

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

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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 6¹⁾: 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 5950-6¹⁾ 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

A British Standard does not purport to include all the necessary provisions of a contract. Users of British Standards are responsible for their correct application.

Compliance with a British Standard does not of itself confer immunity from legal obligations.

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.

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

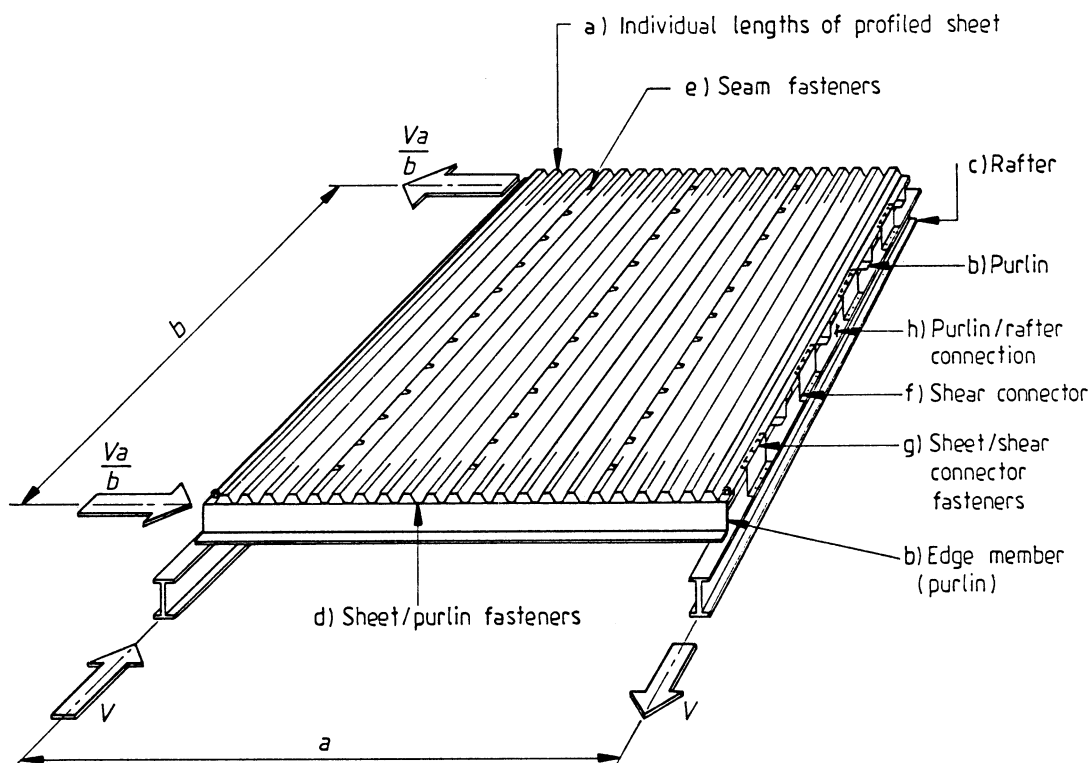
the distance between vertically stiffened frames, over which the diaphragm acts (see Figure 3)

