STRESSED SKIN EFFECT ON THE STABILITY OF STEEL STRUCTURES

HAMK
HÄMEEN AMMATTIKORKEAKOULU
HÄME UNIVERSITY OF APPLIED SCIENCES

Ylemmän ammattikorkeakoulututkinnon opinnäytetyö

Visamäki, Rakentaminen

Kevät, 2020

Mei Wang

ABSTRACT

Degree program in construction and environmental engineering
Visamäki

Author	Mei Wang	Year 2020
Subject	Stressed skin effect on the stability of steel structures	
Supervisors	Risto Nurminen; Zhongcheng Ma	

ABSTRACT

This thesis was commissioned by Sweco Rakennetekniikka Oy.

Single story steel structure with steel roofing made of corrugated steel sheet has been widely used in the industrial building. Corrugated steel roof is often regarded as retaining structure which only takes the vertical loads. However, the corrugated steel sheet has the significant capacity of in plane shear resistance and it can receive a lot of horizontal load through the strong enough connections to building steel structure and transfer the loads to end gables of the building. This phenomenon is called stressed skin effect or diaphragm effect. It could be used to improve the overall stiffness of the main structure and decrease the horizontal deformation of columns at roof level.

In this master's thesis, the author uses the one span and one-story building steel frame with flat roof structure as an example for the analysis. The basic theory and calculation formulas for the stressed skin are presented for different types of the roof systems. The Excel calculation was proposed as a simple design method to figure out the equivalent structure for description for the roof diaphragm, which can be easily conducted in the global FEM structural analysis. Through the comparative analysis of different ways of modelling, we could get the following conclusions: The stressed skin effect of the roof can be simplified into the equivalent horizontal truss structure made of steel or spring members or surface member. All the methods work properly, and the designer of the building steel structure can choose the suitable way to perform structural analysis. Comparison of 3D analysis models of steel structure with or without stressed skin effect shows that by utilizing of stressed skin effect, the roof sheet can replace the often used horizontal roof truss and that it also may decrease the displacements and reduce shear force of the main columns under the same performance.

Keywords Stressed skin diaphragms; Corrugated steel sheet; Global stability; shear flexibility
Pages 83 pages including appendices 17 pages

TIIVISTELMÄ

Rakentaminen
Visamäki

Tekijä	Mei Wang	Vuosi 2020
Työn nimi	Levyjäykistyksen vaikutus rakenteiden vakauteen	
Työn ohjaajat	Risto Nurminen; Zhongcheng Ma	

TIIVISTELMÄ

Tämän opinnäytetyön on tilannut Sweco Rakennetekniikka Oy.

Teollisuuden yksikerroksisissa teräsrunkoisissa rakennuksissa käytetään usein teräsohutlevystä tehtyä profiilipeltikattoa. Suunnitteluvaiheessa profiilipeltikattoa käsitellään usein vain katon pystykuormia kantavan rakenteena. Muotolevyllä on kuitenkin suuri leikkauskestävyys katon tasossa ja se kestää merkittäviä vaakatasoisia kuormia, joka välitetään teräsrakenteesta riittävän vahvojen kiinnikkeiden avulla, ja voi siirtää kuormituksen rakennuksen päätyyn. Tätä ilmiötä kutsutaan nimellä profiilipellin levyvaikutus. Kattorakenteen levyvaikutusta voidaan käyttää parantamaan rakennuksen kokonaisjäykkyyttä ja pienentämään vaakasuuntaisia siirtymiä katon tasossa.

Tässä opinnäytetyössä analysoitava rakenne on yksikerroksinen tasakattoinen teräsrakenteinen rakennus. Levyvaikutuksen toiminta ja laskentakaavat esitetään erityyppisille profiilipellillä toteutettaville kattojärjestelmille. On myös laadittu Excel-laskentapohja, jota on ehdotettu yksinkertaistetuksi suunnittelumenettelyksi, kun määritellään levyvaikutusta vastaava rakenne, joka voidaan helposti luoda globaalissa FEM-laskentamallissa. Vertailevien analyysin kautta voidaan tehdä seuraavat johtopäätökset: Katon levyvaikutus voidaan yksinkertaistaa vastaavaksi vaakaristikkorakenteeksi, jossa on terässauvat tai jousielementit, tai levyvaikutus voidaan kuvata kalvorakenteena, jolla on levyvaikutuksen jäykkyysominaisuudet. Kaikki menetelmät toimivat riittävän hyvin ja teräsrakenteen suunnittelija voivat valita kohteeseen sopivan tavan. Vertaamalla 3D analyysin avulla teräsrunkorakeita levyvaikutuksen kanssa ja ilman, käy ilmi, että profiilipellin levyvaikutus voi korvata katon ristikon ja vähentää vaaka siirtymää ja pääsarakkeiden leikkausvoimaa samalla suorituskyvyllä.

Avainsanat Levyjäykistys; Muotolevy; Globaali teräksen vakaus; Leikkauskestävyys.

Sivut 83 sivua, joista liitteitä 17 sivua

Table of Contents

Foreword

This Master's thesis has been ordered by Sweco Rakenneteknikka Oy. I want to thank to Risto Nurminen and Zhongcheng Ma who were involved in this project. Both are extremely busy in their own work. I appreciated for all the valuable time and guidance I've received from them so that this thesis work can be accomplished smoothly.

Lastly, I would like to thank my family who have supported me throughout my studies and during the thesis. Your encouragement and assistance have made my studies a lot more pleasant.

Helsinki 13.5.2020

Mei Wang

Symbols and definitions

a	Length of diaphragm in the direction perpendicular to the corrugations - [mm].
A	Cross-sectional area of longitudinal edge member [mm2].
b	Depth of diaphragm in the direction parallel to the corrugations [mm].
c	Overall shear flexibility of diaphragm [mm/kN].
$c_{1.1}$	Flexibility due to distortion of the corrugation [mm/kN].
$c_{1.2}$	Flexibility due to shear strain in the sheeting[mm/kN].
$c_{2.1}$	Flexibility due to movement in sheet to purlin fasteners[mm/kN].
$c_{2.2}$	Flexibility due to movement in seam fasteners[mm/kN].
$c_{2.3}$	Flexibility due to movement in shear connectors[mm/kN].
c_3	Flexibility due to axial strain in the purlins [mm/kN].
d	Pitch of corrugations [mm].
Dx, Dy	Orthogonal bending stiffness of profiled sheet per unit length perpendicular and parallel to the corrugation respectively [kN mm2/mm].
E	Modulus of elasticity of steel [kN/m2].
F_p	Design strength of an individual sheet to perpendicular member fastener[kN].
F_{pr}	Design strength of an individual purlin to rafter fastener[kN].
F_s	Design strength of an individual seam fastener [kN].
F_{sc}	Design shear resistance of an individual sheet to shear connector fastener [kN].
K1, K2	Sheeting constants for distortional flexibility.
h	Height of profile [mm].
k	Flexibility of main frame [mm/kN].
l	Width of corrugation crest [mm].
n	Number of panels within length of diaphragm assembly.
n_f	Number of sheets to perpendicular member fasteners per sheet width.
n_s	Total number of seam fasteners per side lap.
n_{sc}	Total number of shear connector fasteners in line.
n_{sh}	Number of sheet width per panel.
n_p	Total number of perpendicular members (purlins) within the depth of the panel.
L	Span of diaphragm between[mm]
P	Point load on the diaphragm [kN].
p	Pitch of the sheet/purlin fasteners [mm].
q	Uniformly distributed load on the diaphragm [kN/m].
s_p	Slip(flexibility) per sheet to purlin (or edge beam) fastener per unit load[mm/kN].
s_s	Slip(flexibility) per seam fastener per unit load[mm/kN].
s_{sc}	Slip(flexibility) per sheet to shear connector fastener per unit load[mm/kN].
s_{pr}	Movement of perpendicular member to parallel member connection per unit load[mm/kN]
t	Net sheet thickness excluding galvanizing and coatings[mm]
ϑ	Deflection of an individual diaphragm [mm]
V	Shear load on an individual diaphragm [kN].

$V_{cr,g}$	Design value of the gobal shear buckling strength of the diaphragm[kN].
$V_{cr,l}$	Design value of the local shear buckling strength of the diaphragm[kN].
V_{Rd}	Design shear capacity of the diaphragm [kN].
V_{red}	Design value of the reduced shear buckling strength of the diaphragm under combined local and global buckling[kN].
u	Perimeter length of a single corrugation [mm]
$\alpha_1, \alpha_2, \alpha_3$	Non-dimensional factors.
α_4, α_5	Non-dimensional factors.
$\beta_1, \beta_2, \beta_3$	Non-dimensional factors.
ϑ	Deflection of an individual diaphragm [mm]
Δ	Mid-span deflection of a diaphragm beam [mm].
$F_{v,Ed}$	Shear force in the plane[kNm].
$F_{t,Ed}$	Pull-out force[kNm].
$F_{b,Rd}$	Resistance with respect to bearing failure[kN]
$F_{p,Rd}$	Resistance with respect to pull-through failure[kN].
$F_{v,Rd}$	Resistance with respect to shear failure[kN].
$F_{o,Rd}$	Resistance with respect to pull-out failure[kN].
$M_{s,Ed}$	Support moment at the intermediate support due to usual transverse action[kNm/mm].
$M_{s,Rd}$	Resistance with respect to support moment[kNm/mm].
$M_{f,Ed}$	Moment in span due to self weight and usual snow load[kNm/mm].
$M_{f,Rd}$	Resistance with respect to moment in the span[kNm/mm].
$R_{e,Ed}$	Reaction at the end support due to transverse action[kN/m].
$R_{es,Rd}$	Resistance with respect to the end support reaction in accordance with EN 1993-1-3[kNm/mm].
$R_{s,Ed}$	Support reaction at the intermediate support due to usual transverse action[kNm/mm].
$R_{s,Rd}$	Resistance with respect to support reaction [kNm/mm].
$R_{V,Ed}$	Support reaction caused by the shear force[kN/m].
V_{Ed}	Maximum shear flow[kN/m].
$V_{g,Rd}$	Design value of shear force resistance with respect to global buckling[kN/m].
$V_{w,Rd}$	Design value of shear force resistance with respect to web buckling [kN/m].
$V_{f,Rd}$	Design value of shear force resistance with respect to flange buckling[kN/m].
$V_{r,Rd}$	Design value of shear force resistance with respect to flexure of the profile corner[kN/m].

List of abbreviations

2D	Two-dimensional
3D	Three-dimensional
FEM	Finite element method
RFEM	3D finite element analysis software developed by Dlubal Software.

1 INTRODUCTION

1.1 Background

Stressed skin effect was original discovered in structural engineering in the early 1950s. The factory and warehouse buildings without intermediate floor were studied at that time. Studies indicated that the walls and roof structures have some capacity to stabilize the whole building and the lateral deflection of the whole roof is significantly decreased due to stiffness of steel sheeting. This effect is called stressed skin effect or diaphragm effect. However, this capacity is often disregarded due to difficulties in performing structural analysis in the design phase and lack of relevant instructions.

In 1982, Bryan & Davies have published a manual regarding the theoretical background of the stressed skin effect, and this was used as a design guideline. Also, a remarkable progress took place in 1995, because at that time the design document European Recommendations for the Application of Metal Sheeting acting as Diaphragm (ECCS) was published. Furthermore, the book named by Stabilization by Stressed Skin Diaphragms Action was written by Torsten Höglund in 2002 and design of the metal sheets and fasteners used for stressed skin were discussed in detail in this book.

The corrugated steel panels are often used as roof structure in the industrial building, sport centers and schools as shown in Figure 1.

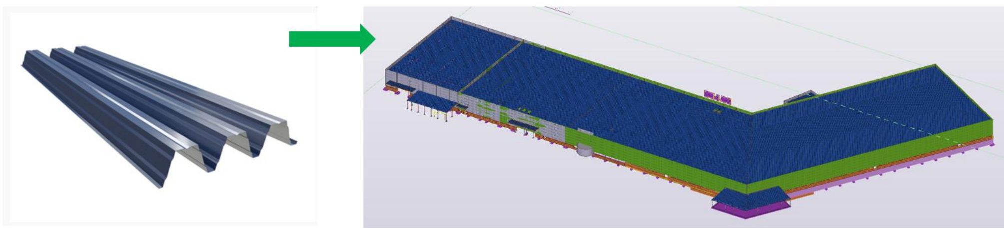


Figure 1. Ruukki Corrugated steel panel is used in Project: Ruukki Birsta Handel (Sweden)

However, the idea of using corrugated steel sheet as stabilization of the steel structure is not popularly applied in Finland. Instead, it is a traditional practice in Finland that the lateral wind bracing system is used to transfer the horizontal force to vertical stabilizing structure at end of the building and the shear capacity of the sheeting structure is not considered at all. Figure 2 represents corrugated steel sheeting of one sport centers in Helsinki. The lateral bracing on the roof has been designed to keep the main truss structure stable, which is proved to be the traditional design approach.

Nowadays, a large number of recent researches indicate that the diaphragm action of the metal sheet can be used to replace the lateral wind bracing system as shown in Figure 3. This Thesis will figure out and how the corrugated sheet acting as stressed skin influence the behavior of the overall steel structure of the building and what is the equivalent steel truss structure of the diaphragm structure so that the stressed skin effect could be correctly taken into consideration in common design procedure.

Figure 2. Roof structure of one sport center in Helsinki.

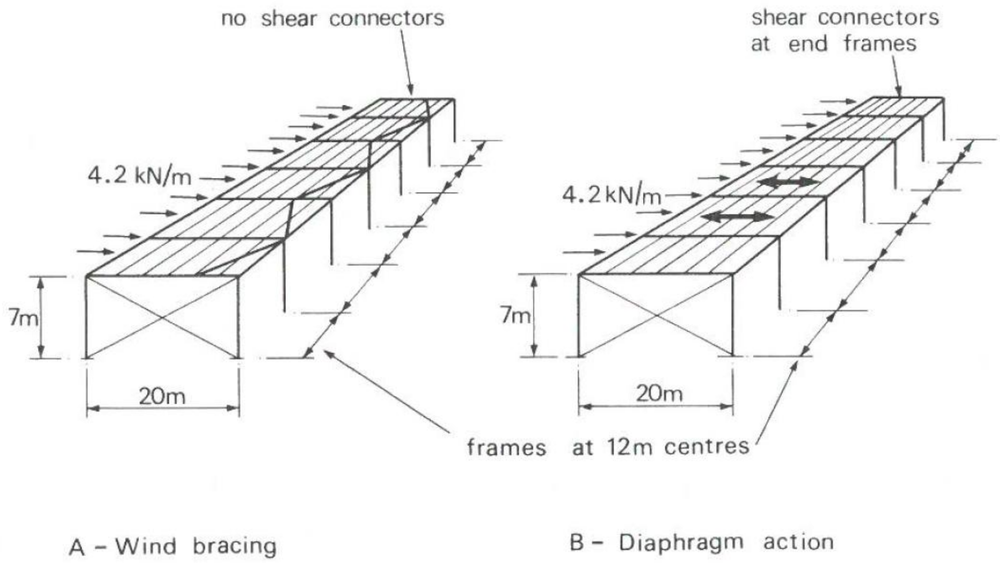
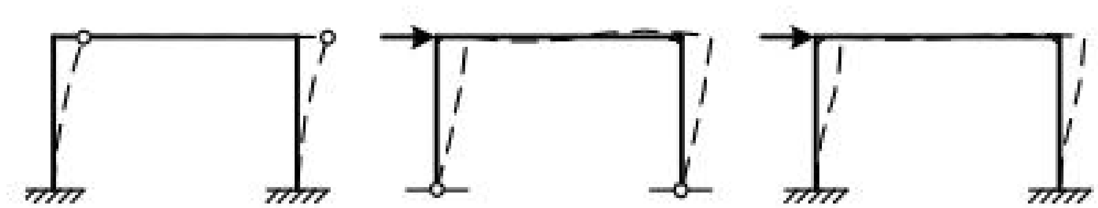


Figure 3. Structure comparison of lateral wind bracing and stresses skin action (J.M. Davies and E.R. Bryan, 1982, P.11)

1.2 Objectives and scope

The primary objective of this master’s thesis is to investigate the stabilization of a simple building steel frame stiffened by corrugated steel sheet and develop the practical design method to consider corrugated sheet as part of stabilizing system of the building steel structure. The studied structure type is the low-rise single-story flat roof structure which is commonly used in the industrial and agricultural building. Basic frame types for this kind of buildings are shown in Figure 4. This thesis gives the comparison of RFEM analysis results for each three types in Chapter 7.



(a) Columns fixed frame (b) Portal frame (c) Rigid frame

Figure 4. Types of load bearing systems (Torsten Höglund,2002,P.1)

The general theory of the stressed skin effect is introduced in this thesis. The diaphragm strength and flexibility of the stressed skin roof sheeting structure are further calculated. An Excel calculation of the shear flexibility of the corrugated roof panel including flexibility of screw fastenings has been developed, so that the designer of structural steel could take the advantage of the stressed skin design and in simple way create a structural model for global analysis of building steel structure during normal design work.

The approach of how to take the stressed skin effect into account in the design work is proposed.

1.3 Method of investigation

To use and utilize stressed skin effect of roof sheeting in real design projects there is a need for a simple enough and easy to use design tool to calculate stiffness and shear strength properties of roof sheeting. An excel calculation sheet to solve this problem has been developed during this thesis investigation.

- Calculation method of stiffness for the stressed skin diaphragm when roof sheeting type and fastenings of the sheet has been specified
- The properties of the equivalent truss members (or spring) for the horizontal truss of same dimensions as stressed skin diaphragm is calculated.
- Another simple calculation method to define an equivalent horizontal truss made of steel tube members and with selected height of truss to describe stiffness properties of corrugated sheet acting as stressed skin.

Three design examples from reference books have been performed by the Excel.

Finite element method (FEM) analysis is performed to verify the results of the Excel tool and the design examples. The finite element analysis program RFEM is used as structural analysis software in this thesis

Thirdly, a parameter study regarding the factors to affect the stressed skin effect is carried out. Three factors such as sheet fastened in every or alternates corrugation, thickness of the sheet, purlin or edge beam are investigated for the parameter study.

1.4 Outline of the Thesis

There are 8 chapters in this thesis and the content is presented as follows.

Chapter 1 introduces the objectives and method of this thesis work.

Chapter 2 presents the literature review regarding the principles of diaphragm action.

Chapter 3 focuses on how to determine the shear strength and shear stiffness of the stressed skin diaphragm.

Chapter 4 explains the interaction between the main frame and the roof diaphragm and the different RFEM modelling approaches.

Chapter 5 presents the design checking list for corrugated steel sheet and relevant fixings.

Chapter 6 proposes the simplified design method and introduce the Excel design tool.

Chapter 7 compares the analysis results between Excel and RFEM analysis models. The parameter study based on Excel is discussed. Furthermore, 3D RFEM models for different load-bearing systems are compared.

Finally, chapter 8 gives the conclusions and recommendations for the stressed skin design and further studies in the future.

2 PRINCIPLES OF STRESSED SKIN DESIGN

There are following introductions and regulations regarding the stressed skin design from Eurocode 3 Design of steel structures - Part 1-3: General rules - Supplementary rules for cold-formed members and sheeting. (SFS-EN 1993-1-3, P.95-96)

In stressed skin design, advantage may be taken of the contribution that diaphragms of sheeting used as roofing, flooring or wall cladding make to the overall stiffness and strength of the structural frame, by means of their stiffness and strength in shear.

Roofs may be treated as deep plate girders extending throughout the length of a building, resisting transverse in-plane loads and transmitting them to end gables of the building, or to intermediate stiffened frames inside building. The panel of sheeting may be treated as a web that resists in-plane transverse loads in shear, with the edge members acting as flanges that resist axial tension and compression forces. It can be seen from Figure 5, how the loads can be transferred to building gable at level of the roof.

Similarly, rectangular wall panels may be treated as bracing systems that act as shear diaphragms to resist in-plane forces.

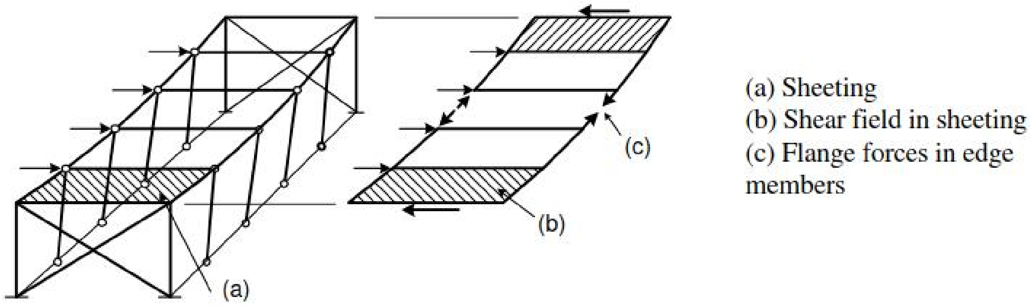


Figure 5. Stressed skin action in a flat-roof building (SFS-EN 1993-1-3)

2.1 Types of diaphragms

There are two principal types of diaphragms as shown in Figure 6. The internal forces in the roof diaphragm are listed in the Table 1. (Torsten Höglund,2002, P.8).

- Corrugated sheets directly on main beams
- Corrugated sheets on purlins

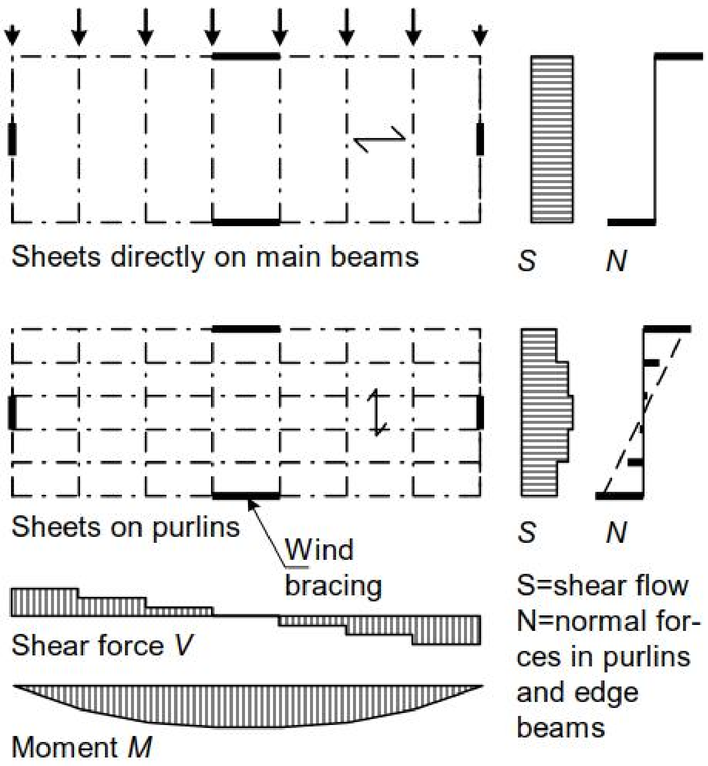


Figure 6. Diaphragm types (Torsten Höglund,2002, P.8)

Type of the forces	Sheets directly on main beams	Sheets on purlins
Shear flow	$S = \frac{V}{b} = \frac{qL}{2b} - \frac{qa}{2b}$	$S \approx 1{,}15\frac{V}{b}$
Normal forces in purlins and edge beams	$N = \frac{M}{b} = \frac{qL^2}{8b}$	$N \approx 0{,}58\frac{M}{b}$
Reaction force on both sides of the building	$V = \frac{qL}{2}$	$V = \frac{qL}{2}$

Table 1. The internal force in the roof diaphragm

q= uniformly distributed load on the diaphragm[kN/m].
L= length of the diaphragm[m].
a= the distance between the rafters[m].
b= the length of diaphragm[m].

2.2 The basic arrangement of the roof diaphragm

The basic roof diaphragm was shown in Figure 7, which consists of trapezoidal sheeting, purlins and edge beams, rafters (main beam or truss of the building), and the connections between these.

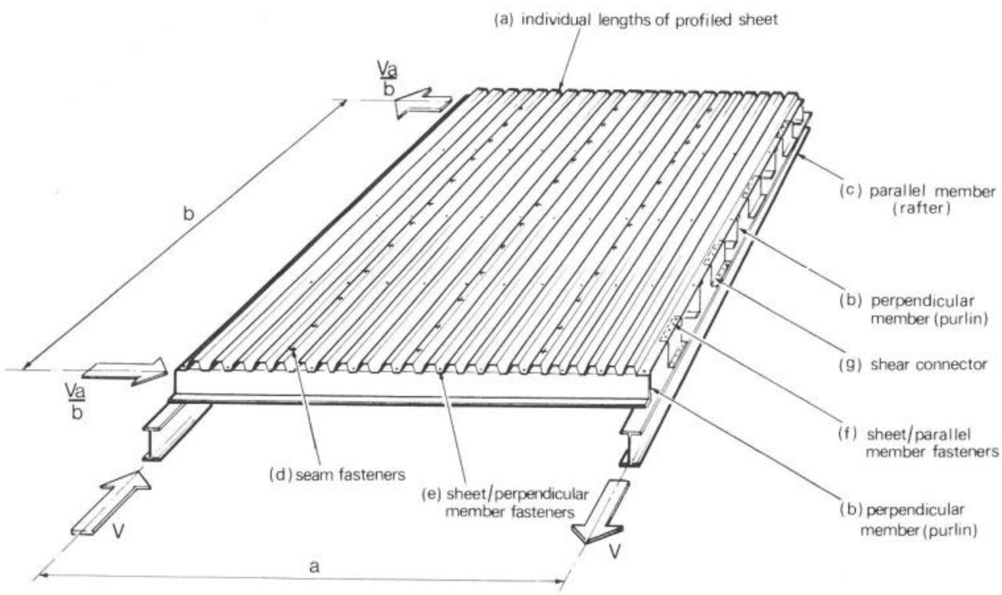


Figure 7. Basic arrangement of an individual diaphragm (J.M. Davies and E.R. Bryan, 1982, P.19)

(a) Individual lengths of profiled steel or aluminum sheeting.
(b) Perpendicular members (Purlin)
(c) Parallel members (Rafters)
(d) Seam fasteners
(e) Sheet to perpendicular member fasteners.
(f) Sheet to parallel member fasters
(g) Shear connectors
(h) Connections between perpendicular and parallel members (Purlin to rafter connections.

Different fastening arrangement can be designed for the diaphragm as shown in Figure 8. Case (1) and (2) require that the tops of the members are at the same level or that shear connectors (additional purlin on top of rafter) are used, allowing four sides of the shear panel to be fastened. Case (3) and (4) occur when the tops of the member are at different levels so that only two sides of the shear panel can be fastened. In case (3), shear connectors must be used at the end rafters. Case (4) is not normally recommended because there are no connectors to the edge member. (ECCS,1995)

This thesis only focuses on Case (1) and (2) and the excel calculations has been prepared only for case (1) and (2), which are 4 sides fastened situations.

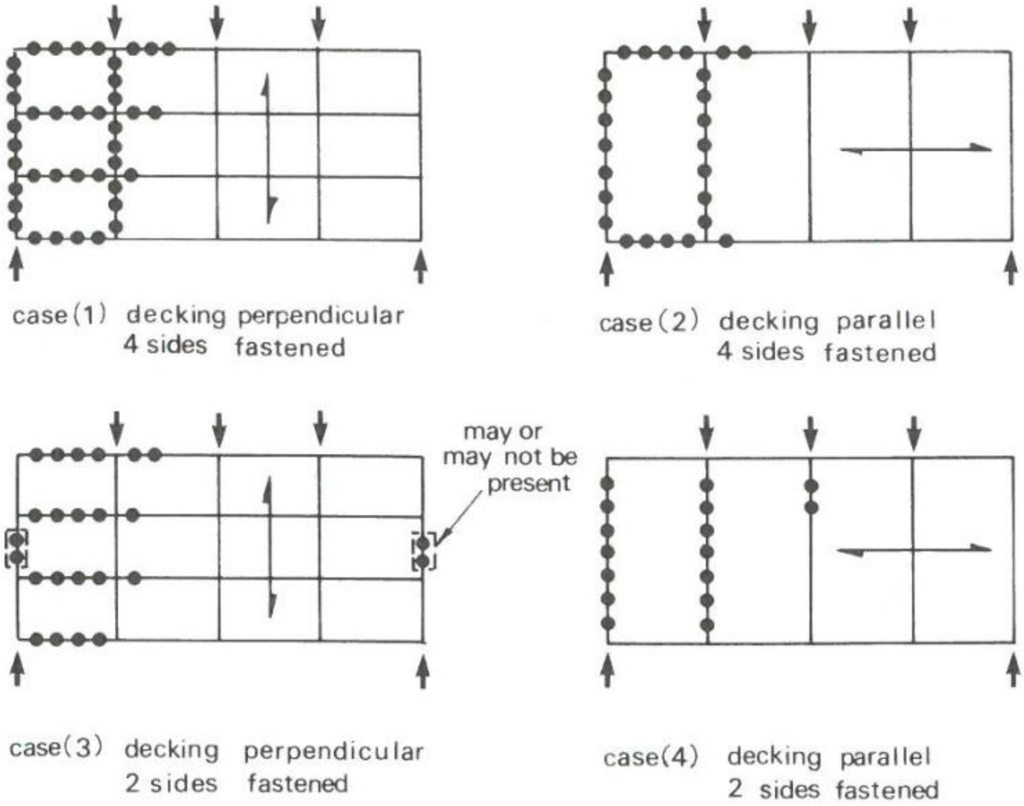


Figure 8. Alternative fastener arrangements (J.M. Davies and E.R. Bryan, 1982, P.18)

2.3 Necessary conditions for stressed skin design (SFS-EN 1993-1-3. P. 96)

Methods of stressed skin design that utilize sheeting as an integral part of a structure, may be used only under the following conditions:

- the use made of the sheeting, in addition to its primary purpose, is limited to the formation of shear diaphragms to resist structural displacement in the plane of that sheeting;

- the diaphragms have longitudinal edge members to carry flange forces arising from diaphragm action;

- the diaphragm forces in the plane of a roof or floor are transmitted to the foundations by means of braced frames, further stressed-skin diaphragms, or other methods of sway resistance;

- suitable structural connections are used to transmit diaphragm forces to the main steel framework and to join the edge members acting as flanges;

- the sheeting is treated as a structural component that cannot be removed without proper consideration;

- the project specification, including the calculations and drawings, draws attention to the fact that the building is designed to utilize stressed skin action;

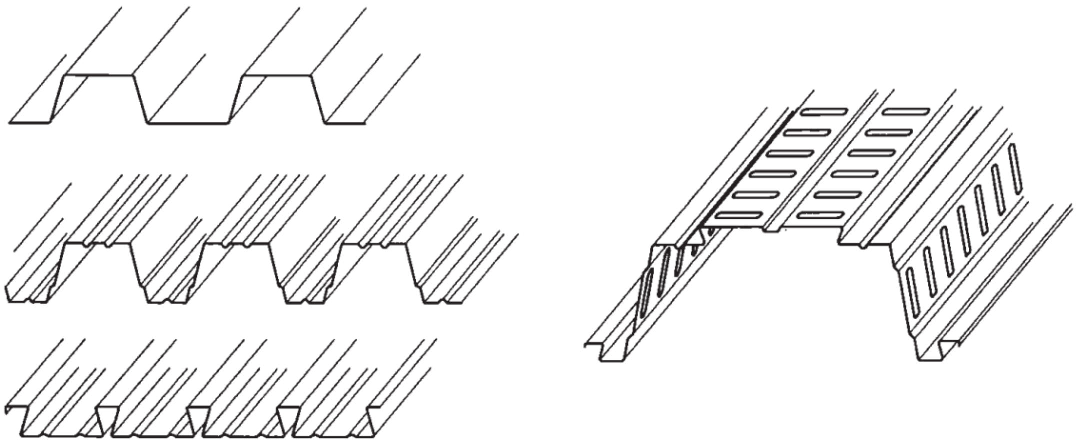
- in sheeting with the corrugation oriented in the longitudinal direction of the roof the flange forces due to diaphragm action may be taken up by the sheeting.

Stressed skin design may be used predominantly in low-rise buildings, or in the floors and facades of high-rise buildings.

Stressed skin diaphragms may be used predominantly to resist wind loads, snow loads and other loads that are applied through the sheeting itself. They may also be used to resist small transient loads, such as surge from light overhead cranes or hoists on runway beams but may not be used to resist permanent external loads, such as those from plant.

2.4 Profiled steel sheeting

Profiled steel sheeting is thin-walled cold formed steel profiles. Some of the trapezoidal sheeting and deck are presented in Figure 9. The depth of the panels usually ranges from 20-250mm, while the thickness is from 0.4 to 1.5 mm.



(a) (b)

Figure 9. Metal sheet profiles and linear trays

(a) Shallow deck (50-100mm) (b) deep deck (150-250 mm)

Material properties are presented in the Table 2. The basic yield strength f_{yb} and ultimte tensile strength f_u are listed.

