BRITISH STANDARD

BS 5950-9:
1994

Incorporating Amendment Nos. 1 and 2

Structural use of steelwork in building —

Part 9: Code of practice for stressed skin design

Committees responsible for this British Standard

The preparation of this British Standard was entrusted by the Technical Committee B/525, Building and civil engineering structures, to Subcommittee B/525/31, Structural use of steel, upon which the following bodies were represented:

Association of Consulting Engineers
British Cement Association
British Constructional Steelwork Association Ltd.
British Masonry Society
Building Employers' Confederation
Department of the Environment (Building Research Establishment)
Department of the Environment (Construction Directorate)
Department of Transport
Federation of Civil Engineering Contractors
Institution of Civil Engineers
Institution of Structural Engineers
National Council of Building Material Producers
Royal Institute of British Architects
Timber Research and Development Association

The following bodies were also represented in the drafting of the standard, through subcommittees and panels:

British Industrial Fasteners Federation
British Railways Board
British Steel Industry
Cold Rolled Sections Association
Department of the Environment (Property Services Agency)
Department of the Environment (Specialist Services)
Health and Safety Executive
Steel Construction Institute
Vice-chairman
Welding Institute

This British Standard, having been prepared under the direction of Technical Committee B/525, Building and civil engineering structures, was published under the authority of the Standards Board and comes into effect on 15 April 1994

The following BSI references relate to the work on this standard:
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Contents

Foreword

This Part of BS 5950 has been prepared under the direction of Technical Committee B/525, Building and civil engineering structures. BS 5950 comprises codes of practice which cover the design, construction and fire resistance of steel structures and specifications for materials, workmanship and erection. It comprises the following Parts and Sections:

— *Part 1: Code of practice for design in simple and continuous construction: hot rolled sections;*

— *Part 2: Specification for materials, fabrication and erection: hot rolled sections;*

— *Part 3: Design in composite construction — Section 3.1 Code of practice for design of simple and continuous composite beams;*

— *Part 4: Code of practice for design of composite slabs with profiled steel sheeting;*

— *Part 5: Code of practice for design of cold formed sections;*

— *Part 6*[1]*: Code of practice for design of light gauge profiled sheeting;*

— *Part 7: Specification for materials and workmanship: cold formed sections;*

— *Part 8: Code of practice for fire resistant design;*

— *Part 9: Code of practice for stressed skin design.*

This Part of BS 5950 gives recommendations for the use of profiled steel sheeting as "stressed skin" shear diaphragms, including the design and construction of such diaphragms, their effects on the design of structural frameworks and the design of frameless steel structures. It also gives worked examples showing the application of the method to several different design cases.

In the course of drafting, a large number of design documents and specifications were consulted. Particular attention was paid to BS 5950-1, BS 5950-4, BS 5950-5, BS 5950-6[1] and BS 5950-7, and to publications of the following relating to stressed skin diaphragm design:

American Iron and Steel Institute;

Commission of the European Communities (Eurocodes);

Deutsches Institut für Normung;

European Convention for Constructional Steelwork;

International Organization for Standardization;

Swedish Institute of Steel Construction.

This Part of BS 5950 applies only to structures of the types described in **1.1**.

It has been assumed in the drafting of this British Standard that the execution of its provisions is entrusted to appropriately qualified and experienced people and that construction and supervision are carried out by capable and experienced organizations.

As a code of practice, this British Standard takes the form of guidance and recommendations. It should not be quoted as if it were a specification and particular care should be taken to ensure that claims of compliance are not misleading.

The full list of organizations who have taken part in the work of the Technical Committee is given on the inside front cover. Professor E. R. Bryan OBE has made a particular contribution in the drafting of this code.

[1] In preparation

Summary of pages

This document comprises a front cover, an inside front cover, pages i to viii, pages 1 to 106, an inside back cover and a back cover.

This standard has been updated (see copyright date) and may have had amendments incorporated. This will be indicated in the amendment table on the inside front cover.

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Section 1. General

1.0 Introduction

1.0.1 Aims of stressed skin design

The aim of structural design is to provide, with due regard to economy, a structure capable of fulfilling its intended function and sustaining the design loads for its intended life. The design should represent the true conditions existing in the structure and should facilitate fabrication, erection and future maintenance.

The structure should behave as one three-dimensional entity. The layout of its constituent parts, such as foundations, steelwork, roof, floors, walls, connections and other structural components, should ensure a robust and stable structure under normal loading to ensure that, in the event of misuse or accident, damage will not be disproportionate to the cause.

To achieve this fully, it is necessary to take into account the strength and stiffness of the roof, floor and wall panels as well as the structural framework, and to define the paths by which the loads are transmitted to the foundations. Any features of the structure which have a critical influence on its overall stability can then be identified and taken account of in design.

Each part of the structure should be sufficiently robust and insensitive to the effects of minor incidental loads applied during service so that the safety of other parts is not prejudiced. Reference should be made to **2.3.5**.

1.0.2 Overall stability

The designer responsible for the overall stability of the structure should ensure the compatibility of design and details of parts and components, including stressed skin shear diaphragms. There should be no doubt as to where the responsibility for overall stability lies when some or all of the design and details are not made by the same designer.

1.0.3 Accuracy of calculation

For the purpose of deciding whether a particular recommendation of this Part of BS 5950 has been met, the final value, observed or calculated, expressing the result of a test or analysis should be rounded off. The number of significant places retained in the rounded off value should be the same as for the value given in this standard.

1.1 Scope

This Part of BS 5950 gives recommendations for the design of shear diaphragms in light gauge profiled steel sheet and the contribution of such diaphragms to the strength and stiffness of structural steelwork in buildings and allied structures. It also gives design recommendations for the effect of profiled steel sheet in lateral bracing to members, diaphragm action in composite floors and folded plate roof construction. Worked examples showing the application of the method to several different design cases are given in Annex A to Annex H.

The recommendations may apply to shear diaphragms in any of the following positions in buildings:

a) sloping roofs;

b) flat roofs;

c) floors; and

d) walls.

NOTE These recommendations are based on the assumption that the materials and construction conform to BS 5950-7.

1.2 References

1.2.1 Normative references

This Part of BS 5950 incorporates, by reference, provisions from specific editions of other publications. These normative references are cited at the appropriate points in the text and the publications are listed on the inside back cover. Subsequent amendments to, or revisions of, any of these publications apply to this Part of BS 5950 only when incorporated in it by amendment or revision.

1.2.2 Informative references

This Part of BS 5950 refers to other publications that provide information or guidance. Editions of these publications current at the time of issue of this standard are listed on the inside back cover, but reference should be made to the latest editions.

1.3 Definitions

NOTE A typical shear panel in a roof or floor is shown in Figure 1.

The components are as follows:

a) individual lengths of profiled steel sheeting or decking (see **1.3.22**);

b) purlins or secondary members perpendicular to the direction of span of the sheeting (perpendicular members) (see **1.3.13**);

c) rafters or main beams parallel to the direction of span of the sheeting (parallel members) (see **1.3.14**);

d) sheet/purlin fasteners;

e) seam fasteners between individual sheet widths;

f) shear connectors to provide attachment between the rafters and sheeting (see **1.3.17**);

g) sheet/shear connector fasteners;

h) purlin/rafter connection.

For the purposes of this Part of BS 5950, the definitions given in BS 5950-1:1990 apply, together with the following.

1.3.1
capacity

the limit of force that can be expected to be carried by a component without causing failure

1.3.2
design shear capacity

the least of the calculated ultimate shear capacities 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 capacity

the capacity of a diaphragm in shear

1.3.5
diaphragm length

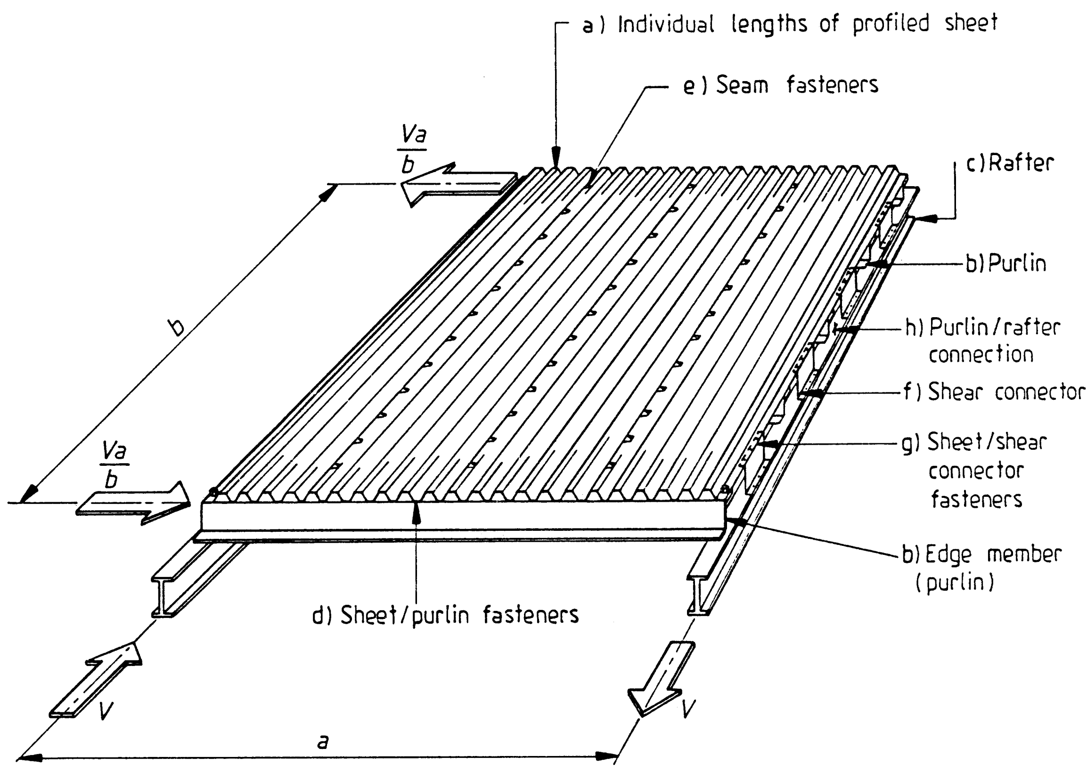
the distance between vertically stiffened frames, over which the diaphragm acts (see Figure 3)

1.3.6
edge member

a member at the extreme edge of the diaphragm running parallel to the length of the diaphragm (see Figure 1 and Figure 3)

1.3.7
fastener slip

the movement at a fastener in the plane of the sheeting per unit shear force per fastener



Key
a is the width of the shear panel in a direction perpendicular to the corrugations (in mm);
b is the depth of the shear panel in a direction parallel to the corrugations (in mm);
V is the applied force on the shear diaphragm (in kN).

Figure 1 — Typical shear panel

1.3.8
fastener tearing resistance

the shear force per fastener necessary to cause elongation or tearing in the sheeting at the fastener

1.3.9
floor panel

shear panel in a floor

1.3.10
frame flexibility

the deflection of the frame per unit load applied in the direction under consideration

1.3.11
frame stiffness

the reciprocal of frame flexibility

1.3.12
panel assembly

general term for an assembly of two or more shear panels

NOTE The term "diaphragm assembly" is sometimes used to mean "panel assembly".

1.3.13
purlin

for sheeting spanning perpendicular to the length of the diaphragm, a member supporting the sheeting and perpendicular to the corrugations (see Figure 1 and Figure 3)

NOTE A purlin may also be referred to as a perpendicular member or a secondary member.

1.3.14
rafter

1) for sheeting spanning perpendicular to the length of the diaphragm, a member supporting the purlins (see Figure 1 and Figure 3)

NOTE 1 In this context, a rafter may also be referred to as a parallel member or a main beam.

2) for sheeting spanning parallel to the length of the diaphragm, a member supporting the sheeting (see Figure 3)

NOTE 2 In this context, a rafter may also be referred to as a perpendicular member or a main beam.

1.3.15
roof panel

shear panel in a roof

1.3.16
shear capacity

capacity of a panel in shear

1.3.17
shear connector

a short length of section to enable attachment of the sheeting to the third and fourth sides of a panel when the side members are not all at the same level (see Figure 1)

1.3.18
shear diaphragm

general term for one or more shear panels or that area of sheeting which resists in-plane deflection by shear

NOTE The term "diaphragm" is sometimes used to mean "shear diaphragm".

1.3.19
shear flexibility

the in-plan deflection of a shear panel or diaphragm per unit shear load

NOTE The term "diaphragm flexibility" is sometimes used to mean "shear flexibility".

1.3.20
shear panel

a panel of sheeting subjected to in-plane shear and bounded by edge members on two sides and rafters on two sides

1.3.21
shear stiffness

the shear load per unit in-plane displacement of a shear panel or diaphragm

NOTE Shear stiffness is the reciprocal of shear flexibility.

1.3.22
sheeting

generic name for roof and floor decking, roof sheeting and side cladding

1.3.23
stressed skin

general term to describe a structure or component in which in-plane shear in the sheeting is taken into account in design

1.3.24
stressed skin action

structural behaviour involving in-plane shear in the sheeting and forces in the edge members

NOTE The term "diaphragm action" is sometimes used to mean "stressed skin action".

1.4 Major symbols

a		Width of a shear panel in a direction perpendicular to the corrugations (in mm) (see Figure 7)
A		Cross-sectional area of a longitudinal edge member (in mm^2)
	or	Semi-area of a fold line member (in mm^2)
	or	Area of openings in a panel (in mm^2)
b		Depth of a shear panel in a direction parallel to the corrugations (in mm) (see Figure 7)
	or	Depth of web in a folded plate roof (in mm) (see Figure 29)
c		Total shear flexibility of a shear panel (in mm/kN) (see Figure 6)
$c_{1.1}$ etc.		Component shear flexibilities (in mm/kN) (see Figure 5)
d		Pitch of the corrugations (in mm) (see Table 10 and Table 11)
D_x, D_y		Orthogonal bending stiffnesses of profiled sheet per unit length (in kN-mm^2/mm)
E		Modulus of elasticity of steel (205 kN/mm^2)
F_p		Design resistance of an individual sheet/purlin fastener (in kN) [see Table 5 a)]
F_{pr}		Design resistance of a purlin/rafter connection (in kN) (see Table 7)
F_s		Design resistance of an individual seam fastener (in kN) [see Table 5 b)]
F_{sc}		Design resistance of an individual sheet/shear connector fastener (in kN) [see Table 5 a)]
h		Height of the sheeting profile (in mm) (see Table 10 and Table 11)
I		Second moment of area of a single corrugation about its neutral axis (in mm^4) (see Figure 8)
k		Frame flexibility (in mm/kN) [see Figure 12 b)]
k_{sp}		Frame flexibility due to spread (in mm/kN) [see Figure 14 b)]
k_{sw}		Frame flexibility due to sway (in mm/kN) [see Figure 14 a)]
K_1, K_2		Sheeting constants for every corrugation fastened and alternate corrugations fastened (see Table 10 and Table 11)
l		Width of the corrugation crest (in mm) (see Table 10 and Table 11)
L		Length of a panel assembly between braced frames (in mm) (see Figure 7)
	or	Length of a folded plate roof between gables (in mm)
m		Number of panels spanned by a single sheet length (for sheeting spanning parallel to the length of the diaphragm) (see case 3 of Table 14)
n		Number of panels in the length of the panel assembly (see Figure 7)
n_b		Number of sheet lengths within the depth of the diaphragm (for sheeting spanning perpendicular to the length of the diaphragm) (see Table 12)
n_f		Number of sheet/purlin fasteners per member per sheet width
n_l		Number of sheet lengths in the length of the panel assembly (see Table 14)
n_p		Number of purlins (edge + intermediate)
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 panel
p		Pitch of the sheet/purlin fasteners (in mm)
P		Panel point load on diaphragm (in kN)
P_{ult}		Ultimate load at a panel point (in kN)
p_y		Design strength of steel sheeting (in N/mm^2)
q		Distributed in-plane line load on the diaphragm (in kN/mm)
r		Relative flexibility of shear panel to frame
s_p		Slip per sheet/purlin fastener per unit load (in mm/kN) [see Table 5 a)]
s_{pr}		Deflection of the top of the purlin at the purlin/rafter connection per unit load (in mm/kN) (see Table 7)
s_s		Slip per seam fastener per unit load (in mm/kN) [see Table 5 b)]
s_{sc}		Slip per sheet/shear connector fastener per unit load (in mm/kN) [see Table 5 a)]
t		Net sheet thickness, excluding galvanizing and coatings (in mm)

u		Perimeter length of a complete single corrugation (see Figure 8)
U_s		Ultimate tensile strength of steel (in N/mm^2)
u		Shear displacement of the diaphragm (in mm) (see Figure 6)
V		Applied shear force on the diaphragm (in kN) (see Figure 6)
V^*		Design shear capacity of the diaphragm (in kN)
V_{ult}		Capacity associated with a given failure mode (in kN)
	or	Ultimate load (in kN)
w		Factored vertical load per unit plan area of a folded plate roof (in kN/mm^2)
Y_s		Yield strength of steel (in N/mm^2)
α_1 to α_5		Factors to allow for intermediate purlins, number of sheet lengths, and sheet continuity (see Table 8, Table 12, Table 14 and Table 15)
β_1, β_2		Factors to allow for the number of sheet/purlin fasteners per sheet width (see Table 6)
β_3		(Distance between outermost fasteners across the sheet width) ÷ (sheet width)
γ		Shear strain (see Figure 9)
γ_f		Overall load factor
Δ		Mid-length deflection of a panel assembly (in mm)
η		Reduction factor for frame forces and moments
θ		Inclination of web of sheeting profile to the vertical (see Table 10 and Table 11)
φ		Inclination of folded plate roof to the horizontal (see Figure 28)
	or	Angle of rafter to the horizontal (see Figure 13, Figure 16 and Figure 17)
ν		Poisson's ratio for steel (0.3)

Section 2. Limit state design

2.1 General principles and methods of design

2.1.1 General

Structures should be designed by considering the limit states at which they would become unfit for their intended use, by applying appropriate factors for the ultimate limit state and the serviceability limit state.

All relevant limit states should be considered but for stressed skin structures it will usually be appropriate to design on the basis of strength at ultimate loading and then to check that the deflection is not excessive under serviceability loading. The ultimate limit state of diaphragms includes yielding, tearing at the fasteners, shear buckling of the sheeting, end collapse of the sheeting profile and failure of the edge members under tension or compression.

The overall factor in any design has to cover variability of:

— material strength γ_m;

— loading γ_l; and

— structural performance γ_p.

In this Part of BS 5950 γ_m is generally taken as 1.0 for the strength of steel (see **3.3.2**) and 1.11 for the strength of fasteners (see **3.4**). Depending on the type of load, values of γ_l and γ_p are assigned. The product of γ_l and γ_p is the overall load factor γ_f by which the specified loads should be multiplied when the strength and stability of a structure are checked (see Table 1).

2.1.2 Methods of design

2.1.2.1 ***General***

The design of any structure or its parts may be carried out by one of the methods given in **2.1.2.2** to **2.1.2.7**.

2.1.2.2 ***Stressed skin design***

The cladding is treated as an integral part of the structure, providing 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 **2.1.2.3** to **2.1.2.7**.

2.1.2.3 ***Simple design***

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

2.1.2.4 ***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.

2.1.2.5 ***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. Reference should be made to BS 5950-1:1990 for detailed design.

2.1.2.6 ***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.

2.1.2.7 ***Testing***

Where design of a structure or element by calculation in accordance with any of the preceding methods is not practicable, or is inappropriate, the strength, stability and stiffness may be confirmed by loading tests in accordance with section **11**.

Such loading tests may be carried out on shear diaphragms not only in profiled steel sheet but also in other types of steel panel such as pressed or formed panels, sandwich panels and composite panels, provided such panels are generally fixed in accordance with **3.4**.

2.2 Loading

2.2.1 General

All relevant loads should be considered separately and in such realistic combinations as to comprise the most critical effects on the components of the structure and the structure as a whole. The magnitude and frequency of fluctuating loads should also be considered. Loading conditions during erection should receive particular attention. Settlement of supports may need to be taken into account.

Table 1 — Load factors and combinations

Loading	Factor γ_f
Dead load	1.4
Dead load restraining uplift or overturning	1.0
Dead load acting with wind and imposed loads combined	1.2
Imposed load	1.6
Imposed load acting with wind load	1.2
Wind load	1.4
Wind load acting with imposed load	1.2
Forces due to temperature effects	1.2

2.2.2 Dead, imposed and wind loading

Reference should be made to BS 6399-1:1984 and BS 6399-3:1988 for the determination of dead and imposed loads, and imposed roof loads. For construction loads on composite floors, reference should be made to BS 5950-4:1993. For loads on agricultural buildings, reference should be made to BS 5502-22:1987. For loads relevant to the design of sheeting and decking, reference should be made to BS 5950-6[2)]. For the determination of wind loads, reference should be made to CP 3:Chapter V-2:1972. In the case of purlins and sheeting rails, local wind pressure (positive or negative) need not be considered.

2.2.3 Cranes and dynamic loading

Reference should be made to BS 2573-1:1983 for loading on overhead cranes and to BS 6399-1:1984 for the determination of dynamic effects.

2.2.4 Temperature effects

Where, in the design and erection of a structure, it is necessary to take account of changes in temperature, it may be assumed that in the UK the average temperature of the internal steelwork varies from – 5 °C to + 35 °C. The actual range, however, depends on the location, type and purpose of the structure and special consideration may be necessary for structures in special conditions, and in locations abroad subject to different temperature ranges.

Where measures are taken in the building structure to allow for temperature expansion or contraction of the sheeting, then such measures should be taken into account in determining the extent and effectiveness of the stressed skin diaphragms. Where such provisions for temperature effects are not made, the sheeting may be considered to be wholly effective as a diaphragm.

2.3 Ultimate limit states

2.3.1 Limit state of strength

2.3.1.1 ***General***

In checking the strength and stability of the structure the loads should be multiplied by the relevant γ_f factors given in Table 1. The factored loads should be applied in the most unfavourable realistic combination for the component or structure under consideration.

The load capacity of each member and its connections, as determined by the relevant provisions of this Part of BS 5950, should be such that the factored loads will not cause failure.

2.3.1.2 ***Overhead cranes***

Where overhead cranes are provided, reference should be made to BS 5950-1:1990 for load factors and to **2.2.3** for loading and dynamic effects. This Part of BS 5950 refers only to light overhead cranes.

2.3.2 Stability limit state

2.3.2.1 ***General***

In considering the overall stability of any structure or part, the loads should be increased by the relevant γ_f factors given in Table 1.

The designer should consider overall frame stability which embraces stability against overturning and sway stability.

2.3.2.2 ***Stability against overturning***

The factored loads should not cause the structure or any part of the structure (including the foundations) to overturn or lift off its seating. The combination of wind, imposed and dead loads should be such as to have the most severe effect on overall stability (see **2.2.1**).

Account should be taken of probable variations in dead load during construction or other temporary conditions.

2.3.2.3 ***Sway stability***

All structures, including portions between expansion joints, should have adequate strength and stiffness against sway. To ensure this, in addition to designing for applied horizontal loads, a separate check should be carried out for notional horizontal forces.

These notional forces mar arise from practical imperfections such as lack of verticality and should be taken as the greater of the following:

a) 1 % of the factored dead load from that level, applied horizontally;

b) 0.50 % of the factored load (dead plus vertical imposed) from that level, applied horizontally.

These notional forces should be assumed to act in any one direction at a time and should be applied to each roof and floor level or their equivalent. They should be taken as acting simultaneously with the factored dead plus vertical imposed loads taken as:

1) 1.4 × (unfactored dead load); and

2) 1.6 × (unfactored vertical imposed load).

The notional force should not:

i) be applied when considering overturning;

ii) be combined with the applied horizontal loads;

iii) be combined with temperature effects;

[2)] In preparation.

iv) be taken to contribute to the net reactions at the foundations.

Sway stability may be provided, for example by braced frames, by joint rigidity or by utilizing staircases, lift cores and shear walls. Whatever system is used, reversal of loading should be accommodated. The cladding, floors and roof should have adequate strength and be so secured to the structural framework as to transmit all horizontal forces to the points of sway resistance. Where such sway stability is provided by construction other than the steel framework, the steelwork designer should state clearly the need for such construction and the forces acting upon it.

2.3.2.4 ***Foundation design***

Foundations should be designed in accordance with BS 8004:1986 and should accommodate all the imposed forces. Attention should be given to the method of connecting the steel superstructure to the foundations and the anchorage of any holding-down bolts.

Where it is necessary to quote the foundation reactions it should be clearly stated whether the forces and moments result from factored or unfactored loads. Where they result from factored loads the relevant γ_f factors for each load in each combination should be stated.

2.3.3 Fatigue

Fatigue need not be considered unless a structure or element is subject to numerous significant fluctuations of load excluding those arising from wind. However, account should be taken of wind induced oscillations where these occur. When designing for fatigue a γ_f factor of **1.0** should be used.

2.3.4 Brittle fracture

Reference should be made to BS 5950-1:1990 for hot rolled sections and BS 5950-5:1987 for cold formed sections.

2.3.5 Structural integrity

2.3.5.1 ***Recommendations for all structures***

All structures should follow the general principles given in BS 5950-1:1990 and BS 5950-5:1987 and in **2.1** of this Part of BS 5950. The additional recommendations given in **2.3.5.2** and **2.3.5.3** apply to buildings.

2.3.5.2 ***Recommendations for all buildings***

By considering the behaviour of the whole building as described in **1.0.1**, stressed skin design may be used to give extra structural integrity to buildings. Every building frame should be effectively tied together at each principal floor and roof level. All columns should be effectively restrained in two directions approximately at right angles at each principal floor or roof which they support. This anchorage may be provided by either beams or tie members.

Members provided for other purposes may be utilized as ties. When members are checked as ties other loading may be ignored. Beams designed to carry the floor or roof loading will generally be suitable provided that their end connections are capable of resisting tension. Ties are not required at roof level where the steelwork supports cladding weighing not more than 0.7 kN/m^2 and carries roof loads only. Where a building is provided with expansion joints, each section between expansion joints should be treated as a separate building for the purpose of this clause.

2.3.5.3 ***Additional recommendations for certain buildings***

Where it is stipulated by appropriate regulations that buildings should be designed to localize accidental damage, reference should be made to BS 5950-1:1990 for the additional recommendations.

2.4 Serviceability limit states

2.4.1 Serviceability loads

Generally, the serviceability loads should be taken as the unfactored imposed loads. When considering dead load plus imposed load plus wind load, only 80 % of the imposed load and wind load need be considered.

2.4.2 Deflection

Subject to the provisions of **4.2**, stressed skin design of a clad building in accordance with this Part of BS 5950 will normally result in deflections significantly less than those of the bare frame calculated in accordance with BS 5950-1:1990 and BS 5950-5:1987, and will give a close estimate of the real deflection of the clad building. The effect will be particularly marked if only one or two frames are loaded.

The deflection under serviceability loads of a building or its members should not impair the strength or efficiency of the structure or its components or cause damage to the finishings.

When checking the deflections the most adverse realistic combination and arrangement of unfactored loads should be assumed, and the structure may be assumed to be elastic.

Table 2 gives recommended deflection limits for certain structural members. Circumstances may arise where greater or lesser values would be more appropriate. Other members may also require a deflection limit to be established, e.g. sway bracing.

The deflections of purlins and side rails should be limited to suit the characteristics of the particular cladding system.

The deflection of sheeting, decking and cladding should be in accordance with BS 5950-6[3)].

Table 2 — Deflection limits

a) Deflection of beams due to unfactored imposed loads	
Cantilevers	Length/180
Beams carrying plaster or other brittle finish	Span/360
All other beams	Span/200
Purlins and sheeting rails	See **2.4.2**
b) Deflection of columns other than portal frames due to unfactored imposed and wind loads	
Top of columns in single-storey buildings	Height/300
In each storey of a building with more than one storey	Height of storey under consideration/300
NOTE 1 On low-pitched and flat roofs the possibility of ponding needs consideration. NOTE 2 The designer of a framed structure, e.g. portal or multistorey, should ensure that the stability is not impaired by the interaction between deflections and axial loads.	

2.5 Durability

In a stressed skin building, the durability of the steel members and sheeting should be considered at the design stage with regard to the following factors:

a) the environment;

b) the degree of exposure;

c) the shape of the members and sheeting, and the structural detailing;

d) the corrosion protection measures adopted;

e) the anticipated life to first maintenance;

f) the degree of maintenance expected.

Reference should be made to BS 5493:1977 in determining suitable protective treatment.

The shear stress in the sheeting should be limited in accordance with **4.2.1** b) so that any deterioration of the sheeting would be apparent in stressed skin action.

Where different materials are connected together, such as in composite construction, the effects of this on the durability of the materials should be taken into consideration. Reference should be made to PD 6484:1979.

[3)] In preparation

Section 3. Materials and components

3.1 General

3.1.1 Sheeting profiles

The provisions of this Part of BS 5950 apply primarily to profiled steel sheeting as used in the roofs, floors and walls of buildings. The calculation procedures refer mainly to trapezoidal profiles but can include traditional corrugated sheeting and re-entrant angle profiled sheets as used in sheet steel/concrete floors.

In this Part of BS 5950, the term sheeting is used when the seams between adjacent sheets occur at the corrugation crests and the term decking is used when the seams occur at the corrugation troughs.

Other types of steel sheeting, decking and cladding such as built-up sections, C-shaped sections, sandwich panels and liner trays, may also be used for stressed skin construction but the shear strength and stiffness of such types should be determined by testing in accordance with section **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 fiat elements.

Where other section properties are required, reference should be made to BS 5950-6[4].

3.2 Thickness

3.2.1 Range of thicknesses

The provisions of this Part of BS 5950 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, as given in **5.3.2**.

For profiles in steel with a nominal yield strength not greater than 280 N/mm^2 the recommended minimum thickness, inclusive of coatings, is given in Table 3. For profiles in steel of thickness less than the recommended minimum, the manufacturer should demonstrate adequate resistance to denting due to construction and maintenance traffic.

Table 3 — Recommended minimum specified sheet thickness

Use	Minimum thickness inclusive of coatings (mm)
Roof sheeting and wall cladding	0.55
Roof decking	0.65
Composite floor decking	0.75

3.2.2 Design thickness

The design thickness of the material should be taken as the nominal base metal thickness exclusive of coatings. For galvanized sheeting, the total thickness of the galvanizing may be assumed to be 0.05 mm unless shown to be different by measurement, i.e. design thickness = specified thickness of sheeting – 0.05 min.

3.3 Properties of materials

3.3.1 General

This Part of BS 5950 covers the design of shear diaphragms made from steel conforming to BS 1449-1.2:1991, BS 1449-1.4:1991 and BS 1449-1.5:1991, and BS EN 10147:1992. Other steels may be used, subject to the approval of the engineer, provided due allowance is made for variation in properties, including ductility. Reference should be made to BS 5950-7:1992.

3.3.2 Strength of steel sheet

The design strength of steel sheet, p_y, should be taken as Y_s but not greater than $0.84U_s$

where

Y_s is the minimum yield strength or, in the case of material with no clearly defined yield, either the 0.2 % proof stress or the stress at 0.5 % total elongation;

U_s is the minimum ultimate tensile strength.

Y_s may normally be taken as specified in the relevant British Standard. For such cases, values of yield, ultimate and design strengths for some of the more common types of steel are given in Table 4.

Where the sheet material is supplied with a certificated minimum yield stress, Y_s may be taken as the certificated value and used for the formed section. Alternatively, for any steel, the strength of the sheet material or the formed section may be determined by testing in accordance with section **11**.

3.3.3 Other properties of steel

The following values for the elastic properties should be used:

— modulus of elasticity E = 205 kN/mm^2;

— shear modulus G = 79 kN/mm^2;

— Poisson's ratio ν = 0.30;

— coefficient of linear thermal expansion $\alpha = 12 \times 10^{-6}$ K^{-1}.

[4] In preparation.

3.4 Fasteners for stressed skin action

3.4.1 Sheet/member fasteners

The sheeting or decking should be attached directly through the troughs to the supporting members 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 means of fastening are self-tapping or self-drilling screws, cartridge fired pins, bolts or welding. Hook bolts, clips or other fasteners which transmit shear forces by friction are not suitable. Welding involves special techniques and consideration of the sheet coatings. The possibility of bimetallic corrosion between the fastener and the sheet should also be considered.

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 means of fastening are self-drilling screws, monel metal or stainless steel blind rivets, bolts or welding. Aluminium blind rivets are not generally suitable. Welding involves special techniques and consideration of the sheet coatings. The possibility of bimetallic corrosion between the fastener and the sheet should also be considered.

3.4.3 Strength of fasteners

The characteristic tearing resistances of fasteners in sheeting may be determined by testing in accordance with **11.3**. The design resistance of fasteners should be taken as (characteristic resistance of fasteners)/1.11, where 1.11 is the material factor γ_m; typical values for commonly used fasteners and sheeting thicknesses are given in Table 5. These values are based on test results and apply to the number of fasteners typically used in stressed skin panels. Reference should be made to **5.3.2**.

3.4.4 Slip of fasteners

The slip values of fasteners in sheeting may be determined by testing in accordance with **11.3**. Typical values for commonly used fasteners are given in Table 5 for serviceability loading conditions. Within the range of sheet thicknesses given, the slip values given in Table 5 may be taken to be independent of net sheet thickness t and yield strength Y_s. Reference should be made to **4.1.4**, **4.2.2** e) and **5.3.2**.

Table 4 — Yield, ultimate tensile and design strengths of steel sheet

British Standard	Grade	Minimum yield strength Y_a N/mm^2	Minimum ultimate tensile strength U_a N/mm^2	Design strength p_y N/mm^2
BS 1449-1.2	HR3 HR4	(170) (170)	(290) (280)	(170) (170)
BS 1449-1.4	HR34/20 HR37/23 HR43/25 HR50/35 HR40/30 HR43/35 HR40 F 30 HR43 F 35	200 230 250 350 300 350 300 350	340 370 430 500 400 430 400 430	200 230 250 350 300 350 300 350
BS 1449-1.5	CR34/20 CR37/23	200 230	340 370	200 230
BS EN 10147	Fe E 220 G Fe E 250 G Fe E 280 G Fe E 350 G Fe E 550 G	220 250 280 350 550	300 330 360 420 560	220 250 280 350 460
NOTE Figures given in parentheses are non-mandatory and are given for guidance only.				

3.5 Diaphragm components

The following considerations apply to diaphragms.

a) The shear diaphragm shown in Figure 1 is typical of a sloping roof panel in which the purlins pass over the rafters. The use of shear connectors enables all four sides of the panel to be fastened.

b) If shear connectors are not used as in Figure 1, so that the 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.

c) 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.

d) In a building in which shear diaphragms are fastened only on two sides, shear connectors or their equivalent (e.g. end closures) should be used on the end rafters at the gables of the building.

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

Section 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 fiat roof building subjected to side load, as shown in Figure 2 a), each of the roof 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 deflections.

b) *Pitched roofs*. In a pitched roof building subjected to vertical load, as shown inFigure 2 b), there is a component of load down the roof slope so that, if the gables are tied, the roof panels may be assumed to act as diaphragms that help prevent the building from spreading. If the gables are braced or sheeted, the roof diaphragms may also be particularly effective in helping to prevent the building from swaying 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 flames.