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)	L	Length of a panel assembly between braced frames (in mm) (see Figure 7)
A	Cross-sectional area of a longitudinal edge member (in mm ²)	<i>or</i>	Length of a folded plate roof between gables (in mm)
<i>or</i>	Semi-area of a fold line member (in mm ²)	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)
<i>or</i>	Area of openings in a panel (in mm ²)	n	Number of panels in the length of the panel assembly (see Figure 7)
b	Depth of a shear panel in a direction parallel to the corrugations (in mm) (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)
<i>or</i>	Depth of web in a folded plate roof (in mm) (see Figure 29)	n_f	Number of sheet/purlin fasteners per member per sheet width
c	Total shear flexibility of a shear panel (in mm/kN) (see Figure 6)	n_l	Number of sheet lengths in the length of the panel assembly (see Table 14)
$c_{1.1}$ etc.	Component shear flexibilities (in mm/kN) (see Figure 5)	n_p	Number of purlins (edge + intermediate)
d	Pitch of the corrugations (in mm) (see Table 10 and Table 11)	n_s	Number of seam fasteners per side lap (excluding those which pass through both sheets and the supporting purlin)
D_x, D_y	Orthogonal bending stiffnesses of profiled sheet per unit length (in kN-mm ² /mm)	n_{sc}	Number of sheet/shear connector fasteners per end rafter
E	Modulus of elasticity of steel (205 kN/mm ²)	<i>or</i>	Number of gable fasteners per gable in a folded plate roof
F_p	Design resistance of an individual sheet/purlin fastener (in kN) [see Table 5 a)]	n'_{sc}	Number of sheet/shear connector fasteners per internal rafter
F_{pr}	Design resistance of a purlin/rafter connection (in kN) (see Table 7)	n_{sh}	Number of sheet widths per panel
F_s	Design resistance of an individual seam fastener (in kN) [see Table 5 b)]	p	Pitch of the sheet/purlin fasteners (in mm)
F_{sc}	Design resistance of an individual sheet/shear connector fastener (in kN) [see Table 5 a)]	P	Panel point load on diaphragm (in kN)
h	Height of the sheeting profile (in mm) (see Table 10 and Table 11)	P_{ult}	Ultimate load at a panel point (in kN)
I	Second moment of area of a single corrugation about its neutral axis (in mm ⁴) (see Figure 8)	p_y	Design strength of steel sheeting (in N/mm ²)
k	Frame flexibility (in mm/kN) [see Figure 12 b)]	q	Distributed in-plane line load on the diaphragm (in kN/mm)
k_{sp}	Frame flexibility due to spread (in mm/kN) [see Figure 14 b)]	r	Relative flexibility of shear panel to frame
k_{sw}	Frame flexibility due to sway (in mm/kN) [see Figure 14 a)]	s_p	Slip per sheet/purlin fastener per unit load (in mm/kN) [see Table 5 a)]
K_1, K_2	Sheeting constants for every corrugation fastened and alternate corrugations fastened (see Table 10 and Table 11)	s_{pr}	Deflection of the top of the purlin at the purlin/rafter connection per unit load (in mm/kN) (see Table 7)
l	Width of the corrugation crest (in mm) (see Table 10 and Table 11)	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)
	<i>or</i> 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)
	<i>or</i> 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 5950-6²⁾. 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 $+35^{\circ}\text{C}$. 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 may 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) $1.4 \times$ (unfactored dead load); and
- 2) $1.6 \times$ (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 5950-6³⁾.

Table 2 — Deflection limits

<i>a) Deflection of beams due to unfactored imposed loads</i>	
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
<i>b) Deflection of columns other than portal frames due to unfactored imposed and wind loads</i>	
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:

- the environment;
- the degree of exposure;
- the shape of the members and sheeting, and the structural detailing;
- the corrosion protection measures adopted;
- the anticipated life to first maintenance;
- 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 flat elements.

Where other section properties are required, reference should be made to BS 5950-6⁴⁾.

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/mm² 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

⁴⁾ In preparation.

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 mm.

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 $E = 205 \text{ kN/mm}^2$;
- shear modulus $G = 79 \text{ kN/mm}^2$;
- Poisson's ratio $\nu = 0.30$;
- coefficient of linear thermal expansion $\alpha = 12 \times 10^{-6} \text{ K}^{-1}$.

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 flat 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 in Figure 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 frames.

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

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

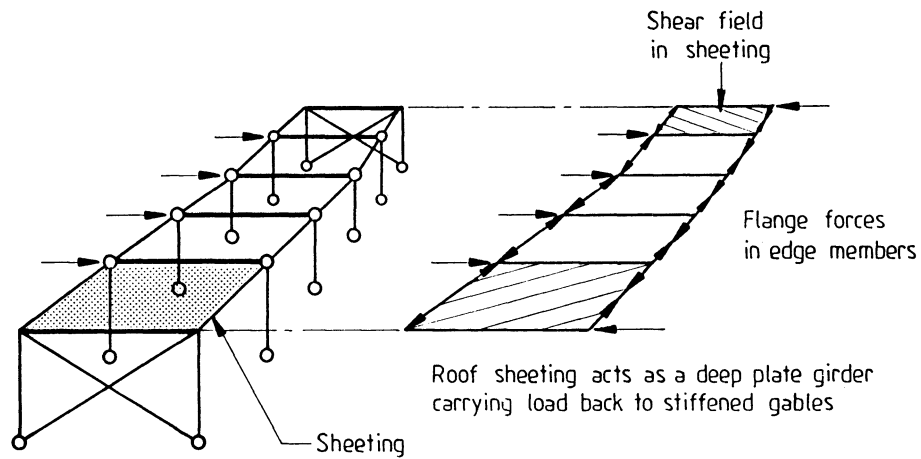
Subject to the determination of dynamic loads given in 2.2.3, no other allowance for repeated loading need normally be made in determining the design strength and shear flexibility of a diaphragm. For unusually severe cases of repeated loading, the shear flexibility of a diaphragm should be increased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