All steels used for cold-formed members and profiled sheets should be suitable for cold-forming and welding, if needed. Steels used for members and sheets to be galvanized should also be suitable for galvanizing. (SFS-EN 1993-1-3. P. 12)

The nominal values of material properties given in this section should be adopted as characteristic values in design calculations. (SFS-EN 1993-1-3. P. 12)

Type of steel	Standard	Grade	f_{yb} N/mm^2	f_u N/mm^2
Cold reduced steel sheet of structural quality	ISO 4997	CR 220	220	300
		CR 250	250	330
		CR 320	320	400
Continuous hot dip zinc coated carbon steel sheet of structural quality	EN 10326	S220GD+Z	220	300
		S250GD+Z	250	330
		S280GD+Z	280	360
		S320GD+Z	320	390
		S350GD+Z	350	420
Hot-rolled flat products made of high yield strength steels for cold forming. Part 2: Delivery conditions for thermomechanically rolled steels	EN 10149: Part 2	S 315 MC	315	390
		S 355 MC	355	430
		S 420 MC	420	480
		S 460 MC	460	520
		S 500 MC	500	550
		S 550 MC	550	600
		S 600 MC	600	650
		S 650 MC	650	700
		S 700 MC	700	750
	EN 10149: Part 3	S 260 NC	260	370
		S 315 NC	315	430
		S 355 NC	355	470
		S 420 NC	420	530
Cold-rolled flat products made of high yield strength micro-alloyed steels for cold forming	EN 10268	H240LA	240	340
		H280LA	280	370
		H320LA	320	400
		H360LA	360	430
		H400LA	400	460
Continuously hot-dip coated strip and sheet of steels with higher yield strength for cold forming	EN 10292	H260LAD	240 2)	340 2)
		H300LAD	280 2)	370 2)
		H340LAD	320 2)	400 2)
		H380LAD	360 2)	430 2)
		H420LAD	400 2)	460 2)
Continuously hot-dipped zinc-aluminium (ZA) coated steel strip and sheet	EN 10326	S220GD+ZA	220	300
		S250GD+ZA	250	330
		S280GD+ZA	280	360
		S320GD+ZA	320	390
		S350GD+ZA	350	420
Continuously hot-dipped aluminium-zinc (AZ) coated steel strip and sheet	EN 10326	S220GD+AZ	220	300
		S250GD+AZ	250	330
		S280GD+AZ	280	360
		S320GD+AZ	320	390
		S350GD+AZ	350	420
Continuously hot-dipped zinc coated strip and sheet of mild steel for cold forming	EN 10327	DX51D+Z	140 1)	270 1)
		DX52D+Z	140 1)	270 1)
		DX53D+Z	140 1)	270 1)

1) Minimum values of the yield strength and ultimate tensile strength are not given in the standard. For all steel grades a minimum value of 140 N/mm^2 for yield strength and 270 N/mm^2 for ultimate tensile strength may be assumed.

2) The yield strength values given in the names of the materials correspond to transversal tension. The values for longitudinal tension are given in the table.

Table 2. The material strength of the Profiles sheeting (SFS-EN 1993-1-3. P. 14)

The most commonly used steel grades for cold formed steel sheets is S280GD+Z, S320GD+Z, S350GD+Z.

There are following regulations can be found in Eurocode regarding profiled panel of stressed skin. (SFS-EN 1993-1-3. P. 97)

Small randomly arranged openings, up to 3% of the relevant area, may be introduced without special calculation, provided that the total number of fasteners is not reduced. Openings up to 15% of the relevant area (the area of the surface of the diaphragm taken into account for the calculations) may be introduced if justified by detailed calculations. Areas that contain larger openings should be split into smaller areas, each with full diaphragm action.

All sheeting that also forms part of a stressed-skin diaphragm should first be designed for its primary purpose in bending. To ensure that any deterioration of the sheeting would be apparent in bending before the resistance to stressed skin action is affected, it should then be verified that the shear stress due to diaphragm action does not exceed 0,25 f_{yb}/γ_{M1}.

The shear resistance of a stressed-skin diaphragm should be based on the least tearing strength of the seam fasteners or the sheet-to-member fasteners parallel to the corrugations or, for diaphragms fastened only to longitudinal edge members, the end sheet-to-member fasteners. The calculated shear resistance for any other type of failure should exceed this minimum value by at least the following:

- for failure of the sheet-to-purlin fasteners under combined shear and wind uplift, by at least 40%;

- for any other type of failure, by at least 25%.

2.5 Fastening

It is extremely important that the connections have the enough strength to make sure that the stressed skin effect works properly. Types of the fasteners are shown in Table 3 and 4.

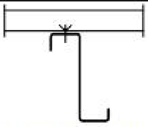
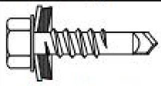
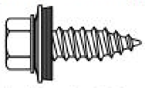
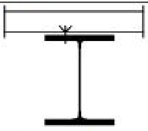
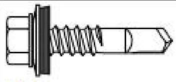
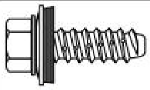
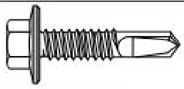
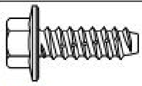
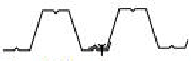
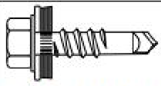
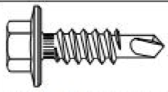
Field of application	Outdoors, stainless austenitic steel, up to Corrosivity Class C4		Indoors. zinc plated carbon steel, Corrosivity Class C1	
	Self-drilling	Thread forming	Self-drilling	Thread forming
 Sheet/lightweight beam	 ϕ 4.8 och 5.5	 ϕ 4.8 och 6.3 C-type thread (AB)	 ϕ 4.8, 5.5 and 6.3	 ϕ 4.8 and 6.3 C-type thread (AB)
 Sheet/rafter	 ϕ 5.5	 ϕ 6.3 B-type thread	 ϕ 5.5 and 6.3	 ϕ 6.3 B-type thread
 Sheet/sheet	 ϕ 4.8, 5.5 and 6.3 See 3.32		 ϕ 4.8, 5.5 and 6.3 See 3.32	

Table 3. Sheetmetal screw (Torsten Höglund,2002, P.12)

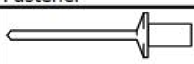
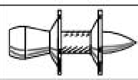
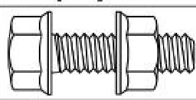
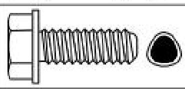
Field of application	Designation	Fastener	Dimension	Corrosivity Class
Side overlap and reinforcement	Pressure-tight rivet with mandrel, blind rivet		ϕ 4.0, 4.8 and 6.4	C1 – C4 (aluminium rivet)
Sheet to steel girder $t > 6.0$ mm	Cartridge fired pin		ϕ 4.5	C1
Predrilled lightweight beam etc	Flanged screw and flanged nut		M12 SF1 M6 MF12	C1
Fixing and jointing of lightweight beam	Thread forming screw type TAP-TITE		M8, M10	C1

Table 4. Other fastener types (Torsten Höglund,2002, P.12)

According to Eurocode 1993-1-3, there are following regulations regarding fastening of stressed skin: (SFS-EN 1993-1-3. P. 97)

1) In a profiled steel sheet diaphragm, both ends of the sheets should be attached to the supporting members by means of self-tapping screws, cartridge fired pins, welding, bolts or other fasteners of a type that will not work loose in service, pull out, or fail in shear before causing tearing of the sheeting. All such fasteners should be fixed directly through the sheeting into the supporting member, for example through the troughs of profiled sheets, unless special measures are taken to ensure that the connections effectively transmit the forces assumed in the design.

2) The seams between adjacent sheets should be fastened by rivets, self-drilling screws, welds, or other fasteners of a type that will not work loose in service, pull out, or fail in shear before causing tearing of the sheeting. The spacing of such fasteners should not exceed 500 mm

3) The distances from all fasteners to the edges and ends of the sheets should be adequate to prevent premature tearing of the sheets.

3 DETERMINATIONS OF SHEAR STRENGTH AND SHEAR FLEXIBILITY FOR A DIAPHRAGM

When an individual panel is loaded in shear up to failure, the load-deflection curve has the form shown in Figure 10.

It is crucial to figure out the shear flexibility, which will be explained in this chapter. Then the deflection of the panel under the shear force can be easily calculated with the following formula.

$$\vartheta = c * V \tag{1}$$

Where c=the flexibility of an individual diaphragm [mm/kN]
V=shear load on an individual diaphragm [kN]
ϑ =deflection of an individual diaphragm [mm]

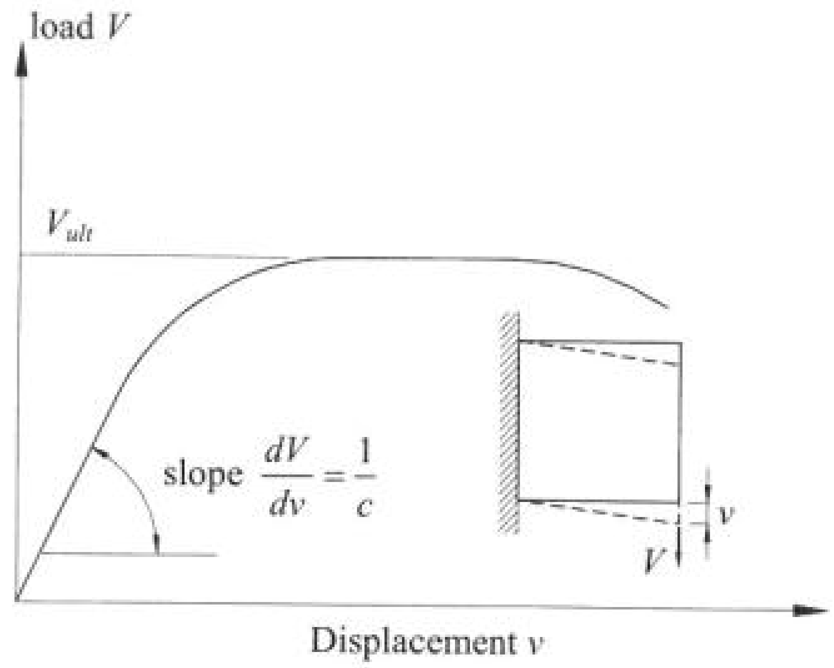


Figure 10. Typical load-deflection curve of a basic shear panel (ECCS, 2012, P.379).

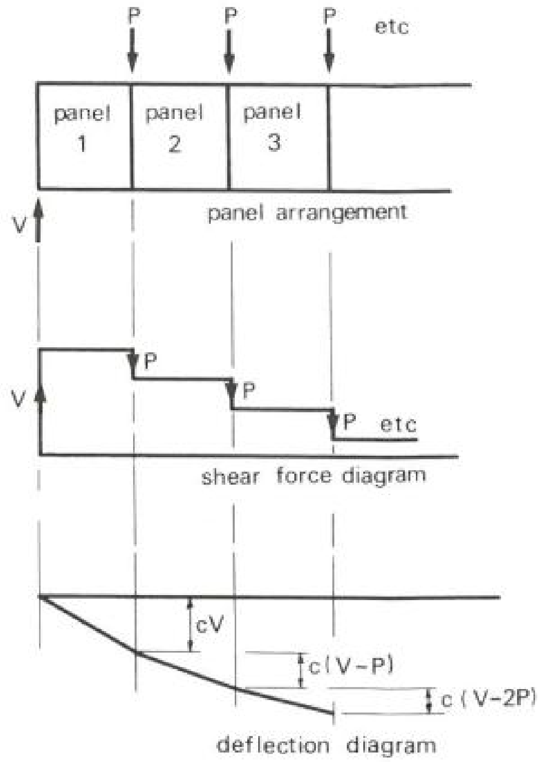


Figure 11. Forces and deflections in flat roof diaphragms (J.M. Davies and E.R. Bryan, 1982, P.44)

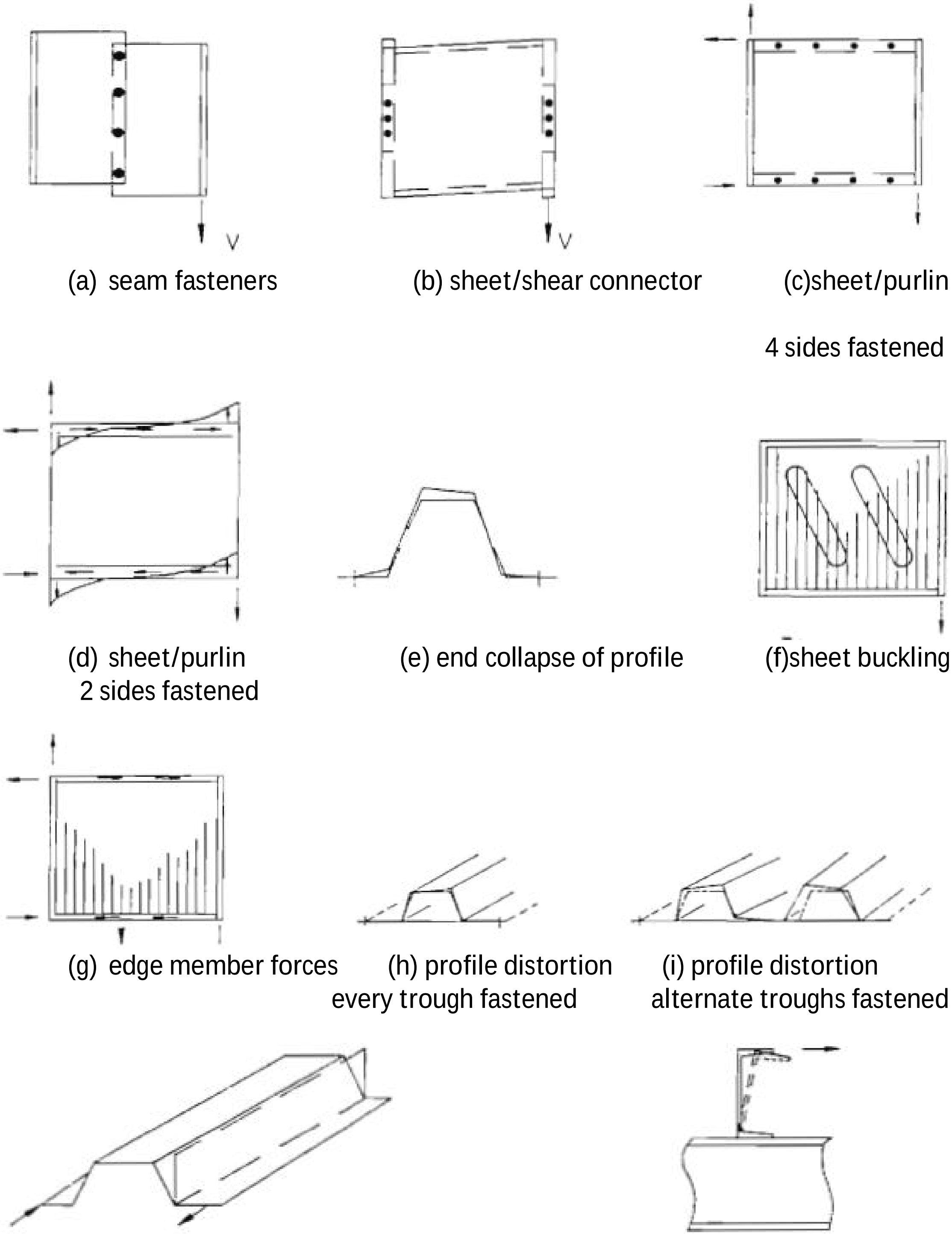
The basic shear panel assembly into the overall diaphragm layout is shown in Figure 11. Therefore, when the shear flexibility of one panel was defined, the total deflection of the whole diaphragms structure can be given by the following formula:

$$\Delta = \frac{n^2}{8} c\, qa \quad \text{or} \quad \Delta = \frac{n^2}{8} c\, P \qquad (2)$$

Where Δ= mid-span deflection of a diaphragm beam[mm].
P= point load on the diaphragm[kN].
q= uniformly distributed load on the diaphragm[kN/m].
a= the distance between the rafters[m].
n= number of panels within length of diaphragm assembly.

3.1 Determination of shear strength of diaphragms

The maximum load of the shear panel is the condition of the diaphragms design. The panel would get failed when the shear force is exceeding to some limitation. The ultimate strength of the diaphragm is obtained by considering all kinds of possible failure modes and the lowest strength would define the panel strength. Figure 12 listed all the possible failure modes. Failures at fasteners (a) (b) (c) (d) are ductile failures which are always aimed for during the stressed skin design. Therefore, the shear strength of diaphragm shall be designed to fail in one of the modes shown in Table 5. The other failure modes shall always be checked in design and they shall show a 25% safety reservation to the lowest load related to the failure modes in Table 5.



(a) seam fasteners (b) sheet/shear connector (c)sheet/purlin 4 sides fastened

(d) sheet/purlin 2 sides fastened (e) end collapse of profile (f)sheet buckling

(g) edge member forces (h) profile distortion every trough fastened (i) profile distortion alternate troughs fastened

(j)shear strain in sheeting (k)Purlin/rafter connection

Figure 12. Design criteria for diaphragm strength and flexibility (ECCS. 1995)

Type of diaphragm	Failure mode
Diaphragm fastened on four sides (direct shear transfer)	- failure at seam fasteners or - failure at shear connector fasteners
Diaphragms fastened on two sides (indirect shear transfer)	- failure at seam fasteners, or - failure at end sheet to purlin fastener

Table 5. Ductile failure mode of diaphragm depending on side fastening (Dan Dubina, ECCS 2012, P.381)

3.1.1 Failure along a line of seam fasteners

The expression for the design strength for this mode of failure is:

$$\boldsymbol{V_{Rd.1} = n_s F_s + \frac{\beta_1}{\beta_3} n_p F_P} \quad (3)$$

Where $\boldsymbol{V_{Rd.1}}$=shear strength of seam fasteners[kN].
$\boldsymbol{n_s}$=total number of seam fasteners per side lap (excluding those, which pass through both sheets and the supporting purlins)- (Red line figure 13)
$\boldsymbol{n_p}$=total number of perpendicular members (purlins or edge beams) within the depth of the panel.
$\boldsymbol{F_s}$=design strength of an individual seam fastener[kN]. See table 12.
$\boldsymbol{F_p}$=design strength of an individual sheet to perpendicular member fastener[kN]. See table 12.
$\boldsymbol{\beta_1}$=defined in Table 5.
$\boldsymbol{n_f}$=number of sheets to perpendicular member fasteners per member per sheet width - (Blue line figure 13)
$\boldsymbol{\beta_3} = \boldsymbol{(n_f - 1)/n_f}$ for case a; =1 for case b.

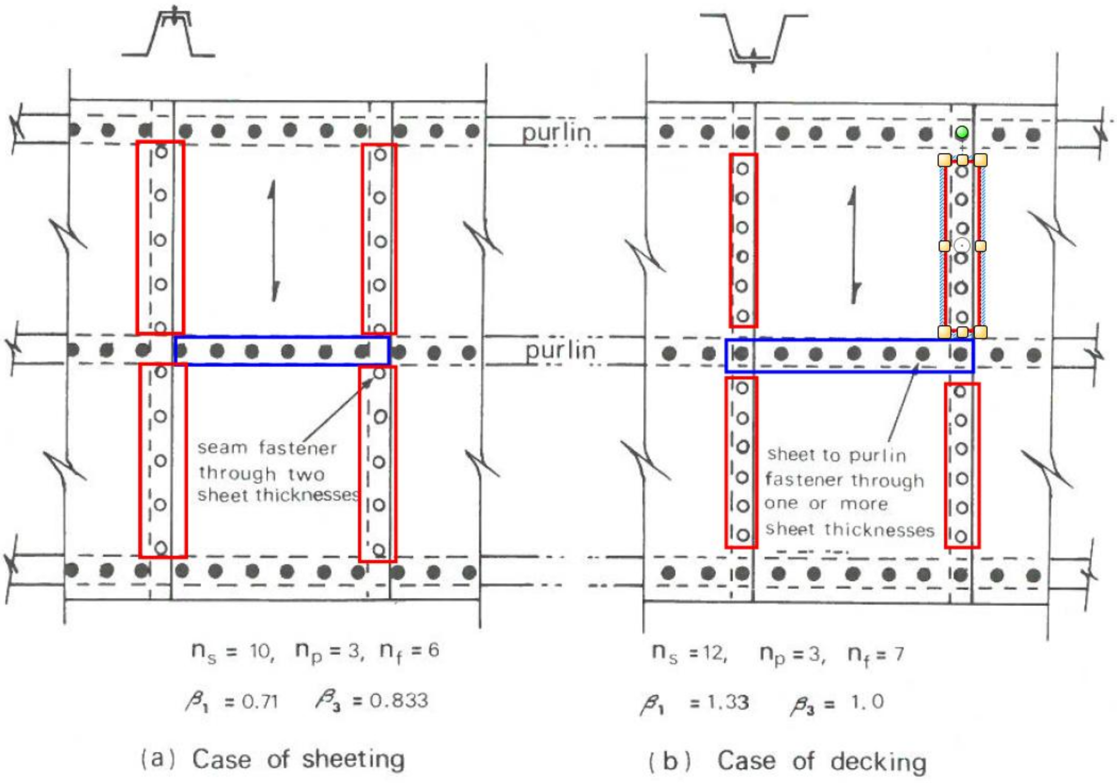


Figure 13. Alternative seam arrangements (a) case of sheeting (b) case of decking

Total number of fasteners per sheet width n_f	Factor β_1		Factor β_2
	Case 1 – sheeting	Case 2 – decking	
2	0.13	1.00	1.00
3	0.30	1.00	1.00
4	0.44	1.04	1.11
5	0.58	1.13	1.25
6	0.71	1.22	1.40
7	0.84	1.33	1.56
8	0.97	1.45	1.71
9	1.10	1.56	1.88
10	1.23	1.68	2.04

Table 6. Factors to allow for the number of sheet/purlin fasteners per sheet width (ECCS,1995)

3.1.2 Failure in fasteners to shear connectors (at end frame)

The expression for the design strength for this failure mode is:

$$V_{Rd.2} = n_{sc}F_{Sc} \quad (4)$$

Where $V_{Rd.2}$=shear strength of fasteners to shear connectors[kN].
n_{sc}=total number of shear connector fasteners in line.

F_{sc}=design strength of an individual shear connector fastener[kN]. See table 12.

3.1.3 Failure in the sheet to perpendicular member fasteners in a direction parallel to the span of the sheeting (Two sides fastened only)

This type of failure shall be checked when it is two sides fastened only, which is not the focus of this thesis. But the method of the calculation is introduced here. This failure mode is complicated, and it includes two parts: Sheet to purlins connections and purlin to rafter connections. Both need to be checked.

The expression for the design strength for the sheet to purlin fasteners is:

$$V_{Rd.3} = \beta_2 n_p F_p \tag{5}$$

Where $V_{Rd.3}$=shear strength of sheet to purlin fasteners in a direction parallel to the span of the sheeting[kN].
n_p=total number of perpendicular members (purlins) within the depth of the panel.
F_p=design strength of an individual sheet to perpendicular member fastener[kN]. See table 12.
β_2=defined in table 6.

The strength through the connection between purlin and rafter.

$$V_{Rd.3} = n_p F_{pr} \tag{6}$$

Where F_{pr}=design strength of an individual purlin to rafter fastener [kN]. See Table 7.

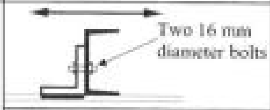
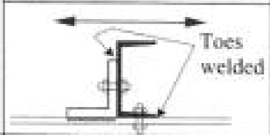
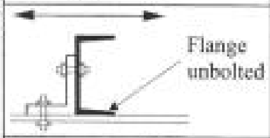
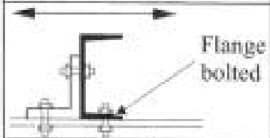
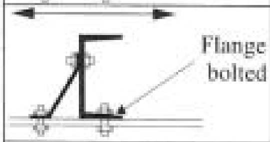
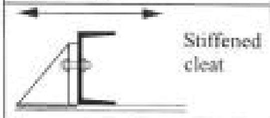
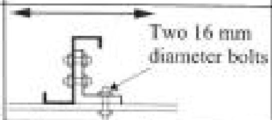
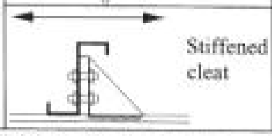
Connection number	Type of purlin (and cleat)	Connection detail	Design resistance F_{pr} [kN]	Flexibility s_{pr} [mm/kN]
1	102 × 51 rolled steel channel (89 × 64 × 7.8 angle cleat × 89 mm long)		4.9	0.84
2			20.0	0.11
3	152 × 76 rolled steel channel (76 × 64 × 6.2 angle cleat × 127 mm long)		14.4	0.60
4			7.2	1.20
5			19.6	0.35
6			25.0	0.13
7			25.0	0.05
8	254 × 102 × 22 kg/m Universal beam		10.0	2.60
9	203 × 51 × 2.0 zed (178 × 89 × 9.4 angle cleat × 127 mm long)		4.4	1.40
10			7.2	0.38
NOTE: The strength and flexibility of other types and sizes of purlin/rafter connections may be estimated from the values given above or may be obtained by test.				

Table 7. Design strengths and flexibilities of purlin/rafter connections. (ECCS,1995)

3.1.4 Failure due to shear buckling (global shear buckling)

In the case of corrugated steel sheet is on top of the purlins, the expression for the design strength [kN] for this failure mode is:

$$V_{cr,g} = \frac{14.4}{b} D_x^{\frac{1}{4}} D_y^{\frac{3}{4}} (n_p - 1)^2 \geq V_{Rd} \qquad (7)$$

Where $V_{cr,g}$=the design value of the gobal shear buckling strength of the diaphragm[kN].
Dx, Dy=Orthogonal bending stiffness of profiled sheet per unit length perpendicular and parallel to the corrugation respectively[kN mm2/mm]. The calculation formula was given by:

$$D_x = \frac{\mathrm{E}t^3 d}{12(1-\vartheta^2)u} \tag{8}$$

$$D_y = \frac{\mathrm{EI}_y}{d} \tag{9}$$

Where I_y= second moment of area about the neutral axis for a single corrugation. [mm4]
u=the perimeter length of a single corrugation[mm]
d=pitch of corrugations[mm]
$\boldsymbol{n_p}$=total number of perpendicular members (purlins) within the depth of the panel.

In the case of corrugated steel sheet is on top of the rafters, the expression for the design strength [kN] for this failure mode is:

-Fasteners in every corrugation

$$V_{cr,g} = \frac{28.8\mathrm{a}}{\mathrm{b}^2}\mathrm{D}_\mathrm{x}^{\frac{1}{4}}\mathrm{D}_\mathrm{y}^{\frac{3}{4}} \geq V_{Rd} \tag{10}$$

-Fasteners in every second corrugation

$$V_{cr,g} = \frac{14.4\mathrm{a}}{\mathrm{b}^2}\mathrm{D}_\mathrm{x}^{\frac{1}{4}}\mathrm{D}_\mathrm{y}^{\frac{3}{4}} \geq V_{Rd} \tag{11}$$

Note: additional safety of 25% was considered in the formulas in this section 3.1.4.

3.1.5 Failure due to shear buckling (local shear buckling)

When the corrugation is unstiffened, the expression for the design strength for this failure mode is:

$$V_{cr,l} = 4.83 \cdot \mathrm{b} \cdot \mathrm{t} \cdot \mathrm{E} \cdot (\frac{t}{l})^2 \geq V_{Rd} \tag{12}$$

Where $V_{cr,l}$=the design value of the local shear buckling strength of the diaphragm[kN].

When the corrugation is stiffened (shown in Figure 14), the expression for the design strength for this failure mode is:

$$V_{cr,l} = 36 \cdot \frac{b}{b_k^2} \sqrt[4]{D_x \cdot D_y^3} \geq V_{Rd} \tag{13}$$

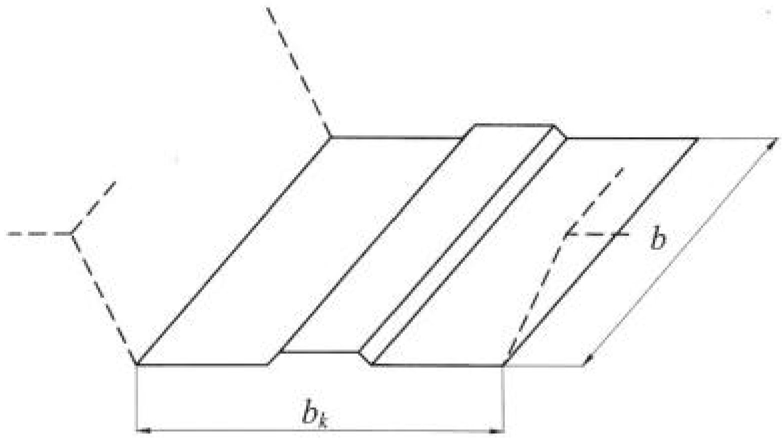


Figure 14. Stiffened corrugation

Where b_k=the width of the stiffened corrugation, See Figure 14.

$$D_x = \frac{\mathrm{E} \cdot I_f}{b_k} \tag{14}$$

$$D_y = \frac{\mathrm{E} \cdot t^3}{10.92} \tag{15}$$

Where I_f=the second moment of area of the corrugation about its horizontal axis.

Note: additional safety of 25% was considered in the formulas in this section 3.1.5.

3.1.6 Failure due to interaction of global and local shear buckling

The interaction of global and local shear buckling can be neglected if the following condition meets:

$$\frac{l}{t} \leq 2.9 \sqrt{\frac{E}{fy}} \tag{16}$$

Otherwise, the expression for the design strength for this failure mode is:

$$V_{red} = \frac{V_{cr,g} \cdot V_{cr,l}}{V_{cr,g} + V_{cr,l}} \geq V_{Rd} \tag{17}$$

Where l=width of the top or bottom flange of the sheeting, whichever is wider[mm].
t=net sheet thickness excluding galvanizing and coatings[mm].

V_{red}=the design value of the reduced shear buckling strength of the diaphragm under combined local and global buckling[kN].
f_y=the nominal yield strength of the steel sheet[N/mm2].

3.1.7 Failure in the sheeting to perpendicular member fasteners in a direction perpendicular to the span of the sheeting

In the case of corrugated steel sheet is on top of the purlins, the expression for the design strength for this mode of failure is:

$$V_{Rd.4} = \frac{\mathbf{0.6}\boldsymbol{bF_p}}{\boldsymbol{p\alpha_3}} \geq V_{Rd} \tag{18}$$

Where $V_{Rd.4}$=shear strength of the sheet to purlins in a direction perpendicular to the span of the sheeting[kN].
b=is depth of diaphragm in the direction parallel to the corrugations[mm].
p= the pitch of the sheet/purlin fasteners. When the sheet is fastened in every trough, p is the same as pitch of corrugation. When the sheet is fastened in every other trough, p is two times of the pitch of corrugation [mm].
α_3=non-dimensional factors. See Table 7.

In the case of corrugated steel sheet is on top of the rafters, the expression for the design strength for this mode of failure is:

$$V_{Rd4} = \frac{\mathbf{0.6}\boldsymbol{aF_p}}{\boldsymbol{p}} \geq V_{Rd} \tag{19}$$

Where $V_{Rd.4}$=shear strength of the sheet to rafters in a direction perpendicular to the span of the sheeting[kN].
a= the length of diaphragm in the direction perpendicular to the corrugations[mm].

Note: in oder to take account of the effect of combined shear and wind uplift load, additional safety of 40% was considered in the formulas in this section 3.1.7.

3.1.8 Failure due to end collapse of sheeting

In the case of corrugated steel sheet is on top of the purlins:

When the roof sheet is fastened in every corrugation, the expression for the design strength[kN] for this mode of failure is:

$$V_{Rd.5} = 0.9 f_y b\sqrt{\frac{t^3}{d}} \geq V_{Rd} \tag{20}$$

Where $V_{Rd.5}$=shear collapse strength of the end sheeting[kN].
d=pitch of corrugations[mm]
t=net sheet thickness excluding galvanizing and coatings[mm].

When the roof sheet is fastened in every other corrugation, the expression for the design strength for this mode of failure is:

$$V_{Rd.5} = 0.3f_y b\sqrt{\frac{t^3}{d}} \geq V_{Rd} \quad (21)$$

In the case of corrugated steel sheet is on top of the rafters:

When the roof sheet is fastened in every corrugation, the expression for the design strength[kN] for this mode of failure is:

$$V_{Rd.5} = 0.9f_y a\sqrt{\frac{t^3}{d}} \geq V_{Rd} \quad (22)$$

When the roof sheet is fastened in every other corrugation, the expression for the design strength for this mode of failure is:

$$V_{Rd.5} = 0.3f_y a\sqrt{\frac{t^3}{d}} \geq V_{Rd} \quad (23)$$

Note: additional safety of 25% was considered in the formulas in this section 3.1.8.

3.1.9 Failure of the edge member in compression or in combined compression and bending

The internal force of edge member shall be calculated by table 1 in Chapter 2.1(Note: additional safety of 25% wasn't considered in the formulas). When the internal force is obtained, the edge member can be designed in compression or in combined compression and bending according to relevant Eurocode regulation.

3.2 Determination of the shear flexibility of diaphragms

3.2.1 Flexibility due to distortion of the sheeting profile

The expression for the distortional flexibility of a panel is (when the sheet is on the purlins)

$$c_{1.1} = \frac{ad^{2.5}\alpha_1\alpha_4 K}{Et^{2.5}b^2} \quad (24)$$

Where $c_{1.1}$ = flexibility due to distortion of the corrugation[mm/kN].
a= the length of diaphragm in the direction perpendicular to the corrugations[mm].

b=is depth of diaphragm in the direction parallel to the corrugations[mm].
d=pitch of corrugations[mm].
E=modulus of elasticity [kN/mm2].
t=net sheet thickness excluding galvanizing and coatings[mm].
K=non dimensional sheeting constant. K can be obtained by linear interpolation from Appendix A.5 and A.6. It is defined by the sharp of the corrugation (θ, l/d, h/d). See Figure 15 and Figure 16. When the sheet is fastened in every trough, K1 shall be used. When the sheet is fastened in alternates trough, K2 shall be used.
α_1, α_4=non-dimensional factors. See Table 8 and 9.

