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EUROPÄISCHE KONVENTION FÜR STAHLBAU

ECCS – Technical Committee 7 – « Thin-walled, cold-formed sheet steel in Building »
Technical Working Group 7.5 – "Practical Improvement of Design Procedures"

European Recommendations for the Application of Metal Sheeting acting as a Diaphragm – Stressed Skin Desing

1995

N° 88

PREFACE

In March 1977 the ECCS published "**EUROPEAN RECOMMENDATIONS FOR THE STRESSED SKIN DESIGN OF STEEL STRUCTURES**". At that time it was the first recommendation concerning diaphragm-action in the world. Since then there have been a number of publications which led, in June 1991, to the decision to up-date the document:

- Eurocode No.3 "Design of Steel Structures", Issue 5 of November 1990 became available; finally that has been published as a pre-standard (see [1]).
- Annex A to Eurocode 3 "Cold Formed Steel Sheeting and Members" of January 1991 became available; later this has been changed to EC 3 part 1.3 (see [2]).
- British Standard BS 5950: Part 9 "Code of practice for stressed skin design", 1994 became available, (see [3]).
- An inventory of comments has been collected in the spring of 1991 from people who have worked with the existing European Recommendations.
- Publication of a survey of literature on profiled metal sheeting in shear (see [4]).

The up-dating work has been carried out by a working group of ECCS committee TC-7 "Thin-walled, cold-formed sheet steel in building". The working group ECCS TWG-7.5 "Practical improvement of design procedures" consists of Messrs:

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In comparison with the 1977 version of the European Recommendations for stressed skin design the following main items have been incorporated:

- table 5.1 with design strengths and slip values for fastenings;
- formulation of criteria to prevent end collapse of the sheeting profile;
- criteria for the interaction of local and global buckling of diaphragms;
- design equations for cassettes acting as a diaphragm;
- design procedures for diaphragms stabilising beams against flexural-lateral buckling and columns against flexural buckling;
- important and practical rules for workmanship and erection;
- a section has been added as Annex A which is meant for e.g. architects and contractors. They can find in a nutshell the important aspects to take into account diaphragm action in design and erection before considering stressed skin design in depth.

Special thanks have to be given to Philip Leach of Salford University for his exelent work in preparing the final draft of the worked examples.

April 1995

Ton Tomà

CONTENTS

ANNEXES

1. INTRODUCTION

1.1 *SCOPE*

Even in buildings which are not conceived as stressed skin structures, the decking, roofing or cladding may carry substantial diaphragm forces. This is because the sheeting is often so stiff in shear that it attracts a high shear load. This can be sufficiently large to cause premature failure in certain types of structures.
Stressed skin design quantifies the stresses and deflections which will occur in practice, and leads to a reliable design.
Diaphragm action should be caused primarily by loads which are applied by means of the sheeting e.g. wind and snow loads. Other short term transient loads, e.g. surge forces from light overhead cranes, may also be considered provided that the shear in any diaphragm from this cause does not exceed 30% of the shear capacity of the diaphragm.

These recommendations do not give rules for following situations:

- cantilever roofs,
- applications where deformation of sheet ends is prevented,
- curved roofs,
- roofs where the sheets are laid in a special pattern,
- fully welded sheets to edge members.

Reference [4] can give guidance in those cases.

1. INTRODUCTION

1.1 *SCOPE*

The Recommendations are concerned with the design, construction and use of shear diaphragms acting as a stressed skin in thin walled steel decking, roofing or side cladding in buildings. They also include the design, construction and use of sheet steel in frameless structures or substructures such as folded plates, hyperbolic paraboloids and curved shells.
They also give design recommendations for the effect of profiled steel sheet in lateral bracing to members and diaphragm action in composite floors.

In principle these Recommendations have been formulated for 'steel'. But in some chapters additional information is given concerning deviations for aluminium. Without a reference to material the clause is valid for steel.

1.2 *ASSUMPTIONS*

a. The following assumptions apply:
- Structures are designed by appropriately qualified and experienced personnel.
- Adequate supervision and quality control is provided in factories, in plants and on site.
- Construction is carried out by personnel having the appropriate skill and experience.
- The construction materials and products are used as specified in these Recommendations or in relevant material or product specifications.
- The structure will be adequately maintained.
- The structure will be used in accordance with the design brief.

b. The design procedures are valid only when the requirements for workmanship and erection given in chapter 12 are also complied with.

c. Numerical values identified by boxes ([ ]) are given as indications. Other values may be specified by National Standards or Eurocodes.

1.3 DEFINITIONS

A typical shear panel is shown in figure C.1.1. The components are as follows:

(1) Individual lengths of profiled steel sheeting or decking;
(2) Purlins or secondary members perpendicular to the direction of span of the sheeting;
(3) Rafters or main beams parallel to the direction of span of the sheeting;
(4) Sheet/purlin fasteners;
(5) Seam fasteners between individual sheet widths;
(6) Shear connectors to provide attachment between the rafters and sheeting;
(7) Sheet/shear connector fasteners;
(8) Purlin/rafter connection.

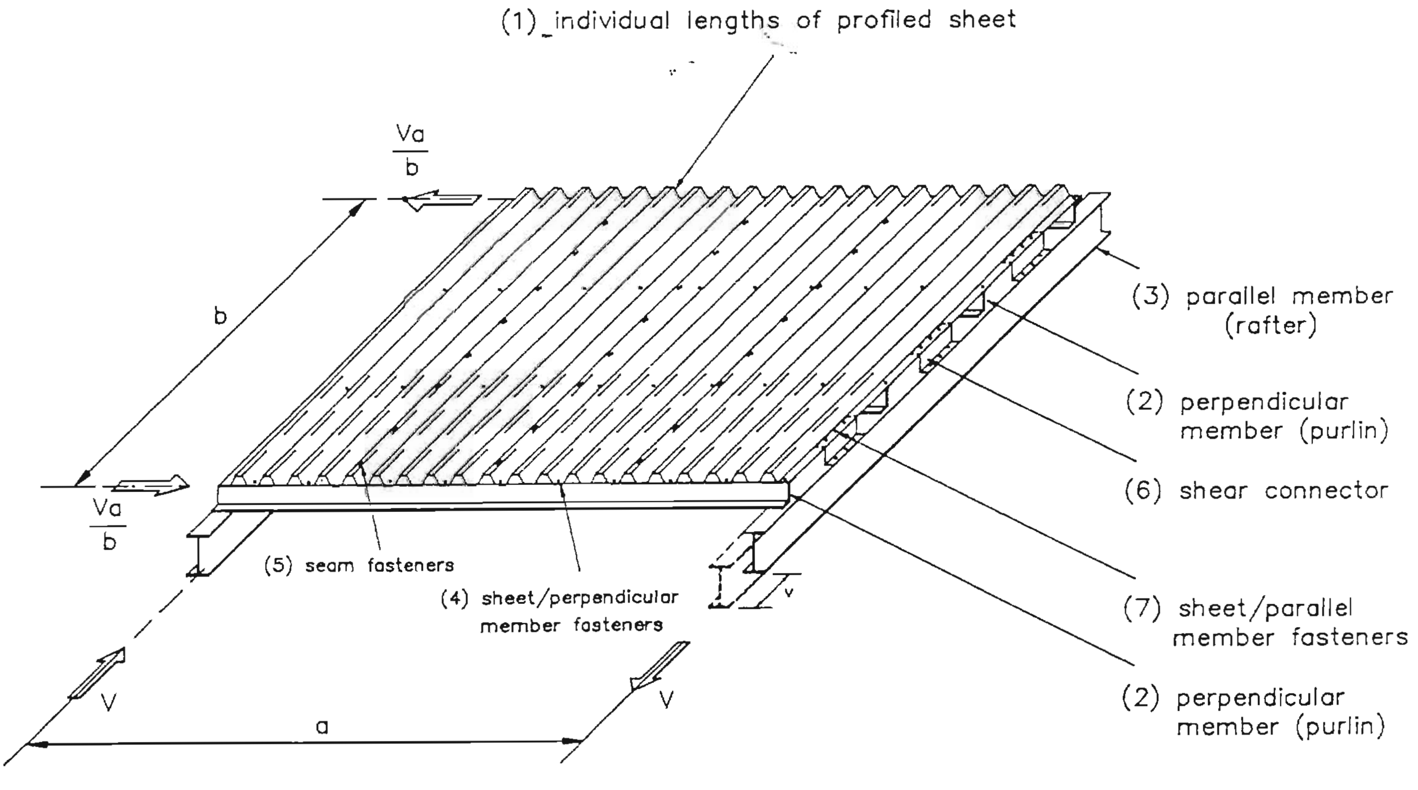


Figure C.1.1: Typical shear panel.

1.3 *DEFINITIONS*

Specific definitions used in stressed skin design are as follows:

1.3.1 *capacity*: Limit of force that can be expected to be carried by a component without causing failure.

1.3.2 *design shear capacity*: Least of the calculated ultimate shear strengths corresponding to the various failure modes of a shear diaphragm.

1.3.3 *diaphragm bracing*: The use of stressed skin diaphragms instead of bracing members to provide lateral support to members or other parts of a structure.

1.3.4 *diaphragm length*: The distance between lines of supports (see chapter 4).

1.3.5 *diaphragm strength*: Capacity of a diaphragm in shear.

1.3.6 *edge member*: Member at the extreme edge of the diaphragm running parallel to the length of the diaphragm.

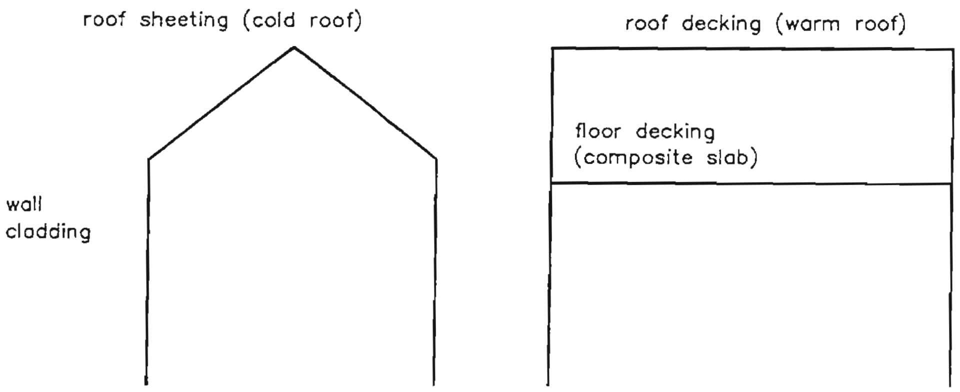
1.3.7 *fastener slip*: The movement at a fastener in the plane of the sheeting per unit shear force per fastener.

1.3.8 *fastener tearing strength*: The shear force per fastener to cause elongation or tearing in the sheeting at the fastener.

1.3.9 *frame flexibility*: Displacement of frame per unit load applied in the direction under consideration.

1.3.10 *frame stiffness*: Reciprocal of frame flexibility.

1.3.11 *shear panel assembly or diaphragm assembly*: Specific term for an assembly of one or more shear panels.



Liner sheets are the inner skin of double skin sheeting.

Figure C.1.2: Definitions of sheeting, decking etc.

1.3.12 *purlin*: Member supporting the sheeting and perpendicular to the corrugations for sheeting spanning perpendicular to the length of the diaphragm. May also be referred to as perpendicular member or secondary member.

1.3.13 *rafter*: (a) Member supporting the purlins for sheeting spanning perpendicular to the length of the diaphragm (in this case, may be referred to as parallel member or main beam).

(b) Member supporting the sheeting for sheeting spanning parallel to the length of the diaphragm (in this case, may be referred to as perpendicular member or main beam).

1.3.14 *roof or floor shear panel*: Shear panel in a roof or floor.

1.3.15 *shear connector*: Short length of section to enable attachment of the sheeting to the third and fourth side of a shear panel when the side members are not all at the same level.

1.3.16 *shear diaphragm or diaphragm*: General term for one or more shear panels or that area of sheeting which resists in-plane displacement by shear.

1.3.17 *shear flexibility or diaphragm flexibility*: In-plane displacement of a shear panel or diaphragm per unit shear load (see 4.2.2).

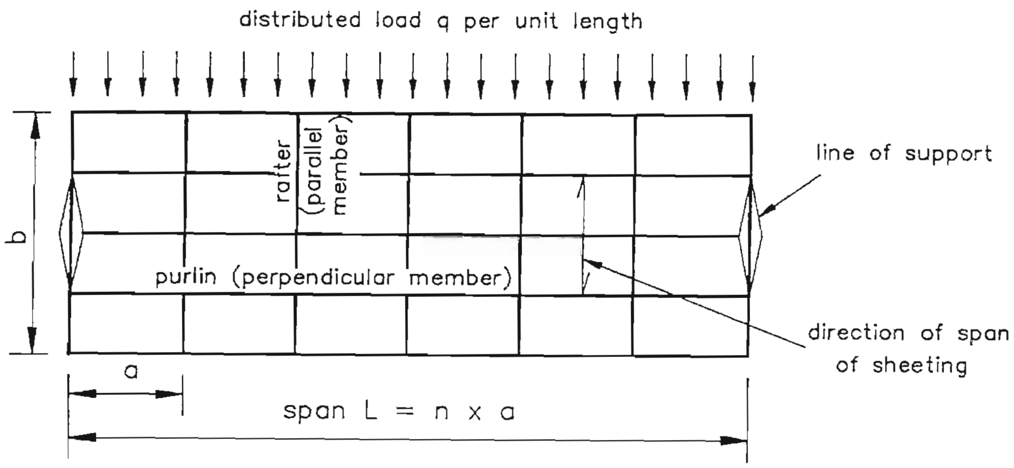
1.3.18 *shear panel*: Shear panel of sheeting subjected to in-plane shear and bounded by edge members on two sides and rafters on the two other sides.

1.3.19 *shear stiffness*: Shear load per unit in-plane displacement of a shear panel or diaphragm (reciprocal of shear flexibility).

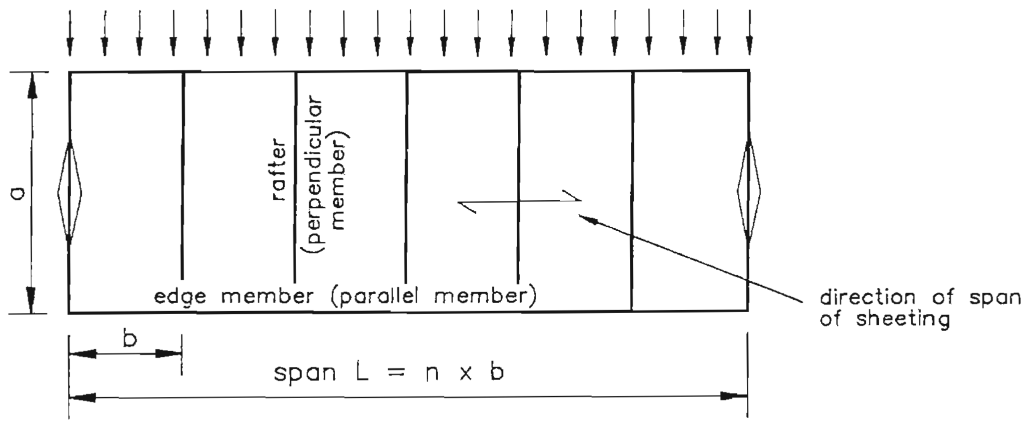
1.3.20 *shear strength*: Capacity of a shear panel in shear.

1.3.21 *sheeting*: Generic name for roof and floor decking, roof sheeting and side cladding.

1.4 *MAJOR SYMBOLS*



a) Sheeting spanning perpendicular to length of building



b) Sheeting spanning parallel to length of building

Figure C.1.3: Direction of span of the sheeting.

1.3.22 *stressed skin or shear diaphragm*: General term to describe a structure or component in which in-plane shear in the sheeting is taken into account in design.

1.3.23 *stressed skin action or diaphragm action*: Structural behaviour involving in-plane shear in the sheeting and forces in the edge members.

1.4 *MAJOR SYMBOLS*

Note: The preferred units are shown in ( ). The symbols used in Annex B are defined separately in that section.

a Dimension of shear panel in a direction perpendicular to the corrugations (mm).

A Cross sectional area of longitudinal edge member (mm^2);
or Semi-area of fold line member (mm^2);
or Area of openings in shear panels (mm^2).

A_d Total area of openings in a shear panel (mm^2).

b Dimension of shear panel in direction parallel to the corrugations (mm);
or Depth of web in folded plate roof (mm).

B Width of a diaphragm composed of cassettes (mm).

B_o Width of the wide flange of a cassette (mm).

B_u Width of the narrow flange of a cassette (mm).

c Total shear flexibility of a shear panel (mm/kN).

$c_{1.1}$ etc Component shear flexibilities (mm/kN).

d Pitch of corrugations (mm).

D_x, D_y Orthogonal bending stiffnesses of profiled sheet per unit length (kNmm²/mm).

E Modulus of elasticity (kN/mm²).

f_u Ultimate tensile strength of steel (kN/mm²).

f_y Design yield strength of steel (kN/mm²).

F_p Design strength of individual sheet/purlin fastener (kN);
or Design strength per fastener between the sheeting and the stabilised element (kN).

F_p^1 Design shear load on a fastener (kN).

F_{pr} Design strength of purlin/rafter connection (kN).

F_s Design strength of individual seam fastener (kN).

F_{sc} Design strength of individual sheet/shear connector fastener (kN).

F_t Design tensile strength of a fastener (kN).

F_t^1 Design tensile load in a fastener (kN).

G Shear modulus (kN/mm²).

h Height of sheeting profile (mm).

H Height of a cassette (mm).

I Second moment of area of a single corrugation about its neutral axis (mm⁴).

I_1 Moment of inertia of the wide flange of a cassette per unit width (mm^4/mm).

k Frame flexibility (mm/kN).

k_{sp} Frame flexibility due to spread (mm/kN).

k_{sw} Frame flexibility due to sway (mm/kN).

K_1, K_2 Sheeting constants for every corrugation fastened and alternate corrugations fastened.

ℓ Width of the top or bottom flange of the sheeting, whichever is wider (mm).

ℓ_S Spacing of seam fasteners in cassettes (mm).

L Length of diaphragm assembly between braced frames (mm);
or Length of folded plate roof between gables (mm);
or Length of the diaphragm in the span direction of the cassettes (mm).

m Number of shear panels spanned by a single sheet length (for sheeting spanning parallel to the length of the diaphragm; see table 5.10).

n Number of shear panels in the length of the diaphragm assembly.

n_b Number of sheet lengths within depth of diaphragm (for sheeting spanning perpendicular to the length of the diaphragm).

n_f Number of sheet/purlin fasteners per member per sheet width;
or Number of fasteners between sheet and stabilised element per sheet width.

n_ℓ Number of sheet lengths in the length of the diaphragm assembly.

n_p Number of purlins (edge + intermediate) within a panel.

n_s Number of seam fasteners per side lap (excluding those which pass through both sheets and the supporting purlin).

n_{sc} Number of sheet/shear connector fasteners per end rafter;
or Number of gable fasteners per gable in a folded plate roof.

n'_{sc} Number of sheet/shear connector fasteners per internal rafter.

n_{sh} Number of sheet widths per shear panel.

p Pitch of sheet/purlin fasteners (mm).

P Shear panel point load on diaphragm (kN).

P_{ult} Ultimate load at a shear panel point (kN).

q Distributed in-plane line load on diaphragm (kN/mm).

r Relative flexibility of shear panel to frame.

R Restraining force provided by the sheeting at each intermediate frame (kN).

s_p Slip per sheet/purlin fastener per unit load (mm/kN).

s_{pr} Deflection of top of purlin at purlin/rafter connection per unit load (mm/kN).

s_s Slip per seam fastener per unit load (mm/kN).

s_{sc} Slip per sheet/shear connector fastener per unit load (mm/kN).

S Shear stiffness (kN/mm).

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t Net sheet thickness (steel core thickness), excluding metalic and other coatings (mm).

t_N Nominal sheet thickness inclusive of metalic coating (mm).

u Perimeter length of a complete single corrugation (mm).

v Shear displacement of diaphragm (mm).

v_o Shear displacement of the diaphragm parallel to the corrugations (mm).

V Applied shear force on diaphragm (kN).

V^* Design shear capacity of diaphragm (kN).

V_g Global shear buckling strength of the diaphragm (kN).

V_ℓ Local shear buckling strength of the diaphragm (kN).

V_o Shear force on the diaphragm parallel to the corrugations (kN).

V_{red} Reduced shear buckling capacity of diaphragm due to interaction of global and local buckling (kN).

V_{ult} Strength associated with a given failure mode (kN). or Ultimate load (kN).

w Vertical load per unit plan area of folded plate roof (kN/mm²).

α Coefficient of thermal expansion.

α_1 to α_5 Factors to allow for intermediate purlins, number of sheet lengths, and sheet continuity.

Comments C.9

β_1, β_2 Factors to allow for the number of sheet/purlin fasteners per sheet width.

β_3 (Distance between outermost fasteners across sheet width)÷(sheet width).

γ Shear strain.

γ_f Overall load factor.

Δ Midspan deflection of a shear panel assembly (mm).

η Reduction factor for frame forces and moments.

ρ Density.

θ Inclination of web of sheeting profile to the vertical;
or Inclination of folded plate roof to the horizontal;
or Inclination of rafter to the horizontal.

ν Poisson's ratio.

2. BASIS OF DESIGN

In chapter 2 of Eurocode 3 the following have been treated:

- Fundamental requirements,
- Definitions and classifications,
- Design requirements,
- Durability,
- Fire resistance.

In design the structural behaviour of diaphragms may be neglected, but then it is necessary that the sheeting, including connections, possess sufficient deformation capacity. This criterion is usually met when the seam fastenings possess sufficient deformation capacity.

2. BASIS OF DESIGN

For the general basis of design reference should be made to chapter 2 of Eurocode 3 (see reference [1]).

The design of any structure or its parts may be carried out by one of the methods given in a to e below:

a. *Stressed skin design.*
The cladding is treated as an integral part of the main structure and provides shear diaphragms which are used to resist structural displacement in the plane of the cladding. This method of design may be used in conjunction with any of the methods given in b to e.

b. *Simple design.*
The connections between members are assumed not to develop moments adversely affecting either the members or the structure as a whole.

c. *Rigid design.*
The connections are assumed to be capable of developing the strength and/or stiffness required by an analysis assuming full continuity. Such analysis may be made using either elastic or plastic methods.

d. *Semi-rigid design.*
The connections provide a predictable degree of interaction between members beyond that of simple design but less than that of rigid design.

e. *Composite design.*
Composite design takes into account the enhanced load capacity and serviceability when steelwork is suitably interconnected to other materials, e.g. concrete, timber and building boards, so as to ensure composite behaviour of the member or structure.

3. MATERIALS AND COMPONENTS

For galvanised sheeting with 275 gr/m^2 for both sides, the total zinc thickness is 0.04 mm.

3. MATERIALS AND COMPONENTS

3.1 *GENERAL*

3.1.1 Sheeting profiles

The provisions of these European Recommendations apply primarily to profiled steel sheeting as used in the roofs, floors and walls of buildings. The calculation procedure refers mainly to trapezoidal profiles and cassettes but can include curved corrugated sheeting and re-entrant angle profiled sheets as used in sheet steel/concrete floors.
Other types of steel sheeting, decking and cladding such as built-up sections and sandwich panels may also be used for stressed skin construction but the shear strength and stiffness of such types must be determined by testing in accordance with chapter 11.

3.1.2 Section properties

In the calculation of section properties for shear diaphragms it is sufficient to assume that the material is concentrated at the mid-line of the section, and that the actual round corners are replaced by intersections of the flat elements.

3.2 *THICKNESS*

3.2.1 Range of thicknesses

The provisions of these European Recommendations apply primarily to sheeting with a thickness of not more than 1.5 mm. Although the use of thicker material is not precluded, special design considerations may apply.

3.2.2 Core thickness

The core thickness of the material should be taken as the nominal base metal thickness exclusive of coatings.

3.3 *PROPERTIES OF MATERIALS*

3.3.1 Properties of steel

3.3.1.1 General

These European Recommendations cover mainly the design of shear diaphragms made of steel supplied to Eurocode 3 Part 1.3. Other steels may be used, subject to the approval of the engineer, provided due allowance is made for variation in properties, including ductility.

3.3.1.2 Strength of steel sheet

The design yield strength of the steel sheet should be taken in accordance with Eurocode 3 Part 1.3.

3.3.1.3 Other properties of steel

The following properties of steel may be assumed in design:

- Modulus of elasticity $E = 210$ kN/mm^2
- Shear modulus $G = 81$ kN/mm^2
- Poisson's ratio $\nu = 0.3$
- Coefficient of linear thermal expansion $\alpha = 12 \times 10^{-6}$ per °C
- Density $\rho = 7850$ kg/m^3

3.3.2 Properties of aluminium

3.3.2.1 General

These European Recommendations cover mainly the design of shear diaphragms made of steel, although some clauses refer to aluminium. This aluminium should be supplied to EN 508-2: 1993. Other types of aluminium may be used, subject to the approval of the engineer, provided due allowance is made for variation in properties, including ductility.

3.3.2.2 Strength of aluminium sheet

The design yield strength of the aluminium sheet should be taken in accordance with EN 508-2: 1993.

3.3.2.3 Other properties of aluminium

The following properties of aluminium may be assumed in design:

- Modulus of elasticity $E = 70$ kN/mm^2
- Shear modulus $G = 27$ kN/mm^2
- Poisson's ratio $\nu = 0.3$

Comments C.13

- Coefficient of linear thermal expansion $\alpha = 23 \times 10^{-6}$ per °C
- Density $\rho = 2700$ kg/m³

3.4 *FASTENERS FOR STRESSED SKIN ACTION*

3.4.1 Sheet/member fasteners

The sheeting should be attached with fasteners which carry shear forces without reliance on friction or bending of the fasteners themselves. The fasteners should be of a type which will not work loose in service and which will neither pull out nor fail in shear before causing tearing of the sheeting. Examples of suitable fasteners are self-tapping or self-drilling screws, shot pins (cartridge fired or air driven), bolts or welding. Hook bolts, clips or other fasteners which transmit shear forces by friction are not suitable.

3.4.2 Seam fasteners

The seams between adjacent sheets should be fastened by fasteners of a type which will not work loose in service and which will neither pull out nor fail in shear before causing tearing of the sheeting. Examples of suitable fasteners are self-drilling screws, monel metal or stainless steel blind rivets, bolts or welding. Aluminium blind rivets are not generally suitable.

3.4.3 Strength of fasteners

The characteristic tearing strengths of fastenings may be determined by tests according to ref. [5] and [6]. The design strength of fastenings should be determined by dividing the characteristic strength by a material factor of [1.25]. For a number of fastener types conservative values can be determined from table 5.1 or from formulae given in Eurocode 3 Part 1.3.

3.4.4 Slip of fasteners

The slip values of fasteners in sheeting may be determined by tests in accordance with ref. [5]. Typical values for commonly used fasteners are given in table 5.1 for serviceability loading conditions. Within the range of sheet thicknesses given, the slip values listed in table 5.1 may be taken to be independent of t and f_y.

b. In other types of shear diaphragm, if the top of the purlins (secondary members) and the top of the rafters (main beams) are at the same level, shear connectors are not necessary in order to fasten all four sides.

3.5 *DIAPHRAGM COMPONENTS*

The following considerations apply to diaphragms:

a. Whenever possible, all four sides of a shear panel should be fastened.

b. The shear diaphragm shown in figure C.1.1 is typical of a sloping roof shear panel in which the purlins pass over the rafters. The use of shear connectors enables all four sides of the shear panel to be fastened.

c. If shear connectors are not used in figure C.1.1, so that sheeting is fastened only on two sides (to the purlins), it is essential that the purlin/rafter connections are sufficiently strong to transmit shear loads from the rafter into the diaphragm.

d. In a building in which shear diaphragms are fastened only on two sides, shear connectors or their equivalent (e.g. end closures) must be used on the lines of support of the diaphragm.

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4. DESIGN PRINCIPLES

4.1 *SUITABLE FORMS OF CONSTRUCTION*

4.1.1 General

1. In pitched roof frames, the flatter the roof pitch, the less effective the diaphragms are in resisting vertical load, but the more effective they are in resisting side load.

2. If the roof pitch is less than 10° it is unlikely that the diaphragms have a significant effect in resisting vertical load.

3. Diaphragm action in the roof sheeting is more likely to have a significant effect in buildings where the length/width ratio does not exceed the following :

Flat roof buildings under horizontal load		4.0
Pitched roof buildings	• under vertical load	2.5
	• under horizontal load	4.0

4. Stressed skin action is particularly effective in buildings where horizontal load is applied to one or two frames only, and in such a case the load distribution is not affected by the length/width ratio.

5. Stressed skin action is also effective in providing lateral restraint to beams and trusses and in providing diaphragm bracing to end gables and eaves of buildings.

4. DESIGN PRINCIPLES

4.1 *SUITABLE FORMS OF CONSTRUCTION*

4.1.1 General

Stressed skin action may be taken account of in design in accordance with the following principles:

a. **Flat roofs**

In a flat roofed building subjected to side load, as shown in figure R.4.1a, each of the roof shear panels may be assumed to act as a shear diaphragm taking load back to the gable ends which are stiffened in their own planes by bracing or shear diaphragms. The action of the roof sheeting may be assumed to cause the roof to behave like a deep plate girder. Under in-plane load, the end gables may be assumed to take the reactions, the sheeting may be assumed to act as a web and take the shear, and the edge members may be assumed to act as flanges and take the axial tension and compression. The sheeting should not be assumed to help the frames resist bending due to any vertical load; it should only be assumed to help resist in-plane displacements.

b. **Pitched roofs**

In a pitched roof building subjected to a vertical load, as shown in figure R.4.1b, there is a component of load down the roof slope so that, if the gables are tied, the roof diaphragms may be assumed to help the building from spreading. If the gables are braced or sheeted, the roof diaphragms may also be assumed to have a significant effect in reducing sway of the building under side load.

c. **Length**

The length of a building should be taken as the distance between gable frames, but where special intermediate stiffened frames are provided, the length should be taken as the distance between these frames.

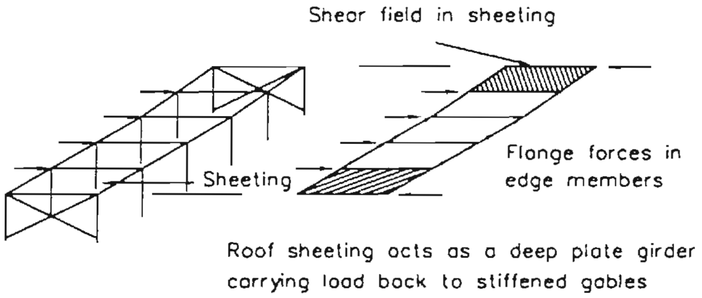
d. **Load distribution**

Where horizontal load, such as crane surge, is applied to one or two frames only, stressed skin action may be used to distribute the load to a number of frames.

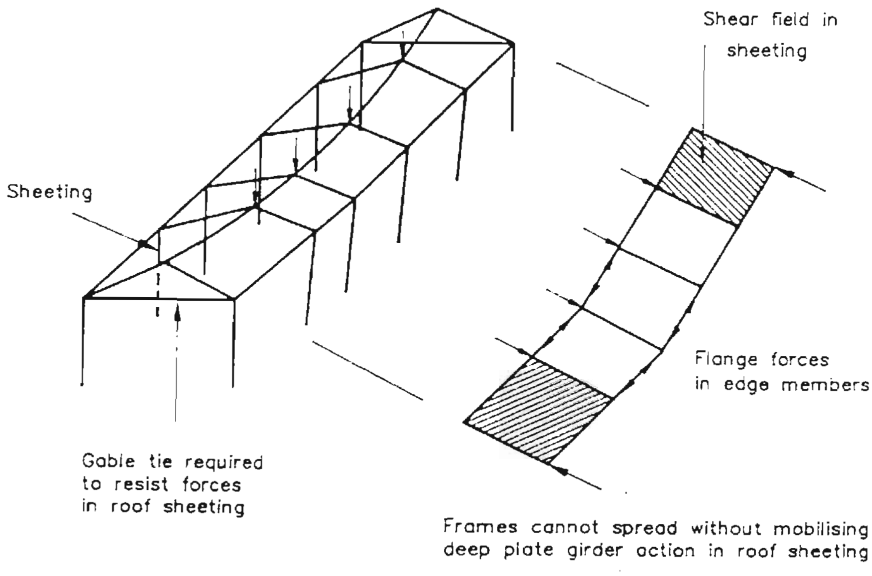
e. **Restraint on beams**

Stressed skin action may be used to provide restraint to members (beams and beam-columns). Where the compression flange is restrained,

efficient restraint is provided against lateral buckling. Where the tension flange is restrained (with the compression flange left unrestrained), economical design may result in considering some degree of restraint against twisting and rotation (especially when dealing with the overall buckling in the torsional mode). In the case of light gauge members such as purlins and cassettes, provisions for the design of the resulting systems are given in Eurocode 3 Part 1.3.



a. Diaphragm action in a flat roofed building



b. Diaphragm action in a pitched roof building

Figure R.4.1: Stressed skin action in buildings.

4.1.2 **Load sharing**

Where the frames are braced during erection, there is no requirement for them to carry the full unfactored horizontal load alone.

4.1.3 **Applications**

For multi-strorey buildings, reference should be made to report 77/95 to ECCS technical committee AC1 "The design of multi-storey buildings stiffened by diaphragm action" by J.M. Davies, August 1978 (reference [7]).

4.1.4 **Repeated or dynamic loadings**

Reversal of loading is nearly always present in blast loadings.

Repeated loadings (static or dynamic) may make connections and other support devices work loose with permanent effects (e.g. as a consequence of elongation of holes). It may therefore be necessary to take a correspondingly higher value of shear flexibility in the deflection calculations.

4.1.2 **Load sharing**

Where the frames shown in figure R.4.1a are pin jointed, the side loads should be assumed to be resisted entirely by stressed skin action. In this case it is essential that the structure is adequately braced during erection and that the sheeting panels are not removed without proper consideration.

Where the frames of figure R.4.1a have rigid joints, the side loads may be assumed to be shared between the frames and the diaphragms (refer to chapter 7 to see how to satisfy this requirement). Where the frames are not braced during erection, they should be designed to carry the full unfactored load without collapse. In the sheeted building the diaphragms may then be assumed to provide the additional resistance required to carry the factored load.

4.1.3 **Applications**

Stressed skin design may be applied to low-rise flat roof buildings under horizontal loads such as wind, crane surge and seismic forces. It may also be applied to pitched roof buildings under horizontal loads and/or vertical loads such as snow.

Stressed skin action may also be applied to buildings which are arched or which have curved shear panels. In such cases the design should take account of the developed length of the sheeting.

Multistorey buildings may also be designed by stressed skin methods, in which case the floors, in addition to the roof, may be designed as horizontal diaphragms to resist horizontal loads such as wind and seismic forces. Stressed skin design should not normally be applied to the vertical frames of tall multistorey buildings in which frame instability is a consideration. However, if it is applied to such frames, special measures should be taken to ensure the permanence and effectiveness of the diaphragm and its connections.

4.1.4 **Repeated or dynamic loadings**

No special allowance for repeated or dynamic loading need normally be made in determining the design strength and shear flexibility of a diaphragm. For unusually severe cases of such loadings (especially when reversal of loading is implied), the shear flexibility of a diaphragm should be increased by 50%

4.2 *CONDITIONS AND RESTRICTIONS*

4.2.1 Necessary conditions

b. The usual rule consists in checking that no yielding occurs under the standard combinations of loads (as defined in Eurocode 1 and Eurocode 3), applied to the whole structure including the sheeting.
In stressed skin design, the bending stress and the shear stress as defined here are respectively given by two different functions of the sheeting and two different loading systems (flexural for out of plane local loads and membrane for the whole structure under the loads considered for diaphragm action).

The present rule provides a simple combination formula to prevent overstressing and to ensure that any deterioration of the sheeting would be apparent in bending before it would be apparent in diaphragm action. If a more exact interaction formula is required, reference should be made to Eurocode 3, Part 1.3, section 4.9.

However, in the most extreme case where the bending stress reaches f_y and the shear stress equals 0.25 f_y (assuming in addition that the loading system is the same), the maximum principal stress is 6 % greater than f_y.

when this rule leads to a more unfavourable design. This rule does not apply to the diaphragm itself.

4.2 *CONDITIONS AND RESTRICTIONS*

4.2.1 Necessary conditions

Stressed skin diaphragms should satisfy the following conditions :

a. The use made of the profiled steel sheeting, in addition to its primary purpose, should be limited to the formation of shear diaphragms to resist structural displacement in the plane of the sheeting.

b. The sheeting should first be designed for its primary purpose in bending according to Eurocode 3 Part 1.3 (either by calculation or by testing). It should then be checked that the maximum shear stress due to diaphragm action does not exceed 25% of the design yield (normal) stress.

c. It may be assumed in design that transverse load on a panel of sheeting will not affect its strength or flexibility as a shear diaphragm.

d. Diaphragm forces in the roof or floor planes should be transmitted to the foundations by means of braced frames, stressed skin diaphragms, or other methods of sway resistance.

e. Structural connections of adequate strength and stiffness should be used to transmit diaphragm forces to the main steel framework.

f. Diaphragms should be provided with edge members. These members, and their connections, should be sufficient to carry the flange forces arising from diaphragm action.

g. Sheeting used as a stressed skin diaphragm should be fastened in accordance with 3.4.1 through every trough or alternate troughs.

h. The seams between adjacent sheets should be fastened in accordance with 3.4.2 at a spacing not exceeding 500 mm.

i. The distances from the fasteners to the edges and ends of the sheets should comply with 5.3.1.

4.2.2 Restrictions

Stressed skin diaphragms are subject to the following restrictions :

a. Diaphragms should not be used to resist permanent external loads (except for lateral support to beams and the dead load from light weight construction) but should be predominantly restricted to resisting

(1) loads applied through the cladding, such as wind loads and snow loads, and

(2) seismic forces and other similar (in magnitude and frequency) transient loads,

(3) crane forces.

Note: the force induced in any fastener or group of fasteners by horizontal surge or braking effects from overhead cranes should not exceed 30% of the capacity of the fastenings.

b. Stressed skin diaphragms should be treated as structural components and should not be removed without consideration of the effect on the stability of the building. Such consideration should not invalidate planned removal of areas of sheeting for maintenance purposes, provided the remaining areas are adequate as a diaphragm or temporary bracing is provided during maintenance.

c. The calculations, drawings and contract documents should draw attention to the fact that the building incorporates stressed skin diaphragms, subject to National rules.

d. Openings totalling more than 3% of the area in each shear panel should not be permitted unless they are treated in accordance with 8.3. Openings of less than this amount may be permitted without special calculation provided the total number of fasteners in each seam with openings is not less than that in a seam without openings.

e. Stressed skin diaphragms should be designed predominantly for short-term imposed loads, unless long term phenomena such as creep are taken into account.

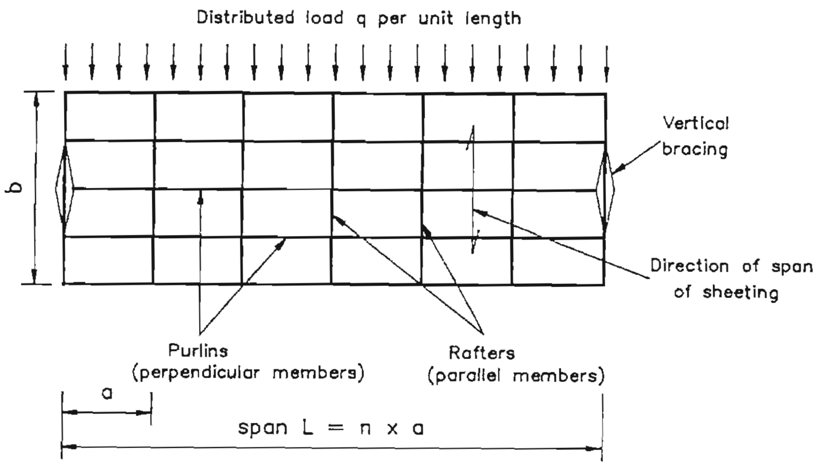
f. In accordance with 4.1.2, stressed skin buildings in which the frames have not been designed to carry the full unfactored load without collapse should be braced during erection. Buildings which utilize the roof or floors as stressed skin diaphragms should be erected so that the roof and floors are sheeted before the walls are cladded.

g. The structural effects of building modifications on stressed skin buildings should be checked. Changes in use or occupancy which might affect the original design assumptions should be noted in the contract documents and notified to the appropriate authority.

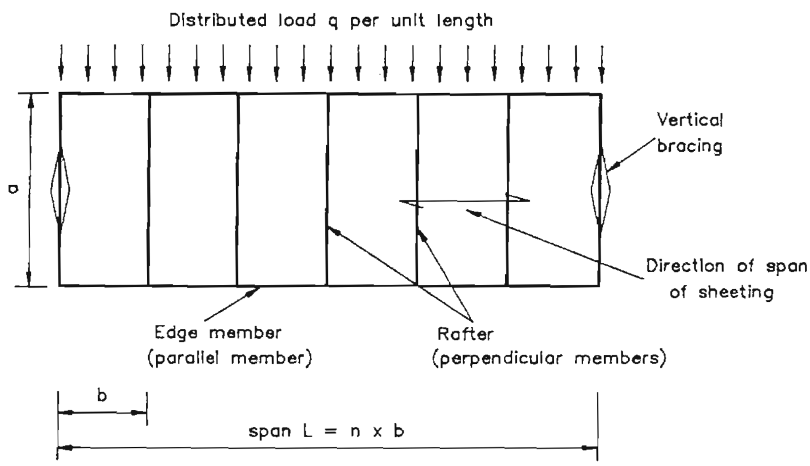
4.3 *TYPES OF DIAPHRAGM*

4.3.1 Direction of span

The sheeting may span perpendicular to the length of the diaphragm (see figure R.4.2a) or parallel to the length of the diaphragm (figure R.4.2b).



a. Sheeting spanning perpendicular to the length of the building



b. Sheeting spanning parallel to the length of the building

Figure R.4.2: Direction of span of the sheeting.

4.3.2 Diaphragm components

For diaphragm components see figure C.1.1 clause 3.5.

4.3.3 Complex diaphragms

In additon to regular diaphragms as found in simple flat or pitched roof buildings, complex diaphragms may occur due to irregular shapes and geometry (e.g. various orientations, large openings) or severe in-plane loadings. Besides the basic requirements as applied to simple diaphragms, special rules are given in chapter 8.

4.3.4 Fastener arrangements

Various sheet/member fastener arrangements may be used for diaphragms, as shown in figure R.4.3. Cases (1) and (2) require that the tops of the members are at the same level or that shear connectors are used, allowing four sides of the shear panel to be fastened. Cases (3) and (4) occur when the tops of the members 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 connections to the edge members.

Whenever practicable, four sides of the shear panel should be fastened in order to give the diaphragm greater strength and stiffness. Fasteners in every trough, rather than alternate troughs, give the diaphragm a greatly improved stiffness.

4.4.1 **Diaphragm strength**

The acceptable modes of failure are modes (a) and (b) in 4.4.1 of the Recommendations, and the design criteria should be based on these modes. The remaining modes, being less ductile, should have greater reserve of safety. The lesser of the strengths of modes (a) and (b) is the diaphragm strength and refers to the direction parallel to the corrugations.

The preferred unit for diaphragm strength is kN.

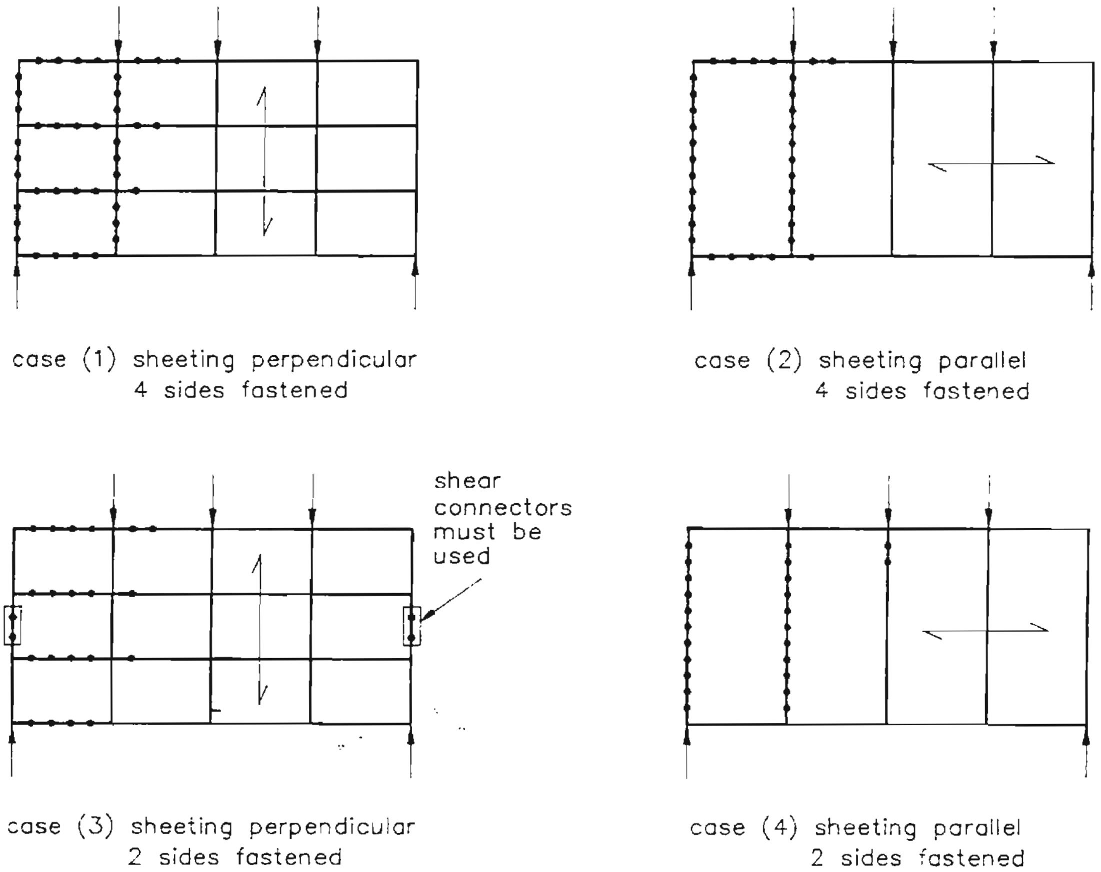


Figure R.4.3: Fastener arrangements.

4.4 *DESIGN CRITERIA*

4.4.1 Diaphragm strength

With reference to figure R.4.4, the possible criteria for diaphragm strength are as follows:

a. sheet tearing along a line of seam fasteners (figure R.4.4a)
b. sheet tearing along a line of shear connector fasteners (figure R.4.4b)
c. sheet tearing in the sheet/purlin fasteners (figures R.4.4c and R.4.4d)
d. end collapse of the sheeting profile (figure R.4.4e)
e. shear buckling of the sheeting (figure R.4.4f)
f. failure of the edge member in tension or compression (figure R.4.4g)

The design expressions for the above criteria are given in Chapter 5.