4.2 Conditions and restrictions

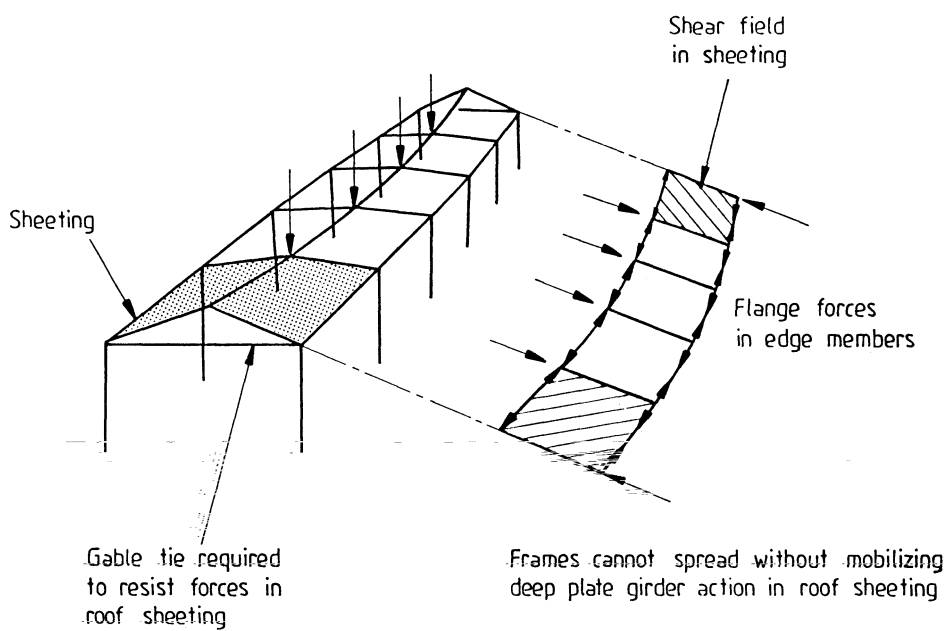
4.2.1 Necessary conditions

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 in 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 5950-6⁵⁾. 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.