The expression for the distortional flexibility of a panel is (when the sheet is connected directly to the rafters)

$$c_{1.1} = \frac{ad^{2.5}\alpha_5 K}{Et^{2.5}b^2} \tag{25}$$

Where α_5=non-dimensional factors. See Table 10 and 11.

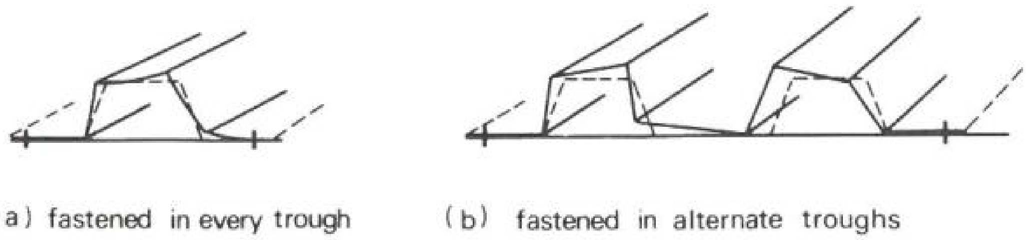


Figure 15. Profiles distortion with alternative fastener arrangements (a) fastened in every trough (b) fastened in alternate troughs. (J.M. Davies and E.R. Bryan, 1982, P.35)

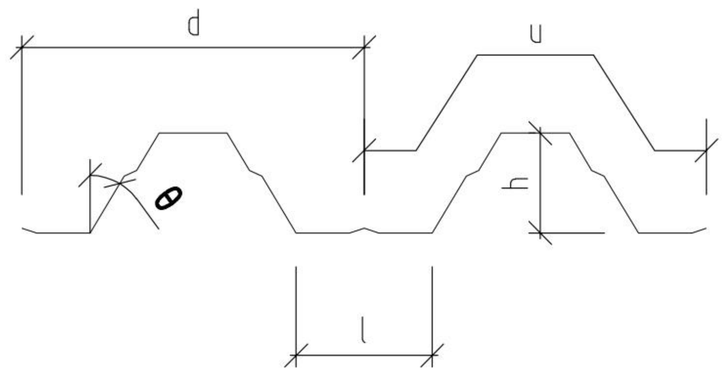


Figure 16. Basic dimension of the corrugated steel sheet

Total number of purlins per panel (or per sheet length for α_1) n_p	Factors		
	α_1	α_2	α_3
2	1.00	1.00	1.00
3	1.00	1.00	1.00
4	0.85	0.75	0.90
5	0.70	0.67	0.80
6	0.60	0.55	0.71
7	0.60	0.50	0.64
8	0.60	0.44	0.58
9	0.60	0.40	0.53
10	0.60	0.36	0.49
11	0.60	0.33	0.45
12	0.60	0.30	0.42
13	0.60	0.29	0.39
14	0.60	0.27	0.37
15	0.60	0.25	0.35
16	0.60	0.23	0.33
17	0.60	0.22	0.31
18	0.60	0.21	0.30
19	0.60	0.20	0.28
20	0.60	0.19	0.27

Table 8. Factors to allow for the effect of intermediate purlins (ECCS, 1995)

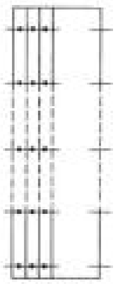
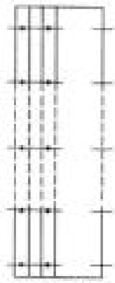
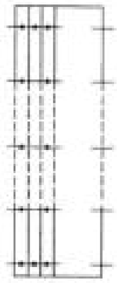
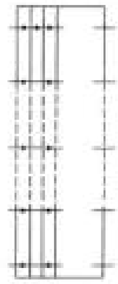
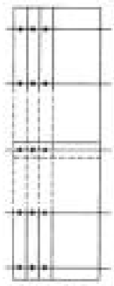
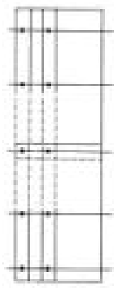
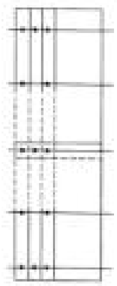
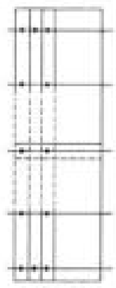
	Fastener positions			
	every corrugation	alternate corrugation	every corrugation at both sheet ends	every corrugation at one sheet end
one sheet length for full depth of diaphragm	 K=K_1 α_1 from Table 5.7 α_4=1 (1)	 K=K_2 α_1 from Table 5.7 α_4=1 (2)	 K=K_1 α_1=1 α_4=1 (3)	 K=K_2 α_1=0.5 α_4=1 (4)
n_b sheet lengths in depth of diaphragm	 K=K_1 α_1 from Table 5.7 for a number of purlins per sheet length α_4=(1+0.3n_b) (5)	 K=K_2 α_1 from Table 5.7 for a number of purlins per sheet length α_4=(1+0.3n_b) (6)	 K=K_1 α_1=1 α_4=(1+0.3n_b) (7)	 K=K_2 α_1 from Table 5.7 for a number of purlins per sheet length $\alpha_4 = (1+0.3n_b)\left(1-\frac{1}{n_b}\right)$ (8)

Table 9. Factors α_4 to allow for the member of sheet lengths in the depth of the diaphragms (ECCS,1995)

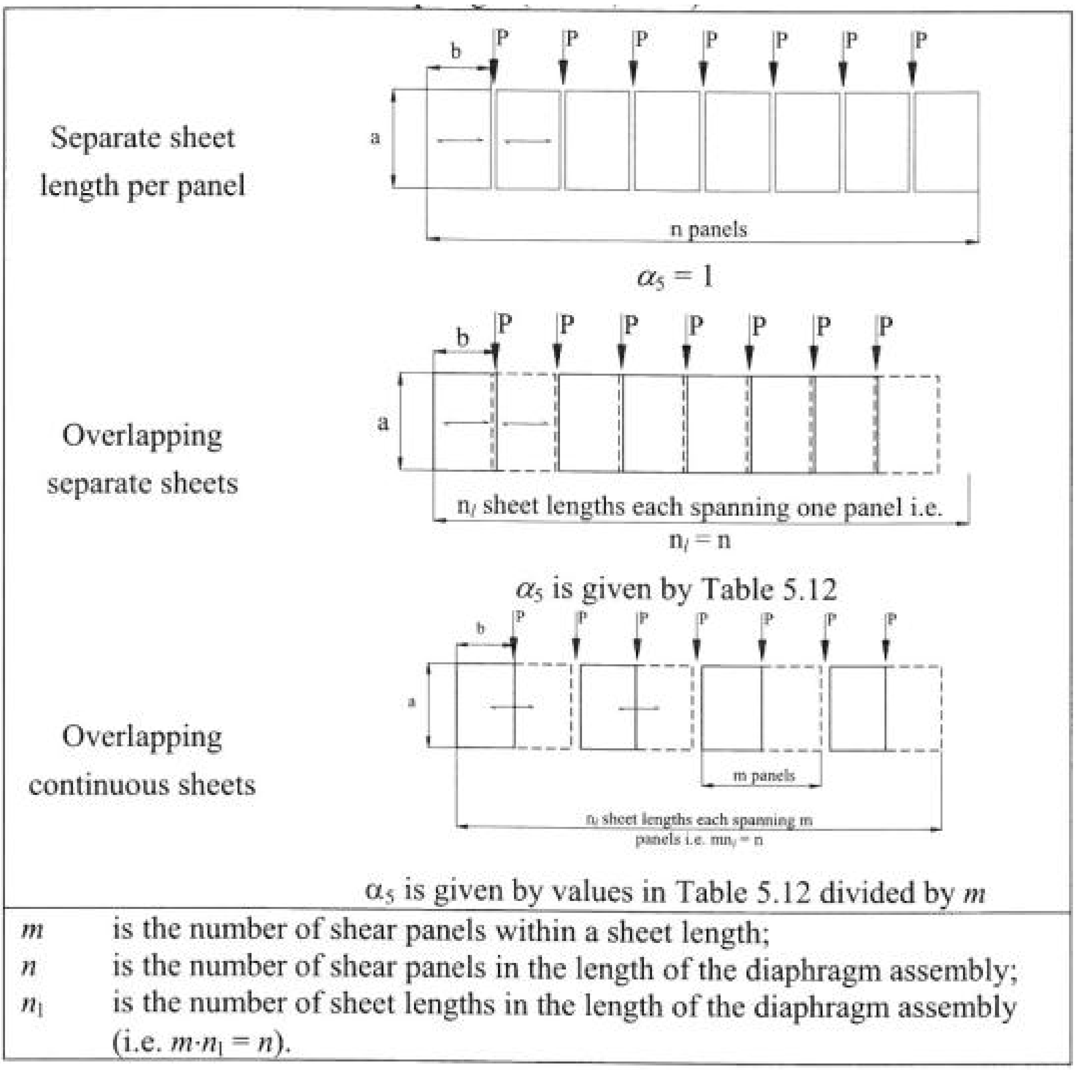


Table 10. Influence of sheet length when sheets are directly on the rafter (ECCS,1995)

Number of sheet lengths n_l	α_5
2	1.0
3	0.9
4	0.8
5 or more	0.7

Table 11. Factor α_5 to allow for sheet continuity

3.2.2 Flexibility due to shear strain in the sheet

The expression for the distortional flexibility of a panel is

$$c_{1.2} = \frac{2a(1+v)\left(1+\left(\frac{2h}{d}\right)\right)}{Etb} \tag{26}$$

Where $c_{1.2}$ =flexibility due to shear strain in the sheeting[mm/kN]
h = height of profile[mm]

3.2.3 Flexibility due to movement at the sheet to perpendicular member fasteners

The expression for the distortional flexibility of a panel is

$$c_{2.1} = \frac{2as_p p}{b^2} \quad (27)$$

Where $c_{2.1}$=flexibility due to movement in sheet to purlin fasteners[mm/kN]
s_p =flexibility per sheet to perpendicular member fastener per unit load[mm/kN]. See Table 12.
a=the length of diaphragm in the direction perpendicular to the corrugations[mm]
b=depth of diaphragm in the direction parallel to the corrugations[mm].
p=pitch of sheet to perpendicular member fasteners[mm].

(1) Sheet/purlin and sheet/shear connector fasteners					
	Washer type	Overall diam. (mm)	Design shear strength F_p and F_{sc}		Slip s_p and s_{sc} (mm/kN)
			Design formula	Design values kN per mm thickness of sheet	
Screws	Collar head	5.5 6.3	$1.9 f_u dt$ $1.9 f_u dt$	6.5 8.0	0.15
	Collar head + Neoprene washer	5.5 6.3	$1.9 f_u dt$ $1.9 f_u dt$	6.5 8.0	0.35
Fired pins	ϕ23 mm steel washer	3.7 to 4.8	$2.9 f_u dt$	8.0	0.10
(2) Seam fasteners (no washers)					
		Overall diam. (mm)	Design shear strength F_s		Slip s_s (mm/kN)
			Design formula	Design values kN per mm thickness of sheet	
Screws		4.1 to 4.8	$2.9(t/d)^{1/2} f_u dt$	2.5	0.25
Steel or monel blind rivets		4.8	$3.2(t/d)^{1/2} f_u dt$	2.8	0.30

Notes:

(1) In the above table, f_u is the specified ultimate tensile strength of the steel sheet (kN/mm^2), d is the nominal diameter of the fastener (mm) and t is the net sheet thickness (mm).

(2) The design strengths and slip values in this table apply to the range of fasteners, number of fasteners, sheet thickness and material strengths typically found in stressed skin panels. For other conditions, lap joint test should be made, in accordance with recommendations given in Chapter 7, to determine the resistance and slip values and to ensure that failure occurs by tearing of the sheeting. Strength formulas from above are in fact the bearing resistances for edge or end fasteners, and they have been obtained by using the relevant formulae of Chapter 7, Tables 7.8, 7.13 and 7.14, using for γ_{M2} the value of 1.1 instead of 1.25 (due to large number of fasteners in panel). For sheet-to-purlin and sheet-to-shear connector fasteners, it is assumed that the two material thickness, t_1 or $t_{sup} > 2.5t$, where t is the sheet thickness. For seam fasteners it assumed that the sheet thicknesses, which overlap, are equal. Alternatively the full calculation procedures, according to Tables 7.8 and 7.13 may be used. However end distance, edge distance and spacing limits for fasteners must be rigorously respected, and it is essential that the absolute limits on design strengths given in Table 5.5 to be not exceeded.

(3) Shear strength and slip values in this table are based on the following assumptions:

- the net sheet thickness is between 0.5 mm and 1.2 mm;
- the nominal yield and ultimate tensile strength of steel sheet do not exceed 355N/mm^2 and 480N/mm^2 respectively.

Table 12. Design strength and slip values for fasteners (ECCS,1995)

3.2.4 Flexibility due to movement in the seams

The expression for the distortional flexibility of a panel is

$$c_{2.2} = \frac{s_s s_p (n_{sh}-1)}{n_s s_p + \beta_1 s_s} \quad (28)$$

Where $c_{2.2}$ =flexibility due to movement in seam fasteners[mm/kN]
n_{sh} =number of sheet width per panel.
s_s =slip(flexibility) per seam fastener per unit load[mm/kN]. See Table 11.
s_p =slip(flexibility) per sheet to purlin (or edge beam) fastener per unit load[mm/kN]. See Table 12.

3.2.5 Flexibility due to movement in the sheet to parallel member fasteners

The expression for the flexibility due to movement in the sheet to parallel member fasteners is, if the sheeting is fastened to the supporting structure on four sides.

$$c_{2.3} = \frac{2 s_{sc}}{n_{sc}} \quad (29)$$

Where $c_{2.3}$ =flexibility due to movement in shear connectors[mm/kN]
n_{sc} =total number of sheet to shear connector fasteners per rafter[mm/kN]
s_{sc} =slip(flexibility) per sheet to shear connector fastener per unit load[mm/kN]. See Table 12.

3.2.6 Flexibility due to movement at the perpendicular member to parallel member (Purlin to rafter) connections

The expression for the flexibility due to movement in the sheet to parallel member fasteners is, if the sheeting is fastened to the supporting structure on four sides.

$$c_{2.3} = \frac{2}{n_p} \left(S_{pr} + \frac{s_p}{\beta_2}\right) \quad (30)$$

Where s_{pr} =movement of perpendicular member to parallel member connection per unit load[mm/kN]. See Table 7.

3.2.7 Flexibility due to axial strain in the purlins or edge members.

The expression for the flexibility due to axial strain in the purlins or edge members is:

$$c_3 = \frac{n^2 b^3}{4.8 E A a^2} \quad (31)$$

Where c_3 [mm/kN] is flexibility due to axial strain in the purlins or edge members.
A=the cross-sectional area of purlins or longitudinal edge member [mm^2]

3.3 Summary

The formulas of calculating the diaphragm shear strength and stiffness for both types are classified in this section, which is also the base of the Excel design tools.

3.3.1 Sheets on purlins

The roof layout for sheets on purlins is demonstrated in Figure 17. Expressions for diaphragm strengths are summarized in the Table 13. Expressions for diaphragm shear flexibility are summarized in the Table 14.

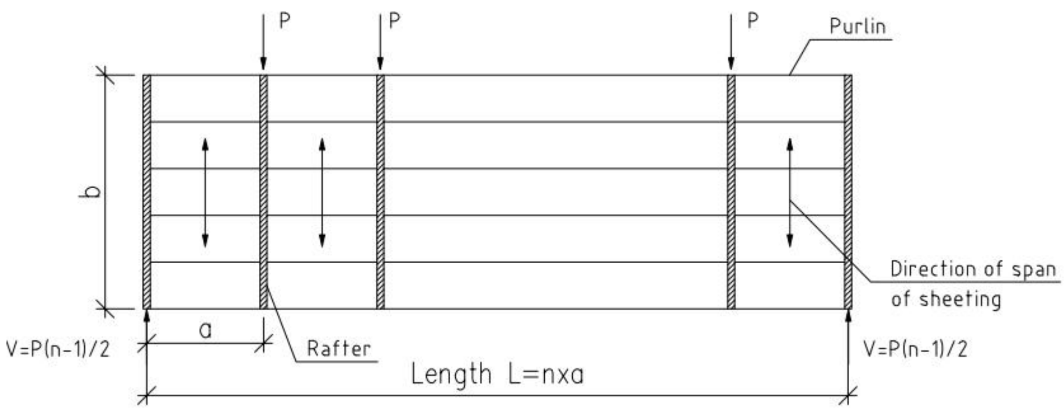


Figure 17. Sheets on purlins

Ultimate loads (kN)	seam strength	$V_{Rd.1} = n_s F_S + \frac{\beta_1}{\beta_3} n_p F_P$
	strength in fastener to shear connectors	$V_{Rd.2} = n_{sc} F_{Sc}$
	Strength at end sheet to purlin fasteners (2 sides fastened only)	$V_{Rd.3} = \beta_2 n_\mathrm{p} F_\mathrm{p}$ or $V_{Rd.3} = n_\mathrm{p} F_\mathrm{pr}$
Design shear capacity V_{Rd}		V_{Rd} is the minimum of the values above
Global shear buckling		$V_{cr.g} = \frac{14.4}{\mathrm{b}} \mathrm{D}_\mathrm{x}^{\frac{1}{4}} \mathrm{D}_\mathrm{y}^{\frac{3}{4}} (\mathrm{n_p} - 1)^2 \geq V_{Rd}$

Local shear buckling	$V_{cr.l} = 4.83btE(\frac{t}{l})^2 \geq V_{Rd}$
The interaction of global and local shear buckling	$V_{red} = \frac{V_{cr,g} \cdot V_{cr,l}}{V_{cr,g} + V_{cr,l}} \geq V_{Rd}$
General requirement for sheet to the perpendicular member fasteners	$V_{Rd.4} = \frac{0.6bF_p}{p\alpha_3} \geq V_{Rd}$
End collapse of sheeting	$V_{Rd.5} = 0.9f_y b\sqrt{\frac{t^3}{d}} \geq V_{Rd}$ (Every corrugation) $V_{Rd.5} = 0.3f_y b\sqrt{\frac{t^3}{d}} \geq V_{Rd}$ (Every two corrugation)

Table 13. Summary of the expressions for diaphragm strength

Where c=overall shear flexibility of diaphragm [mm/kN]

Shear flexibility due to:		Sheets directly on main beams Shear flexibility mm/kN
Sheet deformation	Profile distortion	$c_{1.1} = \frac{ad^{2.5}\alpha_1\alpha_4 K}{Et^{2.5}b^2}$
	Shear strains	$c_{1.2} = \frac{2a\alpha_2(1+v)(1+\left(\frac{2h}{d}\right))}{Etb}$
Fastener deformation	Sheet to purlin	$c_{2.1} = \frac{2as_p p\alpha_3}{b^2}$
	Seam fasteners	$c_{2.2} = \frac{2s_s s_p(n_{sh}-1)}{2n_s s_p + \beta_1 n_p s_s}$
	Connections to Rafters	$c_{2.3} = \frac{4(n+1)s_{sc}}{n^2 n_s{}'}$
Flange forces	Axial strain in purlins	$c_3 = \frac{n^2 a^3 \alpha_3}{4.8EAb^2}$
Total shear flexibility		$c = c_{1.1} + c_{1.2} + c_{2.1} + c_{2.2} + c_{2.3} + c_3$

Table 14. Summary of the expressions for diaphragm shear flexibility

3.3.2 Sheets directly on rafters

The roof layout for sheets on rafters is demonstrated in Figure 18. Expressions for diaphragm strengths are summarized in the Table 15.

Expressions for diaphragm shear flexibilities are summarized in the Table 16. As it can be seen from the Figure 16 and 17, the definition of a and b is different from each other.

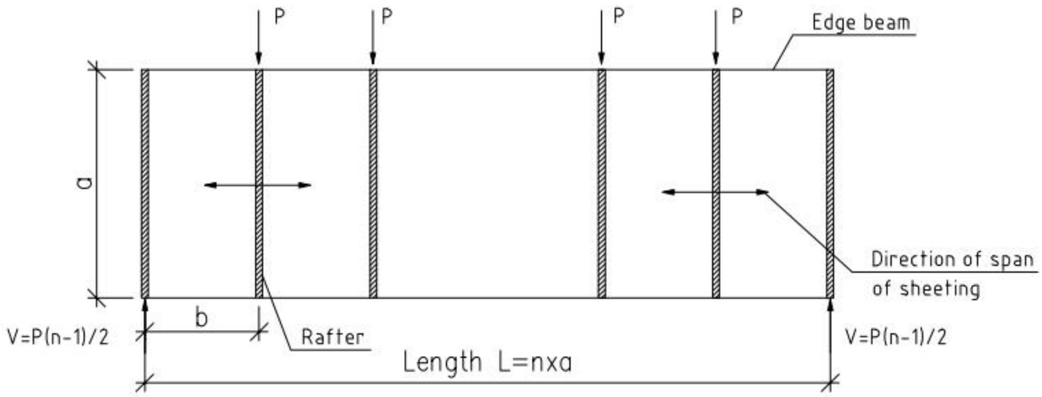


Figure 18. Sheets directly on rafters

Ultimate loads (kN)	seam strength	$V_{Rd.1} = \frac{a}{b}(n_s F_S + \frac{\beta_1}{\beta_3} n_p F_P)$
	strength in fastener to shear connectors	$V_{Rd.2} = \frac{a}{b}(n_{sc} F_{Sc})$
	Strength at end sheet to purlin fasteners (2 sides fastened only)	$V_{Rd.3} = \frac{a}{b}(1.5\beta_2 F_p)$ or $V_{Rd.3} = \frac{a}{b}(1.5F_{pr})$
Design shear capacity V_{Rd}		V_{Rd} is the minimum of the values above
Global shear buckling		$V_{cr.g} = \frac{28.8a}{b^2} D_x^{\frac{1}{4}} D_y^{\frac{3}{4}} (n_p - 1)^2 \geq V_{Rd}$ (Every corrugation) or $V_{cr.g} = \frac{14.4a}{b^2} D_x^{\frac{1}{4}} D_y^{\frac{3}{4}} (n_p - 1)^2 \geq V_{Rd}$ (Every two corrugation)
Local shear buckling		$V_{cr.l} = 4.83 * b * t * E(\frac{t}{l})^2 \geq V_{Rd}$
The interaction of global and local shear buckling		$V_{red} = \frac{V_{cr,g} \cdot V_{cr,l}}{V_{cr,g} + V_{cr,l}} \geq V_{Rd}$
General requirement for sheet to the perpendicular member fasteners		$V_{Rd.4} = \frac{0.6aF_p}{p} \geq V_{Rd}$

End collapse of sheeting	$V_{Rd.5} = 0.9 f_y b \sqrt{\frac{t^3}{d}} \geq V_{Rd}$ (Every corrugation) $V_{Rd.5} = 0.3 f_y b \sqrt{\frac{t^3}{d}} \geq V_{Rd}$ (Every two corrugation)

Table 15. Summary of the expressions for diaphragm strength

Shear flexibility due to:		Sheets on purlins
		Shear flexibility mm/kN
Sheet deformation	Profile distortion	$c_{1.1} = \frac{a d^{2.5} \alpha_5 K}{E t^{2.5} b^2}$
	Shear strains	$c_{1.2} = \frac{2a(1+v)(1+\left(\frac{2h}{d}\right))}{Etb}$
Fastener deformation	Sheet to purlin	$c_{2.1} = \frac{2a s_p p}{b^2}$
	Seam fasteners	$c_{2.2} = \frac{s_s s_p (n_{sh} - 1)}{n_s s_p + \beta_1 s_s}$
	Connections to Rafters	$c_{2.3} = \frac{2 s_{sc}}{n_{sc}}$
Flange forces	Axial strain in purlins	$c_3 = \frac{n^2 b^3}{4.8 E A a^2}$
Total shear flexibility		$c = \frac{b^2}{a^2}(c_{1.1} + c_{1.2} + c_{2.1} + c_{2.2} + c_{2.3}) + c_3$

Table 16. Summary of the expressions for diaphragm shear flexibility

4 INTERACTION OF SHEAR DIAPHRAGMS AND MAIN FRAMES

4.1 Elastic design of the global analysis

The flexibility of the shear diaphragm c and the flexibility of the frame k are the deflections under 1kN horizontal load, which are shown in Figure 19.

The relative flexibility can be represented:

$$\varphi = {}^{c}/_{k} \tag{32}$$

Where k= flexibility of frame of the structure[mm/kN].
c=overall shear flexibility of diaphragm[mm/kN].

Figure 19. Definitions of frame and panel flexibilities: (a) frame (b) sheeting panel

When the horizontal load is applied into the global analysis, part of the load is carried by main frames and part of the load is carried by diaphragm effect on the roof (Figure 21). The distribution of load between the main frames and the claddings is dependent on their relative stiffness. The two systems should work together, and finally they have the same deflections.

The structure with five frames is illustrated in Figures 20 and 21.

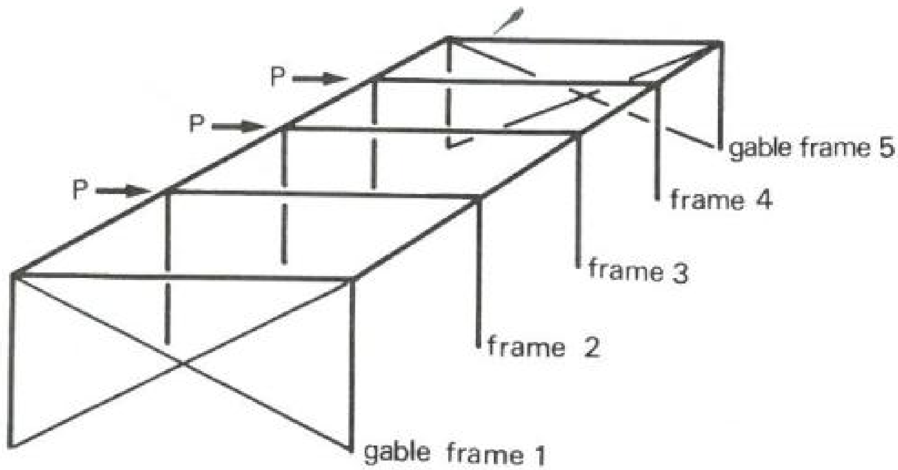


Figure 20. Flat roof structure with stressed skin effect

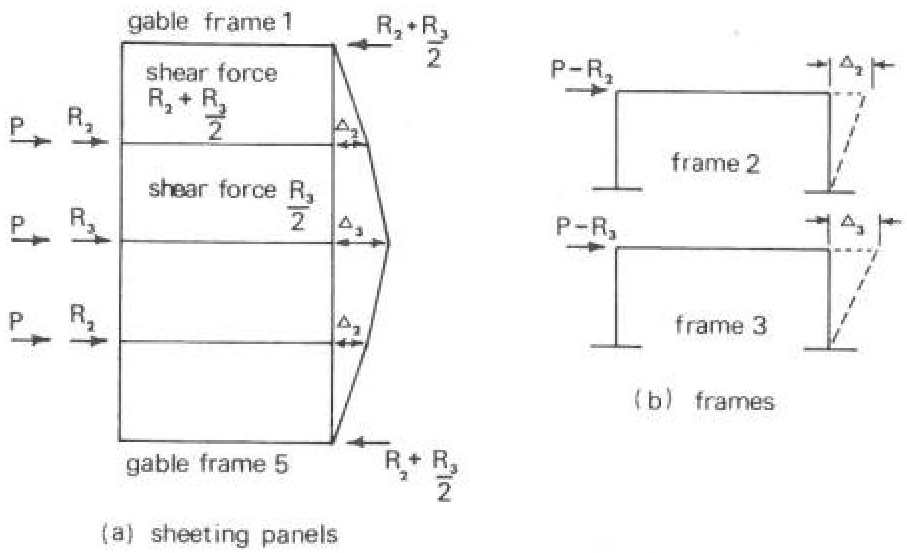


Figure 21. Forces and deflections of components of building (a) sheeting panels. (b) frames

4.2 Modeling of the stressed skin effect in the overall FEM model

In order to solve loading carried by diaphragm, overall load distribution within load bearing structures as well as deformations, it is convenient to use global structural analysis by computer software.

There are diverse ways to model the stressed skin effect is in the FEM model. The shear panel can be simplified into the equivalent truss structure or spring member or surface member in RFEM.

(a) The roof panel between two rafters can be described an equivalent tie as shown in Figure 22.

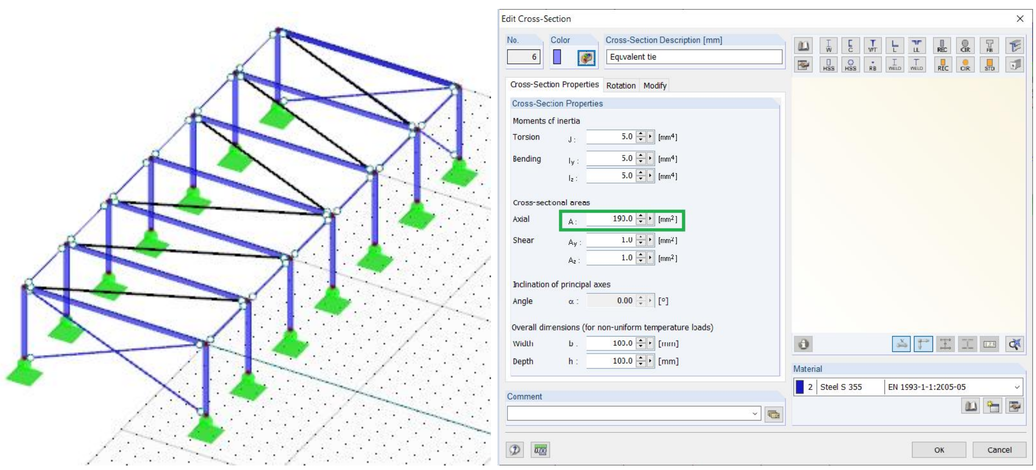


Figure 22. 3D FEM model with equivalent tie

The cross section of the equivalent tie is given by:

$$A = \frac{1}{cos^3\theta}\left(\frac{b}{c\,E}\right) = \frac{L^3}{c\,b^2\,E} \tag{33}$$

$$L = \sqrt[2]{a^2 + b^2}$$

Where A=the cross-sectional area of the equivalent tie[mm2]
L=the length of the equivalent tie[mm]
a= the length of diaphragm in the direction perpendicular to the corrugations [mm].
b=depth of diaphragm in the direction parallel to the corrugations [mm].
c=overall shear flexibility of diaphragm, it is obtained by Table 14 or Table 16 [mm/kN].

(b) The roof panel can be modelled into an equivalent spring as shown in Figure 23. The stiffness of the spring is calculated by the section area and length of the equivalent tie. The principal of this approach is as same as previous approach. It is an easier modelling method than the equivalent tie because only one parameter (the axial stiffness) needs to be inputted.

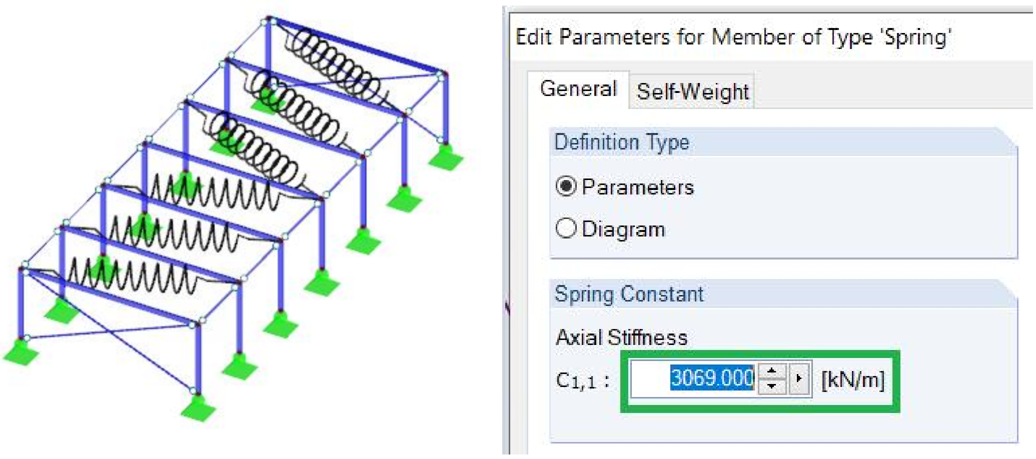


Figure 23. 3D FEM model with spring member

The stiffness of the equivalent spring given by:

$$C_{1,1} = \frac{EL}{A} \tag{34}$$

Where $C_{1,1}$=Axial stiffness of the equvalent spring[kN/m].
A=the cross-sectional area of the equivalent tie[mm2].
L=the length of the equivalent tie[mm].

(c) The roof panel can be modelled as an equivalent truss as shown in Figure 24. In this method, the users can define the height of the truss by themselves. Correspondingly, the top chord and truss chord would be defined by the designer. Figure 25 demonstrates part of the equivalent truss. B_t is the height of the equivalent truss.

The equivalent truss members can be fast obtained by choosing the closest deformation in Excel and then they can be modelled in RFEM to take part into the global analysis.

