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DESIGN OF COMPLEX CROSS-SECTIONS FOR MASONRY

I. Introduction

In this paper a method for calculating the load capacity for complex cross-sections is introduced. Complex cross-sections are every type of cross-sections that are not rectangular.

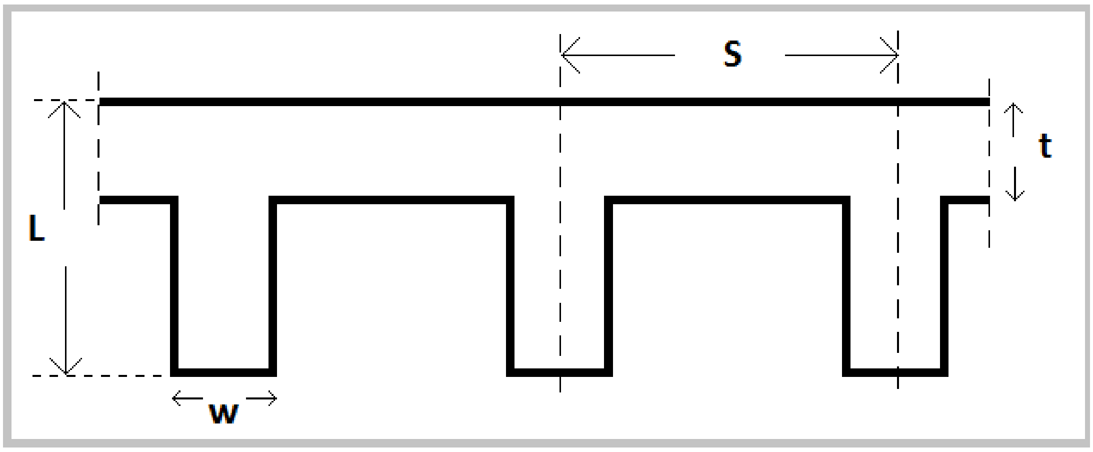

Figure 1. Complex cross-section. Example

In the literature there are some methods for finding the load capacity for complex cross-sections in masonry. They are, however, very sophisticated and require a high degree of numerical calculations and for some cross-sections the use of Finite Element Methods is necessary to find the correct load capacity. All in all not a practical approach for the busy consulting engineer.

The difficulties in designing complex cross-sections have contributed to the current building tradition for masonry. Straight facades and 90 degrees angles in almost all masonry construction (only for veneer walls on steel or concrete frames untraditional constructions can be seen).

In this paper an easy and pragmatic approach for calculating complex structures is introduced using the method of equivalent thickness.

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II. Symbols

A	Area of the complex cross-section
A_{ef}	Area of the transformed cross-section
α	Angle between individual walls
α'	π-α
β	α-$\pi/2$
e	Eccentricity of the load in the complex cross-section
e_{ef}	Eccentricity of the load in the transformed cross-section
f_{vd0}	Design shear strength in the horizontal direction with zero compression
f_{xk1}	Design flexural strength in a horizontal line
f_{xk2}	Design flexural strength in a vertical line
Φ_m	Reduction factor
H	Height of the regarded wall
h_{ef}	Effective column height according to EN 1996-1-1
h	Height of the regarded arch
I	The moment of Inertia (second moment of area)
$I_{flange,i}$	Moment of inertia for flange "i"
I_{body}	Moment of inertia for the body
I_{Total}	The total moment of inertia for S
$I_{stiffening\ wall}$	Moment of inertia of the stiffening wall
$I_{regarded\ wall}$	Moment of inertia of the regarded wall
k	Length of chord of the regarded arch
$L_{equivalent}$	Length of the stiffening flange projected to 90° of the regarded flange
$L_{one\ flange}$	Length of one flange in W-profile
L_1	Length of one flange in profile
L_2	Length of second flange in profile
M_{Rk}	The moment resistance of the wall (general)
M_{Rk1}	The moment resistance in the horizontal direction
M_{Rk2}	The moment resistance in the vertical direction
N_{Ed}	Vertical design load
R	Radius in the regarded arch
S	Design length (unit length) of the complex cross-section
S_i	Part of the design length from flange or body "i"
$S_\square$	First moment of area
S_f	Free span of the flanges/body when examining local buckling, etc
$\sigma_{d,vertical}$	Design compression stress in the construction
η_0	centre of gravity
t	Thickness of the complex cross-section
t_{body}	Height of the body
t_{design}	Value for the thickness used in the line theory calculation
t_{ef}	Effective thickness of the transformed cross-section
$t_{ef,Z1}$	Effective thickness found through Z_1
$t_{ef,Z2}$	Effective thickness found through Z_2
$t_{ef,I}$	Effective thickness found through I
$t_{equivalent}$	Thickness of the stiffening flange projected to 90° of the regarded flange
t_1	Thickness of the upper flange
t_2	Thickness of the lower flange
$t_{1,horizontal}$	Thickness of the flange when projected to a vertical line
τ	Design shear stresses
v	Angle in the regarded arch
W_{Ed}	Horizontal design load

x_1, y_1	Intersection in a coordinate system between L_1 and L_2 when converting an arch
Z_1 and Z_2	Moments of resistance in the upper and lower ends of the cross-section
w	Width/thickness of the body

III. Description of method

Complex (non-rectangular) cross-sections can be designed as a rectangular cross-section using the "equivalent-thickness-method" described below:

Method:

1. For the cross-section (per unit length (S)) the following parameters are determined:

   I: The moment of Inertia

   Z_1 and Z_2: Moments of resistance

   - Equivalent effective thicknesses are determined using:

     $t_{ef,Z1}$: $\sqrt{\frac{6Z_1}{S}}$

     $t_{ef,Z2}$: $\sqrt{\frac{6Z_2}{S}}$

     $t_{ef,I}$: $\sqrt[3]{\frac{12I}{S}}$

   - For mainly horizontal loaded walls t_{ef}: $\min(t_{ef,Z1}, t_{ef,Z2})$
   - For mainly vertical loaded walls t_{ef}: $\min(t_{ef,Z1}, t_{ef,Z2}, t_{ef,I})$
   - With the unit length S and the effective thickness t_{ef} (and the loads, parameters of strength, etc) the "Reduction factor" Φ_t, Φ_m and Φ_b is determined through the rectangular cross-section using annex G in EN 1996-1-1.
   - The load capacity is determined through the area of the complex cross-section (A) (and not A_{ef} based on t_{ef} which is larger)

2. Any eccentricity of the vertical load is proportioned from t to t_{ef}. (see Figure 2)

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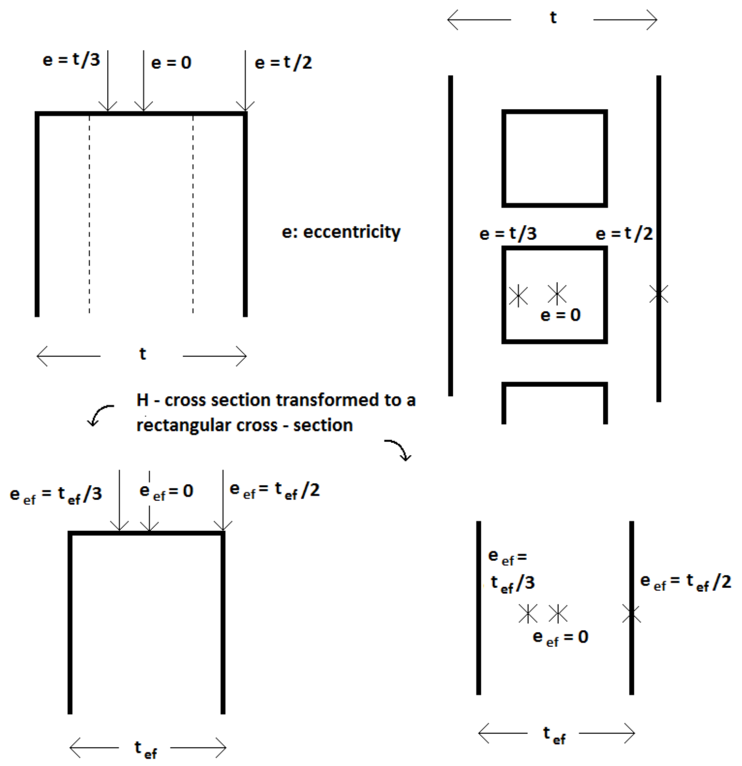

Figure 2. Transformation of the eccentricity from the complex cross-section to the rectangular

3. The shear capacity in the flanges shall be examined. I.e.:

   $\tau \leq f_{vd0}$

4. Instability and load capacity of the individual flanges shall be examined:

   - For vertical loaded walls using the formulas in EN 1996-1-1 (5.6-5.9), 5.5.1.2 (4), etc.
   - For horizontal loaded walls the individual loaded flanges should, if necessary, be examined through the yield line theory and the support evaluated.

For some "inappropriate constructions" phenomena like "lateral buckling" or combined "torsion and buckling" shall be examined. The issue is illustrated in the below figure.