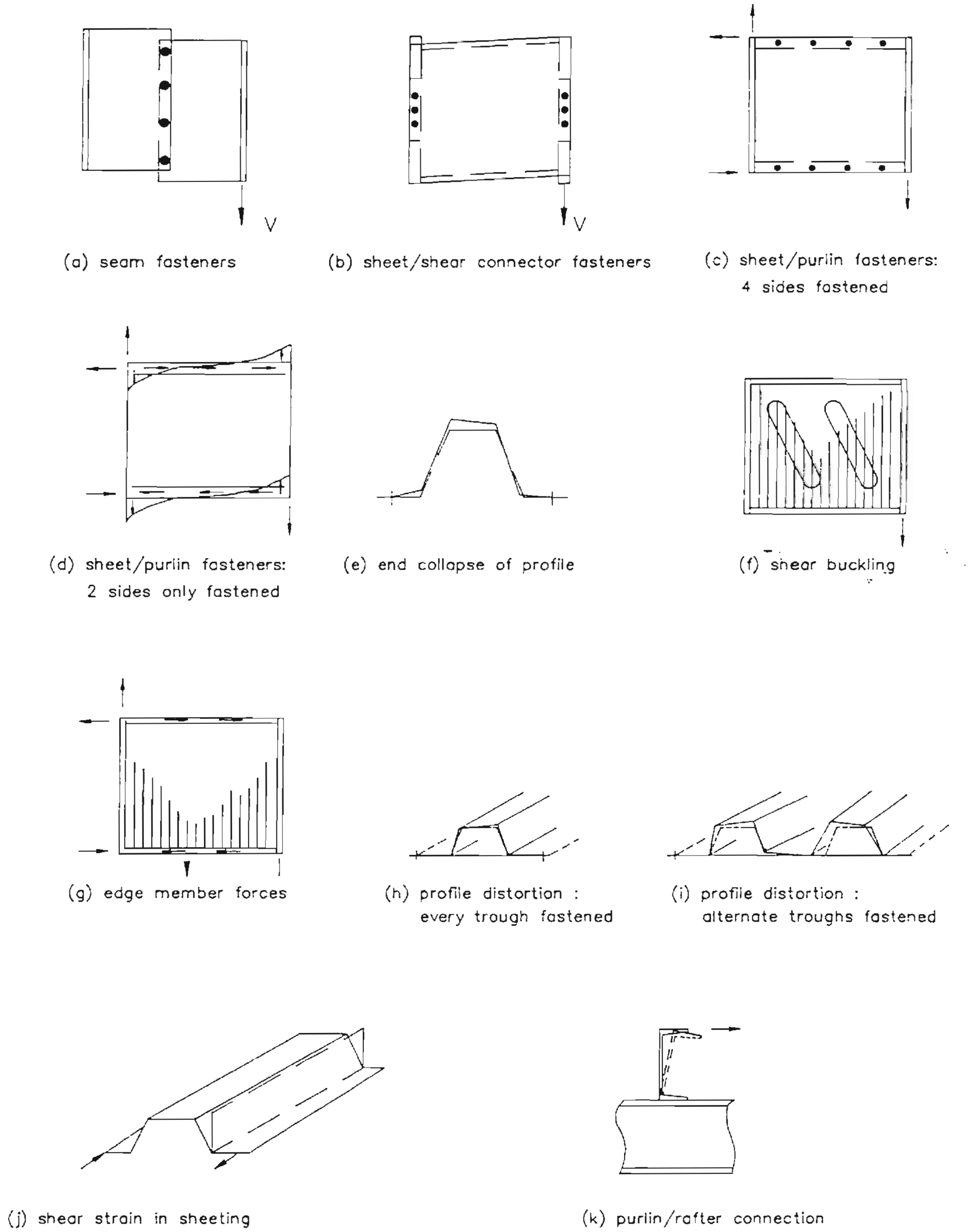


Figure R.4.4: Design criteria for diaphragm strength and flexibility.

4.4.2 Diaphragm stiffness and flexibility

The preferred units for the shear load and shear displacement are kN and mm respectively. Hence, the preferred unit for shear flexibility is mm/kN (and for shear stiffness, kN/mm).

4.4.2 Diaphragm stiffness and flexibility

With reference to figure R.4.5, the displacement of the shear diaphragm is v under the shear load V. The shear flexibility of the diaphragm is $c = v/V$ and the shear stiffness S is the reciprocal of this, $S = V/v$. Throughout these Recommendations (except in chapter 6 and Annex B), the term shear flexibility is used in preference to shear stiffness; it is defined as the shear deformation per unit shear load, and refers to the direction parallel to the corrugations.

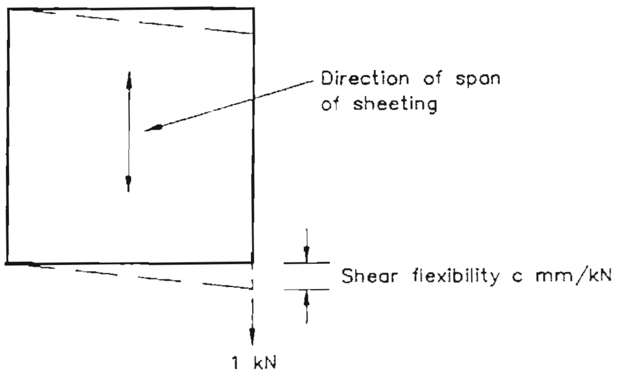


Figure R.4.5: Shear flexibility

With reference to figure R.4.4, the total shear flexibility of a shear panel is the sum of the separate component shear flexibilities due to the following :

a. profile distortion (figures R.4.4h and R.4.4i)

   Remark: limits for the application of the expression for profile distortion are $b/d \geq 10$.

b. shear strain in the sheet (figure R.4.4j)

c. slip in the sheet/purlin fasteners (figures R.4.4c and R.4.4d)

d. slip in the seam fasteners (figure R.4.4a)

e. slip in the sheet/shear connector fasteners (figure R.4.4b)

f. purlin/rafter connections (in the case of the sheet fastened to the purlins only; figure R.4.4k)

g. axial strain in the longitudinal edge members (figure R.4.4g).

5. DESIGN EXPRESSIONS FOR TRAPEZOIDAL PROFILES

Note: All the design expressions given in this chapter are for the diaphragm acting by itself (without collaboration from the frames). The preferred units are given in 1.4.

5.1.1.1 Seam capacity

The definitions for sheeting (seam fasteners in the crests) and decking (seam fasteners in the troughs) are illustrated in figure C.5.1.

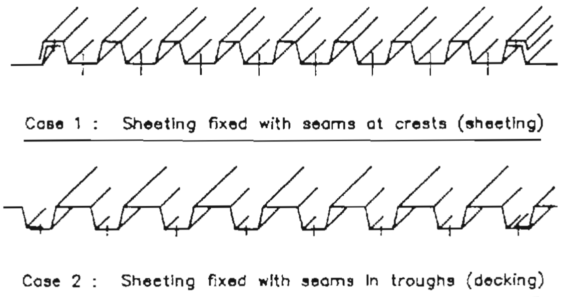


Figure C.5.1: Definitions of sheeting and decking.

5. DESIGN EXPRESSIONS FOR TRAPEZOIDAL PROFILES (STEEL SHEETING)

(For aluminimum sheeting see Chapter 6)

5.1 *DESIGN EXPRESSIONS FOR STRENGTH: SHEETING SPANNING PERPENDICULAR TO LENGTH OF DIAPHRAGM*

5.1.1 Strength of shear panel assemblies

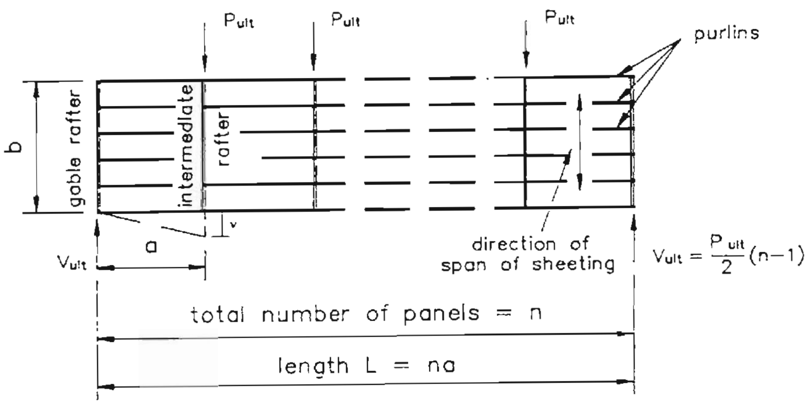


Figure R.5.1: Shear panel assembly: sheeting spanning perpendicular to length of diaphragm.

With reference to figure R.5.1, the shear strength of the shear panel assembly should normally be checked by considering failure modes in the end shear panels and at the internal rafters. The ultimate strengths V_{ult} associated with these failure modes may be obtained as follows:

5.1.1.1 Seam capacity

$$V_{ult} = n_s F_s + \frac{\beta_1}{\beta_3} n_p F_p$$

where n_s is the number of seam fasteners per side lap (excluding those which pass through both sheets and the supporting purlin),

F_s is the design strength of an individual seam fastener. Values are given in table 5.1,

n_p is the number of purlins (edge + intermediate),

F_p is the design strength of an individual sheet/purlin fastener. Values are given in table 5.1,

Table 5.1 Design strengths and slip values of fasteners

(1) Sheet/purlin and sheet/shear connector fasteners

	Washer type	Overall dia. (mm)	Design shear strength F_p and F_{sc} (kN)	Slip s_p and s_{sc} (mm/kN)
Screws	Collar head	5.5 6.3	$1.9\ f_u d_n t$ max. 6.5 $1.9\ f_u d_n t$ max. 8.0	0.15
	Collar head + neoprene washer	5.5 6.3	$1.9\ f_u d_n t$ max. 6.5 $1.9\ f_u d_n t$ max. 8.0	0.35
Fired pins	ϕ 23 mm steel washer	3.7 to 4.8	$2.9\ f_u d_n t$ max. 8.0	0.10

(2) Seam fasteners (no washers)

	Overall dia. (mm)	Design shear strength F_s (kN)	Slip s_s (mm/kN)
Screws	4.1 to 4.8	$2.9(t/d_n)^{1/2} * f_u d_n t$ max 3.8	0.25
Steel or monel blind rivets	4.8	$3.2(t/d_n)^{1/2} * f_u d_n t$ max 3.0	0.30

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

(2) Important The comments and recommendations given in 5.3.2 should be taken into account before using the above design strengths and slip values.

(3) See comment to 5.3.2 regarding the materialfactor included in this table 5.1.

β_1 is a factor to allow for the number of sheet/purlin fasteners per sheet width. Values are given in table 5.2,

β_3 $=(n_f - 1)/n_f$ for sheeting (seam fasteners in the crests),

β_3 $=1.0$ for decking (seam fasteners in the troughs),

n_f is the number of sheet/purlin fasteners per sheet width (including those at the overlaps).

5.1.1.2 Shear connector fastener capacity (at end gables)

$$V_{ult} = n_{sc} F_{sc}$$

where n_{sc} is the number of sheet/shear connector fasteners per end rafter (see note in 5.1.1.3),

F_{sc} is the design strength of an individual sheet/shear connector fastener. Values are given in table 5.1.

5.1.1.3 Shear connector fastener capacity (at internal rafters)

$$P_{ult} = n'_{sc} F_{sc}$$

where P_{ult} is the ultimate load at a shear panel point,

n'_{sc} is the number of sheet/shear connector fasteners per internal rafter,

F_{sc} is the design strength of an individual sheet/shear connector fastener. Values are given in table 5.1,

n is the number of shear panels in the length of the diaphragm assembly.

Note: With reference to figure R.5.1, the force in the end rafters is $\frac{1}{2}(n-1)$ times the force in the internal rafters, so the corresponding numbers of shear connector fasteners should normally be in the same ratio, i.e. $n_{sc} = \frac{1}{2}(n-1)n'_{sc}$.

5.1.1.4 Two sides of shear panels fastened

For sheeting attached to the purlins only and to the end rafters (see figure R.4.3, case 3):

(1) capacity of end fasteners in an internal shear panel

Table 5.2 Factors to allow for the number of sheet/purlin fasteners per sheet width

Total number of fasteners per sheet width n_f	Factor β_1		Factor β_2
	Case 1 - sheeting	Case 2 - decking	
2	0.13	1.0	1.0
3	0.30	1.0	1.0
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

The mathemathical expressions for table 5.2 are given in Annex C 1.

$$P_{ult} = \beta_2 \, n_p \, F_p$$

(2) capacity of purlin/rafter connections

$$P_{ult} = n_p \, F_{pr}$$

where P_{ult} is the ultimate load at a shear panel point,

n_p is the number of purlins (edge + intermediate),

F_p is the design strength of an individual sheet/purlin fastener. Values are given in table 5.1,

F_{pr} is the design strength of a purlin/rafter connection. Values are given in table 5.3,

β_2 is a factor to allow for the number of sheet/purlin fasteners per sheet width. Values are given in table 5.2.

5.1.2 **Design shear capacity**

The design shear capacity V^* may be taken as the least of the values of V_{ult} given in 5.1.1.1 and 5.1.1.2, and the derived values of V_{ult}, given by $V_{ult} = \frac{1}{2} P_{ult}(n-1)$ in 5.1.1.3 and 5.1.1.4, as appropriate to the case considered. It should be checked that the capacity in the other failure modes is greater than V^* as given in 5.1.3.1 to 5.1.3.4.

5.1.3 **Non permissible modes**

5.1.3.1 Sheet/purlin fastener strength

In order to take account of the effect of combined shear and prying action by the sheeting, the capacity of the sheet/purlin fasteners in shear is reduced by 40%. Hence it should be checked that:

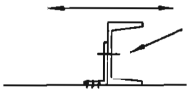
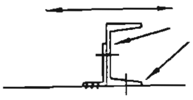
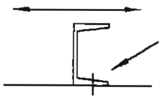
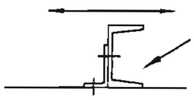
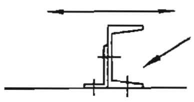
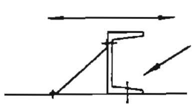
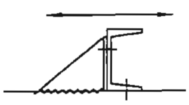
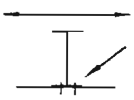
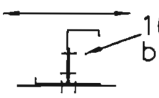
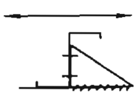
$$\frac{0.6 b F_p}{p \alpha_3} \geq V^*$$

where b is the depth of the shear panel in a direction parallel to the corrugations,

F_p is the design strength of an individual sheet/purlin fastener. Values are given in table 5.1,

p is the pitch of the sheet/purlin fasteners,

Table 5.3 Design strengths and flexibilities of purlin/rafter connections

connection number	type of purlin (and cleat)	connection detail	design strength F_{pr} (kN)	flexibility s_{pr} (mm/kN)
1	102x51 rolled steel channel (89x64x7.8 angle cleat x89mm long)	 two 16 mm bolts	4.9	0.84
2		 toes welded	20.0	0.11
3	152x76 rolled steel channel (76x64x6.2 angle cleat x127mm long)	 two 19 mm bolts	14.4	0.60
4		 flange unbolted	7.2	1.20
5		 flange bolted	19.6	0.35
6		 flange bolted	25.0	0.13
7		 stiffened cleat	25.0	0.05
8	254x102x 22 kg/m Universal Beam	 two 16 mm bolts	10.0	2.60
9	203x51x2.0 zed (178x89x9.4 angle cleat x127mm long)	 16 mm bolts	4.4	1.40
10		 stiffened cleat	7.2	0.38

Note: The strength and flexibility of other types of purlin/rafter connections may be estimated from the values given above or may be obtained by test.

α_3 is a factor to allow for intermediate purlins. Values are given in table 5.4.

5.1.3.2 End collapse of sheeting profile

In order to prevent collapse or gross distortion of the profile at the end of the sheeting, the following limitation on shear force in a shear panel should be observed:

Every corrugation fastened at the end of the sheeting:

$$0.9\ t^{1.5} b\ f_y / d^{0.5} \geq V^*$$

Alternate corrugations fastened at the end of the sheeting:

$$0.3\ t^{1.5} b\ f_y / d^{0.5} \geq V^*$$

where t is the net sheet thickness, excluding metalic and other coatings,

b is the depth of a shear panel in a direction parallel to the corrugations,

f_y is the design yield stress of steel,

d is the pitch of the corrugations.

5.1.3.3 Shear buckling

The shear buckling strength of the sheeting should be checked in accordance with the expressions in 5.4, which include a 25% reserve of safety on global shear buckling.

5.1.3.4 Edge members

The capacity of the edge members and their connections to carry the flange forces arising from diaphragm action should be checked in accordance with 5.5, which includes a 25% reserve of safety.

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 5.4 Factors to allow for the effect of intermediate purlins

The mathematical expressions for table 5.4 are given in Annex C 2.

5.2 *DESIGN EXPRESSIONS FOR FLEXIBILITY: SHEETING SPANNING PERPENDICULAR TO LENGTH OF DIAPHRAGM*

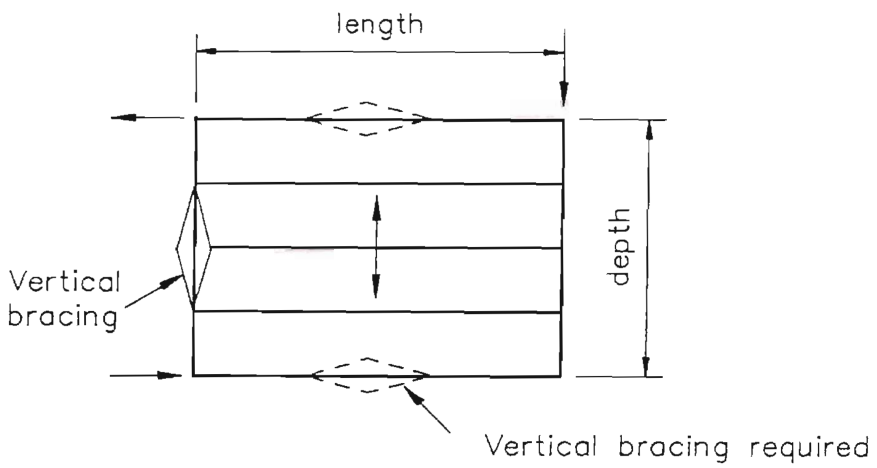


Figure C.5.2: Cantilevered diaphragm: sheeting spanning perpendicular.

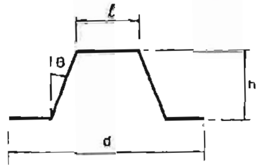
5.2 *DESIGN EXPRESSIONS FOR FLEXIBILITY: SHEETING SPANNING PERPENDICULAR TO LENGTH OF DIAPHRAGM*

5.2.1 **Flexibility of shear panel assemblies**

5.2.1.1 General

The total shear flexibility of a shear panel is the sum of the component shear flexibilities listed in 4.4.2. For shear panel assemblies (see figure R.5.1) the design expressions are given in table 5.5 (column 1). For a cantilevered diaphragm (figure C.5.2), the design expressions are given in table 5.5 (column 2). Notes on the design expressions are given in 5.2.1.2 to 5.2.1.8.

Table 5.6 Values for K_1 for fasteners in every trough



θ	ℓ/d --> / h/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	0.806	0.772	0.877	1.306	2.586	
	0.7	0.591	0.935	1.066	1.028	0.983	1.135	1.756	3.605	
	0.8	0.780	1.191	1.328	1.264	1.208	1.423	2.299	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	0.226	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	0.039	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					
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	0.155	
	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								

Comments

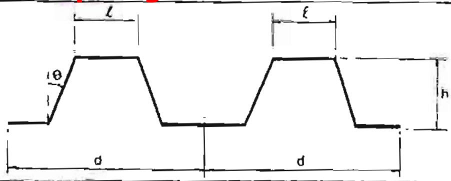
C.30

Table 5.5 Components of shear flexibility: sheeting spanning perpendicular to length of diaphragm

		(1) panel assemblies (see figure R.5.1)	(2) cantilevered diaphr. (see figure C.5.2)
Shear flexibility due to:		shear flexibility mm/kN	shear flexibility mm/kN
sheet deformation	profile distortion	$c_{1.1} = \dfrac{ad^{2.5}\alpha_1\alpha_4 K}{Et^{2.5}b^2}$ 1)	$c_{1.1} = \dfrac{ad^{2.5}\alpha_1\alpha_4 K}{Et^{2.5}b^2}$
	shear strain	$c_{1.2} = \dfrac{2a\alpha_2(1+\nu)[1+(2h/d)]}{Etb}$	$c_{1.2} = \dfrac{2a(1+\nu)[1+(2h/d)]}{Etb}$
fastener deformation	sheet to purlin fastener	$c_{2.1} = \dfrac{2as_p p\alpha_3}{b^2}$	$c_{2.1} = \dfrac{2as_p p}{b^2}$
	seam fasteners	$c_{2.2} = \dfrac{2s_s s_p(n_{sh}-1)}{2n_s s_p + \beta_1 n_p s_s}$	$c_{2.2} = \dfrac{2s_s s_p(n_{sh}-1)}{2n_s s_p + \beta_1 n_p s_s}$
	connections to rafters	4 sides fastened $c_{2.3} = \dfrac{4(n+1)s_{sc}}{n^2 n'_{sc}}$ or 2 sides only fastened with gable shear connect. $c_{2.3} = \dfrac{4(n-1)}{n^2 n_p}\left(s_{pr} + \dfrac{s_p}{\beta_2}\right)$	4 sides fastened $c_{2.3} = \dfrac{2s_{sc}}{n_{sc}}$ or 2 sides only fastened $c_{2.3} = \dfrac{2}{n_p}\left(s_{pr} + \dfrac{s_p}{\beta_2}\right)$
total flexibility in true shear		$c' = (c_{1.1}+c_{1.2}+c_{2.1}+c_{2.2}+c_{2.3})$	
flange forces	axial strain in purlins	$c_3 = \dfrac{n^2 a^3 \alpha_3}{4.8EAb^2}$	$c_3 = \dfrac{2a^3}{3EAb^2}$
Total shear flexibility		$c = c' + c_3$	$c = c' + c_3$

1) The expression for $c_{1.1}$ applies for $b/d \geq 10$.

Table 5.7 Values for K_2 for fasteners in alternate troughs



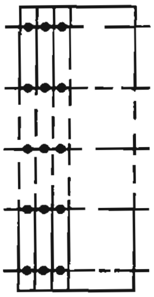
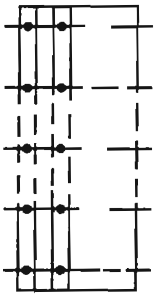
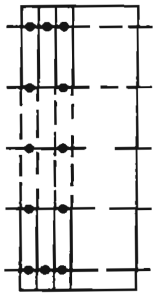
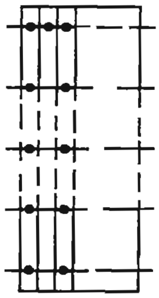
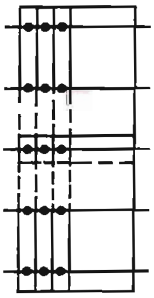
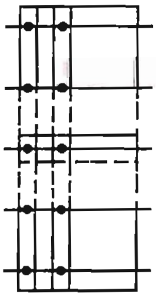
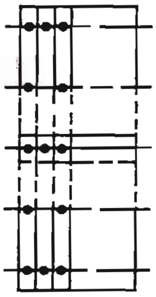
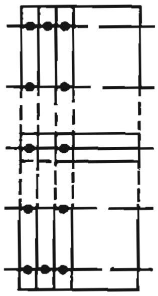
θ	ℓ/d --> h/d ↓	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0°	0.1	0.014	0.025	0.036	0.046	0.054	0.061	0.070	0.108	0.211
	0.2	0.031	0.065	0.099	0.129	0.151	0.169	0.206	0.318	0.649
	0.3	0.054	0.123	0.192	0.252	0.294	0.328	0.402	0.608	1.269
	0.4	0.084	0.202	0.316	0.414	0.482	0.535	0.653	0.968	2.056
	0.5	0.123	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	3.089	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	4.194	4.752	5.588				
	0.7	4.042	4.789	5.494	6.272					
	0.8	5.122	6.013	6.883	7.861					
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	3.199	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								

Comments

In table 5.5

- a is the width of a shear panel in a direction perpendicular to the corrugations,
- A is the cross-sectional area of a longitudinal edge member,
- b is the depth of a shear panel in a direction parallel to the corrugations,
- c is the total shear flexibility of a shear panel,
- $c_{1.1}$ etc. are the component shear flexibilities,
- d is the pitch of the corrugations,
- E is the modulus of elasticity of steel,
- h is the height of the sheeting profile,
- K is a sheeting constant which can take values K_1 or K_2 as given in tables 5.6 and 5.7. See clause 5.2.1.2,
- n is the number of shear panels in the length of the diaphragm assembly,
- n_p is the number of purlins (edge + intermediate),
- n_s is the number of seam fasteners per side lap (excluding those which pass through both sheets and the supporting purlin),
- n_{sc} is the number of sheet/shear connector fasteners per end rafter,
- n'_{sc} is the number of sheet/shear connector fasteners per internal rafter,
- n_{sh} is the number of sheet widths per shear panel,
- p is the pitch of the sheet/purlin fasteners,
- s_p is the slip per sheet/purlin fastener per unit load. Values are given in table 5.1,
- s_{pr} is the deflection of top of purlin at purlin/rafter connection per unit load. Typical values are given in table 5.3,
- s_s is the slip per seam fastener per unit load. Values are given in table 5.1,
- s_{sc} is the slip per sheet/shear connector fastener per unit load. Values are given in table 5.1,
- t is the net sheet thickness, excluding metalic and other coatings,

Table 5.8 Factor α_4 to allow for the number of sheet lengths in the depth of a diaphragm

	fastener positions			
	every corrugation	alternate corrugations	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 5.4 $\alpha_4 = 1$ (1)	 $K = K_2$ α_1 from table 5.4 $\alpha_4 = 1$ (2)	 $K = K_1$ $\alpha_1 = 1$ $\alpha_4 = 1$ (3)	 $K = K_2$ $\alpha_1 = 0.5$ $\alpha_4 = 1$ (4)
n_b sheet lengths in depth of diaphragm	 $K = K_1$ α_1 from table 5.4 for number of purlins per sheet length $\alpha_4 = (1+0.3n_b)$ (5)	 $K = K_2$ α_1 from table 5.4 for number of purlins per sheet length $\alpha_4 = (1+0.3n_b)$ (6)	 $K = K_1$ $\alpha_1 = 1$ $\alpha_4 = (1+0.3n_b)$ (7)	 $K = K_2$ α_1 from table 5.4 for number of purlins per sheet length $\alpha_4 = (1+0.3n_b)(1-\frac{1}{n_b})$ (8)

α_1 to α_4 are factors to allow for intermediate purlins and number of sheet lengths. Values are given in tables 5.4 and 5.8,

β_1, β_2 are factors to allow for the number of sheet/purlin fasteners per sheet width. Values are given in table 5.2,

ν is Poisson's ratio (for steel 0.3).

5.2.1.2 Profile distortion

In the expression for $c_{1.1}$, K can take values K_1 or K_2 given in tables 5.6 and 5.7 depending on whether the sheeting is fastened in every corrugation or alternate corrugations. The factor α_1 takes account of the effect of fasteners to intermediate purlins and is given in table 5.4. The factor α_4 takes account of the effect of the number of sheet lengths in the depth of the diaphragm and is given in table 5.8 for various fastener arrangements.

The expression for $c_{1.1}$ applies for $b/d \geq 10$.

5.2.1.3 Shear strain in the sheet

In the expression for $c_{1.2}$ if there are intermediate purlins present, the shear across the depth of the shear panel is not uniform. The factor α_2 takes account of this effect and is given in table 5.4.

5.2.1.4 Slip in the sheet/purlin fasteners

The flexibility due to this cause, $c_{2.1}$, depends on the slip value and the spacing of fasteners. The factor α_3, given in table 5.4, allows for the effect of fasteners at intermediate purlins.

5.2.1.5 Slip in the seam fasteners

In the case of roof sheeting and side cladding, the seams are usually fastened in the crests by seam fasteners only.

In the case of roof decking, the seams are usually fastened in the troughs by seam fasteners and sheet/purlin fasteners which pass through both sheet thicknesses at the overlaps. The expressions for $c_{2.2}$ takes account of this difference by means of the values of β_1 (table 5.2) and by allowing for the relative values of slip at the seam and the adjacent sheet/purlin fasteners.

5.2.1.6 Slip in the sheet/shear connector fasteners

If four sides of the shear panel are fastened and if $n_{sc} = ½(n - 1)\ n'_{sc}$ (see 5.1.1.3) the shear flexibility $c_{2.3}$ due to this cause is the same at the end shear panels and internal shear panels.

5.2.1.7 Movement in the purlin/rafter connections

For the case of a shear panel with only two sides fastened (plus sheet/shear connector fasteners at the end gable), the expression given for $c_{2.3}$ ignores the small movement at the end rafters in comparison with the movement of the purlin/rafter connections at the internal rafters.

5.2.1.8 Axial strain in the edge members

The flexibility due to this cause is strictly a bending effect but for convenience it is replaced by an equivalent shear flexibility. The expression given for c_3 is an average value over the length of the shear panel assembly. The factor α_3, given in table 5.4, allows for the effect of intermediate purlins.

5.2.2 Deflection

The sum of the component shear flexibilities 5.2.1.2 to 5.2.1.8 gives the total shear flexibility c of the shear panel. The mid-length deflection of the typical shear panel assembly, shown in figure R.5.1, is given by:

$$\Delta = P(n^2/8)c$$

where P is the shear panel point load on the diaphragm,

n is the number of shear panels in the length of the diaphragm assembly,

c is the total shear flexibility of a shear panel as given in table 5.5.

5.3 *FASTENER CHARACTERISTICS*

5.3.1 Edge and end distances

To ensure that the full tearing strength of the sheeting is developed, the edge and end distances (measured from the centre of the hole) should not be less than the following if the fasteners are to be included in the design calculations:

5.3.2 **Fastener strength and slip**

In table 5.1 the typical design strengths and slip values given are based on the following assumptions:

(1) The net sheet thickness is between 0.50 and 1.20 mm.

(2) The specified yield strength and ultimate tensile strength of the steel sheet do not exceed 355 and 480 N/mm² respectively.

In table 5.1, the design strengths of fasteners are obtained from the bearing resistances given in tables 8.1 to 8.3 of Eurocode 3, Part 1.3, using a partial safety factor of 1.1 instead of 1.25 (due to the large number of fasteners in a panel). For sheet/purlin and sheet/shear connector fasteners it is assumed that the base material ≥ 2.5 x sheet thickness, and for seam fasteners it is assumed that the sheet thicknesses are equal. Alternatively, the full calculation procedure may be used.

In table 5.1, the slip values are based on test results and are approximate.

(a)	edge distance of seam fasteners and shear connector fasteners	1.5 x diameter or 8mm,
(b)	edge distance of sheet/purlin fasteners	1.5 x diameter or 10mm,
(c)	end distance of sheet/purlin fasteners	3 x diameter or 20mm.

5.3.2 Fastener strength and slip

The design strengths and slip values given in table 5.1 apply to the range of fasteners, number of fasteners, sheet thicknesses and material strengths typically found in stressed skin panels. For other conditions, lap joint tests should be made in accordance with chapter 11 to determine the characteristic strengths and slip values and to ensure that failure occurs by tearing of the sheeting. It is essential that the absolute limits on design strengths given in table 5.1 are not exceeded.

5.4 *SHEAR BUCKLING*

Shear stresses may cause (1) local buckling of wide flanges and webs of the trapezoidal sheeting and (2) global buckling of the diaphragm as a whole.

The influence of local buckling on the load bearing capacity of the sheeting under transverse loading as well as on the diaphragm flexibility is normally negligible.

The interaction of global and local shear buckling can be neglected if:

$$\ell/t \leq 2.9\ (E/f_y)^{0.5}$$

If $\ell/t > 2.9\ (E/f_y)^{0.5}$ the interaction has to be taken into account by a reduction of the global buckling strength as follows:

$$V_{red} = \frac{V_g V_\ell}{(V_g + V_\ell)} > V^*$$

In the above expressions:

ℓ is the width of the top or bottom flange of the sheeting, whichever is wider (figure R.5.2),

t is the net sheet thickness, excluding metalic and other coatings,

E is the modulus of elasticity of steel,

f_y is the design yield strength of steel,

V_g is the design value of the global shear buckling strength of the diaphragm,

V_ℓ is the design value of the local shear buckling strength of the diaphragm,

V^* is the design shear capacity of the diaphragm,

V_{red} is the design value of the reduced shear buckling strength of the diaphragm under combined local and global buckling.

5.4.1 Global shear buckling

The design value of the global shear buckling strength of the sheeting may be calculated from:

$$V_g = \frac{14.4}{b} D_x^{1/4} D_y^{3/4} (n_p - 1)^2$$

where D_x and D_y are the orthogonal bending stiffnesses given by:

$$D_x = \frac{Et^3 d}{12(1-\nu^2)u}$$

$$D_y = \frac{EI}{d}$$

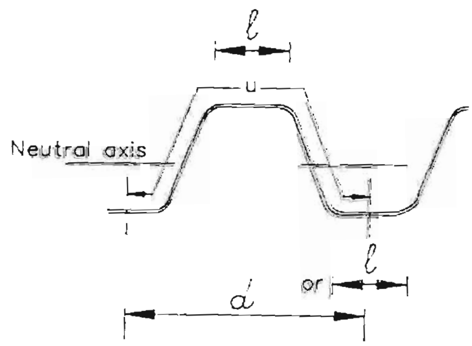


Figure R.5.2: Single corrugation.

where E is the modulus of elasticity of steel,

t is the net sheet thickness, excluding metalic and other coatings,

d is the pitch of the corrugations,

ν is Poisson's ratio for steel (0.3),

u is the perimeter length of a complete single corrugation of pitch d (figure R.5.2),

I is the second moment of area of a single corrugation about its neutral axis (figure R.5.2),

b is the depth of a shear panel in a direction parallel to the corrugations,

n_p is the number of purlins (edge + intermediate).

5.4.2 **Local shear buckling**

Certain sheeting profiles have a relatively wide trough which may or may not be stiffened as shown in figure R.5.3. In such cases, shear failure of the diaphragm may be initiated by local shear buckling of the trough element.

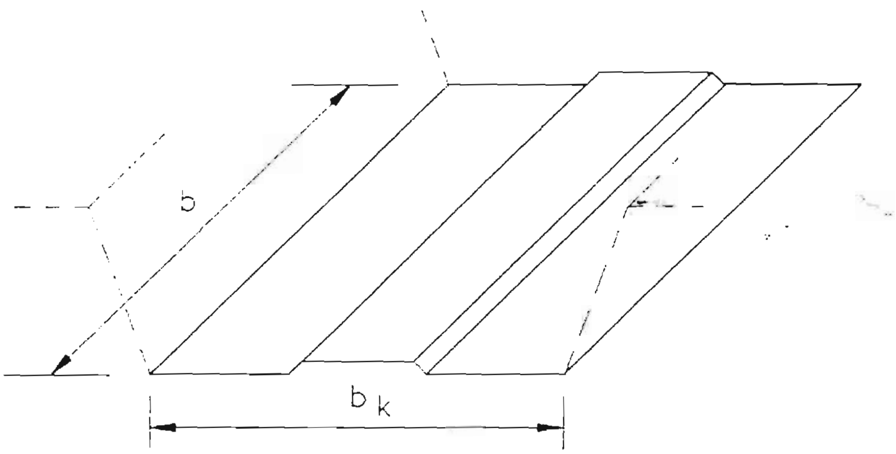


Figure R.5.3: Trough element which may be subject to local shear buckling.

If the trough element is unstiffened, the design value of the local shear buckling strength may be calculated from:

$$V_\ell = 4.83\ E\ (t/\ell)^2\ bt$$

where the terms are as given in 5.4 and 5.4.1.

If the trough element is stiffened, the design value of the local shear buckling strength may conservatively taken as:

$$V_\ell = \frac{36b}{b_k^2}\ D_x^{1/4}\ D_y^{3/4}$$

where, in addition to the terms defined above:

b_k is the width of the trough element,

D_x is EI_f/b_k,

D_y is $Et^3/10.92$,

I_f is the second moment of area of the trough element about a horizontal axis.

5.5 EDGE MEMBERS

The edge members and their connections should be designed in accordance with Eurocode No. 3 (Part 1 or Part 1.3) to carry their designated loads together with 1.25 x the axial load from diaphragm action as given in 5.1.3.4. The maximum axial load, with reference to figure R.5.1, may be taken as $\frac{qL^2\alpha_3}{8b}$

where q is the distributed in-plane line load carried by the diaphragm,

L is the length of the diaphragm assembly between braced frames,

α_3 is a factor to allow for intermediate purlins. Values are given in table 5.4,

b is the depth of a shear panel in a direction parallel to the corrugations.

5.6 COMBINED LOADS

5.6.1 Effect on sheeting

It may be assumed that (1) deterioration of the sheeting would become apparent for other reasons before it could prejudice the behaviour of the diaphragm and that (2) it should not normally be necessary to take diaphragm action into account in the design of the sheeting.

It may be assumed that the effect of normal loads, both downward (i.e. dead and imposed load) and upward (i.e. wind suction load) should not be taken into account in calculating the diaphragm shear strength or flexibility.

5.6.2 **Effect on fasteners**

The only fasteners normally subjected to combined stresses under combined shear and wind suction are the sheet/purlin fasteners.

The coefficient 0.6 applied to F_p takes account of prying action on the fastener (5.1.3.1). No other measure is necessary.

The value of F_t to be taken is the least of (1) pull-over of the sheet over the head of the fastener, (2) pull-out of the fastener from the supporting member and (3) direct tensile strength of the fastener itself.

5.6.2 Effect on fasteners

When fasteners are subjected to combined shear and prying action by diaphragm action in the sheeting, and to direct tensile load by transverse load on the sheeting (e.g. wind suction), the fasteners should satisfy the following expression:

$$\left(\frac{F^1_t}{F_t}\right)^2 + \left(\frac{F^1_p}{0.6F_p}\right)^2 \leq 1$$

where F^1_t is the design tensile load in a fastener,

F_t is the design tensile strength of a fastener (see comment),

F^1_p is the design shear load on a fastener,

F_p is the design shear strength of a fastener.

5.7 *DIAPHRAGM ORIENTATION*

5.7.1 Modified shear strength and flexibility

The values of shear strength and flexibility given in sections 5.1 and 5.2 are calculated in a direction parallel to the corrugations (figure R.5.4a), i.e. V_o and v_o/V_o. The shear strength and flexibility in a direction perpendicular to the corrugations (figure R.5.4b)are given by following expressions:

Shear strength $V = V_o\ a/b$

where V_o is the shear strength as calculated in 5.1.1.

Shear flexibility $c = v/V = v_o/V_o\ (b/a)^2$

where v_o/V_o is the shear flexibility as calculated in 5.2.1.

Note: The modified shear flexibility is applied only to components (a) to (f) in 4.4.2.

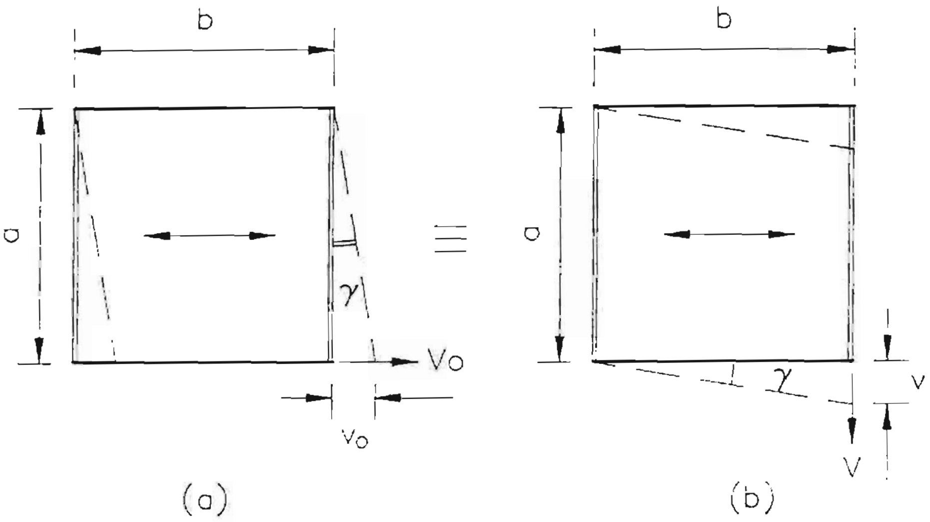


Figure R.5.4: Shear strength and flexibility in a perpendicular direction.

In the before mentioned expressions:

V_o is the shear force on the diaphragm parallel to the corrugations,

v_o is the shear displacement of the diaphragm parallel to the corrugations,

V is the shear force on the diaphragm perpendicular to the corrugations,

v is the shear displacement of the diaphragm perpendicular to the corrugations,

a is the width of the shear panel in a direction perpendicular to the corrugations,

b is the depth of the shear panel in a direction parallel to the corrugations.

5.8 *DESIGN EXPRESSIONS FOR STRENGTH: SHEETING SPANNING PARALLEL TO LENGTH OF DIAPHRAGM*

5.8.1 Strength of shear panel assemblies

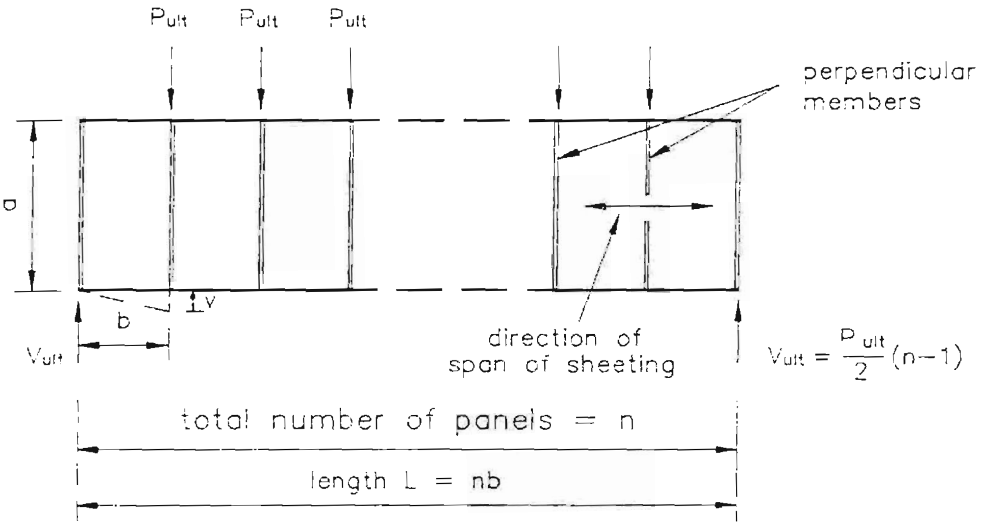


Figure R.5.5: Shear panel assembly: sheeting spanning parallel to length of diaphragm.

With reference to figure R.5.5, the shear strength of the shear panel assembly should normally be checked by considering failure modes in the end shear panels and at the internal rafters. The ultimate strengths V_{ult} associated with these failure modes may be obtained as follows:

5.8.1.1 Seam strength

$$V_{ult} = \frac{a}{b}\left(n_s F_s + \frac{\beta_1 F_p}{\beta_3}\right)$$

The symbols are defined in 5.1.1.

5.8.1.2 Edge member fastener strength

For the sheeting attached to the rafters and edge members:

$$V_{ult} = \frac{a}{b}(n_{sc} F_{sc})$$

The symbols are defined in 5.1.1.

5.8.1.3 Two sides of shear panel fastened

Although not normally recommended (see figure R.4.3 case 4) this case applies to the sheeting attached to the rafters only (not to the edge members). At the end rafter:

$$V_{ult} = \frac{a}{b}(1.5\ \beta_2 F_p)$$

Note: If the value of V_{ult} given by this expression is not sufficient, shear connectors may be added to the edge members in the end shear panels. In this case the design criterion for the end shear panel is given in 5.8.1.2, and the design criterion for an internal panel is:

$$P_{ult} = \frac{a}{b}(1.5\ \beta_2 F_p)$$

The symbols are defined in 5.1.1.

5.8.2 **Design shear capacity**

The design shear capacity V^* may be taken as the lesser of the values of V_{ult} given in 5.8.1.1 and 5.8.1.2 (or 5.8.1.3). It should be checked that the capacity in other failure modes is greater than V^* as given in 5.8.3.1 to 5.8.3.4.

5.8.3 **Non permissible modes**

5.8.3.1 Sheet/rafter fastener strength

It should be checked that:

$$\frac{0.6aF_p}{p} \geq V^*$$

where a is the width of the shear panel in a direction perpendicular to the corrugations and the other symbols are as defined in 5.1.3.

5.8.3.2 End collapse of sheeting profile

The following limitation on shear force in a shear panel should be observed:

Every corrugation fastened at the end of the sheeting:

$$0.9t^{1.5}af_y/d^{0.5} \geq V^*$$

Alternate corrugations fastened at the end of the sheeting:

$$0.3t^{1.5}af_y/d^{0.5} \geq V^*$$

The symbols are defined in 5.1.3.

5.8.3.3 Shear buckling

It should be checked that the design value of the reduced shear buckling strength of the sheeting, V_{red}, is adequate according to 5.4. The design value of the local shear buckling strength, V_ℓ, is as given in 5.4.2 and the design value of the global shear buckling strength may be taken as:

For fasteners in every corrugation:

$$V_g = \frac{28.8a}{b^2} D_x^{1/4} D_y^{3/4}$$

For fasteners in alternate corrugations:

$$V_g = \frac{14.4a}{b^2} D_x^{1/4} D_y^{3/4}$$

The symbols in these expressions are defined in 5.4.

5.8.3.4 Edge members

The edge members and their connections should be designed in accordance with Eurocode No. 3 (Part 1 or Part 1.3) to carry their designated loads together with 1.25 x the axial load from diaphragm action. The maximum axial load may be taken as $\frac{qL^2}{8a}$ where the symbols are as defined in 5.5 and figure R.5.4.

5.8.4 Other effects

The clauses on combined loads given in 5.6.1 apply also to sheeting spanning parallel to the length of the diaphragm. The provisions of clause 5.6.2 ensure that the sheet/rafter fasteners are adequate under combined stresses due to shear prying action and wind uplift.

5.9 *DESIGN EXPRESSIONS FOR FLEXIBILITY: SHEETING SPANNING PARALLEL TO LENGTH OF DIAPHRAGM*

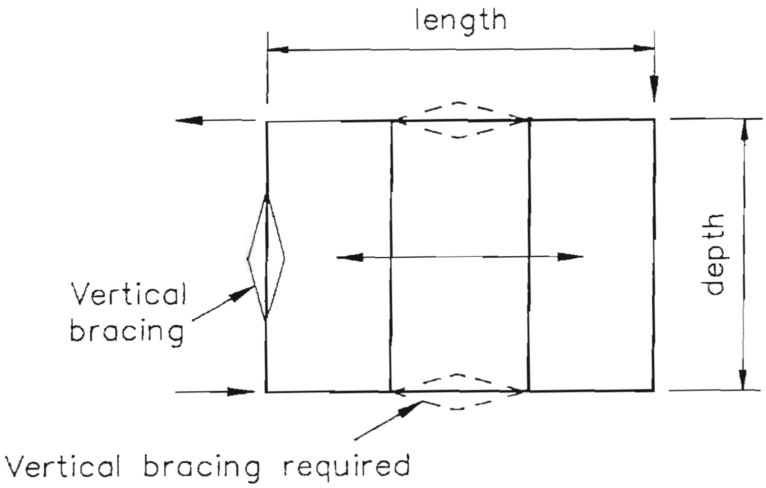


Figure C.5.3: Cantilevered diaphragm: sheeting spanning parallel..

5.9 *DESIGN EXPRESSIONS FOR FLEXIBILITY: SHEETING SPANNING PARALLEL TO LENGTH OF DIAPHRAGM*

5.9.1 **Flexibility of shear panel assemblies**

5.9.1.1 General

The total shear flexibility of a shear panel is the sum of the component shear flexibilities listed in 4.4.2. For shear panel assemblies (see figure R.5.4) the design expressions are given in table 5.9, column (1). For a cantilevered diaphragm (see figure C.5.3) the design expressions are given in table 5.9, column (2). Notes on the design expressions are given in 5.9.1.2 and 5.9.1.3.