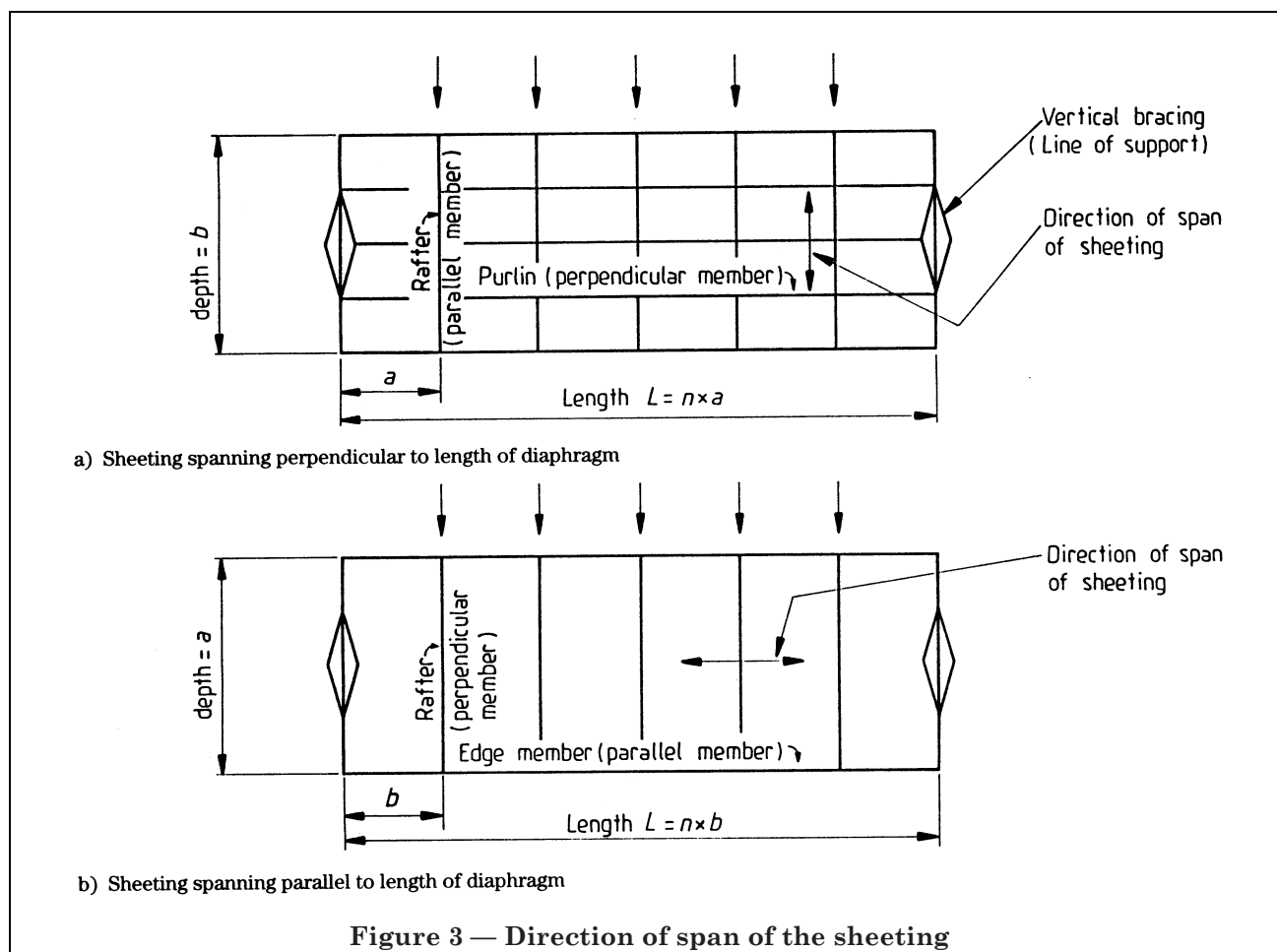
4.4 Design criteria

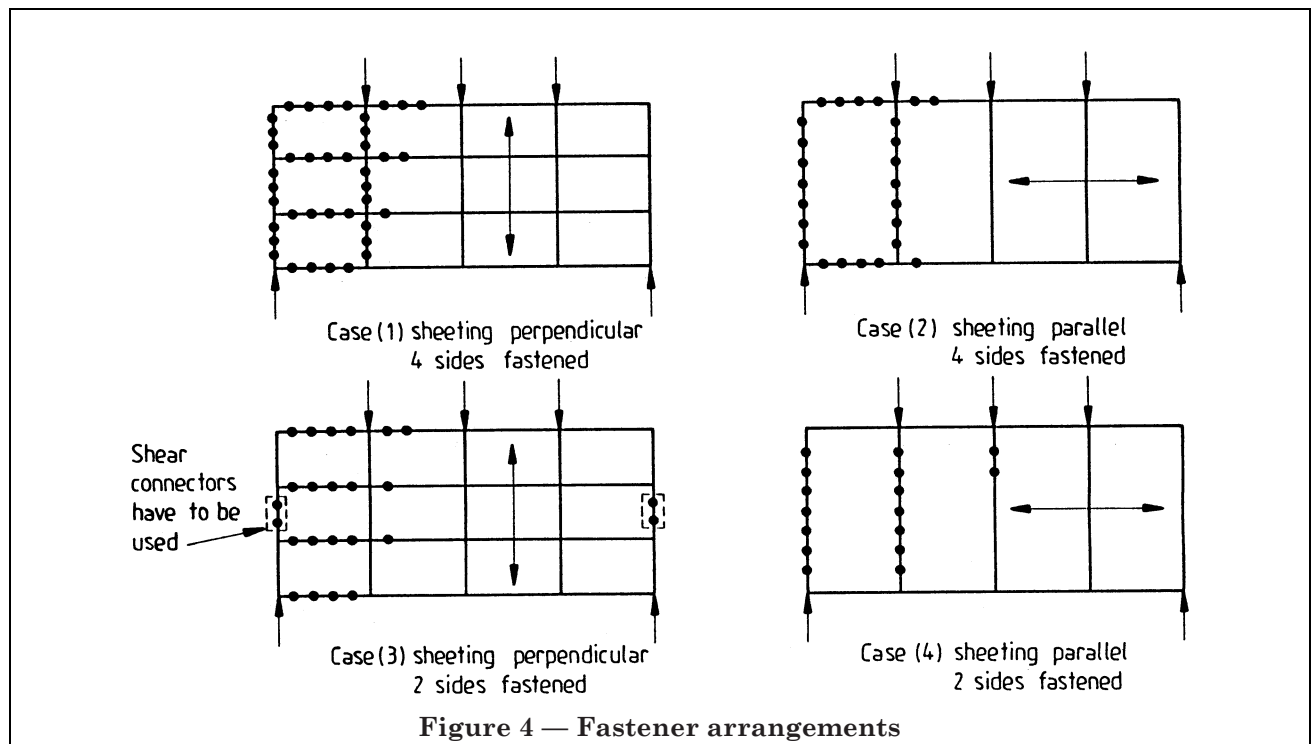
4.4.1 Diaphragm strength

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