Calculation method is simplified and does not give absolutely correct deflection for the equivalent steel truss, but steel profiles and truss geometry based on this approach are correct enough to specify elastic stiffness of stressed skin diaphragm in analysis model.

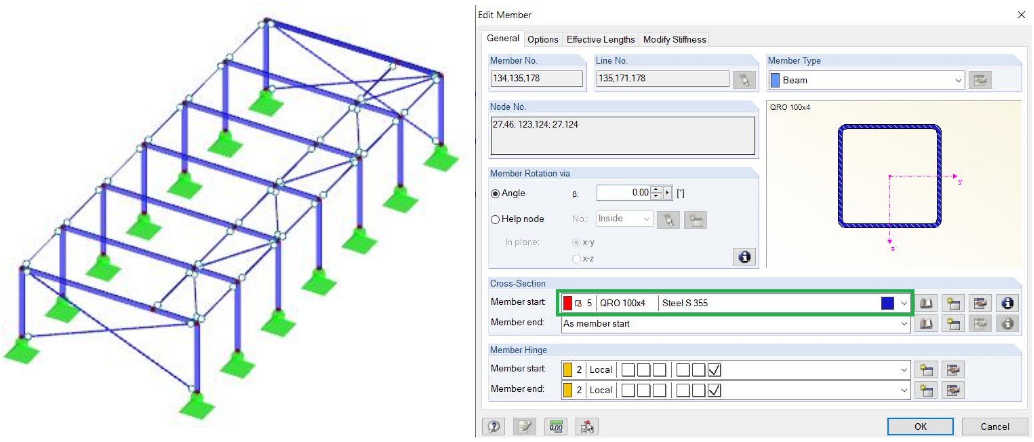


Figure 24. 3D FEM model with equivalent truss

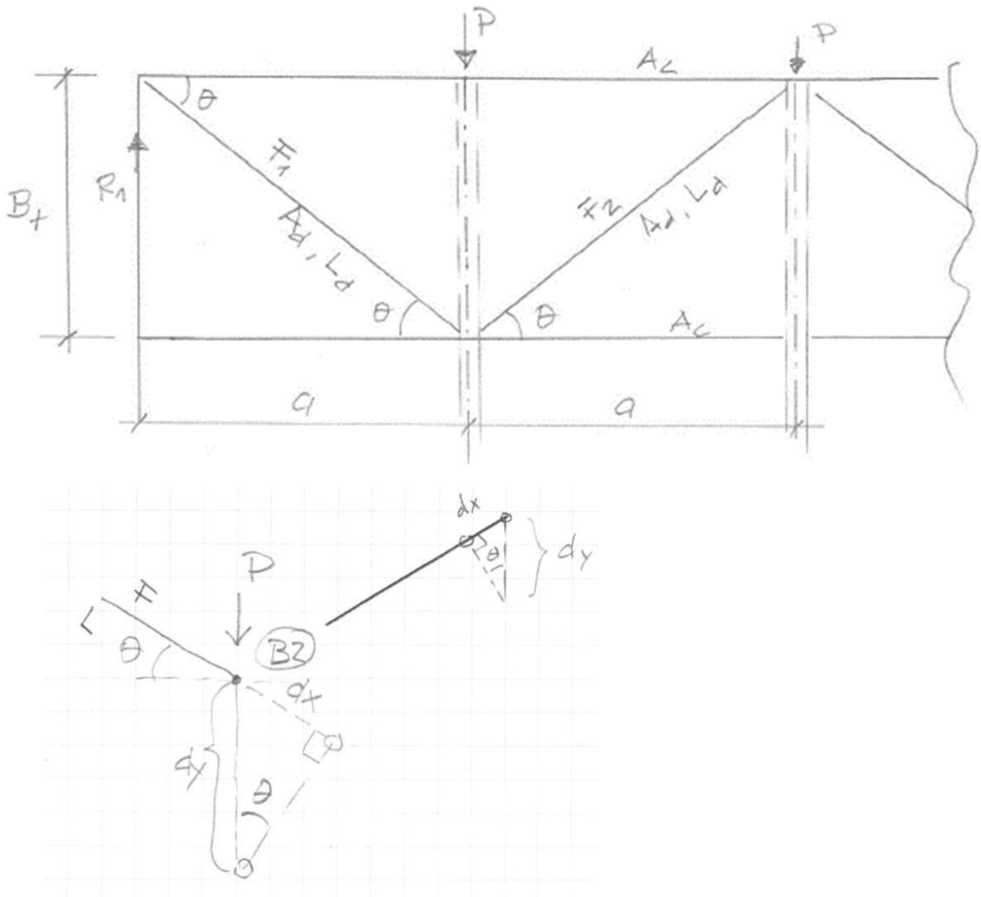


Figure 25. The equivalent truss parameter (part)

Vertical deformation at brace end is given by:

$$dyd = \frac{FB_t}{EA_d \sin\theta^2} \tag{35}$$

Where: B_t=the height of the truss, defined by designer[m].
A_d=the section area of the diagonal member[mm2].
F= the axial force of the diagonal member[kN].
θ=the angle marked in Figure 25.

If 2*m = even number:

$$\sum F = \frac{R_1 m-(m-1)P}{\sin\theta} \tag{36}$$

If 2*m = odd number:

$$\sum F = \frac{R_1(m+1/2)-(m-1/2)P}{\sin\theta} \tag{37}$$

Where: $\sum F$=sum of axial forces in diagonal braces[kN].
P= Horizontal point load on the diaphragm [kN].
R_1 = Reaction force at end of truss. R1=mP.
m= (n-1)/2. number of forces P corresponding to reaction R_1.
n=number of main frames.

Hence, deflection of equivalent steel truss due to elastic deformation of diagonal braces is given by:

If 2*m = even number:

$$\sum dyd = \frac{P(m^2-m+1)B_t}{EA_d \sin\theta^3} \tag{38}$$

If 2*m = odd number:

$$\sum dyd = \frac{P\left(m^2-\frac{m}{2}+{}^1/_2\right)B_t}{EA_d \sin\theta^3} \tag{39}$$

Deflection of equivalent steel truss due to elastic deformation of truss chords is given by:

$$dyc = \frac{5nPL^3}{384EI_0} \tag{40}$$

Where: I_0 = stiffness truss chords $= 2(\frac{B_t^2 A_c}{4} + 2I_a) * 1{,}25$
(1.25 is correction factor defined by the author. It based on the control calculations and it has been proved that it works well.)
A_c=the section area of the chord member[mm2].
I_a = second moment of area about the neutral axis of the chord member[mm4].

The total deflection of the equivalent steel truss is to add up the deflection of diagonal braces and chord memebers:

$$f = \sum dyd + dyc \tag{41}$$

(d) The roof panel can be modelled into an equivalent orthotropic surface panels as shown in Figure 26.

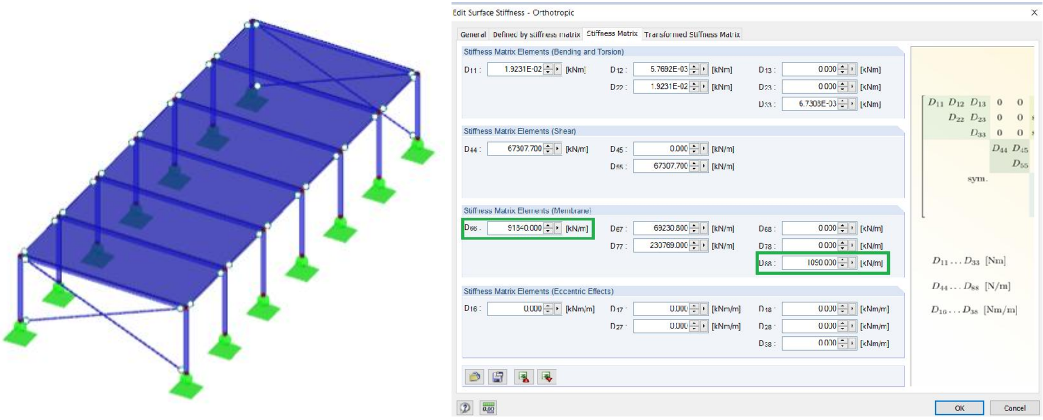


Figure 26. 3D FEM model with equivalent surface member

The following figure shows the general stiffness matrix of an orthotropic surface in RFEM analysis software.

$$\begin{bmatrix} m_x \\ m_y \\ m_{xy} \\ v_x \\ v_y \\ n_x \\ n_y \\ n_{xy} \end{bmatrix} = \begin{bmatrix} D_{11} & D_{12} & D_{13} & 0 & 0 & D_{16} & D_{17} & D_{18} \\ & D_{22} & D_{23} & 0 & 0 & D_{26} & D_{27} & D_{28} \\ & & D_{33} & 0 & 0 & D_{36} & D_{37} & D_{38} \\ & & & D_{44} & D_{45} & 0 & 0 & 0 \\ & & & & D_{55} & 0 & 0 & 0 \\ & & sym. & & & D_{66} & D_{67} & D_{68} \\ & & & & & & D_{77} & D_{78} \\ & & & & & & & D_{88} \end{bmatrix} \cdot \begin{bmatrix} \kappa_x \\ \kappa_y \\ \kappa_{xy} \\ \gamma_{xz} \\ \gamma_{yz} \\ \varepsilon_x \\ \varepsilon_y \\ \gamma_{xy} \end{bmatrix} \tag{42}$$

The designer can input all of them manually. It seems complicated to calculate all of them. However, only the axial action along the purlins or edge member and in-plane shear of roof sheeting matters like illustrated in Figure 27. D66 represents the axial stiffness of the purlin member or edge member and D88 is the in-plane shear stiffness They can be calculated by:

$$D_{66} = \frac{A \times E}{cc} \tag{43}$$

$$D_{88} = \frac{a}{b \times c} \tag{44}$$

Where: A=the cross-sectional area of longitudinal member [mm2]
cc=the distance between the longitudinal members in diaphragm [mm]
c=overall shear flexibility of diaphragm, it is obtained by Table 14 or Table 16 [mm/kN].

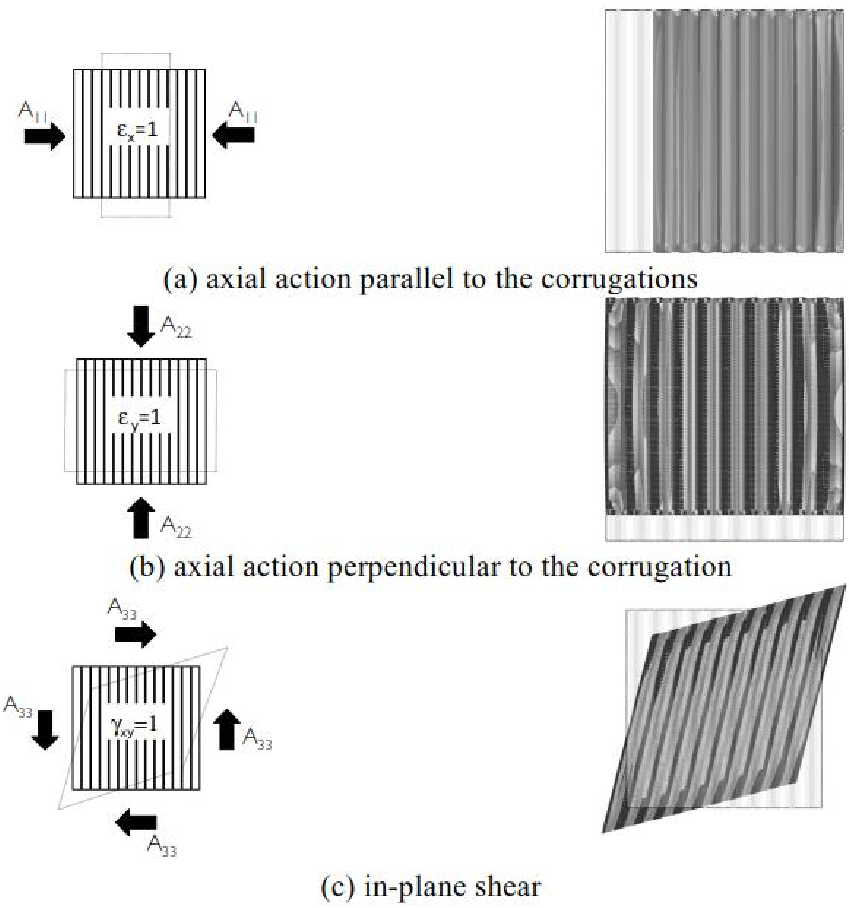


Figure 27. In plane loading of orthotropic member

5 METAL SHEET AND FASTENER DESIGN

The profiled sheeting and fasteners shall be designed for the vertical loads. In addition, roof sheeting needs to be designed for the in plane horizontal loads if the roof structure is working as stressed skin diaphragm.

Since the resistance data of the sheeting and fasteners shall be supplied by the manufacturer, metal sheet and fastenings are usually designed by the manufacturer produced software, such as Poimu (Ruukki), Optimi (Weckmann) and others. There is an option that the stressed skin effect can be considered in the software. The stressed skin effect data can be given in Poimu, which is demonstrated in Figure 28.

In this section, the theory of designing the sheet and fasteners are demonstrated. An example of design metal sheet and fastener made by Poimu is presented in Appendix A.4.

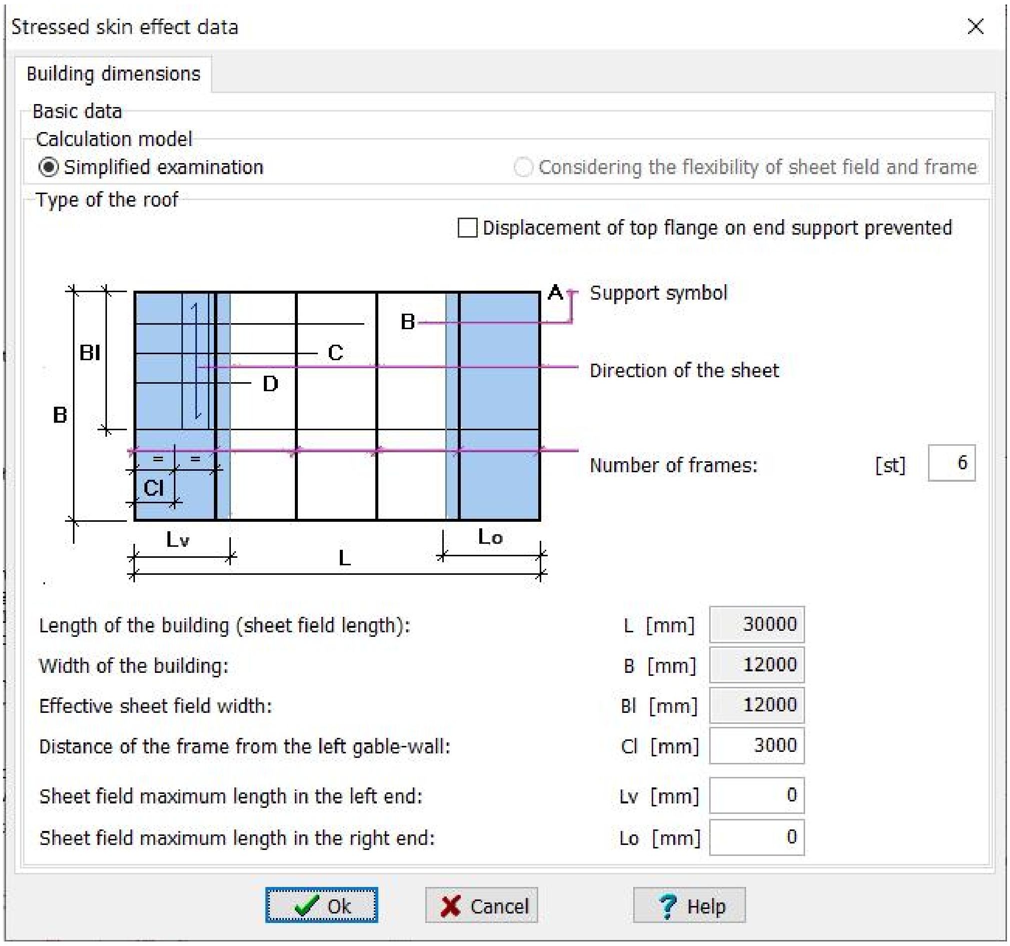


Figure 28. stressed skin effect data by Poimu

5.1 Design for sheeting

It is well known that the sheet member is cold-formed thin wall structure. Therefore, local buckling and global buckling are one of the major considerations in the design of corrugated sheeting. In this section, the design checking list for corrugated steel sheet designed for stressed skin action is summarized according to 'Stabilisation by Stressed Skin Diaphragm Action' (Höglund, 2002, P.24-27).

5.1.1 Individual plate element of corrugated sheet

Local buckling of corrugated sheet web shall be checked by:

$$V_{Ed} \leq V_{w,Rd} \tag{45}$$

Combined moment and shear force shall be checked by:

$$\frac{M_{f,Ed}}{M_{f,Rd}} + \frac{V_{Ed}}{V_{w,Rd}} \leq 1{,}3 \tag{46}$$

Where V_{Ed} =is maximum shear flow in plane of steel sheeting [kN/m]
$V_{w,Rd}$ =is design value of shear force resistance with respect to web buckling, is given by the sheet manufacturers or can be calculated by EN 1993-1-3[kN/m]
$M_{f,Ed}$ =is moment in span due to self-weight and other vertical loads [kNm/mm]
$M_{f,Rd}$ =is resistance to bending moment, is given by the sheet manufacturers or can be calculated by EN 1993-1-3 [kNm/mm]

Local buckling of corrugation sheet flange shall be checked by:

$$V_{Ed} \leq V_{f,Rd} \tag{47}$$

Combined moment and shear force shall be checked by:

$$0.8\frac{M_{f,Ed}}{M_{f,Rd}} + \frac{V_{Ed}}{V_{f,Rd}} \leq 1{,}1 \tag{48}$$

Where $V_{f,Rd}$ =is design value of shear force resistance with respect to flange buckling, is given by the sheet manufacturers or can be calculated by EN 1993-1-3[kN/m]

Global buckling shall be checked by :

$$V_{Ed} \leq V_{g,Rd} \tag{49}$$

Where $V_{g,Rd}$ =is design value of shear force resistance with respect to global buckling, is given by the sheet manufacturers or can be calculated by EN 1993-1-3[kN/m]. See also chapter 3.1.4 where global buckling of corrugated sheet has been discussed.

5.1.2 End support

Flexure of profile corners (end collapse of corrugated sheet profile) shall be checked by:

$$V_{Ed} \leq V_{r,Rd} \tag{50}$$

Where $V_{r,Rd}$ =is design value of shear force resistance with respect to flexure of the profile corner, is given by the sheet manufacturers[kN/m]]. See also chapter 3.1.8 where end collapse of corrugated sheet has been discussed. Design value of end collapse depends in the type profile fastening at end support.

Web crippling shall be checked by:

$$R_{V,Ed} \leq R_{es,\mathrm{Rd}} \tag{51}$$

$$\frac{\mathrm{R_{e,Ed}}}{\mathrm{R_{es,Rd}}} + \frac{\mathrm{R_{V,Ed}}}{\mathrm{R_{es,Rd}}} \leq 1{,}05 \tag{52}$$

Where $R_{V,Ed}$ =is the support reaction caused by the shear force[kN/m].
$R_{e,Ed}$ =is the support reaction at end support due to vertical loads[kN/m]
$R_{es,Rd}$ =is the resistance with respect to the end support reaction in accordance with EN 1993-1-3. [kN/m]

5.1.3 Intermediate supports

Local buckling of web and web crippling shall be checked by:

$$0.8\frac{\mathrm{R_{s,Ed}}}{\mathrm{R_{s,Rd}}} + \frac{\mathrm{V_{Ed}}}{\mathrm{V}_{w,\mathrm{Rd}}} \leq 1{,}1 \tag{53}$$

Where $R_{s,Ed}$ =is the support reaction at the intermediate support due to usual transverse action[kNm/mm]
$R_{s,Rd}$ =is the resistance with respect to support reaction, calculated in accordance with EN 1993-1-3 [kNm/mm]

Local web buckling due to shear force and bending moment shall be checked by:

$$\frac{\mathrm{M_{s,Ed}}}{\mathrm{M_{s,Rd}}} + \frac{\mathrm{V_{Ed}}}{\mathrm{V_{w,Rd}}} \leq 1{,}3 \tag{54}$$

Where $M_{s,Ed}$ =is the support moment at the intermediate support due to usual transverse action[kNm/mm]
$M_{s,Rd}$ =is the resistance with respect to support moment[kNm/mm]

5.1.4 Wind on the end

When the wind load is on the end, it will transfer a normal force in the sheet to the top of the columns.

Normal force in the sheet shall be checked by:

$$\mathrm{N_{Ed}} \leq \mathrm{N}_{c,\mathrm{Rd}} \tag{55}$$

Where N_{Ed} =is the maximum normal force in sheet[kN/m]

$N_{c,Rd}$ =is the resistance with respect to buckling of the sheet, is given by the sheet manufacturers or can be calculated by EN 1993-1-3[kN/m]

Normal force and bending moment shall be checked by:

$$\frac{N_{Ed}}{N_{c,Rd}}(1+0{,}5(1-\frac{N_{Ed}}{N_{c,Rd}}))+\frac{M_{f,Ed}}{M_{f,Rd}}\leq 1{,}0 \qquad (56)$$

Where $M_{f,Ed}$ =is moment in the span due to transverse action [kNm/mm]

$M_{f,Rd}$ =is resistance with respect to moment in the span [kNm/mm]

Normal force, bending moment and diaphragm force shall be checked by:

$$\frac{M_{f,Ed}}{M_{f,Rd}}+\frac{N_{Ed}}{N_{Rd}}+\frac{V_{2,Ed}}{V_{w,Rd}}\leq 1{,}3 \qquad (57)$$

$$0.8\frac{M_{f,Ed}}{M_{f,Rd}}+0{,}8\frac{N_{Ed}}{N_{Rd}}+\frac{V_{2,Ed}}{V_{f,Rd}}\leq 1{,}1 \qquad (58)$$

5.2 Design for fasteners

The most common connections for sheets sections are screws. There are three types of the screws: Self-tapping screws; self-drilling screws; thread-forming screws.

In this section, the design checking list for fasteners is summarized according to Stabilisation by Stressed Skin Diaphragm Action (Höglund, 2002, P.31). The failure modes of the screwed connections are presented in Figure 29. Figure 30 gives a clear image of the fastener component force.

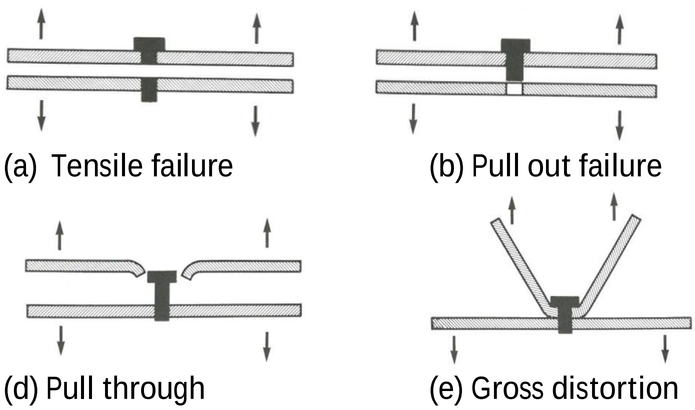


Figure 29. Failure mode

$$F_{v,Ed} = \sqrt{(\sum F_a)^2 + (\sum F_b)^2} \quad (59)$$

$$F_{t,Ed} = \sum F_c \quad (60)$$

Where $F_{v,Ed}$ =is shear force in the plane[kNm]
$F_{t,Ed}$ =is pull-out force[kNm]

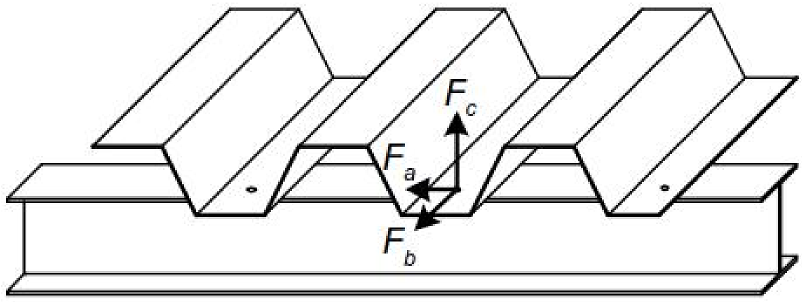


Figure 30. Fastener force components

$$\frac{F_{v,Ed}}{F_{b,Rd}} + \frac{F_{t,Ed}}{F_{p,Rd}} \leq 1{,}0 \quad (61)$$

$$\frac{F_{v,Ed}}{F_{v,Rd}} + \frac{F_{t,Ed}}{1{,}4F_{t,Rd}} \leq 1{,}0 \quad (62)$$

$$F_{v,Ed} \leq F_{v,Rd} \quad (63)$$

$$F_{t,Ed} \leq F_{t,Rd} \quad (64)$$

$$F_{t,Ed} \leq F_{o,Rd} \quad (65)$$

Where $F_{b,Rd}$ =is the resistance with respect to bearing failure[kN]
$F_{p,Rd}$ =is the resistance with respect to pull-through failure[kN]
$F_{v,Rd}$ =is the resistance with respect to shear failure[kN]
$F_{o,Rd}$ =is the resistance with respect to pull-out failure[kN]

6 PROPOSED DEISGN METHOD

6.1 Proposed design procedures for sheeting acting as diaphragm

Design procedure for sheeting acting as diaphragm is described in figure 31. The detailed instructions are as follows:

Step 1: The type of the corrugated steel sheet and the type and number of the screws can be chosen by Ruukki software Poimu or Weckman software Optimi according to the vertical loads, such as self-weight, imposed loads, snow load and etc. Design for in plane horizontal forces should also be performed in step 1.

Step 2: The total shear flexibility of a single sheeting panel between rafters and the maximum deflection of the whole roof sheeting diaphragm can be calculated by Excel calculation tools for the corrugated steel sheets and screws chosen in step 1. In additional, the equivalent truss structure or spring or surface panel can be defined based on the calculated stiffness of stressed skin diaphragm. This is initial data for step 3.

Step 3: The equivalent element member for the roof diaphragm can be modelled in the global analysis by RFEM or other computer analysis program so that the stressed skin effect of roof sheeting is properly taken into account in the structural analysis.

If step 2 or 3 results show that roof sheeting diaphragm is too flexible, the steel sheet and fasteners shall be checked again by Ruukki software Poimu or Weckman software Optimi with the combination of vertical loads and horizontal loads.

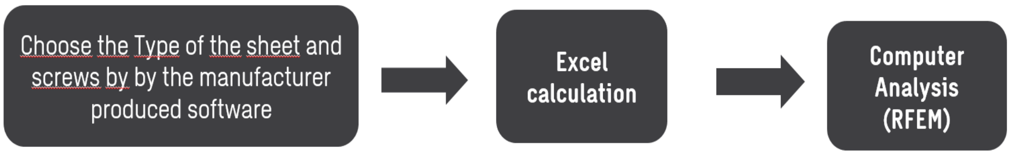


Figure 31. Design method proposed for stressed skin design

6.2 Excel design tool

The Excel calculation template is developed based on the stressed skin theory in Chapter 3. Sheet on the Purlins (four sides fastened) and sheet on the main beams (four sides fastened) are the cases which are included in excel design tool.

There are three pages for each case in the Excel calculation. The excel calculation for sheet on the purlins is displayed in Figure 32, 33, 34. The excel calculation for sheet on the rafters is presented in Appendix A.2.

Here is a short introduction of the Excel calculation. Item 01-05 is the input data and item 06-13 is the calculation results. Yellow colour means that the figure can be changed, and light blues are the calculation results. In item 03 and item 05, the roof sheet and screws can be obtained by sheet manufacturer software Poimu or Optimi as mentioned in Step 1 of the proposed design procedures. In this thesis, some of the commonly used type are implemented into the data base so that the designer can easily choose from the Excel list. The types of the sheet product are demonstrated in Table 17. The fasteners are illustrated in Table 18 and 19.

Weckman product	Ruukki product
W-45JA/900	T45-30L-905
W-70/900	T70-57L-1058
W-115/750	T130M-75L-930
W-130/950	T153-40L-840
W-155/840	

Table 17. The sheet can be chosen from Excel design tool

	Sp	Ssc
Screw collar head 5,5	0,15	0,15
Screw collar head 6,3	0,15	0,15
Screw collar head +Neoprene 5,5	0,35	0,35
Screw collar head +Neoprene 6,3	0,35	0,35

Table 18. Sheet to purlins / shear connector fastener by Excel

	Ss
Screw 4,8	0,25

Table 19. Sheet to sheet fastener by Excel

Item 06 calculates the shear flow and the normal force of the purlins or edge beams according to Table 1. Item 07 gives the diaphragm strength which is based on Table 13 and 15. Item 08 describe the flexibility of the single shear panel according to Table 14 and 16. Item 09 gives the displacement of the middle span, which can be verified by comparing the RFEM results. Item 10-11 give the crucial input data for the global analysis model when the diaphragm effect is taken into account.

As it can be seen, the shear flexibility and middle span displacement can be calculated immediately if we have input data ready. This Excel give a clear guide to the designers that how to simulate the stressed skin effect in the global analysis model. There are three different approaches so that the users can choose their own way depending on which software they are using and sometimes also rely on the project properties. For example, equivalent truss method (Item 11) is the most useful and flexible method because the corresponding truss can be found straightway to describe the stressed skin effect and it can be proceeded with all the calculation software.

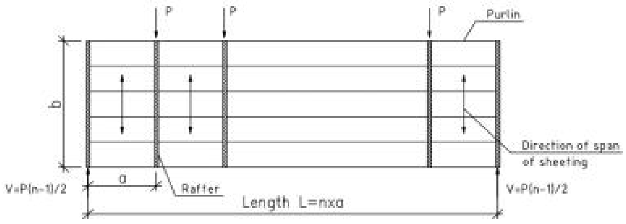
SWECO

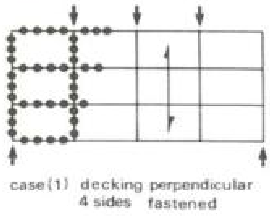
Project:

Subject: Corrugated sheet as stressed skin, deformation calculation

Sheet on the purlins (4 sides fastened)

STRESSED SKIN DESIGN ACCORDING TO ECCS 'Design of Code-formed Steel Structure' (2012)





Input data

01 Data of building (main frame):

Building/Roof sheet dimension

a=	5	m	Length of diaphragm	
b=	12	m	Depth of diaphragm	
H=	5	m	Height of building	
n=	6		Number of panels within diaphragm assebly	
L=	30	m	Length of building	L=n*a

02 Wind load on side of building

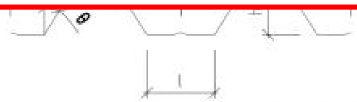
Wind load

q_w=	1,20	kN/m2	Peak velocity pressure (EN 1991-1-4)	
γ_Q=	1,5		Partial load factor	
p=	3,00	kN/m	Wind load at edge of the roof	$p=q_w$*H*0,5
p_d=	4,50	kN/m	Design wind load	pd=p*rQ
P=	15,00	kN	Point load on top of the building column	P=p*a

03 Choosed roof profile: **W-45JA/900_Narrow flange against the support** **(Based on results of roof sheet calculations)**

Type of the sheet can be chosen from the list.

Steel grade: S350GD+Z



f_y=	350	N/mm2	Basic yield strength	
f_u=	420	N/mm2	Ultimate tensile strength	
t=	0,5	mm	Sheet thickness, based on roof sheet design	
t_{nom}=	0,46	mm	Nominal sheet thickness used in design	
E=	210	kN/mm2	Modulus of elasticity	
G=	81	kN/mm2	Shear modulus	
υ=	0,3		Poisson's ratio	
u=	227	mm	Perimeter length of sheet corrugation	Second moment of area of a single corrugation
I_{y1}=	163,77	mm^4/mm	Second moment of area(from manufacture)	$I_{y1}=I_y$*d= 29479 mm^4
d=	180	mm	Pitch of corrugation	
l=	77	mm	See the picture on the right	Sheet profile distortion factor
h/d=	0,24		See the picture on the right	
K_1=	0,110		Fasteners in every corrugation	
K_2=	1,068		Fasteners in every alternate corrugation	

K	p(mm)
0,110	180
1,068	360

04 Choosed Purlin: IPE 80   cp= 1,5 m   Distance between the purlins

Steel grade: S355

Type of purlin can be chosen from the list.

f_y=	355	N/mm2	Basic yield strength	t_p= 5,2 mm
f_u=	510	N/mm2	Ultimate tensile strength	
E=	210	kN/mm2	Modulus of elasticity	
G=	81	kN/mm2	Shear modulus	
υ=	0,3		Poisson's ratio	
A=	764	mm^2	Cross-section	

05 Fastener(Types and amounts)

(a) Sheet to purlin connection **Screws collar head 5,5**

Type of screws can be chosen from the list.