Table 5.9 Components of shear flexibility: sheeting spanning parallel to length of diaphragm

		(1) panel assemblies (see figure R.5.4)	(2) cantilevered diaphr. (see figure C.5.3)
Shear flexibility due to:		shear flexibility mm/kN	shear flexibility mm/kN
sheet deformation	profile distortion	$c_{1.1} = \dfrac{a d^{2.5} \alpha_5 K}{E t^{2.5} b^2}$ 1)	$c_{1.1} = \dfrac{a d^{2.5} \alpha_1 \alpha_4 K}{E t^{2.5} b^2}$
	shear strain	$c_{1.2} = \dfrac{2a(1+\nu)[1+(2h/d)]}{Etb}$	$c_{1.2} = \dfrac{2a(1+\nu)[1+(2h/d)]}{Etb}$
fastener deformation	sheet to perpendicular member fastener	$c_{2.1} = \dfrac{2 a s_p p}{b^2}$	$c_{2.1} = \dfrac{2 a s_p p}{b^2}$
	seam fasteners	$c_{2.2} = \dfrac{s_s s_p (n_{sh}-1)}{n_s s_p + \beta_1 s_s}$	$c_{2.2} = \dfrac{2 s_s s_p (n_{sh}-1)}{2 n_s s_p + \beta_1 n_p s_s}$
	connections to edge members	4 sides fastened $c_{2.3} = \dfrac{2 s_{sc}}{n_{sc}}$ or 2 sides only fastened $c_{2.3} = s_{pr} + \dfrac{s_p}{\beta_2}$	4 sides fastened $c_{2.3} = \dfrac{2 s_{sc}}{n_{sc}}$ or 2 sides only fastened $c_{2.3} = \dfrac{2}{n_p}\left(s_{pr} + \dfrac{s_p}{\beta_2}\right)$
Total flexibility in true shear		$c' = \dfrac{b^2}{a^2}(c_{1.1} + c_{1.2} + c_{2.1} + c_{2.2} + c_{2.3})$	
Flange forces	axial strain in edge members	$c_3 = \dfrac{n^2 b^3}{4.8 E A a^2}$	$c_3 = \dfrac{2 b^3}{3 E A a^2}$
Total shear flexibility		$c = c' + c_3$	$c = c' + c_3$

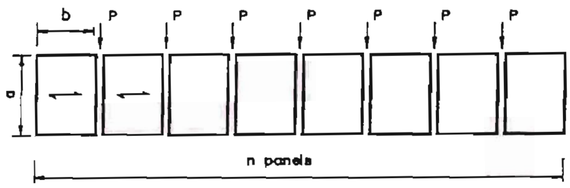
The expression for $c_{1.1}$ applies for $b/d \geq 10$.

Note: here is s_p the slip in the fastening between sheeting and rafter.

Table 5.10 Influence of sheet length for sheeting spanning parallel to length of diaphragm

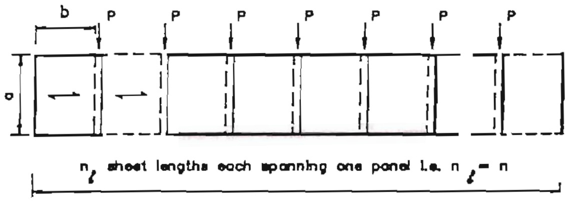
Design case and effect on expression for $c_{1.1}$

Separate sheet length per panel (1)



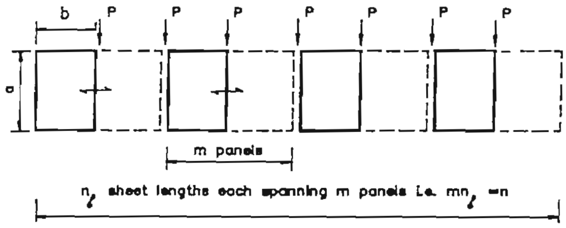
$\alpha_5 = 1$

Overlapping separate sheets (2)



α_5 is given by table 5.11

Overlapping continuous sheets (3)



α_5 is given by values in table 5.11 divided by m

Table 5.11 Factor α_5 to allow for sheet continuity

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

In table 5.9, the symbols are as defined in 5.2.1 with the addition of the following: α_5 is a factor to allow for sheet continuity. See 5.9.1.2.

5.9.1.2 Profile distortion

In the expression for $c_{1.1}$, K can take values K_1 or K_2 given in tables 5.6 and 5.7 depending on whether the sheeting is fastened in every corrugation or alternate corrugations. The factor α_5 takes account of the effect of the sheeting having continuity over two or more spans as illustrated in table 5.10 and given numerically in table 5.11.

In tables 5.10 and 5.11:

- m is the number of shear panels within a sheet length,
- n is the number of shear panels in the length of the diaphragm assembly,
- n_ℓ is the number of sheet lengths in the length of the diaphragm assembly (i.e. $mn_\ell = n$),
- α_5 is a factor to allow for sheet continuity.

The expression for $c_{1.1}$ applies for $b/d \geq 10$.

5.9.1.3 Other data

Values of sheeting, fastener and connection characteristics, and multiplying factors are the same as for sheeting perpendicular to the span and are given in tables 5.1, 5.2, 5.3, 5.6 and 5.7.

5.9.2 **Deflection**

The mid-length deflection of the typical shear panel assembly, shown in figure R.5.4, is given by:

$$\Delta = P(n^2/8)c$$

where
- P is the shear panel point load on the diaphragm,
- n is the number of shear panels in the length of the diaphragm assembly,
- c is the total shear flexibility of a shear panel, as given in table 5.9.

6. DESIGN EXPRESSIONS FOR OTHER PROFILES AND MATERIALS

6.1 *GENERAL*

In principle all types of panels, adequately fastened to the substructure to form an element with certain bending and shear stiffness can be used as a shear diaphragm, provided the quality of the components involved is expected to be unchanged during the expected life time. For certain types of elements, like trapezoidal sheeting and special cassettes, analytical methods can be used to design a diaphragm. In other cases a design by testing is required.

6.2 *SPECIAL REQUIREMENTS*

6.2.1 **Trapezoidal sheeting of aluminium**

It should be taken into account that, due to the lower modulus of elasticity, the flexibility of the diaphragm is increased. In the case of shallow trapezoidal sheeting, the risk of global buckling is more significant than with steel sheeting (Ref. [18]).

6.2.1.1 Guidance values for design shear capacity and flexibility of fasteners are given in the table below (for symbols, which are different from clause 1.4, see page C.47).

Materials in the connection	Type of fasteners; thickness restrictions	Characteristic strength of the connection kN	Flexibility ($c_{2.1}$ - $c_{2.3}$) mm/kN
Aluminium (I) Aluminium (II)	blind rivets d = 4.8 mm	$t_I = t_{II}$ $Q_{Rc} = 1.6 R_m \sqrt{t_I^3 \cdot d}$ $\leq 1.6 R_m \cdot t_I \cdot d$ $t_{II}/t_I \geq 2.5$ $Q_{Rc} = 1.6 R_m \cdot t_I \cdot d$	0.25
Aluminium (I) Steel (II)	self drilling self tapping screw ≥ 5.5 mm	$1.0 < t_{II}/t_I < 2.5$ linear inter-polation	0.4

Note: Contrary to table 5.1 the above table provides *characteristic* strengths and these values are based on a maximum hole elongation of 3 mm.

6. DESIGN EXPRESSIONS FOR OTHER PROFILES AND MATERIALS

6.1 *GENERAL*

Stressed skin design for shear diaphragms other than trapezoidal steel sheeting, may be used provided the functional behaviour of the diaphragm, characterized by its stiffness and load carrying capacity, is proved by tests or a rational analysis.

In the case of non-metallic sheeting, the reliability of the diaphragm to perform its function throughout the expected life-time should be verified.

6.2 *SPECIAL REQUIREMENTS*

6.2.1 **Trapezoidal sheeting of aluminium**

The design principles according to chapter 4 are valid, taking into account the appropriate physical properties of the material.

Regarding the design expressions (chapter 5) however the following variations should be taken into account:

6.2.1.1 Fastener capacities and slip values should be adjusted with respect to thickness and material quality of the sheeting respectively.

6.2.1.2 K-values, characterizing the influence of the profile distortion on the shear flexibility ($c_{1.1}$) should be reduced to 50% of the values given in tables 5.6 and 5.7.

t_I = thickness of the aluminium sheeting,

t_{II} = thickness of the substructure,

R_m = smallest minimum tensile strength of both components I or II,

Q_{Rc} = characteristic shear strength of the connection.

The design strength of the connection is $Q_{Rd} = Q_{Rc}/\gamma_{M2}$ with $\gamma_{M2} = 1.1$

6.2.2 **Cassettes**

The use of the calculation procedure for cassettes as a shear diaphragm is conditional upon a stiffened wide flange and web-connections at a maximum distance of 300 mm.

Failure modes are:

either: local buckling of the wide flange in shear,

or: tearing of fasteners in the webs,

and/or: tearing of fasteners at the end support, where at least 3 fasteners should be arranged.

6.2.2 Cassettes

Cassettes, complying with shapes according to fig. R.6.1 and geometrical properties according to table 6.1 may be calculated as shear diaphragms if the following conditions are met:

Table 6.1: Range of validity

0.75	≤	t_N [mm]	≤	1.5
30	≤	B_O [mm]	≤	60
60	≤	H [mm]	≤	200
300	≤	B_u [mm]	≤	600
I_1	≥	10 mm4/mm		
a_1	≤	1000 mm		
e_S	≤	300 mm		
e_u	≤	30 mm		

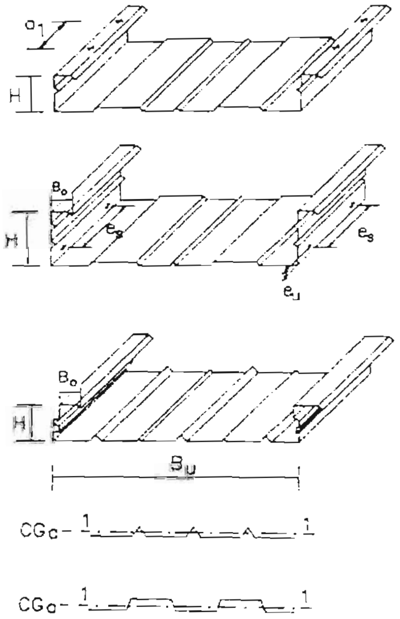


Figure R.6.1 Typical geometry of cassettes.

The fasteners (normally blind rivets) should be located in the webs close to the wide flanges. For an accurate evaluation of flexibility due to fasteners the normal procedure for trapezoidal sheeting should be used.

6.2.2.1 Strength of shear panel assemblies composed with cassettes

The design shear capacity at ultimate limit state of a diaphragm composed of cassettes will be determined by the fasteners in the webs (P_{max}) or shear buckling (V_{buc}). The requirement is that the shear force in the diaphragm caused by the design value of the load should be smaller than P_{max} and V_{buc}.

$$P_{max} = n_s \cdot F_s + \beta_1 \cdot F_p$$

$$V_{buc} = \frac{8.43 \cdot E}{B_u^2} \sqrt[4]{I_1 \cdot t^9} \cdot L$$

where P_{max} is the design shear capacity of a diaphragm when fastening strength is determining,

n_s is the number of seam fasteners (fasteners in the webs) per seam,

F_s is the design strength of an individual seam fastener,

β_1 is a factor to allow for the number of sheet/column fasteners per cassette. Values are given in table 5.2 for case 1 and n_f are the number of sheet/column fasteners per cassette,

F_p is the design strength of an individual cassette/column fastener,

V_{buc} is the design shear capacity of a diaphragm when shear buckling of the wide flange is determining,

I_1 is the moment of inertia of the wide flange in mm⁴ per mm width of the diaphragm,

B_u is the width of the wide flange,

t is the core thickness of the cassette material,

E is the modulus of elasticity,

L is the length of the diaphragm in the span direction of the cassettes.

6.2.2.2 Stiffness of shear panel assemblies composed of cassettes

The stiffness of shear panel assemblies composed of cassettes should be determined from:

$$S_{act} = S'_{act} \cdot L$$

where S_{act} is the shear stiffness of the diaphragm,

S'_{act} is the shear stiffness in force per unit length in N/mm, which (in the absence of test results) can be derived from

Slip of the fasteners is included in the value of the stiffness factor *a*.

6.2.3 **Standing seam profiles**

Such profiles can normally not be used as a shear diaphragm, if the sliding resistance at the connections to the substructure is small.

6.2.4 **Sandwich panels**

Tests have shown that stiffness and ultimate shear load depend on the fastenings, especially with respect to the inner skin, which is directly connected to the substructure and where the sheet thickness and the number of fasteners is important.

$$S'_{act} = \frac{a \cdot L \cdot B_u}{e_s (B - B_u)}$$

L is the length of the diaphragm in the span direction of the cassettes,

B is the width of the diaphragm ($= \Sigma B_u$),

e_s is the spacing of the seam fasteners, with $e_s \leq 300$ mm,

a $= 2 \cdot 10^3$ [N/mm] (= stiffness factor derived from tests).

6.2.2.3 Serviceability state requirement for shear panel assemblies composed of cassettes

The serviceability state requirement for shear panel assemblies composed of cassettes is:

$$\frac{S'_{act}}{375} \geq \frac{\text{actual } T_v}{L}$$

where S'_{act} is the shear stiffness in force per unit length in N/mm, which can be derived according to 6.2.2.2,

L is the length of the diaphragm in the span direction of the cassettes,

actual T_v is the shear force in the diaphragm caused by the load in the serviceability limit state.

6.2.3 Standing seam profiles

Testing is required for the use of standing seam profiles as a shear diaphragm.

6.2.4 Sandwich panels

Testing is required for the use of sandwich panels under in-plane loading. It is essential that the test setup is equivalent to the real structure with respect to the connections between the elements and between sandwich panels and the substructure.

Normally diaphragm action of sandwich panels is governed by the flexibility of the connections between the inner skin and the substructure provided that no other type of failure than tearing of the inner skin occurs.

Increased stiffness can be developed by special edge provisions and fasteners, which connect the inner and outer skins. In such cases, shear buckling as a failure mode may occur.

6.2.5 Two skin roof

1. Testing has demonstrated that the outer skin can provide a significant increase in strength even when the flexibility of the spacing system results in little additional stiffness in the elastic range of loading.

2. Conventional liner tray systems with a thin inner skin generally perform poorly with regard to in-plane shear because the early shear buckling of elements with large *b/t* ratios results in a relatively low strength. If it is required to take advantage of the high in-plane stiffness of such systems, careful design is required in order to avoid slender web or flange elements in the profile of the inner skin.

6.2.6 Sinusoidal sheeting

Appropriate *K*-values are provided in reference [8]. Reference [17] provides amended and significantly improved values.

6.2.5 Two skin roof

This clause is concerned with forms of construction which consist of two layers of metal sheeting with loose insulation placed between them. The two layers are connected together either directly, if for example one of the layers is a cassette, or indirectly through spacing elements.

6.2.5.1 The principles of this document can be applied to either skin, independently of the other skin, in order to determine its strength and flexibility.

6.2.5.2 In general, one skin, usually the inner, will be connected directly to the supporting members. In such cases it is safe to ignore the contribution of the other skin and to base the stressed skin design on the connected skin only.

6.2.5.3 If the spacing system provides a connection between the two skins, then it is permissible to share the in-plane load between the two skins according to their relative flexibilities taking into account the flexibility of the spacer system. It is necessary to determine the flexibility of the spacer system by testing.

6.2.6 Sinusoidal sheeting

Testing is required, if the sheeting is connected to the substructure by fastenings in the top of the corrugation. Otherwise the design can follow the rules for trapezoidal sheeting.

7. SHEAR DIAPHRAGMS AND RIGID JOINTED FRAMES

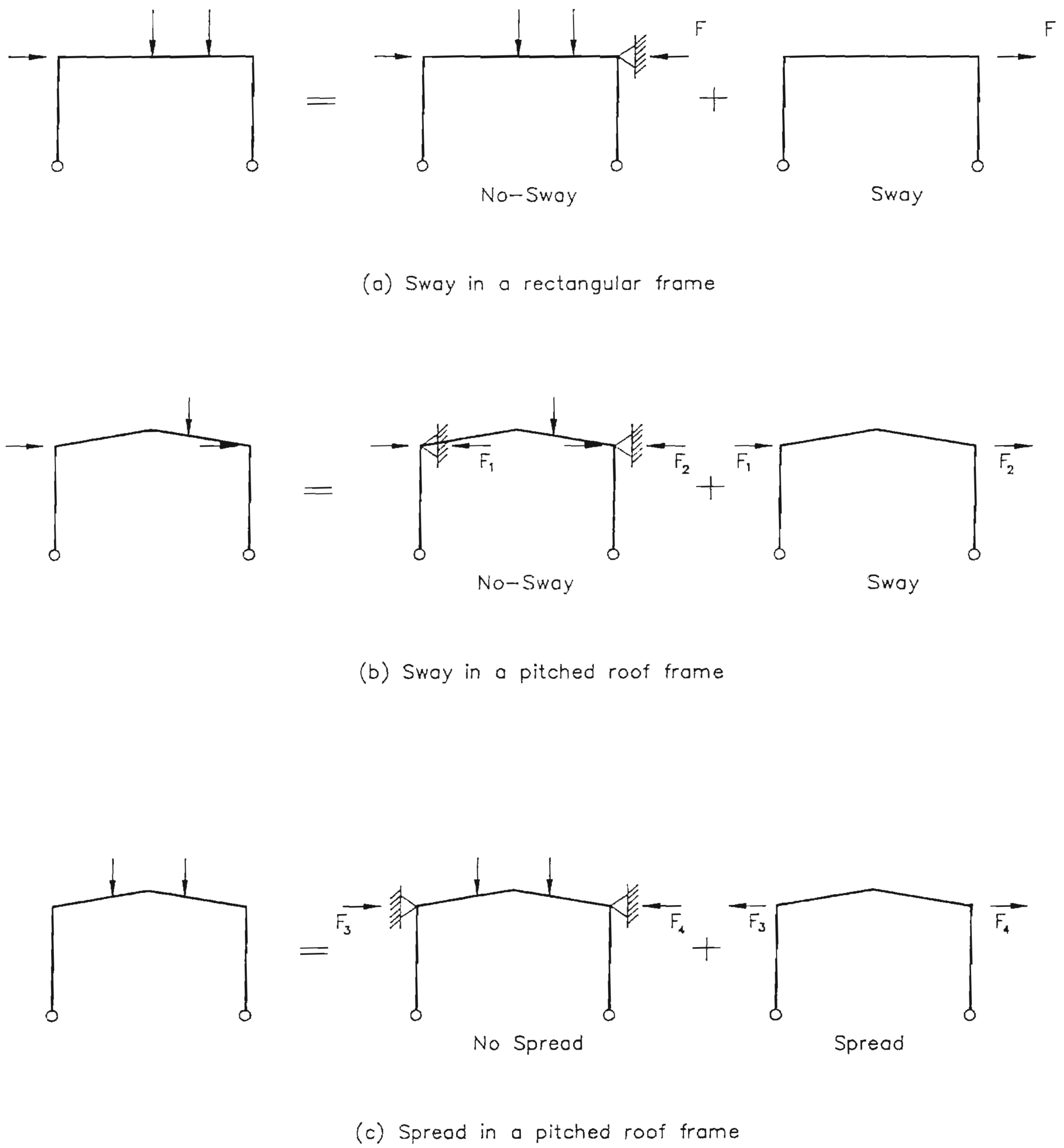


Figure C.7.1: No-sway and sway, no-spread and spread in rectangular and pitched roof portal frames.

7. SHEAR DIAPHRAGMS AND RIGID JOINTED FRAMES

7.1 *DESIGN PRINCIPLES*

Stressed skin action in the sheeting of buildings should be considered as tending to restrain joint movements of the supporting frames, with a consequent reduction in the associated forces and moments. In no case should it be taken as assisting to resist forces and moments out of the plane of the sheeting.

With reference to figure C.7.1, stressed skin action may be seen to have no effect on the no-sway or no-spread moments in a structure, but it may have considerable effect on the sway or spread moments. The amount of the effect may be shown to depend on the shear flexibility of a shear panel of sheeting relative to the frame flexibility.

The treatment given in this section applies only to single-bay flat roof frames and to symmetrical single bay pitched roof frames. It assumes that all frames in a building are similar, that all shear panels are similar and that all foundation and other conditions are similar. For multi-bay frames, or for cases where the above conditions are not met, computer methods should be employed as given in 8.5.

7.2 *ELASTIC DESIGN*

7.2.1 Rectangular frames: all frames loaded

The shear flexibility of a shear panel of sheeting is defined as the shear deflection per unit shear load c (figure C.7.2a) and may be calculated according to 5.2.1 or 5.9.1. The frame flexibility of a rectangular frame is the eaves deflection per unit horizontal eaves load, k (figure C.7.2b) and may be calculated by normal elastic methods. The relative flexibility is defined as $r = c/k$. If r is large (flexible sheeting or stiff frames) then the sheeting will have a small stiffening effect; if r is small (stiff sheeting or flexible frames) then the sheeting will have a large stiffening effect.

Table 7.1 Reductionfactor η for each frame in a building (see Annex C for the mathematical expressions for this table)

*	†	Values of relative flexibility r																								
		0.01	0.02	0.03	0.04	0.06	0.08	0.10	0.12	0.14	0.16	0.18	0.20	0.25	0.30	0.35	0.40	0.45	0.50	0.60	0.70	0.80	0.90	1.00	1.50	2.00
3	2	0.005	0.010	0.015	0.020	0.029	0.038	0.048	0.057	0.065	0.074	0.083	0.091	0.111	0.130	0.149	0.167	0.184	0.200	0.231	0.259	0.286	0.310	0.333	0.429	0.500
4	2	0.010	0.020	0.029	0.038	0.057	0.074	0.091	0.107	0.123	0.138	0.153	0.167	0.200	0.231	0.259	0.286	0.310	0.333	0.375	0.412	0.444	0.474	0.500	0.600	0.667
5	2	0.015	0.029	0.043	0.056	0.082	0.106	0.129	0.150	0.170	0.190	0.208	0.225	0.265	0.301	0.333	0.362	0.388	0.412	0.454	0.490	0.521	0.548	0.571	0.659	0.714
	3	0.020	0.039	0.057	0.075	0.109	0.140	0.170	0.198	0.225	0.250	0.273	0.296	0.347	0.392	0.432	0.468	0.500	0.529	0.580	0.622	0.658	0.688	0.714	0.805	0.857
6	2	0.020	0.038	0.056	0.073	0.104	0.134	0.160	0.185	0.208	0.230	0.250	0.268	0.310	0.347	0.379	0.407	0.432	0.455	0.494	0.526	0.554	0.579	0.600	0.677	0.727
	3	0.029	0.057	0.083	0.108	0.155	0.198	0.237	0.272	0.305	0.336	0.364	0.390	0.448	0.497	0.540	0.576	0.608	0.636	0.684	0.721	0.752	0.778	0.800	0.871	0.909
7	2	0.024	0.047	0.068	0.088	0.124	0.157	0.186	0.213	0.237	0.259	0.280	0.298	0.340	0.375	0.406	0.432	0.456	0.477	0.513	0.543	0.569	0.591	0.611	0.683	0.731
	3	0.039	0.074	0.108	0.139	0.196	0.246	0.291	0.331	0.367	0.400	0.429	0.457	0.515	0.563	0.604	0.638	0.667	0.692	0.734	0.767	0.793	0.815	0.833	0.892	0.923
	4	0.043	0.084	0.121	0.156	0.219	0.275	0.325	0.369	0.408	0.444	0.476	0.506	0.569	0.620	0.663	0.698	0.728	0.754	0.795	0.827	0.852	0.873	0.889	0.938	0.962
8	2	0.029	0.055	0.079	0.101	0.141	0.176	0.207	0.234	0.258	0.280	0.300	0.319	0.359	0.393	0.421	0.447	0.469	0.488	0.522	0.551	0.575	0.597	0.615	0.685	0.732
	3	0.048	0.091	0.130	0.167	0.231	0.286	0.334	0.376	0.413	0.446	0.475	0.502	0.558	0.603	0.641	0.672	0.698	0.721	0.758	0.787	0.811	0.830	0.846	0.898	0.927
	4	0.057	0.109	0.156	0.199	0.274	0.339	0.394	0.443	0.485	0.522	0.555	0.585	0.646	0.695	0.734	0.765	0.792	0.814	0.849	0.875	0.895	0.911	0.923	0.959	0.976
9	2	0.033	0.063	0.089	0.113	0.156	0.191	0.222	0.250	0.274	0.295	0.315	0.333	0.371	0.403	0.430	0.454	0.475	0.494	0.527	0.554	0.578	0.599	0.617	0.686	0.732
	3	0.056	0.106	0.151	0.191	0.260	0.318	0.367	0.409	0.446	0.478	0.506	0.532	0.585	0.627	0.662	0.690	0.715	0.735	0.770	0.796	0.818	0.836	0.851	0.901	0.928
	4	0.070	0.132	0.187	0.237	0.321	0.390	0.449	0.498	0.540	0.577	0.609	0.637	0.695	0.739	0.774	0.802	0.825	0.844	0.874	0.896	0.913	0.926	0.936	0.966	0.979
	5	0.075	0.141	0.199	0.252	0.341	0.414	0.475	0.526	0.570	0.608	0.641	0.670	0.729	0.773	0.808	0.835	0.857	0.875	0.903	0.923	0.938	0.949	0.957	0.981	0.990
10	2	0.037	0.070	0.098	0.124	0.168	0.204	0.235	0.261	0.285	0.306	0.325	0.342	0.379	0.409	0.436	0.458	0.479	0.497	0.529	0.556	0.579	0.599	0.618	0.686	0.732
	3	0.065	0.121	0.170	0.213	0.285	0.344	0.393	0.434	0.469	0.500	0.528	0.552	0.602	0.641	0.673	0.700	0.723	0.743	0.775	0.800	0.821	0.838	0.853	0.901	0.928
	4	0.083	0.154	0.216	0.270	0.360	0.431	0.490	0.539	0.580	0.615	0.645	0.672	0.725	0.766	0.797	0.822	0.843	0.860	0.886	0.906	0.920	0.932	0.941	0.968	0.980
	5	0.092	0.171	0.239	0.298	0.396	0.474	0.536	0.588	0.631	0.668	0.700	0.727	0.780	0.820	0.850	0.873	0.891	0.906	0.929	0.944	0.956	0.964	0.971	0.987	0.993
11	2	0.041	0.076	0.106	0.133	0.177	0.213	0.244	0.270	0.293	0.313	0.331	0.348	0.383	0.413	0.438	0.461	0.481	0.499	0.530	0.556	0.579	0.600	0.618	0.686	0.732
	3	0.073	0.134	0.186	0.231	0.305	0.364	0.412	0.452	0.486	0.516	0.542	0.565	0.612	0.650	0.680	0.706	0.727	0.746	0.777	0.802	0.822	0.839	0.854	0.901	0.928
	4	0.095	0.175	0.241	0.299	0.392	0.464	0.521	0.569	0.608	0.641	0.670	0.695	0.744	0.781	0.810	0.833	0.852	0.867	0.892	0.910	0.924	0.934	0.943	0.969	0.981
	5	0.109	0.199	0.274	0.338	0.441	0.521	0.583	0.633	0.675	0.709	0.738	0.763	0.812	0.847	0.873	0.893	0.909	0.922	0.941	0.954	0.963	0.970	0.976	0.989	0.994
	6	0.113	0.206	0.285	0.351	0.458	0.539	0.603	0.654	0.696	0.731	0.760	0.785	0.833	0.867	0.892	0.911	0.926	0.938	0.954	0.966	0.974	0.980	0.984	0.994	0.997
12	2	0.045	0.082	0.114	0.141	0.185	0.221	0.251	0.276	0.298	0.318	0.335	0.351	0.386	0.415	0.440	0.462	0.482	0.499	0.530	0.557	0.580	0.600	0.618	0.686	0.732
	3	0.081	0.146	0.201	0.247	0.322	0.380	0.427	0.465	0.498	0.526	0.551	0.573	0.618	0.654	0.684	0.709	0.730	0.748	0.779	0.803	0.823	0.840	0.854	0.901	0.928
	4	0.107	0.193	0.264	0.323	0.418	0.489	0.545	0.590	0.628	0.659	0.686	0.709	0.755	0.790	0.817	0.839	0.856	0.871	0.894	0.912	0.925	0.935	0.944	0.969	0.981
	5	0.124	0.224	0.305	0.373	0.478	0.557	0.618	0.666	0.705	0.737	0.764	0.787	0.831	0.863	0.886	0.904	0.918	0.930	0.946	0.958	0.967	0.973	0.978	0.990	0.995
	6	0.133	0.239	0.325	0.397	0.508	0.590	0.653	0.702	0.741	0.774	0.800	0.823	0.865	0.895	0.916	0.932	0.944	0.953	0.967	0.975	0.981	0.986	0.989	0.996	0.998

* Number of frames in building; † frame number. *Note*: The number of frames in the building is inclusive of the gable ends. Frame 1 is the end gable, frame 2 the penultimate frame and so on.

Comments

C.52

A reduction factor, η, may be applied to the sway forces and moments in a bare frame, to take account of the stiffening effect of the sheeting. It may be shown to depend on the value of r, on the number of frames in the building, and on the position of the frame under consideration in the building (figure C.7.4a). Values of η for each frame in a building are given in table 7.1, and the mathematical expressions are given in reference [8].

The moments in a clad rectangular frame may be expressed as:

bending moment in clad frame = no-sway bending moment in bare frame
+ η x sway bending moment in bare frame

The sway force on the clad frame is η x sway force on the bare frame, and the sway force on the sheeting is $(1 - \eta)$ x sway force on the bare frame.

7.2.2 Pitched roof frames: all frames loaded

The in-plane shear flexibility of a shear panel of sheeting, c, may be calculated according to 5.2.1 or 5.9.1, taking the depth of the shear panel as the length of one roof slope. If θ is the angle of the rafter to the horizontal, the horizontal shear flexibility, c_h, is given by:

$$c_h = c \sec^2\theta.$$

The general sway of the frame may be divided into sway (a) and spread (b). The frame flexibility due to sway, k_{sw} is defined in figure C.7.3a and the frame flexibility due to spread, k_{sp} is defined in figure C.7.3b. The corresponding relative flexibilities are $r_{sw} = c_h/k_{sw}$ and $r_{sp} = c_h/k_{sp}$. The reduction factors η_{sw} and η_{sp} may be obtained for each frame in a building (figure C.7.4b) from table 7.1.

The moments in a clad pitched roof portal frame may be expressed as:

bending moment in clad frame = no-sway/no-spread bending moment in bare frame + η_{sw} x sway bending moment in bare frame + η_{sp} x spread bending moment in bare frame

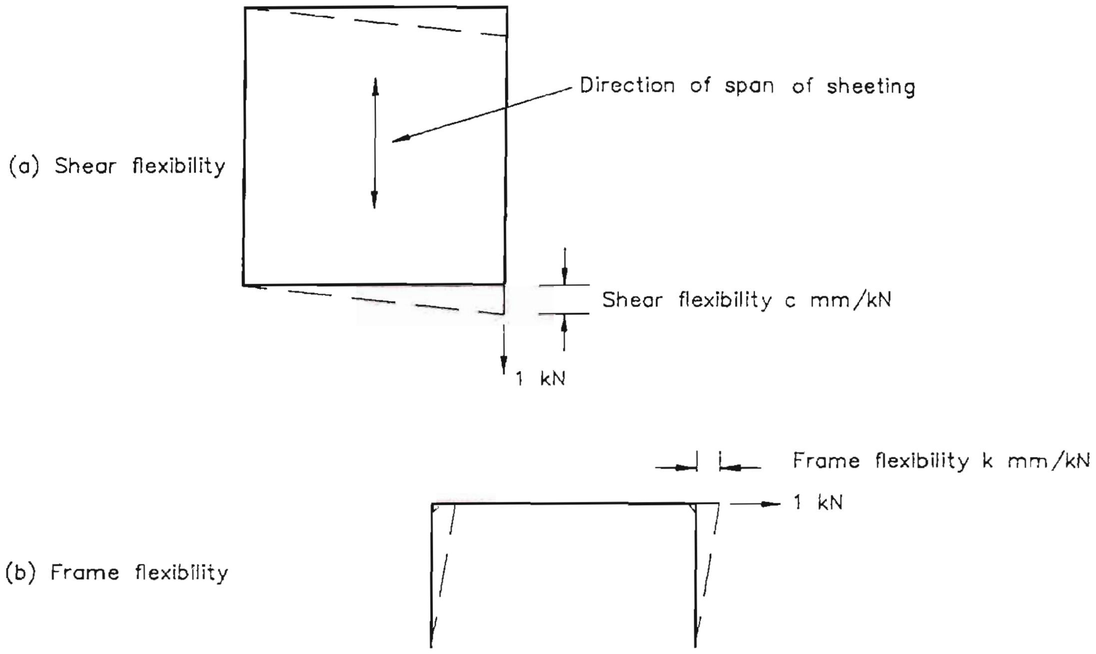


Figure C.7.2: Definitions of shear flexibility and frame flexibility.

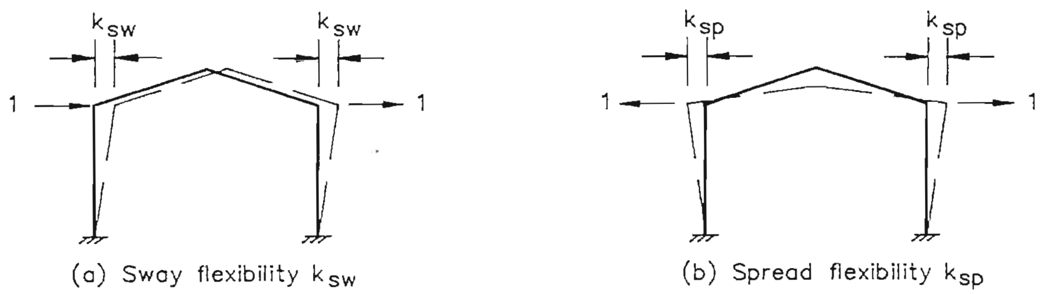


Figure C.7.3: Frame flexibilities for a pitched roof portal frame.

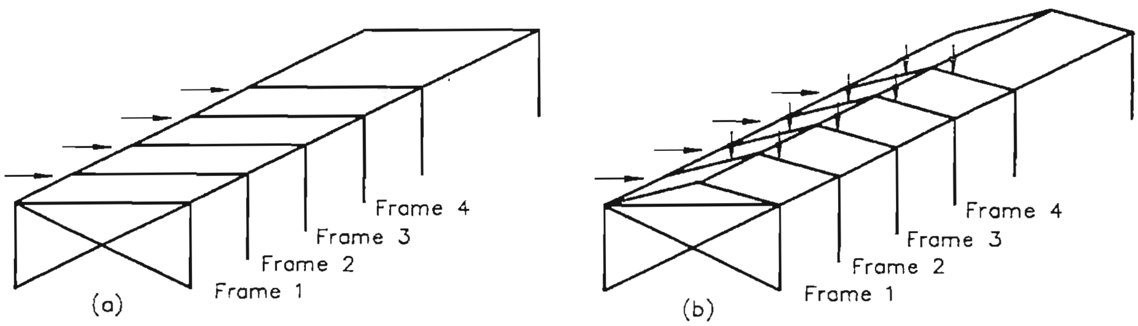


Figure C.7.4: Numbering of frames in clad buildings.

Note: The procedure given may be applied only to symmetrical frames. For asymmetrical frames, reference should be made to 8.5.

7.2.3 One frame loaded

Stressed skin action in a sheeted building may be shown to be especially effective if only one frame is loaded. The effect of sheeting is to distribute the load to a number of frames. Table 7.2 gives factors by which the value of η in table 7.1 should be divided, if only one frame in a building is loaded.

Note: The procedure given may be applied only to symmetrical frames. For asymmetrical frames, reference should be made to 8.5.

7.3 PLASTIC DESIGN OF THE SUBSTRUCTURE

7.3.1 General

Plastic design of the substructure should be in accordance with EC 3 parts 1.1 and 1.3, except that it should be assumed that the effect of stressed skin diaphragms is to modify the loading on frames in a clad building as shown in 7.3.2 and 7.3.3. The designer should satisfy himself that second order effects are not significant or should carry out a rational analysis taking second order effects into account.

Provided that the criterion for the least shear strength of a panel of sheeting is by tearing at the seam fasteners or sheet/shear connector fasteners then the shear panel will generally be able to sustain large shear deformations at the design shear capacity. In this case, at the ultimate load of a building, collapse will occur in all intermediate frames simultaneously and at this stage the forces on each frame will be the same.

7.3.2 Rectangular frames

For a clad flat roof building (figure C.7.5a) the restraining force R provided by the sheeting is the same at each intermediate frame at collapse (figure C.7.5b) and is given by:

$$R = \frac{2V^*}{n-1}$$

Table 7.2 Factors by which η (table 7.1) should be divided for central frame only loaded

value of r	number of frames in building									
	3	4	5	6	7	8	9	10	11	12
0.00	1.00	1.50	2.00	2.50	3.00	3.50	4.00	4.50	5.00	5.50
0.01	1.00	1.50	2.00	2.49	2.98	3.47	3.95	4.43	4.90	5.37
0.02	1.00	1.50	1.99	2.48	2.96	3.43	3.90	4.36	4.81	5.24
0.03	1.00	1.49	1.99	2.46	2.94	3.40	3.86	4.29	4.72	5.12
0.04	1.00	1.49	1.98	2.45	2.92	3.37	3.81	4.23	4.64	5.01
0.06	1.00	1.49	1.97	2.43	2.89	3.31	3.73	4.10	4.48	4.81
0.08	1.00	1.48	1.96	2.41	2.85	3.25	3.65	3.99	4.34	4.63
0.10	1.00	1.48	1.95	2.39	2.82	3.20	3.57	3.89	4.21	4.46
0.12	1.00	1.47	1.94	2.36	2.79	3.14	3.50	3.79	4.08	4.31
0.14	1.00	1.47	1.93	2.34	2.75	3.09	3.43	3.70	3.97	4.18
0.16	1.00	1.46	1.93	2.33	2.72	3.05	3.37	3.62	3.87	4.05
0.18	1.00	1.46	1.92	2.31	2.69	3.00	3.31	3.54	3.77	3.94
0.20	1.00	1.45	1.91	2.29	2.67	2.96	3.25	3.47	3.68	3.83
0.25	1.00	1.44	1.89	2.24	2.60	2.86	3.12	3.30	3.48	3.60
0.30	1.00	1.43	1.87	2.20	2.54	2.77	3.01	3.16	3.31	3.41
0.35	1.00	1.43	1.85	2.17	2.48	2.69	2.90	3.03	3.16	3.24
0.40	1.00	1.42	1.83	2.13	2.43	2.62	2.81	2.92	3.03	3.10
0.45	1.00	1.41	1.82	2.10	2.38	2.55	2.72	2.82	2.92	2.97
0.50	1.00	1.40	1.80	2.07	2.33	2.49	2.65	2.73	2.82	2.86
0.60	1.00	1.38	1.77	2.01	2.25	2.38	2.51	2.58	2.65	2.68
0.70	1.00	1.37	1.74	1.96	2.18	2.29	2.40	2.45	2.50	2.53
0.80	1.00	1.36	1.71	1.91	2.11	2.21	2.30	2.34	2.39	2.40
0.90	1.00	1.34	1.69	1.87	2.05	2.13	2.22	2.25	2.29	2.30
1.00	1.00	1.33	1.67	1.83	2.00	2.07	2.14	2.17	2.20	2.21
1.50	1.00	1.29	1.57	1.69	1.80	1.84	1.88	1.89	1.90	1.91
2.00	1.00	1.25	1.50	1.58	1.67	1.69	1.71	1.72	1.73	1.73

See Annex C for the mathematical expressions for this table.

where V^* = is the design shear capacity of a shear panel,
n = is the number of shear panels in the length of the building.

Each frame should then be plastically designed under a net sidesway force of (applied load - R) as shown in figure C.7.5c.

7.3.3 **Pitched roof frames**

For a clad pitched roof building under side load (figure C.7.6a) or vertical load (figure C.7.7a) the in-plane restraining force R provided by the sheeting on each roof slope is the same at each intermediate frame at collapse (figure C.7.6b and C.7.7b) and is given by:

$$R = \frac{2V^*}{n-1}$$

where the symbols are the same as in 7.3.2.

The horizontal component of the restraining force R_h is given by:

$$R_h = R\cos\theta$$

where θ is the angle of the rafter to the horizontal.

For a clad building under side load, each frame should be plastically designed under the net sidesway force of (applied load - R_h) as shown in figure C.7.6c.

For a clad building under vertical load, each frame should be plastically designed under the action of the vertical applied load and the horizontal restraining force at the eaves, as shown in figure C.7.7c.

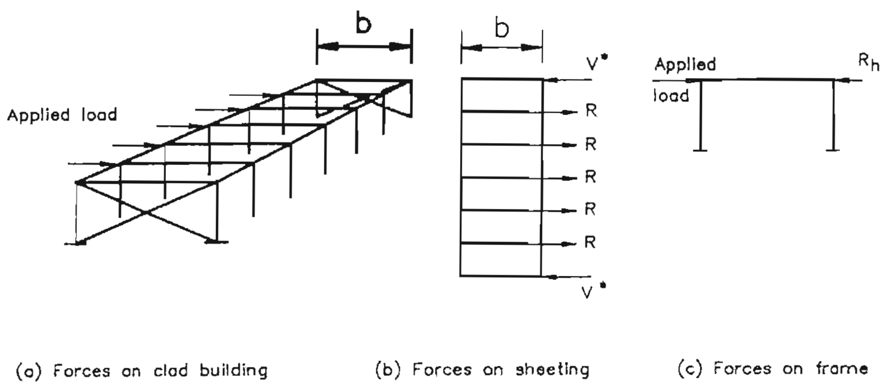


Figure C.7.5: Plastic design of clad rectangular portal frame.

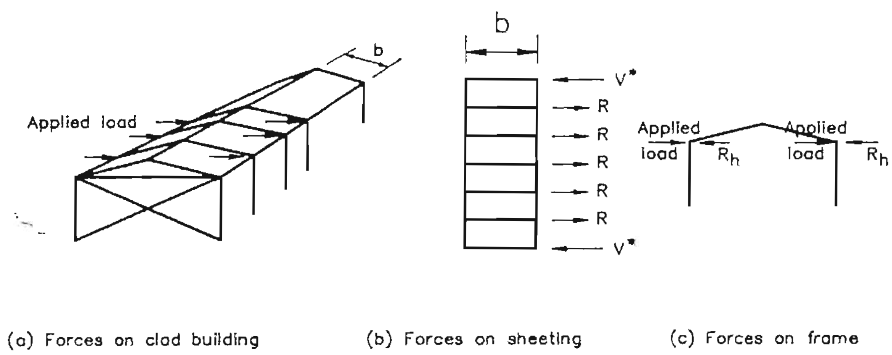


Figure C.7.6: Plastic design of clad pitched roof portal frame under side loads.

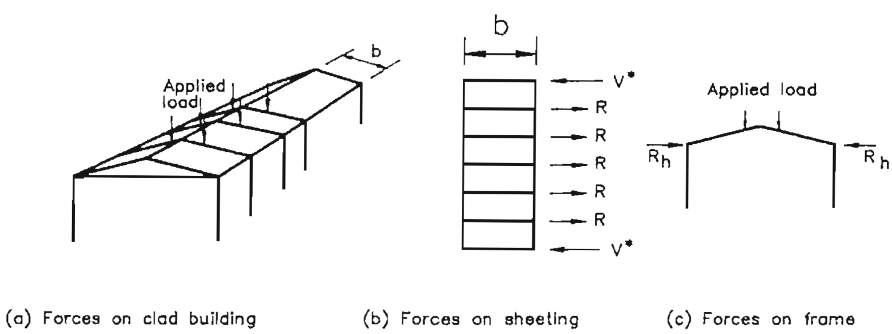


Figure C.7.7: Plastic design of clad pitched roof portal frame under vertical loads.

8. OTHER CONSIDERATIONS IN THE DESIGN OF DIAPHRAGMS

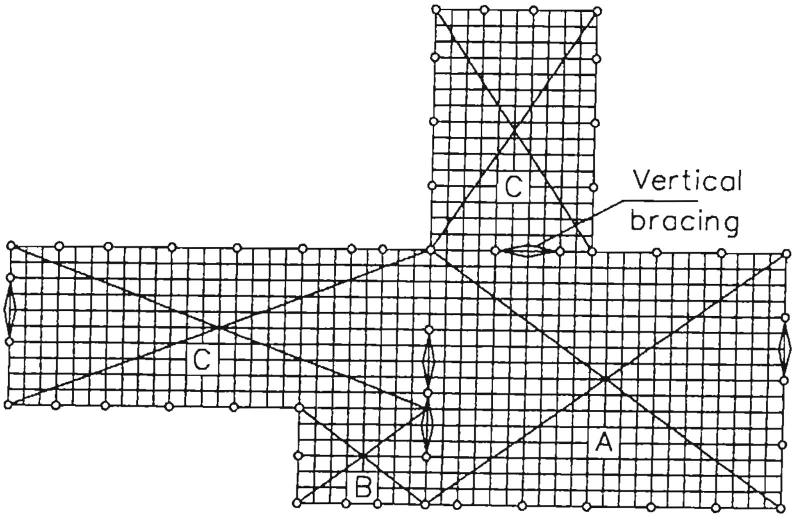


Figure C.8.1: Irregular diaphragm.

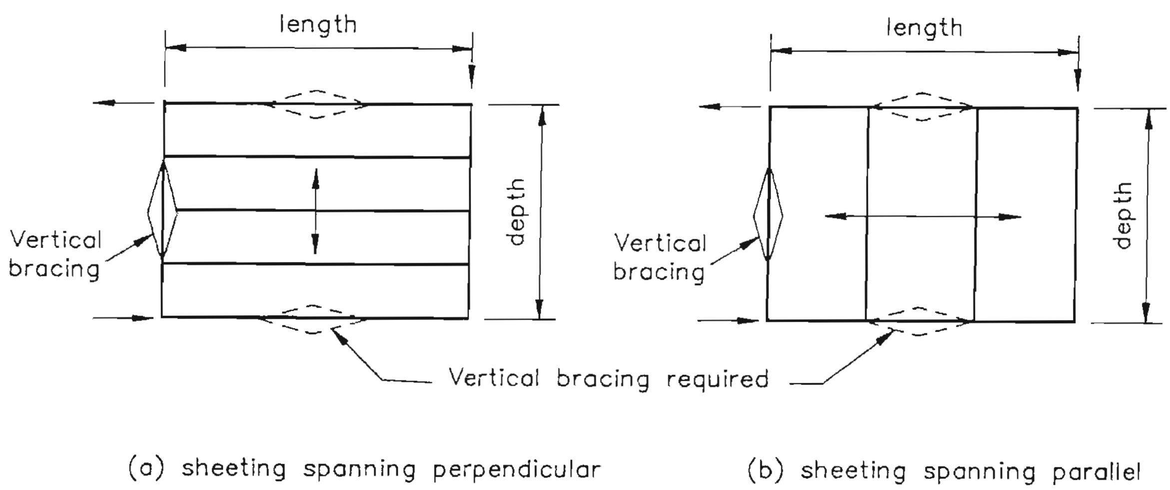


Figure C.8.2: Cantilevered diaphragms.

8. OTHER CONSIDERATIONS IN THE DESIGN OF DIAPHRAGMS

8.1 *INTRODUCTION*

In addition to regular diaphragms in flat or pitched roof buildings, irregular or complex diaphragms may occur such as:

(a) diaphragms in different directions and at different levels in flat roofs,
(b) shear panel assemblies in which one gable end cannot be braced,
(c) diaphragms with openings,
(d) diaphragms with concentrated and distributed in-plane loads,
(e) diaphragms in multi-bay or asymmetrical structures,
(f) composite floor diaphragms.

Information concerning curved surfaces and hyperbolic paraboloid shell elements can be found in references [4] and [8].

8.2 *IRREGULAR DIAPHRAGMS*

In buildings with flat roofs in different directions and at different heights, the requirements for diaphragms should apply separately to (figure C.8.1):

- each area of roof deck at different levels,
- each part of a building adjoining an enclosed open area,
- each wing of a building.

Each diaphragm zone should be bounded by steel frame members and the gable walls supporting each diaphragm should be vertically braced or designed as diaphragms themselves. The conditions and restrictions listed in 4.2. should be observed.

Where a roof diaphragm projects beyond a line of vertical bracing then the diaphragm becomes a cantilevered diaphragm (figure C.8.2) and the two adjacent walls should be braced to prevent body rotation. The length/depth ratio of the cantilever should not normally exceed 2.0.

