using 8.7.2 to 8.7.4, and the resistance in shear should be found using 8.7.5. Interaction between different effects may be taken into account in the same way as for un-stiffened plates (see 8.8.6). The provisions are valid also if the cross-section contains parts that are classified as slender.



Key

(i) open stiffeners

(j) closed stiffeners

(k) partly closed stiffeners

Figure 8.25 — Stiffened plates and types of stiffeners

(4) If the structure consists of flat plating with longitudinal stiffeners, the resistance to transverse normal stress may be taken the same as for an unstiffened plate. With corrugated structure the resistance to transverse normal stress may be neglected.

NOTE Orthotropic plating can have considerable resistance to transverse in-plane normal stress.

8.7.2 Stiffened plates under uniform compression

(1) The cross-section shall be classified as compact or slender in accordance with 8.1.4, considering all the component parts before carrying out either check.

The design value of the compression force, N_{Ed}, shall satisfy Formula (8.108):

$$\frac{N_{Ed}}{N_{Rd}} \leq 1,0 \tag{8.108}$$

where

N_{Rd} is the smallest of $N_{net,Rd}$ $N_{u,Rd}$ and $N_{c,Rd}$ according to (2) and (3).

(2) $N_{net,Rd}$ and $N_{u,Rd}$ should be obtained from 8.2.4.

(3) In columns the plating should be regarded as an assemblage of identical column sub-units, each containing one centrally located stiffener or corrugation and with a width equal to the pitch, $2a$, see Figure 8.25. The design axial resistance, $N_{c,Rd}$, should be taken as given by Formula (8.109):

$$N_{c,Rd} = A_{eff}\chi f_o/\gamma_{M1} \tag{8.109}$$

where

- χ is the reduction factor for flexural buckling of the sub-unit;
- A_{eff} is the effective area of the cross-section of the plating taking into account local buckling and HAZ softening due to longitudinal welds. HAZ softening due to welds at the loaded edges or at transverse stiffeners as well as unfilled holes may be ignored in finding A_{eff}.

The reduction factor, χ, should be obtained from the appropriate buckling curve relevant to column buckling of the sub-unit as a simple strut out of the plane of the plating.

(4) The relative slenderness, $\overline{\lambda}$, in calculating, χ, is given by Formula (8.110):

$$\overline{\lambda} = \sqrt{\frac{A_{eff} f_o}{N_{cr}}} \tag{8.110}$$

where

- N_{cr} is the elastic orthotropic buckling load based on the gross cross-section

(5) For a plate with open stiffeners, N_{cr} should be according to Formulae (8.111) and (8.112):

$$N_{cr} = \frac{\pi^2 EI_y}{L^2} + \frac{L^2 c_e}{\pi^2} \quad \text{if } L < \pi \sqrt[4]{\frac{EI_y}{c_e}} \tag{8.111}$$

$$N_{cr} = 2\sqrt{c_e\, EI_y} \quad \text{if } L \geq \pi \sqrt[4]{\frac{EI_y}{c_e}} \tag{8.112}$$

where

- c_e is the elastic support stiffness from the plate per unit of plate width according to formulae (8.116), (8.117) or (8.118);
- I_y is the second moment of area of all stiffeners and plating within the width, b, with respect to y-axes in Figure 8.25(f).

(6) For a cross-section part with one central or eccentric stiffener, the elastic support stiffness from the plate per unit of plate width may be calculated as given by Formula (8.113) (Figure 8.25(f)):

$$c_e = \frac{0,27Et^3 b}{b_1^2 b_2^2} \tag{8.113}$$

where

t is the thickness of the plate;

b is the overall width of the plate;

b_1 *and* b_2 are the width of plate parts on both sides of the stiffener.