K=	1,068		**Fasteners in every alternate corrugation**	
p=	360,000	mm	Distance between fasteners in connection to purlin	
F_p=	2,02	kN	Design shear resistance of individual sheet-to-purlin fastener	Table 12
s_p =	0,15	mm/kN	Slip(flexibility) per sheet to purlin fastener per unit load.	Table 12
n_f=	4		Number of sheet to purlin fasteners per sheet width. This can be calculated based on d and sheet type.	
n_p=	9		Total number of purlins(Edge+intermediate) width of roof, b	
n_b=	1		Number of sheet lengths within the lengths of diaphragm	
n_{sh}=	6		Number of sheet widths per pannel a	

(b) Seam fasteners **Screw 4,8**   Seam fastners in the crests

F_s=	0,87	kN	Design shear resistance of individual seam fastener	Table 12

Figure 32. Excel calculation for the case when sheet on the purlins (page 1)

SWECO		Designer	Phase / Area	Project nr
			Engineering	
Project:			Date	Page
Subject:	Corrugated sheet as stressed skin, deformation calculation		1.5.2020	

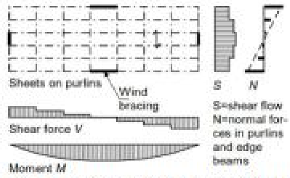
$s_s=$	0,25 mm/kN	Slip(flexibility) per seam fastener per unit load		Table 12
$n_s=$	40	Number of seam fasteners per side lap(excluding those which pass through both shees and the		
$\alpha_1=$	0,6	$\beta_1=$ 0,44	Non-dimensional factors	
$\alpha_2=$	0,4	$\beta_2=$ 1,11	α1, α2, α3 depend on n_p (Table 7)	
$\alpha_3=$	0,53	$\beta_3=$ 0,75	β1, β2 depend on n_f (Table 5)	
$\alpha_4=$	1		α4 (Table 8)	
(c) Sheet to shear connector fasteners		**Screws collar head 5,5**	Shear connectors transfer forces between steel sheet diaphragm and building frame	
$F_{sc}=$	2,02 kN	Design shear resistance of individual sheet-to-shear connector fastener		Table 12
$s_{sc}=$	0,15 mm/kN	Slip(flexibility) per sheet to shear connector fastener per unit load		Table 12
$n_{sc}=$	49,00	Number of sheet to shear connector fasteners per end rafter		
$n_{sc}=$	49,00	Number of sheet to shear connector fasteners per intermediate rafter		

Result

06 Actions on diaphragm due to wind

(a) **Wind on long side**

$R_d=$	67,50 kN	Support force at gable of the building
$N_d=$	24,47 kN	Normal force in edge beam at long sides of building
$V_{dmax}=$	5,39 kN/m	Maximun shear flow



(b) Wind on end (Only for checking the shear flow of the sheet)

$R_{dl}=$	27,00 kN	Support force in long sides
$N_{dl}=$	10,13 kN	Normal force in end rafter
$N_g=$	2,61 kN	Normal force in edge beam

Note: When the wind is on the end of the building, the stressed skin diaphragm height is 2/3*a.

07 Diaphragm strength — **Utility grade**

(a) Seam strength	$V_{Rd.1}=$	45,38 kN	$R_d/V_{Rd.1}=$	**1,49**	Check number and type of fasteners	FAILURE
(b) Strength in fasteners at shear connecto	$V_{Rd.2}=$	98,93 kN	$R_d/V_{Rd.2}=$	**0,68**		OK
(c) Global shear buckling resistance:	Dx=	1,48 kN*mm²/mm				
	Dy=	34391,7 kN*mm²/mm				
$V_{cr.g}=\frac{14{,}4}{b}Dx^{\frac{1}{4}}Dy^{\frac{3}{4}}(n_p-1)^2=$		1712,66 kN	$R_d/V_{cr.g}=$	**0,04**		OK
(d) Local shear buckling resistance:						
$V_{cr.l}=4{,}83btE(\frac{t}{l})^2=$		199,82 kN	$R_d/V_{cr.l}=$	**0,34**		OK
(e) The interaction of global and local shear buckling = Resistance for shear buckling:						
$V_{red}=\frac{V_{cr.g}*V_{cr.l}}{V_{cr.g}+V_{cr.l}}=$		178,94 kN	$R_d/V_{red}=$	**0,38**		OK
(f) Sheet-to-purlin fasteners:						
$V_{Rd.4}=\frac{0{,}6bF_p}{p\alpha 3}=$		76,19 kN	$R_d/V_{Rd.4}=$	**0,89**		OK
(g) End collapse of sheeting profile						
$V_{Rd.5}=0.9f_yb\sqrt{\frac{t^3}{d}}=$		kN	$R_d/V_{Rd.5}=$	**2,304**		FAILURE
$V_{Rd.5}=0.3f_yb\sqrt{\frac{t^3}{d}}=$		29,30 kN			Fasteners in every second corrugation	
Design shear capacity	$V_{Rd}=$	**45,38 kN**				

08 Comonents of shear flexibility:

(a) Flexibility due to profile distortion	$c_{1,1}=$	0,321 mm/kN
(b) Flexibility due to shear strain	$c_{1,2}=$	0,007 mm/kN
(c) Flexibility due to deformation in sheet to purlin fasteners	$c_{2,1}=$	0,002 mm/kN
(d) Flexibility due to deformation in seam fasteners	$c_{2,2}=$	0,029 mm/kN
(e) Flexibility in connections to shear connectors	$c_{2,3}=$	0,002 mm/kN
	c'=	0,361 mm/kN
(f) Equivalent shear flexibility due to axial strain in purlins	$c_3=$	0,021 mm/kN
Total shear fexibility	c =	0,382 mm/kN

Shear flexibility due to:		Sheets on purlins
		Shear flexibility mm/kN
Sheet deformation	Profile distortion	$c_{1.1}=\frac{ad^{2.5}\alpha_1\alpha_4 K}{Et^{2.5}b^2}$
	Shear strains	$c_{1.2}=\frac{2a\alpha_2(1+\nu)(1+(\frac{2h}{d}))}{Etb}$
Fastener deformation	Sheet to pulin	$c_{2.1}=\frac{2as_p p\alpha_3}{b^2}$
	Seam fasteners	$c_{2.2}=\frac{2s_s s_p(n_{sh}-1)}{2n_s s_p+\beta_1 n_p s_s}$
	Connections to Rafters	$c_{2.3}=\frac{4(n+1)s_{sc}}{n^2 n_s}$
Flange forces	Axial strain in purlins	$c_3=\frac{n^2a^3\alpha_3}{4.8EAb^2}$
Total shear flexibility		$c=c_{1.1}+c_{1.2}+c_{2.1}+c_{2.2}+c_{2.3}+c_3$

09 Mid-span deflection

Δ=P(n^2/8)*c= 25,8 mm > Δ_{max}= H/300= 16,67 mm

Figure 33. Excel calculation for the case when sheet on the purlins (page 2)

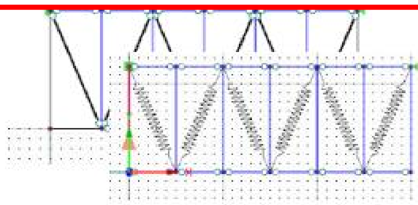
SWECO
Project:
Subject: Corrugated sheet as stressed skin

Input data of three different modelling method to take stressed skin into account in the global model.

10 Equivalent tie or equivalent spring member

Cross section area of equvalent tie	A=	**190,00**	mm2
Length of equvalent tie	L=	13	m
Modulus of elasticity of equivalent tie	E=	210	kN/mm2

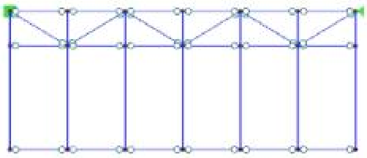
Axial stiffness of the equivalent spring member
K= 3069 kN/mm



11 Equivalent steel truss, simplified calculation of deflection in the middle of the truss
Definition of truss geometry:

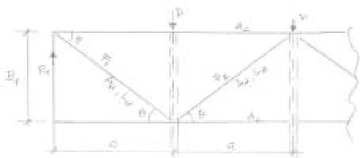
a=	5,00 m		B_t=	4,00	m
n=	6 m		L=	30,00	m

P= 15,0 kN



θ =arctan (B_t/a) = 0,675 radians
38,6598 degrees

m= (n-1)/2 = 2,5
m= number forces P corresponding reaction R1
R1 = m*P = 37,5 kN



Selection of steel tubes in truss

Diagonal brace	**P60x60x4**	A_d =	**855**	mm^2			
Truss chord	**P60x60x4**	A_c =	**855**	mm^2	I_a =	**4,355E+05**	mm^4

Calculation of deflection:

Deflection due to elastic deformation on diagonal braces:

if 2m = even $\Sigma dy_d = \dfrac{P*(m^2-m+1)*B_t}{E*A_d\sin\theta^3}$ mm

if 2m = odd $\Sigma dy_d = \dfrac{P*\left(m^2-\frac{m}{2}+{}^1\!/_2\right)*B_t}{E*A_d\sin\theta^3}$ 7,54 mm

Deflection due to elastic deformation of truss chords:

$dy_c = \dfrac{5*n*P*L^3}{384*E*I_0}$ 20,98 mm

Moment of inertia for truss chords:
$I_0 = (B_t^2/2*A_c + 2I_a)*1.25$ = 7,1829E+09 mm^4

f= $\Sigma dy_d + dy_c$ = **28,5** mm

12 Equivalent surface member

Stiffness Matrix Elements(Membrane)
D_{66}= 91840 kN/m
D_{88}= 1090 kN/m

The following figure shows the general stiffness matrix of an orthotropic surface in RFEM.

$$\begin{bmatrix} m_x \\ m_y \\ m_{xy} \\ v_x \\ v_y \\ n_x \\ n_y \\ n_{xy} \end{bmatrix} = \begin{bmatrix} D_{11} & D_{12} & D_{13} & 0 & 0 & D_{16} & D_{17} & D_{18} \\ & D_{22} & D_{23} & 0 & 0 & D_{26} & D_{27} & D_{28} \\ & & D_{33} & 0 & 0 & D_{36} & D_{37} & D_{38} \\ & & & D_{44} & D_{45} & 0 & 0 & 0 \\ & & & & D_{55} & 0 & 0 & 0 \\ & & \text{sym.} & & & D_{66} & D_{67} & D_{68} \\ & & & & & & D_{77} & D_{78} \\ & & & & & & & D_{88} \end{bmatrix} \cdot \begin{bmatrix} \kappa_x \\ \kappa_y \\ \kappa_{xy} \\ \gamma_{xz} \\ \gamma_{yz} \\ \varepsilon_x \\ \varepsilon_y \\ \gamma_{xy} \end{bmatrix} \quad (4$$

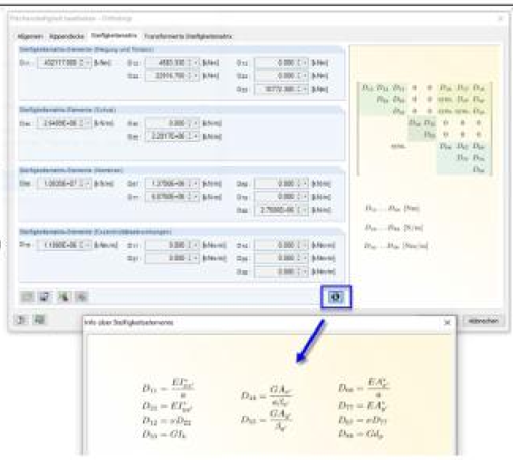


Figure 34. Excel calculation for the case when sheet on the purlins (page 3)

7 VERIFICATION AND COMPARISON OF THE RESULTS

7.1 Varification design examples by Excel design tool

The main target is to develop the Excel calculation to be a design tool. It is worth to test that the Excel works properly in different cases. Therefore, three examples from previous research are used to verify the Excel results. The results obtained by Excel are compared with the results from the original books.

Example 1

This example is referred from the Example 5.1 in the ECCS book, 2012, P.426-435. The corrugated steel sheet is connected with the purlin (the roof sheeting on the purlins). The entire Excel calculation is attached in Appendix A.1.

The building´s geometry is shown in Figure 35. Portal frames of 12 m span, at 5 m centers and purlins at 1.5 m centers. The shear flexibility of single panel which is calculated by Excel is 0,382 mm/kN and the middle span deflection is 25,8mm. See Figure 36. The shear flexibility obtained by ECCS is 0,39 mm/kN and deflection is 26,3mm.

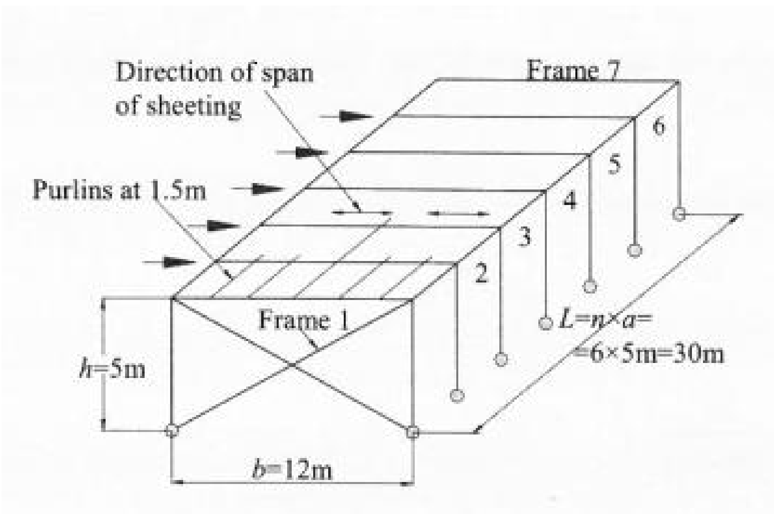


Figure 35. Flat roof building with roof sheets on purlins (ECCS, 2012)

08 Comonents of shear flexibility:

(a) Flexibility due to profile distortion $c_{1,1}$ = 0,321 mm/kN

(b) Flexibility due to shear strain $c_{1,2}$ = 0,007 mm/kN

(c) Flexibility due to deformation in sheet to purlin fasteners
$c_{2,1}$ = 0,002 mm/kN

(d) Flexibility due to deformation in seam fasteners
$c_{2,2}$ = 0,029 mm/kN

(e) Flexibility in connections to shear connectors
$c_{2,3}$= 0,002 mm/kN

c'= 0,361 mm/kN

(f) Equivalent shear flexibility due to axial strain in purlins
c_3= 0,021 mm/kN

Total shear fexibility c = 0,382 mm/kN

Shear flexibility due to:		Sheets on purlins
		Shear flexibility mm/kN
Sheet deformation	Profile distortion	$c_{1.1} = \frac{ad^{2.5}\alpha_1\alpha_4 K}{Et^{2.5}b^2}$
	Shear strains	$c_{1.2} = \frac{2a\alpha_2(1+v)(1+\left(\frac{2h}{d}\right))}{\mathrm{Etb}}$
Fastener deformation	Sheet to pulin	$c_{2.1} = \frac{2as_p p\alpha_3}{b^2}$
	Seam fasteners	$c_{2.2} = \frac{2s_s s_p(n_{sh}-1)}{2n_s s_p + \beta_1 n_p s_s}$
	Connections to Rafters	$c_{2.3} = \frac{4(n+1)s_{sc}}{n^2 n_s'}$
Flange forces	Axial strain in purlins	$c_3 = \frac{n^2 a^3 \alpha_3}{4.8EAb^2}$
Total shear flexibility		$c = c_{1.1} + c_{1.2} + c_{2.1} + c_{2.2} + c_{2.3} + c_3$

09 Mid-span deflection

Δ=P(n^2/8)*c= 25,8 mm < Δ_{max}= H/300= 16,67 mm

Figure 36. Flexibility of single shear panel and total deflection calculated by Excel

Example 2

This example came from example 1 in the book stabilization by stressed skin diaphragm action (Höglund, 2002, P.76). This case is the roof sheeting on the rafters. The entire Excel calculation is attached in Appendix A.2.

The building's geometry is shown in Figure 37. b_{roof} is 25m and the distance between the rafters are different. C_{rafter} is 7 m and $C_{rafter2}$ is 5m. Here, it is practical to use 7m so that the Excel can be applied. The shear flexibility of single panel which is calculated by Excel is 0,165 mm/kN and the middle span deflection is 58mm, which can be seen in Figure 38. The shear flexibility obtained by Höglund is 0,160 mm/kN and deflection is 50,29mm.

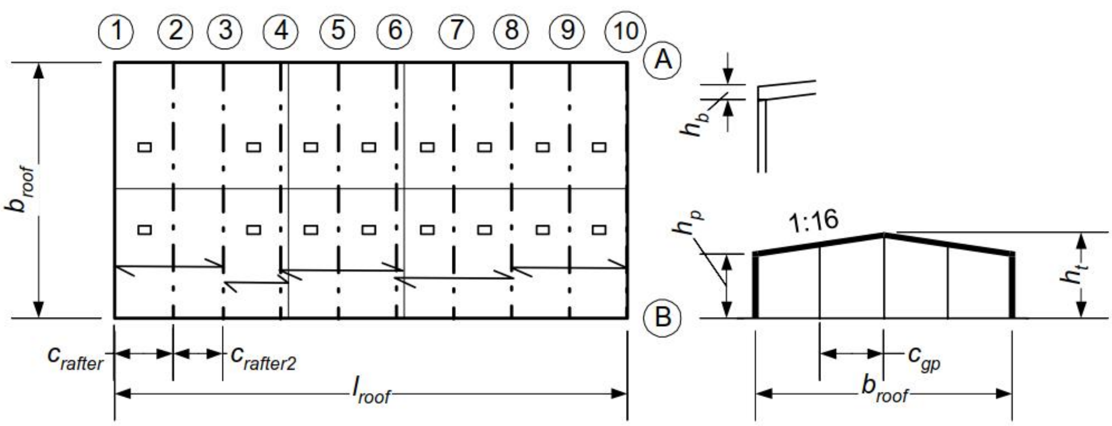


Figure 37. Building with roof sheets on rafters (Höglund, 2002)

08 Comonents of shear flexibility:

(a) Flexibility due to profile distortion	$c_{1,1}$ =	1,340 mm/kN
(b) Flexibility due to shear strain	$c_{1,2}$ =	0,134 mm/kN
(c) Flexibility due to deformation in sheet to purlin fasteners	$c_{2,1}$ =	0,085 mm/kN
(d) Flexibility due to deformation in seam fasteners	$c_{2,2}$ =	0,337 mm/kN
(e) Flexibility in connections to shear connectors	$c_{2,3}$=	0,007 mm/kN
	c'=	1,903 mm/kN
(f) Equivalent shear flexibility due to axial strain in purlins	c_3=	0,016 mm/kN
Total shear fexibility	c =	0,165 mm/kN

Shear flexibility due to:		Sheets on purlins Shear flexibility mm/kN
Sheet deformation	Profile distortion	$c_{1.1} = \frac{ad^{2.5}\alpha_1\alpha_4 K}{Et^{2.5}b^2}$
	Shear strains	$c_{1.2} = \frac{2a\alpha_2(1+v)(1+\left(\frac{2h}{d}\right))}{\mathrm{Etb}}$
Fastener deformation	Sheet to pulin	$c_{2.1} = \frac{2as_p p\alpha_3}{b^2}$
	Seam fasteners	$c_{2.2} = \frac{2s_s s_p(n_{sh}-1)}{2n_s s_p + \beta_1 n_p s_s}$
	Connections to Rafters	$c_{2.3} = \frac{4(n+1)s_{sc}}{n^2 n_s{}'}$
Flange forces	Axial strain in purlins	$c_3 = \frac{n^2 a^3 \alpha_3}{4.8EAb^2}$
Total shear flexibility		$c = c_{1.1} + c_{1.2} + c_{2.1} + c_{2.2} + c_{2.3} + c_3$

09 Mid-span deflection

Δ=P(n^2/8)*c= 58,0 mm < Δ_{max}= H/150= 58,53 mm

Figure 38. Flexibility of single shear panel and total deflection calculated by Excel

Example 3

This example is also refered from example 2 in the book stablilisation by stressed skin diaphragm action (Höglund, 2002, P.89). This case is the roof sheeting on the purlins. The entire Excel calculation is attached in Appendix A.3.

The building's geometry is shown in Figure 39. $b_{roof}\ is\ 36m$ and $C_{rafter}\ is\ 7{,}2\ m$. The distance between the purlin is 2m. In this case, the diaphragm shall be considered as two desperate interacting diaphragms because the roof sheets do not continue at roof ridge. The shear flexibility of single panel which is calculated by Excel is 0,063 mm/kN and the middle span deflection is 12,5mm. See Figure 40. The shear flexibility obtained by Höglund is 0,064 mm/kN and deflection is 12,74mm.

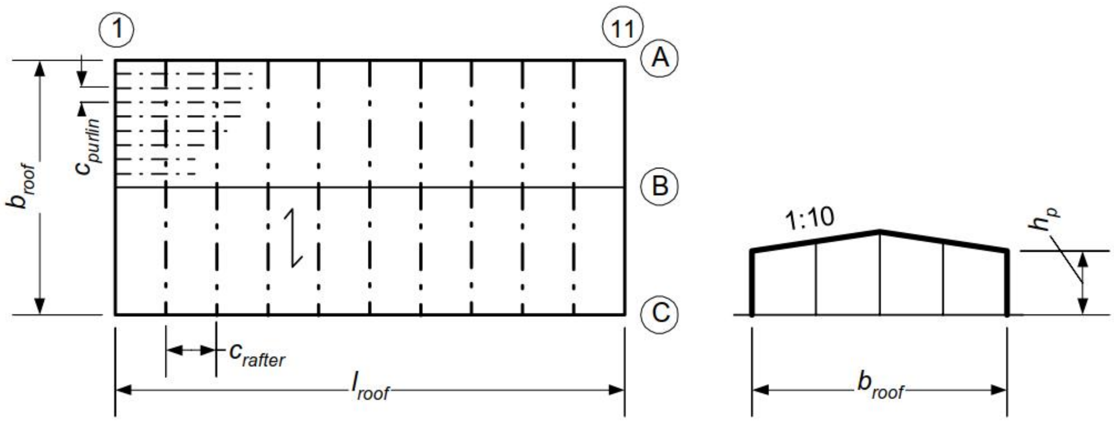


Figure 39. Building with roof sheets on purlins (Höglund, 2002)

08 Comonents of shear flexibility:

(a) Flexibility due to profile distortion	$c_{1,1}$ =	0,004 mm/kN
(b) Flexibility due to shear strain	$c_{1,2}$ =	0,005 mm/kN
(c) Flexibility due to deformation in sheet to purlin fasteners	$c_{2,1}$ =	0,001 mm/kN
(d) Flexibility due to deformation in seam fasteners	$c_{2,2}$ =	0,015 mm/kN
(e) Flexibility in connections to shear connectors	$c_{2,3}$=	0,006 mm/kN
	c'=	0,029 mm/kN
(f) Equivalent shear flexibility due to axial strain in purlins	c_3=	0,034 mm/kN
Total shear fexibility	c =	0,063 mm/kN

Shear flexibility due to:		Sheets on purlins Shear flexibility mm/kN
Sheet deformation	Profile distortion	$c_{1.1} = \frac{ad^{2.5}\alpha_1\alpha_4 K}{Et^{2.5}b^2}$
	Shear strains	$c_{1.2} = \frac{2a\alpha_2(1+v)(1+\left(\frac{2h}{d}\right))}{\mathrm{Etb}}$
Fastener deformation	Sheet to pulin	$c_{2.1} = \frac{2as_p p\alpha_3}{b^2}$
	Seam fasteners	$c_{2.2} = \frac{2s_s s_p(n_{sh}-1)}{2n_s s_p + \beta_1 n_p s_s}$
	Connections to Rafters	$c_{2.3} = \frac{4(n+1)s_{sc}}{n^2 n_s'}$
Flange forces	Axial strain in purlins	$c_3 = \frac{n^2 a^3 \alpha_3}{4.8EAb^2}$
Total shear flexibility		$c = c_{1.1} + c_{1.2} + c_{2.1} + c_{2.2} + c_{2.3} + c_3$

09 Mid-span deflection

Δ=P(n^2/8)*c= 12,5 mm < Δ_{max}= H/300= 29,33 mm

Figure 40. Flexibility of single shear panel and total deflection calculated by Excel

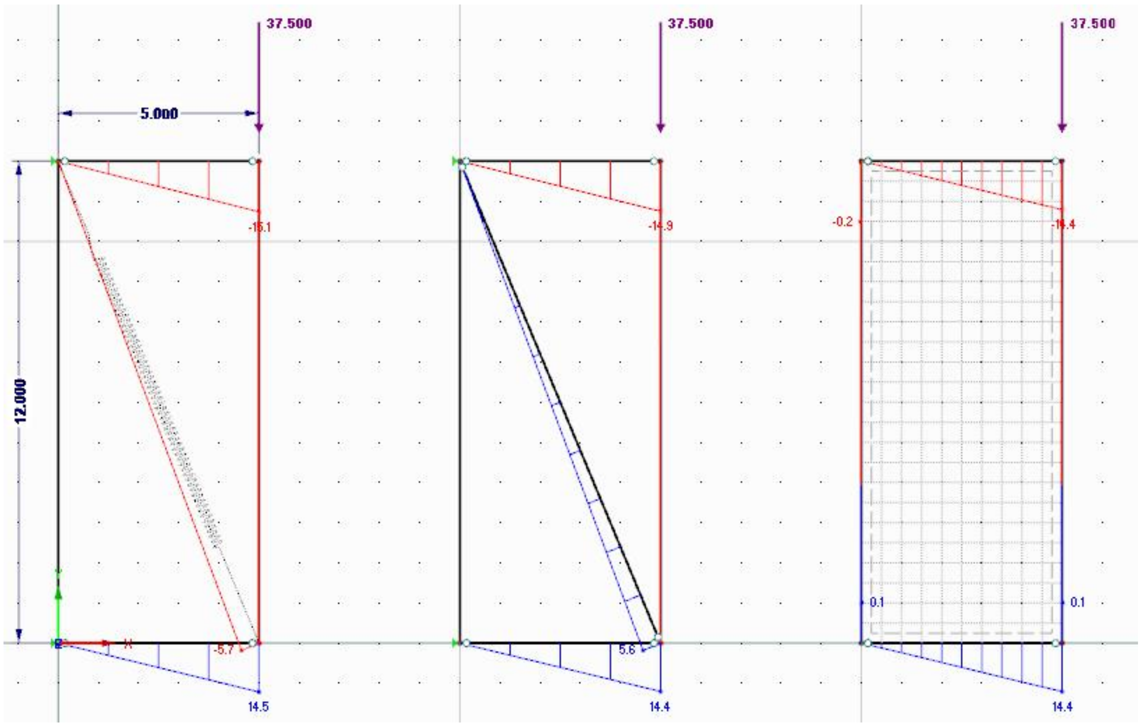
7.2 Verification design examples by RFEM analysis

The RFEM analysis model with different method are conducted to verify the Excel calculation. The same design examples as previous section are used in this section.

The displacement under the horizontal loads can be calculated by RFEM. By comparing the results with Excel, it is easy to figure out how excel is working.

Example 1

The deflection of single shear panel under the shear force was illustrated in Figure 41. As it shows, the results of the different modelling way are similar. The flexibility of the shear panel is 14,4mm/37,5kN= 0,384mm/kN, which is same as the excel calculation.



(a) Spring (b) Bar (c) Surface

Figure 41. Deflection of single shear panel

The whole roof with different methods are modelled in RFEM. It is notice that the middle span deflections of the whole roof panel are 25,8mm to 28,3mm, which are demonstrated as Figure 42. The displacements of the first shear panel are 14,0 mm to 14,7mm, which is same as the results of test by single panel.

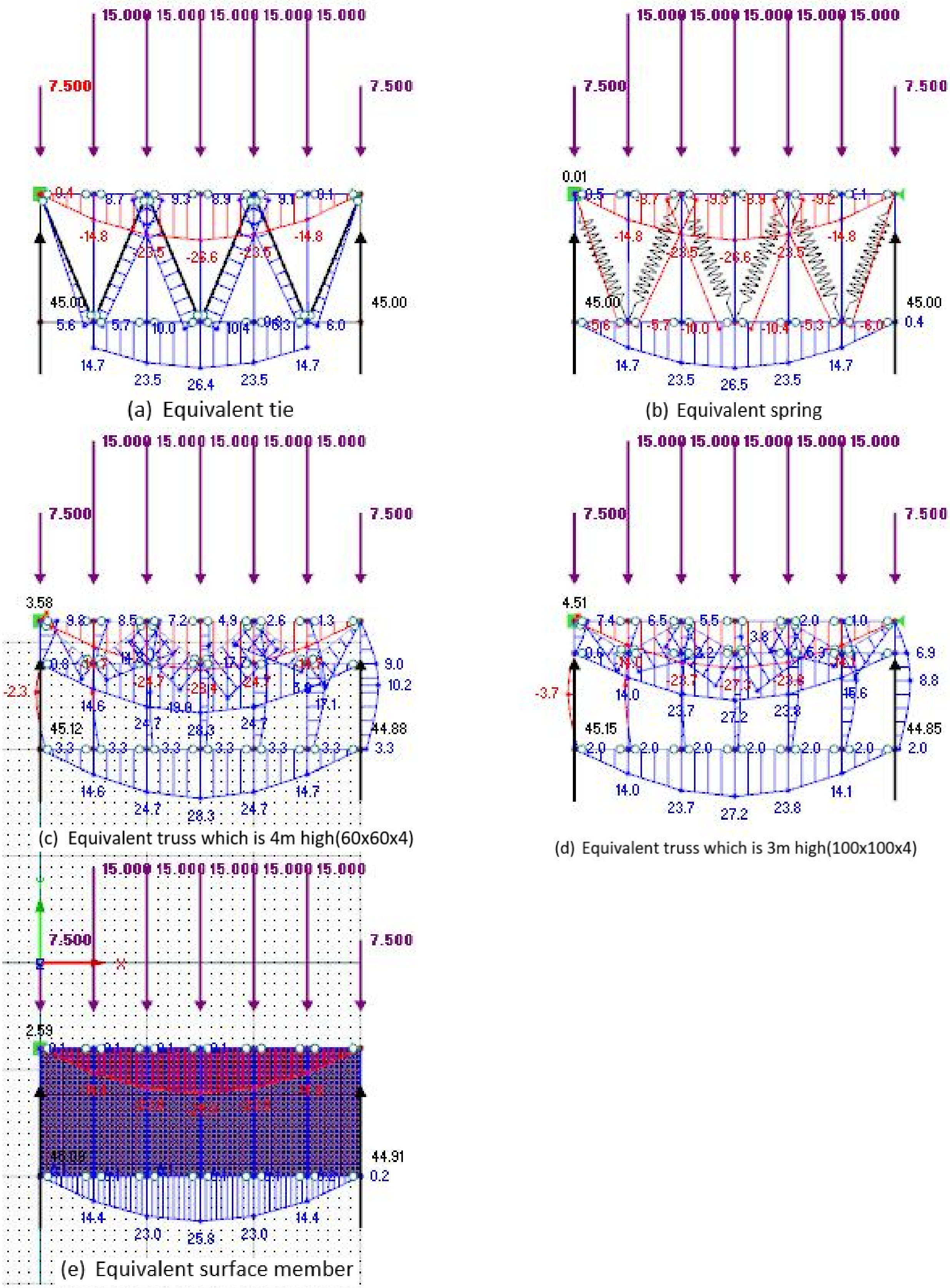
Example 2

Same procedure for example 2. Total deflections of the whole roof panel calculated by RFEM is 53,2mm to 58,8mm, which are illustrated as Figure 43.

Example 3

Total deflections of the whole roof panel calculated by RFEM is 11,8mm to 13,3mm, which are revealed as Figure 44.

The comparison of the calculation results with different approaches for three design examples are summarized in Figure 45. As it is observed, the results obtained by RFEM are very similar to the Excel calculation. Therefore, it shows that the excel results are reliable.