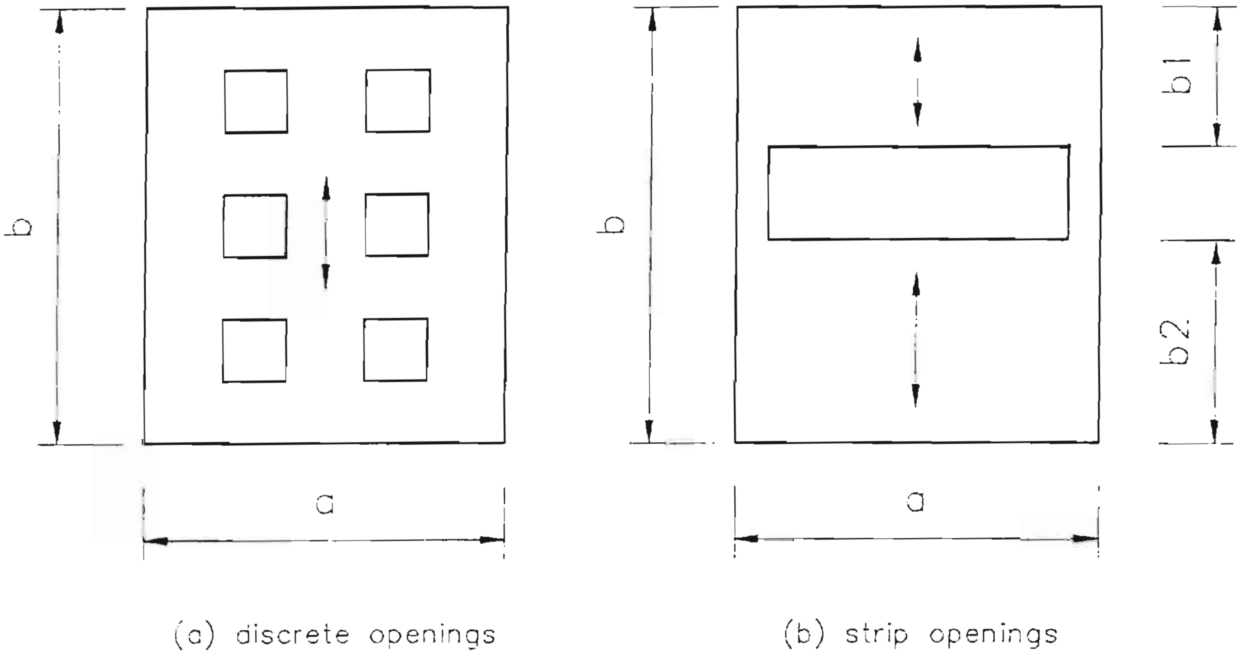


Figure C.8.3: Openings in a diaphragm.

8.3 *DIAPHRAGMS WITH OPENINGS*

Small openings, totalling no more than 3 % of the area in each shear panel, may be ignored for the purpose of calculating diaphragm strength and stiffness. Larger openings may be of two types:

(a) discrete openings (figure C.8.3a),

(b) strip openings (figure C.8.3b).

8.3.1 Requirements for discrete openings

For discrete openings in diaphragms, the following requirements should be fulfilled:

(a) Openings should be bounded on all four sides by steel trimmers attached to the supporting structure.

(b) The sheeting should be fixed in every trough to trimmers running perpendicular to the corrugations and at a spacing not grater than 300 mm to trimmers running parallel to the corrugations.

(c) In a direction perpendicular to the corrugations, the adjacent sheet widths between openings should be equal to or greater than the width of the opening.

(d) In a direction parallel to the corrugations, the total depth of the openings should not exceed 25 % of the depth of the diaphragm.

(e) The requirements (c) and (d) result in a maximum area of openings of 12½ % of the area in each shear panel. Openings up to 15 % of the area may be allowed if further detailed calculations are made in accordance with reference [8].

8.3.2 Strength of diaphragms with discrete openings

For diaphragms with openings conforming to 8.3.1 and a uniform spacing of seam fasteners throughout, the maximum reduction in diaphragm strength may be taken as 50 %. For non reduction in diaphragm strength, the number of seam fasteners on a seam interrupted by an opening should be doubled, or openings should be precluded from the end 25 % of the length of the building.

8.3.3 Flexibility of diaphragms with discrete openings

The increase in the flexibility of a diaphragm due to the presence of openings may be taken into account by applying a multiplying factor to the value of the shear flexibility c obtained using tables 5.5 and 5.9. The value of this multiplying factor is:

Fasteners in every corrugation : $$\frac{1}{1 - 4(\frac{h}{50})^{1/2}(\frac{A_d}{ab})}$$

Fasteners in alternate corrugations: $$\frac{1}{1 - 2.5(\frac{h}{50})^{1/2}(\frac{A_d}{ab})}$$

where h is the height of the profile [mm],

A_d is the total area of openings in a shear panel,

a is the width of the shear panel,

b is the depth of the shear panel.

8.3.4 Deflection of diaphragms with discrete openings

For a diaphragm assembly in which some shear panels contain openings and others do not, the expressions for deflection given in 5.2.2 and 5.9.2 are not applicable. In such cases the mid-length deflection should be calculated as Σ(shear force in shear panel x shear flexibility of shear panel) considered over half the length of the diaphragm.

8.3.5 Strip openings

Diaphragms with strip openings need not comply with the requirements of 8.3.1(c). Provided b_1 and b_2 (figure C.8.3) do not differ by more than 20 % the effective depth may be taken as the sum of b_1 and b_2. In the other cases the diaphragm may be conservatively treated by taking the effective depth as the greater of b_1 or b_2.

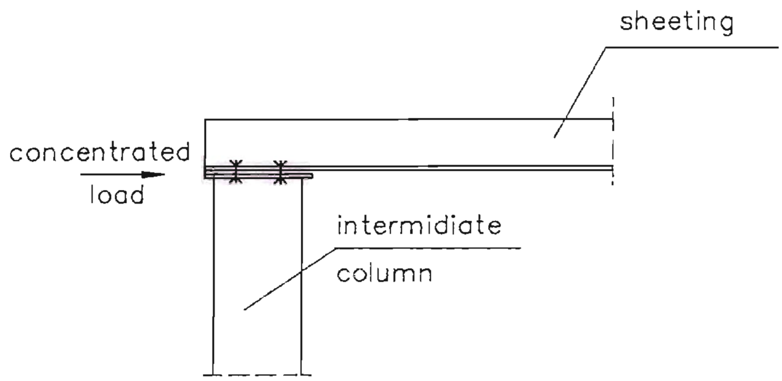
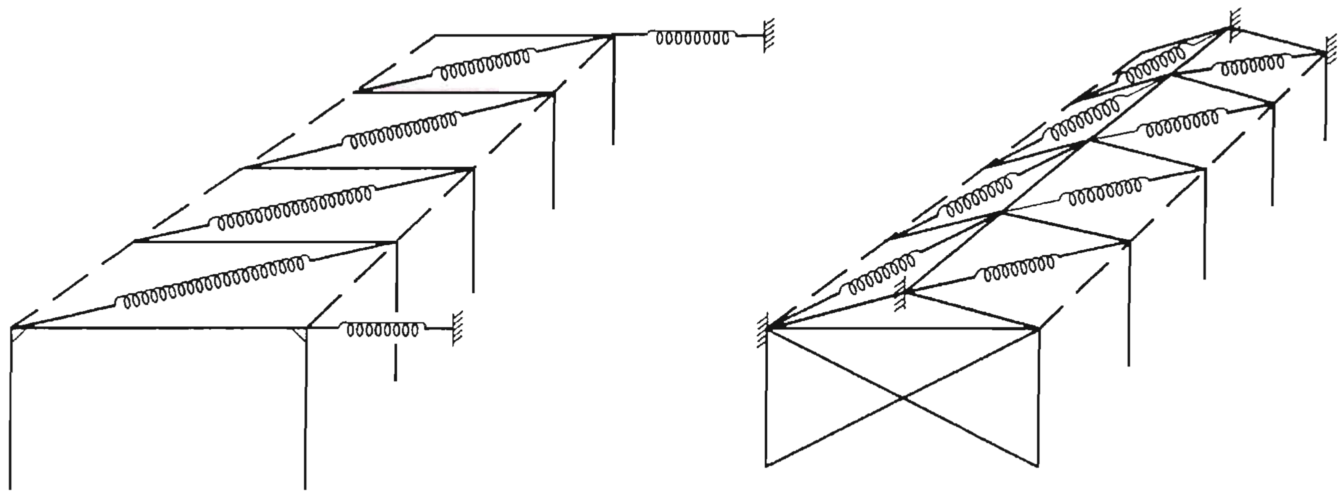


Figure C.8.4: Local force introduced by intermediate column.



(a) Flat roof building (b) pitched roof building

Figure C.8.5: Mathematical models of sheeted buildings.

8.4 *IN PLANE LOADS*

8.4.1 Concentrated loads

Concentrated loads in the plane of the diaphragm may be applied to the diaphragm other than at joints of structural members, if it can be shown by test or calculation (see e.g. reference [19]) that the local forces are within permissible limits. Annex D provides guidelines to check the capacity of a diaphragm with regard to the introduction of concentrated loads.

An example of such a local force is the reaction at the top of an intermediate column, connected to the sheeting (figure C.8.4). When the reaction acts in a direction parallel to the corrugations, the following items must be verified:

- Shear resistance of the connections, according to EC 3 part 1.3,
- Effects (compression and flexure due to the eccentricity) in the sheeting induced by the local force, in addition to the effects resulting from the primary function of the sheeting,
- Influence of the local force must be taken into account in the calculation of both the deflection and the resistance of the shear panel.

8.4.2 Distributed loads

Distributed loads in the plane of the diaphragm may be equivalent to concentrated loads at the joints. Local bending of edge members, due to such distributed load, need only be considered when the member runs parallel to the corrugations.

8.5 *DIAPHRAGMS IN MULTI-BAY OR ASYMMETRICAL STRUCTURES*

The analysis of clad frames given in 7.2 and 7.3 may only be applied to single-bay flat roof frames and to symmetrical single-bay pitched roof frames. For other cases, it is necessary to use computer analysis. Any program suitable for the analysis of plane frames may be used. The procedure is given with reference to figures C.8.5a and C.8.5b.

Each intermediate frame is a plane rigid-jointed frame with pinned or fixed feet. The individual frames are connected together by sheeting panels the shear flexibility of which is summarised by the single quantity c. The computer analysis consists of moving the individual frames close together

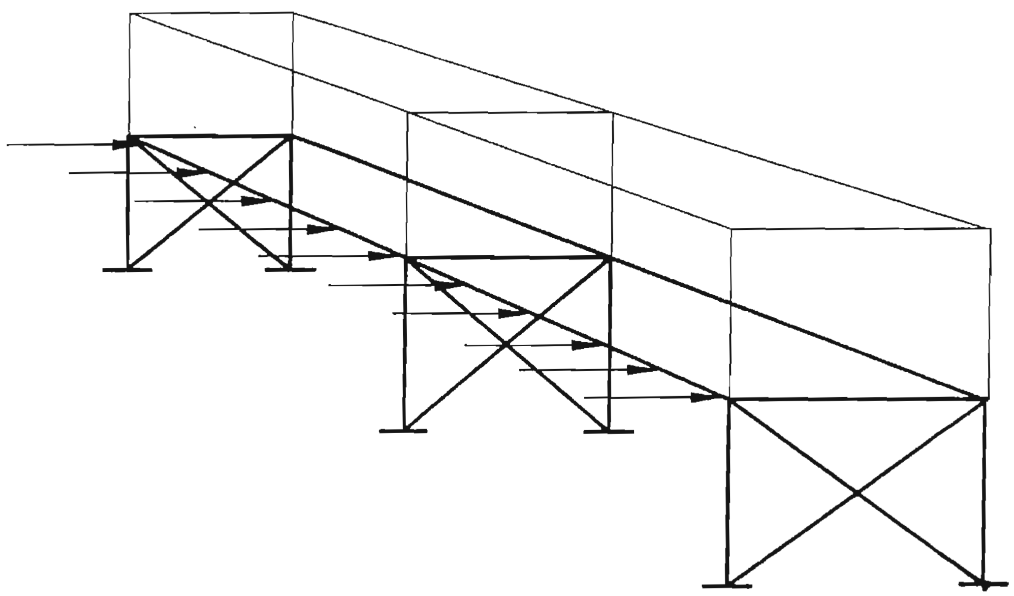


Figure C.8.6: Horizontal floor diaphragm and braced frames.

and replacing the sheeting panels by springs (either tensile or compressive) of the same flexibility c. Flexible end gables may be modelled as shown in figure C.8.5a and rigid end gables may be modelled as shown in figure C.8.5b.

Figure C.8.5 shows the individual frames slightly apart for illustrative purposes, but computationally there is no reason why they cannot be coincident. If the frames are moved into coincidence, the complete three-dimensional structure reduces to a plane frame in which there are a number of joints with the same co-ordinates and a number of members in the same position. This unusual feature does not invalidate the use of a plane frame program to analyse the clad structure.

The spring flexibility c may be modelled as a member of length L, modulus of elasticity E and equivalent cross section area A,

then: $$A = \frac{L^3}{cb^2E}$$

8.6 *COMPOSITE FLOOR DIAPHRAGMS*

8.6.1 Composite floors

Composite sheet steel/concrete floors should first be designed for their primary function in vertical bending according to EUROCODE 4 (reference [20]). They may then be utilised without further checking as horizontal diaphragms to resist transient horizontal load such wind forces and seismic forces as shown in figure C.8.6. The strength of the fasteners must be checked.

8.6.2 General conditions

For a composite floor used as a diaphragm the following conditions should be fulfilled:

(a) Floor diaphragms should comply with the requirements of 4.2.

(b) The directions of span of the sheeting should be in accordance with 4.3.1.

(c) For sheeting fastened on two sides only, as in figure R.4.3 case 3, shear connectors or their equivalent must be used at the end gables.

(d) The fastener arrangements should comply with 4.3.4.

(e) Floor diaphragms with openings should comply with the requirements of 8.3.

8.6.3 Types of floor diaphragm

There are two types of composite floor diaphragm:

(a) Floors in which only the profiled steel sheet is attached to the supporting structure,

(b) Floors with through-deck studs so that both the profiled steel sheet and the concrete topping are attached to the supporting structure.

8.6.4 Erection stage and final stage

Calculations on composite sheet steel/concrete floors should be made under the following conditions :

(a) in the erection stage, when profiled steel sheet acts alone as a diaphragm,

(b) in the final stage, when the profiled steel sheet acts compositely with the concrete as a diaphragm.

Considering wind loads, the diaphragm at stage (a) should be calculated for wind loads acting on the storey at this stage of construction, and the diaphragm at stage (b) should be calculated for wind on the full storey height. The diaphragm should normally be fixed in place before the side cladding is erected or any vertical erection bracing is removed. If the side cladding is erected first, then the building should be designed for this wind condition and temporary horizontal wind bracing should be provided when necessary.

8.6.5 Strength of steel deck alone

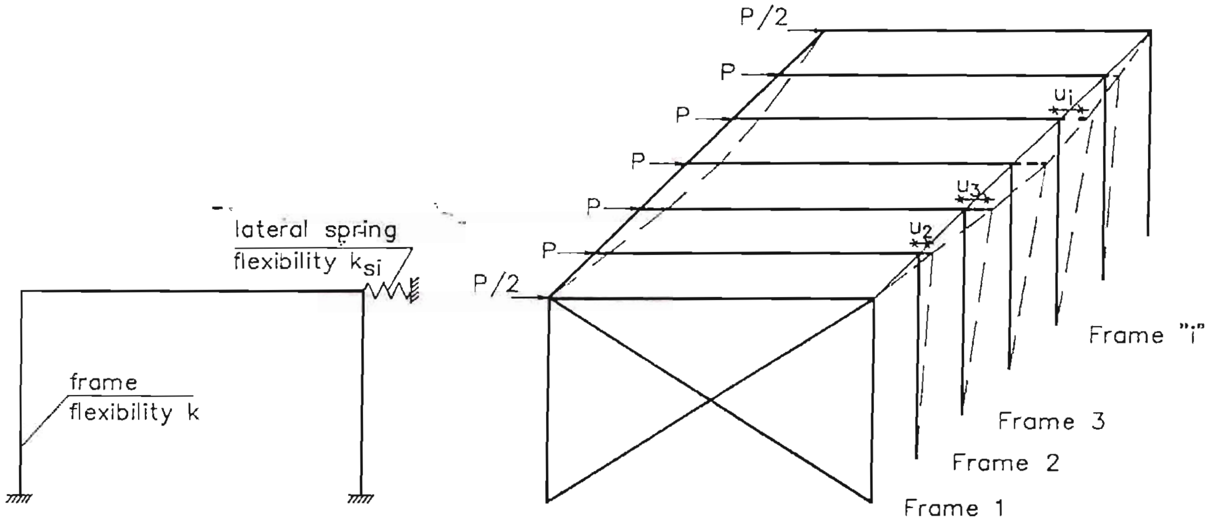
The strength of the steel deck acting alone as a diaphragm in the erection stage, should be calculated in accordance with 5.1 or 5.8.

8.6.6 Deflection of steel deck alone

The deflection of the steel deck acting alone as a diaphragm in the erection stage should be calculated in accordance with 5.2 or 5.9.

8.6.7 Strength of the combined steel deck and concrete slab

The completed composite floor, comprising the steel deck and the concrete slab may be assumed to act as a rigid diaphragm. Its strength will therefore be determined solely by the strength of its connections to the supporting structure. The forces in these connections should be determined by a rational analysis which recognises the rigid-body action of the diaphragm.



(a) Intermediate frame restrained by lateral spring

(b) Lateral deflection of frames

Figure C.8.7: Calculation model for buckling length of columns.

(a) If the connection between the deck and the supporting structure is solely by sheet to supporting member fasteners (e.g. self-tapping screws), the strength of these fasteners is unaffected by the presence of the concrete and the values used for the case when the steel deck acts alone may be used.

(b) If shear studs offering a direct connection between the concrete and the supporting structure are provided, the strength of these studs will determine the strength of the diaphragm. Design expressions for the strength of headed stud shear connectors will be found in Eurocode 4 (reference [20]).
The strength of other types of shear studs should be determined by testing.

8.6.8 Deflection of the combined steel deck and concrete slab

The completed composite floor, comprising the steel deck and the concrete slab may be assumed to act as a rigid diaphragm. Its deflection will therefore be solely due to the flexibility of the connection to the supporting structure. The flexibility of headed stud shear connectors may be ignored.

8.7 *BUCKLING LENGTH OF COLUMNS*

When taking into account the stabilising effect of the diaphragm on the frames, due consideration (see EUROCODE 3 part 1.1 chapter 5) must be made for the supplementary load exercised on the diaphragm by the stabilised frames.

The buckling length of the columns may be calculated using the model shown in figure C.8.7 in which each frame is supported by the sheeting, that acts as an elastic support. Also references [21, 22] provide information concerning this subject.

9. DIAPHRAGM BRACING

Annex B provides a procedure to determine the strength and stiffness of shear panels used for stabilising doubly symmetrical I-sections used as beams or columns. For this procedure it is assumed that beams are braced on the compression flange and possess a constant bending moment. The method is formulated by the Institut für Statik of the TH Darmstadt in Germany. Annex B IV gives an alternative method which is based on the work of Sokol (reference [37]). Furthermore reference [38] shows a procedure developed by Lawson and Nethercot.

References [23] - [28] provide methods to check the strength and stiffness of shear panels as applied in the USA. These methods distinguish between the cases when the shear panels are used for a fully or partially stabilising function:

- fully stabilising : the shear panel prevents buckling of columns or lateral instability of beams so that failure will occur in another mode;
- partially stabilising: buckling of columns or lateral instability of beams is the governing failure mode but failure will occur at a higher load than in the absence of the shear panel.

Furthermore these references give information for the stability of beams braced by a diaphragm on the tension flange (although tension flange bracing is less effective).

9. DIAPHRAGM BRACING

A panel of sheeting with the direction of span perpendicular to beams or columns and fastened to it, will provide a stabilising action (to prevent flexural, flexural-torsional and flexural-lateral buckling or a combination of these). When using this stabilising action, the strength and stiffness of the shear panel should meet the requirements formulated in Annex B.

In the case of a combination of diaphragm action caused by stabilisation of members and by external load, the combined effects should be taken into account when relevant.

A hot rolled I-section with a height smaller than or equal to 200 mm. and fastened to a diaphragm is sufficiently stabilised against lateral instability. The minimum requirements for the fastenings are the same as in stressed skin design.

During erection the stability of the structure should be assured.

10. FOLDED PLATE ROOFS

10.1 *INTRODUCTION*

The design of light gauge steel folded plate roofs involves consideration of the following elements shown in figure C.10.1:

(1) fold line members along the apexes and valleys,

(2) sheeting on each roof slope,

(3) gable framing,

(4) side sheeting or equivalent.

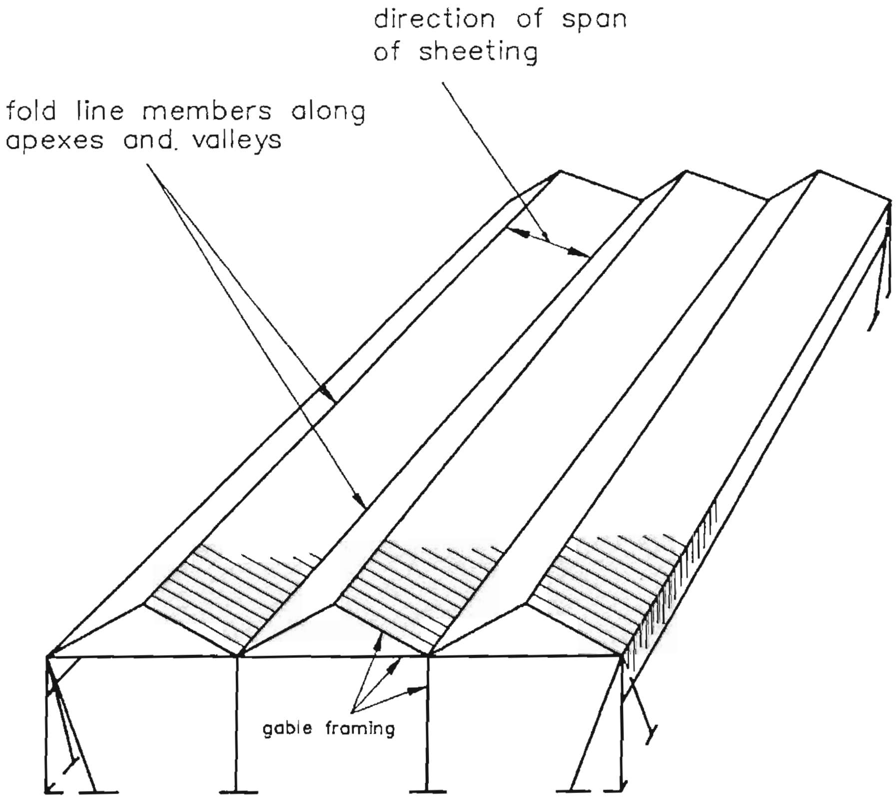


Figure C.10.1: General arrangement of a typical light gauge steel folded plate roof.

Under vertical load, there is a component of load down the roof slope, so that provided the gables are tied, each roof slope acts like an inclined

10. FOLDED PLATE ROOFS

10.1 *INTRODUCTION*

plate girder spanning from gable to gable. Under in-plane load, the gables take the reactions, the sheeting acts as a web and takes the shear, and the fold-line members act as flanges and take the axial tension and compression. The side sheeting or edge columns take the unbalanced vertical forces at the extreme eaves of the building. The action of a folded plate roof is similar to that of a sheeted pitched roof building without intermediate frames.

The stability of a folded plate roof depends entirely on stressed skin action so that the sheeting must not be removed without proper consideration.

10.2 *DESIGN PRINCIPLES*

Note: National loading requirements may be different from the case shown. Deep snow in the valleys may provide the design case.

The preferred units for the design expressions are given in 1.4.

10.2 DESIGN PRINCIPLES

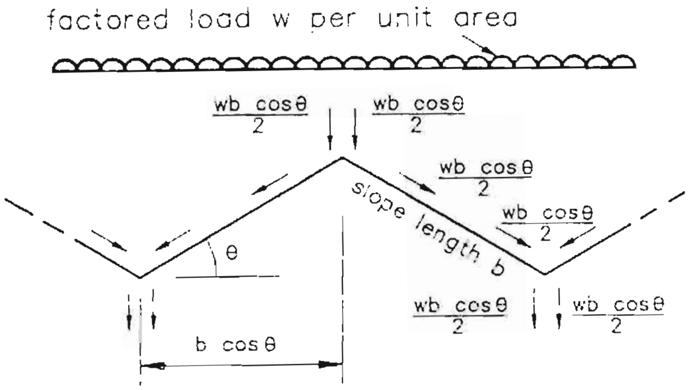


Figure R.10.1: Loads on a typical roof element per unit length.

With reference to figure R.10.1, the factored vertical load per roof slope is $w\ b\ \cos\theta$

and the factored in-plane load per roof slope is $q = w\ b\ \cot\theta$

where q is the factored in-plane distributed line load carried by each roof slope,

w is the factored load per unit plan area of roof,

b is the depth of the web of the folded plate, measured on the slope,

θ is the angle of the web to the horizontal.

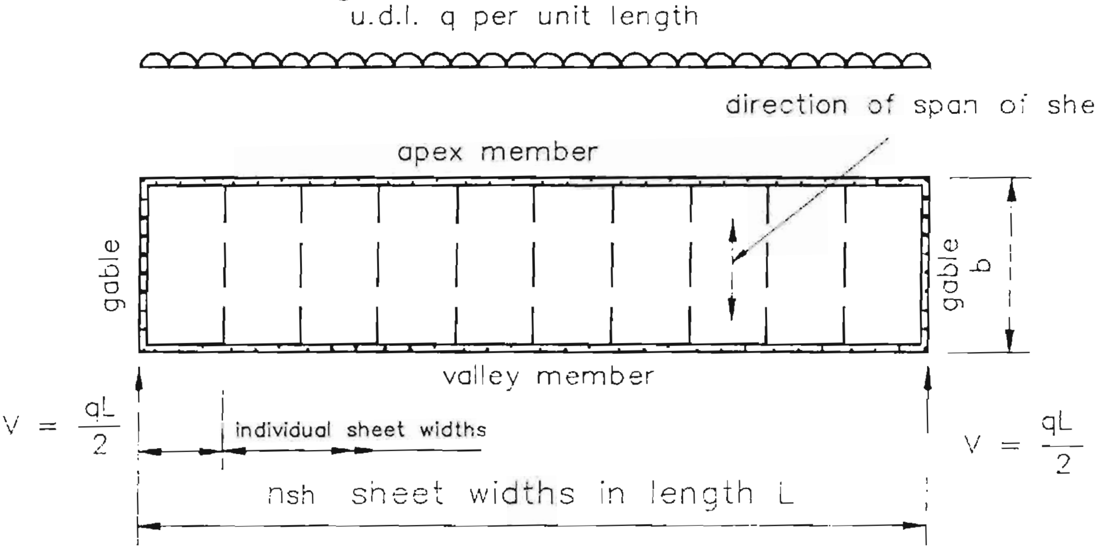


Figure R.10.2: Typical diaphragm girder of a folded plate roof.

Each roof slope should be designed as a plate girder of depth b and length L between gable frames under a distributed load q (figure R.10.2). The fold-line members, which are fully laterally restrained by the sheeting, should be designed as cold formed steel sections under axial load. The web should be designed:

(1) to span between the fold line members as simply supported inclined sheeting under a load per unit plan area, w,

(2) to act as a continuous shear diaphragm over its length L.

The principles of calculation of the diaphragm girder are similar to those given in chapter 5 for sheeting spanning perpendicular to the span of a diaphragm assembly. For a uniform spacing of fasteners, the critical condition is near the end gable, but if the number of seam fasteners is varied along the length of the diaphragm, it could occur elsewhere. Both the in-plane strength and deflection of the roof slope should be calculated. The angle θ should normally lie within the range 35-45°.

10.3 *DESIGN EXPRESSIONS FOR STRENGTH*

10.3.1 Seam capacity

$$V_{ult} = \left(n_s F_s + \frac{2\beta_1}{\beta_3} F_p\right)\left(\frac{n_{sh}}{n_{sh} - 2}\right) = \frac{qL}{2}$$

where F_p is the design strength of an individual sheet/flange fastener (see table 5.1),

F_s is the design strength of an individual seam fastener (see table 5.1),

L is the length of the folded plate roof between gables,

n_s is the number of seam fasteners per side lap (excluding those which pass through both sheets and the supporting flange),

n_{sh} is the number of sheet widths in the length L of the diaphragm,

q is the factored in-plane distributed line load on each roof slope,

V_{ult} is the strength associated with a given failure mode,

β_1 is a factor to allow for the number of sheet/flange fasteners per sheet width,

β_3 see definition in 5.1.1.1.

10.3.2 Capacity of fasteners to end gables

$$V_{ult} = n_{sc}F_{sc} + 2F_p = \frac{qL}{2}$$

where F_{sc} is the design strength of an individual sheet/end gable fastener (see table 5.1),

n_{sc} is the number of sheet/end gable fasteners.

10.3.3 Design shear capacity

The design shear capacity V^* may be taken as the lesser of the values of V_{ult} in 10.3.1 and 10.3.2. It should then be checked that the capacity in other failure modes is greater than V^* as given in 10.3.4 to 10.3.6.

10.3.4 Sheet/fold-line member fasteners

In order to prevent failure of the sheet/fold-line member fasteners under combined wind uplift and shear, a 40% reserve of safety is included in the following expression. It should be checked that:

$$\frac{0.6bF_p}{p} \geq V^*$$

where b is the depth of the web,

p is the pitch of the web/flange fasteners,

F_p is defined in 10.3.1.

10.3.5 End collapse of sheeting profile

In order to prevent collapse or gross distortion of the profile at the end of the sheeting, the following limitations on shear force in the web should be observed:

Every corrugation fastened at the end of the sheeting:

$$0.9\ t^{1.5} b\ f_y / d^{0.5} \geq V^*$$

Alternate corrugations fastened at the end of the sheeting:

$$0.3\ t^{1.5} b\ f_y / d^{0.5} \geq V^*$$

where t is the net sheet thickness, excluding metalic and other coatings,

f_y is the design yield stress of the steel,

d is the pitch of the corrugations,

b is defined in 10.3.4.

10.3.6 Shear buckling

It should be checked that the reduced shear buckling strength of the sheeting, V_{red}, is adequate according to 5.4. The local shear buckling strength, V_ℓ, is as given in 5.4.2 and the global shear buckling strength, V_g, may be taken as:

For fasteners in every corrugation:

$$V_g = \frac{28.8}{b} D_x^{1/4} D_y^{3/4}$$

For fasteners in alternate corrugations:

$$V_g = \frac{14.4}{b} D_x^{1/4} D_y^{3/4}$$

where D_x and D_y are the orthogonal bending stiffnesses given by:

$$D_x = \frac{Et^3 d}{12(1-\nu^2)u}$$

$$D_y = \frac{EI}{d}$$

where E is the modulus of elasticity of steel,

I is the second moment of area of a single corrugation about its neutral axis (figure R.5.2),

u is the perimeter length of a complete single corrugation of pitch d (figure R.5.2),

ν is Poisson's ratio (for steel 0.3),

b, d and t are as defined in 10.3.5.

10.3.7 Fold-line members

The fold-line members should be taken to be fully laterally restrained by the sheeting and designed to carry 1.25 x the factored axial load. The design should be made in accordance with Eurocode 3 (Part 1 or Part 1.3).

10.4 *DESIGN EXPRESSIONS FOR DEFLECTION*

The design expressions for the central in-plane deflection of the diaphragm girder under factored load are given in table 10.1.

Table 10.1 Components of in-plane deflection of diaphragm girder

Deflection due to		Deflection (mm)
sheet deformation	sheet distortion	$\Delta_{1.1} = \dfrac{d^{2.5} K q L^2}{8 E t^{2.5} b^2}$
	shear strain	$\Delta_{1.2} = \dfrac{(1+\nu)[1+(2h/d)]qL^2}{4Etb}$
slip at fasteners	sheet to flange fasteners	$\Delta_{2.1} = \dfrac{s_p q L^2}{4b^2}$
	seam fasteners	$\Delta_{2.2} = \dfrac{s_s s_p (n_{sh} - 2) qL}{8(n_s s_p + \beta_1 s_s)}$
	fasteners to gable members	$\Delta_{2.3} = \dfrac{s_p s_{sc} qL}{2(2\beta_1 s_{sc} + n_{sc} s_p)}$
flange forces	axial strain in fold line members	$\Delta_3 = \dfrac{qL^4}{38.4 E A b^2}$
Total central deflection in plane of element		$\Delta = \Delta_{1.1}+\Delta_{1.2}+\Delta_{2.1}+\Delta_{2.2}+\Delta_{2.3}+\Delta_3$

In table 10.1, the symbols are as defined in 10.2 and 10.3 with the addition of the following:

- A is the semi area of a fold-line member,
- K is a sheeting constant (see 5.2.1.2 and table 5.6 and 5.7),
- h is the height of the sheeting profile,

s_p is the slip per sheet/flange fastener per unit load (see table 5.1),

s_s is the slip per seam fastener per unit load (see table 5.1),

s_{sc} is the slip per sheet/end gable fastener per unit load (see table 5.1).

The sum of the component deflections in table 10.1 gives the total in-plane central deflection Δ of the diaphragm girder under factored load. The vertical central deflection of the folded plate roof Δ_v (figure R.10.3) is given by:

$$\Delta_v = \Delta\ cosec\theta$$

The deflection under unfactored load should conform to the general serviceability limit states given in Eurocode 3.

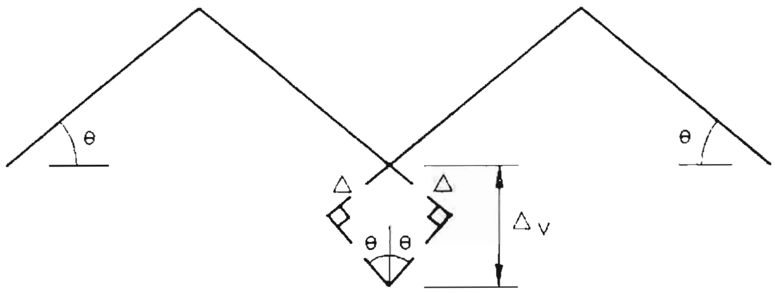


Figure R.10.3: Vertical deflection of a folded plate roof.

11. DESIGN BY TESTING

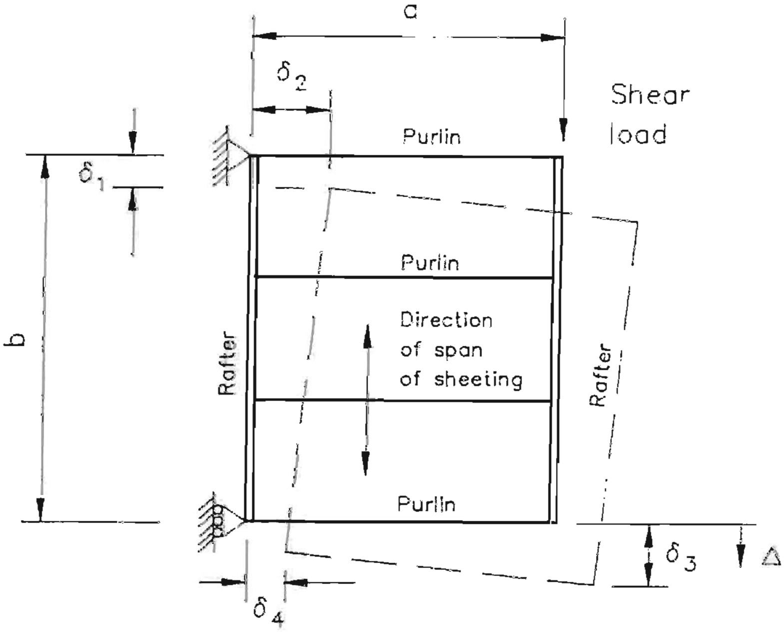
11.1 *GENERAL*

Testing of shear diaphragms may also be carried out in order to:

- verify results of new analytical approaches;
- strengthen existing calculation methods;
- study the effect on the shear strength caused by e.g.:
  - shear panel configuration,
  - shear panel span,
  - type and arrangement of supporting members,
  - type and arrangement of fasteners,
  - material thickness and strength;
- study the effect on the shear stiffness caused by e.g.:
  - deformation of sheeting,
  - slip at the fasteners,
  - axial forces in the edge members,
  - local effects at details and connections.

11.2 *TEST CONDITIONS*

A suggested arrangement for testing a shear diaphragm is shown in figure C.11.1.



$$\Delta = \delta_3 - \left[\delta_1 + \frac{a}{b}(\delta_2 - \delta_4)\right]$$

Figure C.11.1: Shear panel with or without purlins.

11. DESIGN BY TESTING

11.1 *GENERAL*

Testing of steel shear diaphragms is required if the load bearing behaviour at serviceability and/or ultimate state - characterized by e.g. diaphragm stiffness, failure modes or load carrying capacity - cannot be determined on an analytical basis.

Testing of steel shear diaphragms may also form the basis for design, if it is desired to develop shear diaphragm systems of clearly defined components and/or to build a number of similar structures.
Tests should in the main confirm:

- the general structural behaviour,
- the strength against the required factored load,
- the ultimate capacity and mode of failure,
- the stiffness of the shear diaphragm at the serviceability state.

Strength and slip values of the fastenings to be used in the calculation procedures of these Recommendations can be estimated by shear tests according to the European Recommendations for "The design and testing of connections in steel sheeting and sections" (see reference [5]).

11.2 *TEST CONDITIONS*

The test set up for the diaphragm as a whole should be representative of the real structure and the test load should be applied in a manner representing the actual service conditions as closely as is practicable.

Separate tests on structural details used in a stressed skin diaphragm should be executed in a way which is compatible with the conditions in a shear diaphragm.

Separate tests on connections should be carried out according to the relevant standard or appropriate rules according to the European Recommendations for "The design and testing of connections in steel sheeting and sections" (see reference [5]).

Alternatively a one-span beam-test with two point loads may be carried out.

It should be noted that, in most structures, load is applied to the sheeting through rafters or supporting members which are themselves eccentric to the sheeting. The test arrangment should reflect the actual eccentricities in the prototype structure.

The shear load may usually be applied by a jack. If testing under combined loads is required, the loads normal to the plane of the sheeting may be applied by means of weights (downward load) or by means of jacks or pressure bags (upward load) provided these do not offer any restraint to the shear mode.

11.3 *TEST PROCEDURES*

The purpose of the preliminary loading is to find the deflection at the characteristic load. At least 3 increments are required to ensure that the behaviour is not markedly non-linear, and a time of 15 minutes is specified to observe if any creep occurs. The loading should then be removed.

At each of the above stages, the deflections should be measured. After removal of the loading, the recovery of deflection should be at least 80%. If this requiremts is not satisfied, the preliminary test should be repeated and the recovery of deflection should be at least 90%

It is possible that, due particularly to "bedding down" of the fasteners, the recovery of the diaphragm after the initial loading may not reach 80%. Although this may not be serious, it is required in this case that the test be repeated and that the recovery should then be not less than 90%. This requirement ensures that the "bedding down" occurs only once.

In a test on a shear panel, the test arrangement should be representative of the real structure. Prior to the sheeting being fixed, the test frame should be subjected to a preliminary test, up to a deflection in excess of the value expected in the test, in order to verify that it has negligible stiffness.

Provision should be made for measuring the body rotation of the test rig so that the true shear deflection Δ of the shear panel may be obtained.

11.3 *TEST PROCEDURES*

Prior to the test the component may be bedded down by loading to a value not exceeding serviceability loading and removing the load. Deflections should be measured during this loading, but they need not be included in a final assessment of the test.

In the test, the component should be loaded in regular increments (not less than five) and the component examined for signs of distress at each increment. A running plot should be maintained of the load against principal deflection. When this indicates significant non-linearity then the load increments should be reduced in magnitude.

a) Acceptance Test

This test is intended as a non-destructive test for confirming structural performance.

This load, equal to serviceability loading x 1.20, should be maintained for 15 minutes. There should be no undue distortion of the component at this loading. The load should be removed in decrements, and the residual deflection should not exceed 20% of the maximum recorded. If this is not achieved, then the test may be repeated once only and the residual deflection should not then exceed 10% of the maximum recorded in the repeat test.

b) Strength test

This test is used to confirm the calculated capacity of a component or structure. Where a number of items are to be constructed to a common design, and one or more prototypes are tested to confirm their strength,

the others may be accepted without further tests provided they are similar in all relevant aspects to the prototype.

Before carrying out the strength test, the specimen should first be submitted to and satisfy the acceptance test described in 11.3 a).

The component or structure should be reloaded, and at a load equal to the calculated capacity of the component, the load should be maintained for 15 minutes. There should be no gross distortion of the component at this loading. The load should be removed in decrements and the residual deflection should not exceed 80% of the maximum recorded.

c) Test to failure

It is only from a test to failure that the real mode of failure and true capacity of a component or structure can be determined. Where the structure is not required for use in service, it may be advantageous to secure this additional information after a strength test.

Alternatively the objective may be to determine the true design capacity from the ultimate test capacity. In either case, the specimen should first satisfy the strength test described in 11.3 b).

During a test to failure the loading should first be applied in increments up to the strength test load. Consideration of the principal deflection plot should determine the subsequent load increments.

Provided that there is a ductile failure the design capacity of a similar component or structure may be determined from:

$$\text{Design capacity} = K_t \text{x} \left(\frac{\text{design yield strength}}{\text{averaged yield strength in test}} \right) \text{x ult.test load}$$

In the case of a sudden failure, the averaged yield strength should be replaced by 1.2 x the averaged yield strength.

For a single test K_t should be taken as 0.9 unless the resulting design capacity falls below the design capacity confirmed by the strength test, when the latter should be taken. For two or more related tests K_t should

11.4 *FUNCTIONAL REQUIREMENTS*

Paragraph (1) satisfies the requirements of the limit state of deflection.

Paragraphs (2), (3) and (4) satisfy the requirements of the limit state of strength and at the same time ensure that the deflections at the test load are not so large as to cause complete collapse of diaphragm.

Paragraph (4) specifies that the residual deflection of the diaphragm should be less than 80% of the deflection under the strength test load. This ensures that the structure still has a degree of stiffness at the test load.

Paragraph (5) ensures that a primary sudden failure of the connections is avoided by an adequate γ_M - value.

be taken as 1.0 provided that the lowest of the individual ultimate test loads is used.

d) **Shear flexibility**

The design value of shear flexibility of the component, where applicable, should be obtained from the graph of load against deflection as follows:

$$\text{Shear flexibility } c = \frac{\Delta}{0.6\ V_d}$$

where Δ is the mean deflection at a load of 0.6 V_d,

V_d is the design capacity.

11.4 *FUNCTIONAL REQUIREMENTS*

(1) The deflection of the diaphragm in the preliminary test under unfactored loads should not exceed the permissible value. There should be no signs of damage or distress of any kind at this load.

(2) The strength test load specified in clause 11.3 b) should be carried without failure occuring in any part of the diaphragm.

(3) Under the strength test load, the deflections and deformations of the diaphragm or any part should not be of such an extent as to render it unusable.

(4) The recovery of deflection after removal of the strength test load should be at least 20%.

(5) The calculated load carrying capacity of the sheet-purlin connections should be at least 40% higher than the capacity of the other components. For other connections the load carrying capacity should be 25% higher.

11.5 *TEST REPORT*

The Test Report shall also contain all necessary information about fasteners and connections, which may be relevant for a proper evaluation of the test results (e.g. type and material data of fasteners, torque-moment, design and strength values etc.).

11.5 *TEST REPORT*

A report should be prepared for all tests giving the following information:

a) Evidence that the performance of the work is in agreement with the requirements;

b) Sufficient diagrams to indicate the form of the structure or component, position of loading points and location of measuring gauges;

c) The method of loading;

d) Dimensional measurements made on the structure or component;

e) Deflections and any strains measured during the loading and unloading, and the time into the test;

f) A record of observations during the test;

g) An assessment of the load capacity of the structure.

12. WORKMANSHIP AND ERECTION

12.1 *GENERAL REMARKS*

Unlike trapezoidal sheets, cassettes or sandwich panels are normally used only as cladding elements and it is not evident that they are used intentionally as load carrying elements in the case of shear diaphragms. Care must be taken to ensure a correct flow of information. The idea of using these sheets as part of the main structure and/or for limitation of deflections must be known on site as well as by the final customer.

The following recommendations should be a guideline to ensure this flow of information.

12.2 *DRAWINGS AND DESIGN*

To avoid loss of information between the designer and the erection team, the quality of the drawings are of utmost importance.

12. WORKMANSHIP AND ERECTION

12.1 *GENERAL REMARKS*

When using trapezoidal sheets (or cassettes or sandwich panels) as stressed skin sheets in the design of buildings, these elements should be regarded as part of the main load carrying structure and therefore attract the same accuracy and care in design and erection as all other parts of the main structure.

To reach and ensure an adequate quality, it is necessary that:

- the designers,
- the erection team,
- the site supervision,

have appropriate qualifications for their respective jobs.

It is important in this context to transmit the intention of the design engineer to all other people that are engaged with the erection of the building and to ensure that the instructions of the designer are strictly followed. The design engineer is responsible for transmission of information.

12.2 *DRAWINGS AND DESIGN*

The use of stressed skin design should be dependent upon competent calculations and adequate drawings.

The drawings should specify :

- the statical system, particularly the exact definition of which parts are used for stressed skin design,
- the type and situation of the substructure, particularly the kind of shear connection,
- the type, thickness and lay out of the sheets,
- the order (sequence) of laying the sheets,
- the seam and sidelap joints with exact specification of the type and the location of the fasteners and washers,
- the form of the edge and the shear connector fasteners with exact specification of the type and location of the fasteners and washers,

The temporary staying of the structure must kept in place until full completion of the sheeting, including installation of all fasteners.

- all openings and necessary framing with exact specification of the fasteners.

For the purpose of documentation, the drawings should contain a note concerning the intention of the use of stressed skin sheets (e.g. whether the stressed skins are used for control of deflections only or for carrying loads) and a warning that any additional openings should be cleared with the designer prior to installation (see clause 12.5).

Additional specifications may deal with:

- the installation of the fasteners (e.g. drill diameter, torque for screws, ...) if the erection team is not known or does not have general installation rules for all types of fasteners used on site,
- necessary provisions to ensure the stability of the construction before the stressed skin elements are effective.

12.3 *TECHNICAL EXPERTISE OF THE ERECTION TEAM*

The trapezoidal sheets used as a diaphragm must be regarded as normal profiled steel sheets.

They must be handled, stacked and erected in accordance with the European Recommendations "GOOD PRACTICE IN STEEL CLADDING AND ROOFING" (ref. [31]).

In a structure where the presence of trapezoidal sheeting has been taken into account in the design, particular care should be taken on the following:

- The sheets should be installed by firms which possess adequate experience in the installation of roof cladding and employ adequate expert staff. Supervisory staff responsible for installation work on site should be sheet metal workers experienced in roofing or similar trades. They must be fully conversant with the methods used to install steel cladding and with the particular requirements for satisfactory diaphragm action.

- Stressed skin structures may only be erected by firms employing a qualified jobsite supervisor who will be responsible for the stability of the structure at all stages of construction.

12.4 *RECEPTION OF MATERIALS*

12.4.1 General

Example of document for profiled sheet:

Manufacturer : NAME
ADDRESS

Identification : - Corrugation : height : 40 mm
width : 333 mm
- Cover width : 1 m
- Thickness : 0,75 mm
- Raw material :
galva 275 gr/m² } face A
polyester primer 5 μ } face A
polyester 20 μ } (external)

polyester primer 5 μ } face B (internal)

- Yield stress : min. 320 N/mm²
- Specification : 5 x 6000 mm (tol.: -0/+5mm)
4 x 5950 mm ...

- Appropriate tools should be used for both the fixing of the sheeting as well as erection of the structure.
  Particular attention should be given to the necessity to predrill holes of adequate diameter, in accordance with the manufacturers recommendations. Fastenings into concrete and fasteners through insulation should be made strictly in accordance with the stipulated installation drawings.