.





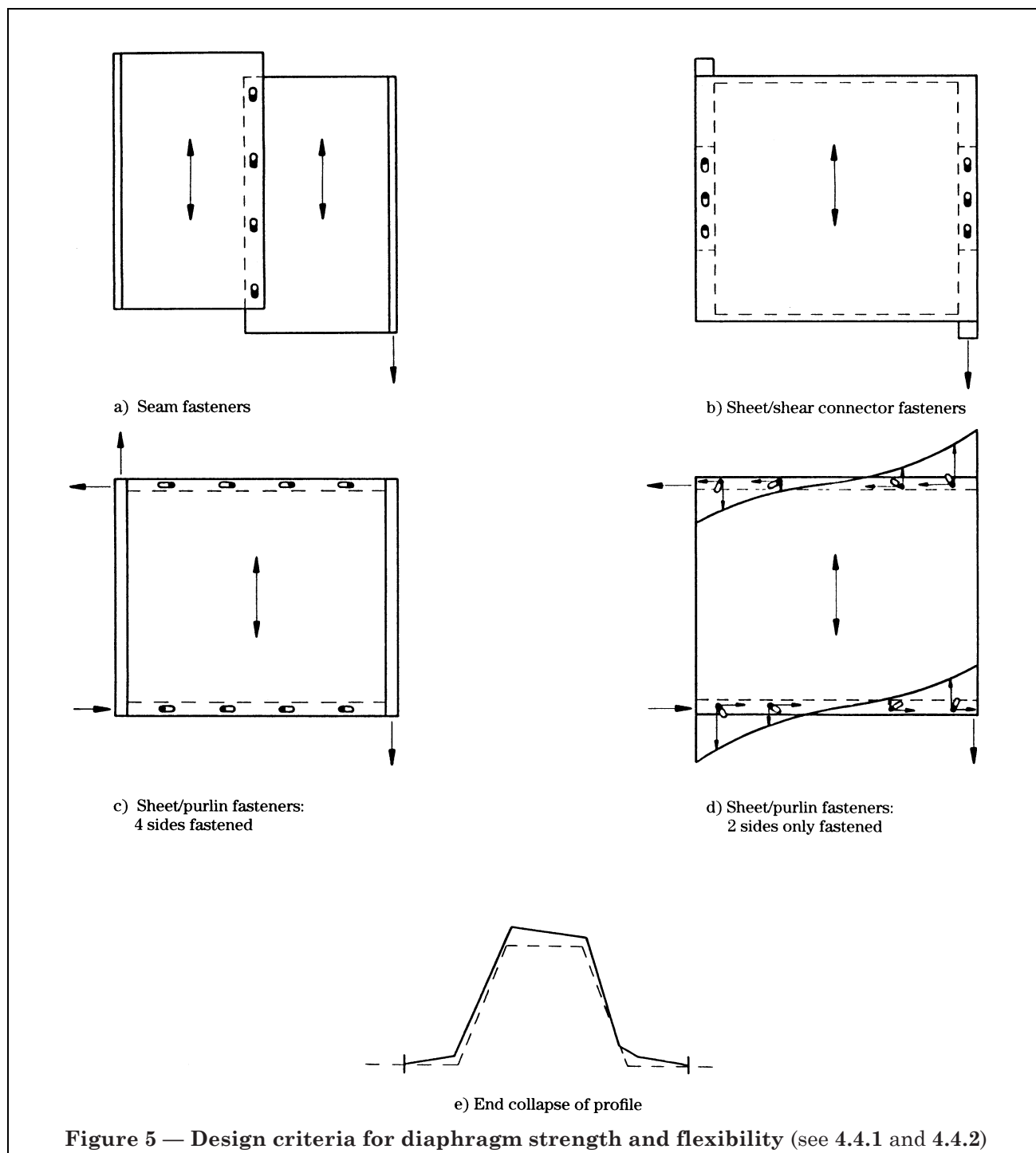
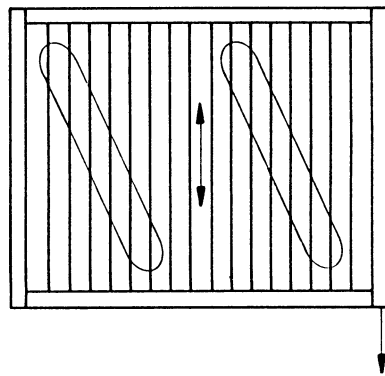
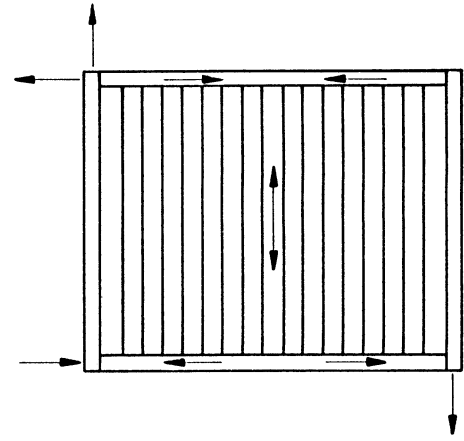