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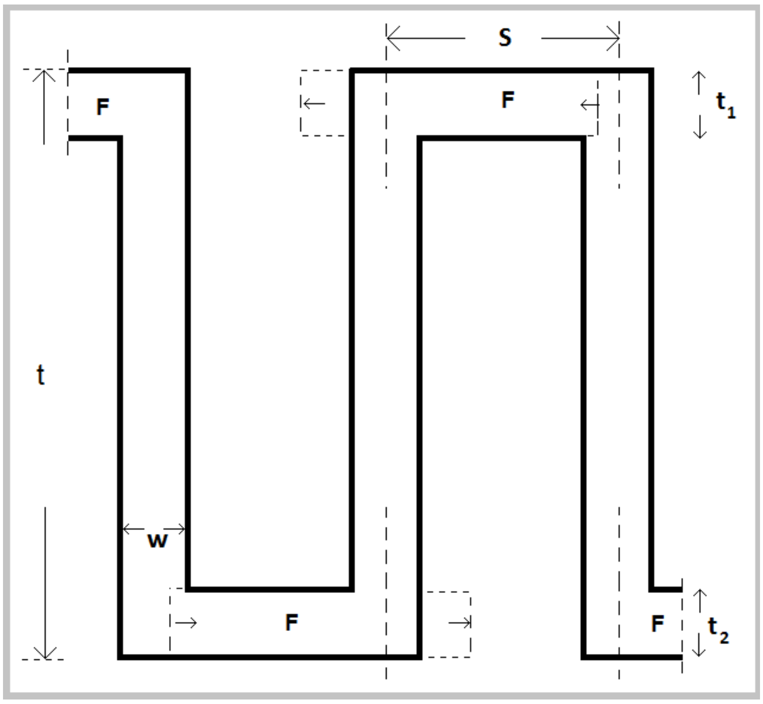

Figure 3. 90 degr serpent wall

For wall types shown in Figure 3 where t >> s failure can occur in the relatively small flanges marked "F" when subject to compression.
At least one of the flanges "F" will be subject to compression when either lateral or vertical loading is applied on the construction.
The failure load is easily determined regarding the flange as a single compressed member with 2nd order deflection in the "strong direction" using the expression in EN 1996-1-1, section 6.4.

Note: these inappropriate constructions are rare and during design the construction shall be optimized securing that the failure loads from lateral buckling and instability of individual flanges are (at least slightly) larger than failure loads from the complex constructions giving the ultimate load determined from t_{ef} .

5. When the complex cross-section is part of a 3- or 4-sided wall to be designed by the yield line theory, t_{ef} shall be implemented in M_{Rk} in the relevant direction only. This procedure is shown in Example 1.

IV. Examples

In this section the method is described through examples.

Calculations (using www.EC6design.com) based on t_{ef} for the masonry are given in Appendix 1.

The intermediate calculations from the spread sheet are shown in Appendix 2.

Example 1: 4 sided – supported diaphragm wall

The numbers (Ref 1-5) correspond to the numbers in the previous section: "III. Description of method".

Ref 1

Determination of t_{ef} (using spread sheet "H O Profiles")

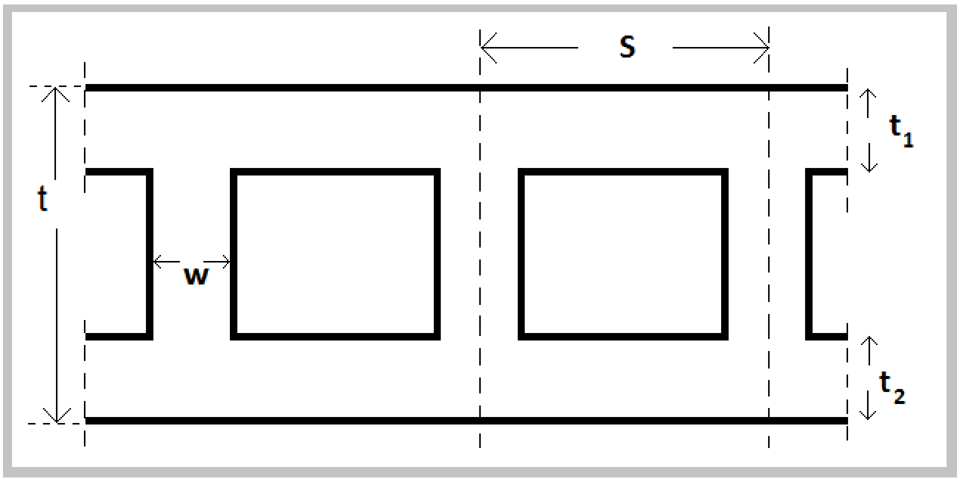

Figure 4. H-cross section/diaphragm wall

A cross-section as shown in Figure 4 is regarded.

Dimensions:

Input

S	1200	mm	H	8.0	m
t	588	mm	N_{Ed}	50	kN/m
t_1	108	mm	W_{Ed}	1.2	kN/m^2
t_2	108	mm			
w	108	mm			

The calculations of I and Z_1 and Z_2 are quite tedious and normally done by a spread sheet. The intermediate calculation (shown in Appendix 2, page 1) gives the output as shown below:

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Output

$t_{ef,I}$:	539 mm
$t_{ef,Z1}$:	516 mm
$t_{ef,Z2}$:	516 mm

Using the introduced formulas, we get:

For mainly horizontal loaded walls t_{ef}: $= \min(t_{ef,Z1}, t_{ef,Z2})$ = **516 mm**
For mainly vertical loaded walls t_{ef}: $= \min(t_{ef,Z1}, t_{ef,Z2}, t_{ef,I})$ = **516 mm**
For the areas:

A: = 299376 mm^2
A_{ef}: = 618989 mm^2

Before finding Φ_m it is necessary to determine the equivalent horizontal load q_{eqv} (See annex I in EN 1996-1-1).
Φ_m is determined in the end of this example.

Ref 2
The vertical load (N_{Ed}) is placed equally on the outer and inner leaf giving a central load.
I.e. the eccentricity is: $e = e_{ef} = 0$

Ref 3
Shear capacity in the flanges.
The shear capacity can be crucial for the load capacity when the cross-section is not massive. The shear stresses are determined through the theory of plasticity:

$$\tau_{max} = [½ \times S \times W_{Ed} \times H]/[w \times (t - t_1/2 - t_2/2)]$$
$$= ½ \times 1.2\text{m} \times 1,2\ \text{kN/m}^2 \times 8\ \text{m} \ \mathbf{/}\ 108\ \text{mm} \times (588 - 54 - 54)\ \text{mm}$$
$$= 0.11\ \text{MPa}$$
$$\leq f_{vd0}$$

Ref 4
Instability and load capacity of the individual flanges.
The outer and inner leaf has the largest effective “free” span (S_f) that is:

$$S_f = 1200 - 108$$
$$=1092\ \text{mm.}$$

In this example

$$t_1 = t_2$$
$$= 108\ \text{mm.}$$

Vertical load
The vertical load on the leaves of the H-cross-section is max: 50/2 kN/m = 25 kN/m giving the stresses:

$$\sigma_{d,vertical} = 25/108$$
$$= 0.23\ \text{MPa}$$
$$\leq f_d$$

When regarding buckling of the compressed flanges h_{ef} is determined using (5.9):

$$h_{ef} = 0{,}5 \times S_f = 0.5 \times 1092 = 546 \text{ mm}$$

and thus:

$$h_{ef}/t_1 = 546/108 = 5.06$$

This value is normally not critical and buckling of the individual flanges is not examined.
If buckling of the individual flanges was regarded as critical it should be examined through the ordinary design rules of vertical loaded walls. The point is, though, to design the construction so buckling of the individual flanges do not occur as in this example.

Horizontal load
Horizontal load on the individual flanges is not regarded crucial for these dimensions.

Ref 5
The cross-section as a part of a 4-sided wall.

The construction can be designed for vertical and horizontal load as a normal rectangular cross-section using the effective thickness

$$t_{ef} = 516 \text{ mm}$$

in the vertical direction.

In this example the wall is assumed being a part of a 4-sided supported wall with the following parameters:

Length = 8 m
Height = 8 m
f_{xk1} = 0.20 MPa
f_{xk2} = 0.50 MPa

Opening and supporting conditions are given in the below figure.

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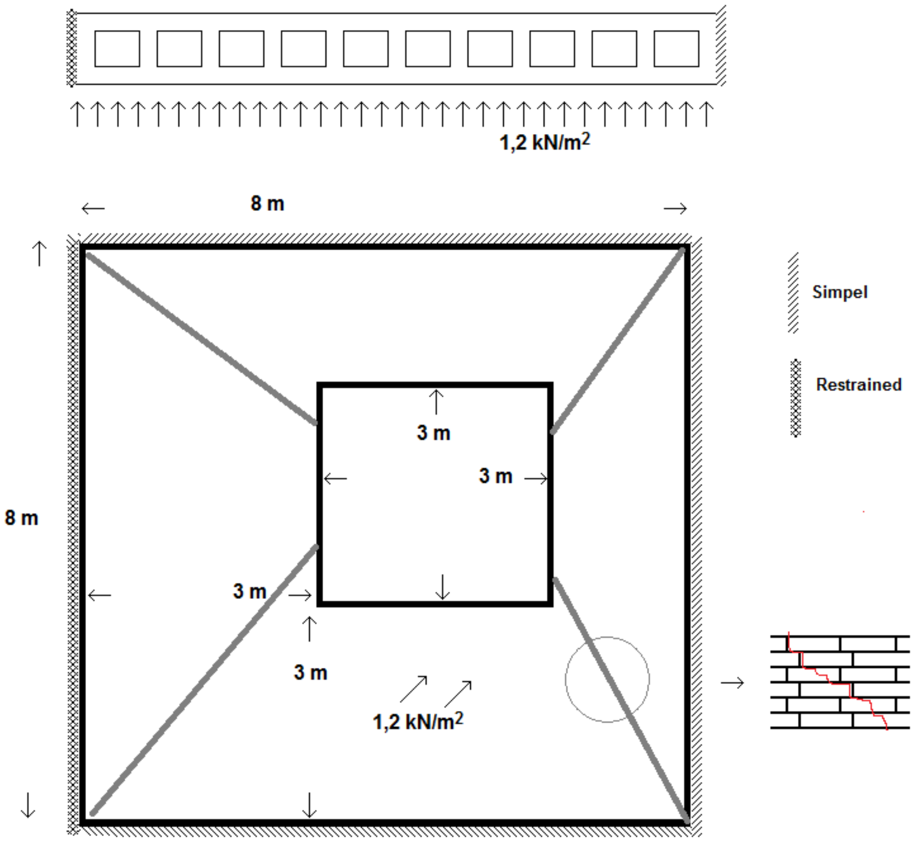