(a) Equivalent tie

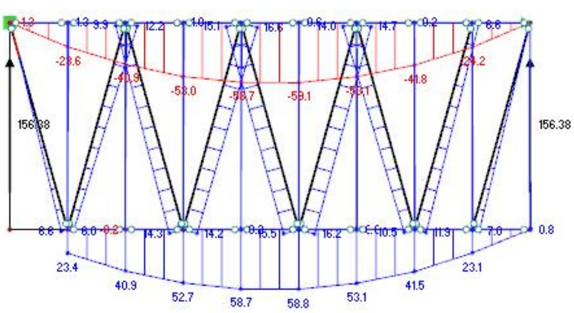
(b) Equivalent spring

(c) Equivalent truss which is 4m high(60x60x4)

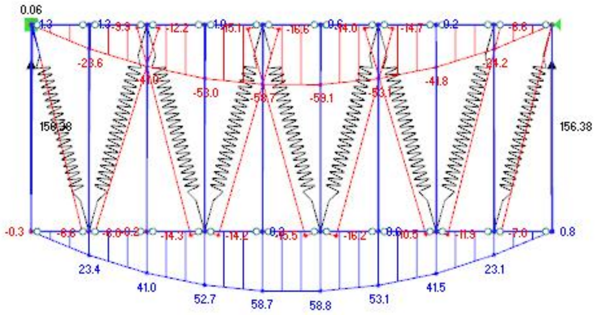
(d) Equivalent truss which is 3m high(100x100x4)

(e) Equivalent surface member

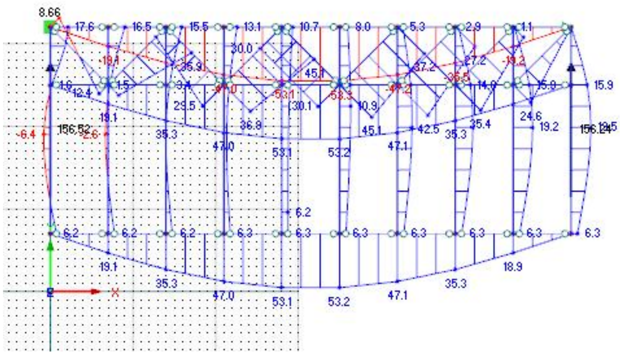
Figure 42. Deflection of the roof diaphragm calculated by Rfem for example 1 (2D)



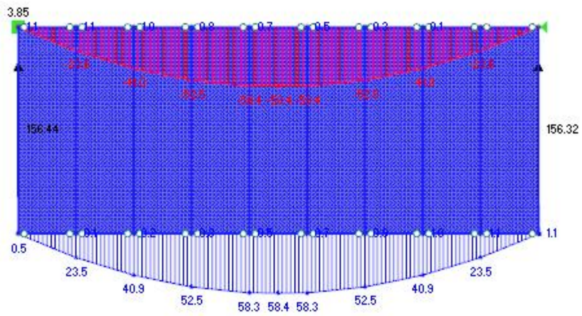
(a) Bars



(b) Springs

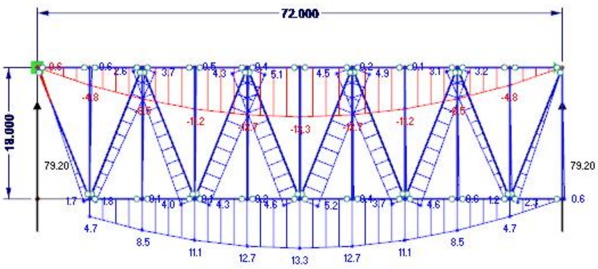


(c) Truss

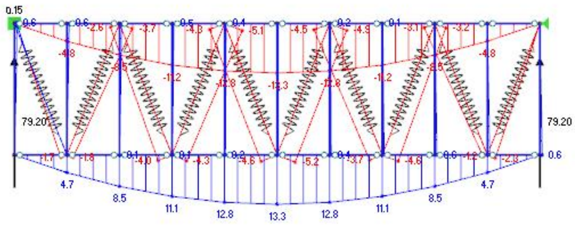


(d) Surfaces

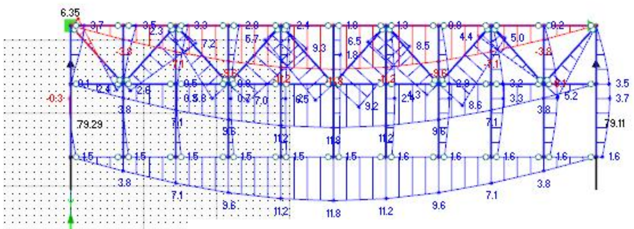
Figure 43. Deflection of the roof diaphragm calculated by RFEM for example 2 (2D)



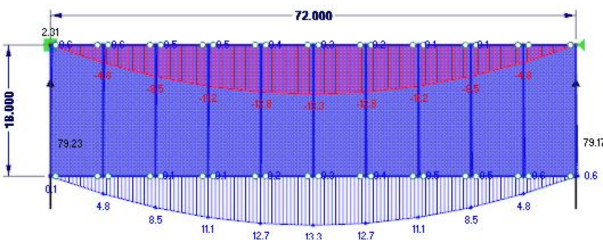
(a) Bars



(b) Springs



(c) Truss



(d) Surfaces

Figure 44. Deflection of the roof diaphragm calculated by RFEM for example 3 (2D)

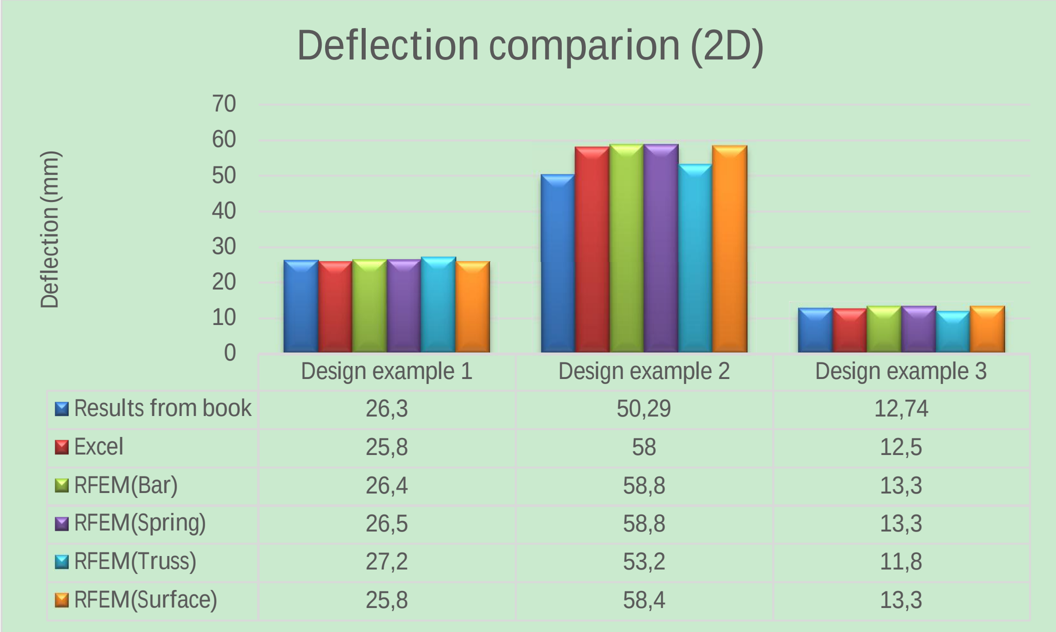


Figure 45. Comparison of deflection (2D)

7.3 Parameter study of stressed skin effect

It is proved that the Excel calculation is reliable as in previous section. Then the Excel can be used to carry out a parameter study.

The parameters of the roof sheet used in design examples are listed in Table 20 and Figure 16 indicates the basic dimension of the roof sheet. As it can be seen, the height of the roof sheet is 43mm, 112mm, 22mm. Usually, the height of the roof sheet is decided by the vertical loads and the distance between the supporting beams. Hence, the type of the roof sheet is not investigated here.

Case	Type of roof sheet	l	d	h	θ	h/d	l/d	K1	K2
Example 1	W-45JA/900	77	180	43	33,00	0,24	0,43	0,110	1,068
Example 2	TRP110	68	237	112	25,00	0,47	0,29	0,209	2,708
Example 3	TRP22	25	90	22	25,50	0,24	0,28	0,130	0,763

Table 20. The parameters of the roof sheet

There are a varies of factors influence the shear stiffness and the horizontal deflections in the diaphragm effect. It is not possible to study all of them. Therefore, three factors which have great impact to the stressed skin effect are investigated in this section: every or alternates corrugation fastened; thickness of the sheet; purlin or edge beam.

7.3.1 Every or alternates corrugation fastened

As it can be noticed from Table 20, that K1 for sheeting fastened in every troughs and K2 for sheeting fastened in alternate troughs have a great difference. Figure 46 shows the test results for both Every and alternates corrugation for all the design examples. It can be proved that every alternate trough fastened make the shear panel much more flexible than every trough fastened. In the example 2, the sheet is connected directly to the rafter and the roof panel is already so flexible that every trough fastened is the only choice in that circumstance.

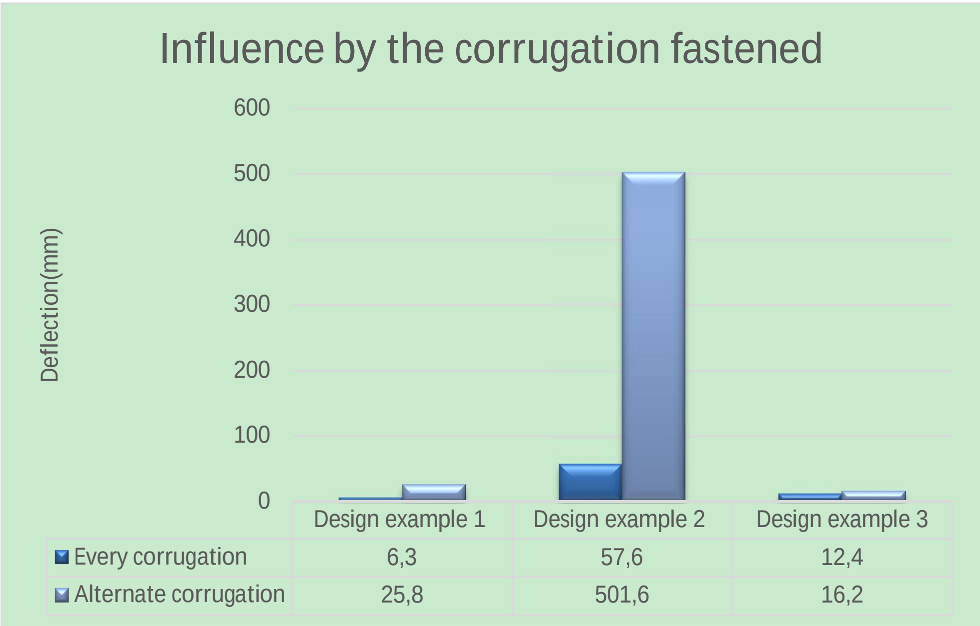


Figure 46. Influence by the corrugation fastened

7.3.2 Thickness of the roof sheet

The roof sheets with thickness of 0.5mm, 0.6mm, 0.7mm, 0.8mm,0.9mm and 1 mm are commonly used. Therefore, all these different thicknesses are tested for three design examples. Figure 47 illustrates the results. As it is observed, increasing of the thickness can make the displacement much smaller. Especially in example 1 and 2.

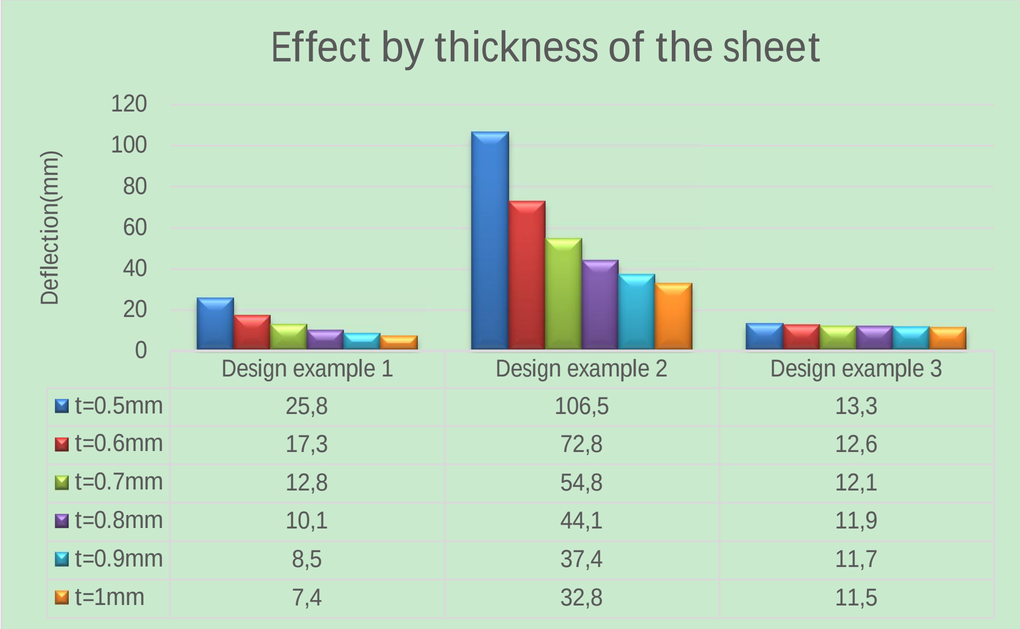


Figure 47. Influence by thickness of the roof sheet

7.3.3 Purlin or edge beam

IPE 80, IPE 100, IPE 120, IPE 140, IPE 160 are tested as the purlin or edge beam. The results are illustrated in Figure 48. It is proved that the size of the Purlin or edge beam plays an important role for stressed skin effect.

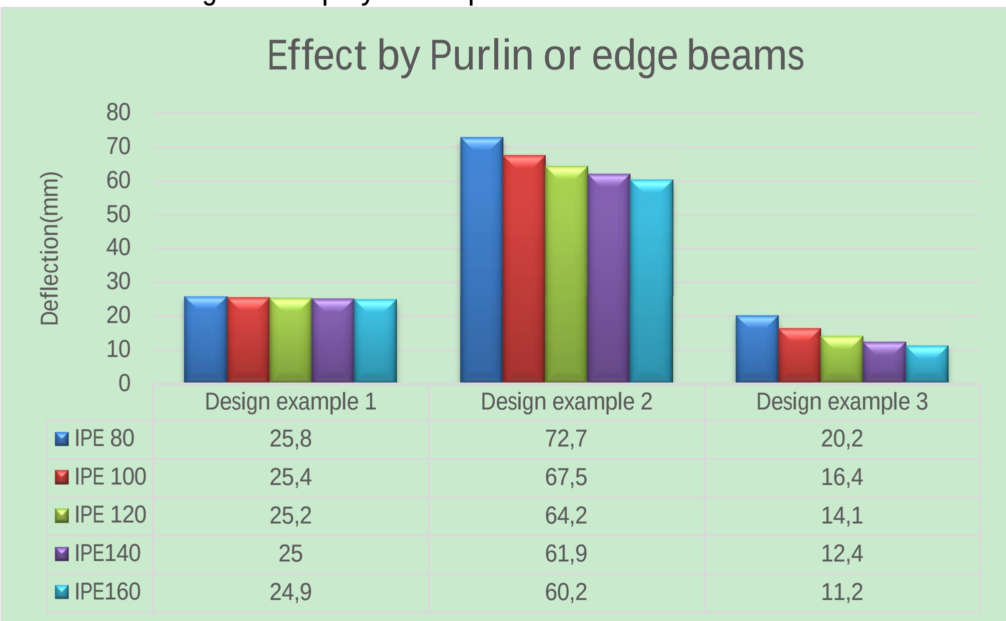


Figure 48. Influence by purlins or edge beams

7.4 Stressed skin behaviours for different load bearing structures

As it has been mentioned in Chapter 4, the main frame and diaphragm together carry all the horizontal loads. In order to figure out how the relative flexibility influence the final deflections, 3D analysis models for different load-bearing structures are executed. Three different structures for design example 1 are implemented in RFEM model. The deflections can be calculated as Figure 50 and the shear force of the columns and normal force of the surface panel are demonstrated in Figure 51.

Figure 49 summarizes the deflection of with and without the stressed skin effect. Firstly, it is clear that the shear panel distributes the horizontal loads evenly and displacement with considering stressed skin effect are smaller than the displacement without stressed skin effect for different load-bearing structures.

Secondly, stress skin effect has greatest influence in portal frame structure and has smallest influence in rigid frame structure.

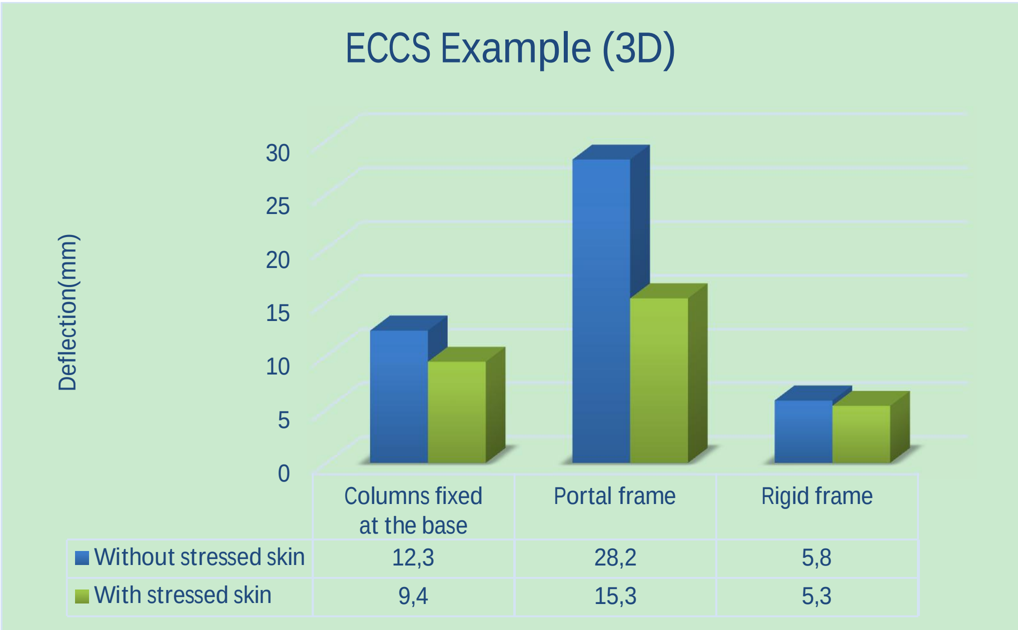


Figure 49. Comparison of the deflection for different structures.

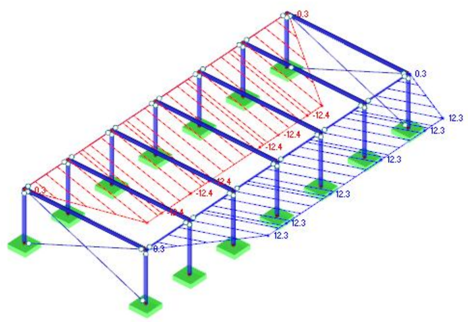
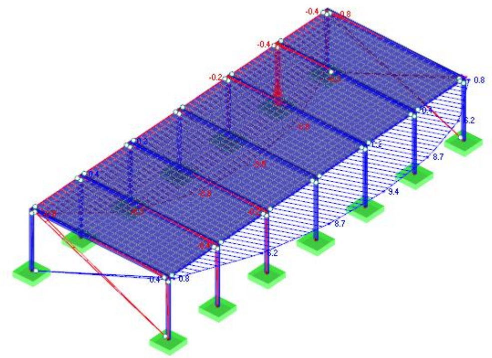
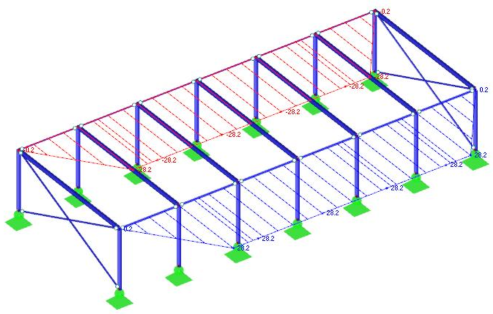
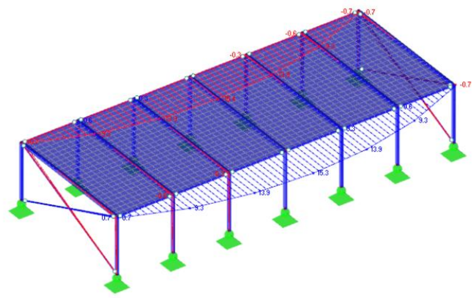
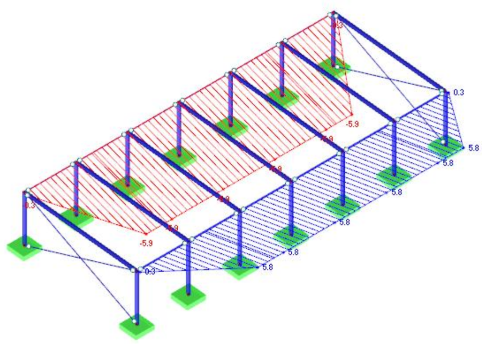
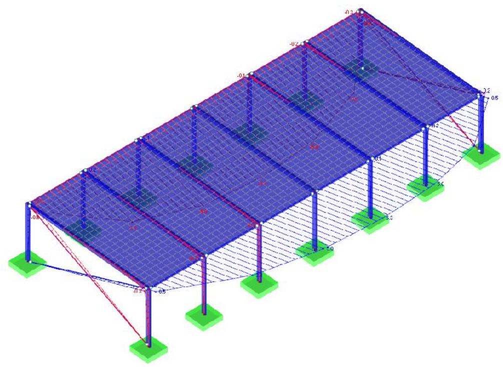
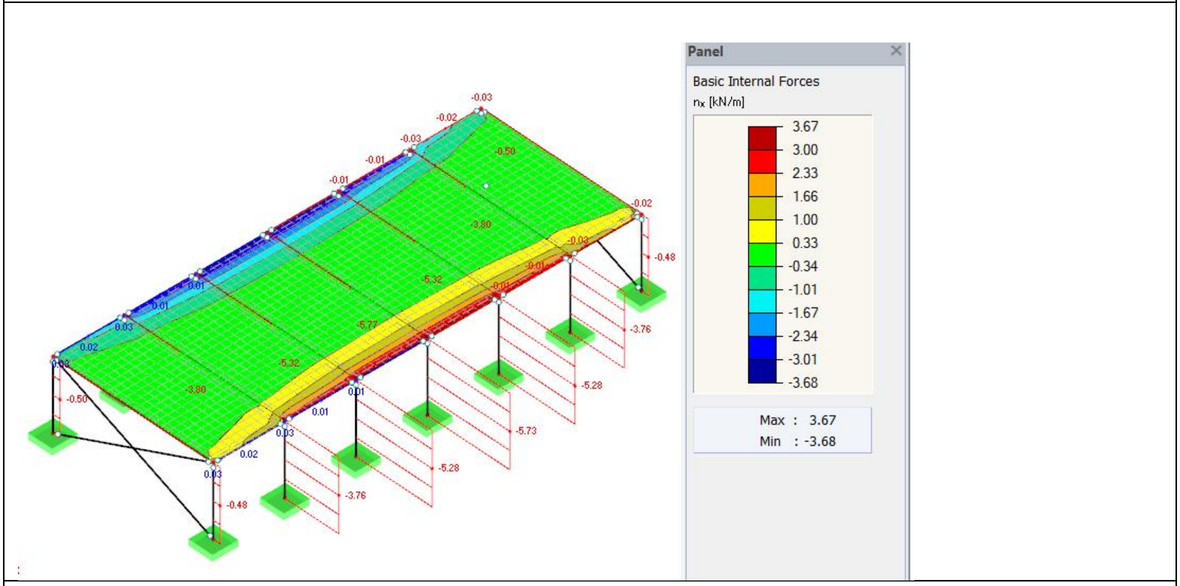
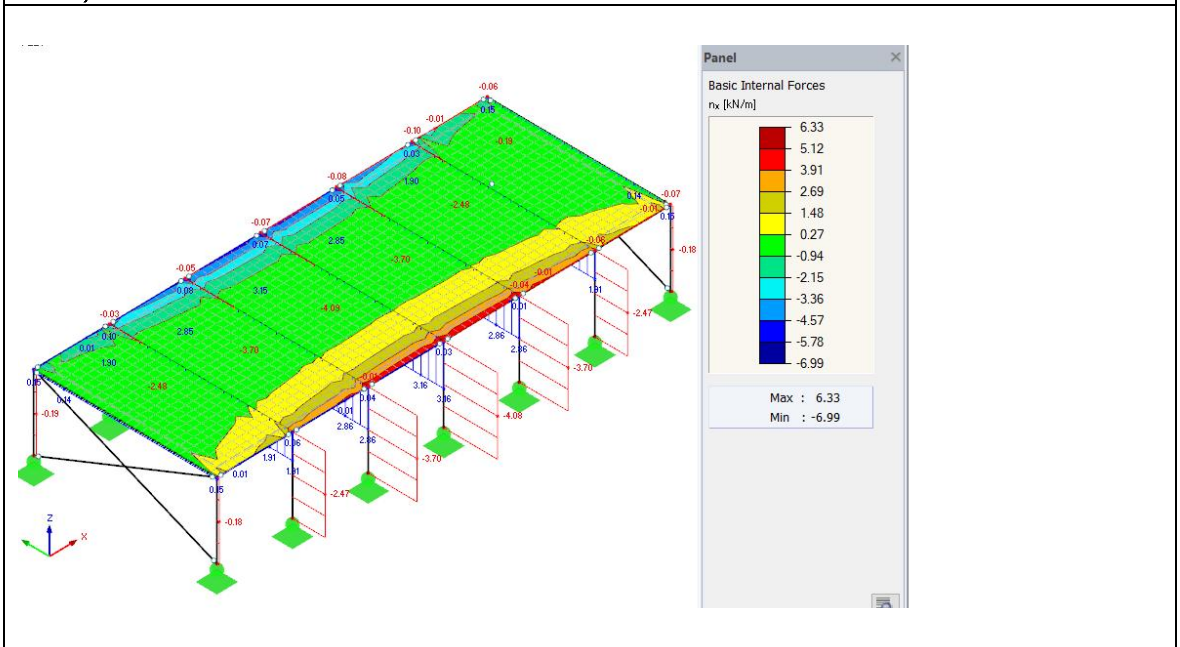
a) Columns fixed at the base	
	
Without stressed skin effect	With stressed skin effect
b) Portal frame	
	
Without stressed skin effect	With stressed skin effect
c) Rigid frame	
	
Without stressed skin effect	With stressed skin effect

Figure 50. Deflection comparison for different structures

a) Columns fixed at the base



b) Portal frame



c) rigid frame

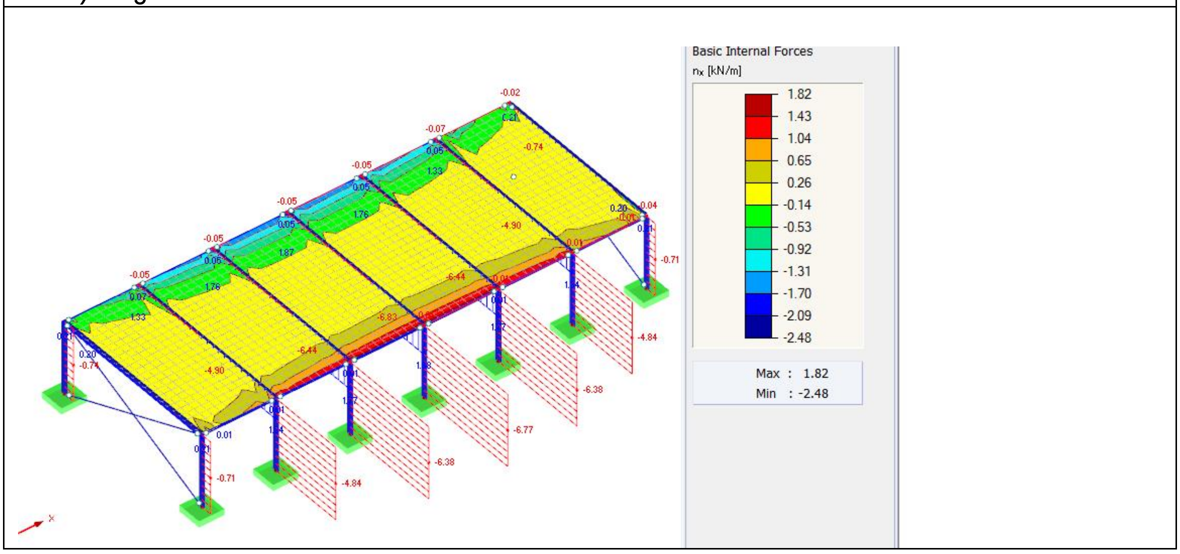


Figure 51. Shear force of the columns and the axial force of the diaphragm

8 CONCLUSIONS AND RECOMMENDATIONS

8.1 Conclusions

It is obvious that the corrugated steel sheet can significantly decrease the lateral deflection under the wind load or horizontal load. The lateral bracing system on the roof aiming to stabilize the whole structure can be replaced by the corrugated roof structure if the fasteners and the corrugated sheet have adequate strength and overall in plane deformation of roof is at acceptable level.

An Excel design tool has been developed to make the design work easier. It is the main goal of this thesis work. By using the Excel calculation table, the designer can get the initial data for the global analysis model. The roof stressed skin effect can be simplified into an equivalent horizontal truss structure at roof, using steel or spring members as diagonals. Another option is to use a surface diaphragm with the equivalent stiffness.

A parameter study is carried out by Excel in this thesis. Two advices are given here. Firstly, Roof sheets fastened in every or alternates corrugation have significantly different stiffness and strength for in plane loading. Sheet fastened in every corrugation is strongly suggested for the stressed skin design, especially when the sheets are connected directly to the rafters. Secondly, in situation when the type of the corrugated sheet has been chosen, increasing thickness of the sheet or increasing the size of the purlin or edge beam can benefit the overall situation.

The roof diaphragm is load-bearing structure, which is working together with the main frame to carry the horizontal load as well as vertical roof loads. The design of roof diaphragm is as important as the main frame. By considering the roof stressed skin effect in structural analysis, the internal force can be distributed properly, and the shear forces of the columns become smaller than without consideration of stressed skin effect. The stressed skin has most impact in the portal frame structures than the others.

8.2 Recommendations for future studies

Stressed skin design is a complicated issue in the real project and there are more than three factors affect the stressed skin effect, such as the number of the purlins, the number of the fasteners between the seams and so on. Therefore, deeply parameter study can be designed and performed.

How does the roof opening affect to the flexibility of the diaphragm is not discussed in this thesis. Nevertheless, it has a lot of engineering value to be studied furthermore.

Different roof structures or wall structures can be also investigated further, such as two layers of corrugated steel sheets plus insulation material inside.

References

Manual of Stressed skin Diaphragm Davies, J. M. and Bryan, E. R., 1982.
Stabilization by stressed skin diaphragms action -Torsten Höglund, 2002
Eurocode 3: 1993-1-1 Design of steel structures. General rules and rules for buildings.
Eurocode 3: 1993-1-3 General rules. Supplementary rules for cold formed members and sheeting.
Design of Cold-formed Steel Structures, Dubina, Ungureanu and Landolfo, 2012
ECCS Publication No.88(1995): European recommendations for the application of metal sheeting acting as a diaphragm
European Recommendations on the stabilization of steel structures by Sandwich panels.

APPENDIX

A.1 Excel calculation regarding sheets on purlin (Design example 1)

A.2 Excel calculation regarding sheets on main beams (Design example 2)

A.3 Excel calculation regarding sheets on purlin (Design example 3)

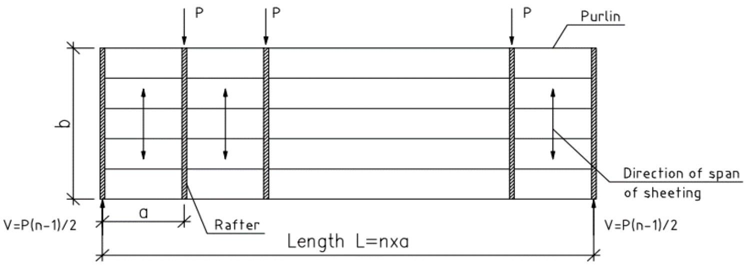
A.4 Sheet and fastener design example made by Poimu (Ruukki software)

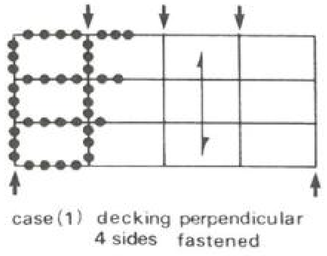
A.5 Values of K1 for fasteners in every trough

A.6 Values of K2 for fasteners in every alternate trough

SWECO	A.1 Excel calcualtion regarding sheets on purlin(Design example 1)	Designer	Phase / Area	Project nr
			Engineering	
Project:			Date	Page
Subject:	Corrugated sheet as stressed skin, deformation calculation		1.5.2020	

STRESSED SKIN DESIGN ACCORDING TO ECCS 'Design of Code-formed Steel Structure' (2012)





Input data

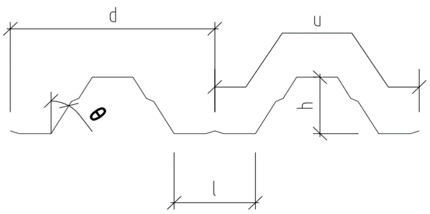
01 Data of building (main frame):

a=	5	m	Length of diaphragm	
b=	12	m	Depth of diaphragm	
H=	5	m	Height of building	
n=	6		Number of panels within diaphragm assebly	
L=	30	m	Length of building	L=n*a

02 Wind load on side of building

q_w=	1,20	kN/m2	Peak velocity pressure (EN 1991-1-4)	
g_Q =	1,5		Partial load factor	
p=	3,00	kN/m	Wind load at edge of the roof	$p=q_w*H*0,5$
p_d=	4,50	kN/m	Design wind load	pd=p*rQ
P=	15,00	kN	Point load on top of the building column	P=p*a

03 Choosed roof profile: **W-45JA/900_Narrow flange against the support** **(Based on results of roof sheet calculations)**



Steel grade:	S350GD+Z			
f_y=	350	N/mm2	Basic yield strength	
f_u=	420	N/mm2	Ultimate tensile strength	
t=	0,5	mm	Sheet thickness, based on roof sheet design	
t_{nom}=	0,46	mm	Nominal sheet thickness used in design	
E=	210	kN/mm2	Modulus of elasticity	
G=	81	kN/mm2	Shear modulus	
u=	0,3		Poisson's ratio	
u=	227	mm	Perimeter length of sheet corrugation	Second moment of area of a single corrugation
I_{y1}=	163,77	mm^4/mm	Second moment of area(from manufacture)	$I_{y1}=I_y*d=$ 29479 mm^4
d=	180	mm	Pitch of corrugation	
l=	77	mm	See the picture on the right	Sheet profile distortion factor
h/d=	0,24		See the picture on the right	
K_1=	0,110		Fasteners in every corrugation	
K_2=	1,068		Fasteners in every alternate corrugation	

Sheet profile distortion factor

K	p(mm)
0,110	180
1,068	360

04 Choosed Purlin: **IPE 80** cp= 1,5 m Distance between the purlins

Steel grade:	S355					
f_y=	355	N/mm2	Basic yield strength	t_p=	5,2	mm
f_u=	510	N/mm2	Ultimate tensile strength			
E=	210	kN/mm2	Modulus of elasticity			
G=	81	kN/mm2	Shear modulus			
u=	0,3		Poisson's ratio			
A=	764	mm^2	Cross-section			

05 Fastener(Types and amounts)

(a) Sheet to purlin connection **Screws collar head 5,5**

K=	1,068		**Fasteners in every alternate corrugation**	
p=	360,000	mm	Distance between fasteners in connection to purlin	
F_p=	2,02	kN	Design shear resistance of individual sheet-to-purlin fastener	Table 12
s_p =	0,15	mm/kN	Slip(flexibility) per sheet to purlin fastener per unit load.	Table 12
n_f=	4		Number of sheet to purlin fasteners per sheet width. This can be calculated based on d and sheet type.	
n_p=	9		Total number of purlins(Edge+intermediate) width of roof, b	
n_b=	1		Number of sheet lengths within the lengths of diaphragm	
n_{sh}=	6		Number of sheet widths per pannel a	

(b) Seam fasteners **Screw 4,8** Seam fastners in the crests

F_s=	0,87	kN	Design shear resistance of individual seam fastener	Table 12

A.1 Excel calcualtion regarding sheets on purlin(Design example 1)	Designer	Phase / Area	Project nr
		Engineering	
Project:		Date	Page
Subject: Corrugated sheet as stressed skin, deformation calculation		1.5.2020	

$s_s =$	0,25 mm/kN	Slip(flexibility) per seam fastener per unit load	Table 12
n_s=	40	Number of seam fasteners per side lap(excluding those which pass through both shees and the	

α_1=	0,6	β_1 =	0,44	Non-dimensional factors
α_2=	0,4	β_2 =	1,11	α1, α2, α3 depend on n_p (Table 7)
α_3=	0,53	β_3 =	0,75	β1, β2 depend on n_f (Table 5)
α_4=	1			α4 (Table 8)

(c) Sheet to shear connector fasteners **Screws collar head 5,5** Shear connectors transfer forces between steel sheet diaphragm and building frame

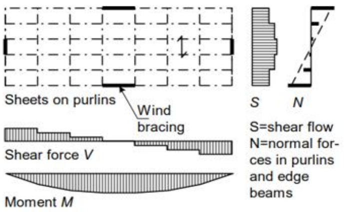
F_{sc}=	2,02 kN	Design shear resistance of individual sheet-to-shear connector fastener	Table 12
s_{sc}=	0,15 mm/kN	Slip(flexibility) per sheet to shear connector fastener per unit load	Table 12
n_{sc}=	49,00	Number of sheet to shear connector fasteners per end rafter	
$n_{sc'}$=	49,00	Number of sheet to shear connector fasteners per intermediate rafter	

Result

06 Actions on diaphragm due to wind

(a) **Wind on long side**

R_d=	67,50 kN	Support force at gable of the building
N_d=	24,47 kN	Normal force in edge beam at long sides of building
V_{dmax}=	5,39 kN/m	Maximun shear flow



(b) Wind on end (Only for checking the shear flow of the sheet)

R_{dl}=	27,00 kN	Support force in long sides
N_{dl}=	10,13 kN	Normal force in end rafter
N_g=	2,61 kN	Normal force in edge beam

Note: When the wind is on the end of the building, the stressed skin diaphragm height is 2/3*a.