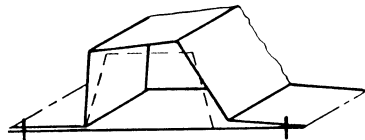
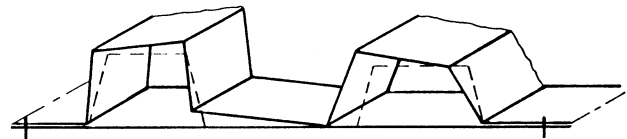
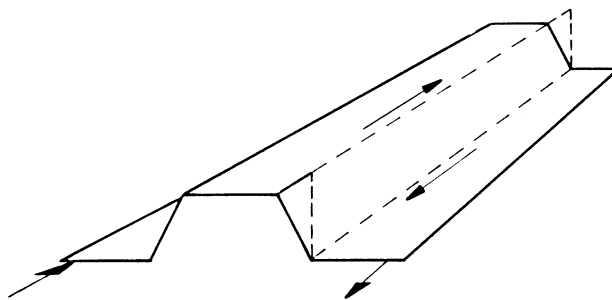
Figure 5 — Design criteria for diaphragm strength and flexibility (see 4.4.1 and 4.4.2)



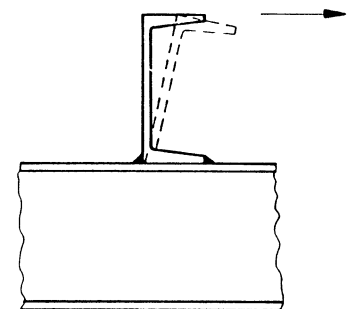
f) Shear buckling



g) Edge member forces

h) Profile distortion:
every trough fastenedi) Profile distortion:
alternate troughs fastened