  All special tools used in the installation of load bearing components, including sheeting, should be used strictly in accordance with the manufacturers instructions and should be properly calibrated.

12.4 *RECEPTION OF MATERIALS*

12.4.1 General

To ensure a sound and workmanlike installation of profiled steel sheets, their shape and dimensions should be carefully checked.

All components, including profiled steel sheets and fasteners, insulation and accessories, should be clearly identified by suitable markings.

Delivery documents should be available at the site and should include the following data:

- manufacturer,
- identification of component,
- dimensions (if necessary, drawings),
- relevant material data (e.g. yield stress).

If the materials do not comply with valid standards or type approvals, copies of the necessary test certificates should be attached to the delivery documents.

12.4.2 Tolerances in profiled steel sheets

Pending the availability of a suitable European standard (for structural sheeting a European standard is in preparation by CEN TC/135 WG 6 and for non-structural sheeting prEN 508-1 is available), National standards may be followed.

12.4.4 Additional components

These requirements are so that adequate checking may be carried out on reception at site.

12.4.3 Fasteners

Requirements for fasteners, including corrosion protection are given in chapter 3.
Independently certified strength values for the materials to be connected should be available.

12.4.4 Additional components

Any additional components, e.g. insulation, ridge members, gutter members, expansion joints, flashings, etc. which influence the strength and stiffness of the structure must be shown on the installation drawings and must be described by adequate details.
All components should be checked against details given on the drawings and bill of materials.

12.5 *SITE SUPERVISION*

12.5.1 Site supervision

The supervisor responsible for the project must be on site during the erection. He should have a jobsite note book and write down all deviations from his understanding of his instructions and clear these with the designer.

This procedure is equally valid for all corrective actions or alterations he has to carry out (like added openings) for which he must seek proper written approval by the designers.

At the end of project, the note book will be basis of the final erection report which should be copied to the designer and the general control.

12.5.2 Start up meeting

The jobsite supervisor should call a meeting with his crew before erection commences.
This meeting should cover the following:

a. Erection drawings
These should have been previously studied by the jobsite supervisor and should be clearly understood by him. Any question

12.6 *INSPECTION AND MAINTENANCE*

12.6.1 General

A yearly inspection is desirable; intervals between inspections should not exceed two years. Preferably a maintenance contract should be concluded at the time the building is handed over by the general contractor to the client. If this is not done, the client should be given a full inspection and maintenance guide.

arising should be cleared with the designers when the complexity of the structure so requires.

The jobsite supervisor should explain the sequence of erection, draw attention to the different types of sheeting used on the job, clearly identify the fastening procedure and show samples of the fasteners to be used at each location.

He should explain the installation of accessories and roof openings with their eventual reinforcements as well as all other pertinent details to the project.

b. Erection procedure

It should be specified at which location and in what quantity the sheeting bundles may be put on the roof. This information should be double checked with the designer.

The supervisor should be informed if temporary planks are necessary to walk on the roof sheeting in the unfastened condition (i.e. the last sheet laid out), together with any other safety requirements.

The supervisor should explain the correct use of drilling and screwing equipment including such factors as the correct drill diameters of self tapping screws and recommended speed of screwing. Eventual sealing operations should be detailed.

12.6 *INSPECTION AND MAINTENANCE*

12.6.1 General

When stressed skin design is used, the sheeting and supports should be maintained in their initial condition.

12.6.2 Inspection

It is essential to inspect the following (others may be relevant on a job to job basis):

- good condition of the sheets (especially when they are hidden by built up insulation), 12.6.3 **Maintenance**

Debris of any nature around exhausts, at the drainage, etc. should be removed carefully and the surface cleaned with water and mild detergent, with a soft bristle brush. If the coating has been attacked by the debris, the sheet supplier should be contacted for advice.

In the case of built up roofs with bituminous weather proofing, it is essential that the bitumen skin is effective. Any repairs should be done after consultation with the supplier. Eventual repair work should be carried out by qualified personnel.

- integrity of the fasteners,
- integrity of the underlying structure,
- condition of the drainage system (very often the starting point of problems if not regularly cleaned),
- condition around exhausts, ventilation systems, roof penetrations .

12.6.3 Maintenance

Repair work becomes necessary most often due to neglected maintenance. The necessary and sufficient maintenance programme should be made available by the sheet supplier.

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Annex A. INFORMATION AND CHECKLIST FOR DECISIONMAKERS TO ESTABLISH THAT DIAPHRAGM ACTION IS POSSIBLE

I. What is diaphragm action

The roof sheeting in a rectangular building under horizontal loads behaves like the web of a deep plate girder spanning between vertical planes of support. In this plate girder the longitudinal edge members behave as flanges, carrying the axial tension and compression. The girder helps resist horizontal loads. In a pitched roof building the roof slopes help to resist both horizontal and vertical loads.

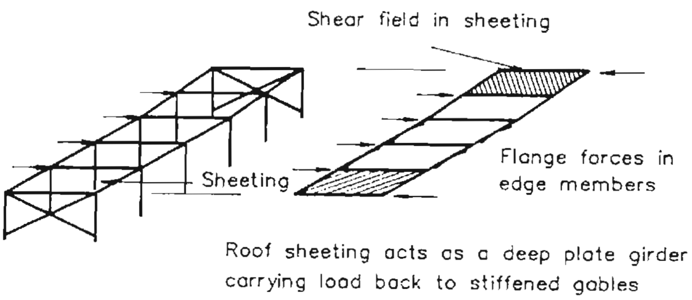


Figure A.1: Diaphragm action in a flat roof on hinged columns.

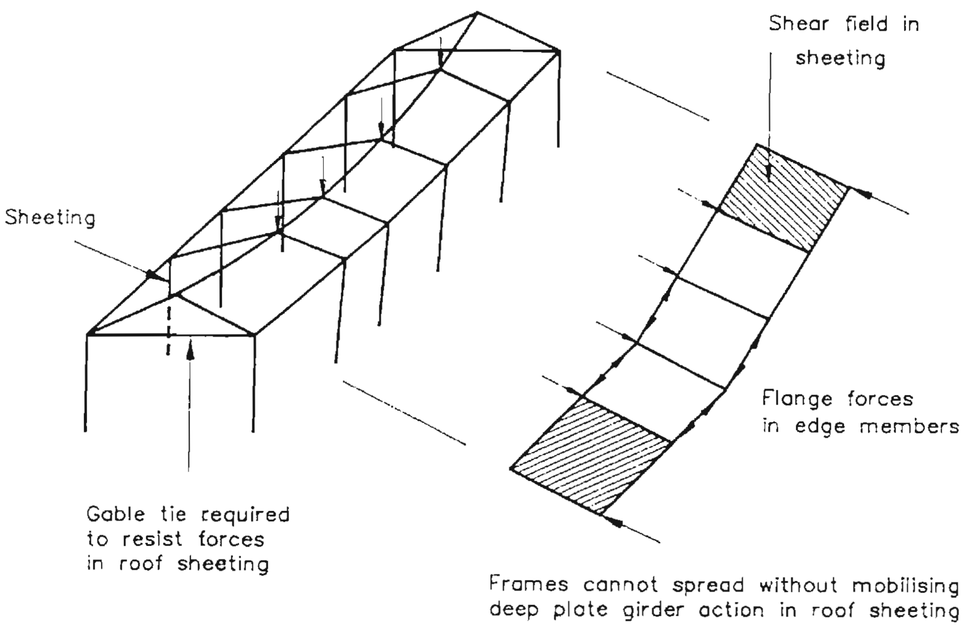


Figure A.2: Diaphragm action in a pitched roof.

Annex A.2

A fundamental element in the theory of diaphragm action is the individual shear panel. A shear panel may include some or all of the following components:

(1) Individual lengths of profiled steel sheeting or decking;
(2) Purlins or secondary members perpendicular to the direction of span of the sheeting;
(3) Rafters or main beams parallel to the direction of span of the sheeting;
(4) Sheet/purlin fasteners;
(5) Seam fasteners between individual sheet widths;
(6) Shear connectors to provide attachment between the rafters and sheeting;
(7) Sheet/shear connector fasteners;
(8) Purlin/rafter connection.

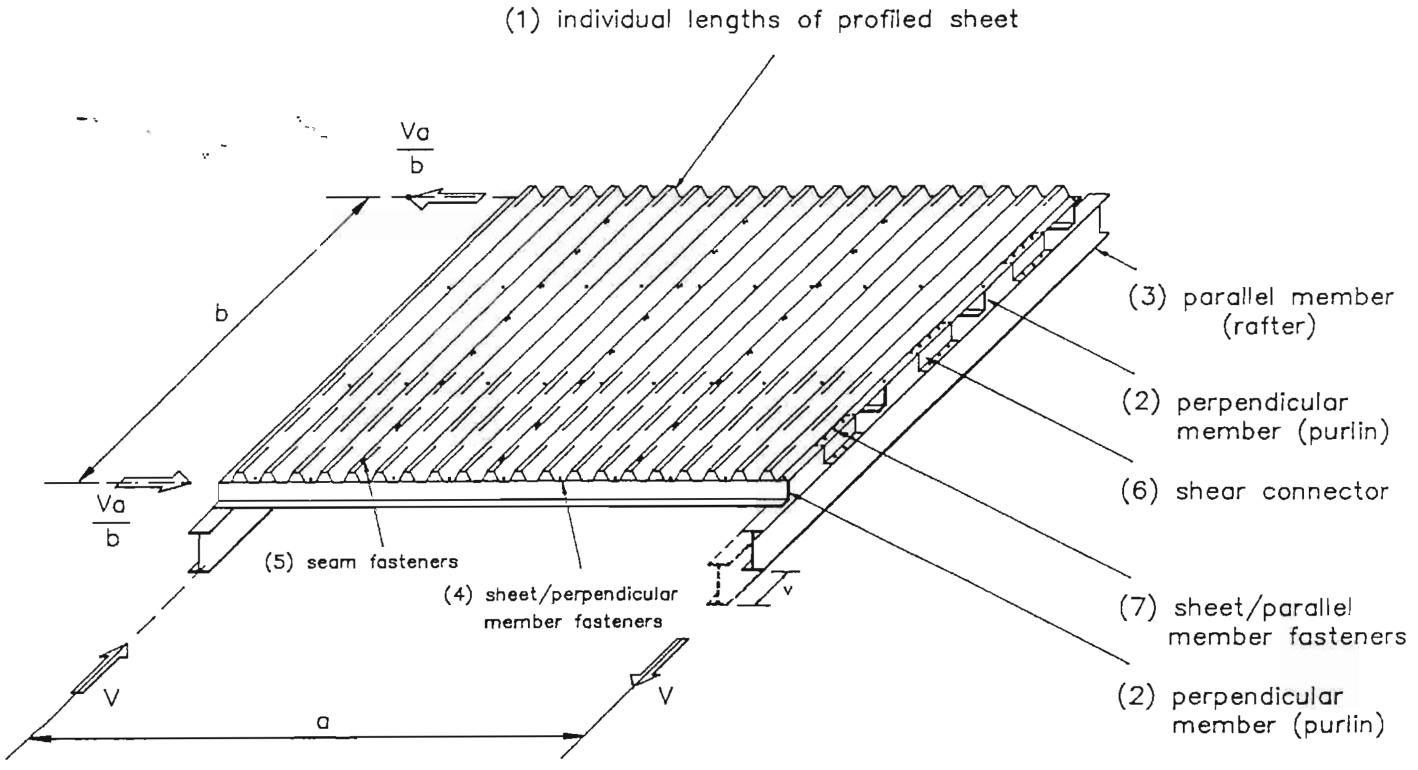


Figure A.3: Individual shear panel.

Stressed skin design quantifies logically and safely the actual stresses and deflections which will occur in a sheeted building.

If the effect of sheeting is ignored, the calculated behaviour of a building may be quite unrealistic and the sheeting itself may be overstressed simply because it is so stiff and attracts a high shear load.

II. When to make use of diaphragm action and when it will occur

II.1 When to make use of diaphragm action

It is meaningful to make explicit use of diaphragm action:

* when with only a few erection provisions the use of permanent bracings can be prevented;
* to provide stability to members to prevent flexural buckling, flexural-torsional buckling and lateral-torsional buckling or a combination of these;
* explicit structures like pitched roofs, folded plate roofs, hyperbolic paraboloid shells etc.

It is less meaningful to make explicit use of diaphragm action (although deformation capacity is still necessary):

* for complex roof plans and when there are no paths to transmit forces to the foundation via walls, facades or bracings;
* for roofs with large openings situated in such a way that separate diaphragms cannot be developed;
* for walls and facades which may not remain permanently during the lifetime of the building.

II.2 When diaphragm action will occur

a. *A steel structure with bracings for stability and with sheeting only for covering.*

The steel structure with bracings should be designed to carry the total loads. Provided that the sheeting possesses sufficient deformation capacity, the diaphragm can follow the deformation of the steel structure without problems arising.

Sufficient deformation capacity may be assumed when fastenings fulfill the requirements of clause 3.4 and a maximum distance of the seam fasteners of 500mm.

b. *A steel structure (flat or pitched roof) with sheeting.*
In this situation the columns are hinged.

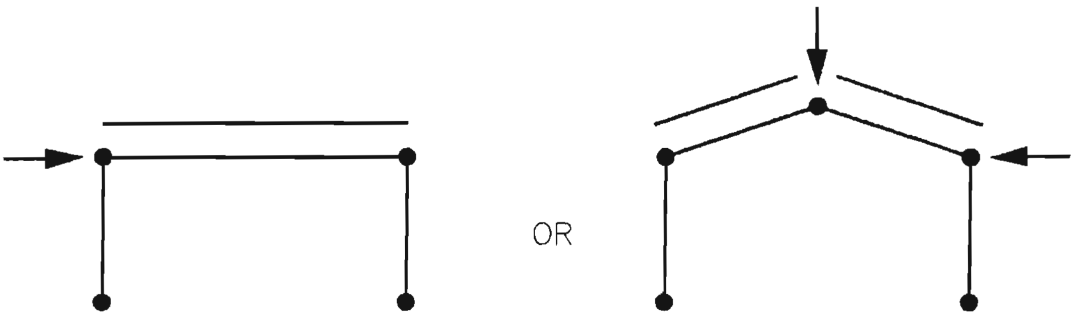


Figure A.4: Steel structure with sheeting and hinged columns.

The task of the bracings has been replaced by diaphragm action of the sheeting. In this situation it should be proved explicitly that the diaphragm can fulfill this function. In this case during erection temporary bracing is often necessary. This will therefore not usually lead to a saving in cost of the wind-bracing.

c. *A steel structure of rigid frames with sheeting.*

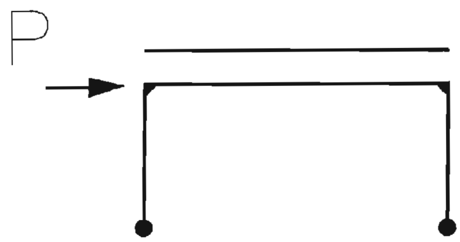


Figure A.5: Steel structure of rigid frames with sheeting.

From a design point of view there are two possibilities:
- The sheeting should be regarded as part of the structure so that the external load will be taken partly by the steel structure and partly by the diaphragm. The proportion depends on the ratio of the stiffness of frames and diaphragm. Both the frames and the diaphragm should be designed for these forces.

- The sheeting should only be regarded as a covering so that the frames should be designed for the total load, which leads to heavier sections. The sheeting should be capable of following the deflections of the frames, which means that the diaphragm must possess sufficient deformation capacity.

d. *The sheeting used as stability bracing.*

Bracing against lateral-torsional buckling

Chapter 9 of these recommendations provides procedures to prove this capability. For European hot rolled sections with a height less than 200mm and with proper connections, a check is not necessary. For the interaction of sheeting and purlins or side-rails, Eurocode 3 Part 1.3 provides a procedure.

Bracing against buckling

This situation can occur, for example at the upper chord of a truss or a column in a wall. Chapter 9 provides procedures to prove this capability.

The element which is braced against lateral-torsional buckling or buckling forms, together with the diaphragm, a stable structure in itself. Special bracing to the foundation is not necessary.

e. *Types of structure only possible thanks to diaphragm action.*

- **Folded plate roofs.**

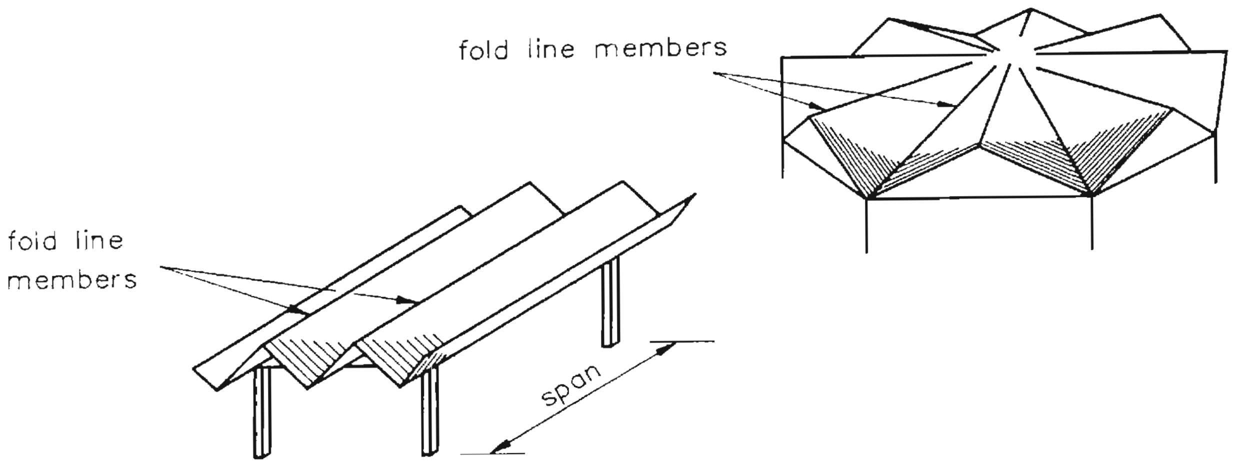


Figure A.6: Folded plate roofs with rectangular or circular plan.

In these situations every roof face is a beam with the ridge and eaves as flanges and the sheeting as a web.

- Hyperbolic paraboloid shells.

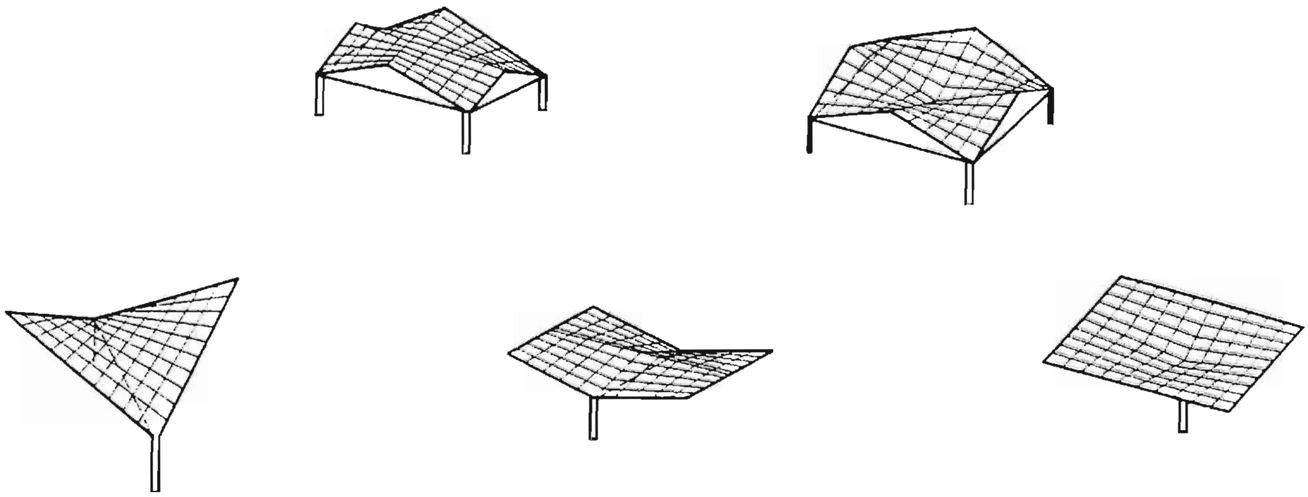


Figure A.7: Hyperbolic paraboloid shells.

These types of structure can only be formulated thanks to the fact that corrugated sheeting can be easily twisted as long as the sheeting is not fastened to the substructure. When all elements have been fastened the capability of diaphragm action is present.

III. <u>Important aspects with regard to diaphragm action</u>

III.1 Aspects with regard to the design.

a. In the case of explicit application of diaphragm action, the design should take account of this from the beginning.

b. With regard to fastenings:

- the spacing of the seam and edge element fasteners should be not exceed a maximum limit; in the absence of national rules this should be 500mm;
- the fastenings should possess sufficient deformation capacity. In general this will be the case when:
  . the main or edge element fastenings consist of screws (typically ϕ 5.5mm to ϕ 6.3mm) or shot pins,
  . the seam fastenings are made of self-drilling screws (typically ϕ 4.1mm to ϕ 4.8mm) or blind rivets (typically ϕ 4.8mm) of monel metal or stainless steel.

c. The main fastenings should be made in the troughs of the sheeting.

d. In some cases it is necessary to apply shear connectors at gables or intermediate rafters.

e. Edge members should always be designed to take axial tension and compression forces.

f. There should be provision to transmit the diaphragm forces to the foundation.

g. The diaphragm can also be used to carry horizontal loads from light cranes. This is allowed when the load in the diaphragm caused by the crane is less than 30% of the failure load of that diaphragm.

h. On drawings and in calculations it should be clearly marked that diaphragm action has been used in the design of the structure.

i. With regard to rebuilding in buildings which are stabilised by diaphragms, these buildings should have clear notices saying that the structure is designed on these principles.

III.2 Aspects with regard to the calculation.

a. In the case of openings distributed over the area of the diaphragm to an amount of less than 3%, these openings may be neglected in the calculation. If the openings are less than 15% of the area of the diaphragm, they should be calculated explicitly. If more than 15% of the area of the diaphragms are openings, than the whole diaphragm should be regarded as a number of smaller ones.

b. The mean shear stress in a shear panel caused by the design load should be less than 25% of the design value of the yield stress of the sheeting.

c. The strength of the diaphragm should be based on a ductile failure mode. The expressions in chapter 5 are designed to ensure this is the case.

III.3 Aspects with regard to the erection.

a. The position of fasteners should be in accordance with the design drawings.

b. If there are any openings not indicated on the drawings, the structural consequence should be proved explicitly.

c. By making use of shear panels, the sheeting becomes an important structural element in the design, erection and maintenance of the building. The supervision at site should be fully aware of this.

d. The stability of the structure (steel framework and sheeting) during erection requires special attention.

IV. The way to calculate

1. Design the sheeting (and supporting members) for its primary purpose as cladding.
2. Determine the in-plane loads on the sheeting and the shear force and maximum bending moment in the deep plate girder.
3. Calculate the shear flexibility and the ultimate shear strength (based on the fasteners) of a shear panel.
4. Check that:
   * the combined stresses in the supporting members are acceptable,
   * the ultimate shear strength of a shear panel is adequate,
   * the in-plane deflection of the diaphragm is acceptable.

Annex B. SHEAR PANELS ACTING AS STABILITY BRACING FOR BEAMS AND COLUMNS

I. General

In chapter 9 the possibility has been given to use shear panels as stability bracing (to prevent flexural, flexural-torsional and lateral-torsional buckling or a combination of these). In annex B section II the procedure has been given for checking the stabilised state of columns and beams made from doubly symmetric I-sections.
The determination of the minimum strength and stiffness of a shear panel which should be available according to this checking procedure has been given in annex B section III.

The formulae in Annex B are based on a sinusoidal form of lateral deflection, which leads to a concentrated reaction force at the support (see figure B.1). The concentrated force can be regarded as an internal force in the diaphragm provided the fasteners, between the sheeting and the element to stabilise, over 1/8 of the span of the element to stabilise can sustain this concentrated force. In these fasteners are unable to sustain this concentrated force, then that force should be introduced into the edge members via the fasteners between the diaphragm and the edge members.

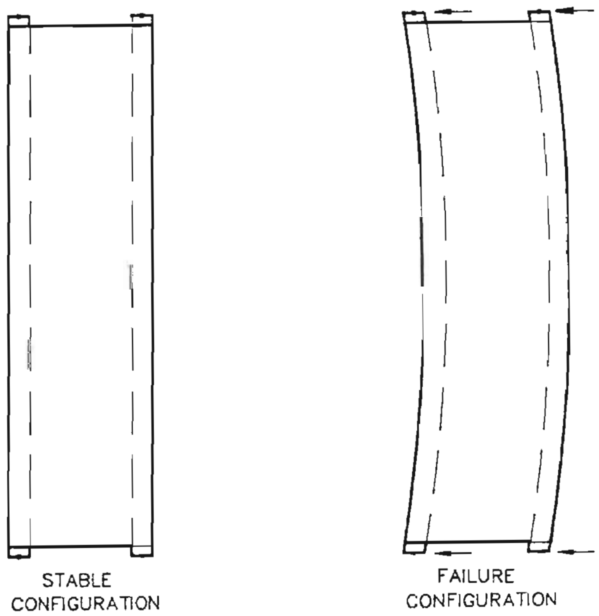


Figure B.1: Assumed deflection of the element to stabilise with the resulting concentrated force at the supports.

II. Determination of M_{cr} and N_{cr} for doubly symmetrical I-sections used as beams or columns and stabilised by a diaphragm

According to chapter 9 shear panels can be used as stability bracing for beams and columns. It is most efficient if the unstiffened flange of the beam is in tension. In this case the member is fully stiffened if the shear stiffness of the diaphragm (S) fulfills the following requirement:

$$S_{act} > S_y = \frac{f_y \cdot A}{2}$$

where S_{act} shear stiffness of the diaphragm according to section III,

S_y required shear stiffness of a diaphragm for fully stabilising a member,

f_y yield stress of the member to be stabilised,

A cross-section area of the member to be stabilised.

If the requirement for full stabilisation is not fulfilled (which is the case when $S_{act} \leq S_y$), the critical section forces M_{cr} and N_{cr} (these are the input parameters for a calculation according to Eurocode 3 part 1.1) follow from:

$$M_{cr} = \psi \cdot M$$

$$N_{cr} = \psi \cdot N$$

where ψ eigenvalue being the lowest positive value of ψ_1 and ψ_2 (if both eigenvalues are negative, no stability problem exists for the given stress resultants M and N)

$$\psi_{1,2} = -\frac{k_1}{2 \cdot k_2} \pm \sqrt{\left(\frac{k_1}{2 \cdot k_2}\right)^2 - \frac{1}{k_2} \cdot \left[W_z \cdot W_w - \left(S \cdot \frac{h}{2 i_p}\right)^2\right]},$$

S = the minimum of S_{act} or S_y

$$k_1 = -\left[N \cdot (W_z + W_w) - M \cdot \frac{S \cdot h}{i_p^2}\right],$$

$$k_2 = \left[N^2 - M^2 \cdot \frac{1}{i_p^2}\right],$$

$$W_z = -(\frac{EI_z \cdot \pi^2}{L^2} + S),$$

$$W_w = -\frac{1}{i_p^2} \cdot [\frac{EC_w \cdot \pi^2}{L^2} + GI_T + S \cdot (\frac{h}{2})^2],$$

A cross-section area of the member to be stabilised,

h height of the member to be stabilised,

$i_p^2 = i_y^2 + i_z^2$,

$i_y^2 = I_y/A$,

$i_z^2 = I_z/A$,

E modulus of elasticity of the member to be stabilised,

G shear modulus of the member to be stabilised,

I_y moment of inertia of the member about the axis parallel to the plane of the shear panel,

I_z moment of inertia of the member about the axis perpendicular to the plane of the shear panel,

I_T St. Venant torsion constant; for a doubly symmetric I-section:

$$I_T = \frac{2bt_f^3 + (h - 2t_f)t_w^3}{3},$$

b : flangewidth of I-section,

t_f : flangethickness of I-section,

h : height of the I-section,

t_w : webthickness of I-section,

C_w warping constant; a good assumption for a doubly symmetric I-section is: $C_w = 1/4\ I_z h^2$,

L length of the member to be stabilised,

M constant bending moment about y-axis to be introduced with the actual sign (for definition see figure B.2),

N constant concentric normal force to be introduced with the actual sign (for definition see figure B.2).

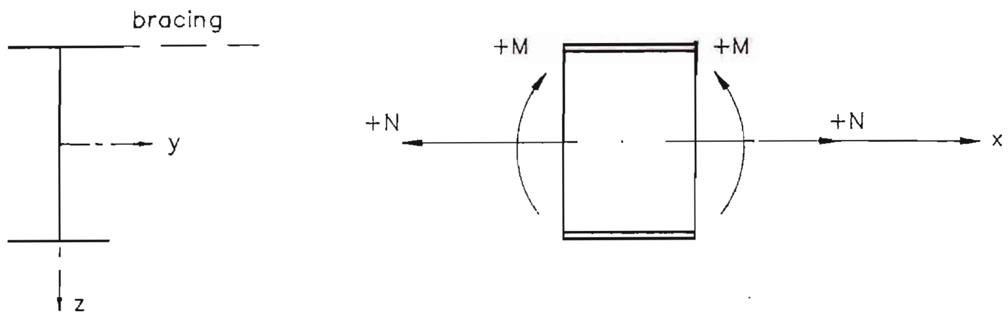


Figure B.2: Notations and sign definitions.

The maximum shear force (T_d) in a shear panel per member to be stabilised follows from:

$$T_d = \frac{\psi}{\psi - 1} \cdot S \cdot \frac{e_o}{L}$$

where e_o horizontal predeformation of the stiffened flange according to chapter 5.2.4.4 of Eurocode 3, part 1.1 (see reference [1]),

S the minimum of S_{act} or S_y.

The other symbols are as before.

The maximum shear force T_d shall be smaller than P_{max} according to section III.

Furthermore one of the following conditions a. or b. shall be fulfilled:

a. The force T_d shall be smaller than $\frac{L}{8 \cdot p} \cdot F_p$

where F_p is the design strength per fastener between the sheeting and the stabilised element,

L is the span of the element to stabilise,

p is the pitch of the fasteners between the sheeting and the stabilised element.

b. The fastening of the sheeting to the edge elements shall be accounted for T_d.

III. Determination of strength and stiffness of a shear panel acting as a stabilising element

When a shear panel has been used as a stabilising element (to prevent flexural, flexural-torsional and flexural-lateral buckling or a combination of these) the stiffness perpendicular to the section, which has to be stabilised, should be known.

From the assumed calculation model (see figure B.3) the following can be determined:

- the shear stiffness of the shear panel,
- the shear strength of the shear panel.

In III.1 and III.2 the shear stiffness and shear strength will be given for respectively a shear panel on purlins and a shear panel on rafters.

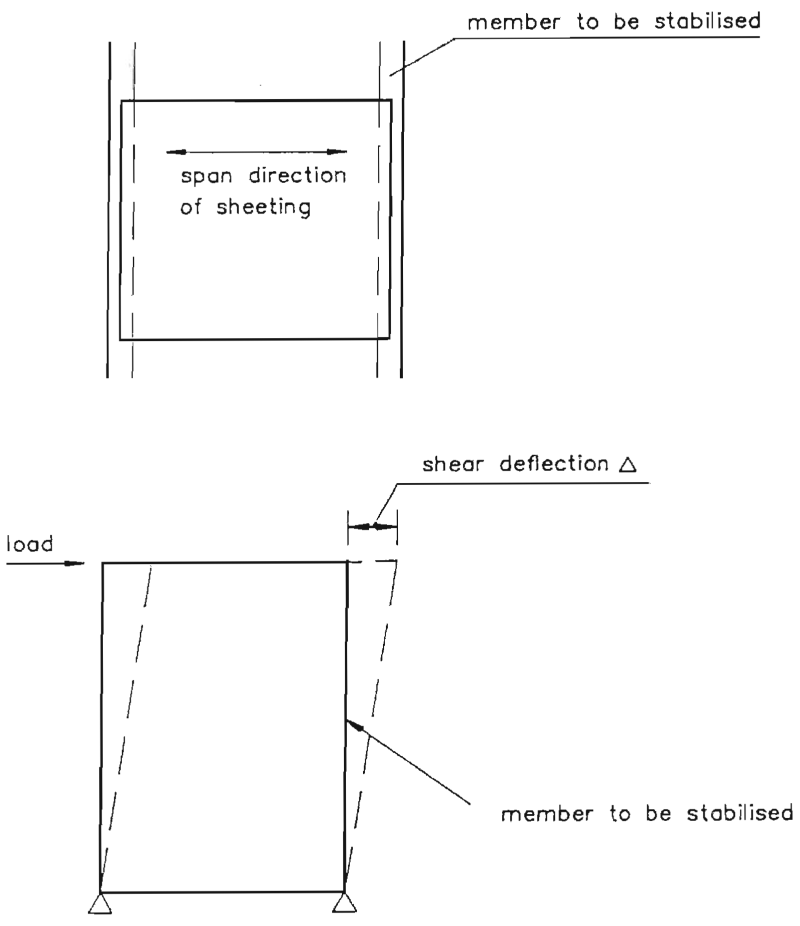


Figure B.3: Schemes for the calculation model of strength and stiffness of a shear panel to stabilise sections loaded in bending and compression.

III.1 Shear stiffness and shear strength of a shear panel on purlins.

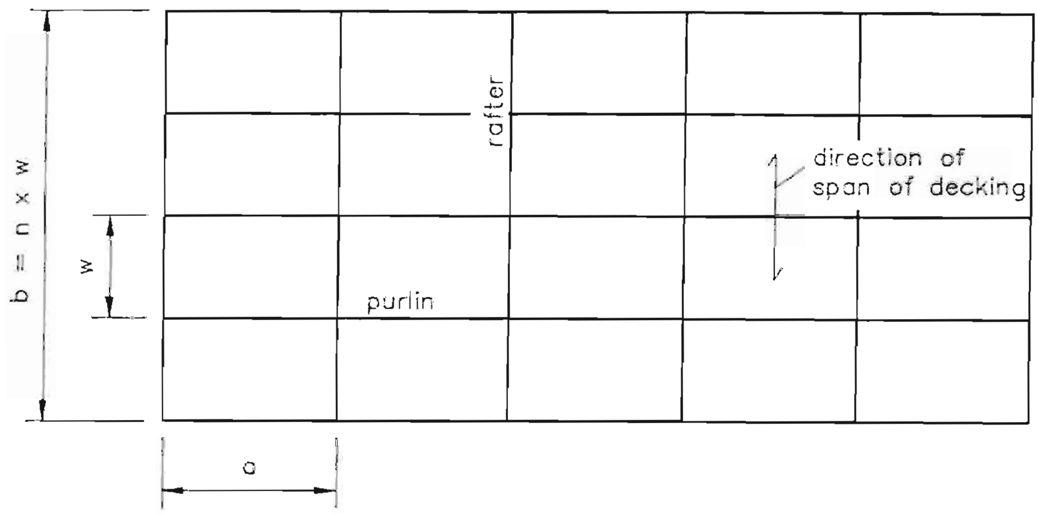


Figure B.4: Shear panels to stabilise purlins.
Notation for determining shear stiffness and shear strength of shear panels.

III.1.1 Shear stiffness of the shear panel to stabilise purlins

The shear flexibility c of the shear panel follows from:

$$c = c_{1.1} + c_{1.2} + c_{2.1} + c_{2.2}$$

in which (see figure B.4)

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

the expression for $c_{1.1}$ applies for $b/d \geq 10$

$$c_{1.2} = \frac{2a\alpha_2(1+\nu)[1+(2h/d)]}{Etnw}$$

$$c_{2.1} = \frac{2as_p p\alpha_3}{(nw)^2}$$

$$c_{2.2} = \frac{2s_s s_p (n_{sh}-1)}{2n_s s_p + \beta_1 n_p s_s}$$

For a, n and w: see figure B.4.

For d, α_1, α_2, α_3, α_4, K, E, t, h, s_p, p, s_s, n_s, n_{sh}, n_p and β_1: see table 5.5.

The shear deflection Δ of the shear panel follows from: $\Delta = P \cdot c$ in which P is the load on a shear panel caused by all the purlins to be stabilised.

The shear stiffness S_{act} of a shear panel (available per purlin to be stabilised) is then defined as:

$$S_{act} = \frac{P}{n+1} : \frac{\Delta}{a} = \frac{a}{c(n+1)}$$

For n see figure B.4.

III.1.2 Shear strength of the shear panel to stabilise purlins

The strength of the shear panel per purlin to stabilise will be determined by P_{max}. Furthermore three requirements shall be fulfilled to prevent non permissible modes.

$$P_{max} = [n_s F_s + \frac{\beta_1}{\beta_3}(n+1)F_p]/n$$

For n_s, F_s, F_p, β_1 and β_3: see 5.1.1.

For n: see figure B.4.

Requirements to prevent non permissible modes:

a. Sheet/purlin fastener strength (comparable to 5.1.3.1):

$$\frac{w}{\alpha_3 \; p} 0.6F_p \geq P_{max}$$

For p, α_3 and F_p see 5.1.3.1.

For w see figure B.4.

b. End collapse of sheeting profile (comparable to 5.1.3.2):

Every corrugation at the end of the sheeting fastened:

$$0.9\ t^{1.5} w f_y / d^{0.5} \geq P_{max}$$

Alternate corrugations at the end of the sheeting fastened:

$$0.3\ t^{1.5} w f_y / d^{0.5} \geq P_{max}$$

For t and d see 5.2.1.1.

For w see figure B.4.

And f_y is the design yield strength of the sheet material.

c. Shear buckling (comparable to 5.1.3.3):

The shear buckling strength of the sheeting should be checked in accordance with the expressions in 5.4 with the requirement:

$$V_{red} \geq n \cdot P_{max}$$

III.2 Shear stiffness and shear strength of a shear panel on rafters.

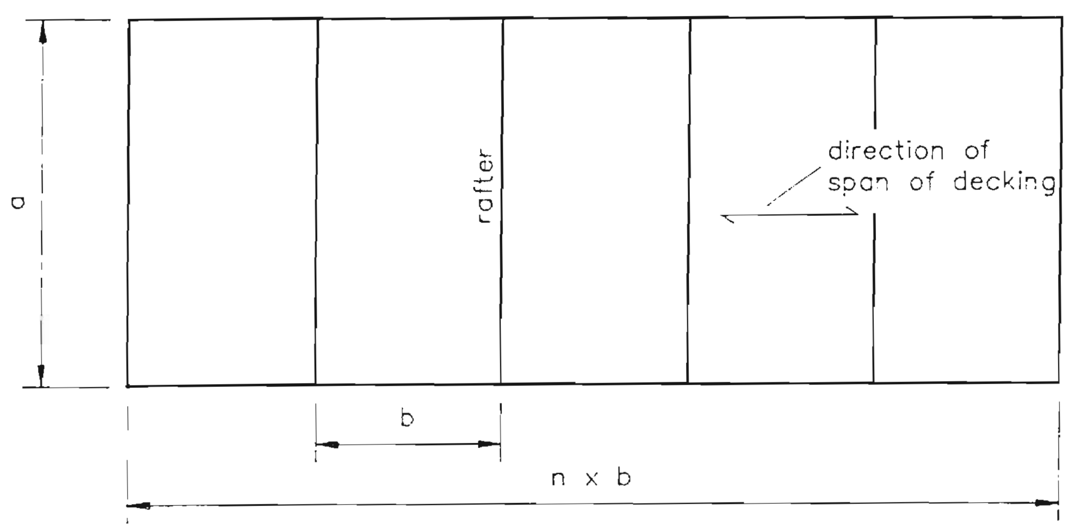


Figure B.5: Shear panels to stabilise rafters.
Notation for determining shear stiffness and shear strength of shear panels.

III.2.1 Shear stiffness of the shear panel to stabilise rafters

The shear flexibility c of the shear panel follows from:

$$c = c_{1.1} + c_{1.2} + c_{2.1} + c_{2.2}$$

in which (see figure B.5)

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

the expression for $c_{1.1}$ applies for $b/d \geq 10$

$$c_{1.2} = \frac{2a(1+\nu)[1+(2h/d)]}{Etb}$$

$$c_{2.1} = \frac{2as_p p}{b^2}$$

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

For a, n and b: see figure B.5.

For d, E, t, h, s_p, p, s_s, n_s, n_{sh} and β_1: see table 5.5.

For α_5 and K: see 5.9.1.2.

s_p is the slip per sheet/rafter fastener per unit load,

n_{sh} is the number of sheet widths over L.

The shear deflection Δ of the shear panel follows from: $\Delta = P \cdot c$ in which P is the load on a shear panel perpendicular to the corrugation.

The shear stiffness S_{act} of a shear panel (available per rafter to be stabilised) is then defined as:

$$S_{act} = P : \frac{\Delta}{b} = \frac{b}{c}$$

For w see figure B.5.

III.2.2 Shear strength of the shear panel to stabilise rafters

The strength of the shear panel per rafter to stabilise will be determined by P_{max}. Furthermore three requirements shall be fulfilled to prevent non permissible modes.

$$P_{max} = [n_s F_s + \frac{\beta_1}{\beta_3} F_p]$$

For n_s, F_s, F_p, β_1 and β_3: see 5.1.1.

Requirements to prevent non permissible modes:

a. Sheet/rafter fastener strength (comparable to 5.8.3.1):

$$\frac{b}{p}\ 0.6F_p \geq P_{max}$$

For p and F_p see 5.1.3.1.

For b see figure B.5.

b. End collapse of sheeting profile (comparable to 5.8.3.2):

Every corrugation at the end of the sheeting fastened:

$$0.9\ t^{1.5}bf_y/d^{0.5} \geq P_{max}$$

Alternate corrugations at the end of the sheeting fastened:

$$0.3\ t^{1.5}bf_y/d^{0.5} \geq P_{max}$$

For t and d see 5.2.1.1.

For b see figure B.5.

And f_y is the design yield strength of the sheet material.

c. Shear buckling (comparable to 5.8.3.3):

The shear buckling strength of the sheeting should be checked in accordance with the expressions in 5.4, with global shear buckling according to 5.8.3.3 and local shear buckling equal to $V_\ell = 4.83\ E(\frac{t}{\ell})^2 at$, with the requirement

$$V_{red} \cdot \frac{b}{a} \geq P_{max}$$

For a and b see figure B.5.

IV. Alternative method for flexural members

This annex gives a procedure to determine the stabilising effect of sheeting in case of doubly or mono-symmetrical flexural members such as I beams (figure B 6) or lattice girders (figure B 7).

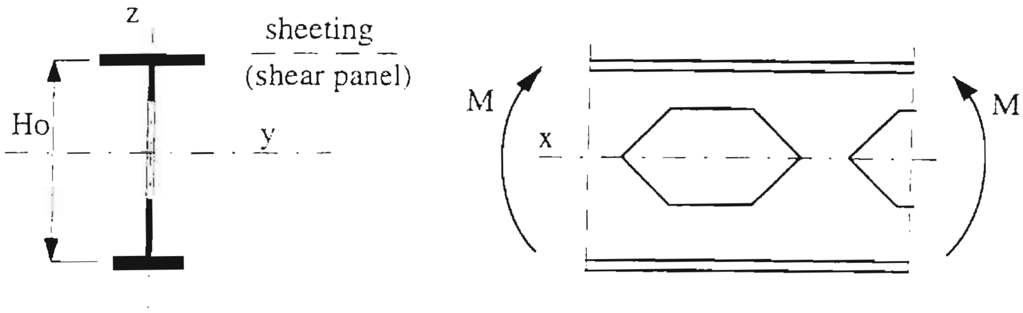


Figure B.6: Example of I beam stabilised by sheeting.

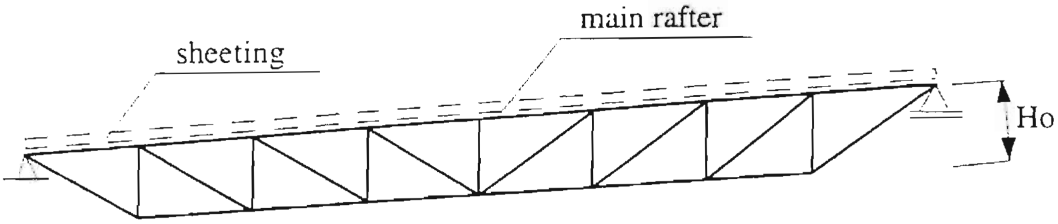


Figure B.7: Example of lattice girder stabilised by sheeting.

The calculation procedure is as follows:

- The in-plane load in the shear panel, caused by the interaction between sheeting and beam to be stabilise, follows from:

$$q = \frac{k_r}{62.5\ (\frac{a}{N} - c)}$$

where N is the normal force in the fictively isolated, compressed upper flange of member, under moment M

$N = M / H_0$

for M and H_0 see figures B.6 and B.7,

c is the shear flexibility defined in III.1.1 (when the shear panel is used to stabilise purlins) or in III.2.1 (when the shear panel is used to stabilise rafters),

k_r is a coefficient depending on the number n_r of members to stabilise:

$$k_r = [0.2 + 1/n_r]^{0.5}$$

$$n_r = n + 1$$

for n see figures B.4 an B.5

- The maximal shear force generated in the shear panel by a current member (purlin or rafter) to be stabilised is:

$$V_{max} = \frac{qa}{2}$$

- To prevent failure of the sheet/member fasteners, the following condition is required:

$$q \le \frac{F_p}{p}$$

which is, taking into account the above equation, equivalent to:

$$V_{max} \le 0.5\ F_p \frac{a}{p}$$

for p and F_p see 5.1.3.1

for a see figures B.4 or B.5.

- To prevent failure of the seam fasteners, the following condition is required:

$$V_{max} \le P_{max}$$

where P_{max} is defined in III.1.2 (when the shear panel is used to stabilise purlins) or in III.2.2 (when the shear panel is used to stabilise rafters).

- Furthermore, to prevent non permissible modes, the conditions defined in III.1.2 or in III.2.2 are required.

Annex C. MATHEMATICAL EXPRESSIONS FOR TABLES 5.2, 5.4, 7.1 AND 7.2

Annex C 1 : Table 5.2 Factors to allow for the number of sheet/purlin fasteners per sheet width

If n_f is an odd number:

Factor β_1: case 1 - sheeting $\beta_1 = \sum_i \left[\frac{2i}{n_f} \right]^3$

case 2 - decking $\beta_1 = \sum_i \left[\frac{2i}{n_f - 1} \right]^3$

Factor β_2: $\beta_2 = \sum_i \left[\frac{2i}{n_f - 1} \right]^2$

where n_f is the number of sheet/purlin fasteners per sheet width (including those at the overlaps)

i is a quantity which increases from 1 to $\frac{n_f - 1}{2}$

If n_f is an even number:

Factor β_1: case 1 - sheeting $\beta_1 = \sum_i \left[\frac{2i-1}{n_f} \right]^3$

case 2 - decking $\beta_1 = \sum_i \left[\frac{2i-1}{n_f - 1} \right]^3$

Factor β_2: $\beta_2 = \sum_i \left[\frac{2i-1}{n_f - 1} \right]^2$

where n_f is the number of sheet/purlin fasteners per sheet width (including those at the overlaps)

i is a quantity which increases from 1 to $\frac{n_f}{2}$

Annex C 2 : Table 5.4 Factors to allow for the effect of intermediate purlins

Factor α_1: this is emperical

Factor α_2: $$\alpha_2 = \frac{1}{1 + \sum_i \left[1 - \frac{2i}{n_p - 1} \right]}$$

Factor α_3: $$\alpha_3 = \frac{1}{1 + \sum_i \left[1 - \frac{2i}{n_p - 1} \right]^2}$$

where n_p is the number of purlins (edge + intermediate)

i is a quantity which increases from 1 to $\frac{n_p - 1}{2}$

Annex C 3 : Table 7.1 Reduction factor η on sway forces and moments for each frame in a clad building - all frames loaded.

Table 7.2 Factors by which η should be divided for one frame only loaded.