07 Diaphragm strength **Utility grade**

(a) Seam strength	$V_{Rd.1}$=	45,38 kN	$R_d/V_{Rd.1}$ =	1,49 Check number and type of fasteners	FAILURE
(b) Strength in fasteners at shear connecto	$V_{Rd.2}$=	98,93 kN	$R_d/V_{Rd.2}$ =	0,68	OK
(c) Global shear buckling resistance:	Dx=	1,48 kN*mm²/mm			
	Dy=	34391,7 kN*mm²/mm			
	$V_{cr.g}=\frac{14,4}{b}Dx^{\frac{1}{4}}Dy^{\frac{3}{4}}(n_p-1)^3$=	1712,66 kN	$R_d/V_{cr.g}$ =	0,04	OK

(d) Local shear buckling resistance:

$V_{cr.l}$=4,83$btE(\frac{t}{l})^2$= 199,82 kN $R_d/V_{cr.l}$ = **0,34** OK

(e) The interaction of global and local shear buckling = Resistance for shear buckling:

$V_{red} = \frac{V_{cr.g}*V_{cr.l}}{V_{cr.g}+V_{cr.l}}$= 178,94 kN R_d/V_{red} = **0,38** OK

(f) Sheet-to-purlin fasteners:

$V_{Rd.4} = \frac{0,6bF_p}{pa3}$= 76,19 kN $R_d/V_{Rd.4}$ = **0,89** OK

(g) End collapse of sheeting profile

$V_{Rd.5} = 0.9f_yb\sqrt{\frac{t^3}{d}}$= kN $R_d/V_{Rd.5}$ = **2,304** FAILURE

$V_{Rd.5} = 0.3f_yb\sqrt{\frac{t^3}{d}}$= 29,30 kN Fasteners in every second corrugation

Design shear capacity V_{Rd}= **45,38 kN**

08 Comonents of shear flexibility:

(a) Flexibility due to profile distortion	$c_{1,1}$ =	0,321 mm/kN
(b) Flexibility due to shear strain	$c_{1,2}$ =	0,007 mm/kN
(c) Flexibility due to deformation in sheet to purlin fasteners	$c_{2,1}$ =	0,002 mm/kN
(d) Flexibility due to deformation in seam fasteners	$c_{2,2}$ =	0,029 mm/kN
(e) Flexibility in connections to shear connectors	$c_{2,3}$=	0,002 mm/kN
	c'=	0,361 mm/kN
(f) Equivalent shear flexibility due to axial strain in purlins	c_3=	0,021 mm/kN
Total shear fexibility	c =	0,382 mm/kN

Shear flexibility due to:		Sheets on purlins
		Shear flexibility mm/kN
Sheet deformation	Profile distortion	$c_{1.1} = \frac{ad^{2.5}\alpha_1\alpha_4K}{Et^{2.5}b^2}$
	Shear strains	$c_{1.2} = \frac{2a\alpha_2(1+v)(1+(\frac{2h}{d}))}{Etb}$
Fastener deformation	Sheet to pulin	$c_{2.1} = \frac{2as_pp\alpha_3}{b^2}$
	Seam fasteners	$c_{2.2} = \frac{2s_ss_p(n_{sh}-1)}{2n_ss_p+\beta_1n_ps_s}$
	Connections to Rafters	$c_{2.3} = \frac{4(n+1)s_{sc}}{n^2n_s'}$
Flange forces	Axial strain in purlins	$c_3 = \frac{n^2a^3\alpha_3}{4.8EAb^2}$
Total shear flexibility		$c = c_{1.1}+c_{1.2}+c_{2.1}+c_{2.2}+c_{2.3}+c_3$

09 Mid-span deflection

Δ=P(n^2/8)*c= 25,8 mm > Δ_{max}= H/300= 16,67 mm

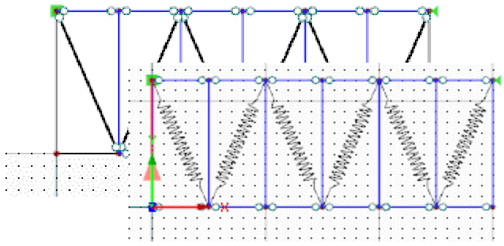
SWECO	A.1 Excel calcualtion regarding sheets on purlin(Design example 1)	Designer	Phase / Area	Project nr
			Engineering	
Project:			Date	Page
Subject:	Corrugated sheet as stressed skin, deformation calculation		1.5.2020	

10 Equivalent tie or equivalent spring member

Cross section area of equivalent tie	**A=**	**190,00**	mm2
Length of equivalent tie	L=	13	m
Modulus of elasticity of equivalent tie	E=	210	kN/mm2

Axial stiffness of the equivalent spring member

K= 3069 kN/mm



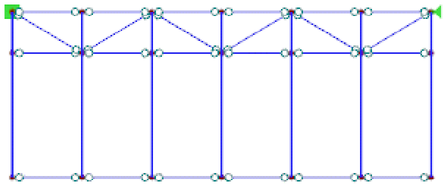
11 Equivalent steel truss, simplified calculation of deflection in the middle of the truss

Definition of truss geometry:

a= 5,00 m B_t= 3,00 m

n= 6 m L= 30,00 m

P= 15,0 kN



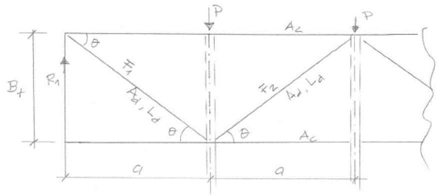
q =arctan (B_t/a) = 0,540 radians

30,963757 degrees

m= (n-1)/2 = 2,5

m= number forces P corresponding reaction R1

R1 = m*P = 37,5 kN



Selection of steel tubes in truss

Diagonal brace	**P100x100x4**	A_d =	**1495**	mm^2
Truss chord	**P100x100x4**	A_c =	**1495**	mm^2

$I_â$ = **2,264E+06** mm^4

Calculation of deflection:

Deflection due to elastic deformation on diagonal braces:

if 2m = even $\Sigma dy_d = \dfrac{P*(m^2-m+1)*B_t}{E*A_d \sin\theta^3}$ mm

if 2m = odd $\Sigma dy_d = \dfrac{P*\left(m^2-\frac{m}{2}+{}^1\!/_2\right)*B_t}{E*A_d \sin\theta^3}$ 5,79 mm

Deflection due to elastic deformation of truss chords:

$dy_c = \dfrac{5*n*P*L^3}{384*E*I_0}$ 21,32 mm

Moment of inertia for truss chords: f= $\Sigma dy_d + dy_c$ = **27,1** mm

$I_0 = (B_t^2/2*A_c + 2I_a)*1.25$ = 7,0686E+09 mm^4

12 Equivalent surface member

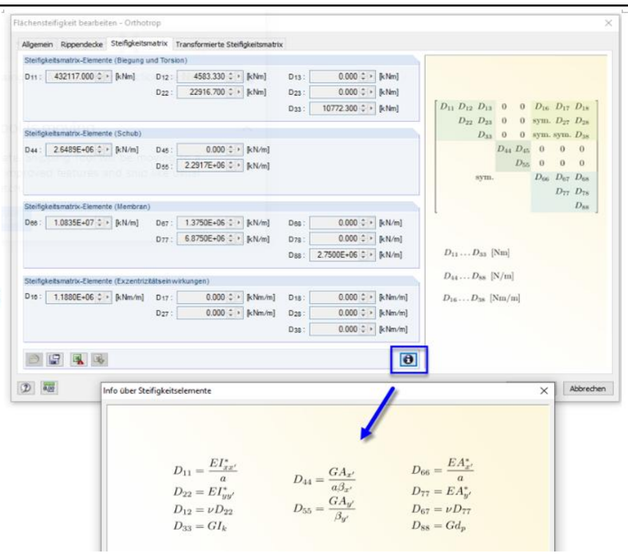
Stiffness Matrix Elements(Membrane)

D_{66}= 107008 kN/m

D_{88}= 1090 kN/m

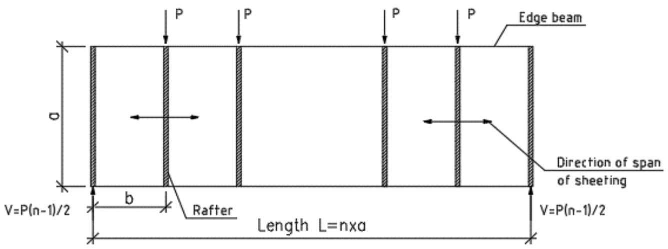
The following figure shows the general stiffness matrix of an orthotropic surface in RFEM.

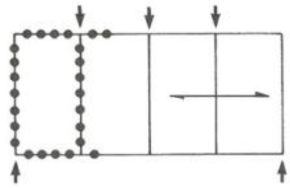
$$\begin{bmatrix} m_x \\ m_y \\ m_{xy} \\ v_x \\ v_y \\ n_x \\ n_y \\ n_{xy} \end{bmatrix} = \begin{bmatrix} D_{11} & D_{12} & D_{13} & 0 & 0 & D_{16} & D_{17} & D_{18} \\ & D_{22} & D_{23} & 0 & 0 & D_{26} & D_{27} & D_{28} \\ & & D_{33} & 0 & 0 & D_{36} & D_{37} & D_{38} \\ & & & D_{44} & D_{45} & 0 & 0 & 0 \\ & & & & D_{55} & 0 & 0 & 0 \\ & & \text{sym.} & & & D_{66} & D_{67} & D_{68} \\ & & & & & & D_{77} & D_{78} \\ & & & & & & & D_{88} \end{bmatrix} \cdot \begin{bmatrix} \kappa_x \\ \kappa_y \\ \kappa_{xy} \\ \gamma_{xz} \\ \gamma_{yz} \\ \varepsilon_x \\ \varepsilon_y \\ \gamma_{xy} \end{bmatrix} \quad (4.1$$



SWECO	A.2 Excel calcualtion regarding sheets directly on main beams	Designer	Phase / Area	Project nr
			Engineering	
Project:			Date	Page
Subject:	Corrugated sheet as stressed skin, deformation calculation		12.1.2020	

STRESSED SKIN DESIGN ACCORDING TO ECCS 'Design of Code-formed Steel Structure' (2012)





case (2) decking parallel
4 sides fastened

Input data

01 Data of building (main frame):

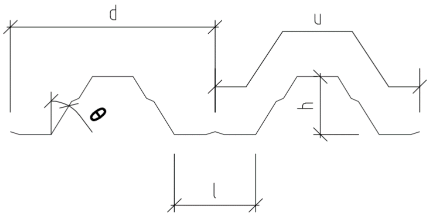
a=	25	m	Length of diaphragm	
b=	7	m	Depth of diaphragm	
H=	8,78	m	Height of building	
n=	9		Number of panels within diaphragm assebly	
L=	63	m	Length of building	L=n*a

02 Wind load on side of building

q_w=	1,13	kN/m2	Peak velocity pressure(EN 1991-1-4)		
g_Q =	1,5		Partial load factor		
p=	**4,96**	kN/m	Wind load at edge of the roof	$p=q_w*H*0,5$	(This load valuse was made
p_d=		kN/m	Design wind load	pd=p*rQ	according to the original book)
P=	34,72	kN	Point load on top of the building column	P=p*a	

03 Choosed roof profile: **TRP110** **(Based on results of roof sheet calculations)**

Steel grade: S350GD+Z



f_y=	350	N/mm2	Basic yield strength	
f_u=	420	N/mm2	Ultimate tensile strength	
t=	0,68	mm	Sheet thickness, based on roof sheet design	
t_{nom}=	0,64	mm	Nominal sheet thickness used in design	
E=	210	kN/mm2	Modulus of elasticity	
G=	81	kN/mm2	Shear modulus	
u=	0,3		Poisson's ratio	
u=	379	mm	Perimeter length of steel corrugation	Second moment of area of a single corrugation
I_{y1}=	1840	mm^4/mm	Second moment of area(from manufacture)	$I_{y1}=I_y*d=$ 436080 mm^4
d=	237	mm	Pitch of corrugation	
l=	68	mm	See the picture on the right	Sheet profile distortion factor
h/d=	0,47		See the picture on the right	
K_1=	0,209		Fasteners in every corrugation	
K_2=	2,708		Fasteners in every alternate corrugation	

K	p(mm)
0,209	237
2,708	474

04 Choosed Edge beam: **P150x150x5**

Steel grade: S355

f_y=	355	N/mm2	Basic yield strength	t_e= 5 mm
f_u=	510	N/mm2	Ultimate tensile strength	
E=	210	kN/mm2	Modulus of elasticity	
G=	81	kN/mm2	Shear modulus	
u=	0,3		Poisson's ratio	
A=	2814	mm^2	Cross-section	

05 Fastener(Types and amounts)

(a) Sheet to edge beam connection **Screws collar head +Neoprene 6,3**

K=	0,209		**Fasteners in every corrugation**	
p=	237,000	mm	Distance between fasteners in connection to edge beam	
F_p=	2,31	kN	Design shear resistance of individual sheet-to-purlin fastener	Table 12
s_p =	0,35	mm/kN	Slip(flexibility) per sheet to edge beam fastener per unit load.	Table 12
n_f=	3		Number of sheet to purlin fasteners per sheet width.This can be calculated based on d and sheet type.	
n_p=	2		Total number of purlins(Edge+intermediate) per width of roof, b	
n_b=	1		Number of sheet lengths within the lengths of diaphragm	
$n_{sh=}$	35		Number of sheet widths per pannel a	
$n_{l=}$	2		Number of sheet lengths	

(b) Seam fasteners **Screw 4,8** Seam fastners in the crests

SWECO	A.2 Excel calcualtion regarding sheets directly on main beams	Designer	Phase / Area	Project nr
			Engineering	
Project:			Date	Page
Subject:	Corrugated sheet as stressed skin, deformation calculation		12.1.2020	

F_s=	0,87 kN	Design shear resistance of individual seam fastener	Table 12
s_s =	0,25 mm/kN	Slip(flexibility) per seam fastener per unit load	Table 12
n_s=	25	Number of seam fasteners per side lap(excluding those which pass through both shees and the supporting purlin	

α_1=	1	β_1 =	0,3	Non-dimensional factors
α_2=	1	β_2 =	1	α1, α2, α3 depend on n_p (Table 7)
α_3=	1	β_3 =	0,67	β1, β2 depend on n_f (Table 5)
α_5=	1			α5 (Table 9)

(c) Sheet to shear connector fasteners **Screws collar head +Neoprene 6,3** Shear connectors transfer forces between steel sheet diaphragm and building frame

F_{sc}=	1,16 kN	Design shear resistance of individual sheet-to-shear connector fastener	Table 12
s_{sc}=	0,35 mm/kN	Slip(flexibility) per sheet to shear connector fastener per unit load	Table 12
n_{sc}=	100,00	Number of sheet to shear connector fasteners per end rafter	
$n_{sc'}$=	100,00	Number of sheet to shear connector fasteners per intermediate rafter	

Result

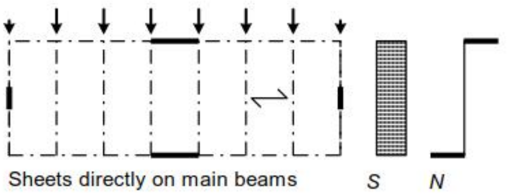
06 Actions on diaphragm due to wind

(a) Wind on long side

R_d=	156,26 kN	Support force at gable of the building
N_d=	98,45 kN	Normal force in edge beam
V_{dmax}=	6,25 kN/m	Maximun shear flow

(b) Wind on end(Only for checking the shear flow of the sheet)

R_{dl}=	14,13 kN	Support force in long sides
N_g=	0,00 kN	Normal force in edge beam
V_{dlmax}=	3,48 kN/m	Shear flow at long sides < V_{max}



Sheets directly on main beams

Note: When the wind is on the end of the building, the stressed skin diaphragm height is 2/3*a.

07 Diaphragm strength

		Value	Utility grade			
(a) Seam strength	$V_{Rd.1}$=	84,93 kN	$R_d/V_{Rd.1}$ =	1,84	Check number and type of fasteners	FAILURE
(b) Strength in fasteners to shear connec	$V_{Rd.2}$=	412,97 kN	$R_d/V_{Rd.2}$ =	0,38		OK

(c) Globle shear buckling resistance:

Dx= 3,15 kN*mm^2/mm

Dy= 386400 kN*mm^2/mm

$V_{cr.g}=28.8\frac{a}{b^2}Dx^{\frac{1}{4}}Dy^{\frac{3}{4}}$= 303,44 kN — $R_d/V_{cr.g}$ = **0,51** Fasteners in every corrugation — OK

$V_{cr.g}=14.4\frac{a}{b^2}Dx^{\frac{1}{4}}Dy^{\frac{3}{4}}$= kN

(d) Local shear buckling resistance:

$V_{cr.l}=4{,}83btE\left(\frac{t}{l}\right)^2$= 402,519 kN — $R_d/V_{cr.l}$ = **0,39** — OK

(e) The interaction of global and local shear buckling = Resistance for shear buckling:

$V_{red} = \frac{V_{cr.g}*V_{cr.l}}{V_{cr.g}+V_{cr.l}}$= 173,014 kN — R_d/V_{red} = **0,90** — OK

(f) Sheet-to-rafter fasteners:

$V_{Rd.4} = \frac{0{,}6aF_p}{p}$= 146,37 kN — $R_d/V_{Rd.4}$ = **1,07** — FAILURE

(g) End collapse of sheeting profile

$V_{Rd.5} = 0.9f_y a\sqrt{\frac{t^3}{d}}$= 261,91 kN — $R_d/V_{Rd.5}$ = **0,597** Fasteners in every corrugation — OK

$V_{Rd.5} = 0.3f_y a\sqrt{\frac{t^3}{d}}$= kN

Design shear capacity V_{Rd}= 84,93 kN

08 Comonents of shear flexibility:

(a) Flexibility due to profile distortion $c_{1,1}$ = 1,340 mm/kN

(b) Flexibility due to shear strain $c_{1,2}$ = 0,134 mm/kN

(c) Flexibility due to deformation in sheet to purlin fasteners $c_{2,1}$ = 0,085 mm/kN

(d) Flexibility due to deformation in seam fasteners $c_{2,2}$ = 0,337 mm/kN

(e) Flexibility in connections to shear connectors $c_{2,3}$= 0,007 mm/kN

c'= 1,903 mm/kN

(f) Equivalent shear flexibility due to axial strain in purlins c_3= 0,016 mm/kN

Total shear fexibility c = 0,165 mm/kN

Shear flexibility due to:		Sheets on purlins
		Shear flexibility mm/kN
Sheet deformation	Profile distortion	$c_{1.1} = \frac{ad^{2.5}\alpha_1\alpha_4 K}{Et^{2.5}b^2}$
	Shear strains	$c_{1.2} = \frac{2a\alpha_2(1+v)(1+\left(\frac{2h}{d}\right))}{Etb}$
Fastener deformation	Sheet to pulin	$c_{2.1} = \frac{2as_p p\alpha_3}{b^2}$
	Seam fasteners	$c_{2.2} = \frac{2s_s s_p(n_{sh}-1)}{2n_s s_p + \beta_1 n_p s_s}$
	Connections to Rafters	$c_{2.3} = \frac{4(n+1)s_{sc}}{n^2 n_s'}$
Flange forces	Axial strain in purlins	$c_3 = \frac{n^2 a^3 \alpha_3}{4.8EAb^2}$
Total shear flexibility		$c = c_{1.1} + c_{1.2} + c_{2.1} + c_{2.2} + c_{2.3} + c_3$

09 Mid-span deflection

Δ=P(n^2/8)*c= 58,0 mm < Δ_{max}= H/150= 58,53 mm

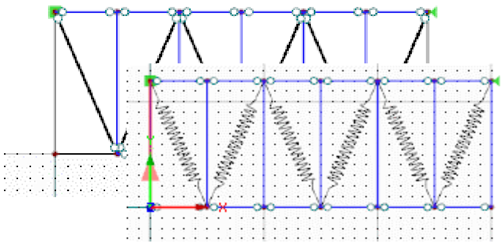
SWECO	A.2 Excel calcualtion regarding sheets directly on main beams	Designer	Phase / Area	Project nr
			Engineering	
Project:			Date	Page
Subject:	Corrugated sheet as stressed skin, deformation calculation		12.1.2020	

10 Equivalent tie or equivalent spring member

Cross section area of equvalent tie	**A=**	**808,60**	mm2
Length of equvalent tie	L=	25,9615	m
Modulus of elasticity of equivalent tie	E=	210	kN/mm2

Axial stiffiness of the equivalent spring member

K= 6541 kN/mm



11 Equivalent steel truss, simplified calculation of deflection in the middle of the truss

Definition of truss geometry:

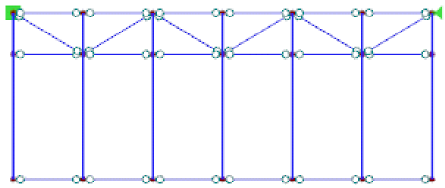
b= 7,00 m B_t= 7,00 m

n= 9 m L= 63,00 m

P= 34,7 kN

q =arctan (B_t/a) = 0,785 radians

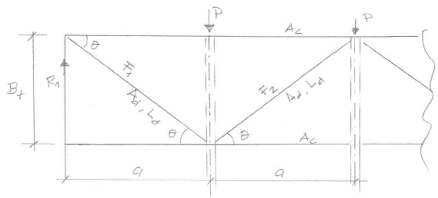
45 degrees



m= (n-1)/2 = 4

m= number forces P corresponding reaction R1

R1 = m*P = 138,9 kN



Selection of steel tubes in truss

Diagonal brace	**P150x150x8**	A_d =	**4324**	mm^2		
Truss chord	**P150x150x8**	A_c =	**4324**	mm^2	$I_â$ = **1,412E+07**	mm^4

Calculation of deflection:

Deflection due to elastic deformation on diagonal braces:

if 2m = even $\sum dy_d = \frac{P*(m^2 - m + 1)*B_t}{E*A_d \sin\theta^3}$ 9,84 mm

if 2m = odd $\sum dy_d = \frac{P*\left(m^2 - \frac{m}{2} + {}^1/_2\right)*B_t}{E*A_d \sin\theta^3}$ mm

Deflection due to elastic deformation of truss chords:

$dy_c = \frac{5*n*P*L^3}{384*E*I_0}$ 43,55 mm

Moment of inertia for truss chords: f= $\sum dy_d + dy_c$ = **53,4** mm

$I_0 = (B_t^2/2*A_c + 2I_a)*1.25$ = 1,1126E+11 mm^4

12 Equivalent surface member

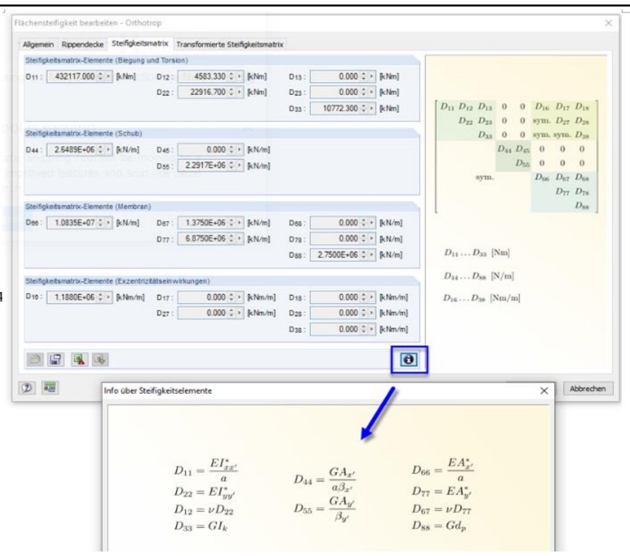
Stiffness Matrix Elements(Membrane)

D_{66}= 23637,6 kN/m

D_{88}= 1698 kN/m

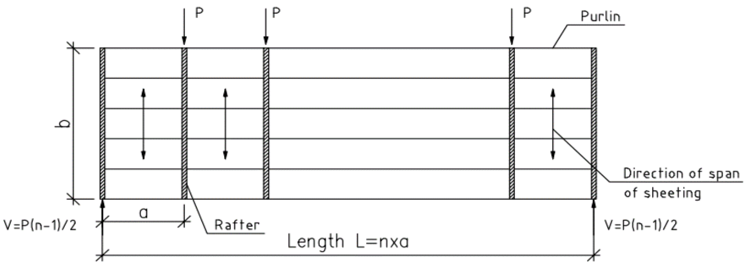
The following figure shows the general stiffness matrix of an orthotropic surface in RFEM.

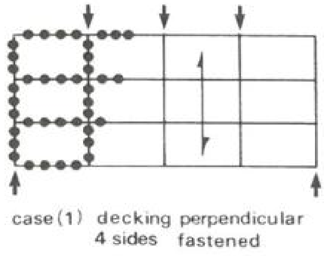
$$\begin{bmatrix} m_x \\ m_y \\ m_{xy} \\ v_x \\ v_y \\ n_x \\ n_y \\ n_{xy} \end{bmatrix} = \begin{bmatrix} D_{11} & D_{12} & D_{13} & 0 & 0 & D_{16} & D_{17} & D_{18} \\ & D_{22} & D_{23} & 0 & 0 & D_{26} & D_{27} & D_{28} \\ & & D_{33} & 0 & 0 & D_{36} & D_{37} & D_{38} \\ & & & D_{44} & D_{45} & 0 & 0 & 0 \\ & & & & D_{55} & 0 & 0 & 0 \\ & & sym. & & & D_{66} & D_{67} & D_{68} \\ & & & & & & D_{77} & D_{78} \\ & & & & & & & D_{88} \end{bmatrix} \cdot \begin{bmatrix} \kappa_x \\ \kappa_y \\ \kappa_{xy} \\ \gamma_{xz} \\ \gamma_{yz} \\ \varepsilon_x \\ \varepsilon_y \\ \gamma_{xy} \end{bmatrix} \quad (4$$



SWECO	A.3 Excel calcualtion regarding sheets on purlin(Design example 3)	Designer	Phase / Area	Project nr
			Engineering	
Project:			Date	Page
Subject:	Corrugated sheet as stressed skin, deformation calculation		1.5.2020	

STRESSED SKIN DESIGN ACCORDING TO ECCS 'Design of Code-formed Steel Structure' (2012)





Input data

01 Data of building (main frame):

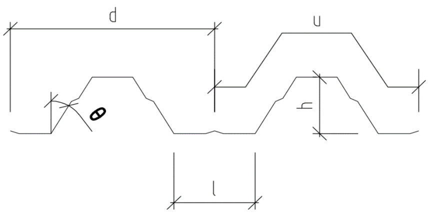
a=	7,2	m	Length of diaphragm	
b=	18	m	Depth of diaphragm	
H=	8,8	m	Height of building	
n=	10		Number of panels within diaphragm assebly	
L=	72	m	Length of building	L=n*a

02 Wind load on side of building

q_w=	0,50	kN/m2	Peak velocity pressure (EN 1991-1-4)		
g_Q =	1,5		Partial load factor		
p=	**2,20**	kN/m	Wind load at edge of the roof	$p=q_w*H*0,5$	(This load valuse was made
p_d=		kN/m	Design wind load	pd=p*rQ	according to the original book)
P=	15,84	kN	Point load on top of the building column	P=p*a	

03 Choosed roof profile: **TRP22** **(Based on results of roof sheet calculations)**

Steel grade: S350GD+Z



f_y=	350	N/mm2	Basic yield strength	
f_u=	420	N/mm2	Ultimate tensile strength	
t=	0,627	mm	Sheet thickness, based on roof sheet design	
t_{nom}=	0,587	mm	Nominal sheet thickness used in design	
E=	210	kN/mm2	Modulus of elasticity	
G=	81	kN/mm2	Shear modulus	
u=	0,3		Poisson's ratio	
u=	118	mm	Perimeter length of sheet corrugation	Second moment of area of a single corrugation
I_{y1}=	60	mm^4/mm	Second moment of area(from manufacture)	$I_{y1}=I_y*d=$ 5400 mm^4
d=	90	mm	Pitch of corrugation	
l=	25	mm	See the picture on the right	Sheet profile distortion factor
h/d=	0,24		See the picture on the right	
K_1=	0,130		Fasteners in every corrugation	
K_2=	0,763		Fasteners in every alternate corrugation	

K	p(mm)
0,130	90
0,763	180

04 Choosed Purlin: **IPE 140** cp= 2 m Distance between the purlins

Steel grade: S355

f_y=	355	N/mm2	Basic yield strength	t_p=	6,9 mm
f_u=	510	N/mm2	Ultimate tensile strength		
E=	210	kN/mm2	Modulus of elasticity		
G=	81	kN/mm2	Shear modulus		
u=	0,3		Poisson's ratio		
A=	1643	mm^2	Cross-section		

05 Fastener(Types and amounts)

(a) Sheet to purlin connection **Screws collar head 6,3**

K=	0,130		**Fasteners in every corrugation**	
p=	90,000	mm	Distance between fasteners in connection to purlin	
F_p=	2,95	kN	Design shear resistance of individual sheet-to-purlin fastener	Table 12
s_p =	0,15	mm/kN	Slip(flexibility) per sheet to purlin fastener per unit load.	Table 12
n_f=	4		Number of sheet to purlin fasteners per sheet width. This can be calculated based on d and sheet type.	
n_p=	10		Total number of purlins(Edge+intermediate) width of roof, b	
n_b=	2		Number of sheet lengths within the lengths of diaphragm	
n_{sh}=	8		Number of sheet widths per pannel a	

(b) Seam fasteners **Screw 4,8** Seam fastners in the crests

F_s=	1,24	kN	Design shear resistance of individual seam fastener	Table 12

SWECO	A.3 Excel calcualtion regarding sheets on purlin(Design example 3)	Designer	Phase / Area	Project nr
			Engineering	
Project:			Date	Page
Subject:	Corrugated sheet as stressed skin, deformation calculation		1.5.2020	

$s_s =$	0,25	mm/kN	Slip(flexibility) per seam fastener per unit load	Table 12
$n_s =$	117		Number of seam fasteners per side lap(excluding those which pass through both shees and the	

$\alpha_1 =$	0,6	$\beta_1 =$	0,44	Non-dimensional factors
$\alpha_2 =$	0,36	$\beta_2 =$	1,11	α1, α2, α3 depend on n_p (Table 7)
$\alpha_3 =$	0,49	$\beta_3 =$	0,75	β1, β2 depend on n_f (Table 5)
$\alpha_4 =$	1,6			α4 (Table 8)

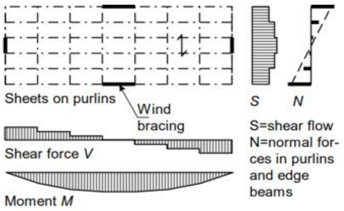
(c) Sheet to shear connector fasteners **Screws collar head 6,3** Shear connectors transfer forces between steel sheet diaphragm and building frame

$F_{sc} =$	2,95	kN	Design shear resistance of individual sheet-to-shear connector fastener	Table 12
$s_{sc} =$	0,15	mm/kN	Slip(flexibility) per sheet to shear connector fastener per unit load	Table 12
$n_{sc} =$	12,00		Number of sheet to shear connector fasteners per end rafter	
$n_{sc}' =$	12,00		Number of sheet to shear connector fasteners per intermediate rafter	

Result

06 Actions on diaphragm due to wind

(a) **Wind on long side**



$R_d =$	79,20	kN	Support force at gable of the building
$N_d =$	45,94	kN	Normal force in edge beam at long sides of building
$V_{dmax} =$	5,06	kN/m	Maximun shear flow

(b) Wind on end (Only for checking the shear flow of the sheet)

$R_{dl} =$	0,00	kN	Support force in long sides
$N_{dl} =$	0,00	kN	Normal force in end rafter
$N_g =$	0,00	kN	Normal force in edge beam

Note: When the wind is on the end of the building, the stressed skin diaphragm height is 2/3*a.