j) Shear strain in sheeting

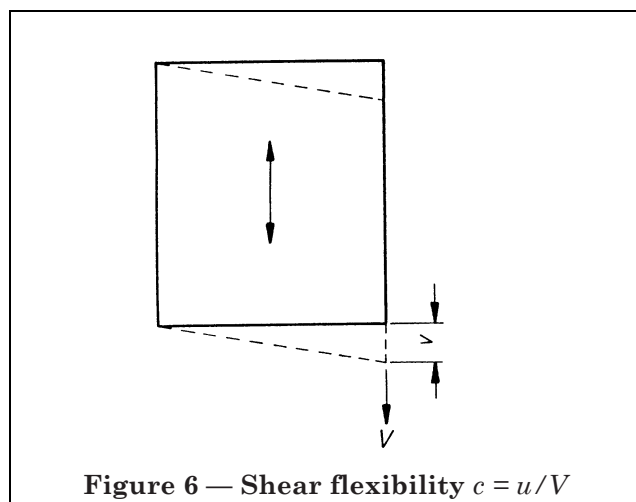


k) Purlin/rafter connection

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.



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;
- $\beta_3 = (n_f - 1)/n_f$ for sheeting (seam fasteners in the crests); or
- $\beta_3 = 1.0$ 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 $\frac{1}{2}(n - 1)$ times the force in the internal rafters, so the corresponding numbers of shear connector fasteners should normally be in the same ratio, i.e. $n_{sc} = \frac{1}{2}(n - 1)n'_{sc}$.

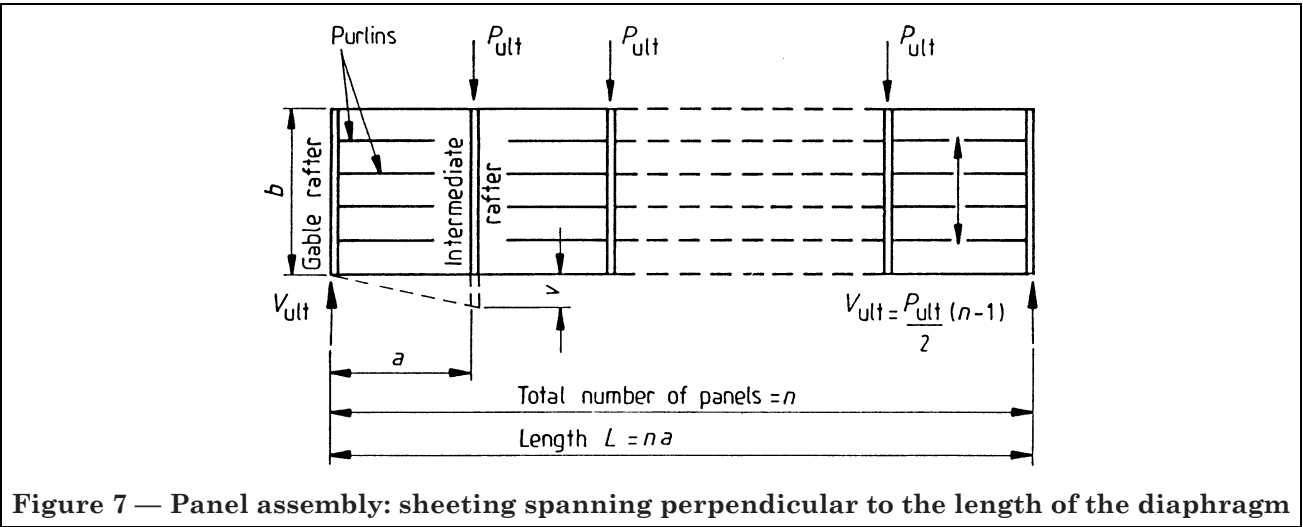


Figure 7 — Panel assembly: sheeting spanning perpendicular to the length of the diaphragm

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.