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.

NOTE 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.

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

NOTE 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:

a) flat roof buildings under horizontal load	4.0
b) pitched roof buildings	
— under vertical load	2.5
— under horizontal load	4.0

NOTE 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 length/width ratio has only a small effect on the load distribution.

NOTE 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.

4.1.2 Load sharing

Where the frames shown in Figure 2 a) are pin jointed, so that they have no in-plane rigidity, 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 shown in Figure 2 a) have rigid joints, the side loads may be assumed to be shared between the frames and the diaphragms. Where the frames are not braced during erection, the sway frames should be designed to carry the full unfactored load with a minimum safety factor of **1.0**. In the sheeted building the diaphragms may then be assumed to provide the additional resistance required to carry the factored load.

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

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 design may also be applied to buildings which are arched or which have curved 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 be used for the vertical frames of tall multistorey buildings where design of such frames is affected by considerations of frame instability.

4.1.4 Repeated loading

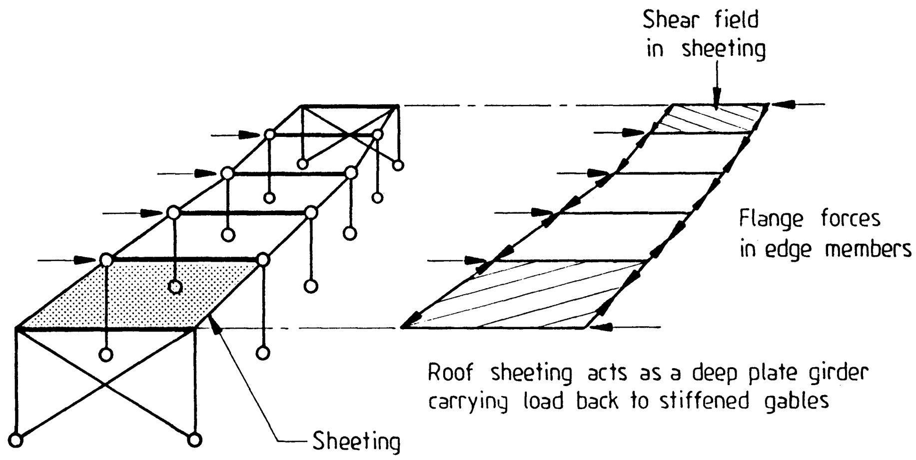
Subject to the determination of dynamic loads given in **2.2.3**, no other allowance for repeated loading need normally be made in determining the design strength and shear flexibility of a diaphragm. For unusually severe cases of repeated loading, the shear flexibility of a diaphragm should be increased by 50 %.

4.2 Conditions and restrictions

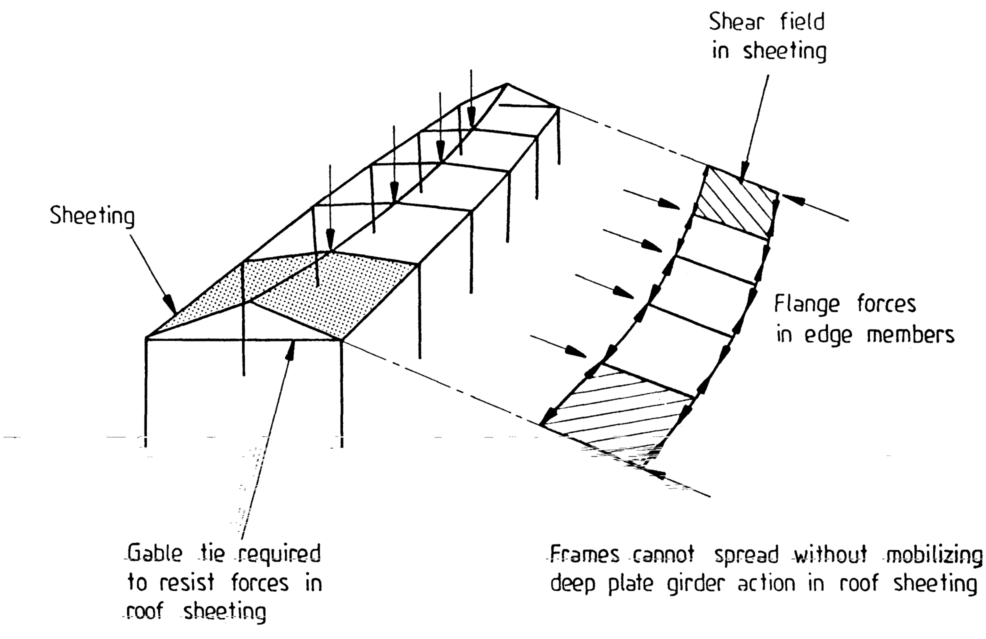
4.2.1 Necessary conditions

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.



a) Stressed skin action ın a flat roof building



b) Stressed skin action in a pitched roof building

Figure 2 — Stressed skin action in buildings

b) The sheeting should first be designed for its primary purpose in bending in accordance with BS 5950-6[5]. It should then be checked that the maximum shear stress due to diaphragm action does not exceed 25 % of the maximum bending stress so that any deterioration of the sheeting would be apparent in bending before it would be apparent in stressed skin action. No other allowance for the combined effects of bending and shear in the sheeting need be made.

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 means of resisting sway.

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 stressed skin 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 fasteners to the edges and ends of the sheets should conform to **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 but should be restricted predominantly to resisting the following:

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

2) seismic forces and small transient loads.

b) Stressed skin diaphragms should be treated as structural components and should not be removed, either wholly or partly, without consideration of the effect on the strength and stiffness of the diaphragm and on the stability of the structure. Such consideration should not preclude planned removal of areas of sheeting where this has been allowed for in design.

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

d) Openings totalling more than 3 % of the area in each panel should not be permitted unless they conform to **8.3**. Openings of less than 3 % of the area in each panel may be permitted without special calculation provided the total number of fasteners in each panel is not reduced.

e) Stressed skin diaphragms should be designed predominantly for short-term imposed loads, unless creep is taken into account.

f) Stressed skin buildings in which the frames have not been designed to carry the full unfactored load without collapse should be braced in accordance with **4.1.2** 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 clad.

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 3 a)] or parallel to the length of the diaphragm [see Figure 3 b)]. In each case "sheet/purlin fasteners" refers to the fasteners between the sheet and the perpendicular members [a purlin in Figure 3 a) and a rafter in Figure 3 b)]. Also in each case "sheet/shear connector fasteners" refers to the fasteners between the sheet and the parallel member [a rafter in Figure 3 a) and an edge member in Figure 3 b)].

NOTE In Figure 3 the double-headed arrow indicating the direction of span of sheeting is labelled; in subsequent figures, the direction of span of sheeting is indicated by an unlabelled, double-headed arrow.

[5] In preparation.

4.3.2 Fastener arrangements

Various sheet/member fastener arrangements may be used for diaphragms, as shown in Figure 4. Cases (1) and (2) require that the tops of the members are level or that shear connectors are used, allowing four sides of the 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 panel can be fastened. In case (3), shear connectors must be used at the end rafters. Case (4) is not normally recommended.

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

4.4 Design criteria

4.4.1 Diaphragm strength

With reference to Figure 5, the possible criteria for diaphragm capacity are as follows:

a) sheet tearing along a line of seam fasteners [see Figure 5 a)];

b) sheet tearing along a line of sheet/shear connector fasteners [see Figure 5 b)];

c) sheet tearing in the sheet/purlin fasteners [see Figure 5 c) and Figure 5 d)];

d) end collapse of the sheeting profile [see Figure 5 e)];

e) shear buckling of the sheeting [see Figure 5 f)];

f) failure of the edge member in tension or compression [see Figure 5 g)].

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

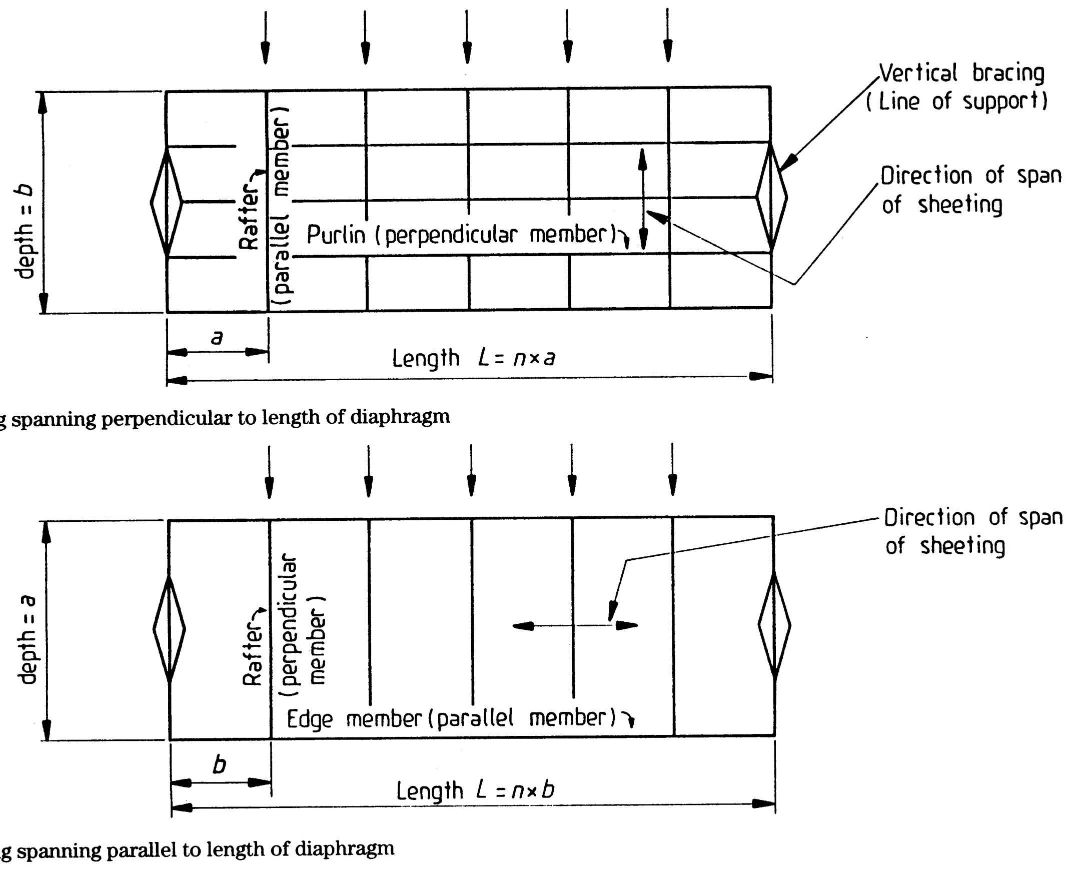


Figure 3 — Direction of span of the sheeting

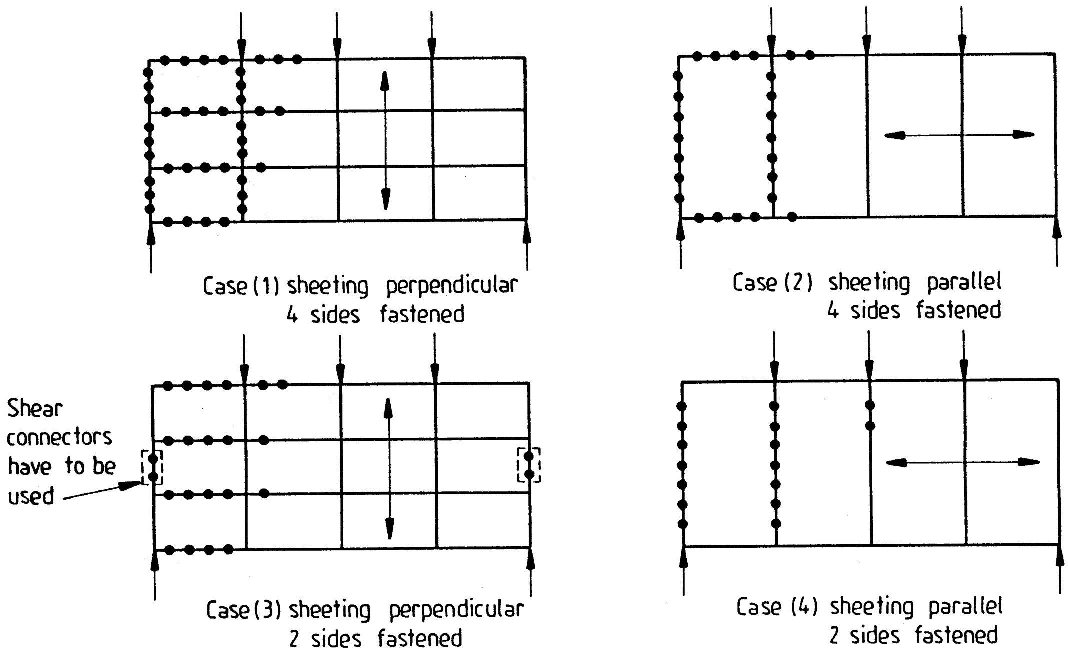


Figure 4 — Fastener arrangements

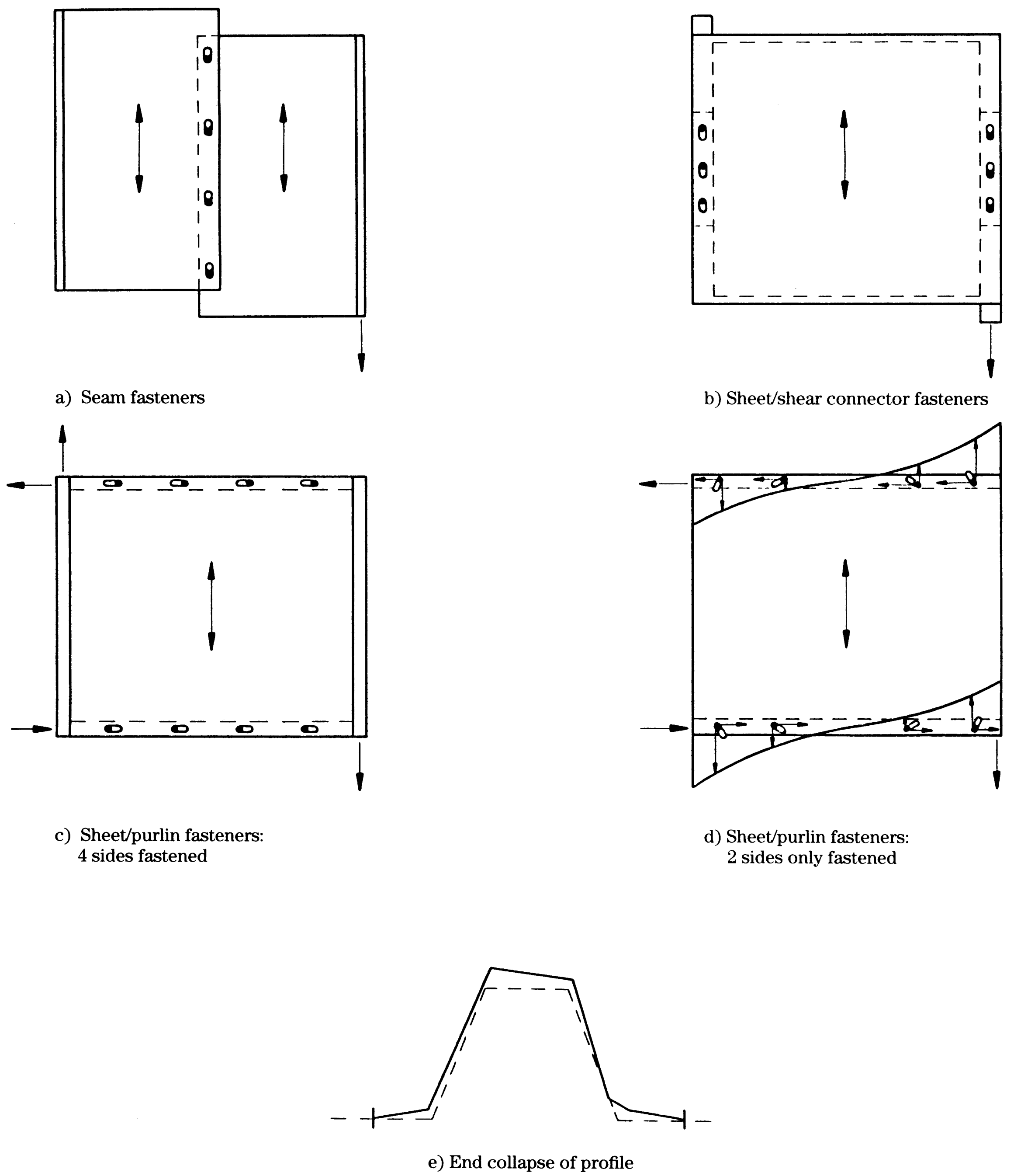
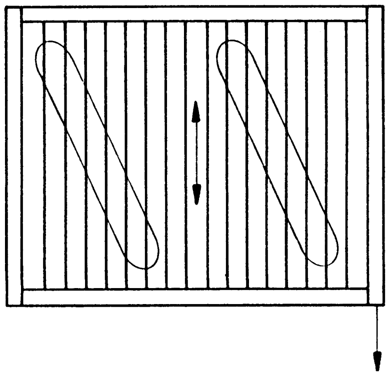
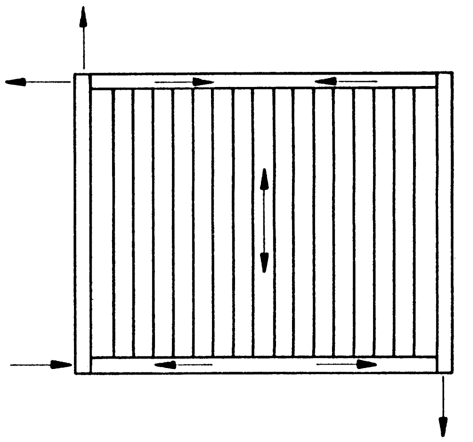


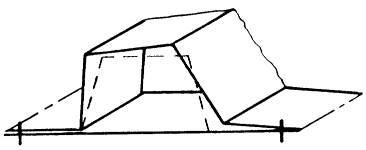
Figure 5 — Design criteria for diaphragm strength and flexibility (see **4.4.1** and **4.4.2**)



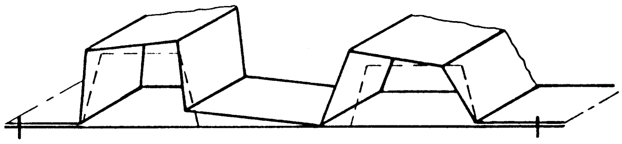
f) Shear buckling



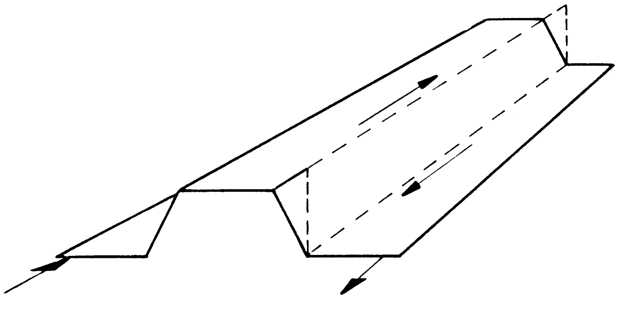
g) Edge member forces



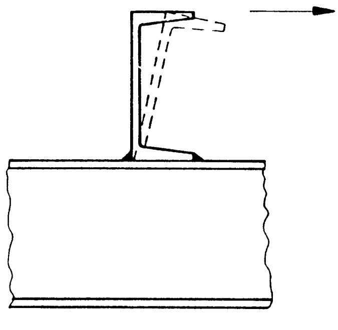
h) Profile distortion:
every trough fastened



i) Profile distortion:
alternate troughs fastened



j) Shear strain in sheeting



k) Purlin/rafter connection

Figure 5 — Design criteria for diaphragm strength and flexibility (see **4.4.1** and **4.4.2**) (*concluded*)

4.4.2 Diaphragm stiffness and flexibility

With reference to Figure 6, the displacement of the shear diaphragm is v under the shear load V. The shear flexibility of the diaphragm is u/V and the shear stiffness is the reciprocal of this, V/u. Throughout this Part of BS 5950 the term shear flexibility is used in preference to shear stiffness; it is defined as the shear deflection per unit shear load, and refers to the direction parallel to the corrugations.

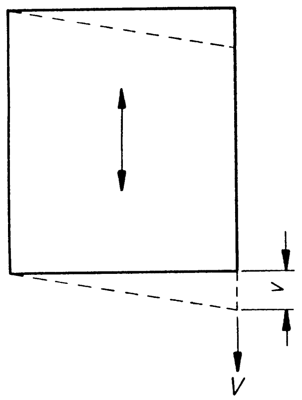


Figure 6 — Shear flexibility $c = u/V$

With reference to Figure 5, the total shear flexibility of a panel is the sum of the separate component shear flexibilities due to the following:

a) profile distortion [see Figure 5 h) and Figure 5 i)];

b) shear strain in the sheeting [see Figure 5 j)];

c) slip in the sheet/purlin fasteners [see Figure 5 c) and Figure 5 d)];

d) slip in the seam fasteners [see Figure 5 a)];

e) slip in the sheet/shear connector fasteners [see Figure 5 b)];

f) purlin/rafter connections (in the case of the sheet fastened to the purlins only) [see Figure 5 k)];

g) axial strain in the longitudinal edge members [see Figure 5 g)].

Section 5. Design expressions: sheeting spanning perpendicular to the length of the diaphragm

5.1 Diaphragm strength

5.1.1 Strength of panel assemblies

5.1.1.1 *General*

With reference to Figure 7, the shear capacity of the panel assembly should normally be checked by considering failure modes in the end panels and at the internal rafters. The shear capacities V_{ult} associated with these failure modes may be obtained as described in **5.1.1.2** to **5.1.1.5**.

5.1.1.2 *Seam capacity*

The seam capacity is given by

$$V_{ult} = n_s F_s + \frac{\beta_1}{\beta_3} n_P F_P$$

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 resistance of an individual seam fastener (in kN);
- n_p is the number of purlins (edge + intermediate);
- F_p is the design resistance of an individual sheet/purlin fastener (in kN);
- β_1 is a factor to allow for the number of sheet/purlin fasteners per sheet width;
- β_3 = $(n_f - 1)/n_f$ for sheeting (seam fasteners in the crests); or
- β_3 = 1.0 for decking (seam fasteners in the troughs);
- n_f is the number of sheet/purlin fasteners per member per sheet width (including those at the overlaps).

NOTE 1 Values of F_s and F_p are given in Table 5; values of β_1 are given in Table 6.

NOTE 2 In roof sheeting and side cladding the seams between adjacent sheets normally occur at the corrugation crests, while in roof and floor decking (except composite sheet steel/concrete decks) the seams normally occur at the corrugation troughs.

5.1.1.3 *Shear connector fastener capacity at the end gables*

The shear connector fastener capacity at the end gables is given by

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

where

- n_{sc} is the number of sheet/shear connector fasteners per end rafter (see note 2 to **5.1.1.4**);
- F_{sc} is the design resistance of an individual sheet/shear connector fastener (in kN).

NOTE Values of F_{sc} are given in Table 5.

5.1.1.4 *Shear connector fastener capacity at the internal rafters*

The shear connector fastener capacity at the internal rafters is given by

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

where

- P_{ult} is the ultimate load at a panel point (in kN);
- n'_{sc} is the number of sheet/shear connector fasteners per internal rafter;
- F_{sc} is the design resistance of an individual sheet/shear connector fastener (in kN).

NOTE 1 Values of F_{sc} are given in Table 5.

NOTE 2 With reference to Figure 7, the force in the end rafters is ½ $(n - 1)$ times the force in the internal rafters, so the corresponding numbers of shear connector fasteners should normally be in the same ratio, i.e. $n_{sc} = ½(n - 1)n'_{sc}$.

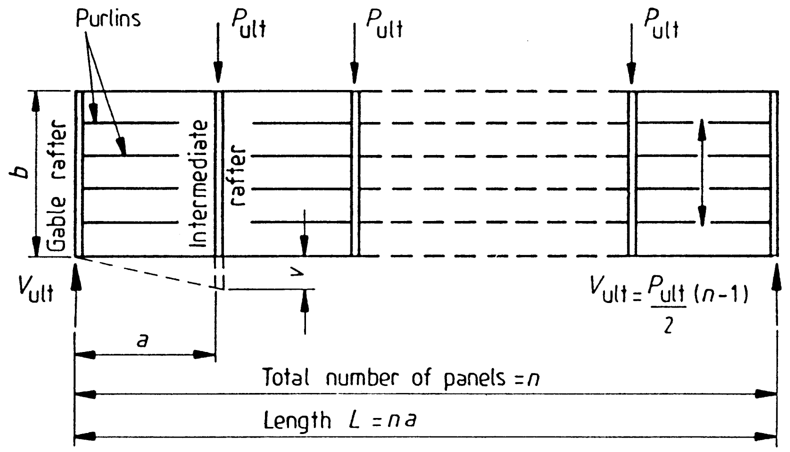


Figure 7 — Panel assembly: sheeting spanning perpendicular to the length of the diaphragm

Table 5 — Design resistances and slip values of fasteners

Fastener type	**Washer type**	**Outside diameter** mm	**Design resistance** kN	**Slip** mm/kN
a) Sheet/purlin and sheet/shear connector fasteners				
Screws	Collar head	6.3 5.5	$F_p = F_{sc} = 6.0t\sqrt{(Y_s/280)}$ but ≤ 9.0 $F_p = F_{sc} = 5.0t\sqrt{(Y_s/280)}$ but ≤ 7.5	$s_p = s_{sc} = 0.15$
	Collar head + neoprene	6.3 5.5	$F_p = F_{sc} = 5.0t\sqrt{(Y_s/280)}$ but ≤ 9.0 $F_p = F_{sc} = 4.0t\sqrt{(Y_s/280)}$ but ≤ 7.5	$s_p = s_{sc} = 0.35$
Fired pins	23 mm diameter steel	3.7 to 4.5	$F_p = F_{sc} = 5.0t\sqrt{(Y_s/280)}$ but ≤ 9.0	$s_p = s_{sc} = 0.10$
b) Seam fasteners (no washers)				
Screws		4.1 to 4.8	$F_s = 2.5t\sqrt{(Y_s/280)}$ but ≤ 3.8	$s_s = 0.25$
Steel or monel blind rivets		4.8	$F_s = 2.8t\sqrt{(Y_s/280)}$ but ≤ 3.0	$s_s = 0.30$

Key

t is the net sheet thickness (in mm);
Y_s is the yield strength of the steel sheet (in N/mm^2).

NOTE The design resistances and slip values apply for the fasteners listed and for net sheet thicknesses between 0.50 mm and 1.25 mm. For other fasteners and sheet thicknesses, see **5.3.2**.

Table 6 — Factors to allow for the number of sheet/purlin fasteners per sheet width

Numbers of fasteners per sheet width (including those at the overlaps) n_f	Factor β_1		Factor β_1
	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
NOTE The expressions from which the above values have been obtained are given in Annex J.			

5.1.1.5 ***Two sides of a panel fastened***

For sheeting attached to the purlins only and to the end rafters (see case (3) in Figure 4):

a) the capacity of the end fasteners in an internal panel is given by

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

b) the capacity of the purlin/rafter connections is given by

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

where

- P_{ult} is the ultimate load at a panel point (in kN);
- n_p is the number of purlins (edge + intermediate);
- F_p is the design resistance of an individual sheet/purlin fastener (in kN);
- F_{pr} is the design resistance of a purlin/rafter connection (in kN);
- β_2 is a factor to allow for the number of sheet/purlin fasteners per sheet width.

NOTE Values of F_p are given in Table 5; values of β_2 are given in Table 6; values of F_{pr} are given in Table 7.

5.1.2 Design shear capacity

The design shear capacity V^* may be taken as the least of the values of V_{ult} obtained from the equations given in **5.1.1.2** and **5.1.1.3**, and the derived values of V_{ult}, given by $V_{ult} = \frac{1}{2} P_{ult}(n - 1)$ in **5.1.1.4** and **5.1.1.5**, as appropriate to the case considered. It should be checked that the capacity in 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 capacity***

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

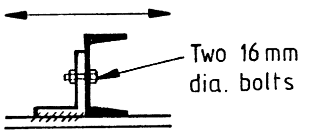
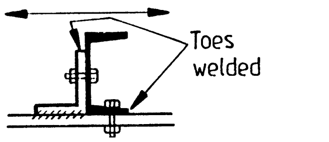
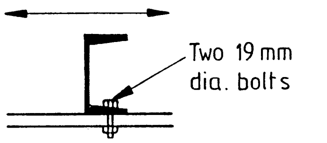
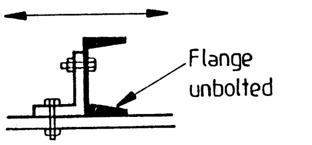
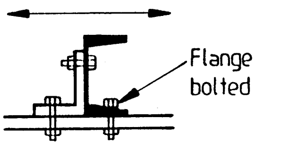
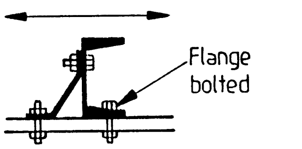
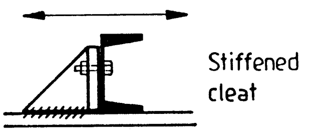
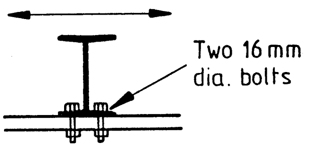
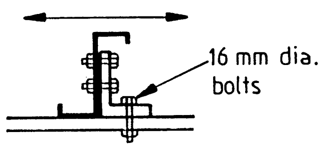
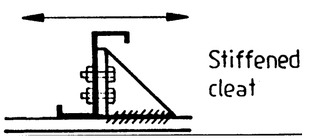
$$\frac{0.6bF_p}{P\alpha_3} \geq V^*$$

where

- b is the depth of the shear panel in a direction parallel to the corrugations (in mm);
- F_p is the design resistance of an individual sheet/purlin fastener (in kN);
- p is the pitch of the sheet/purlin fasteners (in mm);
- α_3 is a factor to allow for intermediate purlins.

NOTE Values of F_p are given in Table 5 and values of $\alpha 3$ are given in Table 8.

Table 7 — Design resistances and flexibilities of purlin/rafter connections

Connection number	Type of purlin (and cleat)	Connection detail	Design resistance F_{pr} kN	Flexibility s_{pr} mm/kN
1	102 × 51 × 10.42 kg/m channel		4.9	0.84
2	(89 × 64 × 7.8 angle cleat × 89 mm long)		20.0	0.11
3	152 × 76 × 17.88 kg/m channel		14.4	0.60
4	(76 × 64 × 6.2 angle cleat × 127 mm long		7.2	1.20
5			19.6	0.35
6			25.0	0.13
7			25.0	0.05
8	254 × 102 × 22 kg/m Universal beam		10.0	2.60
9	203 × 51 × 2.0 zed		4.4	1.40
10	(178 × 89 × 9.4 angle cleat × 127 mm long)		7.2	0.38

NOTE The resistance and flexibility of other types and sizes of purlin/rafter connections may be estimated from the values given above or may be obtained by test.

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 limitations on shear force in a panel should be observed:

a) every corrugation fastened

$0.0009t^{1.5}bY_s/d^{0.5} \geqslant V^*$

b) alternate corrugations fastened

$0.0003t^{1.5}bY_s/d^{0.5} \geqslant V^*$

where

- t is the net sheet thickness, excluding galvanizing and coatings (in mm);
- b is the depth of the shear panel in a direction parallel to the corrugations (in mm);
- Y_s is the yield strength of steel (in N/mm^2);
- d is the pitch of the corrugations (in mm).

5.1.3.3 ***Shear buckling***

The shear buckling capacity of the sheeting should be checked using the expression given in **5.4**, which includes a 25 %reserve of safety.

Table 8 — Factors to allow for the effect of intermediate purlins

Total no. of purlins per panel (or per sheet length for α_1)	Factors		
n_p	α_1	α_2	α_3
2	1	1	1
3	1	1	1
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
NOTE The expressions from which the above values have been obtained are given in Annex J.			

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 using the expression given in **5.5**, which includes a 25 % reserve of safety.

5.2 Shear flexibility

5.2.1 Flexibility of panel assemblies

5.2.1.1 ***General***

The total shear flexibility of a panel is the sum of the component shear flexibilities listed in **4.4.2**. For panel assemblies (see Figure 7) the design expressions are given in column (1) of Table 9. For a cantilevered diaphragm [see Figure 18 a)] the design expressions are given in column (2) of Table 9. Guidance on the design expressions is given in **5.2.1.2** to **5.2.1.8**.

5.2.1.2 ***Profile distortion***

In the expression for $c_{1.1}$, K can take the value K_1 or K_2 as given in Table 10 and Table 11, depending on whether the sheeting is fastened in every corrugation or alternate corrugations. The factor α_1 takes account of the effect of fasteners at intermediate puffins and is given in Table 8. 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 12 for various fastener arrangements.

Table 9 — Components of shear flexibility: sheeting spanning perpendicular to the length of the diaphragm

Component of shear flexibility	(1) Shear flexibility in panel assemblies (see Figure 7) mm/kN	(2) Shear flexibility in cantilevered diaphragms [see Figure 18 a)] mm/kN
Sheet deformation		
Profile distortion	$c_{1.1} = \dfrac{ad^{2.5}\alpha_1\alpha_4 K}{Et^{2.5}b^2}$	$c_{1.1} = \dfrac{ad^{2.5}\alpha_1\alpha_4 K}{Et^{2.5}b^2}$
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 fasteners	$c_{2.1} = \dfrac{2as_\mathrm{p}p\alpha_3}{b^2}$	$c_{2.1} = \dfrac{2as_\mathrm{p}p}{b^2}$
Seam fasteners	$c_{2.2} = \dfrac{2s_\mathrm{s}s_\mathrm{p}(n_\mathrm{sh}-1)}{2n_\mathrm{s}s_\mathrm{p}+\beta_1 n_p s_\mathrm{s}}$	$c_{2.2} = \dfrac{2s_\mathrm{s}s_\mathrm{p}(n_\mathrm{sh}-1)}{2n_\mathrm{s}s_\mathrm{p}+\beta_1 n_p s_\mathrm{s}}$
Connections to rafters	4 sides fastened $c_{2.3} = \dfrac{4(n+1)s_{sc}}{n^2 n^\mathrm{p}_{sc}}$ or 2 sides only fastened with gable shear connectors $c_{2.3} = \dfrac{4(n-1)}{n^2 n_\mathrm{p}}\left(s_\mathrm{pr}+\dfrac{s_\mathrm{p}}{\beta_2}\right)$	4 sides fastened $c_{2.3} = \dfrac{2s_\mathrm{sc}}{n_\mathrm{sc}}$ or 2 sides only fastened $c_{2.3} = \dfrac{2}{n_\mathrm{p}}\left(s_\mathrm{pr}+\dfrac{s_\mathrm{p}}{\beta_2}\right)$
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	$c = c_{1.1} + c_{1.2} + c_{2.1} + c_{2.2} + c_{2.3} + c_3$	$c = c_{1.1} + c_{1.2} + c_{2.1} + c_{2.2} + c_{2.3} + c_3$

Key to Table 9

a	is the width of a shear panel in a direction perpendicular to the corrugations (in mm);
A	is the cross-sectional area of a longitudinal edge member (in mm^2);
b	is the depth of a shear panel in a direction parallel to the corrugations (in mm);
c	is the total shear flexibility of a shear panel (in mm/kN);
$c_{1.1}$ etc.	are the component shear flexibilities (in mm/kN);
d	is the pitch of the corrugations (in mm);
E	is the modulus of elasticity of steel (205 kN/mm^2);
h	is the height of the sheeting profile (in mm);
K	is a sheeting constant which can take the value K_1 or K_2 as given in Table 10 and Table 11 (see **5.2.1.2**);
n	is the number of 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 panel;
p	is the pitch of the sheet/purlin fasteners (in mm);
s_p	is the slip per sheet/purlin fastener per unit load (in mm/kN) (see Table 5);
s_pr	is the deflection of the top of the purlin at the purlin/rafter connection per unit load (in mm/kN) (see Table 7);
s_s	is the slip per seam fastener per unit load (in mm/kN) (see Table 5);
s_sc	is the slip per sheet/shear connector fastener per unit load (in mm/kN) (see Table 5);
t	is the net sheet thickness, excluding galvanizing and coatings (in mm);
α_1 to α_4	are factor to allow for intermediate purlins and the number of sheet lengths (see Table 8 and Table 12);
β_1, β_2	are factors to allow for the number of sheet/purlin fasteners per sheet width (see Table 6);
ν	is Poisson's ratio for steel (0.3).