The general case results in n simultaneous equations for the reduction factors R_i:

$$R_i - R_{i-1} + \frac{r}{n+1} \left(\sum_{j=1}^{n} -jR_j + \sum_{j=1}^{n} (n+1) R_j \right) = \frac{r}{n+1} \left(\sum_{j=1}^{n} -jP_j + \sum_{j=1}^{n} (n+1) P_j \right)$$

There are two cases to consider:

Case 1: All frames equally loaded: $P_j = P$ for all j,

Case 2: One frame loaded: $P_k = P$, $P_j = 0$ for j unequal k.

For table construction, the loaded frame is at the centre of the building. For solution, the equations are expressed in matrix form and the two cases considered together.

$$
\begin{array}{cccccccc}
(1) & (2) & & (i-1) & (i) & & (n-1) & (n) \\
\end{array}
$$

$$
\begin{bmatrix}
\frac{nr}{n+1}+1 & \frac{(n-1)r}{n+1} & \cdots & \frac{(n-i+2)r}{n+1} & \frac{(n-i+1)r}{n+1} & \cdots & \frac{2r}{n+1} & \frac{r}{n+1} \\
\frac{-r}{n+1}-1 & \frac{(n-1)r}{n+1}+1 & \cdots & \frac{(n-i+2)r}{n+1} & \frac{(n-i+1)r}{n+1} & \cdots & \frac{2r}{n+1} & \frac{r}{n+1} \\
\vdots & & & & & & & \\
\frac{-r}{n+1} & \frac{-2r}{n+1} & \cdots & \frac{-(i-1)r}{n+1}-1 & \frac{(n-i+1)r}{n+1}+1 & \cdots & \frac{2r}{n+1} & \frac{r}{n+1} \\
\vdots & & & & & & & \\
\frac{-r}{n+1} & \frac{-2r}{n+1} & \cdots & \frac{-(i-1)r}{n+1} & \frac{-ir}{n+1} & \cdots & \frac{2r}{n+1}+1 & \frac{r}{n+1} \\
\frac{-r}{n+1} & \frac{-2r}{n+1} & \cdots & \frac{-(i-1)r}{n+1} & \frac{-ir}{n+1} & \cdots & \frac{-(n-1)r}{n+1}-1 & \frac{r}{n+1}+1
\end{bmatrix}
\begin{bmatrix}
R_1 \\ \vdots \\ R_2 \\ \vdots \\ R_i \\ \vdots \\ R_{n-1} \\ \vdots \\ R_n
\end{bmatrix}
=
\begin{bmatrix}
r\frac{n}{2} & r\frac{n-k+1}{n+1} \\
r(\frac{n}{2}-1) & r\frac{n-k+1}{n+1} \\
\vdots & \\
r(\frac{n}{2}+1-i) & * \\
\vdots & \\
& \\
r(1-\frac{n}{2}) & r\frac{-k}{n+1}
\end{bmatrix}
P
$$

where term * is $r\dfrac{n-k+1}{n+1}$ if $i \le k$ or $r\dfrac{-k}{n+1}$ if $i > k$

Table 7.1 follows directly from the solution for case 1.
Table 7.2 is the ratio of the solutions for case 1 and case 2 for the loaded frame when this frame is at the centre of the structure.

For the above expressions <u>only</u>, the nomenclature is as follows:

- n is the number of intermediate frames,
- r is the relative flexibility as defined in 7.2.1,
- i is the number of the frame under consideration (counting the first intermediate frame as frame 1),
- j is a quantity which increases from 1 to n or from i to n as all frames are considered within a given equation i,
- k is the number of the loaded frame (counting the first intermediate frame as frame 1),
- R_i is the reduction factor for forces in frame i,
- P_i is the force on frame i (equal to unity for table construction).

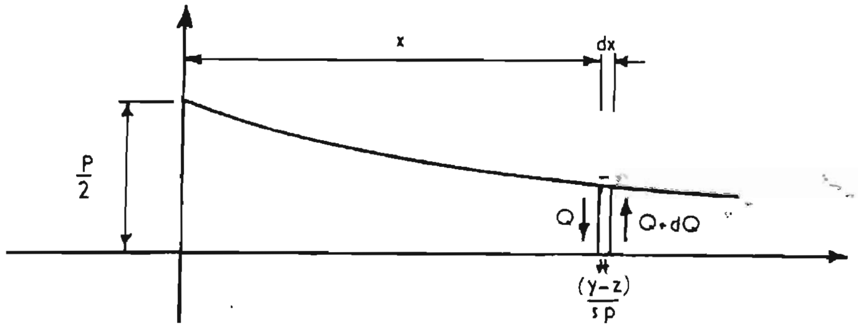
Annex D. DIAPHRAGMS SUBJECT TO CONCENTRATED LOADS

D.1 Introduction

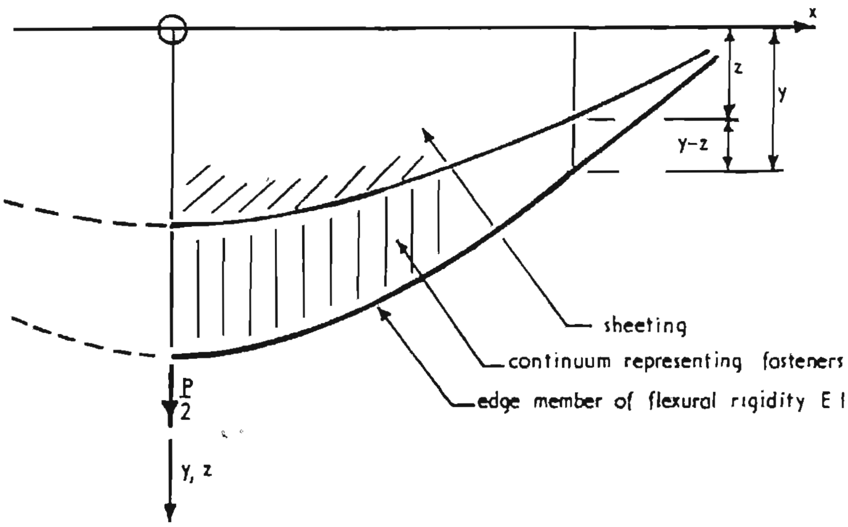
This annex gives design expressions for diaphragms subject to concentrated loads in plane of the diaphragm in situations where there is no primary member in the line of the load. The concentrated load is therefore distributed among the edge member fasteners to an extend that depends on the bending stiffness of the edge member relative to the shear stiffness components of the diaphragm. The full theory is given in reference [D.1].

D.2 Mathematical Model

The mathematical model considered is shown in figure D.1.



(a) Shear force distribution in the sheeting



(b) Relative displacements of sheeting and edge member

Figure D.1: Influence of a point load on a diaphragm

Although figure D.1 shows a tensile point load P (P/2 on half of the panel), the design expressions are unchanged if the load is compressive.

D.3 Design Expressions

Let: EI = flexural rigidity of the edge member,

G = effective shear modulus of the sheeting allowing for distortion of the profile,

s = flexibility (mm/kN) of the edge fasteners,

p = pitch of the fasteners,

b = total depth of the diaphragm,

t = thickness of the sheeting,

$$\alpha = \{(4spEI)^{-0.5} + (4spbtG)^{-1.0}\}^{0.5}$$

The maximum fastener force is:

$$F_{max} = \frac{P}{4\alpha} \left\{ \frac{P}{sEI} \right\}^{0.5}$$

and the maximum bending moment in the edge member is:

$$M_{max} = \frac{P}{4\alpha} \left\{ 1 + \frac{1}{4btG} \left(\frac{EI}{sp} \right)^{0.5} \right\}$$

When checking the edge member fasteners, the force F_{max} should be added to any other forces existing in these fasteners.

It should also be confirmed that the edge member is adequate to carry the bending stresses associated with M_{max} (about an axis normal to the diaphragm) in addition to any other forces present in this member at the point of the load.

Reference

D.1 J.M. Davies
Concentrated loads on light gauge steel diaphragms
J. Struct. Mech., 6(2), 1978 pp 165-194.

Annex E.1

WORKED EXAMPLES

The partial safety factors used in these examples are the boxed values of Eurocode 3. These are:

Dead Load:	γ_G	=	1.35
Imposed Load:	γ_Q	=	1.50
Material factor for yielding:	γ_M	=	1.10
Material factor for fasteners:	see comment to R5.3.2		

E1 SINGLE SHEAR PANEL WITH THE SHEETING SPANNING PERPENDICULAR TO THE SPAN OF THE DIAPHRAGM

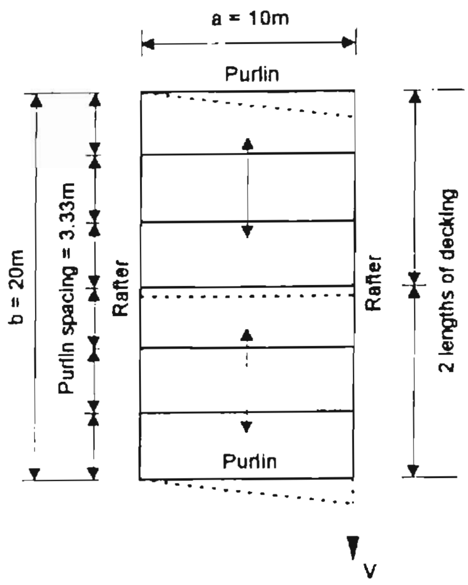
This example illustrates the case of a single shear panel with the sheeting spanning perpendicular to the span of the diaphragm. It illustrates the calculation of the shear capacity and shear flexibility with various fixing conditions. The example shows the application of sections 5.1 and 5.2.

E1.1 Problem

To determine the design shear capacity and shear flexibility of the panel of decking shown when fixed as follows:

(a) on four sides, fasteners in every trough
(b) on four sides, fasteners in alternate troughs
(c) on two sides, fasteners in every trough
(d) on two sides, fasteners in alternate troughs.

Assume the shear buckling strength of the decking and the axial strength of the edge purlins are adequate.

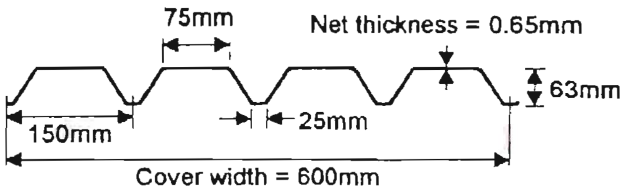


Note: This is a cantilever diaphragm in which the sheeting spans perpendicular to the span of the diaphragm.

Annex E.2

E1.2 Data

Decking profile:



Decking length: 10.2m

Grade of steel: FeE 280 G to EN 10147 (f_u = 0.36 kN/mm^2) with γ_m = 1.1

Purlins: 254 mm x 146 mm Universal Beam x 31 kg/m, Area = 4000mm^2

Sheet/purlin fasteners: 3.7 mm diameter cartridge fired pins

Sheet/rafter fasteners: 3.7 mm diameter cartridge fired pins at 333 mm centres. (for fastening on four sides):

Seam fasteners: 4.8 mm self-drilling, self-tapping screws at 333 mm centres

E1.3 Values of symbols

a = 10,000mm
A = 4000mm^2
b = 20,000mm
d = 150mm
E = 210 kN/mm^2
F_p = 2.9 x 0.360 x 3.7 x 0.65 = 2.51 kN (table 5.1)
F_s = 2.9(0.65/4.8)$^{1/2}$ x 0.36 x 3.7 x 0.65 = 0.924 kN (table 5.1)
F_{sc} = 2.51 kN (= F_p)
h = 63mm
K_1 = 0.278 (see footnote)
K_2 = 2.97 (see footnote)
ℓ = 75mm
n_b = 2
n_f = 5 (every trough fastened)
n_f = 3 (alt. troughs fastened)
n_p = 7
n_s = $\left(\frac{3333}{333} - 1\right) \times 6 = 54$
n_{sc} = 9 x 6 + 7 = 61 = n'_{sc}

n_{sh} = 10 x 1000/600 = 17
p = 150 (every trough fastened
p = 300 (alt. troughs fastened)
s_p = 0.10mm/kN (table 5.1)
s_s = 0.25mm/kN (table 5.1)
s_{sc} = 0.10mm/kN (table 5.1)
t = 0.65mm
f_y = 0.28/1.1 = 0.254 kN/mm^2
α_1 = 0.85 (table 5.4 for n_p = 4)
α_2 = 0.50 (table 5.4 for n_p = 7)
α_3 = 0.64 (table 5.4 for n_p = 7)
α_4 = 1+0.3 x 2 = 1.6 (table 5.8 case 5)

For every trough fastened (n_f = 5):
β_1 = 1.13 (table 5.2, case 2)
β_2 = 1.25 (table 5.2)
β_3 = 1.0 (5.1.1.1)

For alt. troughs fastened (n_f = 3)
β_1 = 1.0 (table 5.2 case 2)
β_2 = 1.0 (table 5.2)
β_3 = 1.0 (5.1.1.1)

Footnotes:

1. For the profile shown, $\ell/d = \frac{75}{150} = 0.5$, $h/d = \frac{63}{150} = 0.420$ and θ = 21.6°. K_1 and K_2 are found by interpolation from tables 5.6 and 5.7, for fasteners in every trough and alternate troughs respectively.

2. Fastener strengths include the material factor γ_m.

Annex E.3

E1.4 Design Shear Capacity

E1.4.1 Case (a) Panel fixed on four sides, fasteners in every trough (Clause 5.1.1)

Seam capacity $V_{ult} = n_s F_s + \frac{\beta_1}{\beta_3} n_p F_p = 54 \times 0.924 + \frac{1.13}{1.0} \times 7 \times 2.51 = 69.8$ kN

Shear connector capacity $V_{ult} = n_{sc} F_{sc} = 61 \times 2.51 = 153.1$ kN

Hence, design shear capacity $V* = 69.8$ kN

Sheet/purlin fasteners - check if $\frac{0.6bF_p}{p\alpha_3} \geq V*$

i.e. $\frac{0.6 \times 20000 \times 2.51}{150 \times 0.64} = 313.8 \text{ kN} > 69.8\text{kN}$

This is satisfactory.

End collapse of profile - check if $0.9t^{1.5}\, bf_y/d^{0.5} \geq V*$

i.e. $\frac{0.9 \times 0.65^{1.5} \times 20000 \times 0.254}{150^{0.5}} = 196.0 \text{ kN} > 69.8 \text{ kN}$

This is satisfactory.

E1.4.2 Case (b) Panel fixed on four sides, fasteners in alternate troughs (Clause 5.1.1)

Seam capacity $V_{ult} = n_s F_s + \frac{\beta_1}{\beta_3} n_p F_p = 54 \times 0.924 + \frac{1.0}{1.0} \times 7 \times 2.51 = 67.5$ kN

Shear connector capacity $V_{ult} = 153.1$ kN as in E1.4.1

Hence, design shear capacity $V* = 67.5$ kN

Sheet/purlin fasteners - check if $\frac{0.6bF_p}{p\alpha_3} \geq V*$

i.e. $\frac{0.6 \times 20000 \times 2.51}{300 \times 0.64} = 156.9$ kN > 67.5 kN

This is satisfactory

End collapse of profile - check if $0.3t^{1.5}\, bf_y/d^{0.5} \geq V*$

i.e. $\frac{0.3 \times 0.65^{1.5} \times 20000 \times 0.254}{150^{0.5}} = 65.4$ kN/m < 67.5 kN

This is marginally unsatisfactory.

End collapse is a non permissible mode. Therefore the number of seam fasteners should be reduced in order to lower the design shear capacity below 65.4kN. For purposes of illustration, the calculations below continue using the original design data.

E1.4.3 Case (c) Panel fixed on two sides, fasteners in every trough (Clause 5.1.1)

Seam capacity	V_{ult} = 69.8 kN as in E1.4.1.
Capacity of end sheet/purlin fasteners	$V_{ult} = \beta_2\, n_p F_p$ = 1.25 x 7 x 2.51 = 22.0 kN
Capacity of purlin/rafter connections	$V_{ult} = n_p F_{pr}$ = 7 x 10.0 = 70.0 kN (table 5.3, take connection 8)

Hence, design shear capacity V* = 22.0 kN

End collapse - $0.9t^{1.5}\, bf_y/d^{0.5}$ = 215.7 kN as in E1.4.1.

Since this is greater than 22.0 kN, this is satisfactory.

E1.4.4 Case (d) Panel fixed on two sides, fasteners in alternate troughs(Clause 5.1.1)

Seam capacity V_{ult} = 67.5 kN as in E1.4.2.

Capacity of end sheet/purlin fasteners $V_{ult} = \beta_2 n_p F_p = 1.0 \times 7 \times 2.51 = 17.6$ kN

Capacity of purlin/ rafter connections V_{ult} = 70.0 kN as in E1.4.3.

Hence, design shear capacity V* = 17.6 kN

End collapse - $0.3t^{1.5}\, bf_y/d^{0.5}$ = 65.4 kN from E1.4.2.

Since this is greater than 17.6 kN, this is satisfactory.

E1.5 Shear Flexibility (Table 5.5, column(2) for cantilever diaphragm)

E1.5.1 Case (a) Panel fixed on four sides, fasteners in every trough Clause 5.2.1)

$$\text{Profile distortion } c_{1.1} = \frac{10000 \times 150^{2.5} \times 0.85 \times 1.6 \times 0.278}{210 \times 0.65^{2.5} \times (20000)^2} = 0.036 \text{ mm/kN}$$

$$\text{Shear strain } c_{1.2} = \frac{2 \times 10000 \times 1.3 \times 1.84}{210 \times 0.65 \times 20000} = 0.018 \text{ mm/kN}$$

$$\text{Sheet/purlin fasteners } c_{2.1} = \frac{2 \times 10000 \times 0.10 \times 150}{20000^2} = 0.001 \text{ mm/kN}$$

$$\text{Seam fasteners } c_{2.2} = \frac{2 \times 0.25 \times 0.10 \times 16}{2 \times 54 \times 0.10 + 1.13 \times 7 \times 0.25} = 0.063 \text{ mm/kN}$$

$$\text{Sheet/shear connector fasteners } c_{2.3} = \frac{2 \times 0.10}{61} = 0.003 \text{ mm/kN}$$

$$\text{Axial strain } c_3 = \frac{2 \times 10000^3}{3 \times 210 \times 4000 \times 20000^2} = 0.002 \text{ mm/kN}$$

c = 0.123 mm/kN

Annex E.6

E1.5.2 Case(b) Panel fixed on four sides, fasteners in alternate troughs (Clause 5.2.1)

$$\text{Profile distortion } c_{1.1} = \frac{10000 \times 150^{2.5} \times 0.85 \times 1.6 \times 2.97}{210 \times 0.65^{2.5} \times 20000^2} = 0.389 \text{ mm/kN}$$

$$\text{Shear strain } c_{1.2} = \text{as in E1.5.1} = 0.018 \text{ mm/kN}$$

$$\text{Sheet/purlin fasteners } c_{2.1} = \frac{2 \times 10000 \times 0.10 \times 300}{20000^2} = 0.002 \text{ mm/kN}$$

$$\text{Seam fasteners } c_{2.2} = \frac{2 \times 0.25 \times 0.10 \times 16}{2 \times 54 \times 0.10 + 1.0 \times 7 \times 0.25} = 0.064 \text{ mm/kN}$$

$$\text{Sheet/shear connector fasteners } c_{2.3} = \text{as in E1.5.1} = 0.003 \text{ mm/kN}$$

$$c = \mathbf{0.476 \text{ mm/kN}}$$

E1.5.3 Case (c) Panel fixed on two sides, fasteners in every trough (Clause 5.2.1)

Profile distortion $c_{1.1}$	= as in E1.5.1	= 0.036 mm/kN
Shear strain $c_{1.2}$	= as in E1.5.1	= 0.018 mm/kN
Sheet/purlin fasteners $c_{2.1}$	= as in E1.5.1	= 0.001 mm/kN
Seam fasteners $c_{2.2}$	= as in E1.5.1	= 0.063 mm/kN
Purlin/rafter connections $c_{2.3}$ (table 5.3, take connection 8)	$= \frac{2}{7}\left(2.6 + \frac{0.10}{1.25}\right)$	= 0.766 mm/kN
Axial strain c_3	= as in E1.5.1	= 0.002 mm/kN

$$c = \mathbf{0.886 \text{ mm/kN}}$$

Annex E.7

E1.5.4 Case (d) Panel fixed on two sides, fasteners in alternate troughs (Clause 5.2.1)

Profile distortion $c_{1.1}$	= as in E1.5.2	= 0.389 mm/kN
Shear strain $c_{1.2}$	= as in E1.5.1	= 0.018 mm/kN
Sheet/purlin fasteners $c_{2.1}$	= as in E1.5.2	= 0.002 mm/kN
Seam fasteners $c_{2.2}$	= as in E1.5.2	= 0.064 mm/kN
Purlin/rafter connections $c_{2.3}$ (table 7, take connection 8)	$= \frac{2}{7}\left(2.6 + \frac{0.10}{1.0}\right)$	= 0.771 mm/kN
Axial strain, c_3	= as in E1.5.1	= 0.002 mm/kN

c = **1.246 mm/kN**

E1.6 Summary of calculations

Case	Four or two sides fastened	Fasteners in every or alt. troughs	Calculated design shear capacity kN	Criterion of strength	Calculated shear flexibility mm/kN
(a)	Four	Every	69.8	Seam fasteners.	0.123
(b)	Four	Alternate	65.4	Seam fasteners.	0.476
(c)	Two	Every	22.0	End sheet/purlin fasteners.	0.886
(d)	Two	Alternate	17.6	End sheet/purlin fasteners.	1.246

The calculations illustrate the following effects:

(1) Fasteners in alternate troughs give a much less stiff diaphragm than fasteners in every trough.

(2) Fixing a panel of sheeting on two sides only (to the purlins) rather than on four sides, reduces the shear capacity considerably.

(3) For a panel of sheeting fixed on two sides only, the flexibility of the purlin/rafter connections greatly affects the panel flexibility. These connections should be stiffened whenever possible.

(4) End collapse of the panel is a non permissible mode. In this example, steps must be taken to prevent this failure mode. In this case it has been suggested that the fastener strength is reduced.

E2 PANEL ASSEMBLY WITH THE SHEETING SPANNING PARALLEL TO THE LENGTH

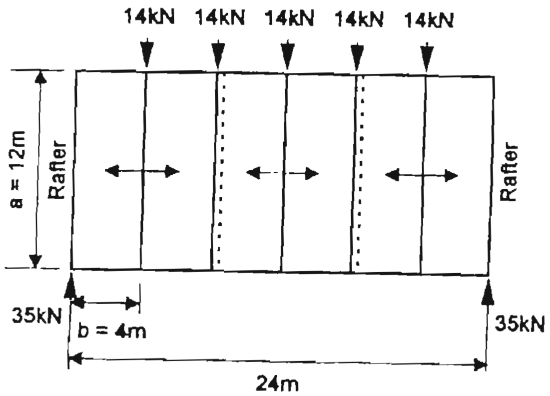
This example illustrates the case of a panel assembly with the sheeting spanning parallel to the length of the diaphragm It gives calculations of shear capacity and deflection including the effects of shear buckling and openings. The example shows the application of sections 5.8, 5.9 and 8.3.

E2.1 Problem

To determine the design shear capacity and shear deflection of the roof deck diaphragm shown, taking into account the following factors:

(a) shear buckling of the decking
(b) effect of openings

Assume the axial strength of the edge members is adequate.



Note
The decking spans parallel to the span of the diaphragm.

E2.2 Data

Edge members:	90 mm x 90 mm x 10 mm equal angle, Area = 1710 mm^2
Decking profile:	as in Appendix E1, except that the net thickness is 0.85 mm
Decking lengths:	8.2m

Decking fastened on all four sides

Grade of steel:	FeE 280 G to EN 10147 (f_u = 0.36 kN/mm^2) with γ_m = 1.1
Sheet/rafter fasteners:	3.7 mm diameter cartridge fired pins in alternate troughs

Sheet/edge member fasteners	6.3 mm diameter self-drilling, self-tapping screws with steel washers at 500 mm centres.
Seam fasteners:	4.8 mm Monel blind rivets at 500 mm centres
Loads:	the loads shown on the roof deck diaphragm are factored. The load factor is 1.5.

E2.3 Values of Symbols

a	=	12000 mm	n_ℓ	=	3 (table 5.10)
A	=	1710 mm^2	n_s	=	(4 x 10^3/500 - 1) = 7
b	=	4000 mm	n_{sc}	=	4 x 10^3/500 = 8
d	=	150 mm	n_{sh}	=	12000/600 = 20
E	=	210 kN/mm^2	p	=	300 (alternate troughs)
F_p	=	1.90 x 0.360 x 6.3 x 0.85	s_p	=	0.10 mm/kN
	=	3.66 kN (table 5.1)	s_s	=	0.30 mm/kN
F_s	=	3.2 (0.85/4.8)^{½} x 0.36 x 4.8 x 0.85 = 1.98 kN (table 5.1)	s_{sc}	=	0.10 mm/kN
			t	=	0.85 mm
F_{sc}	=	3.66 kN (= F_p)	f_y	=	0.28/1.1 = 0.254 kN/mm^2
h	=	63 mm	α_5	=	0.9/2=0.45 (tables 5.10 and 5.11)
K_2	=	2.97 (as in E1.3)			
L	=	24000 mm	β_1	=	1.0 (table 5.2,n_f = 3, case 2)
n	=	6 (table 5.10)	β_2	=	1.0 (table 5.2, n_f = 3)
			β_3	=	1.0 (5.1.1.1)

E2.4 Design Shear Capacity (Clause 5.8.1)

E2.4.1 Capacity of decking

Seam capacity $V_{ult} = \frac{a}{b}\left(n_s F_s + \frac{\beta_1}{\beta_3} F_p\right) = \frac{12000}{4000}\left(7 \times 1.98 + \frac{1.0}{1.0} \times 3.66\right) = 52.6 \text{ kN}$

Fasteners to edge members $V_{ult} = \frac{a}{b}(n_{sc} F_{sc}) = \frac{12000}{4000}(8 \times 3.66) = 87.8\text{Kn}$

Hence design shear capacity $V* = 52.6\text{kN}$

Sheet/rafter fasteners check if $\frac{0.6\ a\ F_p}{p} \geq V*$

i.e. $\frac{0.6 \times 12000 \times 3.66}{300} = 87.8 \text{ kN} > 52.6 \text{ kN}$

This is satisfactory

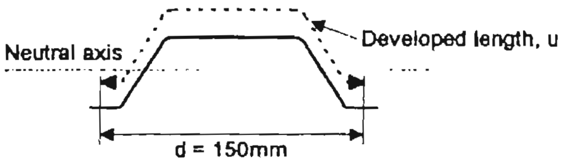
Annex E.10

End collapse of profile - check if $0.3t^{1.5} \, af_y/d^{0.5} \geq V*$

i.e. $\dfrac{0.3 \times 0.85^{1.5} \times 12000 \times 0.254}{150^{0.5}} = 58.6 \text{ kN} > 52.6 \text{ kN}$

This is satisfactory

E2.4.2 Shear buckling of decking



From the manufacturer's data, the second moment of area of the decking profile about its neutral axis is given as 72.1 cm^4/m. For a single corrugation:

$$I = \frac{72.1 \times 10^4}{1000} \times 150 = 108.1 \times 10^3 mm^4$$

The perimeter length, u, of a single corrugation is obtained by calculation from the decking profile shown in E1.2 Thus u = 236 mm.

$$\text{Hence bending stiffness } D_x = \frac{205 \times 0.85^3 \times 150}{12 \times (1-0.3^2) \times 236} = 7.33 \text{ kNmm}$$

$$\text{and bending stiffness } D_y = \frac{205 \times 108.1 \times 10^3}{150} = 147{,}737 \text{ kNmm}$$

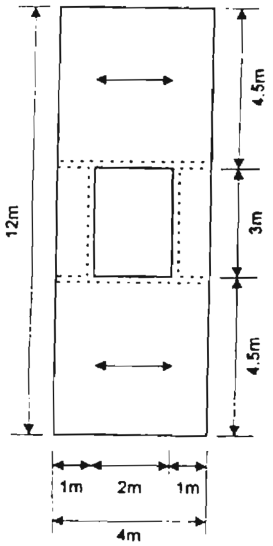
Shear buckling capacity - check if $\dfrac{14.4a}{b^2} D_x^{1/4} D_y^{3/4} \geq V*$

i.e. $\dfrac{14.4 \times 12000}{(4000)^2} \times 7.33^{1/4} \times 147{,}737^{3/4} = 134 \text{ kN} > 52.6 \text{ kN}$

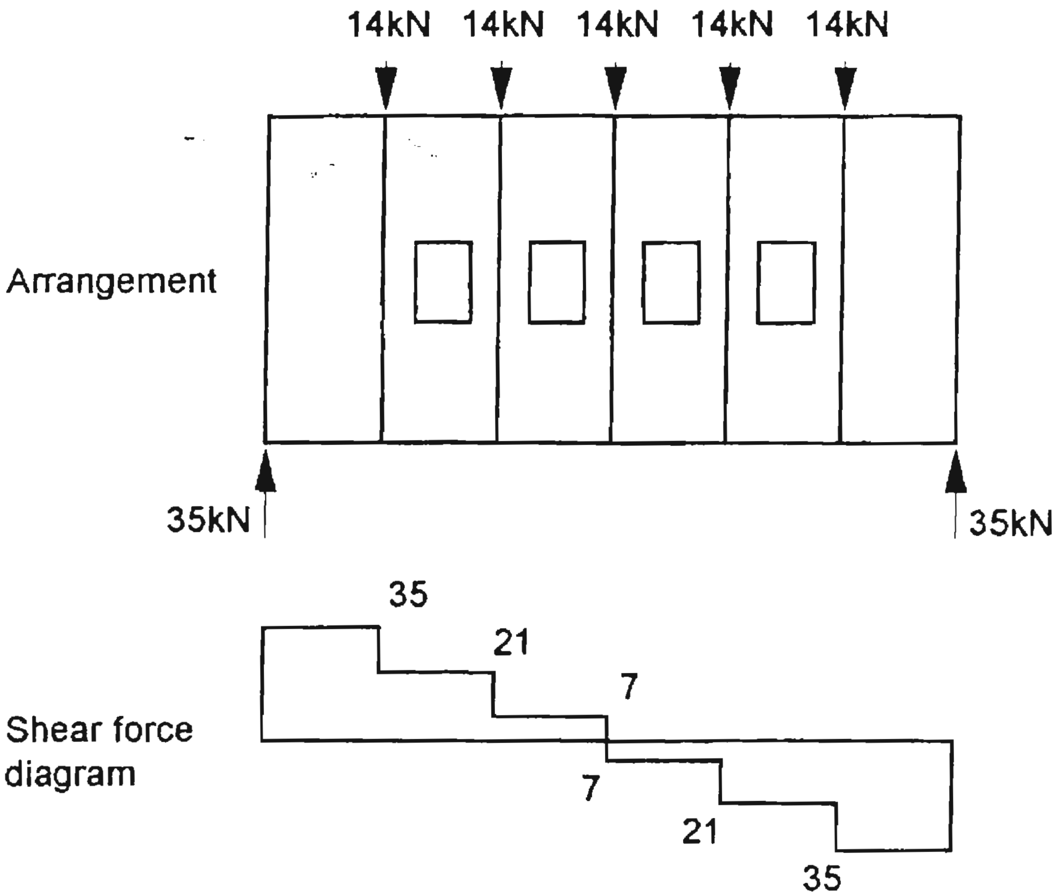
This is satisfactory.

E2.4.3 Effect of openings (Clause 8.3)

All the requirements of 8.3.1 must be satisfied. The maximum size of a single opening is determined by 8.3.1(c) and 8.3.1(d) as shown, i.e. Area = $6m^2$.



If the seam fastener spacing is maintained at 400 mm throughout, the shear capacity of the above panel may be taken as 50% of the shear capacity of a panel without openings.



For the diaphragm arrangement shown, with openings in the interior panels but no openings in the end panels, the design criteria are as follows:

End panels: $V^* \geq 35$ kN

Interior panels: $0.5V^* \geq 21$ kN i.e. $V^* \geq 42$ kN.

Since the capacity of a panel is $V^* = 52.6$ kN, the decking panels with openings are adequately strong.

E2.5 Shear Deflections (Table 5.9, column(1) for panel assemblies)

E2.5.1 Shear flexibility of decking (Clause 5.9)

$$\text{Profile distortion } c_{1.1} = \frac{12000 \times 150^{2.5} \times 0.45 \times 2.97}{210 \times 0.85^{2.5} \times 4000^2} = 1.975 \text{ mm/kN}$$

$$\text{Shear strain } c_{1.2} = \frac{2 \times 12000 \times 1.3 \times 1.84}{210 \times 0.85 \times 4000} = 0.080 \text{ mm/kN}$$

$$\text{Sheet/rafter fasteners } c_{2.1} = \frac{2 \times 12000 \times 0.10 \times 300}{4000^2} = 0.045 \text{ mm/kN}$$

$$\text{Seam fasteners } c_{2.2} = \frac{0.30 \times 0.10 \times 19}{7 \times 0.10 + 1.0 \times 0.30} = 0.570 \text{ mm/kN}$$

$$\text{Sheet/edge member fasteners } c_{2.3} = \frac{2 \times 0.10}{8} = 0.025 \text{ mm/kN}$$

$$c = 2.695 \text{ mm/kN}$$

$$\text{Hence, } c^1 = \frac{4000^2}{12000^2} \times 2.695 = 0.299 \text{ mm/kN}$$

$$\text{Axial strain } c_3 = \frac{6^2 \times 4000^3}{4.8 \times 210 \times 1710 \times 12000^2} = 0.009 \text{ mm/kN}$$

Hence total shear flexibility of a panel of decking is

$$c = c^1 + c_3 = 0.308 \text{ mm/kN}$$

E2.5.2 Effect of openings (Clause 8.3.3)

The multiplying factor to be applied to the shear flexibility of panels, for a profile height of 63 mm is (see 8.3.3)

$$\frac{1}{1 - 2.5\left(\frac{h}{50}\right)^{½}\left(\frac{A_d}{ab}\right)} = \frac{1}{1 - 2.5\left(\frac{63}{50}\right)^{½}\left(\frac{6 \times 10^6}{12000 \times 4000}\right)} = 1.54$$

i.e. modified c for a panel with openings = 0.308 x 1.54 = 0.474 mm/kN.

E2.5.3 Deflection of roof deck diaphragm

For an assembly of six panels, with insulation bonded to the top of the decking, and with roof openings in the four internal panels, as shown in E2.4.3, the mid length deflection at the unfactored load is given by:

$$\Delta = \frac{1}{1.5}(35 \times 0.308 + 21 \times 0.474 + 7 \times 0.474) = 16.0 \text{ mm}$$

E2.6 Summary of calculations

The calculations illustrate the following points:

(1) The procedure for checking the shear buckling capacity of the decking

(2) The method of checking the design shear capacity of diaphragms with openings.

(3) The effect of roof openings in increasing the shear flexibility of decking.

(4) The method of calculating the maximum shear deflection of an assembly of panels with different shear flexibilities.

Annex E.14

E3 FLAT ROOFED BUILDING WITH THE SHEETING SPANNING PERPENDICULAR TO THE LENGTH OF THE DIAPHRAGM

This example illustrates the case of a flat roofed building with the sheeting spanning perpendicular to the length of the diaphragm. It gives calculations for the clad building behaviour with both pin-jointed and rigid-jointed frames. The example shows the application of Section 7.

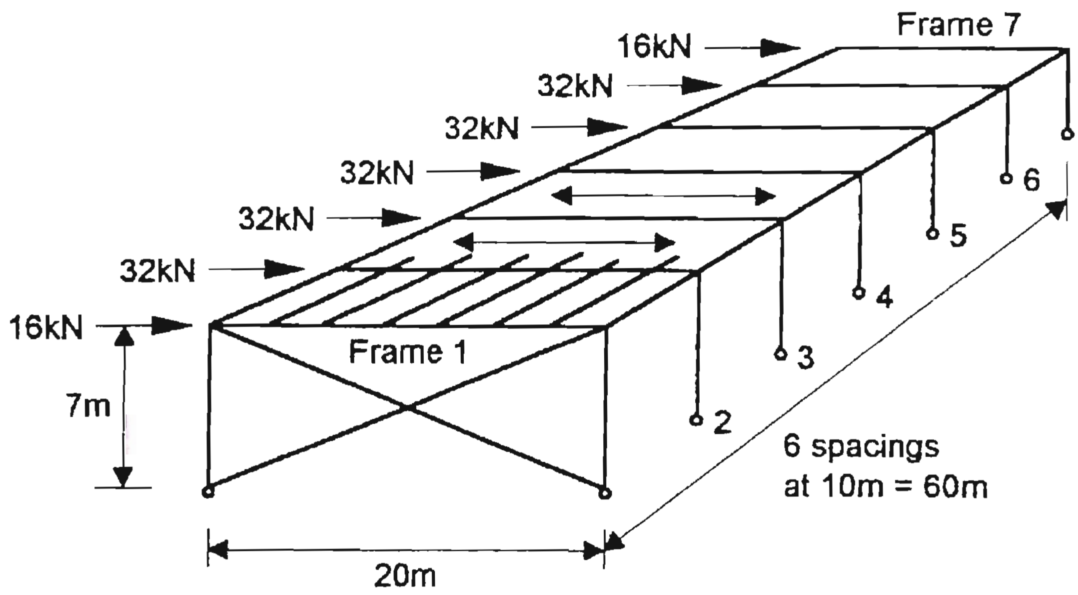
E3.1 Problem

To calculate the strength and deflection of a flat roofed building under side load with panels of decking as in Annex E1, case (a). Consider the following cases:

(a) pin jointed frames, in which the roof diaphragm carries the side load alone.

(b) rigid frames, in which the frames carry the side load alone.

(c) rigid frames with roof diaphragm, which combine to carry the side load.



E3.2 Data

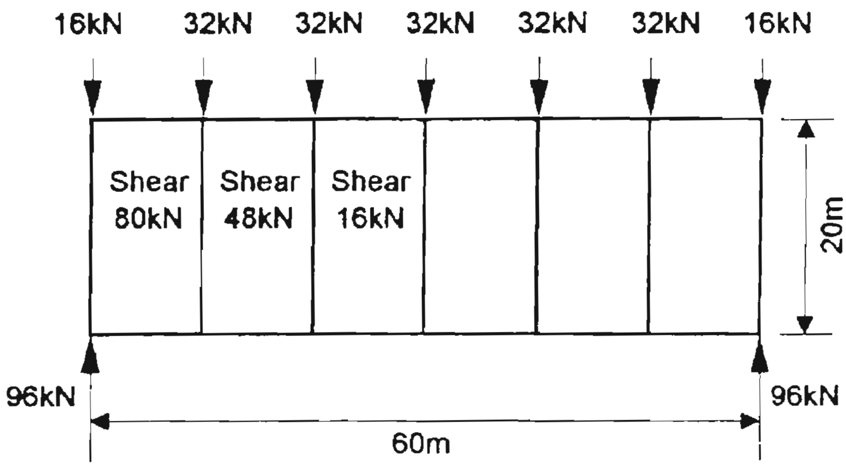
Sheeting: Sheeting spanning perpendicular to length of diaphragm. Panel fixed on four sides, fasteners in every trough.

Frames: 533 mm x 210 mm Universal Beam x 92 kg/m, Area = 11800mm^2 I = 533 x 10^6 mm^4

Loads: The loads shown in the diagram are already factored. The load factor is 1.5.

E3.3 Case (a): Roof Diaphragm Alone

E3.3.1 Diaphragm capacity



From annex E1, case (a), the design shear capacity of a panel is 69.8 kN which is less than the maximum shear force in a panel, 80 kN. The diaphragm capacity is therefore inadequate to carry all of the factored load.

E3.3.2 Diaphragm deflection (Table 5.5, column(1) for panel assemblies)

The shear flexibility of the single diaphragm panel considered in annex E1, case (a), is a little different from that of a panel in a diaphragm assembly. The correct value of shear flexibility is as follows:

Profile distortion $c_{1.1}$ = 0.036 mm/kN

Shear strain $c_{1.2}$ $= 0.5 \times 0.018 = 0.009$mm/kN

Sheet/purlin fasteners $c_{2.1}$ $= 0.64 \times 0.001 = 0.001$mm/kN

Seam fasteners $c_{2.2}$ = 0.063mm/kN

Sheet/shear connector fasteners $c_{2.3} = \dfrac{4\times(6+1)\times0.10}{6^2 \times 61} = 0.001$mm/kN

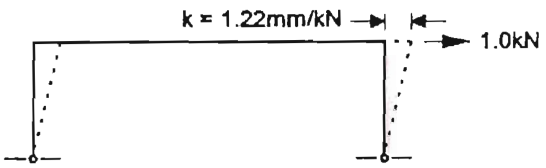
Axial strain $c_3 = \dfrac{6^2\times10000^3\times0.64}{4.8\times210\times40000^2\times20000^2} = 0.014$mm/kN

$c = 0.124$ mm/kN

The mid length deflection of the diaphragm alone under unfactored load is given by:

$$\Delta = \frac{32}{1.5} \times \frac{6^2}{8} \times 0.124 = 11.9 \text{ mm}$$

E3.4 Case (b) Rigid Frames alone



The frame flexibility, k mm/kN, may be calculated by normal elastic methods and is found to be 1.22 mm/kN. Under an unfactored side load per frame of 32/1.5 kN, the sidesway deflection is $\frac{32}{1.5} \times 1.22 = 26.03$ mm.

E3.5 Case (c): Rigid Jointed Frames and Diaphragms (Clause 7.2)

The relative flexibility of the diaphragm to the frame is given by

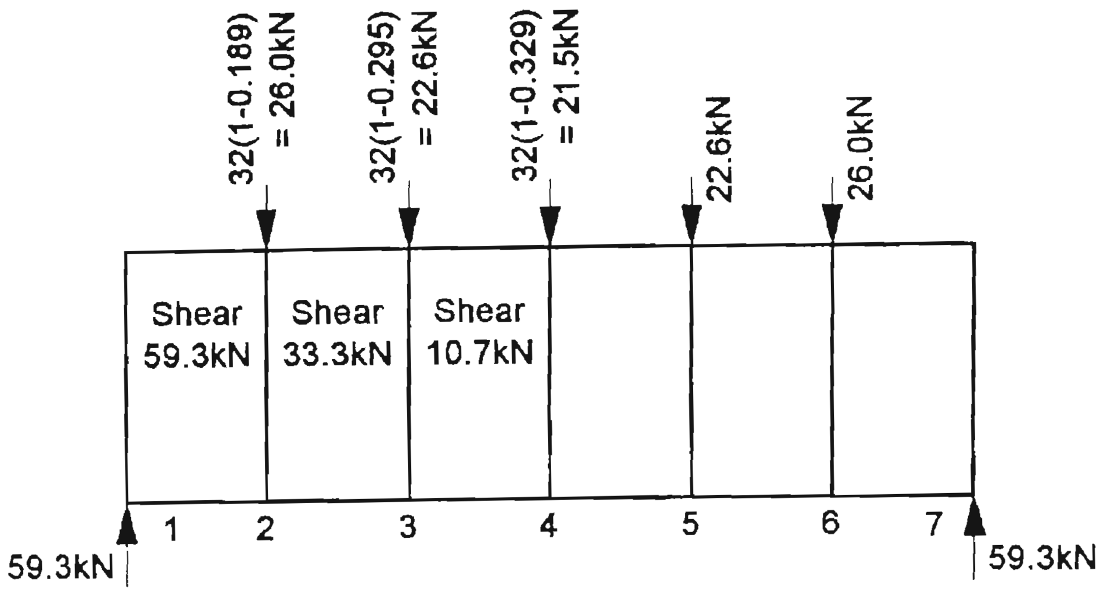
$$r = \frac{c}{k} = \frac{0.124}{1.22} = 0.102$$

The reduction factors η to be applied to the sway forces and deflections in a building of seven frames are obtained by interpolation from Table 7.1:

For frame 2, η = 0.189
For frame 3, η = 0.295
For frame 4, η = 0.329

The sidesway deflection of the central frame (frame 4) in the clad building under unfactored load is thus 0.329 x 27.9 = 9.2 mm.

The factored forces and shears on the roof diaphragm are shown below. It is noted that the shear in the end panel is now less than the design capacity of the diaphragm (69.8 kN).



E3.6 Summary of Calculations

The calculations illustrate the following points:

(1) The shear flexibility of the decking, fastened in every trough, is considerably less than the frame flexibility.

(2) In the central frame of the clad building, the maximum bending moment under side load is only about one third of that in a bare frame.

(3) The sidesway deflection of the central frame in the clad building is only about one third of that of the bare frame.

(4) The shear in the end panel when the diaphragm acts in conjunction with the frames is considerably less than when the diaphragm acts alone.

E4 FLAT ROOFED BUILDING WITH THE SHEETING SPANNING PARALLEL TO THE LENGTH OF THE DIAPHRAGM

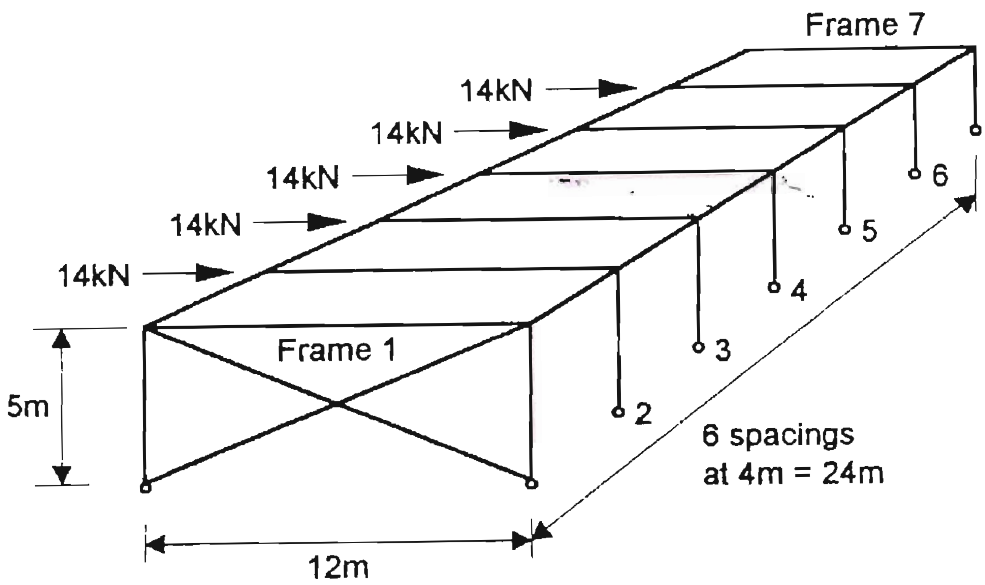
This example illustrates the case of a flat roofed building with sheeting spanning parallel to the length of the diaphragm. If gives calculations of clad building deflections with all frames loaded and one frame loaded.

E4.1 Problem

To calculate the strength and deflection of a flat roofed building under side load with panels of decking as in annex E2. Consider the following cases:

(a) combined action of the rigid frames and roof diaphragms: all frames loaded.

(b) combined action of the rigid frames and roof diaphragms: central frame only loaded:



E4.2 Data

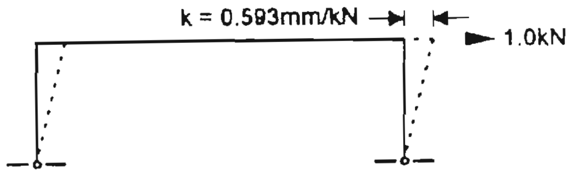
Sheeting: sheeting spanning parallel to length of diaphragm. Panel fixed on four sides, fasteners in alternate troughs.

Frames: 457 mm x 191 mm Universal Beam x 82 kg/m, Area = 10500mm^2, I = 371 x 10^{6}mm^4.

Loads: the loads shown in the diagram are already factored. The load factor is 1.5.

E4.3 Case (a) All Frames Loaded (Clause 7.2)

Consider the case of the decking, fastened in alternate troughs, without roof openings. From annex E2, the shear flexibility c is 0.308 mm/kN.



The frame flexibility may be calculated by normal elastic methods and is found to be k = 0.593 mm/kN.