Figure 5. Geometry and supporting conditions for Example 1

See below figure for yield lines in vertical and horizontal directions

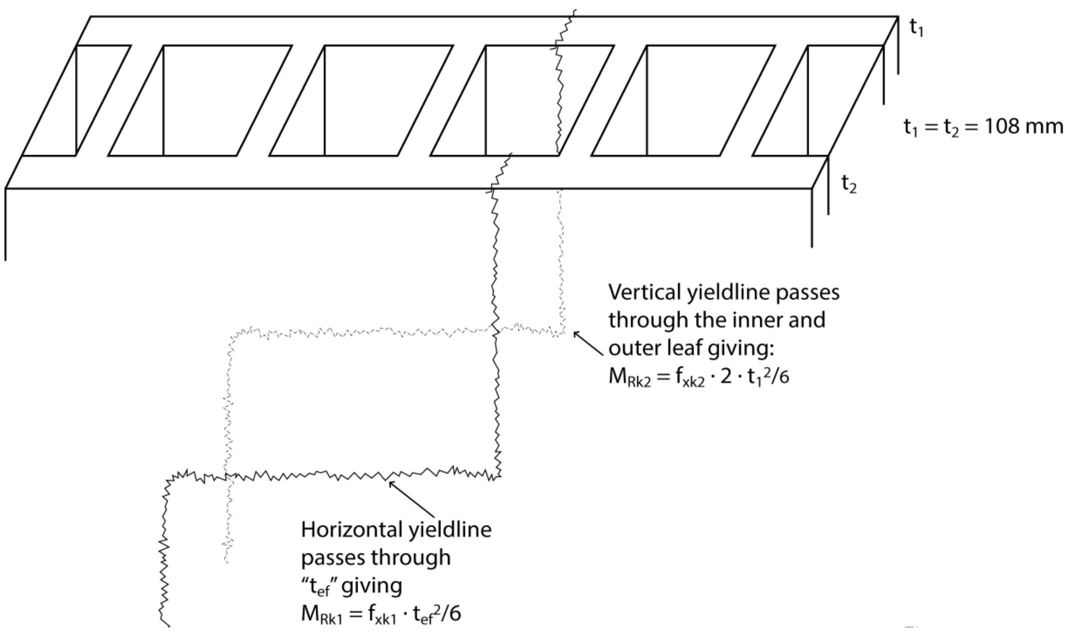

Figure 6. Yield lines in vertical and horizontal directions for continues H-profiles/diaphragm wall

Determining the moment of resistance per unit height and length gives:

$M_{Rk1} = f_{xk1} \times (t_{ef}^2/6)$
$M_{Rk2} = f_{xk2} \times (2 \times t_1^2/6)$

Setting in the relevant parameters:

$M_{Rk1} = 0.20 \times (516^2/6)$
$= 8875$ Nmm/mm

$M_{Rk2} = 0.50 \times (2 \times 108^2/6)$
$= 1944$ Nmm/mm

If the software available only uses one general thickness the values for (f_{xk1}, f_{xk2}) can be adjusted. E.g. to

$t_{design} = t_{ef} = 516$ mm

We get:

$f_{xk1} = 0.20$ MPa
$f_{xk2} = 1944/(516^2/6)$
$= 0.044$ MPa

Using these parameters the load capacity is easily determined when the wall is regarded first as a mainly horizontal loaded wall and then as a mainly vertical loaded wall.

The equivalent horizontal load q_{eqv} (described in Annex I in EN 1996-1-1) will automatically be determined using this approach. In this example we find q_{eqv} = 1.11 kN/m^2

Relevant calculations are given in Appendix 1.

The vertical load is increased due to the window/opening, when determining Φ.

In the calculations we the reduction factor is found to:

$\Phi_m = 0.44$

giving the utility degree:

$U = \sigma / (\Phi_m \times f_d)$
$= F/A / (\Phi_m \times f_d)$
$= 75 \times 10^3/(299376/1,2) /0,44 \times 2,85$
$= 24$ %

Conclusion: The construction has adequate load capacity.

Example 2:
In this example, a chevron wall supporting a casted concrete floor is examined. Geometry, design loads and failure mode (A…C) are shown in the figure below. The wall is regarded as 2-sided supported. Shear stresses are not examined.

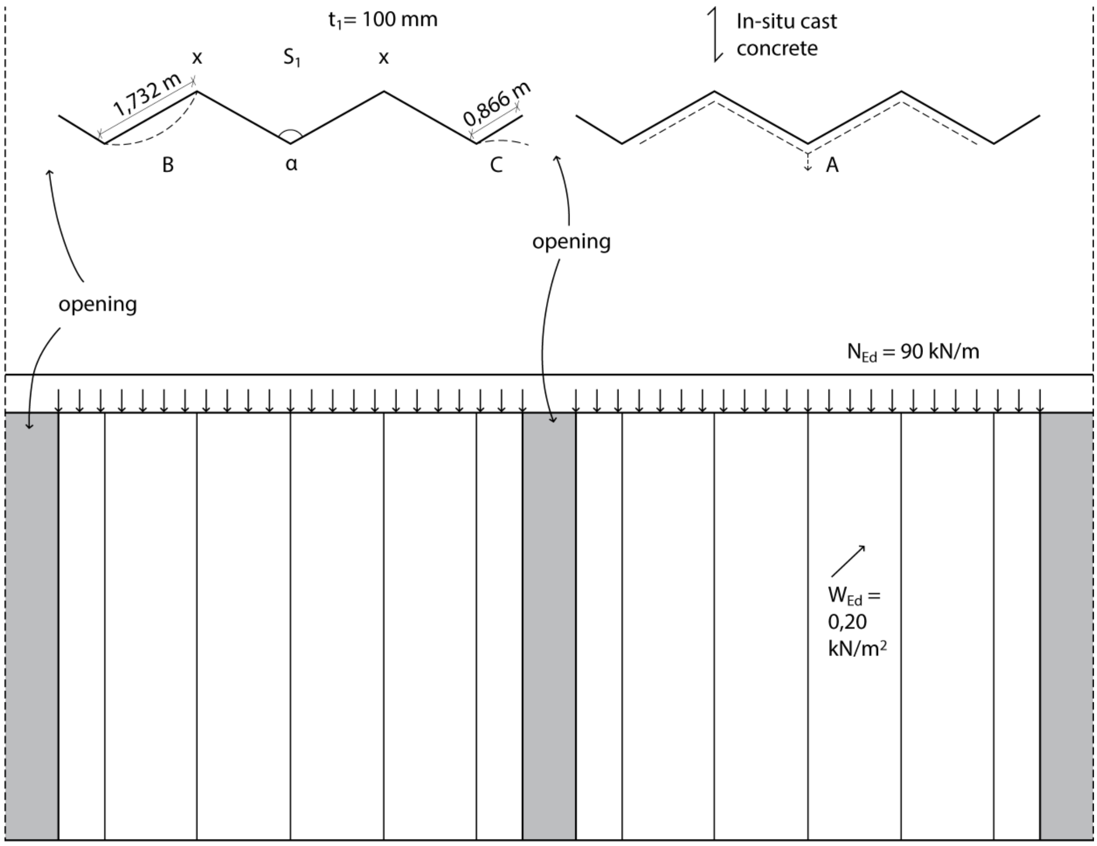

Figure 7. Dimensions and design loads for zigzag walls

The values:

α = 120 °
S1 = 3000 mm

give t_{ef} = 316 mm

(see Appendix 2, page 2)

The walls are stiffening each other. The end walls are 3-sided supported with half width.

The interval of eccentricity is in full length of the wall both in top and bottom.

3 calculations are executed. Relevant parameters are shown in the below table. Parameters like f_k, E, etc. are not relevant for illustration of the method and are not discussed in this note. The values can be seen in the calculations in Appendix 1.

Table 1. Relevant parameters for the calculations

Calcu-lation	Thick-ness (mm)	Eccentricity interval top/bottom (mm)	Vertical load (kN/m)	Length (m)	Max. utility Degree (%)	Num. of line supports	Remarks: This calculation is valid for
A	316	-158 to +158	90	1	88	2	The wall as a unit
B	100	-50 to + 50	78	1,732	50	4	Local buckling for middle wall sec-tions
C	100	-50 to + 50	78	0,866	61	3	Local buckling for end wall sections

Comments to the parameters:

- Thickness.
  A. Regarding the wall as a unit. From the spread sheet 316 mm is found as the equivalent thickness t_{ef}.
  B. (and C) Local buckling – local thickness
- Eccentricity interval
  The concrete slab is regarded casted in the full width of the wall
- Vertical load.
  A. In the overall direction of the wall N_{Ed} = 90 kN/m
  B. Along local walls: N_{Ed} = 90 kN/m × cos (30 °)
  = 78 kN/m
- Length and number of support

A. When only 2-side supported (top and bottom) the length is irrelevant
B. Middle section walls are regarded as supported on 4 sides according to the spread sheet output (given in Appendix 2). The length is the geometrically length (given in the spread sheet)
C. End sections walls are regarded as supported on 3 sides according to the spread sheet output (given in Appendix 2). The length is the geometrically length.

From the calculation given in Appendix 1 Φ_m is found to:

$$\Phi_m = 0{,}31$$

Giving the utility degree:

$$\begin{aligned} U &= \sigma /(\Phi_m \times f_d) \\ &= F/A /(\Phi_m \times f_d) \\ &= 90\times10^3/(346410/3) /0{,}31 \text{ x } 2{,}85 \\ &= 88\ \% \end{aligned}$$

Conclusion: The construction has adequate load capacity.