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 panel is not uniform. The factor α_2 takes account of this effect and is given in Table 8.

5.2.1.4 ***Slip in the sheet/purlin fasteners***

The flexibility $c_{2.1}$ due to slip in the sheet/purlin fasteners depends on the slip value and the spacing of fasteners. The factor α_3, given in Table 8, 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 and floor 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 expression for $c_{2.2}$ takes account of this difference by means of the values of β_1 (see Table 6) 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 panel are fastened and if $n_{sc} = ½(n - 1)n'_{sc}$ (see **5.1.1.4**) the shear flexibility $c_{2.3}$ due to slip in the sheet/shear connector fasteners is the same at the end panels and internal panels.

5.2.1.7 ***Movement in the purlin/rafter connections***

For the case of a 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 purlins***

The flexibility due to axial strain in the purlins 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 panel assembly. The factor β_3, given in Table 8, 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 panel. The mid-length deflection of the typical panel assembly shown in Figure 7 is given by

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

where

- P is the panel point load on the diaphragm (in kN);
- n is the number of panels in the length of the panel assembly;
- c is the total shear flexibility of a shear panel as given in Table 9 (in mm/kN).

5.3 Fastener characteristics

5.3.1 Edge and end distances

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

a) Edge distance of seam fasteners and shear connector fasteners:

(1.5 × diameter) or 8 mm

b) Edge distance of sheet/purlin fasteners:

(1.5 × diameter) or 10 mm

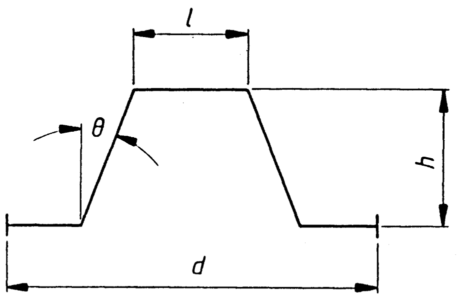
c) End distance of sheet/purlin fasteners:

(3 × diameter) or 20 mm

5.3.2 Fastener strength and slip

The design resistances and slip values given in Table 5 apply to the range of fasteners, number of fasteners and sheet thicknesses typically found in stressed skin panels. For other types of fasteners and sheet thicknesses, lap joint tests should be made in accordance with **11.3** to determine the characteristic resistances and slip values and to ensure that failure occurs by tearing of the sheeting in accordance with **3.4.1** and **3.4.2**. It is essential that the absolute limits on design resistances given in Table 5 are not exceeded.

Table 10 — Values of K_1 for fasteners in every trough



θ	h/d	l/d =								
		0.1	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**
0°	0.1	0.013	0.030	0.041	0.041	0.046	0.050	0.066	0.103	0.193
	0.2	0.042	0.096	0.131	0.142	0.142	0.153	0.199	0.311	0.602
	0.3	0.086	0.194	0.264	0.285	0.283	0.302	0.388	0.601	1.188
	0.4	0.144	0.323	0.438	0.473	0.468	0.494	0.629	0.972	1.925
	0.5	0.216	0.438	0.654	0.705	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	—	—	—	—	—

Table 10 — Values of K_1 for fasteners in every trough

θ	h/d	l/d =								
		0.1	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**
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	—	—	—	—	—	—	—	—

NOTE Intermediate stiffeners in the flange or web may be ignored for the purpose of determining K_1.

5.4 Shear buckling

The shear buckling capacity of the sheeting should satisfy the following expression in accordance with the recommendations of **4.4.1** e) and **5.1.3.3**:

$$\frac{14.4}{b} D_x^{1/4} D_y^{3/4} (n_p - 1)^2 > V^*$$

where D_x and D_y are the orthogonal bending stiffnesses given by

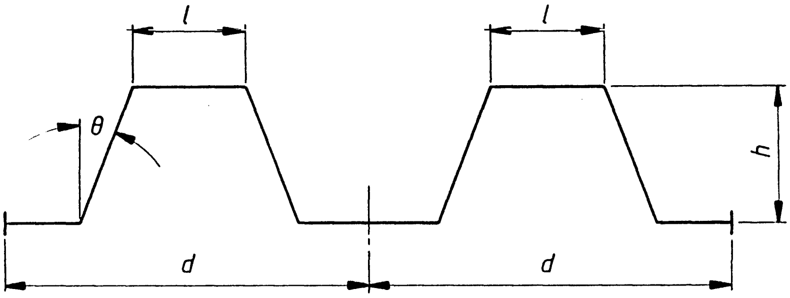
$$D_x = \frac{Et^3 d}{12(1 - \nu^2)u}$$

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

where

- E is the modulus of elasticity of steel (205 kN/mm^2);
- t is the net sheet thickness, excluding galvanizing and coatings (in mm);
- d is the pitch of the corrugations (in mm);
- ν is Poisson's ratio for steel (0.3);
- u is the perimeter length of a complete single corrugation of pitch d (in mm) (see Figure 8);
- I is the second moment of area of a single corrugation about its neutral axis (in mm^4) (see Figure 8);
- b is the depth of a shear panel in a direction parallel to the corrugations (in mm);
- n_p is the number of purlins (edge + intermediate);
- V^* is the design shear capacity of the diaphragm (in kN).

Table 11 — Values of K_2 for fasteners in alternate troughs



θ	h/d	$l/d=$								
		0.1	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**
0°	0.1	0.014	0.025	0.036	0.046	0.054	0.061	0.070	0.108	0.211
	0.2	0.031	0.065	0.099	0.129	0.151	0.169	0.206	0.318	0.649
	0.3	0.054	0.123	0.192	0.252	0.294	0.328	0.402	0.608	1.269
	0.4	0.084	0.202	0.316	0.414	0.482	0.535	0.653	0.968	2.056
	0.5	0.123	0.299	0.468	0.614	0.712	0.790	0.958	1.410	3.006
	0.6	0.169	0.415	0.649	0.846	0.982	1.090	1.318	1.928	4.113
	0.7	0.222	0.549	0.855	1.108	1.286	1.433	1.730	2.525	5.383
	0.8	0.284	0.699	1.086	1.398	1.623	1.818	2.196	3.198	6.811
5°	0.1	0.089	0.138	0.184	0.228	0.269	0.311	0.359	0.432	0.590
	0.2	0.300	0.433	0.564	0.690	0.810	0.934	1.091	1.358	2.046
	0.3	0.627	0.872	1.113	1.345	1.569	1.806	2.125	2.710	4.441
	0.4	1.076	1.453	1.826	2.187	2.535	2.910	3.446	4.498	8.057
	0.5	1.644	2.171	2.694	3.205	3.703	4.244	5.058	6.761	12.94
	0.6	2.280	2.961	3.639	4.313	4.999	5.797	6.971	9.571	—
	0.7	2.961	3.803	4.620	5.443	6.347	7.479	9.206	13.01	—
	0.8	3.802	4.838	5.788	6.612	7.701	9.257	11.76	17.20	—
10°	0.1	0.091	0.140	0.186	0.229	0.270	0.312	0.362	0.440	0.627
	0.2	0.312	0.446	0.575	0.699	0.817	0.943	1.112	1.425	2.472
	0.3	0.665	0.907	1.144	1.370	1.589	1.835	2.204	2.979	—
	0.4	1.156	1.529	1.891	2.239	2.578	2.984	3.655	5.251	—
	0.5	1.793	2.313	2.819	3.305	3.782	4.397	5.519	7.872	—
	0.6	2.533	3.206	3.858	4.509	5.192	6.096	7.875	—	
	0.7	3.334	4.148	4.949	5.780	6.737	8.112	10.82	—	
	0.8	4.236	5.170	6.051	7.066	8.404	10.47	12.59	—	
15°	0.1	0.093	0.142	0.188	0.231	0.271	0.313	0.364	0.448	0.682
	0.2	0.325	0.458	0.586	0.707	0.824	0.953	1.140	1.523	—
	0.3	0.703	0.942	1.174	1.393	1.610	1.874	2.316	3.411	—
	0.4	1.237	1.602	1.953	2.285	2.624	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	9.190	—	—	—	—
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	1.635	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	—	—	—	—	—

Table 11 — Values of K_2 for fasteners in alternate troughs

θ	h/d	l/d =								
		0.1	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**
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	0.144	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	—	—	—	—	—	—	—	—
NOTE Intermediate stiffeners in the flange or web may be ignored for the purpose of determining $K2$.										

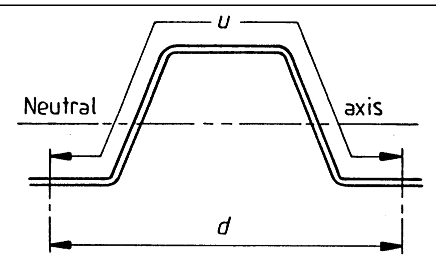


Figure 8 — Single corrugation

5.5 Edge members

The edge members and their connections should be designed in accordance with the recommendations of BS 5950-1:1990 or BS 5950-5:1987 to carry the vertical load from the roof or floor together with 1.25 × the axial load from diaphragm action (see **5.1.3.4**). The maximum axial load, with reference to Figure 7, may be taken as

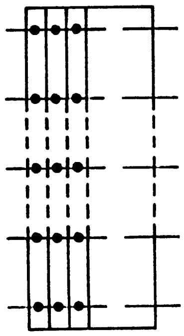
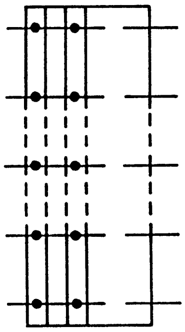
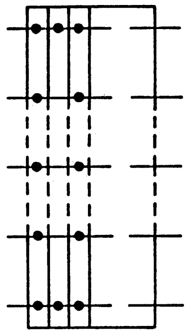
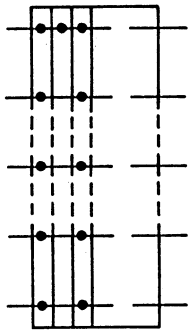
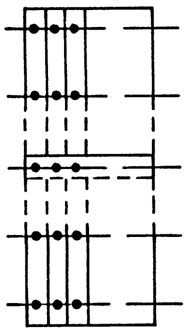
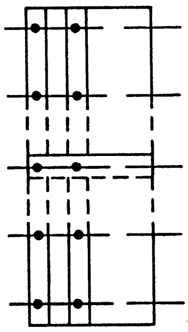
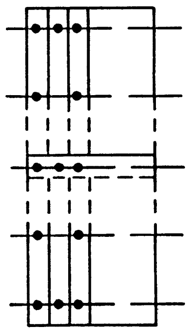
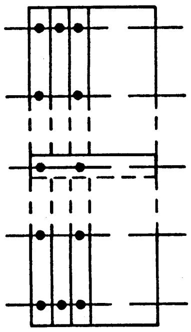
$$\frac{qL^2\alpha_3}{8b}$$

where

- q is the distributed in-plane line load on the diaphragm (in kN/mm);
- L is the length of the panel assembly between braced frames in (mm);
- α_3 is a factor to allow for intermediate purlins;
- b is the depth of a shear panel in a direction parallel to the corrugations (in mm).

NOTE Values of α_3 are given in Table 8.

Table 12 — Factor α_4 to allow for the number of sheet lengths in the depth of a diaphragm

Number of sheet lengths	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	 Case (1) $K = K_1$ α_1 is as given in Table 8 $\alpha_4 = 1$	 Case (2) $K = K_2$ α_1 is as given in Table 8 $\alpha_4 = 1$	 Case (3) $K = K_1$ $\alpha_1 = 1$ $\alpha_4 = 1$	 Case (4) $K = K_2$ $\alpha_1 = 0.5$ $\alpha_4 = 1$
n_b sheet lengths in depth of diaphragm	 Case (5) $K = K_1$ α_1 is as given in Table 8 for the number of purlins per sheet length $\alpha_4 = (1 + 0.3n_b)$	 Case (6) $K = K_2$ α_1 is as given in Table 8 for the number of purlins per sheet length $\alpha_4 = (1 + 0.3n_b)$	 Case (7) $K = K_1$ $\alpha_1 = 1$ $\alpha_4 = (1 + 0.3n_b)$	 Case (8) $K = K_2$ α_1 is as given in Table 8 for the number of purlins per sheet length $\alpha_4 = (1 + 0.3n_b) \times$ $\times \left(1 - \frac{1}{n_b}\right)$

5.6 Bonded insulation

For profiled steel roof decking, with thermal insulation and waterproofing bonded to the top of the decking, e.g. fibreboard bonded with hot bitumen, the following allowances may be made.

a) *Shear strength.* No increase may be permitted in the calculated shear strength of the diaphragm due to the insulation.

b) *Shear flexibility.* For sheeting fastened in alternate corrugations only (in which case $K = K_2$), but not for sheeting fastened in every corrugation, the following multiplying factors may be applied to the component shear flexibility $C_{1.1}$ in Table 9 due to profile distortion:

— for profile height $h \leq 50$ mm, 0.7;

— for profile height $h > 50$ mm, 0.5.

5.7 Combined loads

5.7.1 Effect on sheeting

It may be assumed from **4.2.1** b) that deterioration of the sheeting would become apparent for other reasons before it could prejudice the behaviour of the diaphragm and that it should not normally be necessary to take stressed skin action into account in the design of the sheeting.

It may be assumed from **4.2.1** c) 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.

5.7.2 Effect on fasteners

Under combined shear and wind uplift, the only fasteners subjected to combined stresses are the sheer/purlin fasteners. The reduced shear capacity of these fasteners due to prying action is given in **5.1.3.1**. Under combined load, the fasteners should satisfy the following:

$$\left(\frac{F'_t}{F_t}\right)^2 + \left(\frac{F'_p}{0.06F_p}\right)^2 \leq 1$$

where

F'_t is the design tensile load in a fastener (in kN);

F_t is the design tensile resistance of a fastener (in kN) and is the least of

— pull-over of the sheet over the head of the fastener

— pull-out of the fastener from the supporting member, and

— direct tensile strength of the fastener itself;

F'_p is the design shear load on a fastener (in kN);

F_p is the design shear resistance of a fastener (in kN).

Section 6. Design expressions: sheeting spanning parallel to the length of the diaphragm

6.1 Diaphragm orientation

The values of shear strength and flexibility given in section **5** are calculated in a direction parallel to the corrugations [see Figure 9 a)], i.e. V_1 and v_1/V_1. The shear strength and flexibility in a direction perpendicular to the corrugations [see Figure 9 b)] are given by the following expressions.

a) Shear strength (in kN)

$V_2 = V_1$ a/b

where V_1 is the shear strength calculated as V^* in **5.1.2**.

b) Shear flexibility (in mm/kN)

$v_2/V_2 = v_1/V_1\ (b/a)^2$

where v_1/V_1 is the shear flexibility calculated as c in **5.2.1**.

NOTE The modified shear flexibility is applied only to components a) to f) in **4.4.2**.

In the above expressions:

- V_1 is the shear strength of the diaphragm parallel to the corrugations (in kN);
- v_1 is the shear displacement of the diaphragm parallel to the corrugations (in mm);
- V_2 is the shear strength of the diaphragm perpendicular to the corrugations (in kN);
- u_2 is the shear displacement of the diaphragm perpendicular to the corrugations (in mm);
- a is the width of the shear panel in a direction perpendicular to the corrugations (in mm);
- b is the depth of the shear panel in a direction parallel to the corrugations (in mm).

6.2 Diaphragm strength

6.2.1 Strength of panel assemblies

6.2.1.1 ***General***

With reference to Figure 10, the shear capacity of the panel assembly should normally be checked by considering failure modes in the end panels and at the internal rafters. The shear capacities V_{ult} associated with these failure modes may be obtained as described in **6.2.1.2** to **6.2.1.4**.

6.2.1.2 ***Seam capacity***

The seam capacity is given by

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

where

a and b are defined in **6.1**; and

n_s F_s, β_1, β_3 and F_p are as defined in **5.1.1**.

6.2.1.3 ***Edge member fastener capacity***

For the sheeting attached to the rafters and edge members

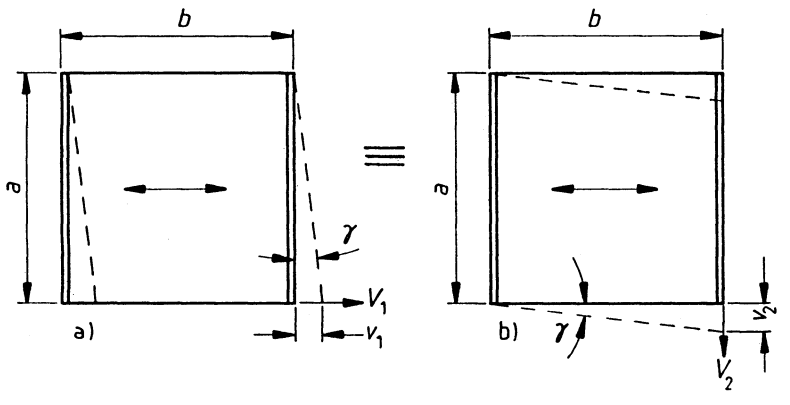
$$V_{ult} = \frac{a}{b}\,(n_{sc} F_{sc})$$

where

a and b are as defined in **6.1**;

F_{sc} is as defined in **5.1.1**; and

n_{sc} is the number of fasteners to the edge member over the length b.



NOTE γ is the shear strain in the sheet and is the same in Figure 9 a) and Figure 9 b).

Figure 9 — Shear strength and flexibility in a perpendicular direction

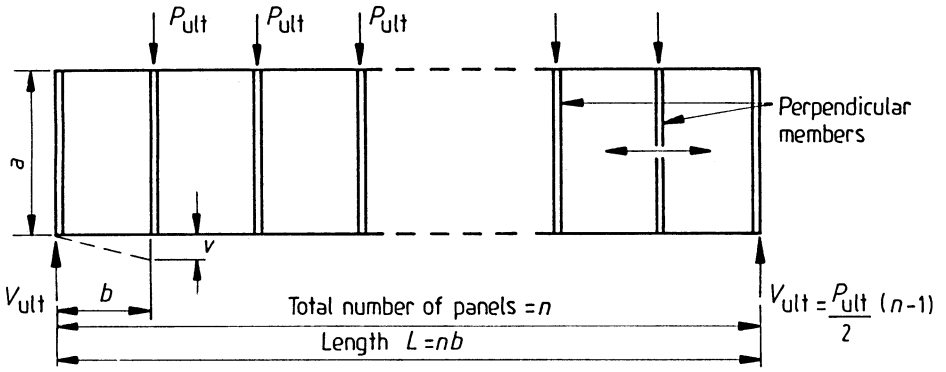


Figure 10 — Panel assembly: sheeting spanning parallel to the length of the diaphragm

6.2.1.4 ***Two sides of panel fastened***

Although not normally recommended (see case (4) of Figure 4) this case applies to the sheeting attached to the rafters only (not to the edge members). At the end rafter

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

where

a and b are as defined in **6.1**; and

β_2 and F_p are as defined in **5.1.1**.

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 panels. In this case the design criterion for the end panel is given in **6.2.1.3**, and the design criterion for an internal panel is

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

6.2.2 Design shear capacity

The design shear capacity V^* may be taken as the lesser of the values of V_{ult} given in **6.2.1.2** and **6.2.1.3** or **6.2.1.4**. It should be checked that the capacity in other failure modes is greater than V^* as given in **6.2.3.1** to **6.2.3.4**.

6.2.3 Non-permissible modes

6.2.3.1 ***Sheet/rafter fastener capacity***

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 (in mm); and
- F_p and p are as defined in **5.1.3**.

6.2.3.2 ***End collapse of sheeting profile***

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

a) every corrugation fastened

$$0.0009t^{1.5}a\ Y_s/d^{0.5} \geq V^*$$

b) alternate corrugations fastened

$$0.0003t^{1.5}a\ Y_s/d^{0.5} \geq V^*$$

where

a is as defined in **6.2.3.1**; and

t, Y_s and d are as defined in **5.1.3**.

6.2.3.3 ***Shear buckling***

It should be checked that the shear buckling capacity of the sheeting is greater than V^* as follows:

a) every corrugation fastened

$$\frac{28.8a}{b^2} D_x^{1/4}\ D_y^{3/4} > V^*$$

b) alternate corrugations fastened

$$\frac{14.4a}{b^2} D_x^{1/4}\ D_y^{3/4} > V^*$$

where

a is as defined in **6.3.2.1**; and

b, D_x and D_y are as defined in **5.4**.

6.2.3.4 ***Edge members***

The edge members and their connections should be designed in accordance with BS 5950-1:1990 or BS 5950-5:1987 to carry the vertical load from the roof or floor together with 1.25 × the axial load from diaphragm action. The maximum axial load (see Figure 10) may be taken as $qL^2/8a$, where the symbols are as defined in **5.5** and **6.2.3.1**.

6.2.4 Other effects

The recommendations on bonded insulation given in **5.6** and on combined loads given in **5.7.1** apply also to sheeting spanning parallel to the length of the diaphragm. The sheet/rafter fasteners are subject to the restrictions given in **5.7.2** concerning combined stresses under shear and wind uplift.

6.3 Shear flexibility

6.3.1 Flexibility of panel assemblies

6.3.1.1 *General*

The total shear flexibility of a panel is the sum of the component shear flexibilities listed in **4.4.2**. For panel assemblies (see Figure 10) the design expressions are given in column (1) of Table 13. For a cantilevered diaphragm [see Figure 18 b)] the design expressions are given in column (2) of Table 13. Guidance on the design expressions is given in **6.3.1.2** and **6.3.1.3**.

6.3.1.2 *Profile distortion*

In the expression for $c_{1.1}$, K can take the value K_1 or K_2, as given in Table 10 and Table 11, 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 14 and given numerically in Table 15.

Where insulation is bonded to the top of the sheeting, the value of $c_{1.1}$ may be reduced in accordance with **5.6** b).

Table 13 — Components of shear flexibility: sheeting spanning parallel to the length of the diaphragm

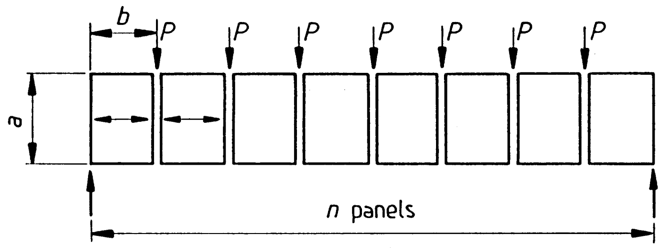
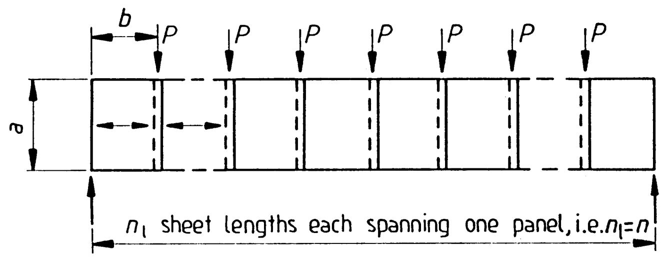
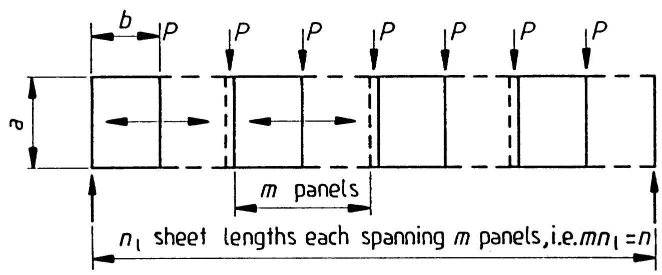
Component of shear flexibility	(1)	(2)
	Shear flexibility in panel assemblies (see Figure 10) mm/kN	**Shear flexibility in cantilevered diaphragms** [see Figure 18 b)] mm/kN
Sheet deformation		
Profile distortion	$c_{1.1} = \dfrac{ad^{2.5}\alpha_5 K}{Et^{2.5}b^2}$	$c_{1.1} = \dfrac{ad^{2.5}\alpha_1\alpha_4 K}{Et^{2.5}b^2}$
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 members fasteners	$c_{2.1} = \dfrac{2as_p p}{b^2}$	$c_{2.1} = \dfrac{2as_p p}{b^2}$
Seam fasteners	$c_{2.2} = \dfrac{s_s s_p(n_{sh}-1)}{n_s s_p + \beta_1 s_s}$	$c_{2.2} = \dfrac{2s_s s_p(n_{sh}-1)}{2n_s s_p + \beta_1 n_p s_s}$
Connections to edge members	4 sides fastened $c_{2.3} = \dfrac{2s_{sc}}{n_{sc}}$ or 2 sides only fastened $c_{2.3} = s_{pr} + \dfrac{s_p}{\beta_2}$	4 sides fastened $c_{2.3} = \dfrac{2s_{sc}}{n_{sc}}$ 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	$c' = \dfrac{b^2}{a^2}(c_{1.1}+c_{1.2}+c_{2.1}+c_{2.2}+c_{2.3})$	$c' = \dfrac{b^2}{a^2}(c_{1.1}+c_{1.2}+c_{2.1}+c_{2.2}+c_{2.3})$
Flange forces		
Axial strain in edge members	$c_3 = \dfrac{n^2 b^3}{4.8EAa^2}$	$c_3 = \dfrac{2b^3}{3EAa^2}$
Total shear flexibility	$c = c' + c_3$	$c = c' + c_3$

Key to Table 13

The symbols are as defined for Table 9 with the addition of the following:

α_5 is a factor to allow for sheet continuity (see **6.3.1.2**).

Table 14 — Influence of sheet length for sheeting spanning parallel to the length of the diagram

Design case	Value of α_5
(1) Separate sheet length per panel 	1
(2) Overlapping separate sheets 	Value given in Table 15
(3) Overlapping continuous sheets 	Value given in Table 15 divided by m

Key

a is the width of the shear panel in a direction perpendicular to the corrugations (in mm);
b is the depth of the shear panel in a direction parallel to the corrugations (in mm);
m is the number of panels within a sheet length;
n is the number of panels in the length of the panel assembly;
n_1 is the number of sheet lengths in the length of the panel assembly (i.e. mn_1 = n);
p is the panel point load on the diaphragm (in kN);
α_5 is a factor to allow for sheet continuity.

Table 15 — Factor α_5 to allow for sheet continuity

Number of sheet lengths n_1	α_5
2	1.0
3	0.9
4	0.8
5 or more	0.7

6.3.1.3 *Other data*

Values of sheeting, fastener and connection characteristics, and multiplying factors are the same as for sheeting spanning perpendicular to the length of the diaphragm and are given in Table 5, Table 6, Table 7, Table 10 and Table 11.

6.3.2 Deflection

The mid-length deflection Δ of the typical panel assembly shown in Figure 10 is given by

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

where

- P is the panel point load on the diaphragm (in kN);
- n is the number of panels in the length of the panel assembly;
- c is the total shear flexibility of a shear panel, as given in Table 13 (in mm/kN).

Section 7. Shear diaphragms and rigid jointed frames

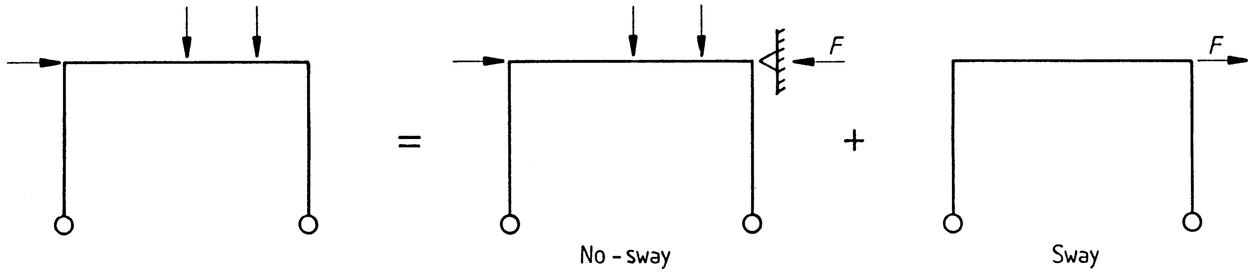
7.1 Design principles

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 resistance to forces and moments out of the plane of the sheeting.

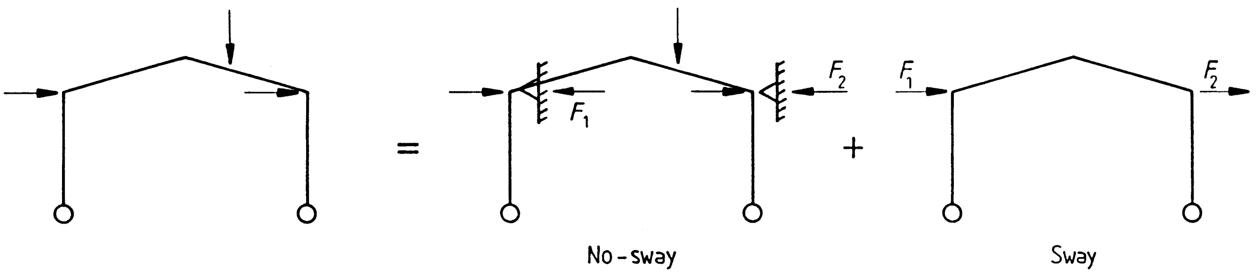
With reference to Figure 11, stressed skin action may be seen to have no effect on the no-sway or no-spread moments in a structure, but it may have considerable effect on the sway or spread moments.

The extent of the effect may be shown to depend on the shear flexibility of a panel of a panel of sheeting relative to the frame flexibility.

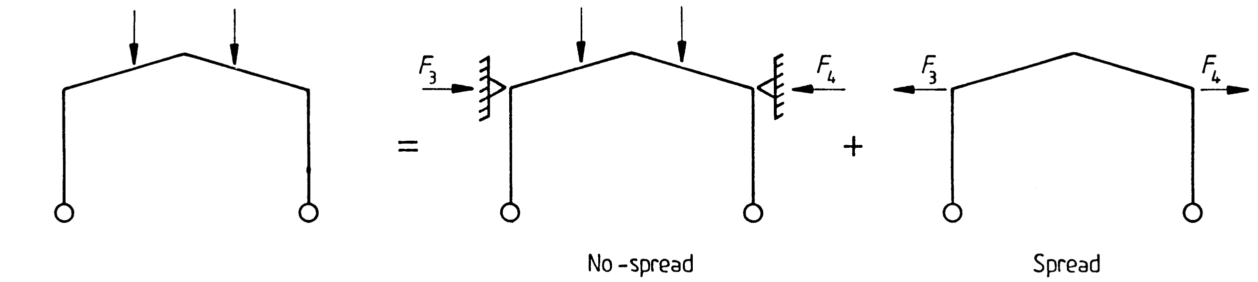
The treatment described in this section applies only to single bay fiat roof frames and to symmetrical single bay pitched roof frames. It is assumed that all frames in a building are similar, that all shear panels are similar and that all foundation and other conditions are similar. Multibay frames and structures for which these conditions are not met should be analysed as described in **8.5**.



a) Sway in a rectangular frame



b) Sway in a pitched roof frame



c) Spread in a pitched roof frame

Figure 11 — No-sway and sway, no-spread and spread in rectangular and pitched roof portal frames

7.2 Elastic design

7.2.1 Rectangular frames: all frames loaded

The shear flexibility c of a panel of sheeting (in mm/kN) is the shear deflection per unit shear load [see Figure 12 a)] and may be calculated in accordance with **5.2** or **6.3**. The frame flexibility k of a rectangular frame (in mm/kN) is the eaves deflection per unit horizontal eaves load [see Figure 12 b)] and may be calculated by normal elastic methods. The relative flexibility is defined as $r = c/k$. If r is large (for flexible sheeting or stiff frames) then the sheeting will have a small stiffening effect; if r is small (for stiff sheeting or flexible frames) then the sheeting will have a large stiffening effect.

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_1 on the number of frames in the building, and on the position of the frame under consideration in the building [see Figure 13 a)]. Values of η for each frame in a building are given in Table 16, and the mathematical expressions are given in Annex J.