07 Diaphragm strength **Utility grade**

(a) Seam strength	$V_{Rd.1} =$	162,43	kN	$R_d/V_{Rd.1} =$	0,49		OK
(b) Strength in fasteners at shear connecto	$V_{Rd.2} =$	35,41	kN	$R_d/V_{Rd.2} =$	2,24	Check number and type of fasteners	FAILURE
(c) Global shear buckling resistance:	Dx=	2,97	kN*mm²/mm				
	Dy=	12600	kN*mm²/mm				
	$V_{cr.g} = \frac{14{,}4}{b} Dx^{\frac{1}{4}} Dy^{\frac{3}{4}} (n_p - 1)^3 =$	910,26	kN	$R_d/V_{cr.g} =$	0,09		OK
(d) Local shear buckling resistance:							
	$V_{cr.l} = 4{,}83btE(\frac{t}{l})^2 =$	5908,45	kN	$R_d/V_{cr.l} =$	0,01		OK
(e) The interaction of global and local shear buckling = Resistance for shear buckling:							
	$V_{red} = \frac{V_{cr.g} * V_{cr.l}}{V_{cr.g} + V_{cr.l}} =$	788,74	kN	$R_d/V_{red} =$	0,10		OK
(f) Sheet-to-purlin fasteners:							
	$V_{Rd.4} = \frac{0{,}6bF_p}{pa3} =$	722,71	kN	$R_d/V_{Rd.4} =$	0,11		OK
(g) End collapse of sheeting profile							
	$V_{Rd.5} = 0.9 f_y b \sqrt{\frac{t^3}{d}} =$	268,79	kN	$R_d/V_{Rd.5} =$	0,295	Fasteners in every corrugation	OK
	$V_{Rd.5} = 0.3 f_y b \sqrt{\frac{t^3}{d}} =$		kN				
Design shear capacity	$V_{Rd} =$	**35,41**	**kN**				

08 Comonents of shear flexibility:

(a) Flexibility due to profile distortion	$c_{1,1} =$	0,004	mm/kN
(b) Flexibility due to shear strain	$c_{1,2} =$	0,005	mm/kN
(c) Flexibility due to deformation in sheet to purlin fasteners	$c_{2,1} =$	0,000	mm/kN
(d) Flexibility due to deformation in seam fasteners	$c_{2,2} =$	0,015	mm/kN
(e) Flexibility in connections to shear connectors	$c_{2,3} =$	0,006	mm/kN
	c'=	0,029	mm/kN
(f) Equivalent shear flexibility due to axial strain in purlins	$c_3 =$	0,034	mm/kN
Total shear fexibility	c =	0,063	mm/kN

Shear flexibility due to:		Sheets on purlins
		Shear flexibility mm/kN
Sheet deformation	Profile distortion	$c_{1.1} = \frac{ad^{2.5}\alpha_1\alpha_4 K}{Et^{2.5}b^2}$
	Shear strains	$c_{1.2} = \frac{2a\alpha_2(1+v)(1+(\frac{2h}{d}))}{Etb}$
Fastener deformation	Sheet to pulin	$c_{2.1} = \frac{2as_p p\alpha_3}{b^2}$
	Seam fasteners	$c_{2.2} = \frac{2s_s s_p(n_{sh}-1)}{2n_s s_p + \beta_1 n_p s_s}$
	Connections to Rafters	$c_{2.3} = \frac{4(n+1)s_{sc}}{n^2 n_s'}$
Flange forces	Axial strain in purlins	$c_3 = \frac{n^2 a^3 \alpha_3}{4.8EAb^2}$
Total shear flexibility		$c = c_{1.1} + c_{1.2} + c_{2.1} + c_{2.2} + c_{2.3} + c_3$

09 Mid-span deflection

$\Delta = P(n^2/8)*c =$ 12,4 mm < $\Delta_{max} = H/300 =$ 29,33 mm

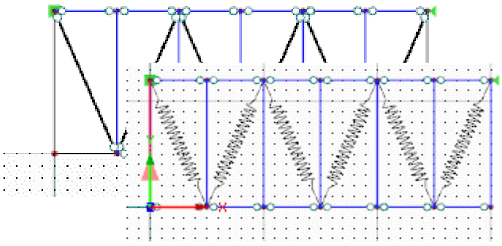
SWECO	A.3 Excel calcualtion regarding sheets on purlin(Design example 3)	Designer	Phase / Area	Project nr
Project:			Engineering	
Subject:	Corrugated sheet as stressed skin, deformation calculation		Date 1.5.2020	Page

10 Equivalent tie or equivalent spring member

Cross section area of equivalent tie	A=	**1705,24**	mm2
Length of equivalent tie	L=	19,3866	m
Modulus of elasticity of equivalent tie	E=	210	kN/mm2

Axial stiffness of the equivalent spring member

K= 18472 kN/mm



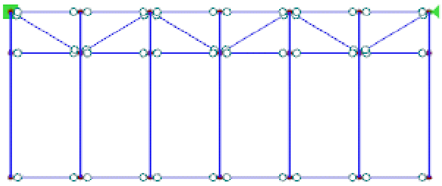
11 Equivalent steel truss, simplified calculation of deflection in the middle of the truss

Definition of truss geometry:

a= 7,20 m $\quad$ B_t= 8,00 m

n= 10 m $\quad$ L= 72,00 m

P= 15,8 kN



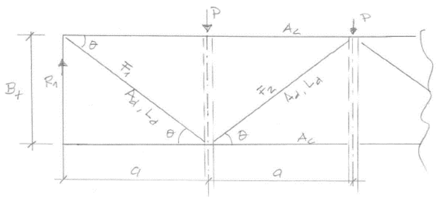
q =arctan (B_t/a) = 0,838 radians

48,012788 degrees

m= (n-1)/2 = 4,5

m= number forces P corresponding reaction R1

R1 = m*P = 71,3 kN



Selection of steel tubes in truss

Diagonal brace	**P300x300x10**	A_d =	**11257**	mm^2
Truss chord	**P300x300x10**	A_c =	**11257**	mm^2

$I_å$ = **1,552E+08** mm^4

Calculation of deflection:

Deflection due to elastic deformation on diagonal braces:

if 2m = even $\quad \sum dy_d = \dfrac{P*(m^2-m+1)*B_t}{E*A_d \sin\theta^3}$ mm

if 2m = odd $\quad \sum dy_d = \dfrac{P*\left(m^2-\frac{m}{2}+{}^{1}\!/_{2}\right)*B_t}{E*A_d \sin\theta^3}$ 2,41 mm

Deflection due to elastic deformation of truss chords:

$dy_c = \dfrac{5*n*P*L^3}{384*E*I_0}$ 9,68 mm

Moment of inertia for truss chords: $\quad$ f= $\sum dy_d + dy_c$ = **12,1** mm

$I_0 = (B_t^2/2*A_c + 2I_a)*1.25$ = 3,7856E+11 mm^4

12 Equivalent surface member

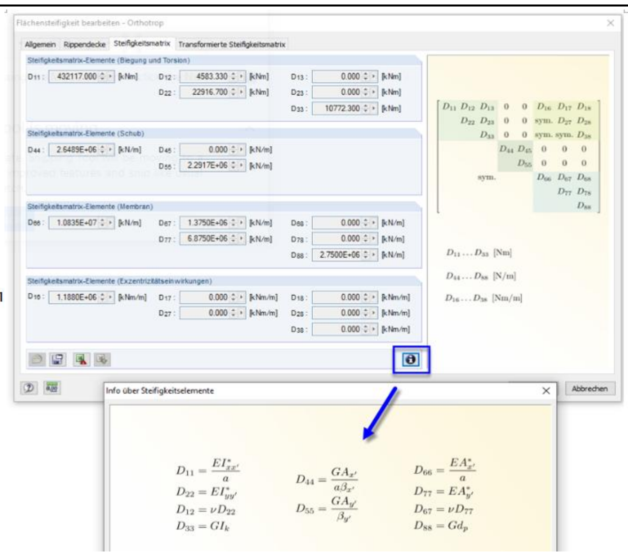
Stiffness Matrix Elements(Membrane)

D_{66}= 172473 kN/m

D_{88}= 6369 kN/m

The following figure shows the general stiffness matrix of an orthotropic surface in RFEM.

$$\begin{bmatrix} m_x \\ m_y \\ m_{xy} \\ v_x \\ v_y \\ n_x \\ n_y \\ n_{xy} \end{bmatrix} = \begin{bmatrix} D_{11} & D_{12} & D_{13} & 0 & 0 & D_{16} & D_{17} & D_{18} \\ & D_{22} & D_{23} & 0 & 0 & D_{26} & D_{27} & D_{28} \\ & & D_{33} & 0 & 0 & D_{36} & D_{37} & D_{38} \\ & & & D_{44} & D_{45} & 0 & 0 & 0 \\ & & & & D_{55} & 0 & 0 & 0 \\ & & \text{sym.} & & & D_{66} & D_{67} & D_{68} \\ & & & & & & D_{77} & D_{78} \\ & & & & & & & D_{88} \end{bmatrix} \cdot \begin{bmatrix} \kappa_x \\ \kappa_y \\ \kappa_{xy} \\ \gamma_{xz} \\ \gamma_{yz} \\ \varepsilon_x \\ \varepsilon_y \\ \gamma_{xy} \end{bmatrix} \tag{4.1}$$



Filename:
S:\FI\Exchange-Area\FIMEIW\Text+Excel\Example_Red_book.pmu

Dimensioning code: SFS-EN 1993-1-3:2006+AC:2009, Finnish NA/NAD

* * * GENERAL INFORMATION OF THE STRUCTURE * * *

Basic data of the structure

Length of the building:	30000 mm
Width of the building:	12000 mm
Width of the sheet field (side-wind):	12000 mm
Length of the sheet field in the left end:	8000 mm
Profiled sheet on purlins	
Frames center distance from constructions left gable:	3000 mm

Safety class :	RC2
Combination type in serviceability limit state	Characteristic combination
Structure type:	Load bearing roof (deck)
Deflection limit:	L/200
Roof slope:	0.0°
Torsion free supports	

Sheet:	**Ruukki T45-30L-905**
- material:	Construction steel
- zinc coating:	Zn275
- modulus of elasticity:	210000 N/mm2
- perforation of the profile:	No perforation

Supports and splices

- support beam material:	Steel beam
- support steel yield strength:	355 N/mm2
- support wall thickness:	3 mm

Support	Support width	Type of splice	Support piece
A	100	End support	No
B	100	Continuing, same sheet	No
C	100	Continuing, same sheet	No
D	100	Continuing, same sheet	No
E	100	Continuing, same sheet	No
F	100	Continuing, same sheet	No
G	100	Continuing, same sheet	No
H	100	Continuing, same sheet	No
I	100	End support	No

Left end support:	Upright support
Right end support:	Upright support

Chosen sheets

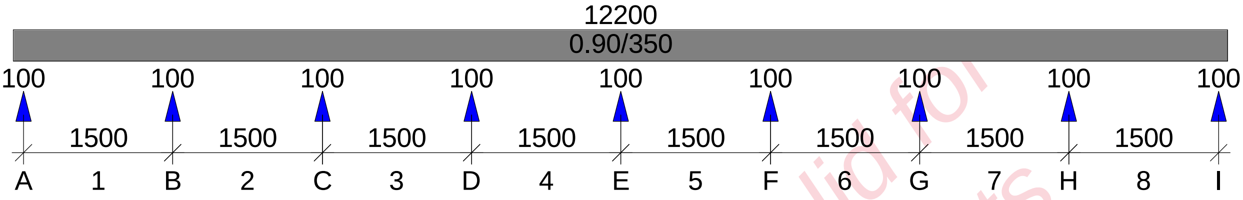
T45-30L-905 Narrow flange against the support

Nr	Thickness/Strength [mm]/[N/mm2]	Overlap* [Folds st]	Length [mm]	Weight [kg/st]
1	0.90/350	0	12200	107.74

Total weight of the sheeting: 9.76 kg/m2
**Side overlapping*

- Chosen sheets fulfill dimension criterions. Maximum utilityrate: 83.8 %
- Chosen fastenings fulfill dimension criterions. Maximum utilityrate: 99.2 %

Structural model



*** * * LOADS * * ***

Dead loads

Structure weight without sheet 0.00 kN/m2

Type	Support	F1	F2	F3	F4	Movement
1 Uniform load	A	0.30		0	12000	-

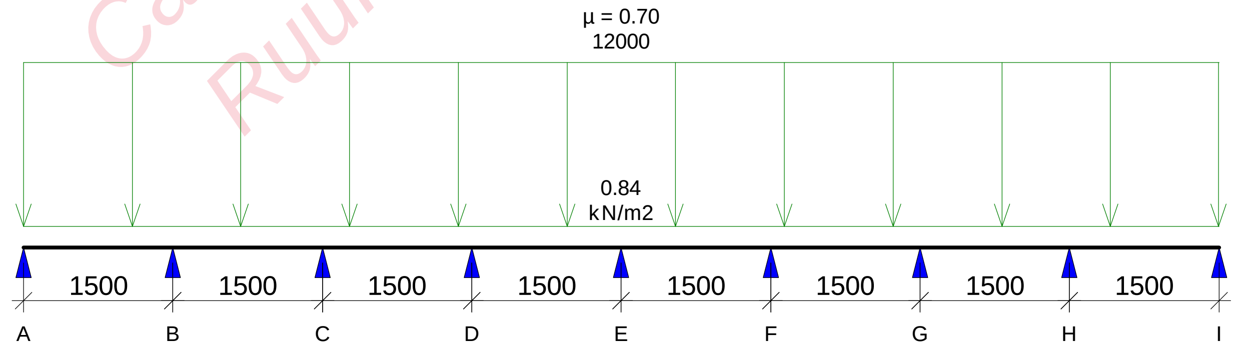
Snow loads - no loads

Wind loads

Basic wind load 1.20 kN/m2

Load case: 1

- Form factors



- Stressed skin design

Line load to side of the building: 3.00 kN/m

Load case: 2

- Stressed skin design

Line load to gable wall: 3.00 kN/m

Load case: 3

Load case: 4

Live loads - no loads

Explanation for loadparameters F1, F2, F3 and F4

- Uniform load:	F1	load intensity [kN/m2]
	F3	distance from the left support to load begin [mm]
	F4	loading length [mm]
- Trapezoid load:	F1	load intensity at left end [kN/m2]
	F2	load intensity at right end [kN/m2]
	F3	distance from the left support to load begin [mm]
	F4	loading length [mm]
- Line load:	F1	load intensity [kN/m]
	F3	distance from the left support to load begin [mm]
	F4	loading length [mm]

Partial safety factors for loads:	Ultimate limit state			Serviceability limit state		
	Max	Min	Comb.fac	Max	Min	Comb.fac
Dead loads:	1.15	0.90		1.00	1.00	
Snow loads:	1.50	0.00	0.70	1.00	0.00	0.70
Wind loads:	1.50	0.00	0.60	1.00	0.00	0.60
Live loads:	1.50	0.00	0.70	1.00	0.00	0.70
Minimum load:	1.35 * Dead loads					

*** * * RESULTS * * ***

Degree of utilization in each sheets

T45-30L-905 Narrow flange against the support

Sheet Nr	Thickness/Strength [mm]/[N/mm2]	Field [%]	Support [%]	Deflection [%]	Stressed skin effect [%]
1	0.90/350	6.8	9.2	7.9	83.8

Maximum utilityrate: 83.8 %
Dimensioning case: Sheet reaction, (stressed skin design)

Degree of utilization in each spans

T45-30L-905 Narrow flange against the support

Field/ Support	M [%]	R/V/N [%]	Combination [%]	Deflection [%]
A	0.0	2.7 R	2.2 M+R	
1	6.8 (603)			7.9 (603)
B	8.5	3.8 R	9.2 M+R	
2	4.8 (163)			1.9 (750)
C	6.1	3.2 R	6.9 M+R	
3	3.8 (750)			3.5 (750)
D	6.8	3.4 R	7.5 M+R	
4	3.6 (750)			3.1 (750)
E	6.6	3.3 R	7.3 M+R	
5	3.6 (750)			3.1 (750)
F	6.8	3.4 R	7.5 M+R	
6	3.8 (750)			3.5 (750)
G	6.1	3.2 R	6.9 M+R	
7	4.8 (1337)			1.9 (750)
H	8.5	3.8 R	9.2 M+R	
8	6.8 (897)			7.9 (897)
I	0.0	2.7 R	2.2 M+R	

(The dimensioning point is printed in braces)
Maximum utilityrate: 83.8 %
Dimensioning case: Sheet reaction, (stressed skin design)

Span results

Span/ Support	Moment kNm/m Msd	Mc,rd	Point load capasity kN/m Fsd	Rw,rd	Deflection mm f	f,allowed
A	0.00	4.43	1.02	37.63		
1	0.30	4.43			-0.6	7.5
B	-0.37	4.36	2.92	77.58		
2	-0.21	4.36			-0.1	7.5
C	-0.27	4.36	2.48	77.58		
3	0.17	4.43			-0.3	7.5
D	-0.30	4.36	2.60	77.58		
4	0.16	4.43			-0.2	7.5
E	-0.29	4.36	2.56	77.58		
5	0.16	4.43			-0.2	7.5
F	-0.30	4.36	2.60	77.58		
6	0.17	4.43			-0.3	7.5
G	-0.27	4.36	2.48	77.58		
7	-0.21	4.36			-0.1	7.5
H	-0.37	4.36	2.92	77.58		
8	0.30	4.43			-0.6	7.5
I	0.00	4.43	1.02	37.63		

* * * STRESSED SKIN DESIGN * * *

Maximum sheet forces

Case	Gable wall Bracing [kN]	Gable wall Edge member [kN]	Side wall Bracing [kN]	Side wall Edge member [kN]	Shear force [kN/m]
1	67.5	0.0	0.0	24.5	5.17
2	0.0	10.1	27.0	0.0	3.38

Dimensioning case: 1

Degree of utilization in each sheets

T45-30L-905 Narrow flange against the support

Sheet Nr	Thickness/Strength [mm]/[N/mm2]	Vsd [kN/m]	Tau [%]	Vw,Rd/M+Vw [%]	Vf,Rd/M+Vf [%]	Vg,Rd/Vg+Vf [%]
1	0.90/350	5.2 (1)	6.9	3.0 / -	3.1 / -	2.7 / -

(The dimensioning wind case is printed in brackets)

- Left end support:
  End collapse of the profile in the end support: Vr,Rd 0.0 %
- Right end support:
  End collapse of the profile in the end support: Vr,Rd 83.8 %

* * * DIMENSIONING FOR FASTENINGS * * *

Fastening to support

Support beam material:	Steel beam
Support steel yield strength:	355 N/mm2
Support wall thickness:	3 mm
Screw material, gasket:	Carbon-steel, hardened, with washer
Screw type:	SD14-T15-5.5*32
Manufacturer:	SFS intec Oy
Number of fasteners/width meter:	60 pc/m

Support	Pc./ trough	Utilityrate [%]	Vd [kN/m]	Fd [kN/m]	Fv [kN]	FvRd [kN]	Ft [kN]	FtRd [kN]	
A	1	38.8	5.2	0.0	0.8	2.0	0.0	2.0	5
B	1	6.3	0.8	0.0	0.1	2.0	0.0	2.0	5
C	1	6.3	0.8	0.0	0.1	2.0	0.0	2.0	5
D	1	6.3	0.8	0.0	0.1	2.0	0.0	2.0	5
E	1	6.3	0.8	0.0	0.1	2.0	0.0	2.0	5
F	1	6.3	0.8	0.0	0.1	2.0	0.0	2.0	5
G	1	6.3	0.8	0.0	0.1	2.0	0.0	2.0	5
H	1	6.3	0.8	0.0	0.1	2.0	0.0	2.0	5
I	1	38.8	5.2	0.0	0.8	2.0	0.0	2.0	5

Side overlap

Screw material, gasket:	Carbon-steel, hardened
Screw type:	SL2-4.8*20
Manufacturer:	SFS intec Oy
Number of fasteners/width meter:	37 pc/m

Span	c/c [mm]	Utilityrate [%]	Fv [kN]	FvRd [kN]	
1	360	99.2	1.9	1.9	5
2	360	99.2	1.9	1.9	5
3	360	99.2	1.9	1.9	5
4	360	99.2	1.9	1.9	5
5	360	99.2	1.9	1.9	5
6	360	99.2	1.9	1.9	5
7	360	99.2	1.9	1.9	5
8	360	99.2	1.9	1.9	5

Dimensioning criterion: -5) Bearing resistance

*** * * SHEET LIST * * ***

Sheet: T45-30L-905

	Thickness/Strength [mm]/[N/mm2]	Total length [mm]	Total weight [kg]
1	0.90/350	12200	107.7

A.5 Values of K1 for fasteners in every trough

θ	h/d \ l/d	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0°	0.1	0.013	0.030	0.041	0.041	0.046	0.050	0.066	0.103	0.193
	0.2	0.042	0.096	0.131	0.142	0.142	0.153	0.199	0.311	0.602
	0.3	0.086	0.194	0.264	0.285	0.283	0.302	0.388	0.601	1.188
	0.4	0.144	0.323	0.438	0.473	0.468	0.494	0.629	0.972	1.935
	0.5	0.216	0.438	0.654	0.703	0.695	0.729	0.922	1.420	2.837
	0.6	0.302	0.674	0.911	0.980	0.965	1.008	1.266	1.938	3.892
	0.7	0.402	0.895	1.208	1.300	1.277	1.329	1.661	2.536	5.098
	0.8	0.516	1.146	1.546	1.662	1.631	1.692	2.107	3.208	6.453
5°	0.1	0.014	0.031	0.041	0.044	0.044	0.049	0.066	0.107	0.205
	0.2	0.050	0.099	0.128	0.134	0.132	0.146	0.198	0.336	0.652
	0.3	0.107	0.202	0.253	0.260	0.254	0.280	0.386	0.681	1.548
	0.4	0.188	0.338	0.413	0.417	0.404	0.448	0.629	1.158	2.639
	0.5	0.295	0.507	0.604	0.601	0.578	0.648	0.934	1.783	
	0.6	0.429	0.706	0.823	0.826	0.772	0.877	1.306	2.586	
	0.7	0.591	0.935	1.066	1.028	0.983	1.385	1.756	3.605	
	0.8	0.780	1.191	1.328	1.264	1.208	1.423	2.990	4.838	
10°	0.1	0.016	0.031	0.040	0.042	0.042	0.048	0.065	0.111	0.221
	0.2	0.056	0.101	0.123	0.125	0.123	0.139	0.200	0.366	0.873
	0.3	0.125	0.204	0.238	0.233	0.260	0.264	0.402	0.786	
	0.4	0.222	0.338	0.375	0.356	0.345	0.418	0.689	1.445	
	0.5	0.349	0.494	0.526	0.486	0.473	0.605	1.082	2.428	
	0.6	0.502	0.668	0.682	0.615	0.608	0.837	1.607		
	0.7	0.677	0.851	0.834	0.736	0.752	1.128	2.308		
	0.8	0.869	1.035	0.975	0.844	0.907	1.494	3.200		
15°	0.1	0.017	0.031	0.040	0.041	0.041	0.047	0.066	0.115	0.241
	0.2	0.062	0.102	0.118	0.115	0.113	0.134	0.209	0.403	
	0.3	0.139	0.202	0.218	0.204	0.200	0.254	0.440	0.945	
	0.4	0.244	0.321	0.325	0.293	0.294	0.414	0.796		
	0.5	0.370	0.448	0.426	0.371	0.396	0.636	1.329		
	0.6	0.508	0.568	0.508	0.434	0.513	0.941			
	0.7	0.646	0.668	0.561	0.483	0.664	1.349			
	0.8	0.768	0.735	0.578	0.527	0.861				
20°	0.1	0.018	0.032	0.03.9	0.039	0.039	0.046	0.066	0.111	0.276
	0.2	0.068	0.101	0.111	0.106	0.104	0.131	0.221	0.452	
	0.3	0.148	0.193	0.194	0.174	0.177	0.255	0.492		
	0.4	0.249	0.289	0.267	0.230	0.259	0.444	0.931		
	0.5	0.356	0.372	0.315	0.270	0.364	0.725			
	0.6	0.448	0.420	0.326	0.303	0.512				
	0.7	0.509	0.423	0.301	0.346					
	0.8	0.521	0.372	0.259	0.413					

θ	h/d \ l/d	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
25°	0.1	0.019	0.032	0.038	0.038	0.038	0.045	0.068	0.126	0.313
	0.2	0.072	0.099	0.103	0.095	0.095	0.129	0.236	0.513	
	0.3	0.151	0.178	0.166	0.144	0.160	0.268	0.557		
	0.4	0.238	0.244	0.204	0.176	0.247	0.494			
	0.5	0.306	0.272	0.203	0.204	0.376				
	0.6	0.333	0.248	0.172	0.241					
	0.7	0.300	0.174	0.142						
	0.8	0.204	0.081							
30°	0.1	0.020	0.032	0.037	0.036	0.036	0.044	0.070	0.133	
	0.2	0.075	0.095	0.094	0.084	0.087	0.132	0.256		
	0.3	0.148	0.157	0.135	0.116	0.152	0.291			
	0.4	0.208	0.186	0.139	0.139	0.253				
	0.5	0.226	0.161	0.112	0.176					
	0.6	0.180	0.089	0.093						
	0.7	0.077								
35°	0.1	0.021	0.032	0.036	0.034	0.034	0.043	0.072	0.142	
	0.2	0.076	0.089	0.083	0.072	0.082	0.137	0.281		
	0.3	0.137	0.130	0.102	0.093	0.151				
	0.4	0.162	0.119	0.082	0.120					
	0.5	0.123	0.059							
	0.6	0.032								
40°	0.1	0.023	0.032	0.034	0.032	0.032	0.043	0.075		
	0.2	0.075	0.081	0.070	0.060	0.077	0.146			
	0.3	0.116	0.096	0.068	0.078					
	0.4	0.100	0.053	0.048						
	0.5	0.024								
45°	0.1	0.024	0.031	0.032	0.029	0.030	0.043	0.079		
	0.2	0.071	0.069	0.056	0.050	0.073				
	0.3	0.086	0.057	0.041						
	0.4	0.032								

A.6 Values of K2 for fasteners in every alternate troughs

θ	h/d \ l/d	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0°	0.1	0.014	0.025	0.036	0.046	0.054	0.061	0.070	0.108	0.211
	0.2	0.031	0.065	0.099	0.129	0.151	0.169	0.206	0.318	0.649
	0.3	0.054	0.123	0.192	0.252	0.294	0.328	0.402	0.608	1.269
	0.4	0.084	0.202	0.316	0.414	0.482	0.535	0.653	0.968	2.056
	0.5	0.123	0.299	0.468	0.614	0.712	0.790	0.958	1.410	3.006
	0.6	0.169	0.415	0.649	0.846	0.982	1.090	1.318	1.928	4.113
	0.7	0.222	0.549	0.855	1.108	1.286	1.433	1.730	2.525	5.383
	0.8	0.284	0.699	1.086	1.398	1.623	1.818	2.196	3.198	6.811
5°	0.1	0.089	0.138	0.184	0.228	0.269	0.311	0.359	0.432	0.590
	0.2	0.300	0.433	0.564	0.690	0.810	0.934	1.091	1.358	2.046
	0.3	0.627	0.872	1.113	1.345	1.569	1.806	2.125	2.710	4.441
	0.4	1.076	1.453	1.826	2.187	2.535	2.910	3.446	4.498	8.057
	0.5	1.644	2.171	2.694	3.205	3.703	4.244	5.058	6.761	12.94
	0.6	2.280	2.961	3.639	4.313	4.999	5.797	6.971	9.571	
	0.7	2.961	3.803	4.620	5.443	6.347	7.479	9.206	13.01	
	0.8	3.802	4.838	5.788	6.612	7.701	9.257	11.76	17.20	
10°	0.1	0.091	0.140	0.186	0.229	0.270	0.312	0.362	0.440	0.627
	0.2	0.312	0.446	0.575	0.699	0.817	0.943	1.112	1.425	2.472
	0.3	0.665	0.907	1.144	1.370	1.589	1.835	2.204	2.979	
	0.4	1.156	1.529	1.891	2.239	2.578	2.984	3.655	5.251	
	0.5	1.793	2.313	2.819	3.305	3.782	4.397	5.519	7.872	
	0.6	2.533	3.206	3.858	4.509	5.192	6.096	7.875		
	0.7	3.334	4.148	4.949	5.780	6.737	8.112	10.82		
	0.8	4.236	5.170	6.051	7.066	8.404	10.47	12.59		
15°	0.1	0.093	0.142	0.188	0.231	0.271	0.313	0.364	0.448	0.682
	0.2	0.325	0.458	0.586	0.707	0.824	0.953	1.140	1.523	
	0.3	0.703	0.942	1.174	1.393	1.610	1.874	2.316	3.411	
	0.4	1.237	1.602	1.953	2.285	2.624	30.89	3.981		
	0.5	1.937	2.443	2.926	3.379	3.869	4.640	6.256		
	0.6	2.778	3.428	4.058	4.664	5.366	6.581			
	0.7	3.692	4.488	5.273	6.081	7.138	8.902			
	0.8	4.648	5.570	6.516	7.628	9.910				
20°	0.1	0.096	0.144	0.190	0.232	0.273	0.315	0.368	0.459	0.680
	0.2	0.339	0.472	0.597	0.716	0.832	0.966	1.177	1.659	
	0.3	0.743	0.978	1.204	1.416	1.633	1.927	2.481		
	0.4	1.317	1.673	2.009	2.325	2.679	3.246	3.840		
	0.5	2.075	2.559	3.011	3.436	3.993	4.969			
	0.6	3.006	3.625	4.197	4.752	5.588				
	0.7	4.042	4.789	5.494	6.272					
	0.8	5.122	6.013	6.883	7.861					

θ	h/d \ l/d	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
25°	0.1	0.098	0.147	0.192	0.234	0.274	0.317	0.373	0.475	0.665
	0.2	0.355	0.485	0.609	0.725	0.840	0.983	1.226	1.566	
	0.3	0.784	1.015	1.233	1.437	1.660	2.000	2.589		
	0.4	1.398	1.740	2.057	2.359	2.753	3.427			
	0.5	2.205	2.659	3.064	3.490	4.114				
	0.6	3.119	3.752	4.218	4.797					
	0.7	4.318	4.941	5.480						
	0.8	5.487	6.132							
30°	0.1	0.101	0.150	0.194	0.236	0.276	0.319	0.378	0.495	
	0.2	0.372	0.500	0.621	0.734	0.850	1.005	1.298		
	0.3	0.827	1.051	1.260	1.456	1.697	2.098			
	0.4	1.477	1.801	2.092	2.393	2.830				
	0.5	2.319	2.727	3.075	3.499					
	0.6	3.320	3.738	4.041						
	0.7	4.378								
35°	0.1	0.105	0.153	0.197	0.238	0.278	0.322	0.385	0.525	
	0.2	0.390	0.516	0.634	0.744	0.862	1.035	1.329		
	0.3	0.872	1.088	1.284	1.476	1.741				
	0.4	1.553	1.849	2.105	2.412					
	0.5	2.400	2.713							
	0.6	3.278								
40°	0.1	0.109	0.156	0.200	0.241	0.280	0.325	0.394	0.569	
	0.2	0.411	0.538	0.647	0.753	0.878	1.077			
	0.3	0.919	1.122	1.301	1.496					
	0.4	1.614	1.859	2.085						
	0.5	2.376								
45°	0.1	0.114	0.160	0.203	0.243	0.282	0.329	0.409		
	0.2	0.434	0.553	0.661	0.764	0.899				
	0.3	0.965	1.148	1.306						
	0.4	1.634								