5.1.1.2 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);

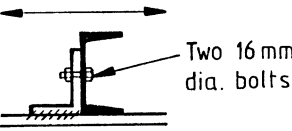
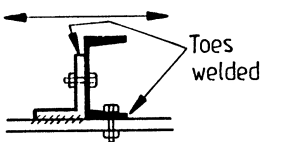
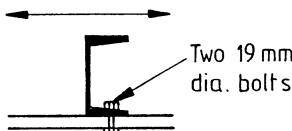
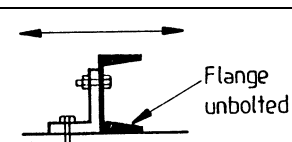
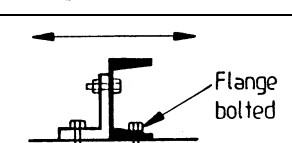
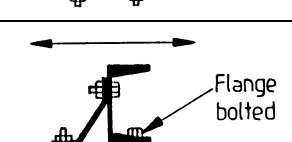
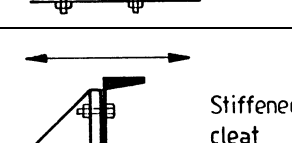
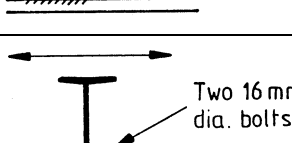
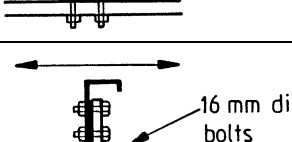
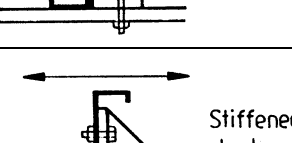
F_p is the design resistance of an individual sheet/purlin fastener (in kN);

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 α_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	 Two 16 mm dia. bolts	4.9	0.84
2	(89 × 64 × 7.8 angle cleat × 89 mm long)	 Toes welded	20.0	0.11
3	152 × 76 × 17.88 kg/m channel	 Two 19 mm dia. bolts	14.4	0.60
4	(76 × 64 × 6.2 angle cleat × 127 mm long)	 Flange unbolted	7.2	1.20
5		 Flange bolted	19.6	0.35
6		 Flange bolted	25.0	0.13
7		 Stiffened cleat	25.0	0.05
8	254 × 102 × 22 kg/m Universal beam	 Two 16 mm dia. bolts	10.0	2.60
9	203 × 51 × 2.0 zed	 16 mm dia. bolts	4.4	1.40
10	(178 × 89 × 9.4 angle cleat × 127 mm long)	 Stiffened cleat	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

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

b) alternate corrugations fastened

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

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/mm²);
- 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) n_p	Factors		
	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 purlins 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)	(2)
	Shear flexibility in panel assemblies (see Figure 7) mm/kN	Shear flexibility in cantilevered diaphragms [see Figure 18 a)] mm/kN
<i>Sheet deformation</i>		
Profile distortion	$c_{1.1} = \frac{ad^{2.5}\alpha_1\alpha_4K}{Et^{2.5}b^2}$	$c_{1.1} = \frac{ad^{2.5}\alpha_1\alpha_4K}{Et^{2.5}b^2}$
Shear strain	$c_{1.2} = \frac{2a\alpha_2(1+\nu)[1+(2h/d)]}{Etb}$	$c_{1.2} = \frac{2a(1+\nu)\{1+(2h/d)\}}{Etb}$
<i>Fastener deformation</i>		
Sheet to purlin fasteners	$c_{2.1} = \frac{2as_p p \alpha_3}{b^2}$	$c_{2.1} = \frac{2as_p p}{b^2}$
Seam fasteners	$c_{2.2} = \frac{2s_s s_p (n_{sh} - 1)}{2n_s s_p + \beta_1 n_p s_s}$	$c_{2.2} = \frac{2s_s s_p (n_{sh} - 1)}{2n_s s_p + \beta_1 n_p s_s}$
Connections to rafters	4 sides fastened $c_{2.3} = \frac{4(n+1)s_{sc}}{n^2 n_{sc}}$ or 2 sides only fastened with gable shear connectors $c_{2.3} = \frac{4(n-1)}{n^2 n_p} \left(s_{pr} + \frac{s_p}{\beta_2} \right)$	4 sides fastened $c_{2.3} = \frac{2s_{sc}}{n_{sc}}$ or 2 sides only fastened $c_{2.3} = \frac{2}{n_p} \left(s_{pr} + \frac{s_p}{\beta_2} \right)$
<i>Flange forces</i>		
Axial strain in purlins	$c_3 = \frac{n^2 a^3 \alpha_3}{4.8EAb^2}$	$c_3 = \frac{2a^3}{3EAb^2}$
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 mm²);
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/mm²);
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).