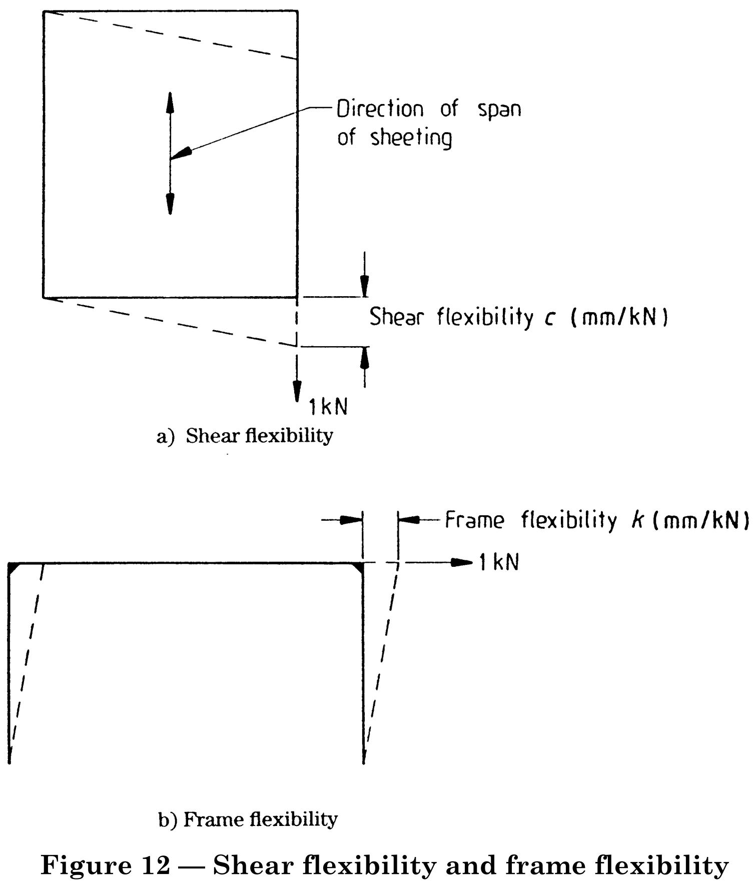
The moments in a clad rectangular frame may be expressed as:

bending moment in clad frame = (no-sway bending moment in bare frame) + η × (sway bending moment in bare frame)

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

7.2.2 Pitched roof frames: all frames loaded

The in-plane shear flexibility c of a panel of sheeting (in mm/kN), may be calculated in accordance with **5.2** or **6.3**, taking the depth of the panel as the length of one roof slope. If φ is the angle of the rafter to the horizontal, the horizontal shear flexibility c_h (in mm/kN) is given by $c_h = c \sec^2 \varphi$.



a) Shear flexibility

b) Frame flexibility

Figure 12 — Shear flexibility and frame flexibility

Table 16 — Reduction factor η on sway forces and moments for each frame in a clad building: all frames loaded

Total number of frames in building	Frame number (see Figure 13)	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	.0333	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	0.200	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

NOTE The expressions from which these values have been obtained are given in Annex J.

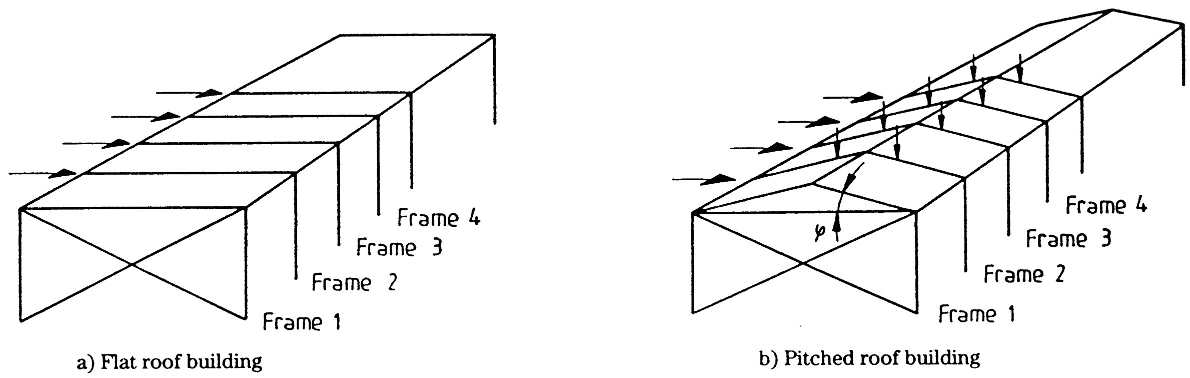


Figure 13 — Numbering of frames in clad buildings

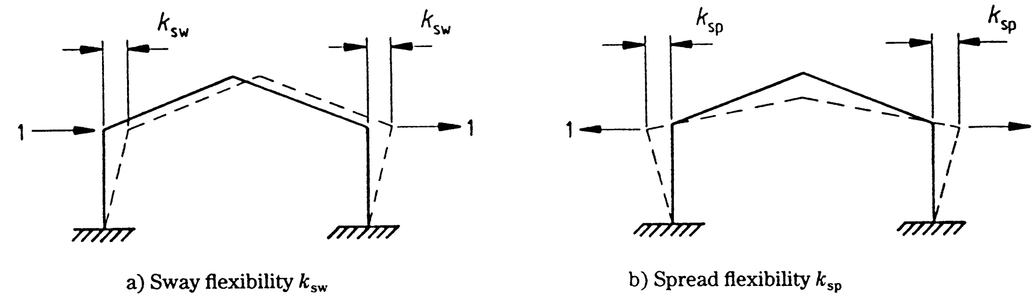


Figure 14 — Frame flexibilities for a pitched roof portal frame

The general sway of the frame may be divided into sway and spread. The frame flexibility due to sway k_{sw} (in mm/kN) is illustrated in Figure 14 a) and the frame flexibility due to spread k_{sp} (in mm/kN) is illustrated in Figure 14 b). 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 [see Figure 13 b)] from Table 16.

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} × (sway bending moment in bare frame)
		+ η_{sp} × (spread bending moment in bare frame)

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 17 gives factors by which the value of η in Table 16 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

7.3.1 General

Plastic design should be in accordance with BS 5950-1-1990 except that it should be assumed that the effect of stressed skin diaphragms is to modify the loading on frames in clad buildings as shown in **7.3.2** and **7.3.3**. If a check on the frame stability is required an elastic analysis should also be carried out. As recommended in **4.1.2** the bare frame should be designed for a minimum load factor of **1.0**.

Table 17 — Factors by which η should be divided for one frame only loaded (see Table 16)

Value of relative flexibility r	Total 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

NOTE These values apply to the case of the central frame only loaded: they are conservative for the case of any one other frame loaded. The expressions from which these values have been obtained are given in Annex J.

Provided that the criterion for the least shear strength of a panel of sheeting is tearing at the seam fasteners or sheet/shear connector fasteners then the 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 fiat roof building [see Figure 15 a)] the restraining force R provided by the sheeting is the same at each intermediate frame at collapse [see Figure 15 b)] and is given by

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

where

V^* is the design shear capacity of a panel;

n is the number of panels in the length of the building.

Each frame should then be plastically designed under a net sway force of (applied load $- R$) as shown in Figure 15 c).

7.3.3 Pitched roof frames

For a clad pitched roof building under side load [see Figure 16 a)] or vertical load [see Figure 17 a)] the in-plane restraining force R provided by the sheeting on each roof slope is the same at each intermediate frame at collapse [see Figure 16 b) and Figure 17 b)] and is given by

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

where V^* and n are as defined in **7.3.2**.

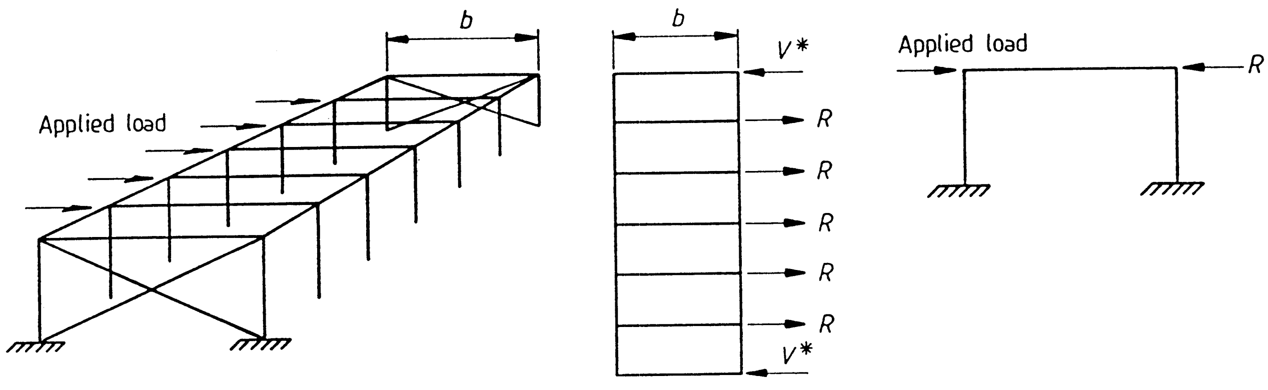
The horizontal component of the restraining force R_h is given by

$$R_h = R \cos \varphi$$

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

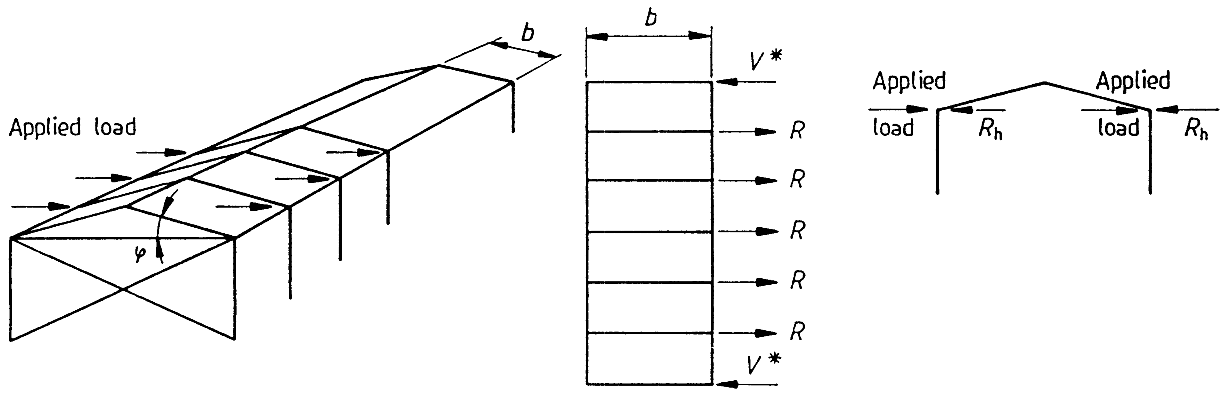
For a clad building under side load, each frame should be plastically designed under the net sway force of (applied load $- R_h$) as shown in Figure 16 c).

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 17 c).



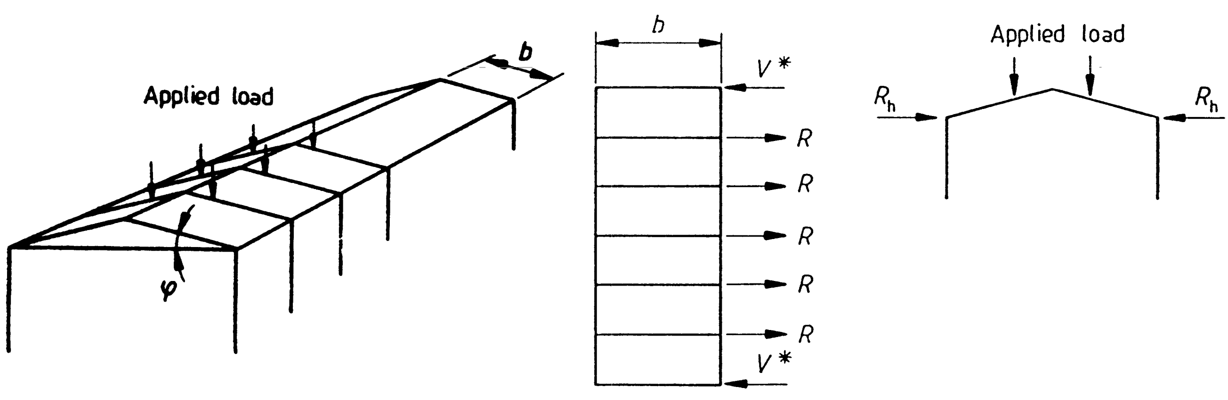
a) Forces on clad building b) Forces on sheeting c) Forces on frame

Figure 15 — Plastic design of a clad rectangular portal frame



a) Forces on clad building b) Forces on sheeting c) Forces on frame

Figure 16 — Plastic design of a clad pitched roof portal frame under side loads



a) Forces on clad building b) Forces on sheeting c) Forces on frame

Figure 17 — Plastic design of a clad pitched roof portal frame under vertical loads

Section 8. Complex diaphragms

8.1 General

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

a) diaphragms in different directions and at different levels on fiat roofs;

b) panel assemblies in which one gable end cannot be braced;

c) diaphragms with openings;

d) diaphragms with concentrated and distributed in-plane loads.

A simplified treatment is given in this Part of BS 5950. Alternatively a more rigorous treatment may be adopted (see J. M. Davies and E. R. Bryan, *Manual of stressed skin diaphragm design* [1]).

8.2 Irregular diaphragms

In buildings with flat roofs in different directions and at different heights, the recommendations for diaphragms should apply separately to:

— areas 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 (see Figure 18) 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.

8.3 Diaphragms with openings

8.3.1 General

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

a) discrete openings [see Figure 19 a)]; or

b) strip openings [see Figure 19 b)].

The recommendations given are for sheeting spanning perpendicular to the length of the diaphragm. They may also be used as a guide for sheeting spanning parallel to the length of the diaphragm in which case the dimensions a and b should be interchanged. Hence for strip openings parallel to the corrugations, the dimensions b_1 and b_2 in **8.3.6** should be replaced by a_1 and a_2.

8.3.2 Discrete openings

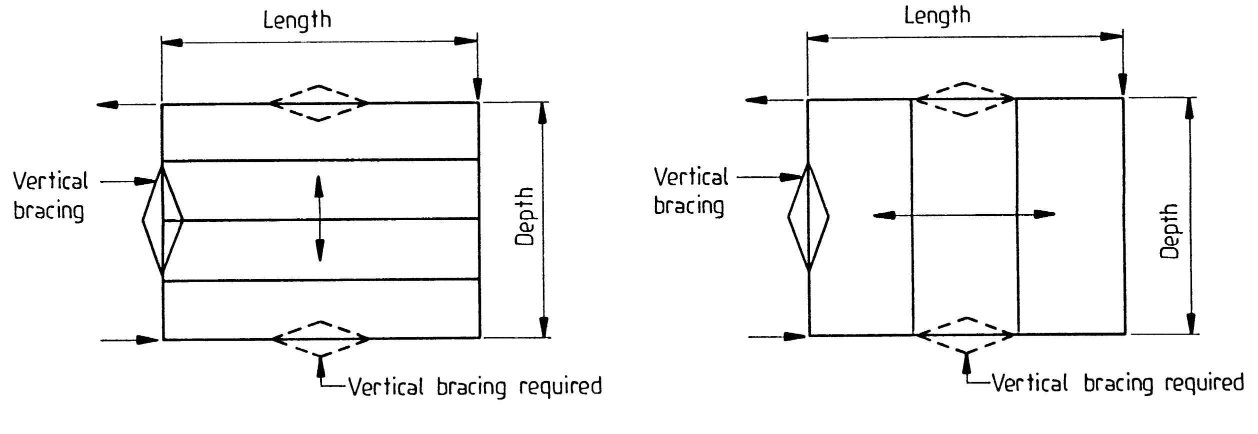
The following recommendations apply to discrete openings.

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 greater 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.



a) Sheeting spanning perpendicular

b) Sheeting spanning parallel

Figure 18 — Cantilevered diaphragms

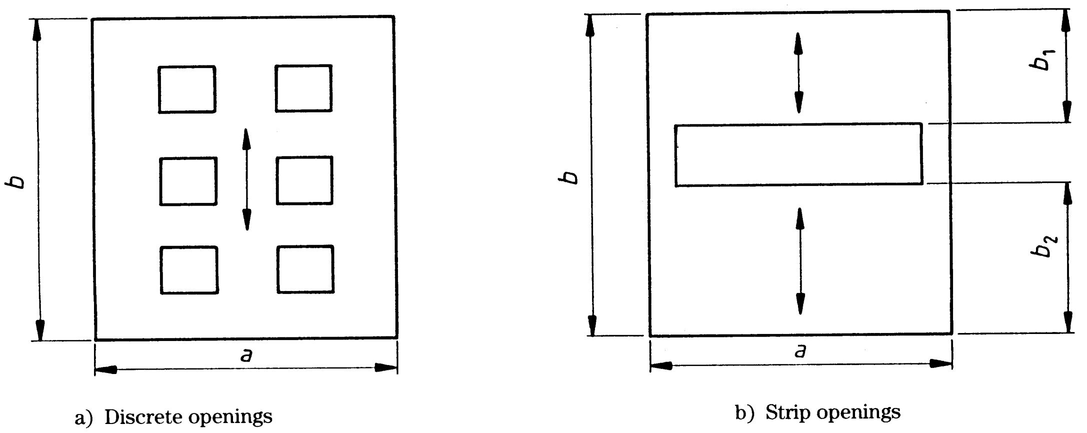


Figure 19 — Openings in a diaphragm

e) Recommendations c) and d) result in a maximum area of openings of 12½ % of the area in each panel. Openings up to 15% of the area may be allowed if further detailed calculations are made in accordance with the reference in **8.1**.

8.3.3 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 no 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.

8.3.4 Flexibility of diaphragms with discrete openings

The increase in shear flexibility due to openings may be taken into account by a multiplying factor to be applied to the value of c obtained from Table 9 and Table 13. The multiplying factor is as follows.

a) For shallow sheeting ($h \leq 50$ mm):

— every corrugation fastened: $\dfrac{1}{1 - 4A/ab}$

— alternate corrugations fastened: $\dfrac{1}{1 - 2.5A/ab}$

b) For deep sheeting (50 mm $< h \leq 85$ mm):

— every corrugation fastened: $\dfrac{1}{1 - 5A/ab}$

— alternate corrugations fastened: $\dfrac{1}{1 - 3A/ab}$

where

A is the area of openings in a panel (in mm^2);

a is the width of the shear panel (in mm);

b is the depth of the shear panel (in mm).

8.3.5 Deflection

For a diaphragm assembly in which some panels contain openings and others do not, the expressions for deflection given in **5.2.2** and **6.3.2** are not applicable. In such cases the mid-length deflection should be calculated as the sum of (shear force in panel × shear flexibility of panel) considered over half the length of the diaphragm.

8.3.6 Strip openings

Diaphragms with strip openings need not conform to **8.3.2** c). Provided b_1 and b_2 [see Figure 19 b)] do not differ by more than 20 % the effective depth may be taken as the same of b_1 and b_2. In other cases the diaphragm may be conservatively treated by taking the effective depth as the greater of b_1 and b_2. The trimmers on the long sides of the openings should conform to **4.2.1** f).

8.4 In-plane loads

8.4.1 Concentrated loads

Concentrated loads in the plane of the diaphragm should not be applied to the diaphragm other than at joints of structural members, unless it can be shown by tests or calculation that the local forces are within permissible limits.

8.4.2 Distributed loads

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

8.5 Diaphragms in multibay or asymmetrical structures

The analysis of clad frames given in **7.2** and **7.3** may only be applied to single bay fiat 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 illustrated in Figure 20 a) and Figure 20 b).

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 summarized by the single quantity c. The computer analysis consists of moving the individual frames close together 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 20 a) and rigid end gables may be modelled as shown in Figure 20 b).

Figure 20 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 joints with the same coordinates 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 by a member of length L (in mm), modulus of elasticity E (in kN/mm^2) and equivalent cross-sectional area A (in mm^2), where $A = L/cE$.

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 in accordance with BS 5950-4:1993. They may then be utilized without further checking as horizontal diaphragms to resist transient horizontal loads such as wind forces and seismic forces as shown in Figure 21. Shear transfer into the diaphragms may be effected by means of the beam/deck fasteners or by means of through-deck welded shear studs (if provided).

8.6.2 General conditions

The following recommendations apply to floor diaphragms.

a) Floor diaphragms should conform to **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 case (3) of Figure 4, shear connectors or their equivalent should be used at the end gables.

d) The fastener arrangements should conform to **4.3.2**.

e) Floor diaphragms with openings should conform to **8.3**.

8.6.3 Type 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.

In this Part of BS 5950, consideration is only given to floors of type a). Floors of type b) will normally be stronger and stiffer than floors of type a), so the treatment described will be conservative for floors of this type.

8.6.4 Erection stage and final stage

Calculations on composite sheet steel/concrete floors should be made at the following stages:

a) in the erection stage, when the 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.

With regard to 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 as 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 **6.2**.

8.6.6 Deflection of steel deck alone

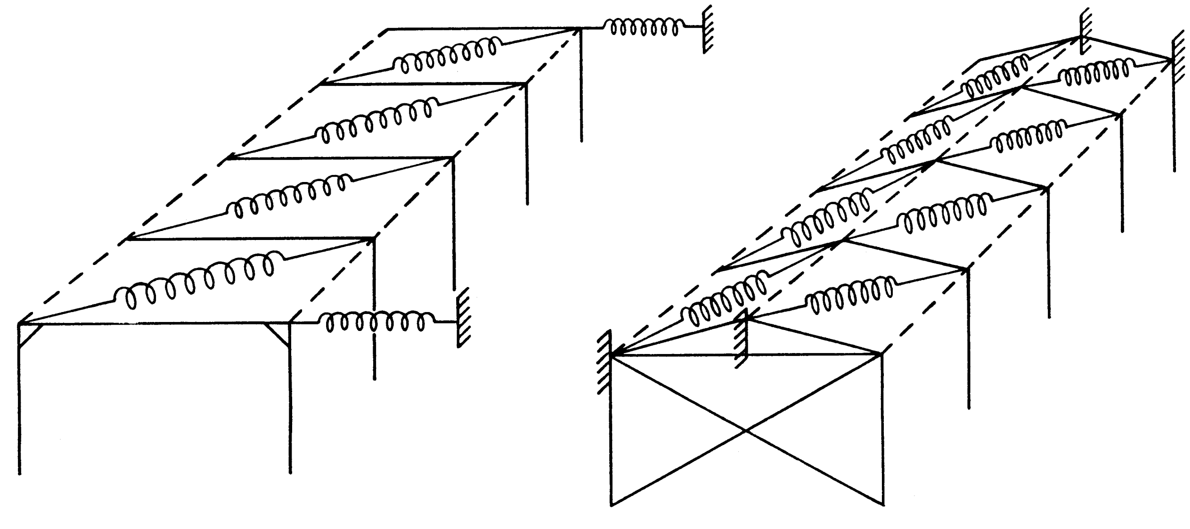
The deflection of steel deck acting alone as a diaphragm in the erection stage should be calculated in accordance with **5.2** or **6.3**.

8.6.7 Strength of combined steel deck and concrete

In the completed composite floor, the concrete should be assumed to carry all the shear force at the sheet seams and to prevent end collapse of the sheeting profile. It should not be assumed to strengthen the connections between the sheet and the supporting structure. The strength of the composite floor acting as a diaphragm should thus be calculated in accordance with **5.1** or **6.2** excluding seam capacity (see **5.1.1.2** and **6.2.1.2**) and end collapse of the profile (see **5.1.3.2** and **6.2.3.2**).

8.6.8 Deflection of combined steel deck and concrete

In the completed composite floor, the concrete should be assumed to prevent shear flexibility due to profile distortion ($c_{1.1}$) and deformation at the seam fasteners ($c_{2.2}$). The flexibility of the composite floor acting as a diaphragm should thus be calculated in accordance with **5.2** or **6.3** excluding the contribution of these items.



a) Flat roof building

b) Pitched roof building

Figure 20 — Mathematical models of sheeted buildings

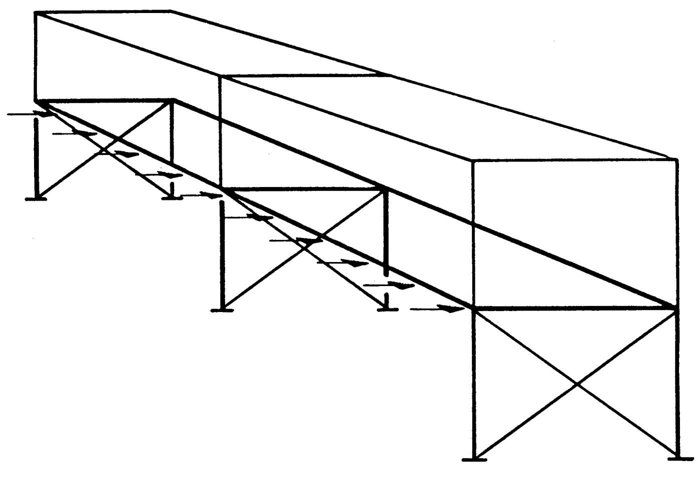


Figure 21 — Horizontal floor diaphragms and braced frames

Section 9. Diaphragm bracing

9.1 General

In addition to its main function as bracing in the plane of the roof, stressed skin action in roof sheeting or decking, and in floor decking, may be used to provide diaphragm bracing as follows:

a) lateral bracing to beams or rafters;

b) wind bracing to end gables;

c) wind bracing to eaves.

If a diaphragm provides more than one function, the diaphragm, including fasteners, should be designed for the sum of the relevant loading cases.

9.2 Lateral bracing to beams or rafters

9.2.1 Sheet used as diaphragm bracing

Sheeting or decking used as diaphragm bracing to provide lateral restraint to beams or rafters should satisfy the following conditions.

a) For a single beam, the diaphragm should be capable of resisting a total force of not less than 2½ % of the maximum factored force in the compression flange, distributed uniformly along the length of the beam.

b) For two or three parallel beams, the diaphragm should be capable of resisting a total force of not less than the sum of the lateral restraint forces required for each beam, obtained as in item a).

c) For more than three parallel beams, the diaphragm should be capable of resisting a total force of not less than the sum of the three largest lateral restraint forces obtained as in item a).

d) The sheeting or decking should be fixed in accordance with **4.2.1** g), **4.2.1** h) and **4.2.1** i), and should be capable of transferring the restraint forces to the beams's effective points of support.

The treatment given in **9.2.2** to **9.2.4** has been simplified by considering only the most common failure modes of diaphragm bracing. For unusual or heavily loaded diaphragms other possible failure modes should also be considered (see **5.1.1.5** b) and **5.1.3**; see also R. M. Lawson and D. A. Nethercot, "Lateral stability of I-beams restrained by profiled sheeting" [2], for further information).

9.2.2 Sheet perpendicular to beam: three or less parallel beams

Sheeting or decking spanning perpendicular to the beams should be considered as a diaphragm of length equal to the span of the beam and depth not greater than the lesser of:

a) the beam spacing × the number of beams; and

b) the span of the beam.

This is illustrated in Figure 22 for the case of three beams. The capacities of the various failure modes should be greater than the maximum shear force V at the ends of the diaphragm as given in the following cases.

1) Sheeting fastened on four sides:

— seam capacity $n_s F_s + \dfrac{\beta_1}{\beta_3} n_p F_p > V$

— shear connector fastener capacity $n_{sc} F_{sc} > V$

2) Sheeting fastened on two sides (to beams only):

— seam capacity $n_s F_s + \dfrac{\beta_1}{\beta_3} n_p F_p > V$

— sheet/beam fastener capacity $\beta_2\ n_p F_p > V$

where

F_p	is the design resistance of an individual sheet/purlin fastener (in kN) (see Table 5);
F_s	is the design resistance of an individual seam fastener (in kN) (see Table 5);
F_{sc}	is the design resistance of an individual sheet/shear connector fastener (in kN) (see Table 5);
n_p	is the number of beams in the diaphragm;
n_{sc}	is the number of seam fasteners per side lap (excluding those which pass through both sheets and the supporting beams);
n_s	is the number of sheet/shear connector fasteners on the member at the ends of the diaphragm;
V	is the applied shear force on a diaphragm (in kN);
β_1 and β_2	are factors to allow for the number of sheet/beam fasteners per sheet width (see Table 6);
β_3	is given by (distance between outermost fasteners across the sheet width) ÷ sheet width (see **5.1.1.2**).

9.2.3 Sheet perpendicular to beam: more than three parallel beams

Sheeting or decking spanning perpendicular to the beam should be considered as a diaphragm of length equal to the span of the beam and depth not greater than the span of the beam. The capacity of the diaphragm should be greater than the maximum shear force V at the ends of the diaphragm as given by the expressions 1) or 2) in **9.2.2**.

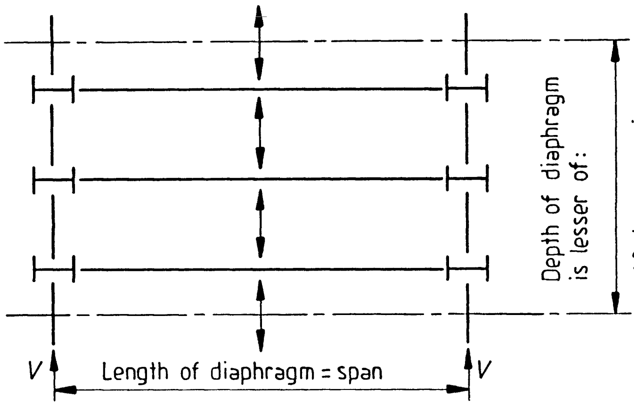


Figure 22 — Diaphragm for lateral restraint to three beams: sheet perpendicular to beams

9.2.4 Sheet parallel to beam

Sheeting or decking spanning parallel to the main beams does not offer continuous lateral restraint to the beam. Restraint is only offered at the positions of the secondary beams. The diaphragm should be calculated by equating the shear per unit depth of the diaphragm to the shear per unit length, and considering the equilibrium of a single sheet. The procedure is illustrated in Figure 23. The depth of the diaphragm for three or less parallel beams should be taken as in **9.2.2** and the depth for more than three parallel beams should be taken as in **9.2.3**.

9.3 Gable bracing

9.3.1 General

Wind bracing in the plane of the roof or floor between the gable and penultimate frames may be replaced by stressed skin action in the roof sheeting or decking, and in the floor decking, provided the conditions given in **9.3.2** and **9.3.3** are satisfied.

9.3.2 Sheet perpendicular to gable

Sheeting or decking spanning perpendicular to the gable should be considered as a diaphragm of length equal to the length of the gable and depth not greater than the lesser of:

a) 4 × beam spacings; and

b) the length of the gable.

This is illustrated in Figure 24. Secondary beams to introduce load into the diaphragm from the gable posts are not normally required.

The capacity of the diaphragm should be greater than the maximum shear force V at the ends of the diaphragm as given by the expressions 1) or 2) in **9.2.2**.

9.3.3 Sheet parallel to gable

Sheeting or decking spanning parallel to the gable should be considered as a diaphragm of length equal to the length of the gable and depth not greater than the lesser of

a) 2 × main beam spacings; and

b) the length of the gable.

This is illustrated in Figure 25. Secondary beams to introduce load into the diaphragm from the gable posts are normally required.

The diaphragm should be calculated by equating the shear per unit depth to the shear per unit length, and considering the equilibrium of a single sheet, as in Figure 23.

9.4 Eaves bracing

9.4.1 General

Wind bracing in the plane of the roof between the eaves puffin and the adjacent purlin may be replaced by stressed skin action in the roof sheeting or decking in fiat roof or pitched roof buildings, provided that:

a) the sheeting or decking spans perpendicular to the line of the eaves; and

b) the conditions given in **9.4.2** are satisfied.

9.4.2 Eaves diaphragm

Sheeting or decking spanning perpendicular to the line of the eaves should be considered as a diaphragm of length equal to the frame spacing and depth not greater than the lesser of:

a) 3 × purlin spacings; and

b) the frame spacing.

This is illustrated in Figure 26. The capacity of the diaphragm should be greater than the maximum shear force V at the ends of the diaphragm as given by the expressions 1) or 2) in **9.2.2**.

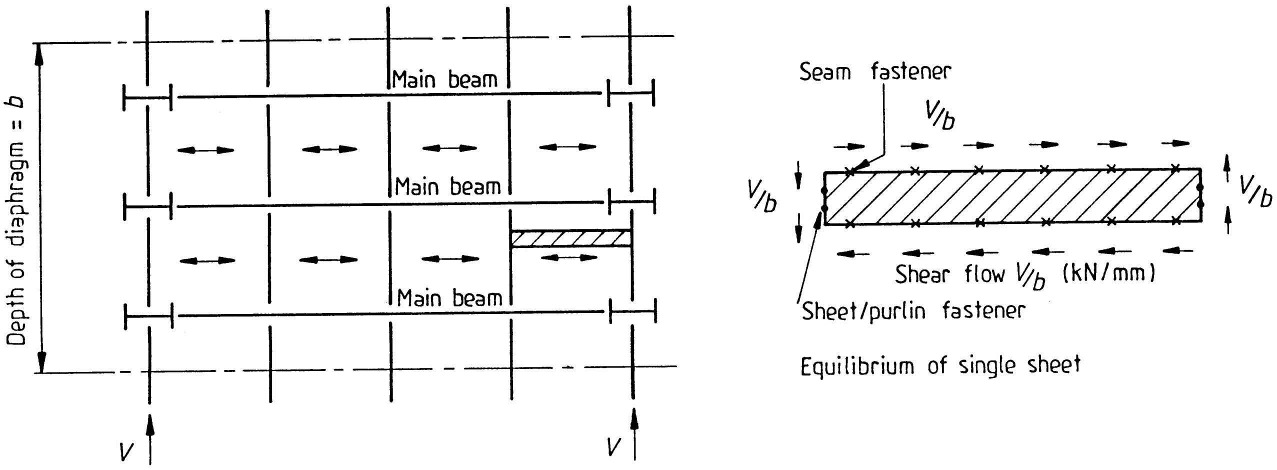


Figure 23 — Diaphragm for lateral restraint to three beams: sheet parallel to beams

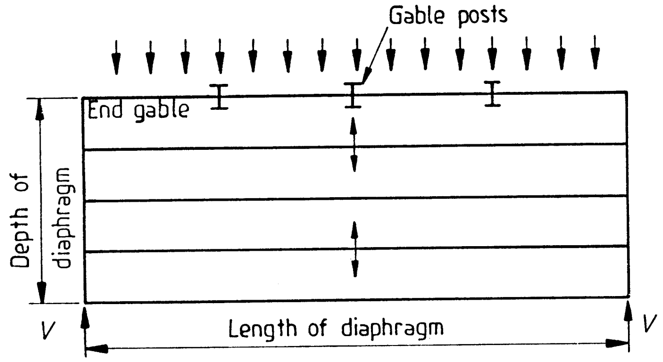


Figure 24 — Diaphragm for gable bracing: sheet perpendicular to gable

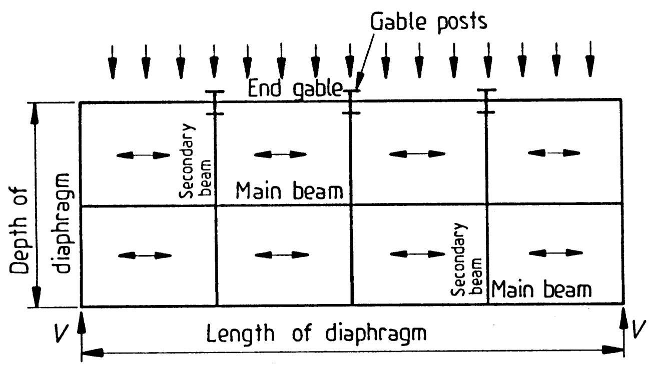


Figure 25 — Diaphragm for gable bracing: sheet parallel to gable

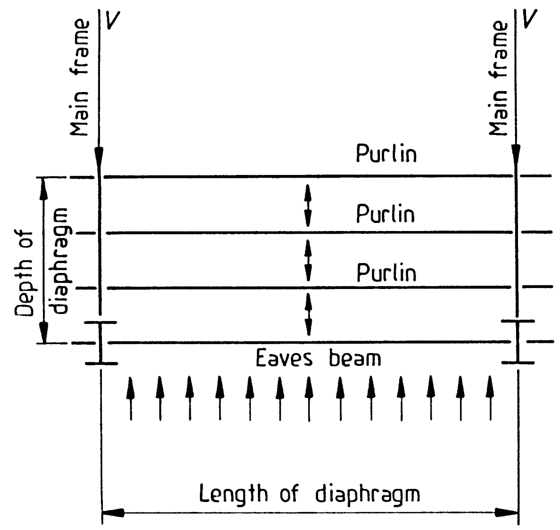


Figure 26 — Diaphragm for eaves bracing

Section 10. Folded plate roofs

10.1 General

The design of light gauge steel folded plate roofs involves consideration of the following elements shown in Figure 27:

a) fold line members along the apexes and valleys;

b) sheeting on each roof slope;

c) gable framing;

d) side sheeting or equivalent.