(7) For a cross-section part with two symmetric stiffeners at a distance, b_1, from the longitudinal supports (see Figure 8.25(g)) the elastic support stiffness per unit of plate width may be calculated using Formula (8.114):

$$c_e = \frac{1,1Et^3}{b_1^2(3b - 4b_1)} \tag{8.114}$$

(8) For a multi-stiffened plate with open stiffeners (Figure 8.25(c), (b) (h) and (i)) with small torsional stiffness the elastic support stiffness from the plate per unit of plate width may be calculated using Formula (8.115):

$$c_e = \frac{8,9Et^3}{b^3} \tag{8.115}$$

(9) For a multi-stiffened plate with closed or partly closed stiffeners (Figure 8.25 (e) and (j)), the elastic orthotropic buckling load may be calculated using the formulae in 8.7.6.

(10) The half-wavelength in elastic buckling, used if the applied action varies in the direction of the stiffener or corrugations (see 8.7.4(3)), should be calculated using Formula (8.116):

$$l_w = \pi \sqrt[4]{\frac{EI_y}{c_e}} \tag{8.116}$$

8.7.3 Stiffened plates under in-plane moment

(1) Two checks should be performed, a yielding check (8.7.3(3)) and a column check (8.7.3(4)).

(2) The cross-section should be classified as one of Class 2, 3 or 4 (see 8.1.4) when carrying out the checks in (1). For the purpose of classifying individual parts, and also when determining effective thicknesses of slender parts, each part should be assumed to be under uniform compression taking $\eta = 1$ in 8.1.4.3. However, in the case of the yielding check only, the calculation of η may be based on the actual stress pattern in parts comprising the outermost region of the plating, and to retain this value for the corresponding parts further in.

NOTE Using in parts of the plate further in from the edge the value of η which is based on the actual stress pattern in the outermost region of the plating, can be favourable if the number of stiffeners or corrugations is small.

(3) In the yielding check the entire cross-section of the plating should be treated as a beam under in-plane bending (see 8.2.5). The design moment resistance, M_{Rd}, should be based on the least favourable cross-section, taking account of local buckling, of any holes and HAZ softening if relevant.

(4) In the column check the plating should be regarded as an assemblage of column sub-units in the same general way as for axial compression (see 8.7.2(3)). The design moment resistance, $M_{c,Rd}$, should be taken as given by Formula (8.117):

$$M_{c,Rd} = \frac{\chi I_{eff} f_o}{y_{st} \gamma_{M1}} \tag{8.117}$$

where

χ is the reduction factor for flexural buckling of the sub-unit, to be determined in the same way as for uniform compression (see 8.7.2(3));

I_{eff} is the second moment of area of the effective cross-section of the plating for in-plane bending;

y_{st} is the distance from the centre of plating to that of the outermost stiffener, see Figure 8.25.

8.7.4 Longitudinal stress gradient on multi-stiffened plates

(1) The provisions of this clause should be satisfied in cases where the applied action effects N_{Ed}, or M_{Rd}, on a multi-stiffened plate varies in the direction of the stiffeners or corrugations.

(2) In the yielding check the design resistance at any cross-section should be not less than the design action effect at that section.

(3) In the column check the design resistance should be compared to the design action effect at a distance $0{,}4l_w$ from the more heavily loaded end of a panel,

where

l_w is the half wavelength in elastic buckling according to 8.7.2(10).

8.7.5 Multi-stiffened plating in shear

(1) A yielding check, see (2), and a buckling check, see (3), should be performed. The methods given in (2) and (3) are valid provided the stiffeners or corrugations, as well as the actual plating, are:

a) effectively connected to the transverse framing at either end;

b) continuous at any transverse stiffener position.

(2) In the yielding check the design shear force resistance, V_{Rd}, should be taken as the same as that for a flat unstiffened plate of the same overall size ($L \times b$) in accordance with 8.6.5(2), taking into account that $L = a$.