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Example 3:

Given a wall constructed as an arch with the geometry:

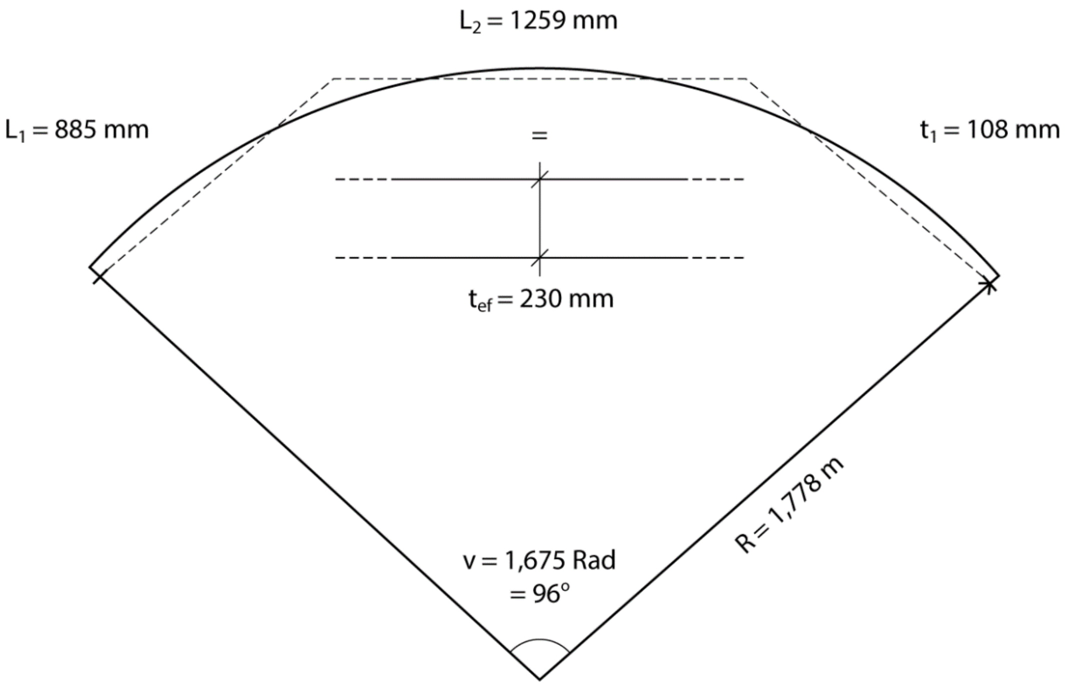

Figure 8. An arch as a load bearing wall

Using the features in the spread sheet n-profile it can be seen that an arch with the parameters:

R	1778	mm
v	1,675	rad

can be represented with the n-profile with the parameters:

L_1	885	mm
L_2	1259	mm
α	141	°

Using these values in combination with:

t_1	108	mm

we do get the effective thickness and the areas:

t_{ef}	230	mm

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A	327132	mm^2
A_{ef}	654760	mm^2

The arch can thus be regarded as a rectangular wall with the thickness of 230 mm (and the length of 2642 mm).

The 3 individual walls give mutual support and can be regarded as 3-sided and 4-sided supported with the thickness of 108 mm.

The load capacity is not determined. The purpose with this example is to show the procedure for arches.

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Calculations from EC6design.com using t_{ef}

The notation and symbols in the prints in this appendix can vary from the notations and symbols used in the paper.

Example 1

- Horizontal loaded rectangular wall

Danish Technological Institute Kongsvang Allé DK-8000 Aarhus C	Initials: pdc Date: 8/29/2013 Time: 10:39 AM
Project: Complex Cross-sections Component: Example 1	Number: 1355724-03 Module: Horizontal loaded rectangular wall / EC6design v.7.0

Input

The wall is made of: Masonry

Dimensions:

Length= 8.000 m Height= 8.000 m Thickness= 516 mm

Supporting conditions and moment on edges, if any, for the 4 edges of the wall:

Left vertical edge	: Restrained
Right vertical edge	: Simple; moment on the edge = 0 Nm/m
Lower horizontal edge	: Simple; moment on the edge = 0 Nm/m
Upper horizontal edge	: Simple; moment on the edge = 0 Nm/m

Form, placement and size of openings:

Form	Coordinates to the lower left corner of the opening		Width	Height
	x (m)	y (m)	(m)	(m)
quadrangle	3.000	3.000	3.000	3.000

Material parameters and loads:

Characteristic flexural strengths:	f_{xk1} = 0.20 MPa	f_{xk2} = 0.04 MPa
Consequence class= Normal	Inspection class= Normal	
Design horizontal load	w = 1.20 kN/m²	
(see also moment of edges, above)		
Design vertical load	n = 50.00 kN/m	

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Calculations

The area of the wall and the total horizontal load:	A	$= 64.0\ m^2$	W	$= 76.8\ kN$
Safety factors for parameters of strength			γ_c	$= 1.70$
Design flexural strength:	f_{xd1}	$= 0.12$ MPa	f_{xd2}	$= 0.02$ MPa
Design vertical yield moment	m_{vu}	$= f_{xd1}*t^2/6$		= 1044 Nm/m
Design horizontal yield moment:				
contribution from flexural strength	m_0	$= f_{xd1}*t^2/6$		= 5221 Nm/m
contribution from vertical load	m_1	$= n*t/6$		= 4300 Nm/m
Resulting horizontal yield moment			m_{lu}	= 9521 Nm/m

Result

Design load capacity (horizontal load) q_u = 1.29 kN/m²

based on the design yield moment m_{su} = 1044 Nm/m and m_{lu} = 9521 Nm/m

The horizontal load is w = 1.20 kN/m² The utilization ratio is $UR = w / q_u$ UR = 93 %

Conclusion: The utilization ratio is < 100 %: The load capacity is adequate.

Additional design

The wall is subject to vertical load and should (if the vertical load is dominant) be designed as a vertical loaded wall, subjected to an equivalent horizontal load

The supporting conditions and the actual reductions of the cross-section due to openings gives the lateral load qu the moment	m_{hu}	= 9521 Nm/m
For a wall with a simple vertical span and without openings the horizontal load qu will produce the simple moment:	$m_s = q_u * h^2/8$	= 10289 Nm/m
Factor of reduction k_a	$= m_{lu}/m_s =$ 9521/10289	= 0.93
Equivalent horizontal load = k_a * w	= 0.93 * 1.20 kN/m²	= 1.11 kN/m²

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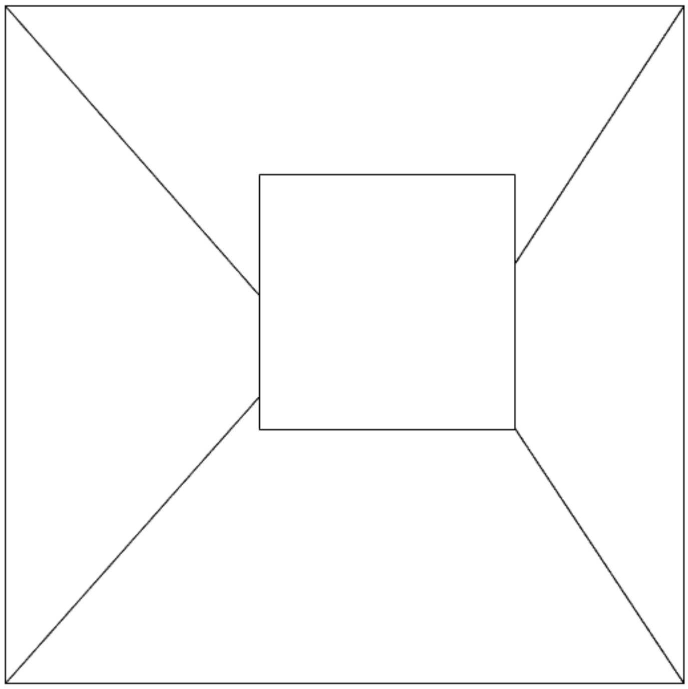

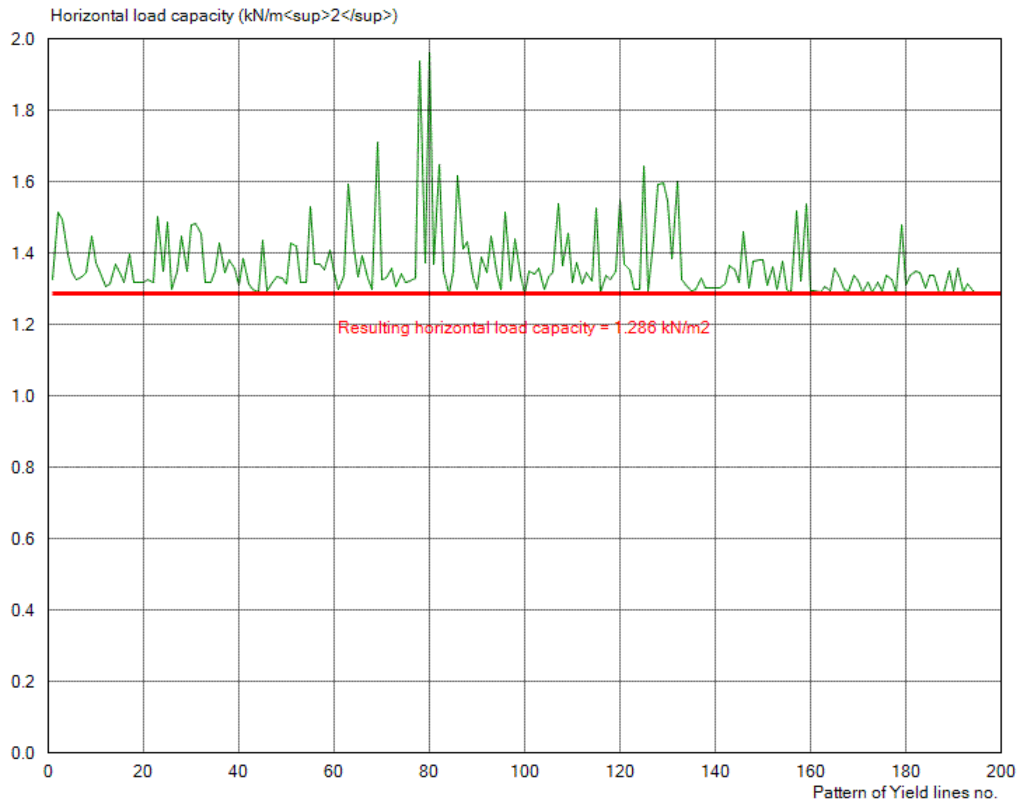

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Example 1

- Vertical loaded diaphragm wall

Danish Technological Institute Kongsvang Allé 8000 Århus C	Initials: pdc Date: 2/28/2014 Time: 10:11 AM
Project: Complex Cross-sections Component: Example 1	Number: 1355724-03 Module: Vertical loaded wall / EC6design v.7.01