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 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 [see Figure 2 b)] without intermediate frames.

The stability of a folded plate roof depends entirely on stressed skin action; thus the sheeting should not be removed without proper consideration.

10.2 Design principles

With reference to Figure 28, the factored vertical load per roof slope is $wb \cos \varphi$ and the factored in-plane load per roof slope is

$$q = wb \cot \varphi$$

where

- q is the factored in-plane distributed line load on each roof slope (in kN/mm);
- w is the factored vertical load per unit plan area of roof (in kN/mm^2);
- b is the depth of the web of the folded plate, measured on the slope (in mm);
- φ is the angle of the web to the horizontal.

Each roof slope should be designed as a plate girder of depth b and length L between gable frames under a distributed load q (see Figure 29). 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:

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

b) 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 section **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 capacity and deflection of the roof slope should be calculated. The angle φ should normally lie within the range of 35° to 45°.

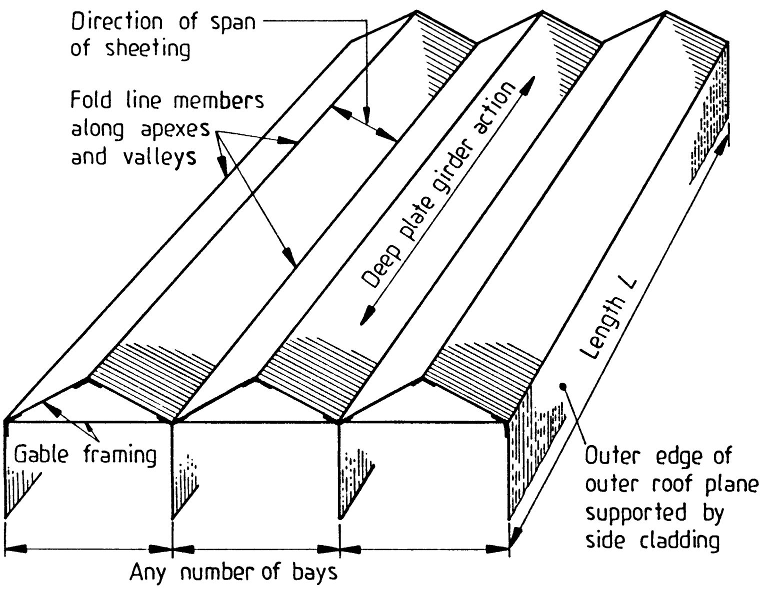


Figure 27 — General arrangement of a typical light gauge steel folded plate roof

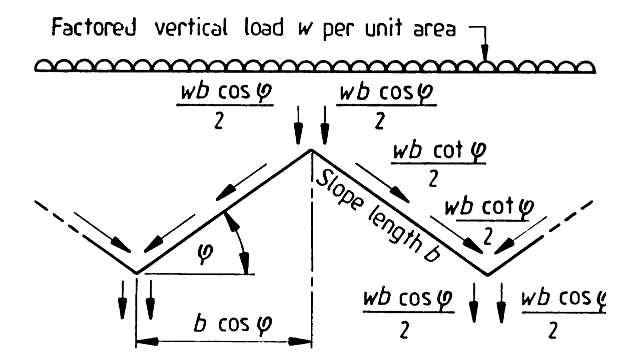


Figure 28 — Loads on a typical roof element per unit length

10.3 Design expressions for strength

10.3.1 Seam capacity

The seam capacity is given by

$$V_{\mathrm{ult}} = \left(n_{\mathrm{s}}F_{\mathrm{s}} + \frac{2\beta_1}{\beta_3}F_{\mathrm{p}}\right)\left(\frac{n_{\mathrm{sh}}}{n_{\mathrm{sh}}-2}\right) = \frac{qL}{2}$$

where

- F_{p} is the design resistance of an individual sheet/flange fastener (in kN) (see Table 5);
- F_{s} is the design resistance of an individual seam fastener (in kN) (see Table 5);
- L is the length of the folded plate roof between gables (in mm);
- 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 (in kN/mm);
- V_{ult} is the shear capacity associated with a given failure mode (in kN);
- β_1 is a factor to allow for the number of sheet/flange fasteners per sheet width;
- β_3 is given by (distance between outermost fasteners across sheet width) ÷ (sheet width) (see **5.1.1.1**).

10.3.2 Capacity of fasteners to end gables

The capacity of the fasteners to the end gables is given by

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

where

- F_{sc} is the design resistance of an individual sheet/end gable fastener (in kN) (see Table 5);
- 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} obtained as 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 take account of the effect of combined shear and prying action by the sheeting, the capacity of sheet/flange fasteners in shear is reduced by 40%. Hence it should be checked that

$$\frac{0.6bF_{\mathrm{p}}}{P} \geq V^*$$

where

- b is the depth of the web (in mm);
- p is the pitch of the sheet/flange fasteners (in mm);
- F_{p} is defined in **10.3.1**.

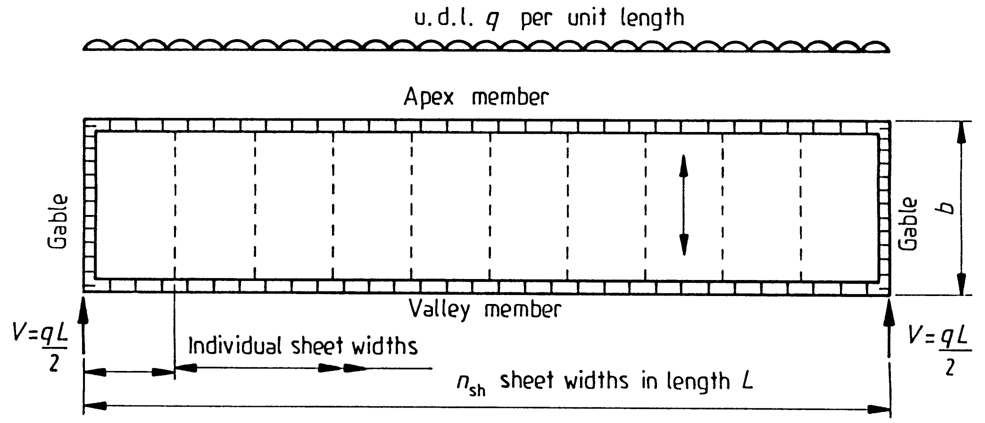


Figure 29 — Typical diaphragm girder of a folded plate roof

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:

a) every corrugation fastened
$0.0009t^{1.5}b\ Y_s/d^{0.5} \geqslant V^*$

b) alternate corrugations fastened
$0.0003t^{1.5}b\ Y_s/d^{0.5} \geqslant V^*$

where

- t is the net sheet thickness, excluding galvanizing and coatings (in mm);
- Y_s is the yield stress of the steel (in N/mm^2);
- d is the pitch of the corrugations (in mm);
- b is as defined in **10.3.4**.

10.3.6 Shear buckling

The following expressions for shear buckling include a 25 % reserve of safety. For fasteners in every trough, it should be checked that

$$\frac{28.8}{b}D_x^{1/4}D_y^{3/4} > V^*$$

For fasteners in alternate troughs, it should be checked that

$$\frac{14.4}{b}D_x^{1/4}D_y^{3/4} > V^*$$

where D_x and D_y are the orthogonal bending stiffnesses given by

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

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

where

- E is the modulus of elasticity of steel (205 kN/mm^2);
- I is the second moment of area of a single corrugation about its neutral axis (in mm^4) (see Figure 8);
- u is the perimeter length of a complete single corrugation of pitch d (in mm) (see Figure 8);
- 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 × the factored axial load. The design should be in accordance with BS 5950-5:1987.

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 18.

The sum of the component deflections in Table 18 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 (see Figure 30) is given by

$$\Delta_v = \Delta \operatorname{cosec} \varphi$$

The deflection under unfactored load should conform to the recommendations for general serviceability limit states given in **2.4.2**.

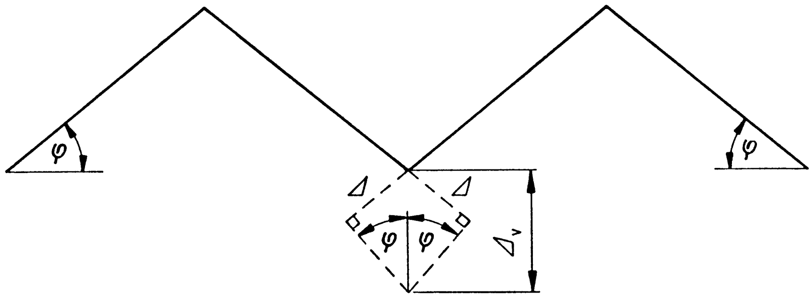


Figure 30 — Vertical deflection of a folded plate roof

Table 18 — Components of in-plane deflection of a diaphragm girder

Component of deflection	**Deflection** mm
Sheet deformation	
Sheet distortion	$\Delta_{1.1} = \dfrac{d^{2.5}KqL^2}{8Et^{2.5}b^2}$
Shear strain	$\Delta_{1.2} = \dfrac{(1+\nu)\{1+(2h/d)\}qL^2}{4Etb}$
Slip at fasteners	
Sheet to flange fasteners	$\Delta_{2.1} = \dfrac{s_p pqL^2}{4b^2}$
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 force	
Axial strain in fold line members	$\Delta_3 = \dfrac{qL^4}{38.4EAb^2}$
Total central deflection in plane of element	$\Delta = \Delta_{1.1} + \Delta_{1.2} + \Delta_{2.1} + \Delta_{2.2} + \Delta_{2.3} + \Delta_3$

Key

The symbols are as defined in **10.2** and **10.3** with the addition to the following:

- A is the semi-area of a fold line member (in mm^2);
- K is a sheeting constant (see **5.2.1.2** and Table 10 and Table 11);
- h is the height of the sheeting profile (in mm);
- s_p is the slip per sheet/flange fastener per unit load (in mm/KN) (see Table 5);
- s_s is the slip per seam fastener per unit load (in mm/KN) (see Table 5);
- s_{sc} is the slip per sheet/end gable fastener per unit load (in mm/kN) (see Table 5).

Section 11. Design by testing

11.1 General

The design of sheet steel shear diaphragms should normally be carried out by calculation in accordance with sections 1 to 10 of this Part of BS 5950.

Design by testing may be carried out if any of the following circumstances apply:

a) the properties of the steel or fasteners need to be determined;

b) the design or construction is not entirely in accordance with sections **1** to **10** of this Part of BS 5950 and use is made of experimental vertification as recommended in **2.1.2.7**;

c) adequate analytical procedures are not available for the component, assembly or structure to be designed;

d) it is desired to build a number of similar structures on the basis of a prototype test structure.

Any or all of the following tests may be undertaken in connection with cases a) to d):

— for case a), tensile tests on specimens to determine the material properties and lap joint tests to determine the shear charactertistics of sheeting fasteners;

— for cases b), c) and d), tests on structural details or connections, tests on shear panels, and tests on assemblies and complete structures.

Loading tests may be carried out for cases b), c) and d) on the following basis:

1) an acceptance test for confirmation of general structural behaviour;

2) a strength test against the required factored loads;

3) a test to determine the ultimate capacity and mode of failure.

Testing should be carried out in an established testing laboratory under the supervision of an appropriately qualified or experienced person. Whenever possible, the tests should be made in an independent testing laboratory, but if this is not possible they should be witnessed by an independent appropriately qualified or experienced person who should sign the test report.

11.2 Tensile tests

The material properties of the sheet steel should be obtained experimentally from coupon tests in accordance with BS EN 10002-1:1990.

11.3 Fastener lap joint tests

11.3.1 Test arrangement

The strength, flexibility and ductility of joints using fasteners can only be determined by testing. The test specimen should be so designed that the joint fails in the desired manner. A suitable test arrangement is shown in Figure 31 and suitable specimen dimensions are given in Table 19.

11.3.2 Test procedure

The rate of loading in the initial stages of testing should not exceed 1 kN/min and the rate of straining in the final stages should not exceed 1 mm/min.

Extension readings should initially be taken at load increments of approximately one-tenth of the expected ultimate load of the joint. A running plot should be maintained of the load against the average extension. When this indicates significant non-linearity then the load increments should be reduced in magnitude. Loading should be continued until the applied load can no longer be maintained.

11.3.3 Number of tests

For each type of fastener and each nominal thickness of sheet at least five tests should be carried out. If any one of these tests results in an ultimate load which differs from the mean ultimate load by more than 10 % of the mean, then at least two further tests should be carried out. The test results should be evaluated in accordance with **11.3.4**.

If it is believed that non-representative results are obtained from any test, the test may be rejected provided it is replaced by two or more equivalent tests. Any test so rejected should be included in the test report (see **11.8**) and the reason for the rejection clearly stated.

11.3.4 Characteristic and design strengths

The characteristic tearing resistance F_k of a fastener in steel sheet should be obtained from

$F_k = F_m - k \times SD$

where

F_m is the mean ultimate tearing resistance obtained from the tests;

k is a coefficient which depends on the number of tests (see Table 20);

SD is the standard deviation where, if the ultimate tearing resistance in a test is F,

$$SD = \sqrt{\left\{\frac{\Sigma(F - F_m)^2}{n - 1}\right\}}$$

or

$$SD = \sqrt{\left\{\frac{n\Sigma F^2 - (\Sigma F)^2}{n\,(n - 1)}\right\}}$$

n is the number of tests.

The design resistance of a fastener should be taken as $F_k/1.11$.

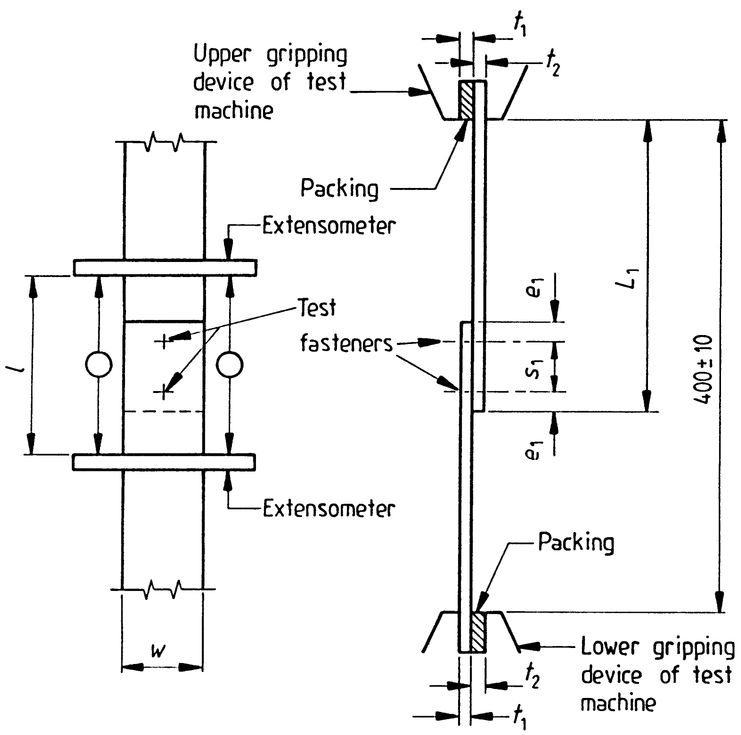


Figure 31 — Arrangement of fastener lap joint test

Table 19 — Dimensions for fastener lap joint tests

Dimensions in millimetres

Fastener diameter d	Specimen dimensions				
	w	L_1	e_1	s_1	l
⩽ 6.5	60	260	30	60	150
> 6.5	$10d$	$200 + 10d$	$5d$	$10d$	$20d + 30$
Tolerance	± 2	± 5	± 1	± 1	± 5

Table 20 — Number of tests and coefficient of standard deviation

Number of tests n	Coefficient n
3	3.15
4	2.68
5	2.46
6	2.33
8	2.19
10	2.10
16	1.98
20	1.93
∞	1.64

11.3.5 Fastener slip

The design value of fastener slip s should be obtained from the fastener lap joint tests using the relationship

$$s = \frac{s_m}{0.6F_k}$$

where

s_m is the mean slip at a load of $0.6F_k$ (in mm/kN);

F_k is the characteristic tearing resistance of a fastener (in kN).

The mean slip s_m should be determined as follows with reference to Figure 32. The slip of each lap joint (i.e. joint movement), s_1, s_2, etc. should be obtained from the graphs at a load of $0.6F_k$. The mean slip s_m is then the mean of these values.

11.4 Tests on components and structures

11.4.1 General

The tests described in **11.4.2** to **11.4.6** may be carried out on the following:

a) structural details or connections;

b) shear panels;

c) assemblies and complete structures.

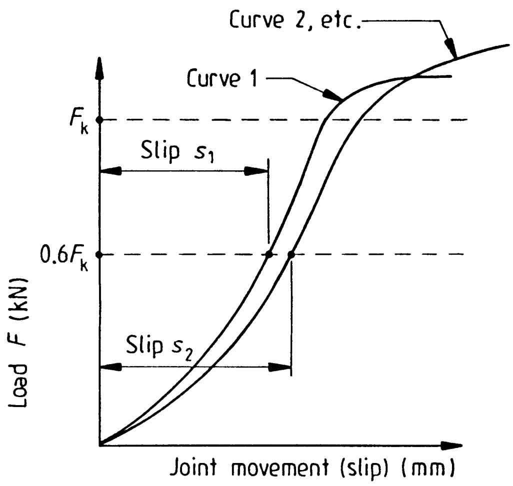


Figure 32 — Typical load/slip graphs of fasteners

The test procedure and type of test (i.e. acceptance test, strength test and test to failure) are generally similar for all three cases and are given in **11.4.2** to **11.4.5**. The determination of shear flexibility is given in **11.4.6**. Arangements which are specific to each case are given in **11.5** to **11.7**.

11.4.2 Test procedure

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.

11.4.3 Acceptance test

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

At a load equal to 1.2 × unfactored (dead load + imposed load), the load should be maintained for 15 min. At this stage the plot should be substantially linear. 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.

11.4.4 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 respects to the prototype.

Before the strength test is carried out, the specimen should first be submitted to and satisfy the acceptance test described in **11.4.3**.

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 min. 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.

11.4.5 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 specimen 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. Before the test, the specimen should first satisfy the strength test described in **11.4.4**.

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 then determine subsequent load increments.

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

$$\text{design capacity} = K_1 \times \left(\frac{\text{design strength}}{\text{averaged yield strength}}\right) \times (\text{ultimate test load})$$

where the averaged yield strength of the specimen is determined from coupon tests.

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

For a single test K_t should be taken as 0.9 and for two or more related tests K_t should be taken as 1.0 provided that the lowest of the individual ultimate test loads is used. If the resulting design capacity falls below the design capacity confirmed by the strength test, the latter should be taken.

11.4.6 Shear flexibility

The design value of shear flexibility of the component (in mm/kN), where applicable, should be obtained from the graph of load against deflection as follows:

$$\text{shear flexibility} = \frac{\Delta}{0.6 V_{des}}$$

where

Δ is the mean deflection at a load of $0.6V_{des}$ (in mm);

V_{des} is the design capacity (in kN).

11.5 Structural details or connections

Tests on structural details or connections used in a stressed skin diaphragm (e.g. purlin/rafter connections; see Table 7) should be carried out if these components influence the shear capacity or shear flexibility of the diaphragm and are not easily calculated.

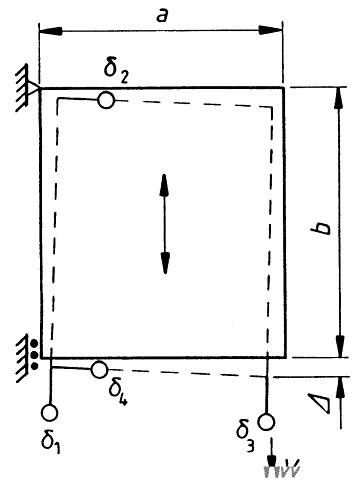


Figure 33 — Arrangement for shear test on panel

11.6 Shear panels

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 panel may be obtained. With reference to Figure 33, this is obtained as follows:

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

11.7 Assemblies and complete structures

Tests on assemblies and complete structures should be carried out in accordance with **11.4** to confirm serviceability (acceptance test), to confirm the calculated capacity (strength test), and to determine the true design capacity (test to failure). The test results, especially any observed weaknesses, may be advantageously used by the manufacturers to improve the design of a prototype.

Tests on prototype structures should, whenever possible, be carried out in the presence of manufacturers so that those responsible for design can observe the actual behaviour of structures loaded to failure.

11.8 Test report

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

a) the identity of the structure or component and all material properties;

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 after the start of the test at which they were taken;

f) a record of observations during the test;

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

Annex A (informative) Worked example 1: single shear panel with sheeting spanning perpendicular to the length of the diaphragm

A.1 General

This example illustrates the case of a single shear panel with sheeting spanning perpendicular to the length of the diaphragm. It gives calculations of shear capacity and shear flexibility under various fixing conditions. The example shows the application of section **5**. Throughout the example, references to the appropriate parts of the text are given in square brackets at the right-hand side of the page.

A.2 Problem

Determine the design shear capacity and shear flexibility of the panel of decking shown in Figure A.1 when the panel is 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. Regard the panel as a cantilever diaphragm (see Figure 18 a) and column 2) of Table 9).

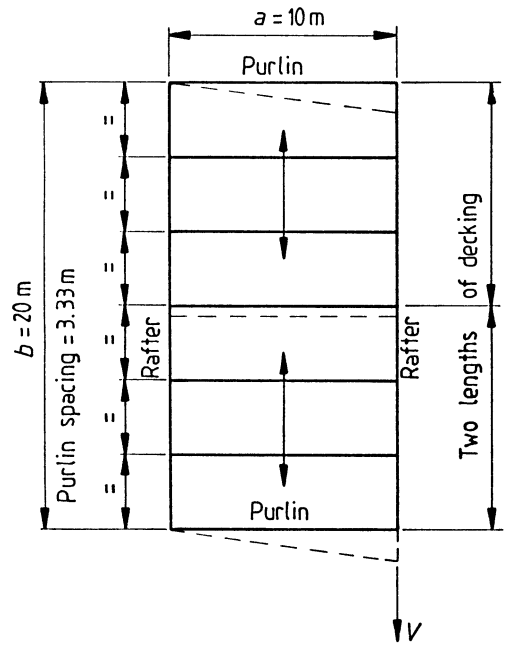


Figure A.1 — Worked example 1: single shear panel

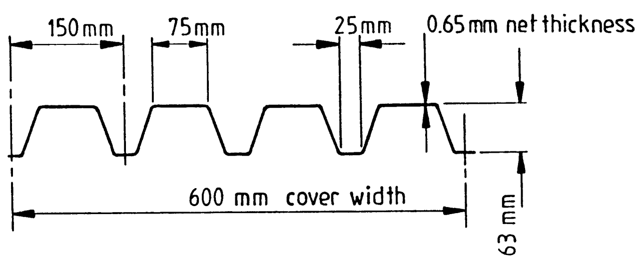


Figure A.2 — Decking profile for worked example 1

A.3 Data

Purlins:	254 mm × 146 mm Universal beam × 1 kg/m.
Decking profile:	As shown in Figure A.2.
Decking lengths:	10.2 m.
Grade of steel:	Fe E 280 G conforming to BS EN 10147
Sheet/purlin fasteners:	3.7 mm diameter cartridge fired pins.
Sheet/rafter fasteners (for fastening on four sides):	3.7 mm diameter cartridge fired pins at 333 mm centres.
Seam fasteners:	4.8 mm self-drilling screws at 333 mm centres.

A.4 Values of variables

a = 10×10^3 mm

A = 40.0×10^2 mm^2

b = 20×103 mm

d = 150 mm

E = 205 kN/mm^2

F_p = 0.65×5.0

= 3.25 kN (see Table 5)

F_s = 0.65×2.5

= 1.62 kN (see Table 5)

F_{sc} = 3.25 kN (= F_p)

h = 63 mm

K_1 = 0.278 (see note)

K_2 = 2.97 (see note)

l = 75 mm

n_b = 2

n_f = 5 (every trough fastened)

n_f = 3 (alternate troughs fastened)

n_p = 7

$$n_s = \left(\frac{3.33 \times 10^3}{333} - 1\right) \times 6 = 54$$

n_{sc} = $9 \times 6 + 7 = 61 = n'_{sc}$

n_{sh} = 10 × 1 000/600 = 17 **[5.1.1]**

p = 150 (every trough fastened)

p = 300 (alternate troughs fastened)

s_p = 0.10 mm k/N (see Table 5)

s_s = 0.25 mm/kN (see Table 5)

s_{sc} = 0.10 mm/kN (see Table 5)

t = 0.65 mm

Y_s = 280 N/mm^2

α_1 = 0.85 (see Table 8 for $n_p = 4$)

α_2 = 0.50 (see Table 8 for $n_p = 7$)

α_3 = 0.64 (see Table 8 for $n_p = 7$)

α_4 = $1 + 0.3 \times 2 = 1.6$ (see case (5) in Table 12)

For every trough fastened ($n_f = 5$):

β_1 = 1.13 (see case (2) in Table 6)

β_2 = 1.25 (see Table 6)

β_3 = 1.0 (see **5.1.1.2**)

For alternate troughs fastened ($n_f = 3$):

β_1 = 1.0 (see case (2) in Table 6)

β_2 = 1.0 (see Table 6)

β_3 = 1.0 (see **5.1.1.2**)

NOTE For the profile shown $l/d = 75/150 = 0.5$, $h/d = 63/150 = 0.42$ and $\theta = 21.6°$. K_1 and K_2 are found by interpolation from Table 10 and Table 11, for fasteners in every trough and alternate troughs respectively.

A.5 Design shear capacity

A.5.1 *Case a). Panel fixed on four sides, with fasteners in every trough*

Seam capacity $V_{\text{ult}} = n_s F_s + \dfrac{\beta_1}{\beta_3} n_p F_p = 54 \times 1.62 + \dfrac{1.13}{1.0} \times 7 \times 3.25 = 113.2\text{kN}$ **[5.1.1.2]**

Shear connector capacity $V_{\text{ult}} = n_{sc}F_{sc} = 61 \times 3.25 = 198.2$ kN **[5.1.1.3]**

Hence, design shear capacity $V^* = 113.2$ kN **[5.1.2]**

Sheet/purlin fasteners: check whether $0.6bF_p/p\alpha_3 \geqslant V^*$ **[5.1.3.1]**

$$0.6bF_p/p\alpha_3 = \frac{0.6 \times 20 \times 10^3 \times 3.25}{150 \times 0.64} = 406 \text{ kN}$$

Since 406 kN > 113.2 kN, this is satisfactory.

End collapse of profile: check whether $0.0009t^{1.5}b\,Y_s/d^{0.5} \geqslant V^*$ **[5.1.3.2]**

$$0.0009t^{1.5}bY_s/d^{0.5} = \frac{0.0009 \times 0.65^{1.5} \times 20 \times 10^3 \times 280}{150^{0.5}} = 215 \text{ kN}$$

Since 215 kN > 113.2 kN, this is satisfactory.

A.5.2 *Case b). Panel fixed on four sides, with fasteners in alternate troughs*

Seam capacity $V_{\text{ult}} = n_s F_s + \dfrac{\beta_1}{\beta_3} n_p F_p = 54 \times 1.62 + \dfrac{1.0}{1.0} \times 7 \times 3.25 = 110.2$ kN **[5.1.1.2]**

Shear connector capacity, as in **A.5.1**, $V_{\text{ult}} = 198.2$ kN **[5.1.1.3]**

Hence, design shear capacity $V^* = 110.2$ kN **[5.1.2]**

Sheet/purlin fasteners: check whether $0.6bF_p/p\alpha_3 \geqslant V^*$ **[5.1.3.1]**

$$0.6bF_p/p\alpha_3 = \frac{0.6 \times 20 \times 10^3 \times 3.25}{300 \times 0.64} = 203 \text{ kN}$$

Since 203 kN > 110.2 kN, this is satisfactory.

End collapse of profile: check whether $0.0003t^{1.5}b\,Y_s/d^{0.5} \geqslant V^*$ **[5.1.3.2]**

$$0.0003t^{1.5}b\,Y_s/d^{0.6} = \frac{0.0003 \times 0.65^{1.5} \times 20 \times 10^3 \times 280}{150^{0.5}} = 71.9 \text{ kN}$$

Since 71.9 kN < 110.2 kN, the design shear capacity should be reduced to 71.9 kN to prevent end collapse of the sheeting profile. This may be done by reducing the number of seam fasteners.

A.5.3 ***Case c). Panel fixed on two sides, with fasteners in every trough***

Seam capacity, as in **A.5.1**, V_{ult} = 113.2 kN [**5.1.1.2**]

Capacity of end sheet/purlin fasteners $V_{ult} = \beta_2 n_p F_p$ = 1.25 × 7 × 3.25 = 28.4 kN [**5.1.1.5** a)]

Capacity of purlin/rafter connections (using connection 8 in Table 7) $V_{ult} = n_p F_{pr}$

= 7 × 10.0 = 70.0 kN [**5.1.1.5** b)]

Hence, design shear capacity V^* = 28.4 kN [**5.1.2**]

End collapse: $0.0009t^{1.5}b\ Y_s/d^{0.6}$ = 215 kN as in **A.5.1**.

Since 215 kN > 28.4 kN, this is satisfactory [**5.1.3.2**]

A.5.4 ***Case d). Panel fixed on two sides, with fasteners in alternate troughs***

Seam capacity, as in A.5.2, V_{ult} = 110.2 kN [**5.1.1.2**]

Capacity of end sheet/purlin fasteners $V_{ult} = \beta_2 n_p F_p$ = 1.0 × 7 × 3.25 = 22.7 kN [**5.1.1.5** a)]

Capacity of purlin/rafter connections, as in **A.5.3**, V_{ult} = 70.0 kN [**5.1.1.5** b)]

Hence, design shear capacity V^* = 22.7 kN [**5.1.2**]

End collapse: $0.0003t^{1.5}b\ Y_s/d^{0.6}$ = 71.9 kN as in **A.5.2**.

Since 71.9 kN > 22.7 kN, this is satisfactory [**5.1.3.2**]

A.6 Shear flexibility

A.6.1 ***Case a). Panel fixed on four sides, with fasteners in every trough*** [**5.2** and column (2) of Table 9]

Shear flexibility due to profile distortion

$$c_{1.1} = \frac{10 \times 10^3 \times 150^{2.5} \times 0.85 \times 1.6 \times 0.278}{205 \times 0.65^{2.5} \times (20 \times 10^3)^2} = 0.037 \text{ mm/kN}$$

Shear flexibility due to shear strain

$$c_{1.2} = \frac{2 \times 10 \times 10^3 \times 1.3 \times 1.84}{205 \times 0.65 \times 20 \times 10^3} = 0.018 \text{ mm/kN}$$

Shear flexibility due to sheet/purlin fasteners

$$c_{2.1} = \frac{2 \times 10 \times 10^3 \times 0.10 \times 150}{(20 \times 10^3)^2} = 0.001 \text{ mm/kN}$$

Shear flexibility due to seam fasteners

$$c_{2.2} = \frac{2 \times 0.25 \times 0.10 \times 16}{2 \times 54 \times 0.10 + 1.13 \times 7 \times 0.25} = 0.063 \text{ mm/kN}$$

Shear flexibility due to sheet/shear connector fasteners

$$c_{2.3} = \frac{2 \times 0.10}{61} = 0.003 \text{ mm/kN}$$

Shear flexibility due to axial strain

$$c_3 = \frac{2 \times (10 \times 10^3)^3}{3 \times 205 \times 40.0 \times 10^2 \times (20 \times 10^3)^2} = 0.002 \text{ mm/kN}$$

$$c = \overline{0.124 \text{ mm/kN}}$$

A.6.2 ***Case b). Panel fixed on four sides, with fasteners in alternate troughs*** [**5.2** and column (2) of Table 9]

Shear flexibility due to profile distortion

$$c_{1.1} = \frac{10 \times 10^3 \times 150^{2.5} \times 0.85 \times 1.6 \times 2.97}{205 \times 0.65^{2.5} \times (20 \times 10^3)^2} = 0.399 \text{ mm/kN}$$

Shear flexibility due to shear strain, as in **A.6.1**,

$$c_{1.2} = 0.018 \text{ mm/kN}$$

Shear flexibility due to sheet/purlin fasteners

$$c_{2.1} = \frac{2 \times 10 \times 10^3 \times 0.10 \times 300}{(20 \times 10^3)^2} = 0.001 \text{ mm/kN}$$

Shear flexibility due to seam fasteners

$$c_{2.2} = \frac{2 \times 0.25 \times 0.10 \times 16}{2 \times 54 \times 0.10 + 1.0 \times 7 \times 0.25} = 0.064 \text{ mm/kN}$$

Shear flexibility due to sheet/shear connector fasteners, as in **A.6.1**,

$$c_{2.3} = 0.003 \text{ mm/kN}$$

Shear flexibility due to axial strain, as in **A.6.1**,

$$c_3 = \underline{0.002 \text{ mm/kN}}$$
$$c = \underline{0.487 \text{ mm/kN}}$$

A.6.3 ***Case c). Panel fixed on two sides, with fasteners in every trough*** [**5.2** and column (2) of Table 9]

Shear flexibility due to profile distortion, as in **A.6.1**, $c_{1.1} = 0.037$ mm/kN

Shear flexibility due to shear strain, as in **A.6.1**, $c_{1.2} = 0.018$ mm/kN

Shear flexibility due to sheet/purlin fasteners, as in **A.6.1**, $c_{2.1} = 0.001$ mm/kN

Shear flexibility due to seam fasteners, as in **A.6.1**, $c_{2.2} = 0.063$ mm/kN

Shear flexibility due to purlin/rafter connections (using connection 8 in Table 7) $c_{2.3} = \frac{2}{7}\left(2.6 + \frac{0.10}{1.25}\right) = 0.766$ mm/kN

Shear flexibility due to axial strain, as in **A.6.1**,

$$c_3 = \underline{0.002 \text{ mm/kN}}$$
$$c = \underline{0.887 \text{ mm/kN}}$$

A.6.4 ***Case d). Panel fixed on two sides, with fasteners in alternate troughs*** [**5.2** and column (2) of Table 9]

Shear flexibility due to profile distortion, as in **A.6.2**, $c_{1.1} = 0.399$ mm/kN

Shear flexibility due to shear strain, as in **A.6.1**, $c_{1.2} = 0.018$ mm/kN

Shear flexibility due to sheet/purlin fasteners, as in **A.6.2**, $c_{2.1} = 0.001$ mm/kN

Shear flexibility due to seam fasteners, as in **A.6.2**, $c_{2.2} = 0.064$ mm/kN

Shear flexibility due to purlin/rafter connections (using connection 8 in Table 7) $c_{2.3} = \frac{2}{7}\left(2.6 + \frac{0.10}{1.0}\right) = 0.771$ mm/kN

Shear flexibility due to axial strain, as in **A.6.1**,

$$c_3 = \underline{0.002 \text{ mm/kN}}$$
$$c = \underline{1.255 \text{ mm/kN}}$$

A.7 Summary of calculations

The calculations are summarized in Table A.1.