The relative flexibility of the diaphragm to the frame is given by

$$r = c/k = \frac{0.308}{0.593} = 0.519$$

For a building with seven frames, the reduction factors to be applied to the sway forces and deflections are from table 7.1:

For frame 2, η = 0.484
frame 3, η = 0.700
frame 4, η = 0.762

The sidesway deflection of a bare frame under unfactored load is given by

$$\Delta = \frac{14}{1.5} \times 0.593 = 5.5 \text{ mm}$$

The sidesway deflection of the central frame in the clad building when all frames are loaded with unfactored loads is given by

$$\Delta = \frac{14}{1.5} \times 0.593 \times 0.762 = 4.2 \text{ mm}$$

E4.4 Case (b) Central Frame Only Loaded (Clause 7.2)

If the central frame only is loaded, the reduction factor η for all frames loaded, is divided by a further factor as given in table 7.2. For a building of seven frames, with r equal to 0.519, this further factor is found to be 2.31.

The sidesway deflection of the central frame is then

$$\Delta = \frac{4.2}{2.31} = 1.8 \text{ mm}$$

E4.5 Summary of Calculations

The calculations illustrate the following points:

(1) When all frames are loaded, the deflection of the frames adjacent to the gables is considerably less than the deflection of the central frame, which in turn is considerably less than the deflection of a bare frame.

(2) When the central frame only is loaded, the sidesway deflection of the clad frame compared with that of the bare frame, is small.

(3) The effect of the fasteners in alternate troughs rather than in every trough (worked example E3) reduces the benefit of stressed skin action, but still results in moments and deflections in the central frame of 0.76 times the bare frame values.

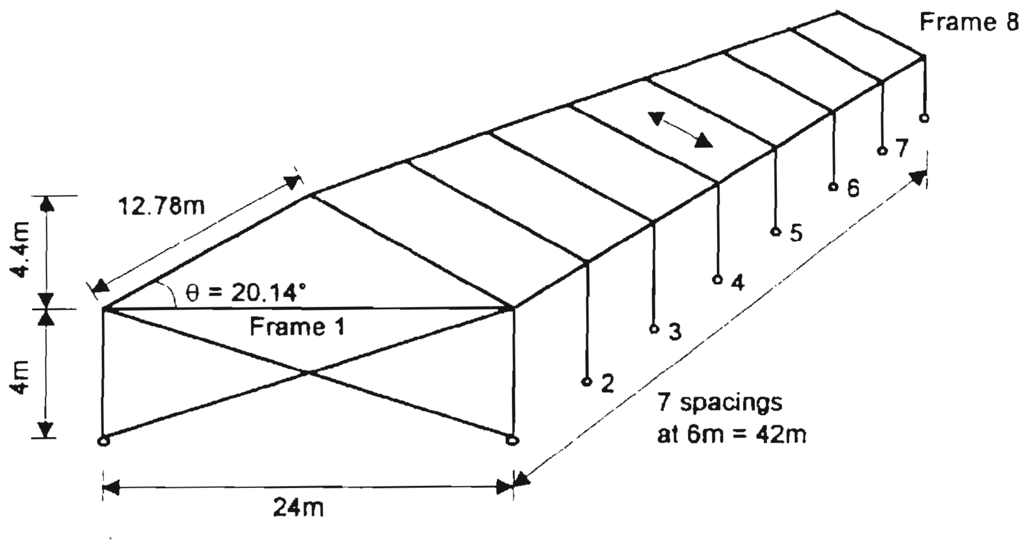
E5 PITCHED ROOF BUILDING UNDER VERTICAL LOAD

This example illustrates the case of a pitched roof building under vertical load and demonstrates the calculation of clad building behaviour using elastic analysis. The forces for the plastic design of clad frames are also calculated. The example shows the application of Sections 7.2 and 7.3.

E5.1 Problem

(a) To calculate the strength and eaves deflection of a sheeted pitched roof portal frame building under vertical dead load and imposed load.

(b) To determine the forces to be used in the plastic design of the sheeted frame.



E5.2 Data

Frames: Two-pinned portal frames of section 406 mm x 178 mm Universal Beam x 60 kg/m, Area = 7600 mm^2, I = 215 x 10^{6}mm^4, design moment resistance $M_{c.Rd}$ = 297.3 kNm. (f_y = 275 N/mm^2, γ_m = 1.1)

Sheeting panels: Assume that the panels are all similar with a design shear capacity of 100 kN and a shear flexibility of 0.12 mm/kN.

Loads: Assume that the dead load is 0.5 kN/m^2 (load factor 1.35) and that the imposed load is 0.75 kN/m^2 (load factor 1.5).

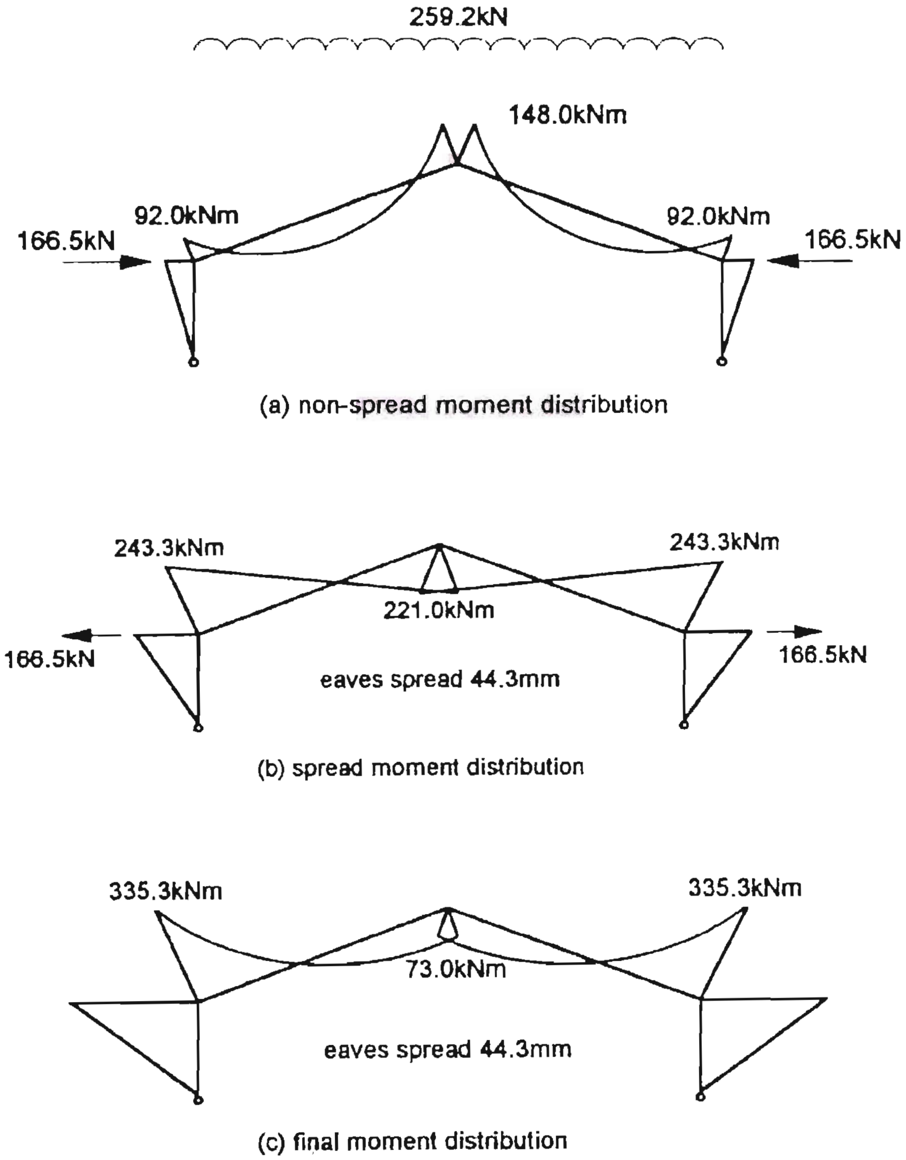
E5.3 Elastic Calculations for the Bare Frame

The total factored vertical load per frame is given by

$$W = 24 \times 6 \times [0.5 \times 1.35 + 0.75 \times 1.5] = 259.2 \text{ kN}$$

Annex E.22

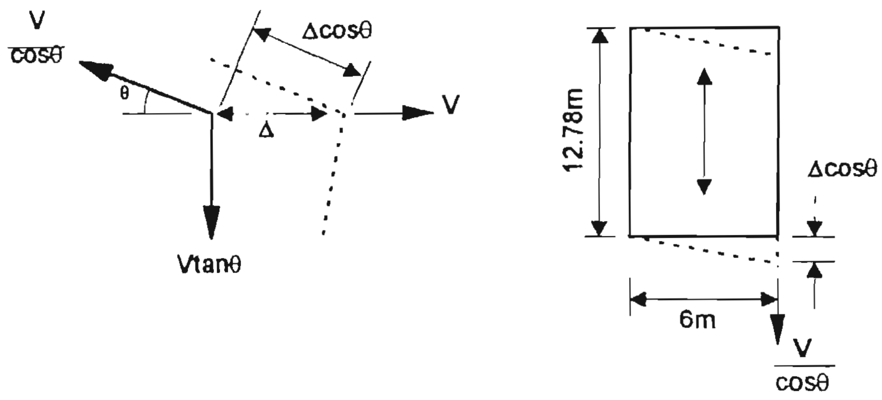
The bending moment diagram for the frame may be obtained by conventional elastic analysis and may be considered to be the sum of (a) a non-spread moment distribution with movement at the eaves prevented by a notional horizontal eaves force and (b) a spread moment distribution due to reversal of this eaves force to give (c) a final bending moment distribution. These three diagrams are shown below:



It is seen from bending moment diagram (c) that the maximum bending moment in the bare frame is 335.3 kNm which exceeds the yield moment of the section, 297.3 kNm. The design is therefore inadequate.

E5.4 Elastic Calculations for the Clad Frame (Clause 7.2)

E5.4.1 For the sheeting



Eaves detail

Panel of sheeting

In the plane of the sheeting the shear flexibility c is given by

$$c = \frac{\Delta\cos\theta}{V/\cos\theta} = \frac{\Delta}{V}\cos^2\theta$$

In the horizontal direction, the horizontal shear flexibility c_h is given by

$$c_h = \frac{\Delta}{V}$$

Hence, from the above two equations, $c_h = c\,\sec^2\theta$

For the panel in question, c = 0.12 mm/kN and $\theta = 20.14^0$, hence c_h = 0.136 mm/kN.

E5.4.2 For the frame:

From bending moment diagram (b) in E5.3, the spread flexibility of the frame is given by

$$k_{sp} = \frac{44.3}{166.5} = 0.266 \text{ mm/kN}$$

E5.4.3 For the clad frames:

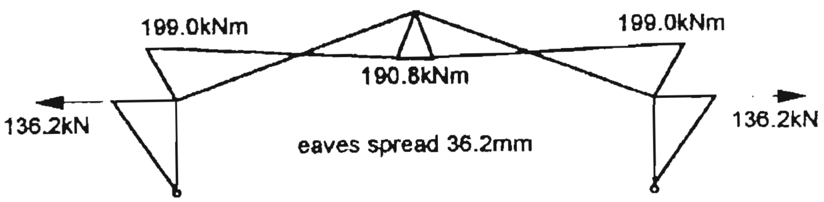
The relative flexibility of the sheeting to the frame is given by

$$r_{sp} = \frac{c_h}{k_{sp}} = \frac{0.136}{0.266} = 0.511$$

so that the reduction factors to be applied to the spread moments in an eight frame building (table 7.1) are:

For frame 2, η = 0.492
frame 3, η = 0.725
frame 4, η = 0.818

For the worst case, frame 4, the spread moment distribution for the clad frame is 0.818 x diagram (b), i.e.



(d) spread moment distribution in clad frame

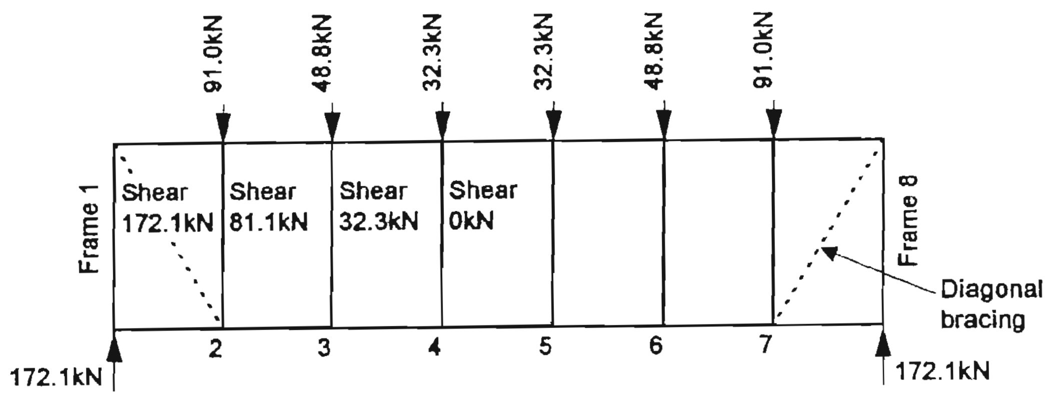
The final bending moment diagram in the clad frame is the sum of diagrams (a) and (d),and the maximum bending moment in the frame is 92.0 + 199.0 = 291.0 kNm which is a reduction of 13% and is less than the yield moment of the section.

E5.4.4 Forces on the sheeting

The horizontal forces on the sheeting at frames 2, 3 and 4 are respectively:

at frame 2, V = (1 - 0.492) x 166.5 = 84.6 kN
frame 3, V = (1 - 0.725) x 166.5 = 45.8 kN
frame 4, V = (1 - 0.818) x 166.5 = 30.3 kN

Hence, the forces in the plane of the sheeting, $\dfrac{V}{\cos\theta}$, are



Annex E.25

The shear in the end panel exceeds the design shear capacity of the sheeting, 100 kN, and is therefore not allowable. By inserting a diagonal bracing member in the end panels (this member could be erection bracing), the critical panel moves to the penultimate panel where the shear is 81.1 kN, which is less than the design shear capacity of the sheeting. This design is therefore adequate.

E5.5 Forces for the Plastic Design of the Clad Frame (Clause 7.3)

E5.5.1 With no end bracing

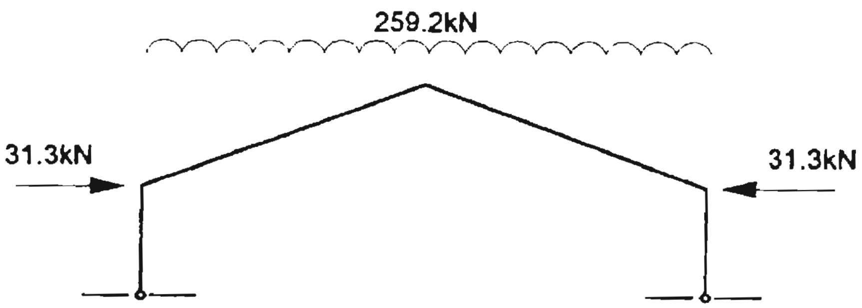
The restraining force R at each frame, in the plane of the sheeting is given by

$$R = \frac{2V^*}{n-1} = \frac{2 \times 100}{7-1} = 33.3 \text{ kN}$$

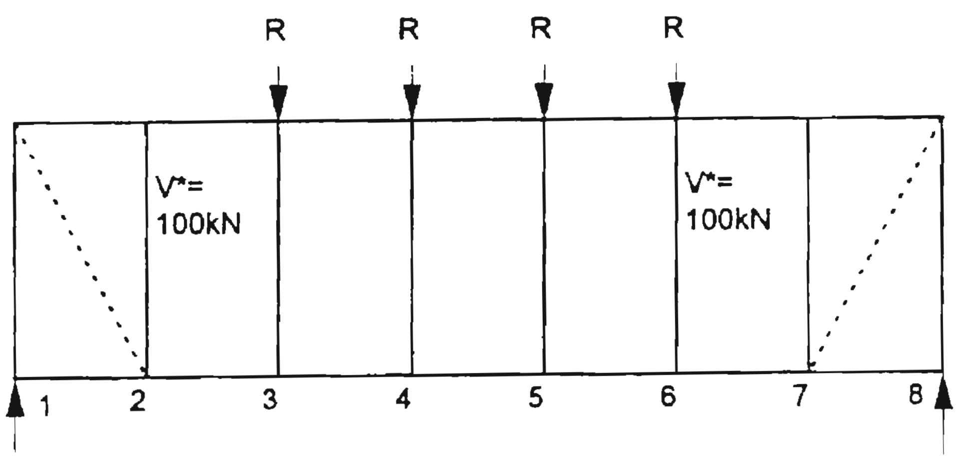
and the horizontal restraining force V at each frame is

$$V = R\cos\theta = 33.3 \times 0.939 = 31.3 \text{ kN}$$

Each frame should then be plastically designed under the action of the following factored forces:



E5.5.2 With end bracing

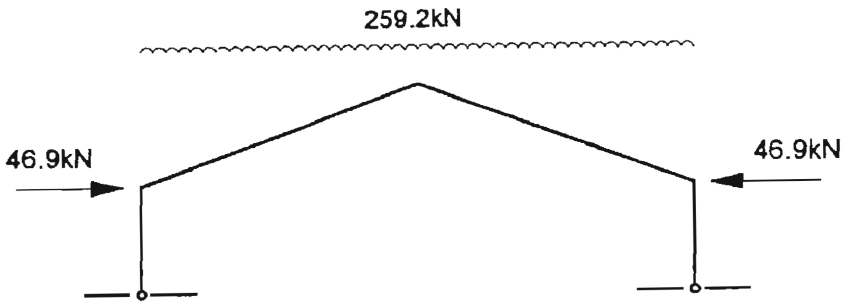


The diaphragm assembly is effectively reduced to five panels so that

$$R = \frac{2V^*}{n-1} = \frac{2 \times 100}{5 - 1} = 50 \text{ kN}$$

$$\text{and} \quad V = R\cos\theta = 46.9 \text{ kN}$$

Each frame may then be plastically designed under the action of the following factored forces:



E5.6 Summary of calculations

The calculations illustrate the following points:

(1) That the roof sheeting may make a useful contribution to resisting vertical load, for roof pitches of the order of 20°.

(2) The resulting shear forces in the end panels may be substantial.

(3) By utilising diagonal bracing in the end panels, the diaphragm assembly may be strengthened considerably.

(4) Eaves forces, to be used in the plastic design of sheeted frames, are simply obtained.

<u>Note</u>: although the calculations are not given, the roof sheeting would make a substantial contribution to resisting horizontal load and sidesway deflection.

E6 THE DIAPHRAGM STRENGTH AND SHEAR STIFFNESS OF A COMPOSITE FLOOR DECK IN VARIOUS STAGES OF CONSTRUCTION

This example shows how to calculate the diaphragm strength and shear stiffness of a composite floor deck in various stages of construction. It illustrates the application of section 8.6.

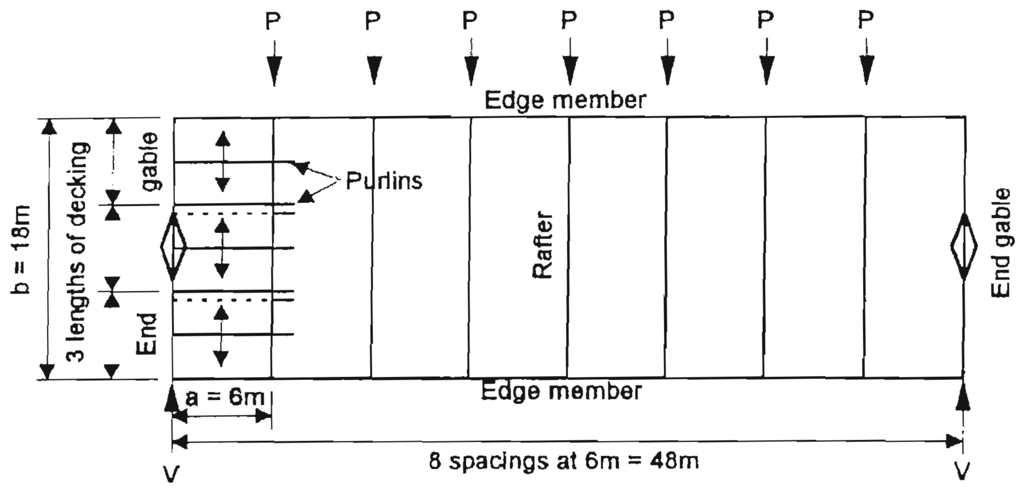
E6.1 Problem

To check the diaphragm strength and shear deflection of the composite floor deck shown, at the following stages:

(a) in the erection stage, when the steel deck acts alone as a diaphragm.

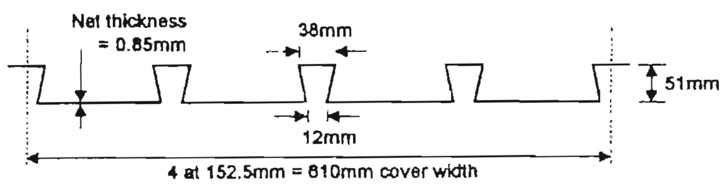
(b) in the final stage, when the steel deck/concrete floor acts as a diaphragm.

Assume the shear buckling strength of the decking and the axial strength of the edge members are adequate.



E6.2 Data

Decking profile:



Decking: fastened to purlins only

Decking length: 6.2m

Annex E.28

Grade of steel:	FeE 280 G to EN10147 (f_u = 0.36 kN/mm²) with $\gamma_m = 1.1$
Sheet/purlin fasteners:	3.7 mm diameter cartridge fired pins in alternate troughs
Sheet/end gable fasteners:	20 - 3.7 mm diameter cartridge fired pins
Seam fasteners:	4.8 mm self drilling screws (6 per 3m span)
Edge member:	Area 4000 mm²
Load:	take the wind pressure as 1.0 kN/m². Assume it acts over an effective depth in the erection stage of 2.0m per storey, and in the final stage of 3.5m per storey. The load factor is 1.5.

E6.3 Values of Symbols

a = 6000 mm
A = 4000 mm²
b = 18000 mm
d = 152.5 mm
E = 210 kN/mm²
F_p = 2.9 x 0.36 x 3.7 x 0.85
= 3.28 kN (Table 5.1)
F_s = $2.9(0.85/4.8)^{1/2}$ x 0.36 x 4.8 x 0.85 = 1.79 kN (table 5.1)
F_{sc} = 3.28 kN (= F_p)
h = 51 mm
K_2 = 0.191 (see footnote)
ℓ = 38 mm
L = 48000 mm
n = 8
n_b = 3
n_f = 2 (alternate troughs fastened)
n_p = 7
n_s = 6 x 6 = 36
n_{sc} = 20
n_{sh} = $\frac{6000}{510}$ = 10
p = 305 mm
s_p = 0.10 mm/kN (table 5.1)
s_s = 0.25 mm/kN (table 5.1)
s_{sc} = 0.10 mm/kN (table 5.1)
t = 0.85 mm
f_y = 0.28/1.1 = 0.254 kN/mm²
α_1 = 1.0 (table 5.4 for n_p = 3)
α_2 = 0.50 (table 5.4 for n_p = 7)
α_3 = 0.64 (table 5.4 for n_p = 7)
α_4 = 1 + 0.3 x 3 = 1.9 (table 5.8, case 5)
β_1 = 0.13 (table 5.2, case 1)
β_2 = 1.0 (table 5.2)
β_3 = $\frac{2-1}{2}$ = 0.50 (5.1.1.1)

Footnote:

As an approximation, to determine K_2,

consider the ribs to be rectangular (not dovetailed) i.e. $\theta = 0°$.

Now, $\ell/_d = \frac{38}{152.5} = 0.249, \qquad \frac{h}{d} = \frac{51}{152.5} = 0.334$

From table 5.7 by interpolation, K_2 = 0.191

E6.4 Factored Loads on Diaphragm

E6.4.1 Case (a) - steel deck alone

Factored panel load P = 1.00 x 6 x 2.0 x 1.5 = 18.0 kN
Factored end reaction V = 18.0 x 7/2 = 63.0 kN

E6.4.2 Case (b) - completed composite floor

Factored panel load P = 1.00 x 6 x 3.5 x 1.5 = 31.5 kN
Factored end reaction V = 31.5 x 7/2 = 110.25 kN

E6.5 Design Shear Capacity

E6.5.1 Case (a) - steel deck alone (Clause 5.1.1)

Seam capacity $V_{ult} = n_s F_s + \frac{\beta_1}{\beta_3} n_p F_p = 36 \times 1.79 + \frac{0.13}{0.50} \times 7 \times 3.28 = 70.4$ kN

Sheet/end gable fastener capacity $V_{ult} = n_{sc} F_{sc} = 20 \times 3.28 = 65.6$ kN

Internal panel end fastener capacity $P_{ult} = \beta_2 n_p F_p = 1.0 \times 7 \times 3.28 = 23.0\text{kN} > 18.5\text{kN}$

This is satisfactory.

Hence, design shear capacity V* = 65.6 kN

Sheet/purlin fasteners - check if $\frac{0.6bF_p}{p\alpha_3} > V^*$

i.e. $\frac{0.6 \times 18000 \times 3.28}{305 \times 0.64} = 181.5 \text{ kN} > 65.6 \text{ kN}$

This is satisfactory

End collapse of profile - check if $0.3t^{1.5}bf_y/d^{0.5} \geq V*$

i.e. $\frac{0.3 \times 0.85^{1.5} \times 18000 \times (0.28/1.1)}{152.5^{0.5}} = 87.4 \text{ kN} > 65.6 \text{ kN}$

This is satisfactory

Also, the design shear capacity, 65.6 kN > maximum factored shear in the diaphragm (= V = 63.0 kN).

The strength of the diaphragm is therefore adequate.

E6.5.2 Case (b) - completed composite floor (Clause 8.6)

If the seam capacity and end collapse of the profile are excluded from the above, the design shear capacity is as follows:

(a) For sheet/end gable fastener capacity, V_{ult} = 65.6 kN (< 110.25 kN)

(b) For internal panel end fastener capacity, P_{ult} = 23.0 kN (< 31.5 kN)

Since these criteria are not satisfied, the following measures must be taken:

(a) The number of sheet/end gable fasteners must be increased to $\frac{110.25}{3.28} = 33.6$ say 34

(b) The sheet/purlin fasteners should be doubled at the longitudinal edge members and the purlins adjacent to the edges.

(Note: this effectively increases the number of purlins from 7 to 11 so that $P_{ult} = \beta_2 n_p F_p$ = 1.0 x 11 x 3.28 = 36.1 kN (> 31.5 kN which is satisfactory).

E6.6 Shear Deflection

E6.6.1 Case (a) - steel deck alone (Clause 5.2.1)

$$\text{Profile distortion } c_{1.1} = \frac{6000\times152.5^{2.5}\times1.0\times1.9\times0.191}{210\times0.85^{2.5}\times18000^2} = 0.014 \text{ mm/kN}$$

$$\text{Shear strain } c_{1.2} = \frac{2\times6000\text{x}0.5\text{x}1.3\text{x}1.67}{210\times0.85\times18000} = 0.004 \text{ mm/kN}$$

$$\text{Sheet/purlin fasteners } c_{2.1} = \frac{2\times6000\times0.10\times305\times0.64}{18000^2} = 0.001 \text{ mm/kN}$$

$$\text{Seam fasteners } c_{2.2} = \frac{2\times0.25\times0.10\times9}{2\times36\times0.10 + 0.13\times7\times0.25} = 0.061 \text{ mm/kN}$$

$$\text{Sheet/end gable fasteners } c_{2.3} = \frac{4 \times 7}{8^2 \times 7}\left(0 + \frac{0.10}{1.0}\right) = 0.006 \text{ mm/kN}$$

$$\text{Axial strain } c_3 = \frac{8^2 \times 6000^3 \times 0.64}{4.8 \times 210 \times 4000 \times 18000^2} = 0.007 \text{ mm/kN}$$

$$c = \mathbf{0.093 \text{ mm/kN}}$$

The mid length deflection of the decking diaphragm alone under unfactored load is given by:

$$\Delta = \frac{18.0}{1.5} \times \left(\frac{8^2}{8}\right) \times 0.093 = 8.9 \text{ mm}$$

E6.6.2 Case (b) - completed composite floor (Clause 8.6)

If $c_{1.1}$ and $c_{2.2}$ are excluded from the calculation in accordance with 8.6.8, then the value of c becomes 0.018 mm/kN and the mid length deflection of the steel deck/concrete diaphragm is given by:

$$\Delta = \frac{31.5}{1.5} \times \left(\frac{8^2}{8}\right) \times 0.018 = 3.0 \text{ mm}$$

E6.7 Summary of Calculations

The calculations illustrate the following points:

(1) That the procedure for calculating the strength and deflection of the steel deck diaphragm alone is as given in Chapter 5.

(2) That the wind loads on the steel deck diaphragm alone may be less than on the final steel deck/concrete diaphragm.

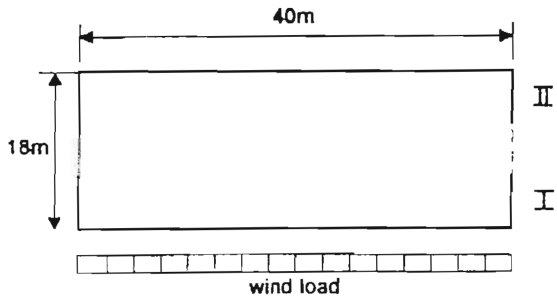
(3) That the procedure for calculating the strength and deflections of the steel deck/concrete diaphragm is as given in Chapter 5 except that, for strength, the seam capacity and end collapse of the profile need not be considered and, for deflection, the shear flexibility due to profile distortion and seam slip need not be considered.

E7 USE OF CASSETTES INSTEAD OF WALL BRACING

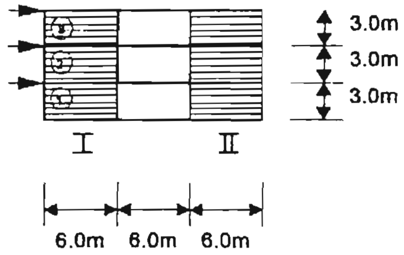
This example illustrates the use of cassettes instead of wall bracing. It illustrates the stabilisation of a 3-storey building with cassettes in the gable walls and shows the application of section 6.2.2.

E7.1 Problem

To investigate the strength and deflection of a three-storey wall panel comprising light gauge steel cassettes spanning horizontally.



Plan of building



Elevation on gable

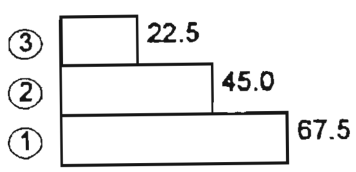
E7.2 Data

Wind load $=$ $0.5\ kN/m^2$ with load factor $\gamma_Q = 1.5$

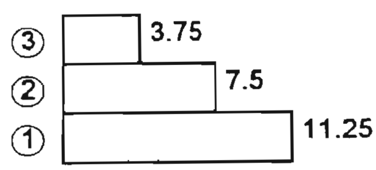
$\therefore$ Design wind load $= 0.5 \times 1.5 = 0.75\ kN/m^2$

$\therefore$ Distributed panel load $w_I = w_{II} = 0.5 \times 0.75 \times 40/2 = 7.5\ kN/m$

Panel load at each floor or roof level $= 7.5 \times 3.0 = 22.5\ kN$



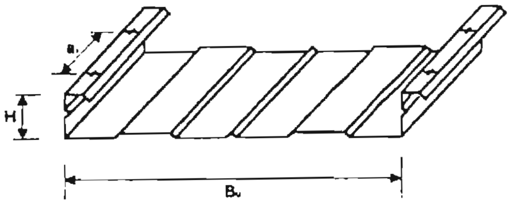
Design shear force $V_D^I = V_D^{II}$ (kN)



Design shear flow $T_{V,D}^I = T_{V,D}^{II}$ (kN/m) (idealized)

Annex E.33

Cassette



Width: B_u = 600mm

Height: H = 130mm

Grade of steel: FeE 320G to EN10147

Thickness: Panel 1 t_N = 1.25mm, t = 1.21, I_l = 19mm^4/mm

Panel 2 t_N = 1.00mm, t = 0.96, I_l = 15mm^4/mm

Panel 3 t_N = 0.75mm, t = 0.71, I_l = 11mm^4/mm

Fasteners to supporting members: 6.3mm diameter collar head screws (three per cassette)

Seam fasteners: 4.0mm diameter monel metal blind rivets at 300mm centres

Shear connector fasteners (to floor and roof members along top and bottom edges of panels): 6.3mm diameter collar head fasteners at 300mm centres

E7.3 Values of Symbols

a	=	3000mm	n_{sh}	=	5
b	=	6000mm	p	=	200mm
d	=	600mm	s_p	=	0.15 mm/kN
E	=	210 kN/mm^2	s_s	=	0.30 mm/kN
f_u	=	0.39 kN/mm^2	s_{sc}	=	0.15 mm/kN
f_y	=	0.32/1.1 = 0.291 kN/mm^2	t	=	1.21, 0.96 or 0.71mm
F_p	=	5.65 kN, 4.48 kN or 3.31 kN	β_1	=	0.30
F_s	=	3.64 kN, 2.57 kN or 1.64 kN	β_3	=	(3-1)/3 = 0.667
F_{sc}	=	F_p	B_u	=	600mm
K	=	0 (no sheet distortion with cassette profiles)	B	=	3000mm (=ΣB_u)
			ℓ_s	=	300mm
n_f	=	3			
n_p	=	2			
n_s	=	6000/300 = 20			
n_{sc}	=	6000/300 = 20			

E7.4 Design Shear Capacity of Cassettes (clause 6.2.2)

Maximum shear flow $T_v = \dfrac{8.43E}{B_u^2} \sqrt[4]{I_1 \, t^9}$

Panel 1 $\dfrac{V_{buc}}{L} = \dfrac{8.43 \times 210 \times 10^3}{600^2} \sqrt[4]{19 \times 1.21^9} = 15.8$ N/mm(kN/m) > 11.25

Panel 2 $\dfrac{V_{buc}}{L} = \dfrac{8.43 \times 210 \times 10^3}{600^2} \sqrt[4]{15 \times 0.90^9} = 8.8$ N/mm(kN/m) > 7.5

Panel 3 $\dfrac{V_{buc}}{L} = \dfrac{8.43 \times 210 \times 10^3}{600^2} \sqrt[4]{11 \times 0.71^9} = 4.1$ N/mm(kN/m) > 3.75

The design of the cassettes is satisfactory for strength.

E7.5 Design Strength of Fasteners

Fasteners to supporting members: $F_p = 1.9 \, f_u \, d_n t$ (table 5.1)

The comment to clause 6.2.2 suggests that at least 3 fasteners are required at each end of the cassette.

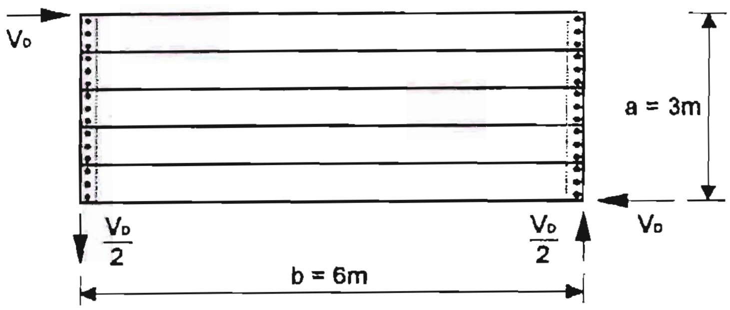
Panel 1 $F_p = 1.9 \text{ x } 0.390 \text{ x } 6.3 \text{ x } 1.21 = 5.65$ kN

Panel 2 $F_p = 1.9 \text{ x } 0.390 \text{ x } 6.3 \text{ x } 0.96 = 4.48$ kN

Panel 3 $F_p = 1.9 \text{ x } 0.390 \text{ x } 6.3 \text{ x } 0.71 = 3.31$ kN

Strength with respect to fastener failure

$V_{ult} = 2 \text{ x } 15 \text{ x } F_p$



Panel 1 $V_{ult} = 2 \text{ x } 15 \text{ x } 5.65 = 169$ kN > 67.5 kN

Panel 2 $V_{ult} = 2 \text{ x } 15 \text{ x } 4.48 = 134$ kN > 45.0 kN

Panel 3 $V_{ult} = 2 \text{ x } 15 \text{ x } 3.31 = 99.3$ kN > 22.5 kN

The design of these fasteners is satisfactory

Seam fasteners: $F_s = 3.2\,(t/d_n)^{½}\, f_u d_n t$ (table 5.1)

Panel 1 $F_s = 3.2\,(1.21/4.8)^{½} \times 0.390 \times 4.8 \times 1.21 = 3.64$ kN

Panel 2 $F_s = 3.2\,(0.96/4.8)^{½} \times 0.390 \times 4.8 \times 0.96 = 2.57$ kN

Panel 3 $F_s = 3.2\,(0.71/4.8)^{½} \times 0.390 \times 4.8 \times 0.71 = 1.64$ kN

Strength with respect to failure of seam fasteners: $V_{ult} = n_s\, F_s + \frac{\beta_1}{\beta_3}\, n_p\, F_p$

Conservatively, take $\beta_1 = 0.30$ (table 5.2 for sheeting with $n_f = 3$)
$\beta_3 = 0.667$ so that $\beta_1/\beta_3 = 0.45$

Panel 1 $V_{ult} = 20 \times 3.64 + 0.45 \times 2 \times 5.65 = 77.9$ kN $>$ 67.5 kN

Panel 2 $V_{ult} = 20 \times 2.57 + 0.45 \times 2 \times 4.49 = 55.4$ kN $>$ 45.0 kN

Panel 3 $V_{ult} = 20 \times 1.64 + 0.45 \times 2 \times 3.31 = 35.8$ kN $>$ 22.5 kN

This is satisfactory.

E7.6 Deflections at Serviceability Limit State

At the serviceability limit state, load factor $\gamma_Q = 1.0$

The design loads are, therefore:

	Panel 1	Panel 2	Panel 3
Shear load V (kN)	45.0	30.0	15.0
Shear flow T_v (kN/m)	7.5	5.0	2.5

Limitation of the shear flow according to clause 6.2.2(2)

$$T_v \leq \frac{2000L\ B_u}{375\ell_s\ (B-B_u)}$$

with L = 6000mm = length of diaphragm ϵ the direction of span of the cassettes

ℓ_s = 300mm = spacing of seam fastener

B = 3000mm = width of diaphragm (ΣB_u)

Panel 1 $$\frac{2000 \times 6000 \times 600}{375 \times 300 \times (3000-600)} = 26.7\text{N/mm(kN/m)} > 7.5\text{kN/m}$$

Annex E.36

Deflections according to table 5.5 (cantilever diaphragm with cassette spanning perpendicular to span of the diaphragm)

$$c_{1.2} = \frac{2 \times 3000 \times 1.3}{210 \times 1.21 \times 6000} = \mathbf{0.005\ mm/kN}$$

$$c_{2.1} = \frac{2 \times 3000 \times 0.15 \times 200}{6000^2} = \mathbf{0.005\ mm/kN}$$

$$c_{2.2} = \frac{2 \times 0.30 \times 0.15\ (5-1)}{2 \times 20 \times 0.15 + 0.30 \times 2 \times 0.30} = \mathbf{0.058\ mm/kN}$$

$$c_{2.3} = \frac{2 \times 0.15}{20} = \mathbf{0.015\ mm/kN}$$

$$c' = \mathbf{0.083\ mm/kN}$$

Panel 1 $\Delta_1 = 67.5 \text{ x } 0.083 = 5.6\text{mm} = \text{height}/536$

Panel 2 $\Delta_2 \approx 45.0 \text{ x } 0.083 = 3.7\text{mm}$

Panel 3 $\Delta_3 \approx 22.5 \text{ x } 0.083 = 1.9\text{mm}$

Total deflection at roof $= 11.2\text{mm} = \text{height}/804$

The design is satisfactory.

E8 THE USE OF A SHEAR PANEL TO STABILIZE HOT ROLLED SECTIONS

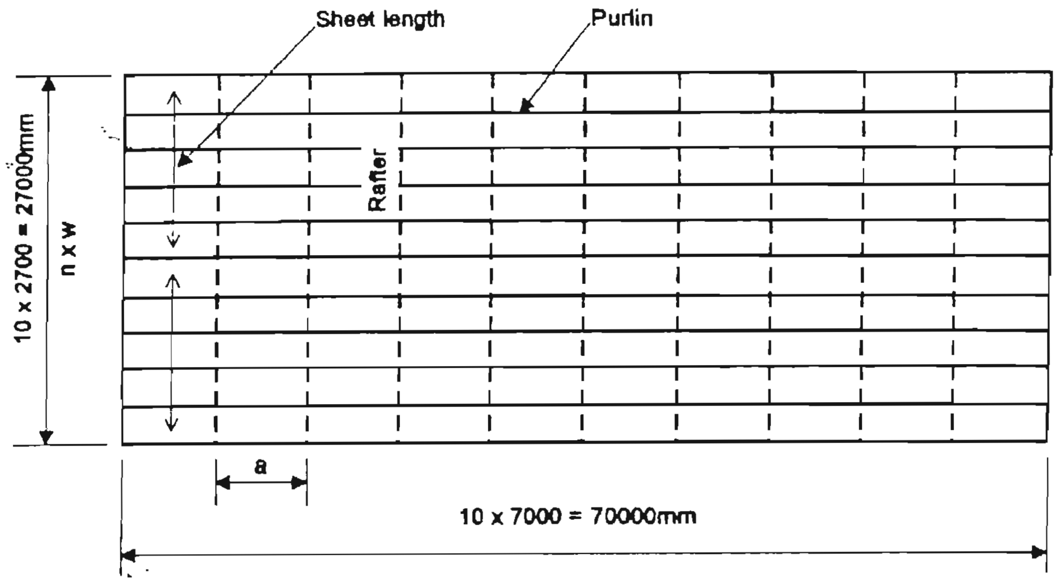
This example illustrates the use of shear panels to stabilize hot rolled sections. It shows the application of section 9 and Annex B

E8.1 Shear panel used for the stabilisation of a hot rolled section purlin.

Although it was stated in Chapter 9 that conventional diaphragm action is sufficient to stabilise any hot-rolled I-section of height less than 200mm, the check of such a case is presented here. This serves to illustrate the procedure for the stability calculation and also to demonstrate that the above statement implies an adequate reserve of safety.

E8.1.1 Problem

To check the stability of I-section purlins stabilised by diaphragm action. The general arrangement is shown below:



E8.1.2 Data

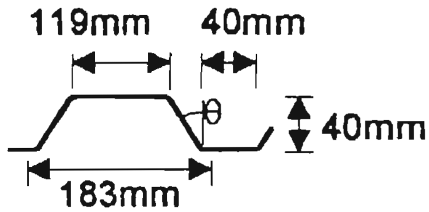
Purlins: profile IPE 180 with $f_y = 0.235/1.1 = 0.214$ kN/mm^2

Sheeting: profile E40 with: $f_y = 0.280/1.1 = 0.254$ kN/mm^2
$f_u = 0.400$ kN/mm^2

t = 0.70mm net of coatings

5 corrugations in a sheet width

2 sheet lengths in depth of diaphragm



Sheet to purlin fasteners:	6.3mm diameter self-drilling self-tapping screws fastened in every trough at the sheet ends and in alternate troughs at intermediate purlins.
Seam fasteners:	4.8mm diameter blind rivets at 500mm centres.

Gravity load case:

Dead load (sheeting + purlin + insulation)	$= 0.40\ kN/m^2$
Imposed load	$= 1.00\ kN/m^2$
Load factor dead load γ_G	$= 1.35$
Load factor imposed load γ_Q	$= 1.5$
Design load $= 1.35 \times 0.40 + 1.5 \times 1.0$	$= 2.04\ kN/m^2$

Uplift load case:

Wind suction	$= 1.00\ kN/m^2$
Load factor for wind γ_Q	$= 1.5$
Load factor for dead load γ_G	$= 1.0$
Design load $= 1.5 \times 1.00 - 1.0 \times 0.40$	$= 1.10\ kN/m^2$

E8.1.3 Values of Symbols for Diaphragm Calculation

$a = 7000mm$
$A = 2390mm^2$
$b = 27000mm$
$d = 183mm$
$E = 210\ kN/mm^2$
$F_p = 3.0\ kN$
$F_s = 1.5\ kN$
$h = 40mm$
$K_1 = 0.208$
$\ell = 119mm$
$n_b = 2$
$n_f = 3$ (at intermediate purlins)
$n_p = 11$
$n_s = \dfrac{27000}{500} = 54$

$n_{sh} = 7.65$
$p = 366mm$ (at intermediate purlins)
$s_p = 0.15\ mm/kN$
$s_s = 0.30\ mm/kN$
$t = 0.70mm$
$\alpha_1 = 0.60$ (6 purlins per sheet length)
$\alpha_2 = 0.33$ (11 purlins per panel)
$\alpha_3 = 0.45$
$\alpha_4 = 1 + 0.3 \times 2 = 1.6$
$\beta_1 = 1.0$
$\beta_3 = 1.0$ (seam fasteners in troughs)

Maximum span bending moments:

Gravity: $M_{b,Sd}$ $9/128 \times 2.7 \times 2.04 \times 7^2 = 19.0\ kNm$

Uplift: $M_{b,Sd}$ $9/128 \times 2.7 \times 1.10 \times 7^2 = 10.2\ kNm$

Note: For the stability checks of the purlins, they are conservatively assumed to be simply supported over a span of 7.0m and subject to a constant bending moment equal to the maximum simply supported bending moment.

E8.1.4 Design Shear Capacity.

E8.1.4.1 Stiffness of shear panel (Clause B.III.1.1)

$$c_{1.1} = \frac{7000 \times 183^{2.5} \times 0.6 \times (1+0.3 \times 2) \times 0.208}{210 \times 0.7^{2.5} \times 27000^2} = \mathbf{0.010\ mm/kN}$$

$$c_{1.2} = \frac{2 \times 7000 \times 0.33 \times (1 + 0.3) \times [1+(2 \times 40/183)]}{210 \times 0.7 \times 27000} = \mathbf{0.002\ mm/kN}$$

$$c_{2.1} = \frac{2 \times 7000 \times 0.15 \times 366 \times 0.45}{27000^2} = \mathbf{0.000\ mm/kN}$$

$$c_{2.2} = \frac{2 \times 0.30 \times 0.15 \times (7.65 - 1)}{2 \times 54 \times 0.15 + 1.0 \times 11 \times 0.30} = \mathbf{0.031\ mm/kN}$$

$$c = \mathbf{0.043\ mm/kN}$$

Actual Shear Stiffness:

$$S_{act} = \frac{7000}{0.043 \times (10 + 1)} = 14800\ \text{kN}$$

E8.1.4.2 Strength of shear panel (Clause B.III.1.2)

$$P_{max} = \frac{54 \times 1.5 + \frac{1.0}{1.0} \times (10 + 1) \times 3.0}{10} = 11.4\ \text{kN}$$

Check for other failure modes:

a) Sheet/purlin fastener strength

$$\frac{2700}{0.45 \times 366} \times 0.6 \times 3.0 = 29.5\ \text{kN} > P_{max}$$ Satisfactory

b) End collapse of sheeting profile

$$\frac{0.9 \times 0.7^{1.5} \times 2700 \times 280/1.1 \times 10^{-3}}{183^{0.5}} = 26.8\ \text{kN} > P_{max}$$ Satisfactory

Annex E.40

c) Shear buckling

$$D_x = \frac{210 \times 10^3 \times 0.7^3 \times 183}{12 \times (1 - 0.3^2)(119 + 40 + 2 \times 41.8)} = 4976 \quad Nmm$$

$$D_y = \frac{210 \times 10^3 \times 246 \times 183}{183} = 51.7 \times 10^6 \quad Nmm$$

Where I = 246 mm⁴/mm

$$V_g = \frac{14.4}{27000} \times 4976^{0.25} \times (51.7 \times 10^6)^{0.75} \times (11 - 1)^2 = 273 \times 10^3 \ N = 273 \quad kN$$

$$V_l = 4.83 \times 210 \times (\frac{0.7}{119})^2 \times 27000 \times 0.7 = 663 \quad kN$$

$$V_{red} = \frac{1}{\frac{1}{273} + \frac{1}{663}} = 193 \quad kN > 10 \times 11.4$$ Satisfactory

E8.1.4.3 Stiffness check of shear panel (Clause B.II)

$$S_y = \frac{235/1.1 \times 2390}{2} = 255 \times 10^3 \ N = 255 \quad kN < S_{act}$$ Satisfactory

Therefore purlin is fully stabilized.