Input

Dimension

Length= 3.000 m — Thickness= 516 mm — Height= 8.000 m

Thickness of outer leaf= 0 mm — (thickness=0 means no outer leaf)

Supporting conditions

Number of supports (2-4) = 3 — ρ_2 = 1.00

Parameters:

Compressive strength f_k = 4.56 MPa — Flexural strength f_{xk1} = 0.20 MPa

E-module E_{0k} = 3000 MPa — E-module (outer leaf, if any) = 3000 MPa

Density = 0 kg/m^3 — Consequence class= Normal — Inspection class = Normal

Design load on the load carrying wall

Vertical load= 75.0 kN/m — Horizontal load (wind load) = 1.11 kN/m^2 (positive leftwards)

Interval of eccentricity before correcting e_{init} in disfavour. Positive rightwards:

Top: from -30 to 30 mm; — Bottom: from -30 to 30 mm

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Calculations

Geometric conditions			
Area = 1.548 m^2	ρ_3 = 0.56	ρ_4 = 0.19	
Eff. height= 4469 mm	Eff. thickness= 516 mm	Ratio of slenderness = 8.7	
Initial eccentricity e_{init} = 10 mm	Creep eccentricity e_k = 0 mm		

Parameters	Charac. value	Safety fact.	Design value
Compressive strength	f_k = 4.56 MPa	1.60	f_d = 2.85 MPa
Flexural strength	f_{xk1} = 0.20 MPa	1.70	f_{xd1} = 0.12 MPa
E-module	E_{0k} = 3000 MPa	1.60	E_{0d} = 1875 MPa
Density		0.00000000 N/mm^3	

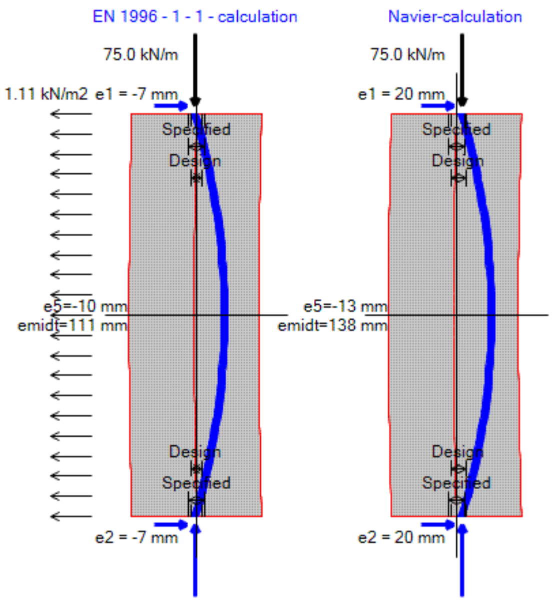
Capacity conditions	top	middle	bottom
Design normal force, N_{Ed}	75.0 N/mm	75.0 N/mm	75.0 N/mm
Minimal width of compression zone, N_{Ed} / f_{cd}	26 mm	26 mm	26 mm
Calculations			
The eccentricity of the arch of compression	-7 mm	111 mm	-7 mm
Deflection of the wall, e_5	-	-10 mm	-
Resulting eccentricity, e_{mr}	7 mm	121 mm	7 mm
(in the middle, at least 1/20 × thickness)			
Factor of reduction Φ (ref. EN 1996 - 1 - 1(6.4) og (G.1))	0.97	0.44	0.97
Design load capacity N_{Rd} (ref. EN 1996 - 1 - 1(6.2))	1431.2 N/mm	648.7 N/mm	1431.2 N/mm
Utilization ratio: N_{Ed} / N_{Rd}	5 %	12 %	5 %
Navier-calculations			
Eccentricity of the arch of compression	20 mm	138 mm	20 mm
Deflection of the wall, e_5	-	-13 mm	-
Resulting eccentricity, e_{mr}	20 mm	152 mm	20 mm
1. order moment $M_0 = N_{Ed} \times e$		11120 Nmm/mm	
Euler - load N_{cr}		3310.4 N/mm	
Factor of second order effect: $a = N_{cr} / (N_{cr} - N_{Ed})$		1.02	
Resulting moment $M_{max} = a \times M_0$		11377 Nmm/mm	
Section modulus Z		44376 mm^3/mm	
Flexural stress abs(M_{max}) / Z		0.256 MPa	
Normal stress N_{Ed} / thickness of wall		0.145 MPa	
Edge tension stress and -strength		0.111 MPa	0.118 MPa
Edge compression stress and -strength		0.402 MPa	2.850 MPa
Largest utilization ratio = Edge stress / strength		94 %	

Result

Utilizations ratios	5 %	12 %	5 %

Conclusion

One set of the utilizations ratios < 100 %. Thus the load capacity is adequate



The grey area is the wall. The blue area is the arc of compressio
symmetrically to the compression line and with the width NEd/fd

The curved red line through the middle is the axis of the wall.
The vertical black line is the "system" - line. The compression lin
is not indicated, but is in the middle of the zone of compression.

The design interval of eccentricities is shown at the top and botto
The zone of compression shall be inside these design interval o
eccentricities.

For the Navier-calculation is the eccentricity at the top in the mid
of the design interval of eccentricities and in the bottom 1/6 of th
of the design interval of eccentricities in disfavour of the construc

All loads are marked with black arrows, reactions with blue.
All horizontal dimensions are enlarged a factor 5 relative to verti

The calculation has given following result:
The load capacity is adequate.

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Example 2

- A. chevron wall, thickness=316 mm, 2-sided supported

Danish Technological Institute Kongsvang Allé DK-8000 Aarhus C	Initials: pdc Date: 8/29/2013 Time: 10:42 AM
Project: Complex Cross-sections Component: A: Zigzag wall. t=316 mm	Number: 1355724-03 Module: Vertical loaded wall / EC6design v.7.0

Input

Dimension

Length= 1.000 m Thickness= 316 mm Height= 8.000 m

Thickness of outer leaf= 0 mm (thickness=0 means no outer leaf)

Supporting conditions

Number of supports (2-4) = 2 ρ_2 = 1.00

Parameters:

Compressive strength f_k = 4.56 MPa Flexural strength f_{xk1} = 0.21 MPa

E-module E_{0k} = 3000 MPa E-module (outer leaf, if any) = 3000 MPa

Density = 1800 kg/m^3 Consequence class= Normal Inspection class = Normal

Design load on the load carrying wall

Vertical load= 90.0 kN/m Horizontal load (wind load) = 0.20 kN/m^2 (positive left-wards)

Interval of eccentricity before correcting e_{init} in disfavour. Positive rightwards:

Top: from -158 to 158 mm; Bottom: from -158 to 158 mm

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Calculations

Geometric conditions

Area = 0.316 m^2	ρ_3 = 0.30	ρ_4 = 0.06
Eff. height= 8000 mm	Eff. thickness= 316 mm	Ratio of slenderness = 25.3
Initial eccentricity e_{init} = 18 mm	Creep eccentricity e_k = 4 mm	

Parameters	Charac. value	Safety fact.	Design value
Compressive strength	f_k = 4.56 MPa	1.60	f_d = 2.85 MPa
Flexural strength	f_{xk1} = 0.21 MPa	1.70	f_{xd1} = 0.12 MPa
E-module	E_{0k} = 3000 MPa	1.60	E_{0d} = 1875 MPa
Density		0.00001765 N/mm^3	

Capacity conditions	top	middle	bottom
Design normal force, N_{Ed}	90.0 N/mm	112.3 N/mm	134.6 N/mm
Minimal width of compression zone, N_{Ed} / f_{cd}	32 mm	39 mm	47 mm
Calculations			
The eccentricity of the arch of compression	-19 mm	0 mm	-11 mm
Deflection of the wall, e_5	-	22 mm	-
Resulting eccentricity, e_{mr}	19 mm	22 mm	11 mm
(in the middle, at least 1/20 × thickness)			
Factor of reduction Φ (ref. EN 1996 - 1 - 1(6.4) og (G.1))	0.88	0.31	0.93
Design load capacity N_{Rd} (ref. EN 1996 - 1 - 1(6.2))	792.7 N/mm	283.2 N/mm	837.3 N/mm
Utilization ratio: N_{Ed} / N_{Rd}	11 %	40 %	16 %
Navier-calculations			
Eccentricity of the arch of compression	59 mm	73 mm	59 mm
Deflection of the wall, e_5	-	-33 mm	-
Resulting eccentricity, e_{mr}	59 mm	106 mm	59 mm
1. order moment $M_0 = N_{Ed} \times e$		10177 Nmm/mm	
Euler - load N_{cr}		760.3 N/mm	
Factor of second order effect: $\alpha = N_{cr} / (N_{cr} - N_{Ed})$		1.17	
Resulting moment $M_{max} = \alpha \times M_0$		11941 Nmm/mm	
Section modulus Z		16643 mm^3/mm	
Flexural stress abs(M_{max}) / Z		0.717 MPa	
Normal stress N_{Ed} / thickness of wall		0.355 MPa	
Edge tension stress and -strength		0.362 MPa	0.124 MPa
Edge compression stress and -strength		1.073 MPa	2.850 MPa
Largest utilization ratio = Edge stress / strength		293 %	