Table A.1 — Summary of calculations

Case	Four or two sides fastened	Fasteners in every or alternate troughs	Calculated design shear capacity kN	Criterion of strength	Calculated shear flexibility mm/kN
a)	Four	Every	113.2	Seam fasteners	0.124
b)	Four	Alternate	71.9	End collapse of profile	0.487
c)	Two	Every	28.4	End sheet/purlin fasteners	0.887
d)	Two	Alternate	22.7	End sheet/purlin fasteners	1.255

The calculations illustrate the following effects.

— Fasteners in alternate troughs give a much more flexible diaphragm than fasteners in every trough.

— Fixing a panel of sheeting on two sides only (to the purlins), rather than on four sides, reduces the shear capacity considerably.

— 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.

Annex B (informative) Worked example 2: panel assembly with sheeting spanning parallel to the length of the diaphragm

B.1 General

This annex illustrates the case of a panel assembly with sheeting spanning parallel to the length of the diaphragm. It gives calculations of shear capacity and shear deflection including the effects of shear buckling, insulation and openings. This example shows the applications of sections **6** and **8**. Throughout the example, references to the appropriate parts of the text are given in square brackets at the right-hand side of the page.

B.2 Problem

Determine the design shear capacity and shear deflection of the roof deck diaphragm shown in Figure B.1, taking into account the following factors:

a) shear buckling of the decking;

b) insulation bonded to the top of the decking;

c) effect of openings.

Assume the axial strength of the edge members is adequate.

B.3 Data

Edge members:	90 mm × 90 mm × 10 mm equal angle.
Decking profile:	As in Annex A, worked example 1, except that the net thickness is 0.85 mm.
Decking lengths:	8.2 m.
Decking:	Fastened on all four sides.
Grade of steel:	Fe E 280 G conforming to BS EN 10147.
Sheet/rafter fasteners:	3.7 mm diameter cartridge fired pins in alternate troughs.
Sheet/edge member fasteners:	3.7 mm diameter cartridge fired pins at 500 mm centres.
Seam fasteners:	4.8 mm Monel blind rivets at 400 mm centres.
Loads:	The loads shown on the roof deck diaphragm are factored. The load factor is **1.4**.

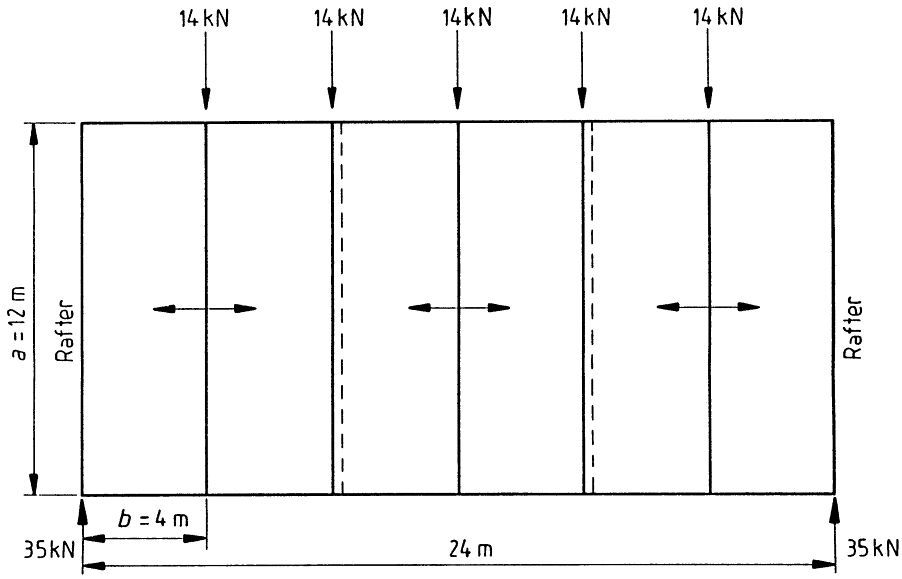


Figure B.1 — Worked example 2: roof deck diaphragm

B.4 Values of variables

a = 12×10^3 mm
A = 17.1×10^2 mm^2
b = 4×10^3 mm
d = 150 mm
E = 205 kN/mm^2
F_p = 0.85×5.0
= 4.25 kN (see Table 5)
F_s = 0.85×2.8
= 2.38 kN (see Table 5)
F_{sc} = 4.25 kN (= F_p)
h = 63 mm
K_2 = 2.97 (as in **A.4**)
L = 24×10^3 mm
n = 6 (see Table 14)
n_l = 3 (see Table 14)
n_s = $(4 \times 10^3/400 - 1) = 9$
n_{sc} = $4 \times 10^3/500 = 8$
n_{sh} = 12 000/600 = 20
p = 300 (alternate troughs fastened)
s_p = 0.10 mm/kN
s_s = 0.30 mm/kN
s_{sc} = 0.10 mm/kN
t = 0.85 mm
Y_s = 280 N/mm^2
α_5 = 0.9/2 = 0.45 (see Table 14 and Table 15)
β_1 = 1.0 [see Table 6, $n_f = 3$, case (2)]
β_2 = 1.0 (see Table 6, $n_f = 3$)
β_3 = 1.0 (see **5.1.1.2**)

B.5 Design shear capacity

B.5.1 *Capacity of decking* [6.2.1]

[6.2.1.2]

Seam capacity $V_{ult} = \frac{a}{b}\left(n_s F_s + \frac{\beta_1 F_p}{\beta_3}\right) = \frac{12 \times 10^3}{4 \times 10^3}\left(9 \times 2.38 + \frac{1.0}{1.0} \times 4.25\right) = 77.0 \text{ kN}$

[6.2.1.3]

Fasteners to edge members $V_{ult} = \frac{a}{b}(n_{sc} F_{sc}) = \frac{12 \times 10^3}{4 \times 10^3}(8 \times 4.25) = 102.0 \text{ kN}$

Hence, design shear capacity $V^* = 77.0$ kN [6.2.2]

Sheet/rafter fasteners: check whether $0.6aF_p/p > V^*$ [6.2.3.1]

$$0.6aF_p/p = \frac{0.6 \times 12 \times 10^3 \times 4.25}{300} = 102.0 \text{ kN}$$

Since 102.0 kN > 77.0 kN, this is satisfactory.

End collapse of profile: check whether $0.0003t^{1.5}a\, Y_s/d^{0.5} > V^*$ [6.2.3.2]

$$0.0003t^{1.5}a\, Y_s/d^{0.5} = \frac{0.0003 \times 0.85^{1.5} \times 12 \times 10^3 \times 280}{150^{0.5}} = 64.5 \text{ kN}$$

Since 64.5 kN < 77.0 kN, the design shear capacity should be reduced to 64.5 kN to prevent end collapse of the sheeting profile. This is still considerably greater than the maximum shear in the decking, 35 kN.

B.5.2 *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 (see Figure B.2)

$$I = \frac{72.1 \times 10^4}{1000} \times 150 = 108.1 \times 10^3 \text{ mm}^4$$

The perimeter length, u, of a single corrugation is obtained by calculation from the decking profile shown in Figure A.2. Thus $u = 236$ mm.

Hence bending stiffness $D_x = \frac{205 \times 0.85^3 \times 150}{12 \times (1 - 0.3^2) \times 236} = 7.33 \text{ kN·mm}$

and bending stiffness $D_y = \frac{205 \times 108.1 \times 10^3}{150} = 147\ 737 \text{ kN·mm}$

Shear buckling capacity: check whether $(14.4a/b^2)D_x^{1/4}\, D_y^{3/4} \geqslant V^*$

$$(14.4a/b^2)D_x^{1/4}\, D_y^{3/4} = \frac{14.4 \times 12 \times 10^3}{(4 \times 10^3)^2} \times 7.33^{1/4} \times 147\ 737^{3/4} = 134 \text{ kN}$$

Since 134 kN > 64.5 kN, this is satisfactory.

B.5.3 *Insulation bonded to decking* [5.6]

No increase in the design shear capacity may be permitted due to insulation.

B.5.4 *Effect of openings* [8.3.2]

All the recommendations of **8.3.2** should be satisfied. The maximum size of a single opening is determined in accordance with **8.3.2** c) and **8.3.2** d) as shown, i.e. $A = 6\ m^2$.

If the seam fastener spacing is maintained at 400 mm throughout, the shear capacity of the panel shown in Figure B.3 may be taken as 50 % of the shear capacity of a panel without openings. [8.3.3]

For the diaphragm arrangement shown in Figure B.4, with openings in the interior panels but no openings in the end panels, the design criteria are as follows:

end panels: $V^* \geqslant 35$ kN

interior panels: $0.5\ V^* \geqslant 21$ kN, i.e. $V^* \geqslant 42$ kN **[8.3.3]**

Since the capacity of a panel is V^* = 64.5 kN, the decking panels with openings are adequately strong.

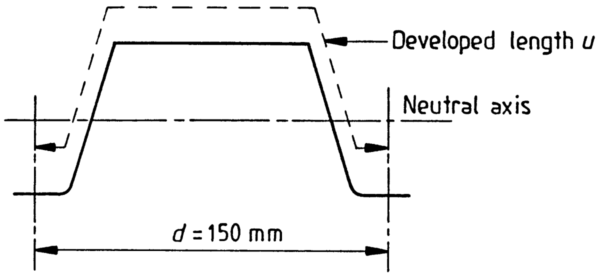


Figure B.2 — Decking profile for worked example 2

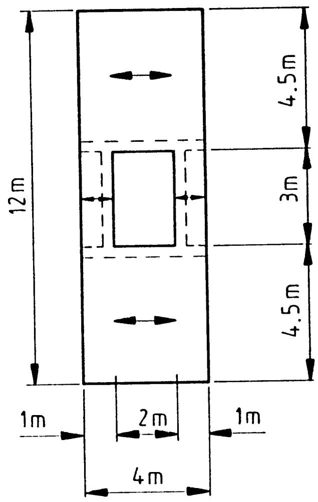


Figure B.3 — Shear panel with opening

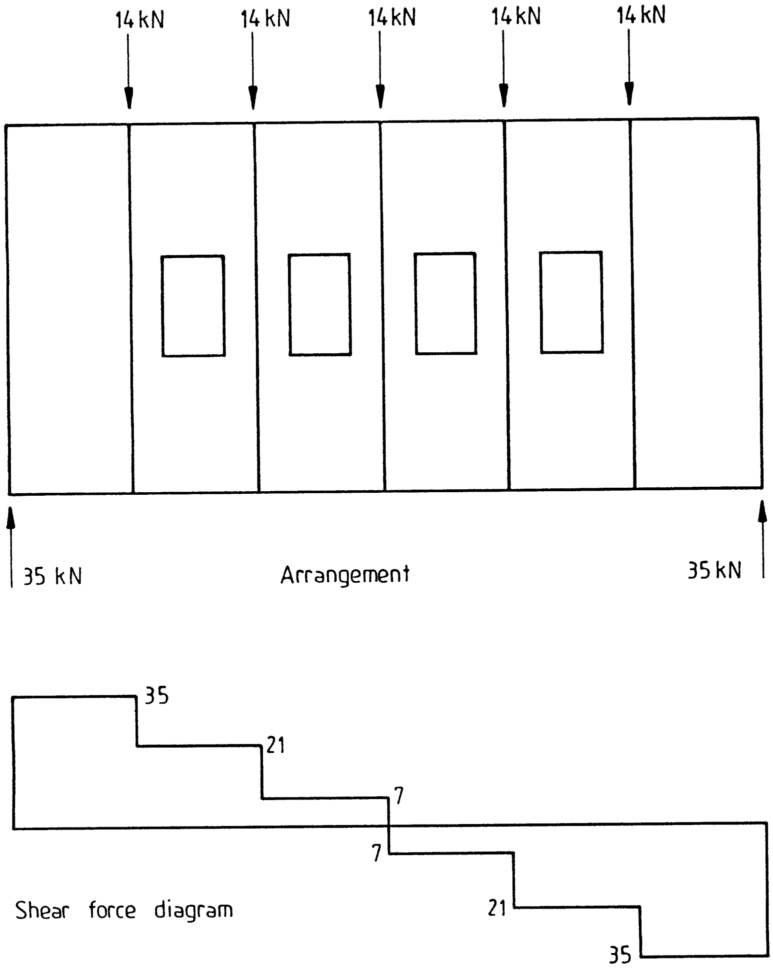


Figure B.4 — Panel assembly with openings

B.6 Shear deflection

[**6.3**]

B.6.1 *Shear flexibility of decking*

[column (1) of Table 18]

Shear flexibility due to profile distortion

$$c_{1.1} = \frac{12 \times 10^3 \times 150^{2.5} \times 0.45 \times 2.97}{205 \times 0.85^{2.5} \times (4 \times 10^3)^2} = 2.023 \text{ mm/kN}$$

Shear flexibility due to shear strain

$$c_{1.2} = \frac{2 \times 12 \times 10^3 \times 1.3 \times 1.84}{205 \times 0.85 \times 4 \times 10^3} = 0.082 \text{ mm/kN}$$

Shear flexibility due to sheet/rafter fasteners

$$c_{2.1} = \frac{2 \times 12 \times 10^3 \times 0.10 \times 300}{(4 \times 10^3)^2} = 0.045 \text{ mm/kN}$$

Shear flexibility due to seam fasteners

$$c_{2.2} = \frac{0.30 \times 0.10 \times 19}{9 \times 0.10 + 1.0 \times 0.30} = 0.475 \text{ mm/kN}$$

Shear flexibility due to sheet/edge member fasteners

$$c_{2.3} = \frac{2 \times 0.10}{8} = \underline{0.025 \text{ mm/kN}}$$

$$2.650 \text{ mm/kN}$$

Hence,

$$c' = \frac{(4 \times 10^3)^2}{(12 \times 10^3)^2} \times 2.650 = 0.294 \text{ mm/kN}$$

Shear flexibility due to axial strain

$$c_3 = \frac{6^2 \times (4 \times 10^3)^3}{4.8 \times 205 \times 17.1 \times 10^2 \times (12 \times 10^3)^2} = 0.010 \text{ mm/kN}$$

Hence, total shear flexibility of a panel of decking is

$$c = c' + c_3 = 0.304 \text{ mm/kN}$$

B.6.2 ***Insulation bonded to decking*** [**5.6** b) and **6.3.1.2**]

The effect of insulation, for a profile height of 63 mm, is to multiply the value of $c_{1.1}$ by a factor of 0.5 to give a modified value of $(c_{1.1} + c_{1.2} + c_{2.1} + c_{2.2} + c_{2.3})$ equal to 1.638 mm/kN.

Hence, modified $c' = \dfrac{(4 \times 10^3)^2}{(12 \times 10^3)^2} \times 1.638 = 0.182$ mm/kN.

Therefore, modified $c = 0.182 + 0.010 = 0.192$ mm/kN.

B.6.3 ***Effect of openings*** [**8.3.4**]

The multiplying factor to be applied to the shear flexibility of panels with openings, for a profile height of 63 mm is

$$\frac{1}{1 - 3A/ab} = \frac{1}{1 - (3 \times 6 \times 10^8)/(12 \times 4 \times 10^6)} = 1.60$$

i.e. modified c for a panel with insulation and openings = 0.192 × 1.60 = 0.307 mm/kN.

B.6.4 ***Deflection of roof deck diaphragm*** [**8.3.5**]

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 Figure B.4, the mid-length deflection at the unfactored load is given by

$$\Delta = \frac{1}{1.4}(35 \times 0.192 + 21 \times 0.307 + 7 \times 0.307) = 10.9 \text{ mm}$$

B.7 Summary of calculations

The calculations illustrate the following:

— the procedure for checking the shear buckling capacity of the decking;

— the method of checking the design shear capacity of diaphragms with openings;

— the effect of insulation, bonded to the top of the decking, in reducing the shear flexibility of decking which is fastened in alternate troughs;

— the effect of roof openings in increasing the shear flexibility of decking;

— the method of calculating the maximum shear deflection of an assembly of panels with different shear flexibilities.

Annex C (informative)
Worked example 3: flat roof building with panels of decking as in worked example 1

C.1 General

This annex illustrates the case of a flat roof building with sheeting spanning perpendicular to the length of the diaphragm. It gives calculations for the clad building behaviour with pin-jointed and rigid jointed frames. This example shows the application of section **7**. Throughout the example, references to the appropriate parts of the text are given in square brackets at the right-hand side of the page.

C.2 Problem

Calculate the strength and deflection of a flat roof building under side load with panels of decking as in Annex A, worked example 1, case a) (see Figure C.1). Consider the following cases:

a) pin-jointed frames, in which the roof diaphragm carries the side load alone;

b) rigid jointed frames, in which the frames carry the side load alone;

c) rigid jointed frames with roof diaphragm, which combine to carry the side load.

C.3 Data

Sheeting:	Sheeting spanning perpendicular to length of diaphragm. Panel fixed on four sides, fasteners in every trough.
Frames:	533 mm × 210 mm Universal beam × 92 kg/m.
Loads:	The loads shown in the diagram are already factored. The load factor is **1.4**.

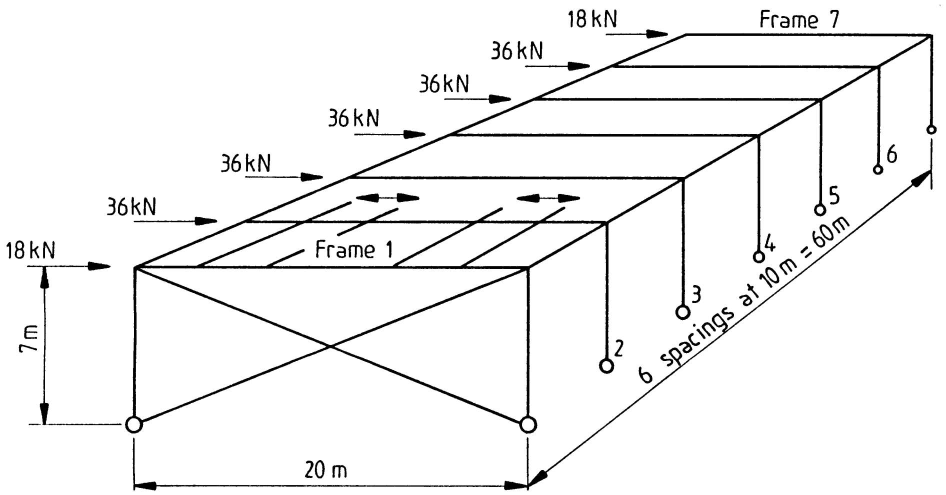


Figure C.1 — Worked example 3: flat roof building

C.4 Case a). Roof diaphragm alone

C.4.1 *Diaphragm capacity* **[A.5.1]**

From worked example 1, case a), the design shear capacity of a panel is 113.2 kN which is greater than the maximum shear force in a panel, 90 kN (see Figure C.2). The diaphragm capacity is therefore satisfactory.

C.4.2 *Diaphragm deflection* [**A.6.1**, **5.2.1.1** and column (1) of Table 9]

The shear flexibility of the single diaphragm panel considered in worked example 1, case a), is a little different from that of a panel in a panel assembly. The correct value of shear flexibility is as follows:

Shear flexibility due to profile distortion	$c_{1.1}$ = 0.037 mm/kN
Shear flexibility due to shear strain	$c_{1.2}$ = 0.009 mm/kN
Shear flexibility due to sheet/purlin fasteners	$c_{2.1}$ = 0.001 mm/kN

Shear flexibility due to seam fasteners $c_{2.2} = 0.063$ mm/kN

Shear flexibility due to sheet/shear connector fasteners

$$c_{2.3} = \frac{4 \times (6+1) \times 0.10}{6^2 \times 61} = 0.001 \text{ mm/kN}$$

Shear flexibility due to axial strain

$$c_3 = \frac{6^2 \times (10 \times 10^3)^3 \times 0.64}{4.8 \times 205 \times 40.0 \times 10^2 \times (20 \times 10^3)^2} = 0.015 \text{ mm/kN}$$

$$c = 0.126 \text{ mm/kN}$$

The mid-length deflection of the diaphragm alone under unfactored load is given by **[5.2.2]**

$$\Delta = \frac{36}{1.4} \times (6^2/8) \times 0.126 = 14.6 \text{ mm}$$

C.5 Case b). Rigid jointed frames alone

The frame flexibility k (in mm/kN), may be calculated by normal elastic methods and is found to be 1.22 mm/kN (see Figure C.3). Under an unfactored side load per frame of 36/1.4 kN, the sway deflection is (36/1.4) × 1.22 = 31.4 mm.

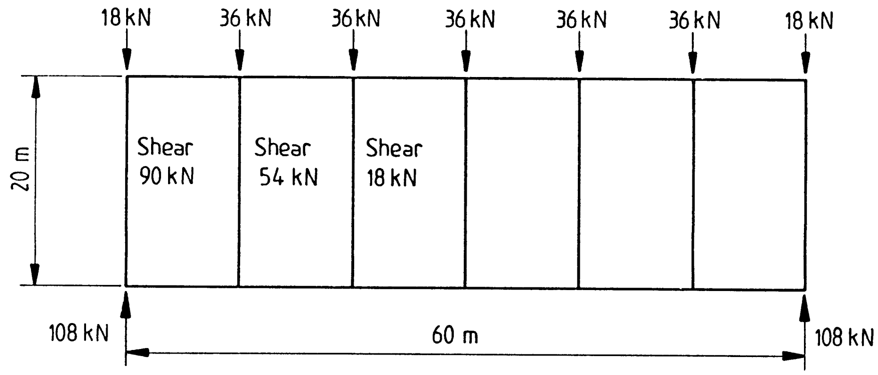


Figure C.2 — Roof diaphragm

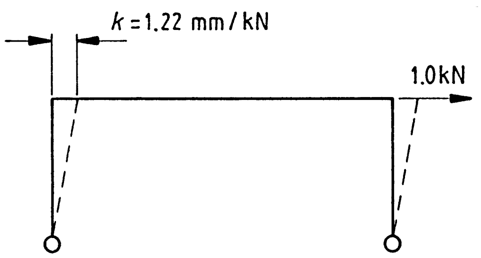


Figure C.3 — Frame flexibility of a frame shown in Figure C.1

C.6 Case c). Rigid jointed frames and diaphragms **[7.2.1]**

The relative flexibility of the diaphragm to the frame is given by

$$r = c/k = \frac{0.126}{1.22} = 0.104$$

[Table 16]

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 16:

— for frame 2, $\eta = 0.191$;

— for frame 3, $\eta = 0.299$;

— for frame 4, $\eta = 0.334$.

The sidesway deflection of the central frame (frame 4) in the clad building under unfactored load is thus $0.334 \times 31.4 = 10.5$ mm.

The factored forces and shears on the roof diaphragm are shown in Figure C.4. It is noted that the shear in the end panel is considerably less than the design capacity of the diaphragm.

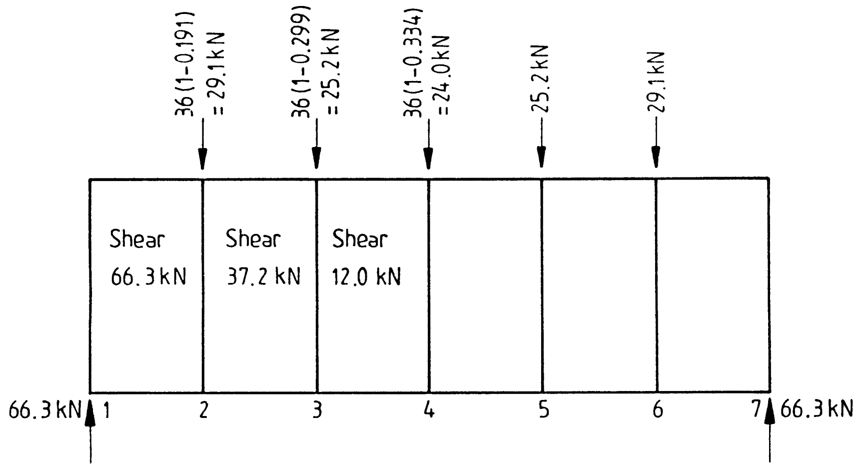


Figure C.4 — Factored forces and shears on the roof diaphragm

C.7 Summary of calculations

The calculations illustrate the following.

— The shear flexibility of the decking, fastened in every trough, is considerably less than the frame flexibility.

— In the central frame of the clad building, the maximum bending moment under side load is only one-third of that in a bare frame.

— The sway deflection of the central frame in the clad building is only one-third of that of the bare frame.

— The shear in the end panel when the diaphragm acts in conjunction with the frames is considerably less than when the diaphragm acts alone.

Annex D (informative)
Worked example 4: flat roof building with panels of decking as in worked example 2

D.1 General

This annex illustrates the case of a flat roof building with sheeting spanning parallel to the length of the diaphragm. It gives calculations of clad building deflections with all frames loaded, and with one frame only loaded. This example shows the further application of section **7**. Throughout the example, references to the appropriate parts of the text are given in square brackets at the right-hand side of the page.

D.2 Problem

Calculate the strength and deflection of a fiat roof building under side load with panels of decking as in Annex B (see Figure D.1). Consider the following cases.

a) Combined action of the rigid frames and roof diaphragm: all frames loaded.

b) Combined action of the rigid frames and roof diaphragm: central frame only loaded.

D.3 Data

Sheeting:	Sheeting spanning parallel to length of diaphragm. Panel fixed on four sides, fasteners in alternate troughs.
Frames:	457 mm × 191 mm Universal beam × 82 kg/m.
Loads:	The loads shown in the diagram are already factored. The load factor is **1.4**.

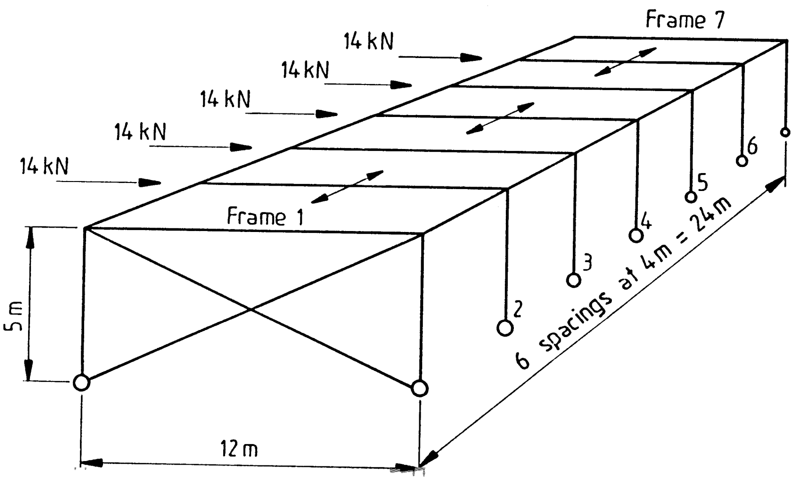


Figure D.1 — Worked example 4: flat roof building under side load

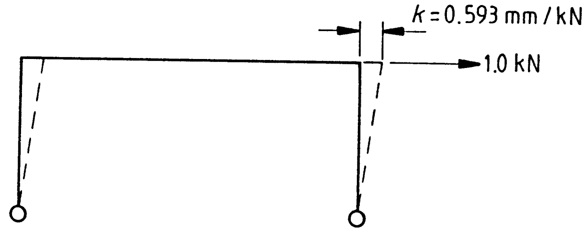


Figure D.2 — Frame flexibility of a frame shown in Figure D.1

D.4 Case a). All frames loaded **[7.2.1]**

Consider the case of the decking, fastened in alternate troughs, with insulation bonded to the top of the decking, without roof openings. From **B.6.2** the shear flexibility c is 0.192 mm/kN. [**B.6.2**]

The frame flexibility may be calculated by normal elastic methods and is found to be k = 0.593 mm/kN (see Figure D.2).

The relative flexibility of the diaphragm to the frame is given by

$$r = c/k = \frac{0.192}{0.593} = 0.324$$

For a building with seven frames, the reduction factors to be applied to the sway forces and deflections are obtained by interpolation from Table 16: [Table 16]

— for frame 2, η = 0.390;

— for frame 3, η = 0.583;

— for frame 4, η = 0.641.

The sway deflection of a bare frame under unfactored load is given by

$$\Delta = \frac{14}{1.4} \times 0.593 = 5.9 \text{ mm}$$

The sway deflection of the central frame in the clad building when all frames are loaded with unfactored loads is given by

$$\Delta = \frac{14}{1.4} \times 0.593 \times 0.641 = 3.8 \text{ mm}$$

D.5 Case b). Central frame only loaded **[7.2.3]**

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 17. For a building of seven frames, with r equal to 0.324, this further factor is found to be 2.51. [Table 17]

The sway deflection of the central frame is then

$$\Delta = \frac{3.8}{2.51} = 1.5 \text{ mm}$$

D.6 Summary of calculations

The calculations illustrate the following.

— 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.

— When the central frame only is loaded, the sway deflection of the clad frame, compared with that of the bare frame, is very small.

— The effect of the fasteners in alternate troughs rather than in every trough (see Annex C) reduces the benefit of stressed skin action, but still results in moments and deflections in the central frame of 0.64 × the bare frame values.

Annex E (informative)
Worked example 5: pitched roof building with panels of sheeting

E.1 General

This annex illustrates the case of a pitched roof building under vertical load. It shows the calculation of clad building behaviour using elastic analysis and gives forces for the plastic design of clad frames. This example shows the further application of section **7**. Throughout the example, references to the appropriate parts of the text are given in square brackets at the right-hand side of the page.

E.2 Problem

E.2.1 Calculate the strength and eaves deflection of a sheeted pitched roof portal frame building under vertical dead load and imposed load (see Figure E.1).

E.2.2 Determine the forces to be used in the plastic design of the sheeted frame.

E.3 Data

Frames:	Two-pinned portal frames of section 457 mm × 191 mm Universal beam × 67 kg/m, yield moment M_y = 356.4 kN·m.
Sheeting panels:	Assume that the panels are all similar with a design shear capacity of 100 kN and shear flexibility of 0.12 mm/kN.
Loads:	Assume that the dead load is 0.5 kN/m^2 (load factor **1.4**) and that the imposed load is 0.80 N/m^2 (load factor 1.6).

E.4 Elastic calculations for the bare frame

The total factored vertical load per frame is given by

$W = 24 \times 6 \times (0.5 \times 1.4 + 0.80 \times 1.6) = 285.1$ kN

The bending moment diagram for the frame may be obtained by conventional elastic analysis as the sum of the following:

a) non-spread moment distribution with movement at the eaves prevented by a horizontal eaves force; and

b) spread moment distribution due to reversal of this eaves force.

The bending moment diagrams for cases a) and b), and their sum are shown in Figure E.2.

It is seen from the bending moment diagram in Figure E.2 c) that the maximum bending moment in the bare frame is 368.8 kN·m which exceeds the yield moment of the section, 356.4 kN·m. The design is therefore inadequate.