(3) In the buckling check the design shear force resistance should be calculated according to 8.9.3, using the following plate stiffnesses per unit width of the plate. (Note the different coordinate system, x and y in Figure 8.25 and z and x in Figure 8.36):

B_y $= Et^3/10{,}9$ for a flat plate with stiffeners, otherwise see 8.7.6(1);

B_x $= EI_y/b$ where I_y is the second moment of area of stiffeners and plating within the width, b, about a centroidal axis parallel to the plane of the plating;

h_w is the buckling length, l, which may be conservatively taken as the unsupported length, L, (see Figure 8.25). If L greatly exceeds b, a less conservative result may be obtained by setting $\tau_{cr,g}$ equal to the elastic orthotropic shear buckling stress given by Formula (8.129), which should be modified if $\phi > 1$ (see 8.6.6(2)). HAZ softening may be neglected in the buckling check.

8.7.6 Buckling load for orthotropic plates

(1) For an orthotropic plate under uniform compression the procedure in 8.7.2 may be used. The elastic orthotropic buckling load, N_{cr}, for a simply supported orthotropic plate may be calculated as given by Formulae (8.118) and (8.119):

$$N_{cr} = \frac{\pi^2}{b}\left[\frac{B_x}{(L/b)^2} + 2H + B_y(L/b)^2\right] \quad \text{if } \frac{L}{b} < \sqrt[4]{\frac{B_x}{B_y}} \tag{8.118}$$

$$N_{cr} = \frac{2\pi^2}{b}\left[\sqrt{B_x B_y} + H\right] \quad \text{if } \frac{L}{b} \geq \sqrt[4]{\frac{B_x}{B_y}} \tag{8.119}$$

Formulae for B_x, B_y and H for different cross-sections are given in Table 8.9 or Formulae (8.120) to (8.128). Indices x and y indicates rigidity in section x = constant and y = constant, respectively.

Table 8.9, Case No. 2:

$$B_y = \frac{2Ba}{2a_4 + \dfrac{2a_1 a_3 t_1^3 (4a_2 t_3^3 + a_3 t_2^3)}{a_3 t_1^3 (4a_2 t_3^3 + a_3 t_2^3) + a_1 t_3^3 (12a_2 t_3^3 + 4a_3 t_2^3)}} \tag{8.120}$$

$$H = 2B + \frac{GI_t}{2a} \frac{1}{2 + \dfrac{8GI_t}{Lab}\left[\varphi_{plt} + \varphi_{dis}\right]} \tag{8.121}$$

where

$$\varphi_{plt} = \frac{2a_4^2}{aEt_1^3}\left[a_4 + \frac{a_1(3\alpha_3 + 4\alpha_2)}{3\alpha_3 + 4\alpha_2 + 4\alpha_1 + 4\alpha_1\alpha_2/\alpha_3}\right] \tag{8.122}$$

$$\varphi_{dis} = \frac{ha_2\left(3\alpha_3^2 + 4\alpha_1\alpha_2 + 4\alpha_1\alpha_3 + 4\alpha_2\alpha_3\right)}{2a_2\alpha_2 + f2a_1\alpha_1 + (2+f)a_2\alpha_3 + (1+2f)a_1\alpha_3} \tag{8.123}$$

$$B = \frac{Et_1^3}{12(1-\nu^2)} \tag{8.124}$$

$$\alpha_1 = \frac{2a_1}{Et_1^3}, \quad \alpha_2 = \frac{2a_2}{Et_2^3}, \quad f = 1 + \frac{(a_1 - a_2)a_3}{ha_2} \tag{8.125}$$

Table 8.9, Case No. 5:

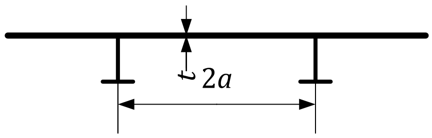
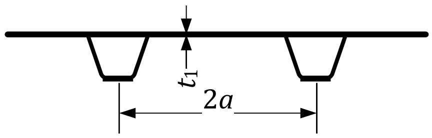
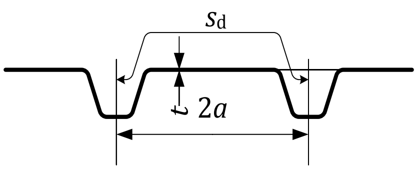
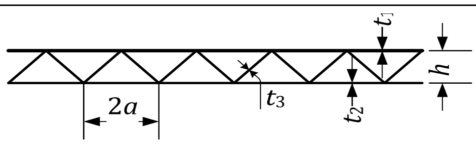
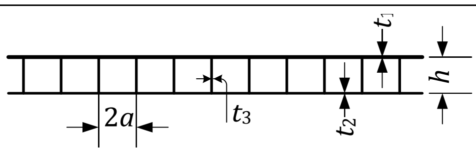
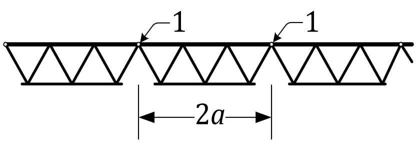
$$B_y = \frac{1}{\dfrac{1}{D_{em}} + \dfrac{t_1 + t_2}{Et_1 t_2 h^2}} \tag{8.126}$$

where

$$D_{\mathrm{em}} = \frac{Et_1^3}{12(1-\nu^2)} \frac{10b^2}{32a^2} \frac{at_3^3 + at_2^3 t_3^3 / t_1^3 + 6ht_2^3}{at_3^3 + 2h(t_1^3 + t_2^3) + 3h^2 t_1^3 t_2^3 / (at_3^3)} \tag{8.127}$$

$$H = \frac{2E}{3\left(1 - \frac{t_3}{2a}\right)^3} \left[\frac{t_1^3}{1 + \frac{6t_1}{2a - t_3}} + \frac{t_2^3}{1 + \frac{6t_2}{2a - t_3}} \right] \tag{8.128}$$

Table 8.9 — Flexural and torsional rigidity of orthotropic plates

Case No	Cross-section	B_x (corresponds to EI_y)	B_y (corresponds to EI_x)	H
1		$\frac{EI_L}{2a}$	$\frac{Et^3}{12\left(1-\nu^2\right)}$	$\frac{Gt^3}{6}$
2	 For details, see Figure 8.26	$\frac{EI_L}{2a}$	Eq. (8.120)	Eq. (8.121)
3		$\frac{EI_L}{2a}$	$\frac{2a}{s_d} \frac{Et^3}{12(1-\nu^2)}$	$\frac{2a}{s_d} \frac{Gt^3}{6}$
4		$\frac{EI_L}{2a}$	$\frac{Et_1 t_2 h^2}{t_1 + t_2}$	$\frac{GI_t}{2a}$
5		$\frac{EI_L}{2a}$	Eq. (8.126)	Eq. (8.128)
6		$\frac{EI_L}{2a}$	0	$\frac{GI_t}{2a}$

Key

I_L is the second moment of area of one stiffener and adjacent plating within $2a$

I_t is the torsional constant of the same cross-section

1 groove and tongue

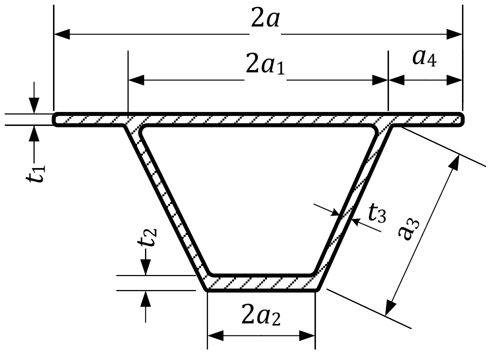


Figure 8.26 — Cross-section notations of closed stiffener

(2) The shear force resistance for an orthotropic plate with respect to global buckling, for $\varphi \leq 1$, may be calculated according to 8.9.3(3), by using Formulae (8.129) to (8.132):

$$\tau_{\text{cr,g}} = \frac{k_\tau \pi^2}{LA} \sqrt[4]{B_y B_x^3} \tag{8.129}$$

$$k_\tau = 3{,}25 - 0{,}567\varphi + 1{,}92\varphi^2 + (1{,}95 + 0{,}1\varphi + 2{,}75\varphi^2)\eta_h \tag{8.130}$$

$$\varphi = \frac{L}{b} \sqrt[4]{\frac{B_y}{B_x}} \tag{8.131}$$

$$\eta_h = \frac{H}{\sqrt{B_x\ B_y}} \qquad \text{(Valid for } \eta_h < 1{,}5) \tag{8.132}$$

B_x, B_y and H are given in Table 8.9 and A is the cross-sectional area in the smallest section for y = constant.