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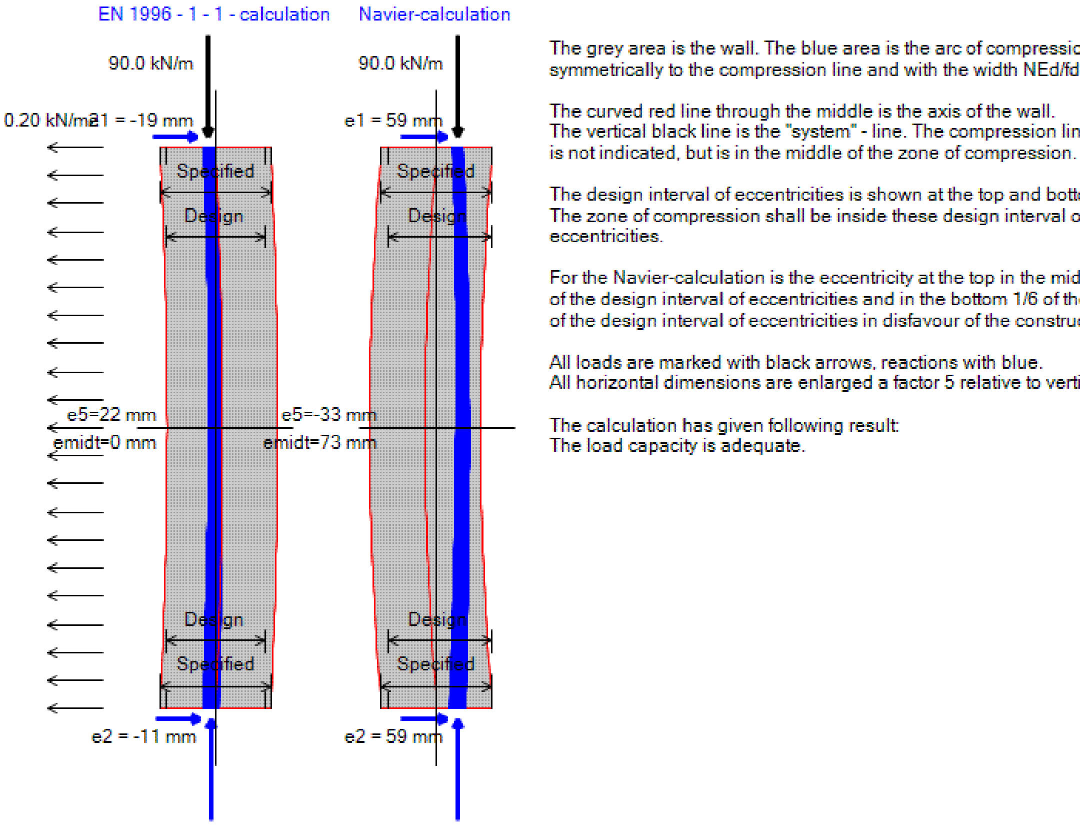
Result

Utilizations ratios	11 %	40 %	16 %

Conclusion

One set of the utilizations ratios $\leq$ 100 %. Thus the load capacity is adequate



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Example 2

- B. Local buckling of a 4-sided wall

Danish Technological Institute *Kongsvang Allé* *DK-8000 Aarhus C*	*Initials: pdc* *Date: 8/29/2013* *Time: 10:43 AM*
Project: Complex Cross-sections Component: B: Zigzag wall. t=100. 4 sid-ed	Number: 1355724-03 Module: Vertical loaded wall / EC6design v.7.0

Input

Dimension

Length= 1.732 m	Thickness= 100 mm	Height= 8.000 m
Thickness of outer leaf= 0 mm	(thickness=0 means no outer leaf)	

Supporting conditions

Number of supports (2-4) = 4	ρ_2 = 1.00

Parameters:

Compressive strength f_k = 4.56 MPa	Flexural strength f_{xk1} = 0.21 MPa	
E-module E_{0k} = 3000 MPa	E-module (outer leaf, if any) = 3000 MPa	
Density = 1800 kg/m^3	Consequence class= Normal	Inspection class = Normal

Design load on the load carrying wall

Vertical load= 78.0 kN/m	Horizontal load (wind load) = 0.20 kN/m^2 (positive left-wards)

Interval of eccentricity before correcting e_{init} in disfavour. Positive rightwards:

Top: from -50 to 50 mm; Bottom: from -50 to 50 mm

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Calculations

Geometric conditions

Area = 0.173 m^2 | ρ_3 = 0.32 | ρ_4 = 0.11

Eff. height= 866 mm | Eff. thickness= 100 mm | Ratio of slenderness = 8.7

Initial eccentricity e_{init} = 2 mm | Creep eccentricity e_k = 0 mm

Parameters	Charac. value	Safety fact.	Design value
Compressive strength	f_k = 4.56 MPa	1.60	f_d = 2.85 MPa
Flexural strength	f_{xk1} = 0.21 MPa	1.70	f_{xd1} = 0.12 MPa
E-module	E_{0k} = 3000 MPa	1.60	E_{0d} = 1875 MPa
Density		0.00001765 N/mm^3	

Capacity conditions	top	middle	bottom
Design normal force, N_{Ed}	78.0 N/mm	85.1 N/mm	92.1 N/mm
Minimal width of compression zone, N_{Ed} / f_{cd}	27 mm	30 mm	32 mm
Calculations			
The eccentricity of the arch of compression	-20 mm	0 mm	-18 mm
Deflection of the wall, e_5	-	5 mm	-
Resulting eccentricity, e_{mr}	20 mm	5 mm	18 mm
(in the middle, at least 1/20 × thickness)			
Factor of reduction Φ (ref. EN 1996 - 1 - 1(6.4) og (G.1))	0.60	0.83	0.65
Design load capacity N_{Rd} (ref. EN 1996 - 1 - 1(6.2))	170.1 N/mm	235.9 N/mm	184.3 N/mm
Utilization ratio: N_{Ed} / N_{Rd}	46 %	36 %	50 %
Navier-calculations			
Eccentricity of the arch of compression	17 mm	36 mm	17 mm
Deflection of the wall, e_5	-	51 mm	-
Resulting eccentricity, e_{mr}	17 mm	15 mm	17 mm
1. order moment $M_0 = N_{Ed} \times e$		3236 Nmm/mm	
Euler - load N_{cr}		24.1 N/mm	
Factor of second order effect: $a = N_{cr} / (N_{cr} - N_{Ed})$		-0.40	
Resulting moment $M_{max} = a \times M_0$		-1279 Nmm/mm	
Section modulus Z		1667 mm^3/mm	
Flexural stress abs(M_{max}) / Z		0.767 MPa	
Normal stress N_{Ed} / thickness of wall		0.851 MPa	
Edge tension stress and -strength		-0.083 MPa	0.124 MPa
Edge compression stress and -strength		1.618 MPa	2.850 MPa
Largest utilization ratio = Edge stress / strength		57 %	

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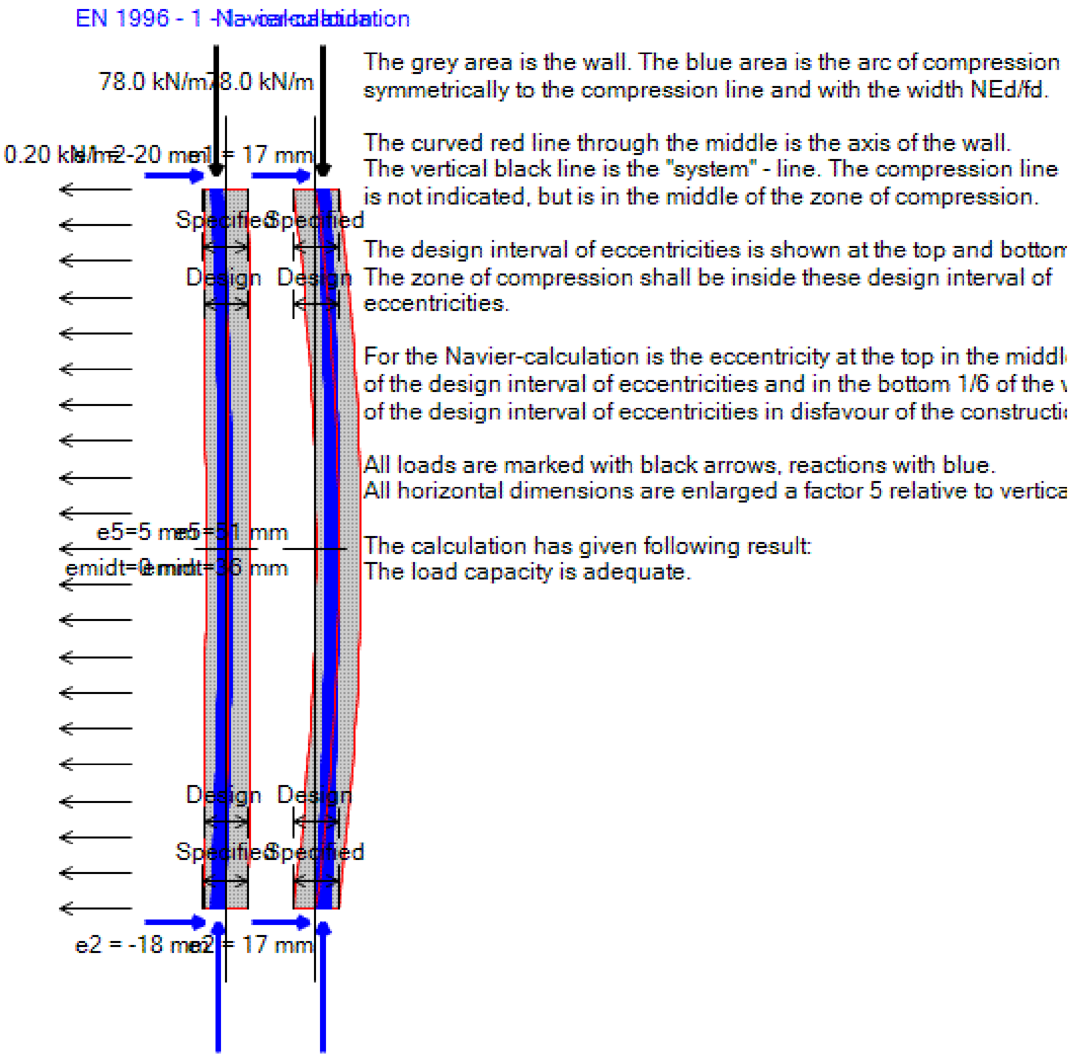
Result