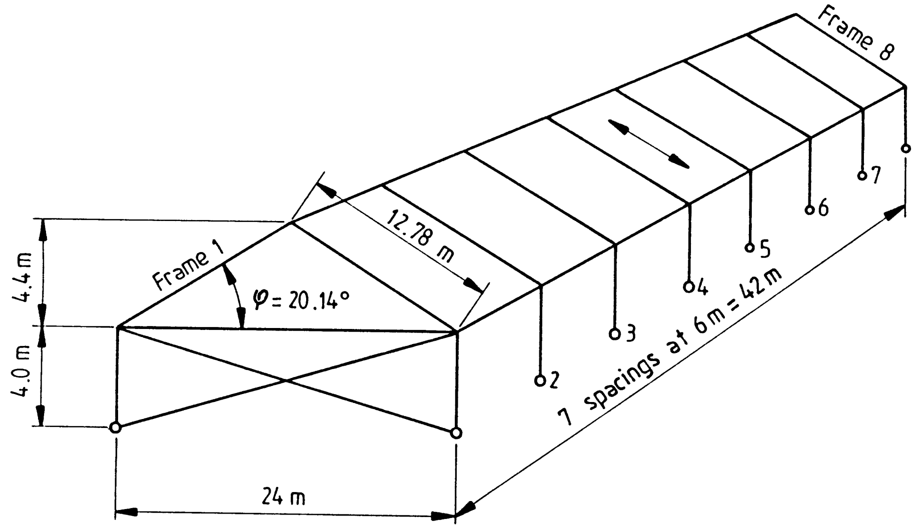


Figure E.1 — Worked example 5: sheeted pitched roof portal frame building

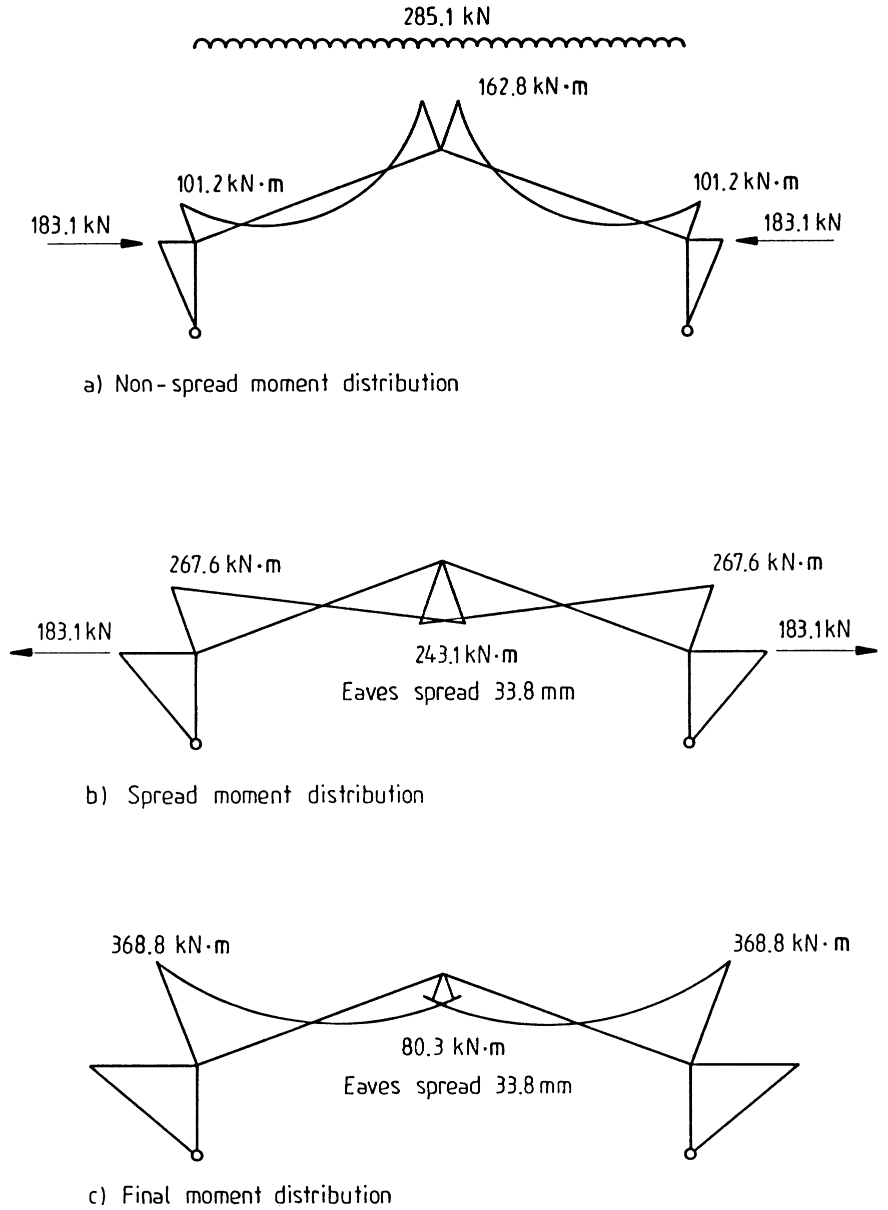


Figure E.2 — Bending moment diagrams for the frame shown in Figure E.1

E.5 Elastic calculations for the clad frame [7.2.2]

E.5.1 *Elastic calculations for the sheeting*

In the plane of the sheeting the shear flexibility c (see Figure E.3) is given by

$$c = \frac{\Delta \cos \varphi}{V/\cos \varphi} = \frac{\Delta}{V} \cos^2 \varphi$$

In the horizontal direction, the horizontal shear flexibility c_h (see Figure E.4) is given by

$$c = \frac{\Delta}{V}$$

Hence, from the equations for c and c_h, $c_h = c \sec^2 \varphi$

For the panel in question, $c = 0.12$ mm/kN and $\varphi = 20.14°$

Hence $c_h = 0.136$ mm/kN

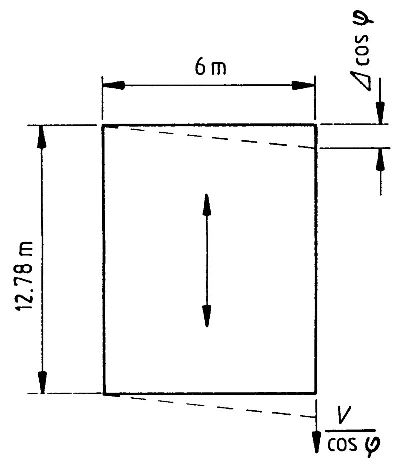


Figure E.3 — Panel of sheeting

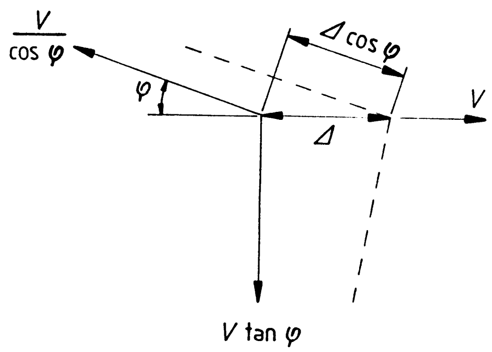


Figure E.4 — Eaves detail

E.5.2 ***Elastic calculations for the frame*** [Figure 13 b]

From Figure E.2 b), the spread flexibility of the frame is given by

$$k_{sp} = \frac{33.8}{183.1} = 0.184 \text{ mm/kN}$$

E.5.3 ***Elastic calculations for the clad frames***

The relative flexibility of the sheeting to the frame is given by

$$r_{sp} = \frac{c_h}{k_{sp}} = \frac{0.136}{0.184} = 0.739$$

so that the reduction factors to be applied to the spread moments in an eight frame building (see Table 16) are: [Table 16]

— for frame 2, $\eta = 0.560$;

— for frame 3, $\eta = 0.796$;

— for frame 4, $\eta = 0.883$.

For the worst case, frame 4, the spread moment distribution for the clad frame is 0.883 × the distribution in Figure E.2 b), i.e. as shown in Figure E.5.

The final bending moment diagram in the clad frame is the sum of the diagrams in Figure E.2 a) and Figure E.5. The maximum bending moment in the frame is 101.2 + 236.3 = 337.5 kN·m which is 8.5 % less than the maximum moment in the bare frame and is less than the yield moment of the section. The eaves spread of the clad frame is 29.8 mm which is 11.8 % less than the eaves spread of the bare frame.

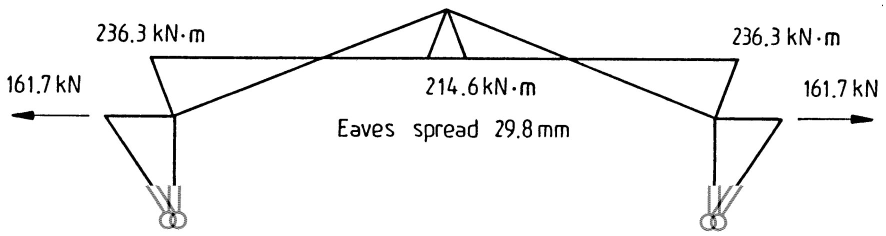


Figure E.5 — Spread moment distribution in the clad frame

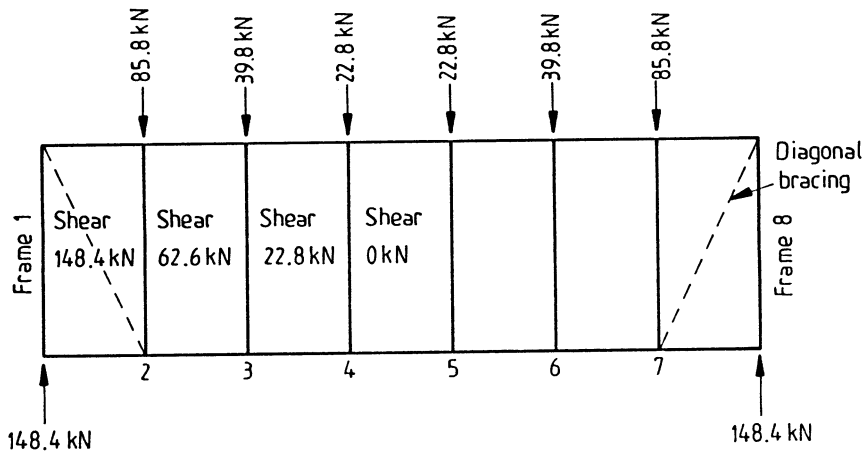


Figure E.6 — Forces in the plane of the sheeting

E.5.4 ***Elastic calculations for the forces on the sheeting***

The horizontal forces on the sheeting at frames 2, 3 and 4 are respectively:

— at frame 2, $V = (1 - 0.560) \times 183.1 = 80.6$ kN;

— at frame 3, $V = (1 - 0.796) \times 183.1 = 37.4$ kN;

— at frame 4, $V = (1 - 0.883) \times 183.1 = 21.4$ kN.

Hence, the forces in the plane of the sheeting, $V/\cos \varphi'$, are shown in Figure E.6.

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 also be used as erection bracing), the critical panel moves to the penultimate panel where the shear is 62.6 kN, which is less than the design shear capacity of the sheeting. This design is therefore adequate.

E.6 Forces for the plastic design of the clad frame [7.3.3]

E.6.1 ***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 given by

$$V = R \cos \varphi = 33.3 \times 0.939 = 31.3 \text{ kN}$$

Each frame should then be plastically designed under the action of the factored forces shown in Figure E.7.

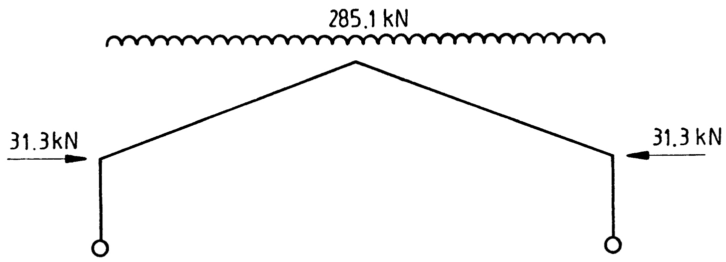


Figure E.7 — Factored forces for the plastic design of the clad frame with no end bracing

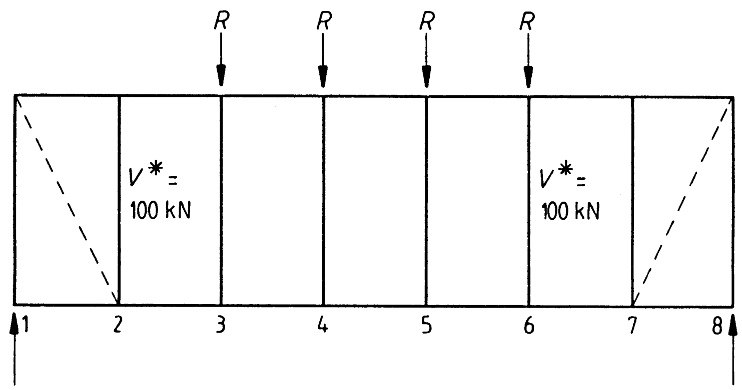


Figure E.8 — Roof diaphragm with end bracing

E.6.2 ***With end bracing***

The panel assembly is effectively reduced to five panels (see Figure E.8) so that

$$R = \frac{2V^*}{n-1} = \frac{2 \times 100}{5-1} = 50 \text{ kN}$$

and $V = R \cos \varphi = 46.9$ kN

Each frame may then be plastically designed under the action of the factored forces shown in Figure E.9.

E.7 Summary of calculations

The calculations illustrate the following

— The roof sheeting may make a useful contribution to resisting vertical load, for roof pitches of the order of 20°.

— The resulting shear forces in the end panels may be substantial.

— By utilizing diagonal bracing in the end panels, the panel assembly may be strengthened considerably.

— Eaves forces, to be used in the plastic design of sheeted frames, are simply obtained.

NOTE Although the calculations are not given, the roof sheeting would make a substantial contribution to resisting horizontal load and sway deflection as follows:

$$k_{sw} = 1.48 \text{ mm/kN and } r_{sw} = \frac{0.136}{1.48} = 0.092$$

so that, from Table 16, the maximum value of η is 0.372, i.e the maximum moment and deflection in the clad frame due to horizontal eaves forces is only 37 % of that in the bare frame. [**7.2.2** Figure 14 a), Table 16]

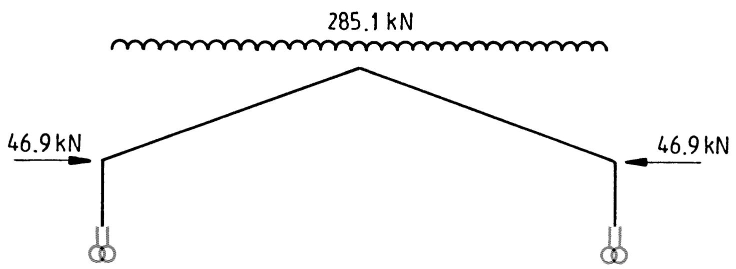


Figure E.9 — Factored forces for the plastic design of the clad frame with end bracing

Annex F (informative)
Worked example 6: composite steel deck/concrete floor

F.1 General

This annex illustrates the case of a composite floor deck diaphragm. It shows the calculation of shear capacity and shear deflection for the steel deck alone and for the completed floor. This example shows the application of section **8**. Throughout the example, references to the appropriate parts of the text are given in square brackets at the right-hand side of the page.

F.2 Problem

Check the diaphragm strength and shear deflection of the composite floor deck shown in Figure F.1, 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.

F.3 Data

Decking profile:	As shown in Figure F.2.
Decking:	Fastened to purlins only.
Decking lengths:	6.2 m.
Grade of steel:	Fe E 280 G conforming to BS EN 10147.
Sheet/purlin fasteners:	3.7 mm diameter cartridge fired pins in alternate troughs.
Sheet/end gable fasteners:	20 cartridge fired pins of diameter 3.7 mm.
Seam fasteners:	4.8 mm self-drilling screws (6 per 3 m span).
Edge member:	Area 40.0 cm^2.
Load:	Take the wind pressure as 1.10 kN/m^2. Assume it acts over an effective depth in the erection stage of 2.0 m per storey, and in the final stage of 3.5 m per storey. The load factor is 1.4.

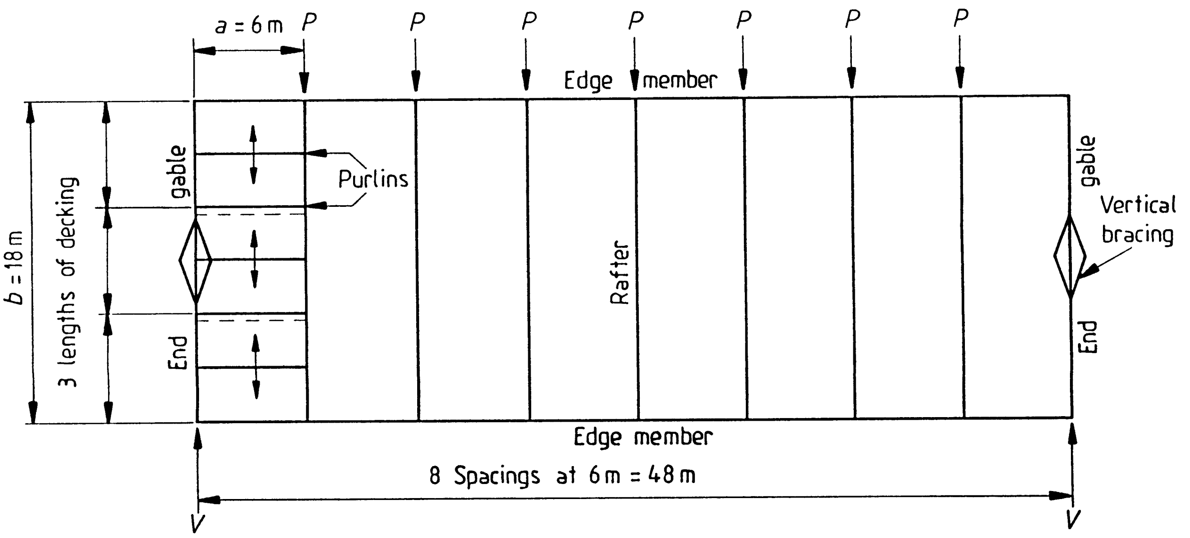


Figure F.1 — Worked example 6: composite floor deck

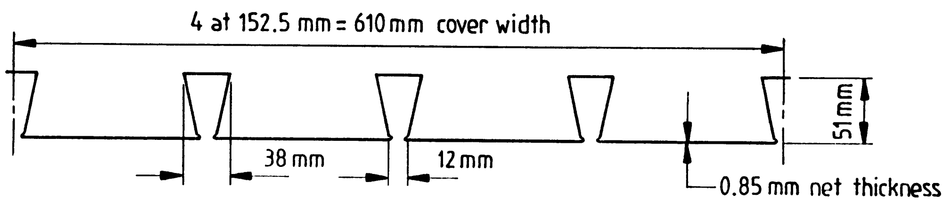


Figure F.2 — Decking profile for composite floor

F.4 Values of variables [**5.1** and **5.2**]

a = 6×10^3 mm
A = 40×10^2 mm^2
b = 18×10^3 mm
d = 152.5 mm
E = 205 kN/mm^2
F_p = 0.85×5.0
= 4.25 kN (see Table 5)
F_s = 0.85×2.5
= 2.12 kN (see Table 5)
F_{sc} = 4.25 kN ($= F_p$)
h = 51 mm
K_2 = 0.191 (see note)
l = 38 mm
L = 48×10^3 mm
n = 8
n_b = 3
n_f = 2 (alternate troughs fastened)
n_p = 7
n_s = $6 \times 6 = 36$

n_{sc} = 20

$n_{sh} = \frac{6000}{610} = 10$

p = 305 mm

s_p = 0.10 mm/kN (see Table 5)

s_s = 0.25 mm/kN (see Table 5)

s_{sc} = 0.10 mm/kN (see Table 5)

t = 0.85 mm

Y_s = 280 N/mm^2

α_1 = 1.0 (see Table 8 for $n_p = 3$)

α_2 = 0.50 (see Table 8 for $n_p = 7$)

α_3 = 0.64 (see Table 8 for $n_p = 7$)

α_4 = 1 + 0.3 × 3 = 1.9 (see case (5) in Table 12)

β_1 = 0.13 (see case (1) in Table 6)

β_2 = 1.0 (see Table 6)

$\beta_3 = \frac{2-1}{2} = 0.50$ (see **5.1.1.2**)

NOTE To determine K_2 consider the ribs to be rectangular (not dovetailed) i.e. $\theta = 0°$. Thus

$$\frac{t}{d} = \frac{38}{152.5} = 0.249$$

$$\frac{h}{d} = \frac{51}{152.5} = 0.334$$

From Table 11 by interpolation, $K_2 = 0.191$ [Table 11]

F.5 Factored loads on diaphragm

F.5.1 *Case a). Steel deck alone*

Factored panel load P = 1.10 × 6 × 2.0 × 1.4 = 18.5 kN

Factored end reaction V = 18.5 × 7/2 = 64.7 kN

F.5.2 *Case b). Completed composite floor*

Factored panel load P = 1.10 × 6 × 3.5 × 1.4 = 32.3 kN

Factored end reaction V = 32.3 × 7/2 = 113.2 kN

F.6 Design shear capacity [**5.1.1** and **8.6.5**]

F.6.1 *Case a). Steel deck alone* [**5.1.1.2**]

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

$$= 36 \times 2.12 + \frac{0.13}{0.50} \times 7 \times 4.25 = 84.1 \text{ kN}$$

Sheet/end gable fastener capacity $V_{ult} = n_{sc} F_{sc}$ = 20 × 4.25 = 85.0 kN [**5.1.1.3**]

Internal panel end fastener capacity $P_{ult} = \beta_2 n_p F_p$ = 1.0 × 7 × 4.25 = 29.8 kN [**5.1.1.5**]

Since 29.8 kN > 18.5 kN, this is satisfactory.

Hence, design shear capacity V^* = 84.1 kN [**5.1.2**]

Sheet/purlin fasteners: check whether $0.6bF_p/p\alpha_3 > V^*$ [**5.1.3.1**]

$$0.6bF_p/p\alpha_3 = \frac{0.6 \times 18 \times 10^3 \times 4.25}{305 \times 0.64} = 235 \text{ kN}$$

Since 235 kN > 84.1 kN, this is satisfactory.

End collapse of profile: check whether $0.0003t^{1.5}bY_s/d^{0.5} \geqslant V^*$ **[5.1.3.2]**

$$0.0003t^{1.5}bY_s/d^{0.5} = \frac{0.0003 \times 0.85^{1.5} \times 18 \times 10^3 \times 280}{152.5^{0.5}} = 96.0 \text{ kN}$$

Since 96.0 kN > 84.1 kN, this is satisfactory.

Also, the design shear capacity, 84.1 kN > maximum factored shear in the diaphragm (= V = 64.7 kN).

The strength of the diaphragm is therefore adequate.

F.6.2 *Case b). Completed composite floor* **[5.1.1.2]**

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} = 85.0 kN (< 113.2 kN);

b) for internal panel end fastener capacity, P_{ult} = 29.8 kN (< 32.3 kN).

Since these criteria are not satisfied, the following measures should be taken.

1) The number of sheet/end gable fasteners should be increased to

$$\frac{113.2}{4.25} = 26.6 \approx 27$$

2) The sheet/purlin fasteners should be doubled at the extreme edge members only.

NOTE This effectively increases the number of purlins from 7 to 9 so that $P_{ult} = \beta_2 n_p F_p = 1.0 \times 9 \times 4.25 = 38.2$ kN (> 32.3 kN; this is satisfactory).

F.7 Shear deflection

F.7.1 *Case a). Steel deck alone*

[**5.2.1**, **8.6.6**, and column (1) of Table 9]

Shear deflection due to profile distortion

$$c_{1.1} = \frac{6 \times 10^3 \times 152.5^{2.5} \times 1.0 \times 1.9 \times 0.191}{205 \times 0.85^{2.6} \times (18 \times 10^3)^2} = 0.014 \text{ mm/kN}$$

Shear deflection due to shear strain

$$c_{1.2} = \frac{2 \times 6 \times 10^3 \times 0.5 \times 1.3 \times 1.67}{205 \times 0.85 \times 18 \times 10^3} = 0.004 \text{ mm/kN}$$

Shear deflection due to sheet/purlin fasteners

$$c_{2.1} = \frac{2 \times 6 \times 10^3 \times 0.10 \times 305 \times 0.64}{(18 \times 10^3)^2} = 0.001 \text{ mm/kN}$$

Shear deflection due to seam fasteners

$$c_{2.2} = \frac{2 \times 0.25 \times 0.10 \times 9}{2 \times 36 \times 0.10 + 0.13 \times 7 \times 0.25} = 0.061 \text{ mm/kN}$$

Shear deflection due to sheet/end gable fasteners

$$c_{2.3} = \frac{4 \times 7}{8^2 \times 7}\left(0 + \frac{0.10}{1.0}\right) = 0.006 \text{ mm/kN}$$

Shear deflection due to axial strain

$$c_3 = \frac{8^2 \times (6 \times 10^3)^3 \times 0.64}{4.8 \times 205 \times 40 \times 10^2 \times (18 \times 10^3)^2} = 0.007 \text{ mm/kN}$$

$$c = 0.093 \text{ mm/kN}$$

The mid-length deflection of the decking diaphragm alone under unfactored load is given by: [**5.2.2**]

$$\Delta = \frac{18.5}{1.4} \times \frac{8^2}{8} \times 0.093 = 9.8 \text{ mm}$$

F.7.2 ***Case b). Completed composite floor*** [**5.2.1** and **8.6.8**]

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

$$\Delta = \frac{32.3}{1.4} \times \frac{8^2}{8} \times 0.018 = 3.3 \text{ mm}$$

F.8 Summary of calculations

The calculations illustrate the following.

— The procedure for calculating the strength and deflection of the steel deck diaphragm alone is as given in sections **5** and **6**.

— The wind loads on the steel deck diaphragm alone may be less than on the final steel deck/concrete diaphragm.

— The procedure for calculating the strength and deflection of the steel deck/concrete diaphragm is as given in sections **5** and **6** 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.

Annex G (informative) Worked example 7: flat and pitched roof buildings with decking and sheeting acting as diaphragm bracing to beams, wind forces on end gables and wind forces on eaves

G.1 General

This annex illustrates the action of roof decking and sheeting in providing lateral restraint to beams, and diaphragm bracing to end gables and eaves. This example shows the application of section **9**. Throughout the example, references to the appropriate parts of the text are given in square brackets at the right-hand side of the page.

G.2 Problem

G.2.1 Consider the action of roof decking in a flat roof building in providing lateral support to the main beams supporting the roof decking.

G.2.2 Consider the action of roof decking in a flat roof building in providing bracing to resist wind on the end gable.

G.2.3 Consider the action of roof sheeting in a pitched roof building in providing bracing to resist wind on the end gables.

G.2.4 Consider the action of roof sheeting in a pitched roof building in providing bracing to resist wind on the eaves.

G.3 Case a). Lateral support of beams

G.3.1 ***Arrangement***

Figure G.1 shows the plan of a typical roof area consisting of three 3.6 m beam spacings with the decking spanning perpendicular to the beams and fastened on two sides (to the beams only).

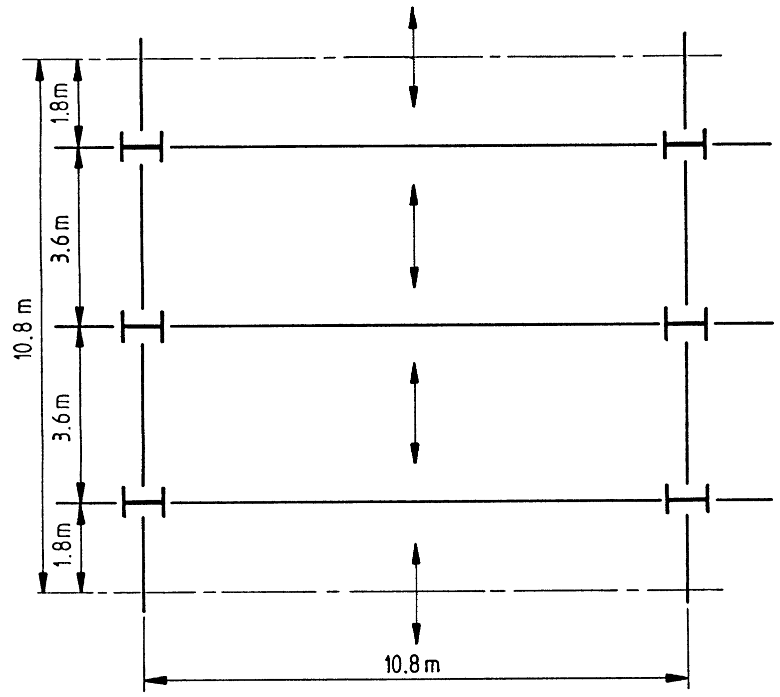


Figure G.1 — Worked example 7: plan of roof deck

Recommendations for lateral restraint of beams are given in BS 5950-1. The lateral restraints should be capable of resisting a total force of not less than 2.5 % of the maximum factored force in the compression flange.

If the lateral restraint is provided by a diaphragm, the depth of the diaphragm should be equal to the beam spacing × the number of beams. Hence, for the case illustrated of three parallel beams at a spacing of 3.6 m, the total depth of the diaphragm is 10.8 m, and the span of the beam is also 10.8 m. [**9.2.2**]

G.3.2 ***Data***

Beam:	457 mm × 152 mm Universal beam × 52 kg/m (the section modulus about the x-x axis Z_{xx} = 949.0 cm^3, flange width = 152.4 mm, flange thickness = 10.9 mm).
Decking:	0.85 mm net thickness; profile and grade of steel as in **A.3**.
Sheet/beam fasteners:	3.7 mm diameter cartridge fired pins in every trough (150 mm spacing).
Seam fasteners:	4.8 mm Monel blind rivets at 450 mm spacing.
Factored vertical load:	3.0 kN/m^2.

G.3.3 ***Lateral restraint force***

The factored maximum bending moment in a beam is

$$M_{\max} = \frac{3.0 \times 3.6 \times 10.8^2}{8} = 157.4 \text{ kN} \cdot \text{m}$$

The maximum stress in the compression flange is

$$\frac{157.4 \times 10^6}{949.0 \times 10^3} = 165.8 \text{ N/mm}^2$$

Hence the maximum force in the compression flange is

$$165.8 \times 152.4 \times 10.9 \times 10^{-3} = 275.4 \text{ kN}$$

So the lateral restraint force required for three beams is [**9.2.1** b]

$3 \times 2.5\ \% \times 275.4 = 20.6$ kN

G.3.4 *Shear capacity of diaphragm*

For the shear diaphragm shown in Figure G.2 [**9.2.2**, 2]

F_p = 4.25 kN (as in **B.4**)

F_s = 2.38 kN (as in **B.4**)

n_p = 3 [**B.4**, Table 6]

n_s = 21

β_1 = 1.13 (5 troughs/sheet; see Table 6)

β_2 = 1.25 (5 troughs/sheet; see Table 6)

β_3 = 1.0 (see **5.1.1.2**)

Seam capacity: $21 \times 2.38 + \frac{1.13}{1.0} \times 3 \times 4.25 > 10.3$

i.e. 64.3 kN > 10.3 kN. This is satisfactory.

For decking fastened to the beams only:

Sheet/beam fastener capacity: $1.25 \times 3 \times 4.45 > 10.3$

i.e. 15.9 kN > 10.3 kN. This is satisfactory.

Hence the decking is adequate to provide lateral restraint to the beams.

G.4 Case b). Bracing to the end gable: decking perpendicular to the gable

G.4.1 *Arrangement* [**9.3.1**]

Figure G.3 shows a plan view of decking spanning perpendicular to the end gable acting as a stressed skin diaphragm to resist wind on the gable. The depth of the diaphragm is taken as four frame spacings. The decking is fastened on four sides. [**9.3.2**]

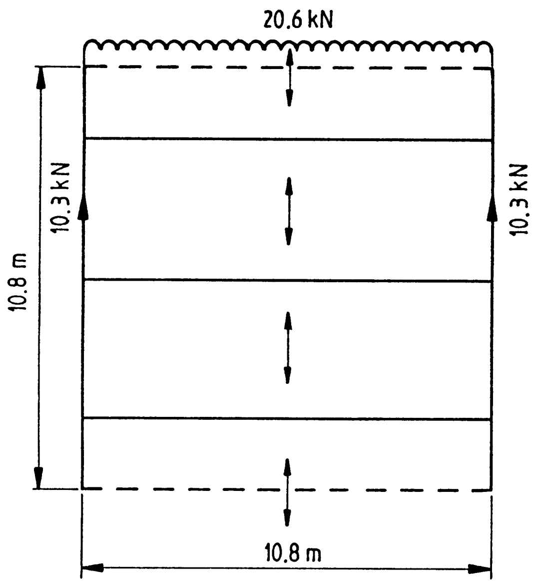


Figure G.2 — Extent of diaphragm

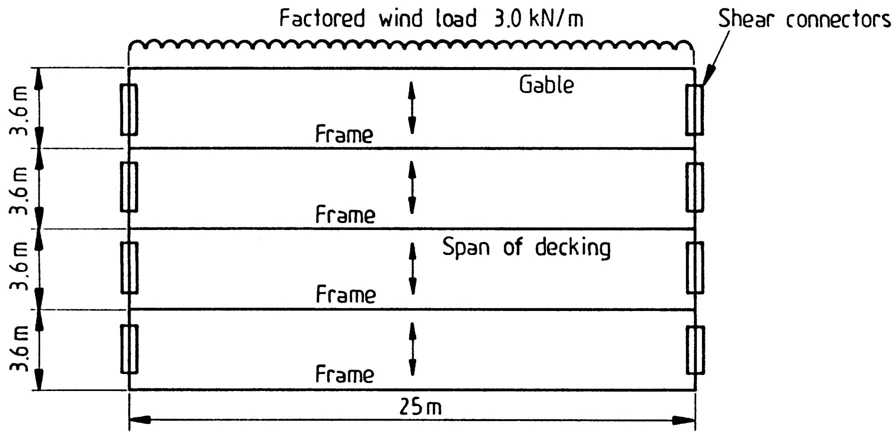


Figure G.3 — Plan of roof decking

G.4.2 ***Data***	[**A.3**]
Decking:	0.85 mm net thickness; profile and grade of steel as in **A.3**.
Fasteners:	As in **G.3.2**.
Factored wind load:	3.0 kN/m.
Shear connectors:	Four fasteners in each.

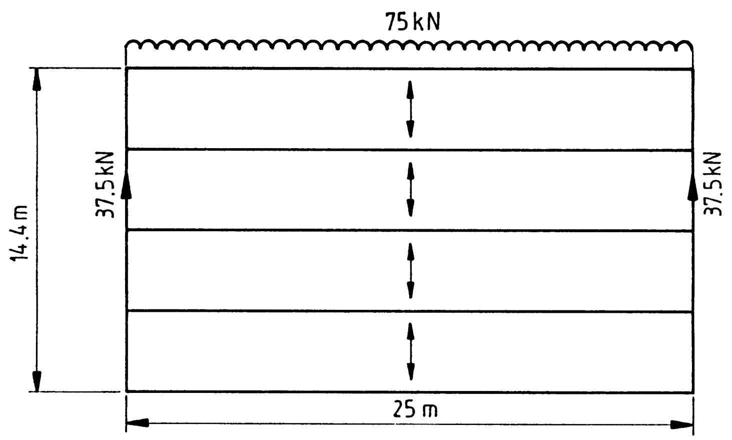


Figure G.4 — Loads on roof diaphragm

G.4.3 ***Shear capacity of diaphragms***

For the shear diaphragm shown in Figure G.4 [**9.2.21**]

F_p = 4.25 kN = F_{sc}

F_s = 2.38 kN

n_p = 5

n_s = 28

β_1 = 1.13 (5 troughs/sheet; see Table 6)

β_2 = 1.25 (5 troughs/sheet; see Table 6)

β_3 = 1.0 (see **5.1.1.2**)

Seam capacity: $28 \times 2.38 + \frac{1.13}{1.0} \times 5 \times 4.25 > 37.5$

i.e. 90.6 kN > 37.5 kN. This is satisfactory.

Shear connector capacity: 16 × 4.25 > 37.5.

i.e. 68.0 kN > 37.5 kN. This is satisfactory.

Hence the decking is adequate to provide wind bracing to the end gable.

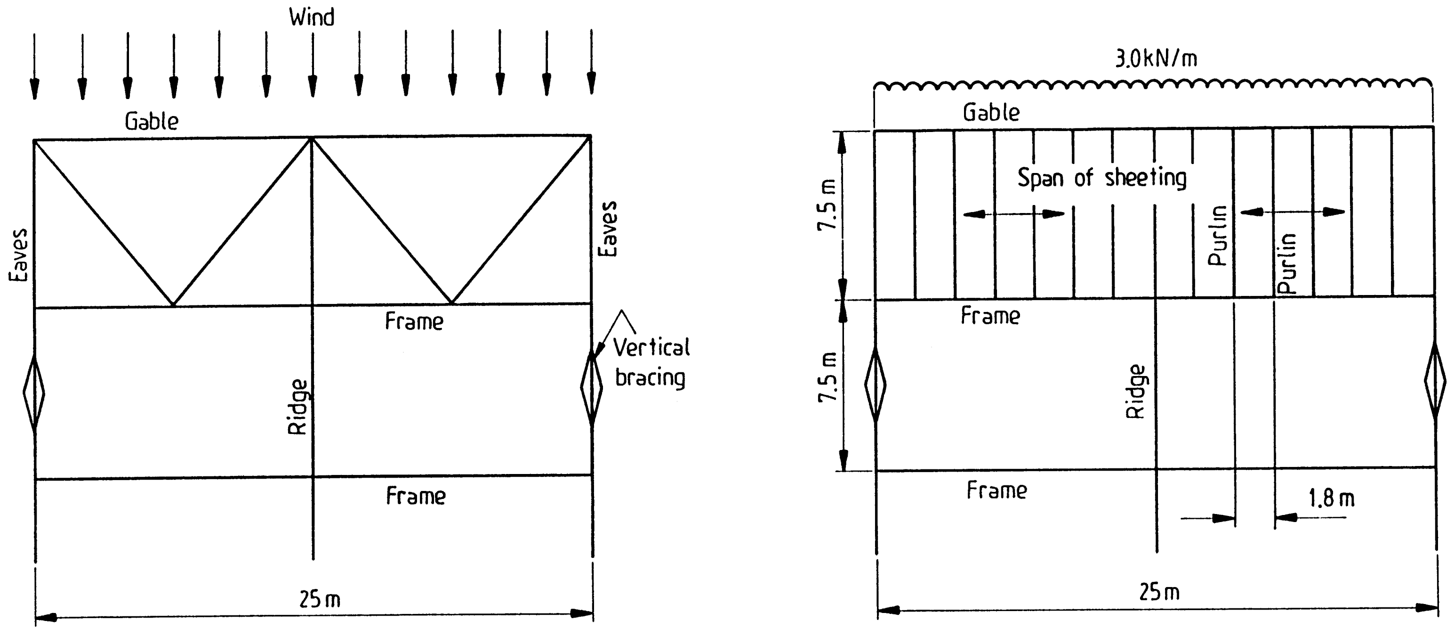
G.5 Case c). Bracing to the end gable: sheeting parallel to the gable

G.5.1 *Arrangement* [**9.3.1**]

Figure G.5 a) shows a plan view of typical triangulated bracing used to resist wind on the end gable. Figure G.5 b) shows how it may be replaced by sheeting spanning parallel to the end gable, acting as a stressed skin diaphragm. The depth of the diaphragm is taken as two frame spacings. The sheeting is fastened on two sides (to the purlins only). [**9.3.3**]

G.5.2 *Data*

Decking:	0.65 mm net thickness, 6 troughs wide, steel Fe E 280 G conforming to BS EN 10047.
Sheet/purlin fasteners:	6.3 mm self-drilling screws in every trough.
Seam fasteners:	4.8 mm self-drilling screws at 450 mm centres.
Factored wind load:	3.0 kN/m.



a) Plan of triangulated gable wind bracing

b) Plan of diaphragm gable wind bracing

Figure G.5 — Wind bracing to end gables

G.5.3 ***Shear capacity of diaphragm*** **[9.2.22]**

F_p = 0.65 × 6.0 = 3.90 kN

F_s = 0.65 × 2.5 = 1.62 kN

β_1 = 0.71 (6 troughs/sheet; see Table 6)

β_2 = 1.40 (6 troughs/sheet; see Table 6)

$\beta_3 = \frac{6-1}{6} = 0.83$ (see **5.1.1.2**)

For a diaphragm depth of two frame spacings, the maximum shear per metre is given by q = 37.5/15 = 2.5 kN/m.