$A = Lt$ for cases 1, 2 and 3 in Table 8.9 and $A = L(t1 + t2)$ for cases 4 and 5. The formulae are not applicable to case 6.

(3) The shear force resistance for an orthotropic plate with respect to global buckling for $\varphi > 1$, may be calculated according to 8.9.3(3), using the Formulae of (2) with subscripts x and y as well as the width, b, and the length, L, in (8.129) to (8.132) interchanged and with $A = b\Sigma t$.

8.7.7 Out-of-plane loading

8.7.7.1 General

(1) For an orthotropic plate under out-of-plate loading, the bending moments, $m_{\text{x,Ed}}$ and $m_{\text{y,Ed}}$, shall be less than the resistances of the plate per unit width. The resistances depend on the direction of loading, on the plate side, $m_{\text{x,p,Rd}}$ and $m_{\text{y,p,Rd}}$, or on the stiffener side, $m_{\text{x,s,Rd}}$ and $m_{\text{y,s,Rd}}$, see 8.2.5 and Figure 8.27, given by Formulae (8.133) and (8.134):

$$m_{\text{x,p,Rd}} = m_{\text{x,s,Rd}} = \frac{\alpha_x W_{x,\text{el}}}{2a} \frac{f_o}{\gamma_{\text{M1}}} \tag{8.133}$$

$$m_{\mathrm{y,p,Rd}} = m_{\mathrm{y,s,Rd}} = \frac{\alpha_y W_{y,\mathrm{el}}}{L} \frac{f_{\mathrm{o}}}{\gamma_{\mathrm{M1}}} \tag{8.134}$$

where

$W_{\mathrm{x,el}}$	is the section modulus for x-x-axis bending of one stiffener including the adjacent plate within $2a$;
α_{x}	is the shape factor according to 8.2.5.1(2) referring to the compression side of the stiffened plate;
$2a$	is the distance between stiffeners.

— for case 1, 2 and 3 in Table 8.9: $\alpha_{\mathrm{y}} W_{\mathrm{y,el}} = Lt^2/4$;

— for case 4 and 5: $W_{\mathrm{y,el}}$ is the section modulus for y-y-axis bending of the entire plate length, L, and α_{y} is the shape factor according to 8.2.5.1(2) referring to the compression side of the stiffened plate.