Utilizations ratios	46 %	36 %	50 %

Conclusion

One set of the utilizations ratios ≤ 100 %. Thus the load capacity is adequate



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Example 2

- C. Local buckling of a 3-sided end wall

Danish Technological Institute Kongsvang Allé DK-8000 Aarhus C	Initials: pdc Date: 8/29/2013 Time: 10:44 AM
Project: Complex Cross-sections Component: C: Zigzag wall. t=100. 3 sided	Number: 1355724-03 Module: Vertical loaded wall / EC6design v.7.0

Input

Dimension

Length= 0.866 m   Thickness= 100 mm   Height= 8.000 m

Thickness of outer leaf= 0 mm   (thickness=0 means no outer leaf)

Supporting conditions

Number of supports (2-4) = 3 ρ_2 = 1.00

Parameters:

Compressive strength f_k = 4.56 MPa   Flexural strength f_{xk1} = 0.21 MPa

E-module E_{0k} = 3000 MPa   E-module (outer leaf, if any) = 3000 MPa

Density = 1800 kg/m^3 Consequence class= Normal   Inspection class = Normal

Design load on the load carrying wall

Vertical load= 78.0 kN/m   Horizontal load (wind load) = 0.20 kN/m^2 (positive leftwards)

Interval of eccentricity before correcting e_{init} in disfavour. Positive rightwards:

Top: from -50 to 50 mm;   Bottom: from -50 to 50 mm

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Calculations

Geometric conditions

Area = 0.087 m^2	ρ_3 = 0.30	ρ_4 = 0.05
Eff. height= 2400 mm	Eff. thickness= 100 mm	Ratio of slenderness = 24.0
Initial eccentricity e_{init} = 5 mm	Creep eccentricity e_k = 1 mm	

Parameters	Charac. value	Safety fact.	Design value
Compressive strength	f_k = 4.56 MPa	1.60	f_d = 2.74 MPa
Flexural strength	f_{xk1} = 0.21 MPa	1.70	f_{xd1} = 0.12 MPa
E-module	E_{0k} = 3000 MPa	1.60	E_{0d} = 1800 MPa
Density		0.00001765 N/mm^3	

Capacity conditions	top	middle	bottom
Design normal force, N_{Ed}	78.0 N/mm	85.1 N/mm	92.1 N/mm
Minimal width of compression zone, N_{Ed} / f_{cd}	29 mm	31 mm	34 mm
Calculations			
The eccentricity of the arch of compression	-20 mm	0 mm	-18 mm
Deflection of the wall, e_5	-	6 mm	-
Resulting eccentricity, e_{mr}	20 mm	6 mm	18 mm
(in the middle, at least 1/20 × thickness)			
Factor of reduction Φ (ref. EN 1996 - 1 - 1(6.4) og (G.1))	0.60	0.36	0.65
Design load capacity N_{Rd} (ref. EN 1996 - 1 - 1(6.2))	163.0 N/mm	98.0 N/mm	177.1 N/mm
Utilization ratio: N_{Ed} / N_{Rd}	48 %	87 %	52 %
Navier-calculations			
Eccentricity of the arch of compression	18 mm	37 mm	18 mm
Deflection of the wall, e_5	-	53 mm	-
Resulting eccentricity, e_{mr}	18 mm	16 mm	18 mm
1. order moment M_0 = N_{Ed} × e		3623 Nmm/mm	
Euler - load N_{cr}		23.1 N/mm	
Factor of second order effect: a = N_{cr} / (N_{cr} - N_{Ed})		-0.37	
Resulting moment M_{max} = a × M_0		-1353 Nmm/mm	
Section modulus Z		1667 mm^3/mm	
Flexural stress abs(M_{max}) / Z		0.812 MPa	
Normal stress N_{Ed} / thickness of wall		0.851 MPa	
Edge tension stress and -strength		-0.039 MPa	0.119 MPa
Edge compression stress and -strength		1.662 MPa	2.735 MPa
Largest utilization ratio = Edge stress / strength		61 %	

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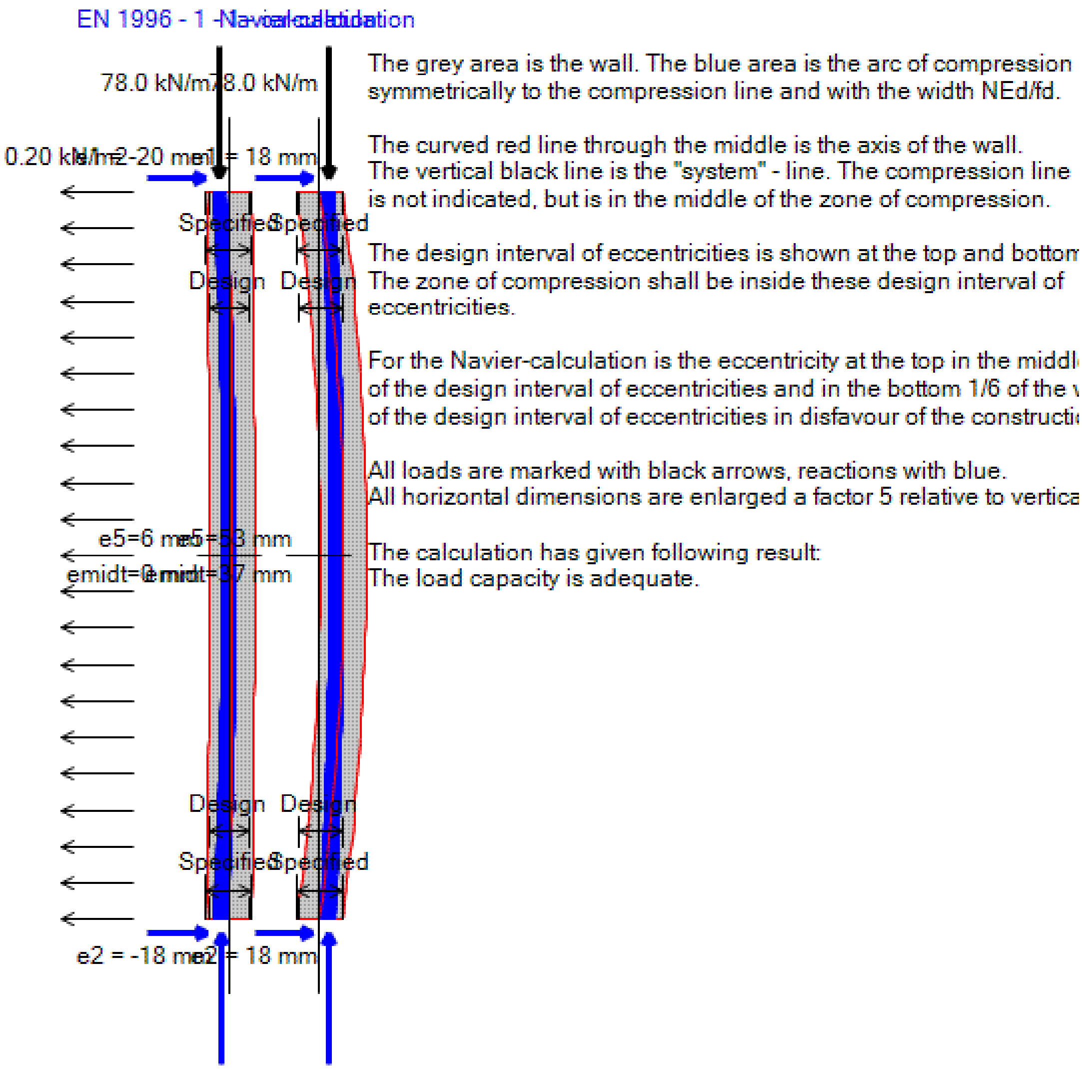
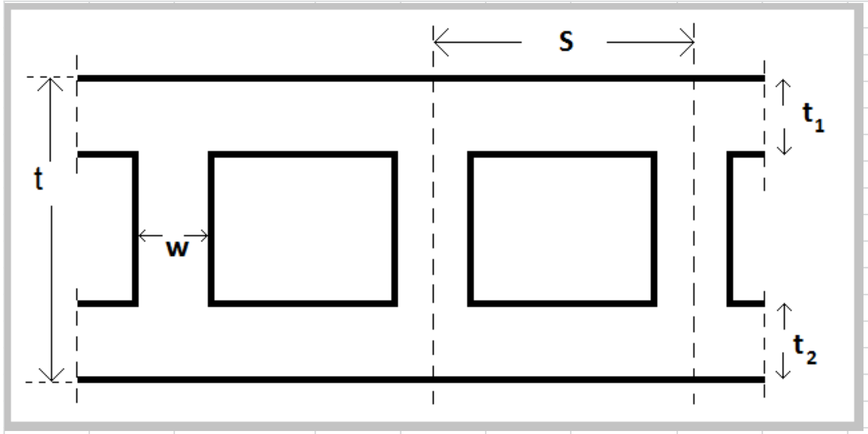
Result