Design strength of purlin is thus:

$$M_p = 1.15 \text{ x } 146 \text{ x } 10^3 \text{ x } 235/1.1 = 35.9 \text{ x } 10^6 \text{ Nmm}$$

$$= 35.9 \text{ kNm} > M_{b,Sd}$$ Satisfactory

Annex E.41

E8.1.4.4 Strength check of shear panel (Clause B.II)

a) Gravity Load

$$k_1 = \frac{19.0 \times 10^6 \times 255 \times 10^3 \times 180}{5926} = 0.147 \times 10^{12}$$

$$k_2 = -\frac{(19.0 \times 10^6)^2}{5926} = -6.09 \times 10^{10}$$

$$W_w = -\frac{1}{5926}\left(\frac{(210 \times 10^3 \times 1/4 \times 101 \times 10^4 \times 180^2 \times \pi^2}{7000^2} + \right.$$

$$\left. + 0.8 \times 10^5 \times \left(\frac{2 \times 91 \times 8^3 + (180 - 2 \times 8)}{3}\right) + 255 \times 10^3 \times 90^2 \right) =$$

$$= -0.827 \times 10^6$$

$$W_z = -\left(\frac{210 \text{ x } 10^3 \text{ x } 101 \text{ x } 10^4 \text{ x } \pi^2}{7000^2} + 255 \text{ x } 10^3\right) = -0.298 \text{ x } 10^6$$

Hence:

$$\psi_1 = -0.80$$

$$\psi_2 = 3.22$$

From EC3 Part 1.1 chapter 5.2.4.4

$$n_r = 11$$

$$k_r = \left(0.2 + \frac{1}{11}\right)^{0.5} = 0.539$$

$$e_0 = 0.539 \text{ x } \left(\frac{7000}{500}\right) = 7.55 \text{ mm}$$

$$T_d = \pi \times \frac{1}{3.22 - 1} \times 255 \times \frac{7.55}{7000} = 0.39 \text{ kN}$$

Annex E.42

Requirements:

$T_d < P_{max}$ i.e. $0.39\ kN < 11.4\ kN$ Satisfactory

$T_d < \frac{L}{8 \times p} \times F_p$ i.e. $0.39\ kN < \frac{7000 \times 3.0}{8 \times 366} = 7.2\ kN$ Satisfactory

b) Uplift Load

$$k_1 = -\left(\frac{10.2 \times 10^6 \times 255 \times 10^3 \times 180}{5926}\right) = -7.90 \times 10^{10}$$

$$k_2 = -\frac{(10.2 \times 10^6)^2}{5926} = -1.76 \times 10^{10}$$

$W_Z = -0.298 \times 10^6$

$W_w = -0.827 \times 10^6$

Hence:

$\psi_1 = -6.60$

$\psi_2 = 2.12$

The value of e_0 is as for gravity load

Hence:

$$T_d = \pi \times \frac{1}{2.12 - 1} \times 255 \times \frac{7.55}{7000} = 0.77\ kN$$

Requirements:

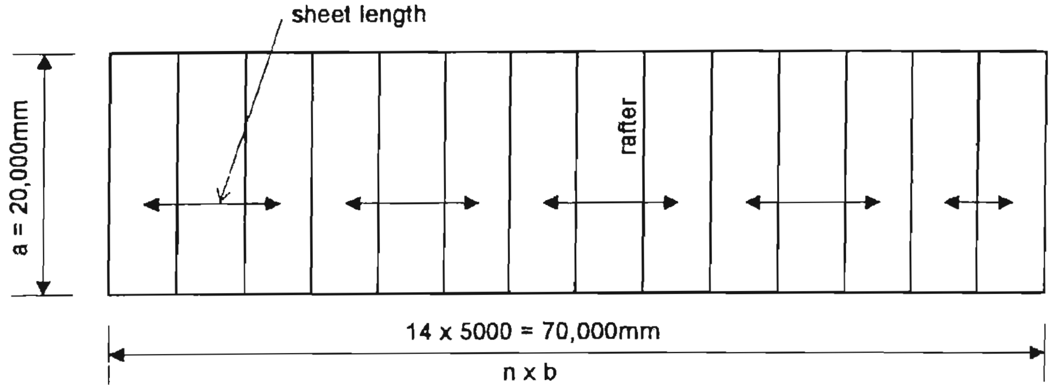
$T_d < P_{max}$ i.e. $0.77\ kN < 11.4\ kN$ Satisfactory

$T_d < \frac{L}{8 \times p} \times F_p$ i.e. $0.77\ kN < \frac{7000 \times 3.0}{8 \times 366} = 7.2\ kN$ Satisfactory

E8.2 Shear panel used to stabilize hot rolled rafters.

E8.2.1 Problem

To check the stability of rafter sections stabilised by diaphragm action. The general arrangement is shown below:



E8.2.2 Data

Purlins: profile IPE 550 with $f_y = 0.235/1.1 = 0.213$ kN/mm^2

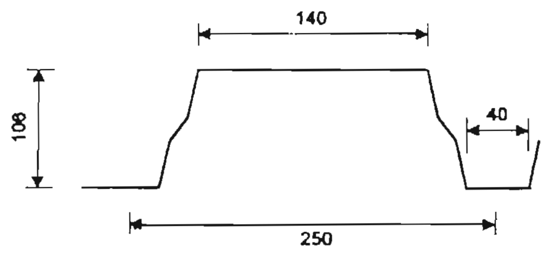
Sheeting: profile E106 with: $f_y = 0.280/1.1 = 0.254$ kN/mm^2

$f_u = 0.400$ kN/mm^2

$t = 0.70$mm net of coatings

3 corrugations in a sheet width

2 sheet lengths in depth of diaphragm



Sheet to rafter fasteners: 4.5mm diameter powder actuated fasteners in every trough.

Seam fasteners: 4.5mm diameter blind rivets at 500mm centres.

Annex E.44

Critical load case (gravity):

Dead load (sheeting + purlin + insulation)	= 0.30 kN/m^2
Imposed load	= 0.50 kN/m^2
Dead Load factor γ_G	= 1.35
Imposed Load factor γ_Q	= 1.5
Design load = 1.35 x 0.3 + 1.5 x 0.5	= 1.16 kN/m^2

E8.2.3 Values of Symbols for Diaphragm Calculation

a = 20000mm
A = 13400mm^2
b = 5000mm
d = 250mm
E = 210 kN/mm^2
F_p = 3.2 kN
F_s = 1.5 kN
h = 106mm
K_1 = 0.416
ℓ = 140mm
n_f = 4
$n_s = \frac{5000}{500} - 1 = 9$

n_{sh} = 26.7
p = 250mm
s_p = 0.10 mm/kN
s_s = 0.30 mm/kN
t = 0.70mm
α_5 = 0.70 (5 sheet lengths per diaphragm)
β_1 = 1.04

Maximum span bending moment:

Single span rafter: $M_{b,Sd}$ 1/8 x 5.0 x 1.16 x 20^2 = 290.0 kNm

Additional compression force: $N_{c,Sd}$ = 200 kN

Note: For the stability checks of the rafter, it is conservatively assumed subject to a constant bending moment equal to the maximum simply supported bending moment.

E8.2.4 Design shear capacity

E8.2.4.1 Stiffness of shear panel (Clause B.III.2.1)

$$c_{1.1} = \frac{20000 \times 250^{2.5} \times 0.7 \times \times 0.416}{210 \times 0.7^{2.5} \times 5000^2} = \mathbf{2.67\ mm/kN}$$

$$c_{1.2} = \frac{2 \times 20000 \times (1 + 0.3) \times [1 + (2 \times 106/250)]}{210 \times 0.7 \times 5000} = \mathbf{0.13\ mm/kN}$$

$$c_{2.1} = \frac{2 \times 20000 \times 0.10 \times 250}{5000^2} = \mathbf{0.04\ mm/kN}$$

Annex E.45

$$c_{2.2} = \frac{0.30 \times 0.10 \times (26.7 - 1)}{9 \times 0.10 + 1.04 \times 0.30} = 0.636 \text{ mm/kN}$$

$$c = 3.476 \text{ mm/kN}$$

Actual shear stiffness:

$$S_{act} = \frac{5000}{3.476} = 1438 \text{ kN}$$

E8.2.4.2 Strength of shear panel (Clause B.III.2.2)

$$P_{max} = 9 \times 1.5 + \frac{1.04}{1.0} \times 3.2 = 16.8 \text{ kN}$$

Check for other failure modes:

a) Sheet/rafter fastener strength

$$\frac{5000}{250} \times 0.6 \times 3.2 = 38.4 \text{ kN} > P_{max}$$ Satisfactory

b) End collapse of sheeting profile

$$\frac{0.9 \times 0.7^{1.5} \times 5000 \times 280/1.1 \times 10^{-3}}{250^{0.5}} = 42.4 \text{ kN} > P_{max}$$ Satisfactory

c) Shear buckling

$$D_x = \frac{210 \times 10^3 \times 0.7^3 \times 250}{12 \times (1 - 0.3^2)(403)} = 4092 \text{ Nmm}$$

$$D_y = \frac{210 \times 10^3 \times 1803 \times 250}{250} = 378.6 \times 10^6 \text{ Nmm}$$

Where $I = 1803 \text{ mm}^4/\text{mm}$

Annex E.46

$$V_g = \frac{28.8 \times 20000}{5000} \times 4092^{0.25} \times (378.6 \times 10^6)^{0.75} = 500 \times 10^3 \text{ N} = 500 \text{ kN}$$

$$V_l = 4.83 \times 210 \times (\frac{0.7}{140})^2 \times 20000 \times 0.7 = 355 \text{ kN}$$

$$V_{red} = \frac{1}{\frac{1}{500} + \frac{1}{355}} = 208 \text{ kN}$$

$$\frac{208 \times 13400}{20000} = 52 \text{ kN} > P_{max}$$ Satisfactory

E8.2.4.3 Stiffness check of shear panel (Clause B.II)

$$S_y = \frac{235/1.1 \times 13400}{2} = 1441 \times 10^3 \text{ N} = 1441 \text{ kN} > S_{act}$$ Unsatisfactory

Therefore rafter is not fully stabilized.

Stability check of rafter without full stabilization:

$$k_1 = [-200 \times 10^3 \times (-1.58 \times 10^6 - 4.20 \times 10^6)$$

$$- 290 \times 10^6 \times \frac{(1438 \times 10^3 \times 550)}{52082}] = 3.25 \times 10^{12}$$

$$k_2 = \left[(-200 \times 10^6)^2 - \frac{(300 \times 10^6)^2}{52082}\right] = -1.57 \times 10^{12}$$

$$W_z = -\left[\frac{210 \times 10^3 \times 2670 \times 10^4 \times \pi^2}{20000^2} + 1438 \times 10^3\right] = -1.58 \times 10^6$$

$$W_w = -\frac{1}{52082}(\frac{210 \times 10^3 \times 1/4 \times 2670 \times 10^4 \times 550^2 \times \pi^2}{20000^2} +$$

$$+ 0.8 \times 10^5 \times \left(\frac{2 \times 210 \times 17.2^3 + (550 - 2\times17.2) \times 11.1^3}{3}\right)$$

$$+ 1438 \times 10^3 \times 275^2) = -4.20 \times 10^6$$

Hence:

$$\psi_1 = +2.88$$

$$\psi_2 = 0.80$$

$$M_{cr} = 2.88 \times 290 \times 10^6 \text{ Nmm} = 835 \text{ kNm}$$

$$N_{cr} = 2.88 \times 200 \times 10^3 \text{ N} = 576 \text{ kN}$$

Since an IPE 550 member is a Class 1 section, the equations (5.51) and (5.52) of EC3 Part 1.1 apply:

$$\gamma_{M1} = 1.1$$

$$f_y = 235 \text{ N/mm}^2$$

$$W_{pl.y} = 1.15 \times 2440 \times 10^3 \text{ mm}^3$$

$$\bar{\lambda} = \frac{\lambda}{\bar{\lambda}} = \frac{\dfrac{20000}{\sqrt{\dfrac{67210 \times 10^4}{13400}}}}{93.9} = 0.952 > \chi_y = 0.698$$

$$\beta_{My} = 1.3$$

$$\mu_y = 0.952 (2 \times 1.3 - 4) + \frac{1.15 - 1.0}{1.0} = -1.18$$

$$k_y = 1 - \frac{-1.18 \times 200 \times 10^3}{0.698 \times 13400 \times 235} = 1.11$$

$$\lambda_z = \sqrt{\frac{13400 \times 235}{576 \times 10^3}} = 2.34 \quad \text{Hence } \chi_z = 0.157$$

Annex E.48

Substituting into equation (5.51) gives:

$$\frac{200 \times 10^3}{0.157 \times 13400 \times \dfrac{235}{1.1}} + \frac{1.11 \times 290 \times 10^6}{1.15 \times 2440 \times 10^3 \times \dfrac{235}{1.1}} = 0.98$$

$\leq$ 1.0 Satisfactory

$$\overline{\lambda_{LT}} = \sqrt{W_{pl,y}\, f_y / M_{cr}} = 0.829 \quad > \quad \chi_{LT} = 0.778$$

$$\mu_{LT} = 0.15 \text{ x } 2.34 \text{ x } 1.3 - 0.15 = 0.306$$

$$k_{LT} = 1 - \frac{0.306 \times 200 \times 10^3}{0.157 \times 13400 \times 235} = 0.876$$

Substituting into equation (5.52) gives:

$$\frac{200 \times 10^3}{0.157 \times 13400 \times \dfrac{235}{1.1}} + \frac{0.876 \times 290 \times 10^6}{0.778 \times 1.15 \times 2440 \times 10^3 \times \dfrac{235}{1.1}} = 0.99$$

$\leq$ 1.0 Satisfactory

E8.2.4.4 Strength check of shear panel (Clause B.II)

From EC3 Part 1.1 chapter 5.2.4.4

$$n_r = 15$$

$$k_r = \left(0.2 + \frac{1}{15}\right)^{0.5} = 0.516$$

$$e_0 = 0.516 \text{ x } \left(\frac{20000}{500}\right) = 20.7 \text{ mm}$$

$$T_d = \pi \times \frac{1}{2.88 - 1} \times 1438 \times \frac{20.7}{20000} = 2.49 \text{ kN}$$

Requirements:

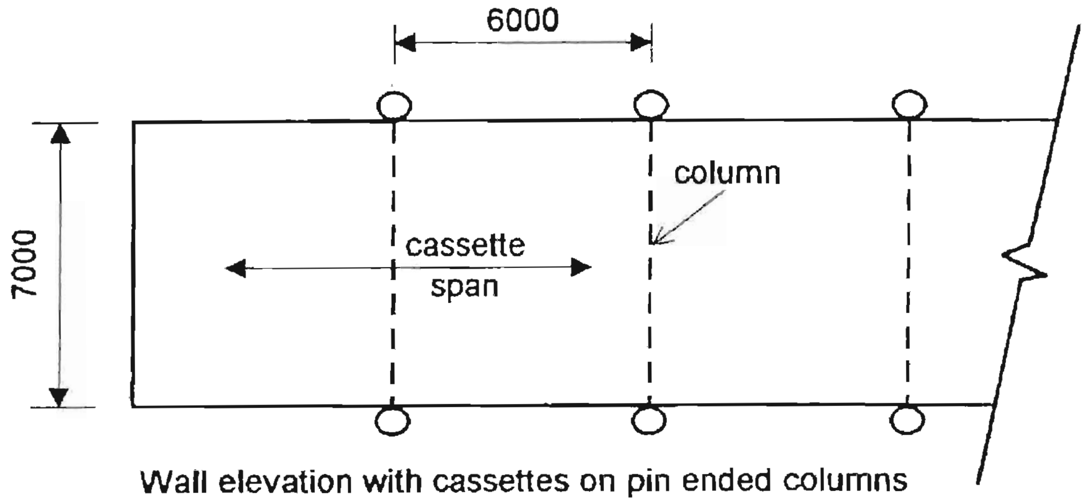
$T_d < P_{max}$ i.e. 2.49 kN < 16.8 kN Satisfactory

$T_d < \dfrac{L}{8 \times p} \times F_p$ i.e. 2.49 kN < $\dfrac{20000 \times 3.2}{8 \times 250} = 32.0$ kN Satisfactory

E8.3 Cassettes used to stabilize columns.

E8.3.1 **Problem**

To check the stability of pin ended columns which are stabilised by the diaphragm action of cassette cladding. The general arrangement is show below:



Wall elevation with cassettes on pin ended columns

E8.3.2 **Data**

Columns: profile IPE 120 with $f_y = 0.235/1.1 = 0.213$ kN/mm^2

Cassettes: profile 90/600 - 0.75

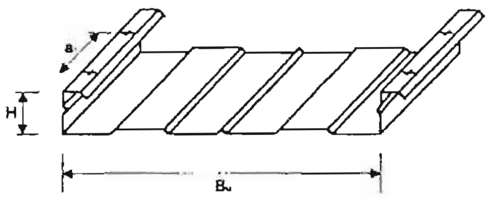
$f_y = 0.320/1.1 = 0.291$ kN/mm^2
$f_u = 0.390$ kN/mm^2

$t = 0.71$mm net of coatings

$B_u = 600$mm

$H = 90$mm

$I_1 = 11.4$mm^4/mm



Cassette to column 3 screws, 6.3mm diameter in each cassette at a pitch of 250mm:

Seam fasteners: 4.8mm diameter blind rivets at 300mm centres.

Factored Column loading:

$$N_{Sd} = 100 \text{ kN}$$

E8.3.3 Values of Symbols for Cassette Calculation

a	=	2×10^3 N/mm	s_p	=	0.15 mm/kN
B	=	7000mm	s_s	=	0.30 mm/kN
L	=	6000mm			
e_s	=	300mm			
E	=	210 kN/mm^2			
F_p	=	3.0 kN			
F_s	=	1.5 kN			

E8.3.4 Design shear capacity

E8.3.4.1 Stiffness of shear panel (Clause 6.2.2.2)

$$S_{act} = \frac{2 \times 10^3 \times 6000\ 600}{300(\ 7000 - 600\ )} \times 6000 = 22.5 \times 10^6 \text{ N} = 22500 \text{ kN}$$

E8.3.4.2 Strength of shear panel (Clause 6.2.2.1)

i) Check fastener strength is critical:

$$P_{max} = \frac{6000}{300} \times 1.5 + 0.30 \times 3.0 = 30.9 \text{ kN}$$

ii) Check local buckling of the wide flange:

$$V_{buc} = 8.43 \times \frac{210}{600^2} \times \sqrt[4]{11.2 \times 0.71^9} \times 6000 = 25.0 \text{ kN}$$

E8.3.4.3 Stiffness check of shear panel (Clause B.II)

$$S_y = \frac{235/1.1 \times 1320}{2} = 141 \times 10^3 \text{ N} = 141 \text{ kN} < S_{act}$$

Therefore column is fully stabilized

Annex E.51

Design strength $N_{c,Rd}$ of column:

Buckling about the z axis; (fully stabilized by the cassettes):

$$N_{c,Rd} = 1350 \times 235 / 1.1 = 288 \times 10^3 \text{ N} = 288 \text{ kN}$$

> 100 kN Satisfactory

Buckling about the y axis;

Buckling length = 7000mm

$$\lambda_y = \frac{7000}{49} = 143$$

$$\lambda_1 = 93.9$$

$$\overline{\lambda}_y = \frac{143}{93.9} = 1.52$$

Reduction factor χ_y = 0.365 (according to table 5.5.2 of EC3)

$$N_{c,Rd} = 0.365 \times 1350 \times 235 / 1.1 = 103 \times 10^3 \text{ N} = 103 \text{ kN}$$

> 100 kN Satisfactory

E8.3.4.4 Strength check of shear panel (Clause B.II)

$$W_z = -\left[\frac{210 \times 10^3 \times 27.7 \times 10^4 \times \pi^2}{7000^2} + 0.141 \times 10^6\right] = -0.153 \times 10^6$$

$$W_w = -\frac{1}{2611}\left(\frac{210 \times 10^3 \times 1/4 \times 27.7 \times 10^4 \times 120^2 \times \pi^2}{7000^2} + \right.$$

$$+ 80 \times 10^3 \times \left(\frac{2 \times 64 \times 6.3^3 + (120 - 2\times 6.3) \times 4.4^3}{3}\right)$$

$$\left. + 0.141 \times 10^6 \times 60^2 \right) = -0.687 \times 10^6$$

$$k_1 = -[-100 \times 10^3 (-0.153 \times 10^6 - 0.687 \times 10^6)] = -8.40 \times 10^{10}$$

$$k_2 = (-100 \times 10^3)^2 = 10^{10}$$

Hence:

$$\psi_1 = 7.34$$

$$\psi_2 = 1.06$$

From EC3 Part 1.1 chapter 5.2.4.4

$$n_r = 4$$

$$k_r = \left(0.2 + \frac{1}{4}\right)^{0.5} = 0.671$$

$$e_0 = 0.671 \text{ x } \left(\frac{7000}{500}\right) = 9.39 \text{ mm}$$

$$T_d = \pi \times \frac{1}{1.06 - 1} \times 155 \times \frac{9.39}{7000} = 10.9 \text{ kN}$$

Requirements:

$T_d < P_{max}$ i.e. $10.9 \text{ kN} < 30.9 \text{ kN}$ Satisfactory

$T_d < V_{buc}$ i.e. $10.9 \text{ kN} < 25.0 \text{ kN}$ Satisfactory

$T_d < \frac{L}{8 \times p} \times F_p$ i.e. $10.9 \text{ kN} > \frac{7000 \times 3.0}{8 \times 250} = 10.5 \text{ kN}$ Not Satisfactory

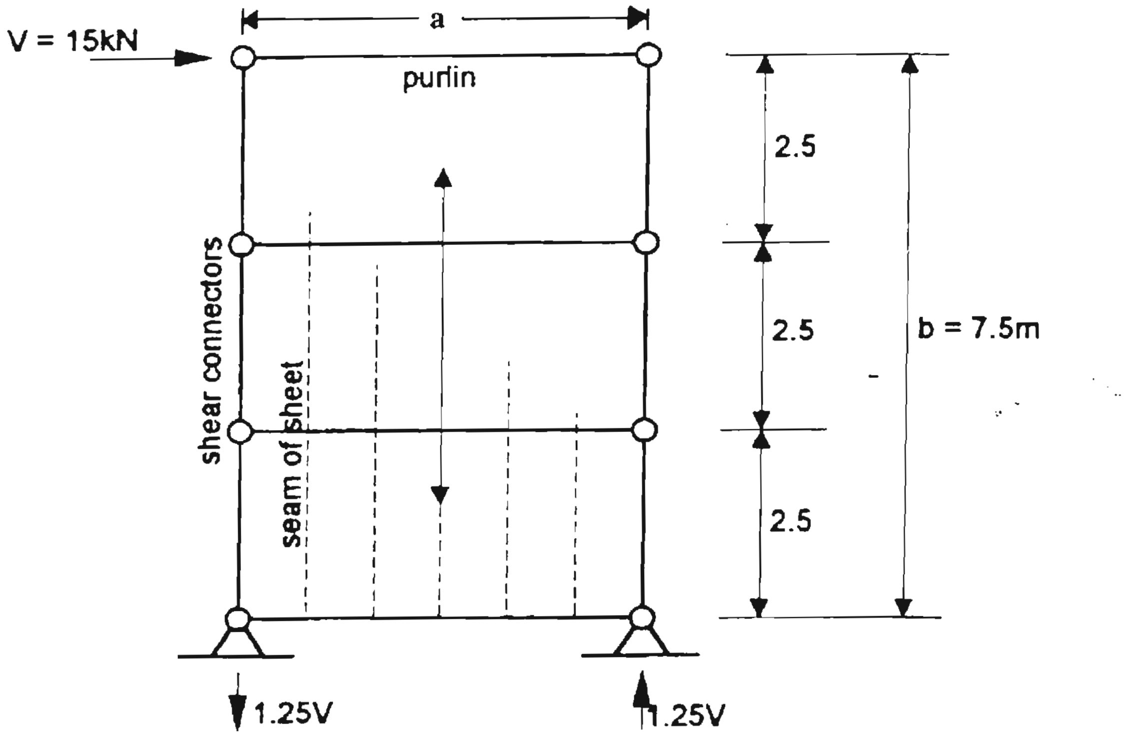
So the force $T_d = 10.9$ kN should be introduced into the edge members by 4 - 6.3mm ϕ screws per span of 6000mm (screw capacity = 4 x 3.0 = 12.0 kN)

E9 DIAPHRAGM OF ALUMINIUM

E9 This example illustrates the calculation of design shear capacity and the shear flexibility of a single shear panel using trapezoidally profiled aluminium sheeting. It shows the application of section 6.

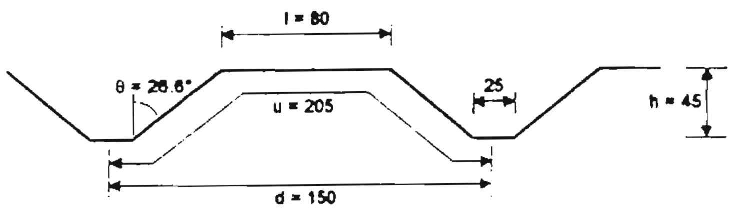
E9.1 Problem

To calculate the shear strength and shear stiffness of aluminium panels.



E9.2 Data

Decking: Aluminium sheet 45/150



t=0.8mm
f_y = 180 N/mm^2
R_m = 240 N/mm^2
Note that f_y is the design value (including γ_M). γ_M = 1.25 is assumed for fasteners in aluminium sheets.

6 corrugations per metre.

Sheet/purlin fasteners: 5.5 mm diameter self tapping screws at 300 mm centres in intermediate purlins and at 150 mm centres in end purlins. Refer to Table 5.8 case (3).

Seam fasteners:	4.8 mm diameter aluminium blind rivets, at 500 mm centres.
Shear connectors:	5.5 mm diameter self drilling screws, at 500 mm centres.

E9.3 Values of symbols

a	=	6,000mm	n_{sh}	=	6
A	=	760000mm^2	p	=	150 (every trough fastened
b	=	7,500mm	p	=	300 (alt. troughs fastened)
d	=	150mm	s_p	=	0.40mm/kN (table 5.1)
E	=	70 kN/mm^2	s_s	=	0.25mm/kN (table 5.1)
F_p	=	1.6 x 0.240 x 5.5 x 0.8/1.25 = 1.35 kN Clause 6.2.1.1	s_{sc}	=	0.4mm/kN (table 5.1)
			t	=	0.80mm
F_s	=	$1.6(0.8 \times 5.5)^{1/2} \times 0.240/1.25 \times 0.8$ = 0.52 kN	f_y	=	0.18 kN/mm^2
			α_1	=	1.0 (table 5.8)
K_1	=	0.197 (table 5.6)	α_4	=	1.0 (table 5.8)
K_2	=	1.78	β_1	=	1.33 (table 5.2)
ℓ	=	80mm	β_3	=	1.0 (5.1.1.1)
n_f	=	7 (end purlin)	ℓ/d	=	80/150 = 0.533
n_f	=	4 (intermediate purlin)	h/d	=	45/150 = 0.3
n_p	=	4			
n_s	=	3 x 9 = 27			
n_{sc}	=	16			

E9.4 Design Shear Capacity

Seam strength

$$V_{ult} = \frac{6.0}{7.5} \times \left(27.0 \times 0.52 + \frac{1.33}{1.0} \times 4 \times 1.35\right) = 17.0 \text{ kN}$$

Shear connector strength

$$V_{ult} = \frac{6.0}{7.5} \times 16 \times 1.35 = 17.3 \text{ kN}$$

Hence, design shear capacity $V^* = 17.0$ kN

Sheet/purlin fasteners - check if $\dfrac{0.6bF_p}{p\alpha_3} \geq V*$

$$\frac{0.6 \times 6000 \times 1.35}{150} = 32.4 \text{ kN} > V^*$$ Satisfactory

Annex E.55

End collapse of profile - check if $0.9t^{1.5}\ bf_y/d^{0.5} \geq V*$

$$\frac{0.9 \times 0.8^{1.5} \times 6000 \times 0.18}{150^{0.5}} = 56.8 \text{ kN} > V^*$$ Satisfactory

Global shear buckling

$$V_{ult} = \frac{14.4 \times 6000}{7500^2} \times 2.39^{0.25} \times 23404^{0.75} \times (4 - 1) = 32.5 \text{ kN}$$

D_x = 2.39 kNmm

D_y = 23404 kNmm

Local shear buckling

$$\ell/t \leq 2.9\sqrt{\frac{70}{0.180}} = 57.2$$

$$80/0.8 = 100 \nleq 57.2$$

Hence global buckling strength is reduced.

$$V_1 = 4.83 \times 70000 \times (\frac{0.8}{80})^2 \times 7500 \times 0.8 \times 10^{-3} = 202.2 \text{ kN}$$

$$V_{red} = \frac{32.5 \times 202.9}{(32.5 + 202.9)} = 28.0 \text{ kN}$$

Panel has adequate shear strength.

E9.5 Shear Flexibility

$$\text{Profile distortion } c_{1.1} = 0.5 \times \frac{6000 \times 150^{2.5} \times 1.1 \times 0.197}{70 \times 0.8^{2.5} \times (7500)^2} = 0.072 \text{ mm/kN}$$

(See Clause 6.2.1.2)

$$\text{Shear strain } c_{1.2} = \frac{2 \times 6000 \times (1 + 0.3) \times (1 + 2{\times}45/150)}{70 \times 0.8 \times 7500} = 0.059 \text{ mm/kN}$$

$$\text{Sheet/purlin fasteners } c_{2.1} = \frac{2 \times 6000 \times 0.40 \times 150}{7500^2} = 0.013 \text{ mm/kN}$$

$$\text{Seam fasteners } c_{2.2} = \frac{2 \times 0.25 \times 0.40 \times (6 - 1)}{2 \times 27 \times 0.40 + 1.33 \times 4 \times 0.25} = 0.044 \text{ mm/kN}$$

$$\text{Sheet/shear connector fasteners } c_{2.3} = \frac{2 \times 0.40}{16} = 0.050 \text{ mm/kN}$$

$$\text{Axial strain } c_3 = \frac{2 \times 7500^3}{3 \times 210 \times 760000 \times 6000^2} = 0.000 \text{ mm/kN}$$

$$c' = \left(\frac{7500}{6000}\right) \times (0.072 + 0.059 + 0.013 + 0.044 + 0.050) = 0.037 \text{ mm/kN}$$

$$c = 0.37 + 0.0 = 0.37 \text{ mm/kN}$$

$$\Delta = \frac{15}{1.5} \times 0.37 = 3.7 \text{ mm} = \frac{b}{2000}$$

Panel has adequate shear stiffness.

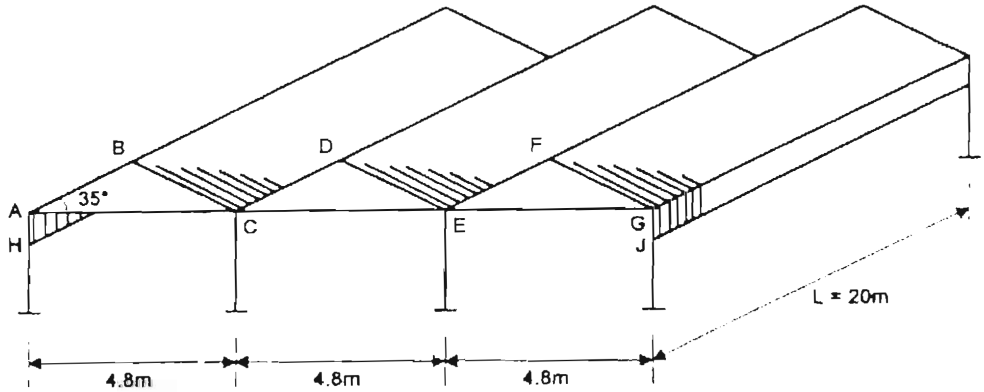
Annex E.57

E10 THE STRENGTH AND DEFLECTION OF A LIGHT GAUGE STEEL FOLDED PLATE ROOF UNDER VERTICAL LOAD

This example illustrates the case of a light gauge steel folded plate roof under vertical load. It shows the application of section 10.

E10.1 Problem

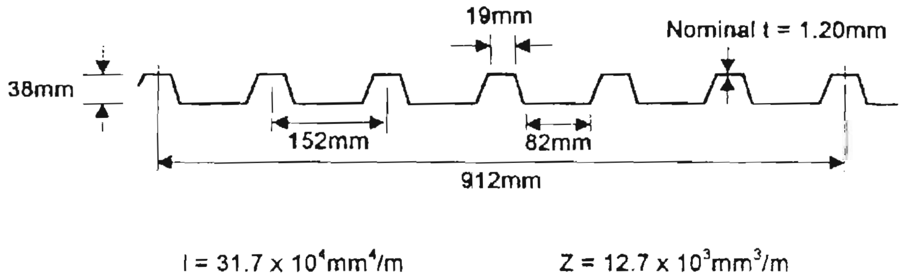
To calculate the strength and deflection of the light gauge steel folded plate roof shown under the given vertical loads.



E10.2 Data

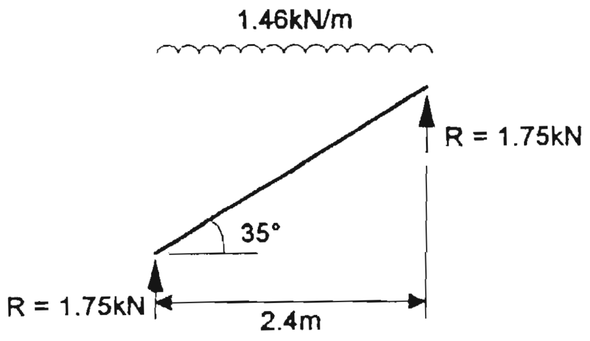
Loads: Snow load 0.70 kN/m^2, uniformly distributed in plan.
Self weight of sheeting, insulation etc. 0.30 kN/m^2.

Sheeting profile:



Grade of steel:	FeE 350 G to EN 10147 (f_u = 420 N/mm^2)
Sheet/edge member fasteners:	6.3mm self-drilling, self-tapping screws with metal washers in every trough.
Seam fasteners:	4.8mm self-drilling, self-tapping screws
Factored load:	1.35 x 0.30 + 1.5 x 0.7 = 1.46 kN/m^2

E10.3 Sheeting in Bending (per metre length)



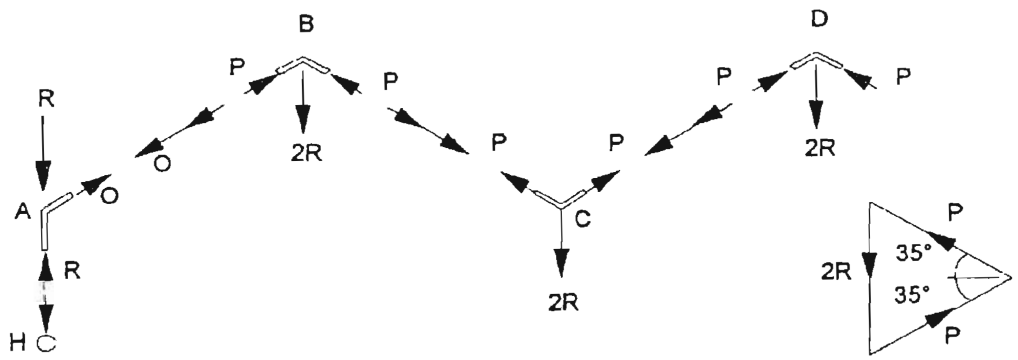
Maximum factored bending moment in sheeting $= \dfrac{1.46 \times 2.4^2}{8} = 1.05$ kNm/mm

Hence, maximum factored bending stress in sheeting $= \dfrac{1.05 \times 10^6}{12.7 \times 10^3}$ N/mm^2

$= 82.7$ N/mm^2

E10.4 Factored Loading on Fold-Line Members (per metre length)

Roof slope BC



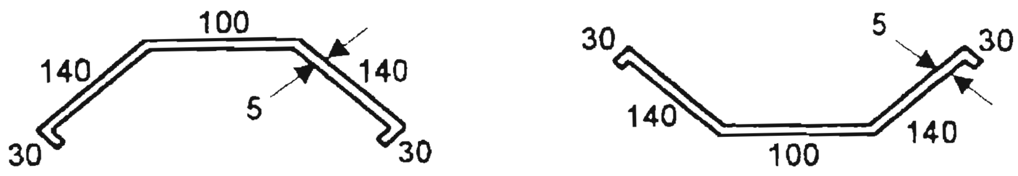
For interior roof slopes, $2R = 2P \sin 35°$

Hence, $P = \dfrac{R}{\sin 35°} = \dfrac{1.75}{0.574} = 3.05$ kN/m

Thus the factored load per metre for the design of the inclined interior plate girder BC is 2P = 6.10 kN/m.

Annex E.59

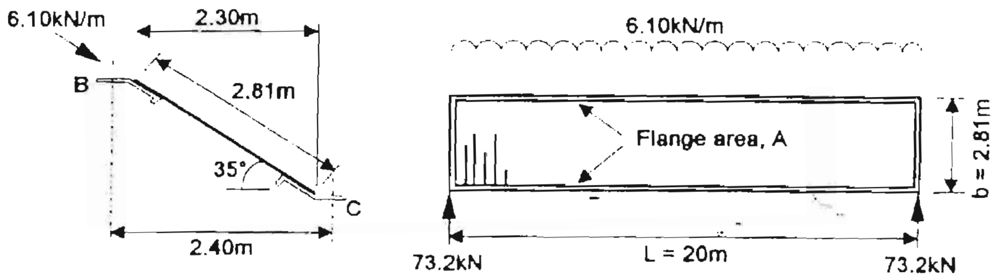
Assume the fold-line members are as shown below:



Apex member

Valley member

so that the semi-area of each is A = 1100 mm² and the dimensions of the inclined plate girder BC are girder BC are



For the inclined plate girder BC,

$$I = 2A\left(\frac{b}{2}\right)^2 = \frac{Ab^2}{2} \quad \text{and } Z = \frac{I}{b/2} = Ab = 1100 \times 2810 = 3.09 \times 10^6 \text{ mm}^3$$

$$\text{Maximum factored bending moment} = \frac{6.10 \times 24^2}{8} = 439.2 \text{ kNm}$$

so maximum factored axial stress in the edge members at B and C

$$= \frac{439.2 \times 10^6}{3.09 \times 10^6} = 142 \text{ N/mm}^2$$

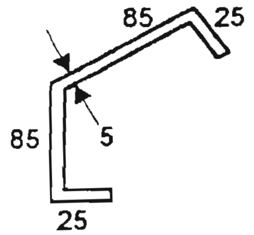
The edge members are fully restrained by the sheeting, and should be designed to Part 1.3 of Eurocode 3.

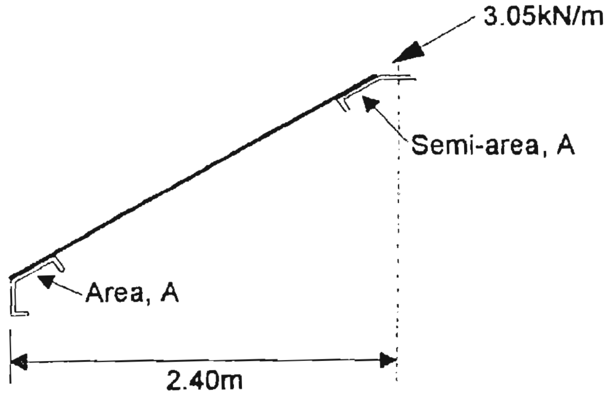
<u>Roof slope AB</u>

The factored load per metre for the design of the inclined end plate girder AB is P = 3.05 kN/m.

Assume that the fold-line member at A has the following section (area = 1100 mm²)

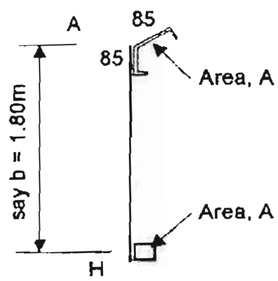
Annex E.60





The design process is similar to that for an interior roof slope, except that the in-plane load is halved. Hence, the maximum factored axial stress in the edge members = 71 N/mm^2. As before, the edge members are fully restrained by the sheeting and should be designed to Part 1.3 of Eurocode 3.

Vertical sheeting AH



From the diagram given previously, the factored load per metre for the design of the vertical plate girder AH is R = 1.75 kN/m. Assume that the member at H has a cross sectional area A of 1100 mm^2.

Then, for the vertical plate girder AH

$$I = 2A\left(\frac{b}{2}\right)^2 = \frac{Ab^2}{2}$$

and

$$Z = \frac{I}{b/2} = Ab = 1100 \times 1800 = 1.98 \times 10^6 \text{ mm}^3$$

$$\text{Maximum factored bending moment} = \frac{1.75 \times 24^2}{8} = 126 \text{ kNm}$$

so maximum factored axial stress in the edge members

$$= \frac{126 \times 10^6}{1.98 \times 10^6} = 64 \text{ N/mm}^2$$

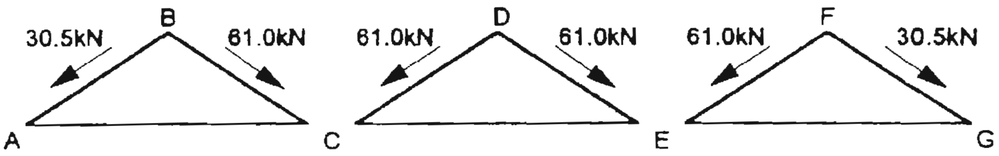
The combined axial stress in the edge member at A is therefore 71 + 64 = 135 N/mm^2.

Annex E.61

The edge member at A is fully restrained by the sheeting, but the edge member at H is restrained only in the vertical direction.

An alternative to providing the vertical sheeting AH, is to support the edge member at A with columns at intervals along the side wall.

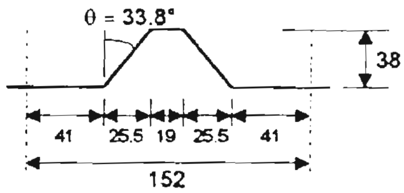
E10.5 End Frame Members



The end frames should be designed for the above forces from the inclined plate girders.

E10.6 Values of Symbols for the design of folded plate elements

A	$= 1100\ mm^2$	t	$= 1.16$ mm (Net of coatings)
b	$= 2810$ mm	u	$= 192$ mm
d	$= 152$ mm	f_y	$= 0.35/1.1 = 0.318\ kN/mm^2$
E	$= 210$ kN/mm	ν	$= 0.3$
F_p	$= 1.9 \times 0.42 \times 6.3 \times 1.16$	β_1	$= 0.71$ (table 5.2 with $n_f = 6$)
	$= 5.83$ kN (table 5.1)	β_3	$= 5/6 = 0.83$ (5.1.1.1)
F_s	$= 2.9(1.16/4.8)^{1/2} \times 0.42 \times 4.8 \times 1.16 = 3.33$ kN (table 5.1)		
F_{sc}	$= 5.83$ kN ($= F_p$)		
h	$= 38$ mm		
I	$= 48200\ mm^4$ per corr.		
L	$= 24000$ mm		
n_s	$= 15$ say		
n_{sh}	$= 20000/912 = 22$		
n_{sc}	$= 10$		
q	$= 6.10 \times 10^{-3}$ kN/mm		
p	$= 152$ mm		
s_p	$= 0.15$ mm/kN		
s_s	$= 0.25$ mm/kN (table 5.1)	ℓ/d	$= 0.125$, $h/d = 0.25$
s_{sc}	$= 0.15$ mm/kN	K_1	$= 0.113$ (table 5.6)



E10.7 Design Strength of folded plate elements

Seam Capacity

$$V_{ult} = \left(n_s F_s + \frac{2\beta_1}{\beta_3} F_p\right)\left(\frac{n_{sh}}{n_{sh} - 2}\right) = \left(15 \times 3.33 + \frac{2 \times 0.71}{0.83} \times 5.83\right)\frac{27}{25} = 65.9 \text{ kN}$$

Capacity of fasteners to end gable

$$V_{ult} = n_{sc}F_{sc} + 2F_p = 10 \times 5.83 + 2 \times 5.83 = 70.0 \text{ kN}$$

Design shear capacity

The design shear capacity V* is 65.9 kN which is the smaller of the above two values, i.e. just a little greater than the required value of 61.0 kN.

Sheet/fold-line member fasteners

Check if $\frac{0.6 b F_p}{p} > V*$

i.e. $\frac{0.6 \times 2.810 \times 10^3 \times 5.83}{152} = 64.7 \text{ kN} \approx 65.9 \text{ kN}$

The strength with respect to the sheet to fold line fasteners is marginally less than V^* but greater than the required capacity of 61.0 kN. This is satisfactory.

End collapse of sheeting profile

Check if $0.9 t^{1.5} b f_y/d^{0.5} \geq V*$

i.e. $\frac{0.9 \times 1.16^{1.5} \times 2810 \times 0.35/1.1}{152^{0.5}} = 81.5 \text{ kN} > 65.9 \text{ kN}$

This is satisfactory

Shear buckling

Check if $\frac{28.8}{b} D_x^{1/4} D_y^{3/4} > V*$

$$D_x = \frac{E t^3 d}{12(1 - \nu^2)u} = \frac{210 \times 1.16^3 \times 152}{12 \times (1 - 0.3^2) \times 192} = 23.8 \text{ kNmm}$$

$$D_y = \frac{EI}{d} = \frac{210 \times 48200}{152} = 66590 \text{ kNmm}$$

Thus $\frac{28.8}{2810} \times 23.8^{1/4} \times 66590^{3/4} = 93.8 \text{ kN} > 65.9\text{kN}$

This is satisfactory

E10.8 Deflection at Factored Load

For unit load:

$$\Delta_{1.1} = \frac{152^{2.5} \times 0.113 \times 20000^2}{8000 \times 210 \times 1.16^{2.5} \times 2810^2} = 0.667 \text{ mm/kN/m}$$

$$\Delta_{1.2} = \frac{(1 + 0.3)(1 + 2 \times 38/152) \times 20000}{4000 \times 210 \times 1.16 \times 2810} = 0.285 \text{ mm/kN/m}$$

$$\Delta_{2.1} = \frac{0.15 \times 152 \times 20000^2}{4000 \times 2810^2} = 0.289 \text{ mm/kN/m}$$

$$\Delta_{2.2} = \frac{0.25 \times 0.15(22 - 2) \times 20000}{8000(15 \times 0.15 + 0.71 \times 0.25)} = 0.772 \text{ mm/kN/m}$$

$$\Delta_{2.3} = \frac{0.15 \times 0.15 \times 20000}{2000 \times (2 \times 0.71 \times 0.15 + 10 \times 0.15)} = 0.131 \text{ mm/kN/m}$$

$$\Delta_3 = \frac{20000^4}{38400 \times 210 \times 1100 \times 2810^2} = 2.284 \text{ mm/kN/m}$$

$$\Delta' = 4.428 \text{ mm/kN/m}$$

Hence, under the factored load in the plane of the diaphragm of q = 6.10 kN/m, the in-plane deflection is

$$\Delta = 6.10 \times 4.428 = 27.0 \text{ mm}$$

The vertical central deflection of the folded plate roof, Δ_v, is then

$$\Delta_v = \Delta \operatorname{cosec} \theta = 27.0 \operatorname{cosec} 35° = 47.1\text{mm}.$$

Note: Under unfactored load, i.e. 0.3 + 0.7 = 1.00 kN/m^2,

$$\Delta_v = 47.1 \times \frac{1.00}{1.46} = 32.3 \text{ mm} = \frac{\text{span}}{620}$$

E10.9 Summary of Calculations

The calculations illustrate the following points:

(1) That profiled steel sheeting and cold formed steel apex and valley members may be used to form folded plate roofs of medium span.

(2) That the calculation procedure is simple.

(3) That a typical steel folded plate roof has a small deflection in relation to its span.