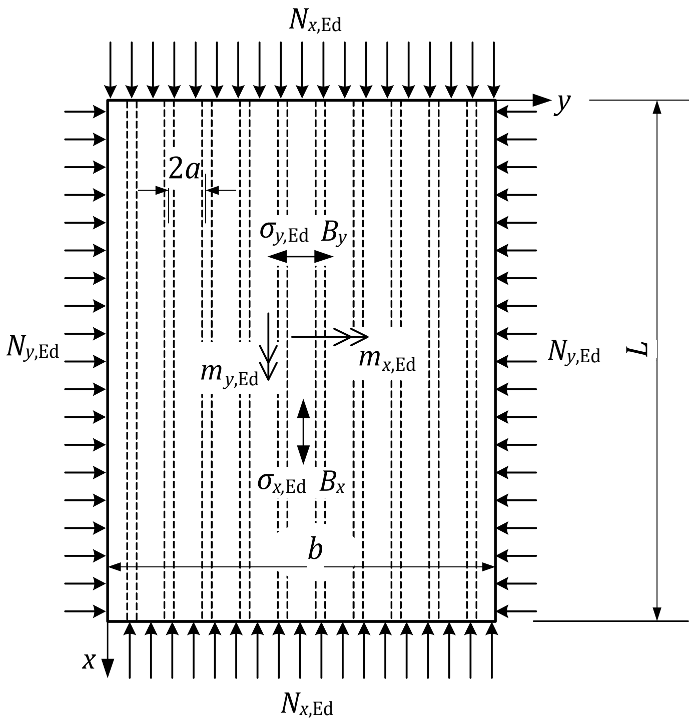


Figure 8.27 — Notations for stiffened plate

8.7.7.2 Uniform distributed loading

(1)The deflection, w, and the moments, $m_{\mathrm{x,Ed}}$ and $m_{\mathrm{y,Ed}}$, may be calculated using numerical methods e.g. the finite elements method (see Annex B), or, for a simply supported plate, with the Formulae (8.135) to (8.137):

$$m_{x,\mathrm{Ed}} = B_x \frac{16 q_{\mathrm{Ed}}}{\pi^6} \sum_{m=1,3,5}^{\infty} \sum_{n=1,3,5}^{\infty} \left(\frac{\left(\frac{m\pi}{L}\right)^2 \cdot \sin\left(\frac{m\pi x}{L}\right) \sin\left(\frac{n\pi y}{b}\right)}{mn\left(\frac{m^4}{L^4} B_x + \frac{2m^2 n^2}{L^2 b^2} H + \frac{n^4}{b^4} B_y\right)} \right) \tag{8.135}$$

$$m_{y,\mathrm{Ed}} = B_y \frac{16q_{\mathrm{Ed}}}{\pi^6} \sum_{m=1,3,5}^{\infty} \sum_{n=1,3,5}^{\infty} \left(\frac{\left(\frac{n\pi}{b}\right)^2 \cdot \sin\left(\frac{m\pi x}{L}\right) \sin\left(\frac{n\pi y}{b}\right)}{mn\left(\frac{m^4}{L^4} B_x + \frac{2m^2n^2}{L^2b^2} H + \frac{n^4}{b^4} B_y\right)} \right) \quad (8.136)$$

$$w = \frac{16q_{\mathrm{Ed}}}{\pi^6} \sum_{m=1,3,5}^{\infty} \sum_{n=1,3,5}^{\infty} \left(\frac{\sin\left(\frac{m\pi x}{L}\right) \sin\left(\frac{n\pi y}{b}\right)}{mn\left(\frac{m^4}{L^4} B_x + \frac{2m^2n^2}{L^2b^2} H + \frac{n^4}{b^4} B_y\right)} \right) \quad (8.137)$$

NOTE 1 For cases 1 and 3 in Table 8.9 the influence of the transverse bending stiffness can be disregarded when calculating the bending moments. This means that the plate works as a series of beams.

NOTE 2 Usually three terms in the series in (8.135), (8.136) and (8.137), are sufficient.

8.7.7.3 Patch loading

(1) For determination of deflection and moments under patch loading numerical methods may be used. See Annex B.

8.7.8 Resistance under combined loading

8.7.8.1 General

(1) Two methods may be used to verify the resistance of a stiffened plate under combined loading:

1) The stressed skin design method according to 8.7.8.2;

2) The interaction method according to 8.7.8.2.

8.7.8.2 Stressed skin design method

(1) This design method may be used for plate panels with the main function to carry in-plane shear loads. All axial membrane stresses should be carried by the adjoining framing which should be verified for them. In the analysis the plate panels may be modelled with only shear stiffness.

(2) Interaction between the shear force, V_{Ed}, and the bending moments, $m_{\mathrm{x,Ed}}$ and $m_{\mathrm{y,Ed}}$, due to out-of-plane loading may be ignored if $V_{\mathrm{Ed}}/V_{\mathrm{Rd}} \leq 0{,}32$ or $\eta_{\mathrm{m}} \leq 0{,}32$. Otherwise the condition given by Formulae (8.138) and (8.139) should be satisfied:

$$U = \sqrt{0{,}9\left(\frac{V_{\mathrm{Ed}}}{V_{\mathrm{Rd}}}\right)^2 + 0{,}9\eta_m^2} \leq 1 \quad (8.138)$$

where

U is utilization grade,