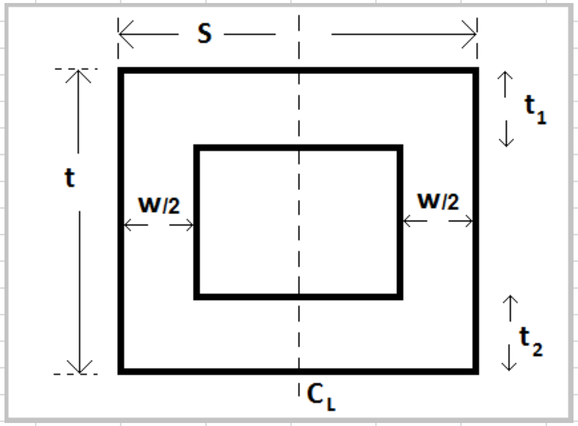
Utilizations ratios	48 %	61 %	52 %

Conclusion

One set of the utilizations ratios $\leq$ 100 %. Thus the load capacity is adequate







Input			**Output**			**For**
S	1200	mm	t_{ef}	516	mm	Mainly lateral loading
t	588	mm	t_{ef}	516	mm	Mainly vertical loading
t_1	108	mm	A	299376	mm^2	
t_2	108	mm	A_{ef}	618989	mm^2	
w	108	mm				

Intermediate calculation				
t_{body}	372	mm		
A	299376	mm^2		
S_ξ	88016544	mm^3		
η_0	294	mm		
$I_{flange,1}$	7590931200	mm^4		
I_{body}	463309632	mm^4		
$I_{flange,2}$	7590931200	mm^4		
I_{total}	15645172032	mm^4	$t_{ef,I}$	539
Z_1	53214871	mm^3	$t_{ef,Z1}$	516
Z_2	53214871	mm^3	$t_{ef,Z2}$	516

Example 1. Spread sheet HO-profiles

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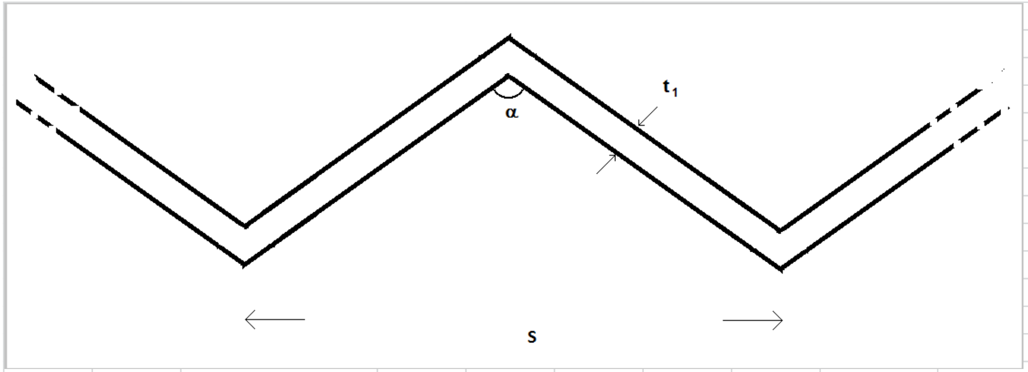

Input			Output			For
S	3000	mm	t_{ef}	316	mm	Mainly lateral loading
α	120	°	t_{ef}	316	mm	Mainly vertical loading
t_1	100	mm	A	346410	mm^2	
			A_{ef}	948683	mm^2	

Intermediate calculation						Stiffening walls - active:		
α	2,094		pi	3,141593		β	0,5236	
$t_{horizontal}$	200	mm				$t_{equivalent}$	115	mm
t	866	mm				$L_{equivalent}$	693	mm
$L_{one\ flange}$	1732	mm				$I_{stiffening\ wall}$	3199999971	mm^4
A	346410	mm^2				$I_{regarded\ wall}$	144337566	mm^4
S_ξ	149999993	mm^3				$I_{stiff.}/I_{regard.}$	22,2	
η_0	433	mm				3 or 4 -sided	**Yes**	
I_{total}	21650633354	mm^4	$t_{ef,l}$	442	mm			
Z_1	49999998	mm^3	$t_{ef,Z1}$	316	mm			
Z_2	49999998	mm^3	$t_{ef,Z2}$	316	mm			

Slender constructions should allways be arranged with the stiffening walls active. E.g. For S=6000 mm and t_1 = 100 mm the angle α < 154 °

Local buckling shall always be evaluated, e.g. according to EN 1996-1-1 (5.6 - 5.9), 5.5.1.2 (4), etc.

Compact constructions can not allways be arranged with the stiffening walls active. E.g. For S= 450 mm, t_1 = 100 mm and α = 90 ° the stiffening wall is not active. This only means the compact construction is acting in unison.

Example 2. Spread sheet W-profiles

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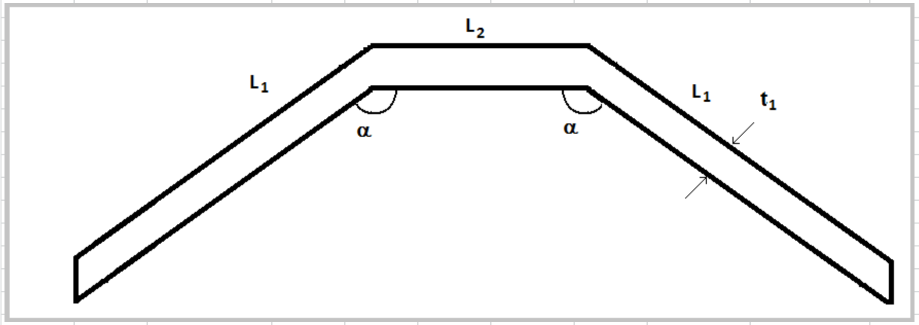

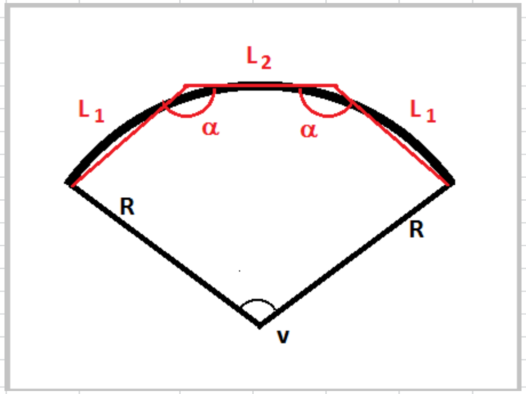

Input			Output			For
L_1	885	mm	t_{ef}	230	mm	Mainly lateral loading
L_2	1259	mm	t_{ef}	230	mm	Mainly vertical loading
$\alpha \geq 90$	141	°	S	2851	mm	
t_1	108	mm	A	327132	mm^2	
			A_{ef}	654760	mm^2	

Intermediate calculation

α	2,461	rad	pi	3,141593	
$t_{1,horizontal}$	172	mm			
t	557	mm			
A	327132	mm^2			
S_ξ	53233142	mm^3			
η_0	163	mm			
I_{total}	11235120177	mm^4	$t_{ef,I}$	362	mm
Z_1	51840014	mm^3	$t_{ef,Z1}$	330	mm
Z_2	25065986	mm^3	$t_{ef,Z2}$	230	mm

Stiffening walls - active:		
β	0,8901	
$t_{equivalent}$	172	mm
$L_{equivalent}$	396	mm
$I_{stiffening\ wall}$	889146492	mm^4
$I_{regarded\ wall}$	132164784	mm^4
$I_{stiff.}/I_{regard.}$	6,7	
3 or 4 -sided	**Yes**	

Slender constructions should allways be arranged with the stiffening walls active. E.g. For L_1 = 1200, L_2 = 800, t_1 = 100 mm and the angle α < 131 °

Local buckling shall always be evaluated, e.g. according to EN 1996-1-1 (5.6 - 5.9), 5.5.1.2 (4), etc.

Compact constructions can not allways be arranged with the stiffening walls active. E.g. For L_1 = 600, L_2 = 300, t_1 = 150 mm and α = 135° the stiffening wall is not active. This only means the compact construction is acting in unison.

Input			Output		
R	1778	mm	L_1	885	mm
v	1,675	rad	L_2	1259	mm
			α	141	°

Intermediate calculation

v(gr)	96,0	gr
Pi	3,1415927	
k	2642	mm
h	588	mm
k/h	4,49	
x1	692	mm
y1	551,4	mm
α'	0,67	rad
α	2,47	rad

An circular arc can be represented by the n - profile. See figure.
Input/output above gives a proposal for L_1, L_2 and α based on R and v.

When representing the arc through the n-profile locale buckling and local load capacity can "easily" be found through the formulas given in EN 1996-1-1.

Example 3. Spread sheet n-profiles