Considering the equilibrium of one sheet (shown shaded in Figure G.6 and enlarged in Figure G.7): [Figure 23]

Shear force in x direction, V = 2.5 × 1.8 = 4.5 kN.

Seam capacity: $3 \times 1.62 + \frac{0.71}{0.83} \times 2 \times 3.90 > 4.5$

i.e. 11.5 kN > 4.5 kN. This is satisfactory.

Sheet/purlin fastener capacity: 1.40 × 2 × 3.90 > 4.5

i.e. 10.9 kN > 4.5 kN. This is satisfactory.

Hence the sheeting is adequate to provide wind bracing to the end gable.

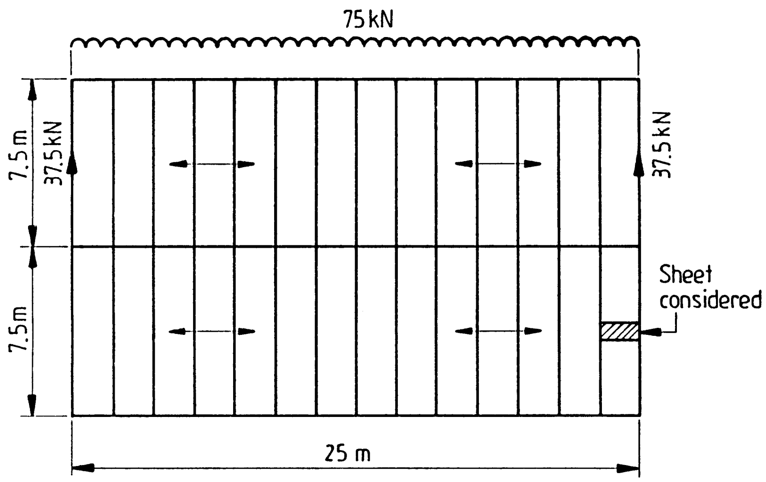


Figure G.6 — Roof diaphragm to gable

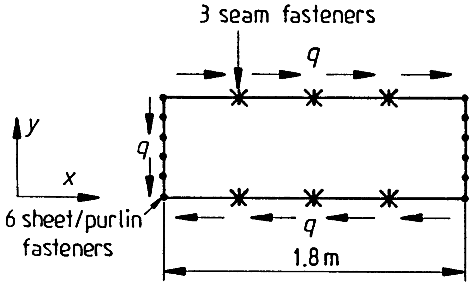


Figure G.7 — Enlarged diagram of sheet considered in Figure G.6

G.6 Case d). Bracing to the eaves

G.6.1 *Arrangement* [9.4.1]

Figure G.8 a) shows a plan view of triangulated eaves bracing. Figure G.8 b) shows how it may be replaced by roof sheeting acting as a stressed skin diaphragm (shown hatched). The depth of the diaphragm is taken as two purlin spacings rather than three as permitted. The sheeting is fastened on two sides (to the purlins only).

G.6.2 *Data*

Building dimensions and data are as given in **G.5.1**, **G.5.2** and **G.5.3**.

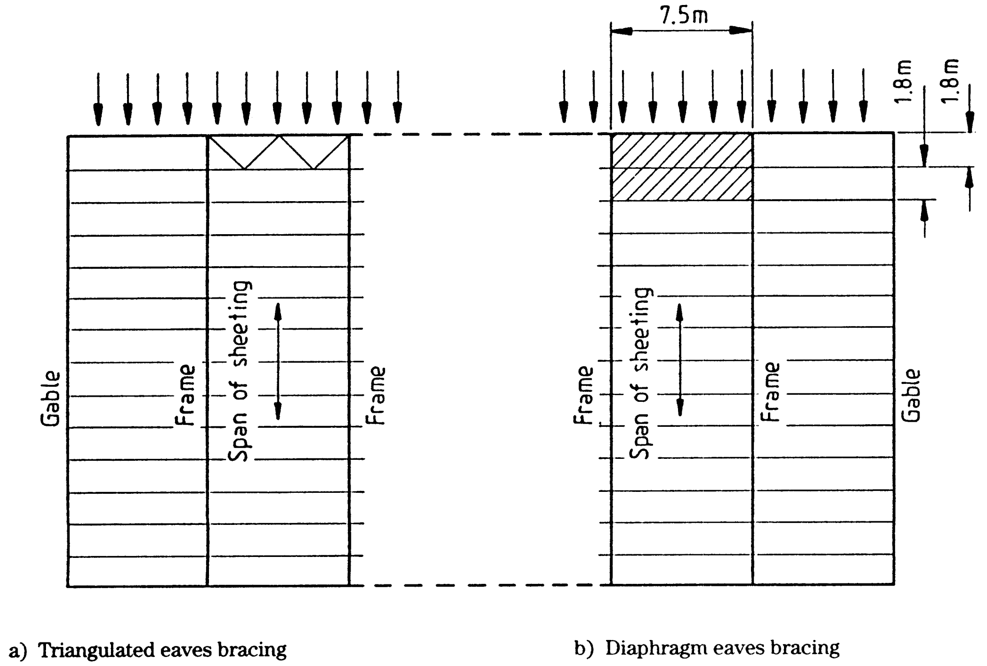


Figure G.8 — Plan of building

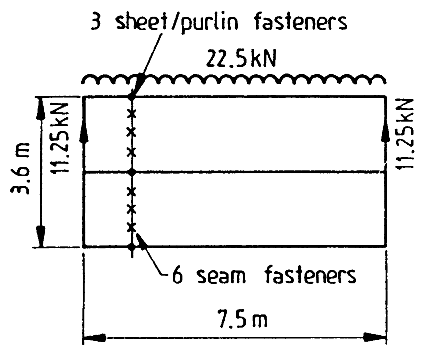


Figure G.9 — Enlarged diagram of sheets considered in Figure G.8

G.6.3 ***Shear capacity of diaphragm*** **[9.4.2]**

The forces acting on the shaded diaphragm shown in Figure G.8 and the detailed arrangement are as shown in Figure G.9.

For a diaphragm depth of two purlin spacings, the capacities are as follows. [**9.2.22**]

Seam capacity: $6 \times 1.62 + \frac{0.71}{0.83} \times 3 \times 3.90 > 11.25$

i.e. 19.7 kN > 11.25 kN. This is satisfactory.

Sheet/purlin fastener capacity: 1.40 × 3 × 3.90 > 11.25

i.e. 16.3 kN > 11.25 kN. This is satisfactory.

Hence the sheeting is adequate to provide wind bracing to the eaves.

G.7 Summary of calculations

The calculations illustrate the following.

— Roof or floor decking may be used to provide lateral restraint to the compression flanges of the supporting beams.

— Roof or floor decking, spanning perpendicular to the gable end, may be used to provide diaphragm wind bracing to the gable.

— Roof sheeting, spanning parallel to the gable end, may be used to provide diaphragm wind bracing to the gable.

— Roof sheeting may be used to provide diaphragm wind bracing to the eaves.

NOTE 1 If the diaphragm provides more than one function, the diaphragm, including fasteners, should be designed for the sum of the relevant loading cases. [**9.1**]

NOTE 2 This example considers only the most common failure modes of diaphragm bracing. For many practical applications this treatment is adequate, but for unusual or heavily loaded diaphragms other possible failure modes should also be considered (see **5.1.1.5** b) and **5.1.3**). [**9.2.22**] [**5.1.1.5** b) and **5.1.3**]

Annex H (informative) Worked example 8: folded plate roof

H.1 General

This example illustrates the case of a folded plate roof. It shows design calculations for strength and deflection. This example shows the application of section **10**. Throughout the example, references to the appropriate parts of the text are given in square brackets at the right-hand side of the page.

H.2 Problem

Calculate the strength and deflection of the light gauge steel folded plate roof shown in Figure H.1 under the given vertical loads.

H.3 Data

Loads:	Snow load 0.75 kN/m^2. Self-weight of sheeting, insulation, etc. 0.40 kN/m^2.
Sheeting profile:	As shown in Figure H.2.
Grade of steel:	Fe E 350 G Conforming to BS EN 10147.
Sheet/edge member fasteners:	6.3 mm self-drilling screws in every trough.
Seam fasteners:	4.8 mm self-drilling screws (21 per seam).
Factored load:	1.4 × 0.40 + 1.6 × 0.75 = 1.76 kN/m^2
Sheet/gable end member fasteners:	6.3 mm self-drilling screws (9 per roof slope).

H.4 Sheeting in bending (per metre length)

Maximum factored bending moment in sheeting (see Figure H.3) = $\dfrac{1.76 \times 2.4^2}{8} = 1.27\ \text{kN} \cdot \text{m/m}$

Hence, maximum factored bending stress in sheeting = $\dfrac{1.27 \times 10^6}{12.7 \times 10^3}\ \text{N/mm}^2 = 100\ \text{N/mm}^2$

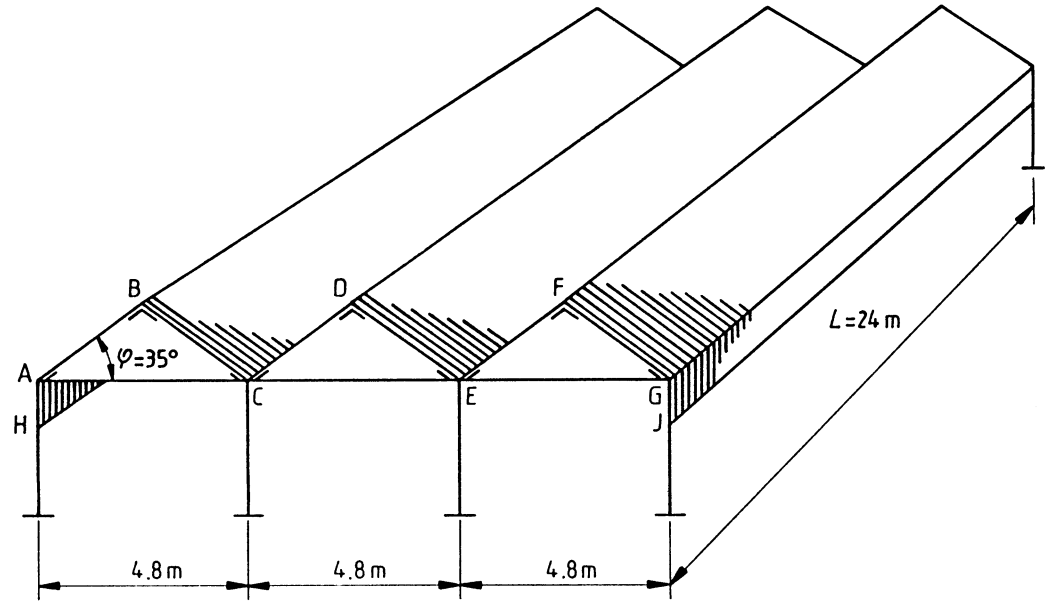


Figure H.1 — Worked example 8: folded plate roof

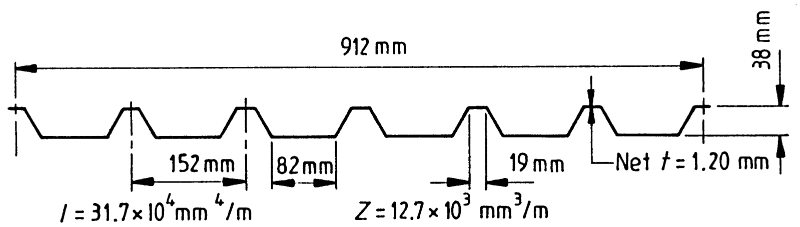


Figure H.2 — Sheeting profile

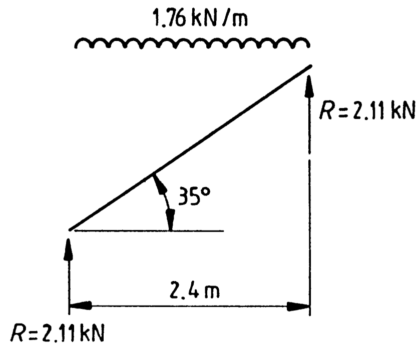


Figure H.3 — Bending of roof sheeting

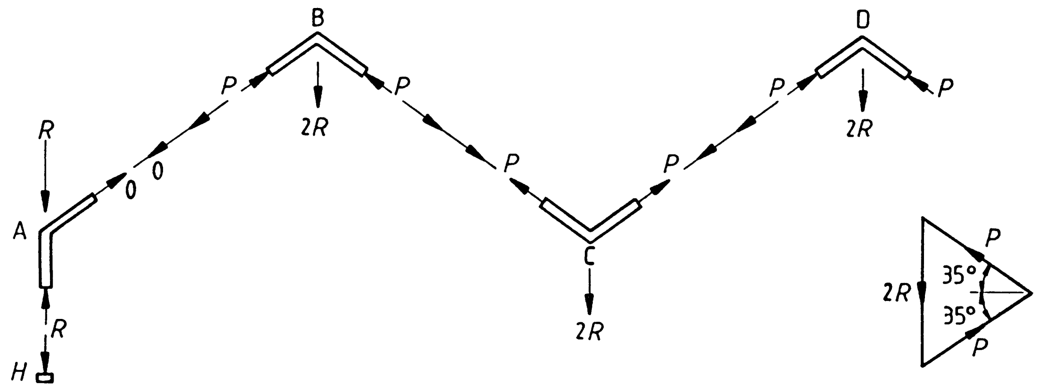


Figure H.4 — In-plane forces in roof slopes

H.5 Factored loading on fold line members (per metre length)

H.5.1 *Roof slope BC* [10.2]

For interior roof slopes (see Figure H.4), $2R = 2P \sin 35°$

Hence

$$P = \frac{R}{\sin 35°} = \frac{2.11}{0.574} = 3.68 \text{ kN/m}$$

Thus, the factored load per metre for the design of the inclined interior plate girder BC is $2P = 7.36$ kN/m.

Assume the fold line members are as shown in Figure H.5 so that the semi-area of each is $A = 1\ 100 \text{ mm}^2$ and the dimensions of the inclined plate girder BC are as shown in Figure H.6.

For the inclined plate girder BC,

Second moment of area $= I = 2 \times A(b/2)^2 = \dfrac{Ab^2}{2}$

and the section modulus $= \dfrac{I}{b/2} = Ab = 1100 \times 2810 = 3.09 \times 10^6 \text{ mm}^3$

Maximum factored bending moment $= \dfrac{7.36 \times 24^2}{8} = 530 \text{ kN} \cdot \text{m}$

so maximum factored axial stress in the edge members at B and C is given by

$$\frac{530 \times 10^6}{3.09 \times 10^6} = 172 \text{ N/mm}^2$$

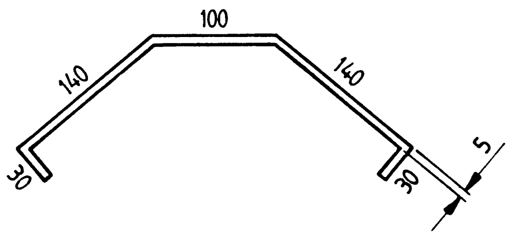
The edge members are fully restrained by the sheeting, and should be designed in accordance with BS 5950-5.

H.5.2 *Roof slope AB*

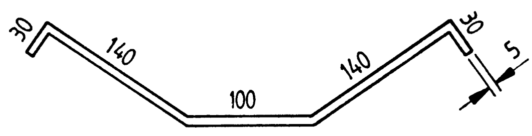
From Figure H.4, the factored load per metre for the design of the inclined end plate girder AB is $P = 3.68$ kN/m.

Assume that the fold line member at A has the section shown in Figure H.7 (area = 1 100 mm^2).

The design process is similar to that in **H.5.1**, except that the in-plane load is halved. Hence, the maximum factored axial stress in the edge members is 86 N/mm^2. As before, the edge members are fully restrained by the sheeting and should be designed in accordance with BS 5950-5.



a) Apex member



b) Valley member

Figure H.5 — Fold line members

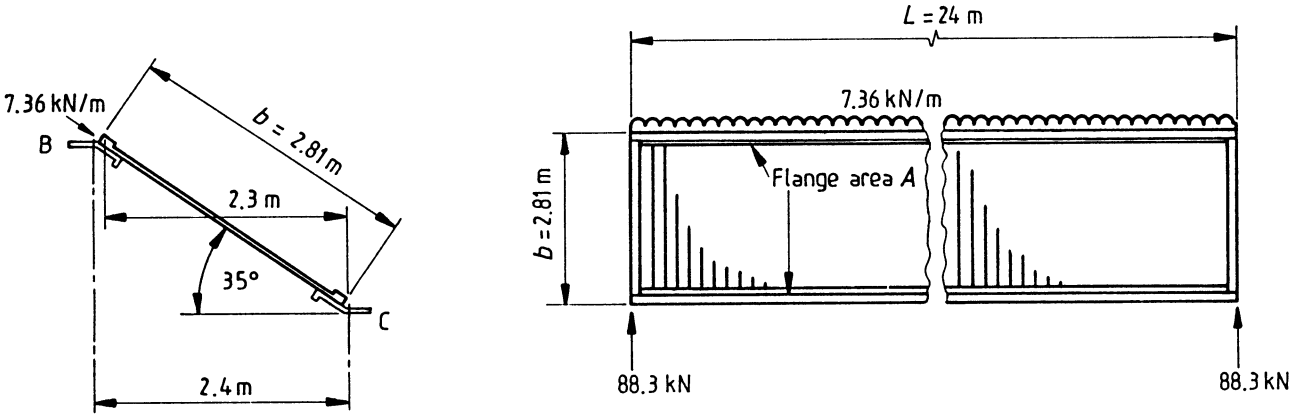
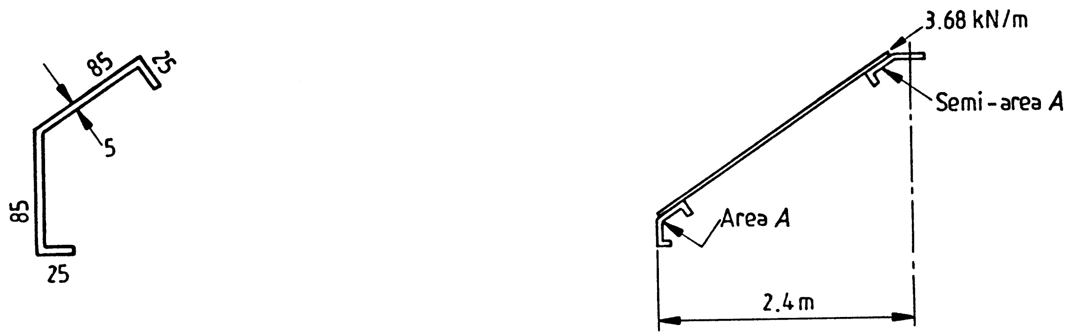
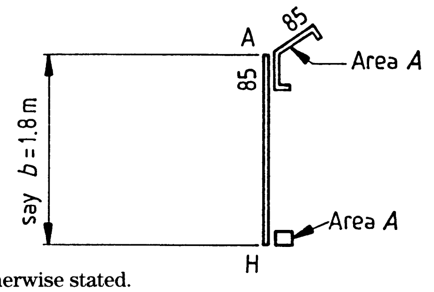


Figure H.6 — In-plane forces on plate girder BC



Dimensions are in millimetres, unless otherwise stated.

Figure H.7 — In-plane forces on plate girder AB



Dimensions are in millimetres, unless otherwise stated.

Figure H.8 — Vertical plate girder AH

H.5.3 *Vertical sheeting AH*

From Figure H.4, the factored load per metre for the design of the vertical plate girder AH is R = 2.11 kN/m. Assume that the member at H has a cross-sectional area A of 1 100 mm^2.

Then, for the vertical plate girder AH (see Figure H.8)

$$I = 2 \times A(b/2)^2 = \frac{Ab^2}{2}$$

and

$$Z = \frac{I}{b/2} = Ab = 1100 \times 1800 = 1.98 \times 10^6 \text{ mm}^3$$

Maximum factored bending moment = $\dfrac{2.11 \times 24^2}{8} = 152 \text{ kN} \cdot \text{m}$

so maximum factored axial stress in the edge members is given by

$$\frac{152 \times 10^6}{1.98 \times 10^6} = 77 \text{ N/mm}^2$$

The combined axial stress in the edge member at A is therefore 86 + 77 = 163 N/mm^2.

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.

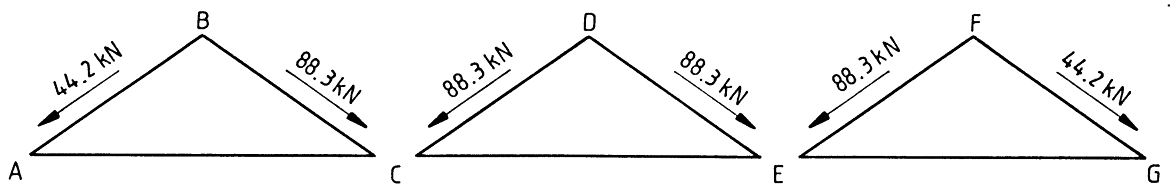


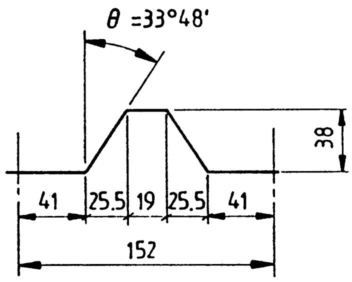
Figure H.9 — Axial forces on end frame members

H.6 End frame members

The end frames should be designed for the forces shown in Figure H.9 from the inclined plate girders.

H.7 Values of variables [**5.1** and **5.2**]

A = 1 100 mm^2
b = 2.81×10^3mm
d = 152 mm (see Figure H.10)
E = 205 kN/mm^2
F_p = $1.20 \times 6.0 \times \sqrt{(350/280)}$
= 8.04 kN (see Table 5)
F_s = $1.20 \times 2.5 \times \sqrt{(350/280)}$
= 3.35 kN (see Table 5)
F_{sc} = 8.04 kN (= F_p)
h = 38 mm (see Figure H.10)
I = 4.82×10^4 mm^4 per corrugation
L = 24×10^3 mm
n_s = 21
n_{sh} = $24 \times 10^3/912 = 27$
n_{sc} = 9
q = 7.36×10^{-3} kN/mm
p = 152 mm
s_p = 0.15 mm/kN (see Table 5)
s_s = 0.25 mm/kN (see Table 5)
s_{sc} = 0.15 mm/kN (see Table 5)
t = 1.20 mm
u = 192 mm
Y_* = 350 N/mm^2
ν = 0.3
β_1 = 0.71 (see Table 6)
β_3 = 5/6 = 0.83 (see **5.1.1.2**)
l/d = 0.125, h/d = 0.25
K_1 = 0.109 (see Table 10)



Linear dimensions are in millimetres.

Figure H.10 — Detail of typical profile in Figure H.2

H.8 Design strength [10.3]

Seam capacity $V_{ult} = \left(n_s F_s + \dfrac{2\beta_1}{\beta_3} F_p\right)\left(\dfrac{n_{sh}}{n_{sh} - 2}\right) = \left(21 \times 3.35 + \dfrac{2 \times 0.71}{0.83} \times 8.04\right) \dfrac{27}{25} = 90.8\ \text{kN}$ [10.3.1]

Capacity of fasteners to end gable $V_{ult} = n_{sc}F_{sc} + 2F_p = 9 \times 8.04 + 2 \times 8.04 = 88.4$ kN [10.3.2]

Hence the design shear capacity V^* is 88.4 kN which is almost exactly equal to the required value $qL/2$, i.e. 88.3 kN [10.3.3]

Sheet/fold line member fasteners: check whether $0.6bF_p/p > V^*$ [10.3.4]

$$0.6bF_p/p = \frac{0.6 \times 2.81 \times 10^3 \times 8.04}{152} = 89.1\ \text{kN}$$

Since 89.1 kN > 88.4 kN, this is satisfactory.

End collapse of sheeting profile: check whether $0.0009t^{1.5}bY_s/d^{0.5} \geqslant V^*$ [10.3.5]

$$0.0009t^{1.5}bY_s/d^{0.5} = \frac{0.0009 \times 1.20^{1.5} \times 2.81 \times 10^3 \times 350}{152^{0.5}} = 94.3\ \text{kN}$$

Since 94.3 kN > 88.4 kN, this is satisfactory.

Shear buckling: check whether $(28.8/b)\ D_x^{1/4}\ D_y^{3/4} > V^*$ [10.3.6]

$$D_x = \frac{Et^3 d}{12(1 - \upsilon^2)u} = \frac{205 \times 1.20^3 \times 152}{12 \times (1 - 0.3^2) \times 192} = 25.6\ \text{kN·mm}$$

$$D_y = \frac{EI}{d} = \frac{205 \times 4.82 \times 10^4}{152} = 65\,006\ \text{kN·mm}$$

$$(28.8/b)D_x^{1/4}D_y^{3/4} = \frac{28.8}{2.81 \times 10^3} \times 25.6^{1/4} \times 65\,006^{3/4} = 93.8\ \text{kN}$$

Since 93.8 kN > 88.4 kN, this is satisfactory.

The above values are all nearly equal to, or just above, the design shear capacity. The design is therefore satisfactory.

H.9 Deflection at factored load

[**10.4** and Table 18]

$$\Delta_{1.1} = \frac{152^{2.5} \times 0.109 \times 7.36 \times 10^{-3} \times (24 \times 10^3)^2}{8 \times 205 \times 1.20^{2.5} \times (2.81 \times 10^3)^2} = 6.44 \text{ mm}$$

$$\Delta_{1.2} = \frac{(1 + 0.3)(1 + 2 \times 38/152) \times 7.36 \times 10^{-3} \times (24 \times 10^3)^2}{4 \times 205 \times 1.20 \times 2.81 \times 10^3} = 2.99 \text{ mm}$$

$$\Delta_{2.1} = \frac{0.15 \times 152 \times 7.36 \times 10^{-3} \times (24 \times 10^3)^2}{4 \times (2.81 \times 10^3)^2} = 3.06 \text{ mm}$$

$$\Delta_{2.2} = \frac{0.25 \times 0.15 \times (27 - 2) \times 7.36 \times 10^{-3} \times 24 \times 10^3}{8(21 \times 0.15 + 0.71 \times 0.25)} = 6.22 \text{ mm}$$

$$\Delta_{2.3} = \frac{0.15 \times 0.15 \times 7.36 \times 10^{-3} \times 24 \times 10^3}{2 \times (2 \times 0.71 \times 0.15 + 9 \times 0.15)} = 1.27 \text{ mm}$$

$$\Delta_3 = \frac{7.36 \times 10^{-3} \times (24 \times 10^3)^4}{38.4 \times 205 \times 1100 \times (2.81 \times 10^3)^2} = 35.71 \text{ mm}$$

$$\Delta = \overline{55.7 \text{ mm}}$$

Hence, total deflection Δ in the plane of the roof slope is 55.7 mm. The vertical central deflection of the folded plate roof, Δ_v, is

$\Delta_v = \Delta \operatorname{cosec} \varphi = 55.7 \times \operatorname{cosec} 35° = 97 \text{ mm}$

NOTE Under unfactored load, i.e. 0.4 + 0.75 = 1.15 kN/m^2,

$$\Delta_v = 97 \times \frac{1.15}{1.76} = 64 \text{ mm} = \frac{\text{span}}{377}$$

H.10 Summary of calculations

The calculations illustrate the following.

— Profiled steel sheeting and cold formed steel apex and valley members may be used to form folded plate roofs of medium span.

— The calculation procedure is simple.

— For the particular profile and fastener spacing chosen, the capacities of all the modes are very close.

— A typical steel folded plate roof has a small deflection in relation to its span.

Annex J (informative) Mathematical expressions for Table 6, Table 8, Table 16 and Table 17

J.1 Derivation of factors β_1 and β_2 (see Table 6)

If n_f is an odd number, factors β_1 and β_2 are obtained as follows.

a) *Factor β_1*

— Case 1: sheeting $\beta_1 = \sum_i \left(\frac{2i}{n_f}\right)^3$

— Case 2: decking $\beta_1 = \sum_i \left(\frac{2i}{n_f - 1}\right)^3$

b) *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 $(n_f - 1)/2$.

If n_f is an even number, factors β_1 and β_2 are obtained as follows.

1) *Factor β_1*

— Case 1: sheeting $\beta_1 = \sum_i \left(\frac{2i - 1}{n_f}\right)^3$

— Case 2: decking $\beta_1 = \sum_i \left(\frac{2i - 1}{n_f - 1}\right)^3$

2) *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 $n_f/2$.

J.2 Derivation of factors α_1, α_2 and α_3 (see Table 8)

Factors α_1, α_2 and α_3 are obtained as follows.

a) *Factor α_1*. This is empirical.

b) *Factor α_2*. $\alpha_2 = \frac{1}{1 + \sum_i \{1 - 2i/(n_p - 1)\}}$

c) *Factor α_3*. $\alpha_3 = \frac{1}{1 + \sum_i \{1 - 2i/(n_p - 1)\}^2}$

where

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

i is a quantity which increases from 1 to $(n_{p-1)/2}$.

J.3 Derivation of reduction factor η and factors by which η should be divided (see Table 16 and Table 17)

The general case results in η simultaneous equations for the reduction factors R_i:

$$R_i - R_{i-1} + \frac{r}{n+1}\left(\sum_{j=1}^{n} -jR_j + \sum_{j=i}^{n} (n+1)R_j\right) = \frac{r}{n+1}\left(\sum_{j=1}^{n} -jP_j + \sum_{j=i}^{n} (n+1)P_j\right)$$

There are two cases to consider.

— Case 1: all frames are equally loaded ($P_j = P$ for all j).

— Case 2: one frame is loaded, $P_k = P$, $P_j = 0, j \neq k$.

To derive the values given in Table 16 and Table 17, it is assumed that the loaded frame is at the centre of the building. The equations are solved by expressing them in matrix form and considering the two cases together.

$$\begin{array}{cccccccc}
(1) & (2) & & (i-1) & (i) & & (n-1) & (n) \\
\end{array}$$

$$\begin{bmatrix}
\frac{nr}{n+1}+1 & \frac{(n-1)r}{n+1} & \cdot\cdot & \frac{(n-i+2)r}{n+1} & \frac{(n-i+1)r}{n+1} & \cdot\cdot & \frac{2r}{n+1} & \frac{r}{n+1} \\
\frac{-r}{n+1} & \frac{(n-1)r}{n+1}+1 & \cdot\cdot & \frac{(n-i+2)r}{n+1} & \frac{(n-i+1)r}{n+1} & \cdot\cdot & \frac{2r}{n+1} & \frac{r}{n+1} \\
\cdot & \cdot & & \cdot & \cdot & & \cdot & \cdot \\
\cdot & \cdot & & \cdot & \cdot & & \cdot & \cdot \\
\frac{-r}{n+1} & \frac{-2r}{n+1} & \cdot\cdot & \frac{-(i-1)r}{n+1}-1 & \frac{(n-i+1)r}{n+1}+1 & \cdot\cdot & \frac{2r}{n+1} & \frac{r}{n+1} \\
\cdot & \cdot & & \cdot & \cdot & & \cdot & \cdot \\
\cdot & \cdot & & \cdot & \cdot & & \cdot & \cdot \\
\frac{-r}{n+1} & \frac{-2r}{n+1} & \cdot\cdot & \frac{-(i-1)r}{n+1} & \frac{-ir}{n+1} & \cdot\cdot & \frac{2r}{n+1}+1 & \frac{r}{n+1} \\
\frac{-r}{n+1} & \frac{-2r}{n+1} & \cdot\cdot & \frac{-(i-1)r}{n+1} & \frac{-ir}{n+1} & \cdot\cdot & \frac{-(n-1)r}{n+1}-1 & \frac{r}{n+1}+1
\end{bmatrix}
\begin{bmatrix}
R_1 \\ R_2 \\ \cdot \\ \cdot \\ R_i \\ \cdot \\ \cdot \\ R_{n-1} \\ R_n
\end{bmatrix}
=
\begin{bmatrix}
r\frac{n}{2} & r\frac{n-k+1}{n+1} \\
r\left(\frac{n}{2}-1\right) & r\frac{n-k+1}{n+1} \\
\cdot & \cdot \\
\cdot & \cdot \\
r\left(\frac{n}{2}+1-i\right) & * \\
\cdot & \cdot \\
\cdot & \cdot \\
\cdot & \cdot \\
r\left(1-\frac{n}{2}\right) & r\frac{-k}{n+1}
\end{bmatrix} P$$

where the term * is $r\,\dfrac{n-k+1}{n+1}$ if $i \leqslant k$ or $r\,\dfrac{-k}{n+1}$ if $i > k$

The values in Table 16 follow directly from the solution for case 1.

The values in Table 17 are obtained from 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 expressions in this subclause only, the symbols are 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_j is the force on frame j (equal to unity for the derivation of the values given in Table 16 and Table 17).

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List of references (see 1.2)

Normative references

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I2] LAWSON, R.M., and D.A. NETHERCOT. Lateral stability of I-beams restrained by profiled sheeting. *The Structural Engineer,* March 1985, 63B(1).

[6] In preparation.

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