# THIN-WALLED CONSTRUCTION

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**ESDEP WG 9**

**THIN-WALLED CONSTRUCTION**

# Lecture 9.1: Thin-Walled Members and Sheeting

### OBJECTIVE/SCOPE

To introduce cold-formed members, and to discuss their manufacture, applications and design.

### PREREQUISITES

Lecture 6.2: General Criteria for Elastic Stability

Lectures 6.6: Buckling of Real Structural Elements

Lecture 8.1: Introduction to Plate Behaviour and Design

### RELATED LECTURES

Lecture 2.4: Steel Grades and Qualities

Lecture 4A.3: Practical Corrosion Protection for Buildings

Lecture 14.1.1: Single-Storey Buildings: Introduction and Primary Structure

### SUMMARY

This lecture introduces cold-formed sections and members; it discusses methods of manufacture and applications and shows how these sections have certain advantages over more conventional steelwork. The design methods generally used are explained and advice is given on practical considerations.

## 1. INTRODUCTION TO THE DESIGN OF COLD- FORMED SECTIONS

Formerly the use of cold-formed thin-walled steel sections was mainly confined to products where weight saving was of prime importance, e.g. in the aircraft, railway and motor industries. Simple types of cold-formed profiles (mainly similar to hot-rolled shapes), as well as profiled sheeting, have also been used as non-structural elements in building for about one hundred years.

Systematic research work, carried out over the past four decades, as well as improved manufacturing technology, protection against corrosion, increased material strength and the availability of codes of practice for design, have led to wider use of cold-formed sections within the building industry. In many countries cold-formed steel construction is the fastest growing branch of the structural steel market.

## 1.1 Typical Products and Uses

Cold-formed sections are prismatic elements, of constant sheet thickness, formed by a sequence of plane sub-elements and folds in order to perform specific load bearing functions for members and also sometimes a space-covering function (see Figures 1-3).

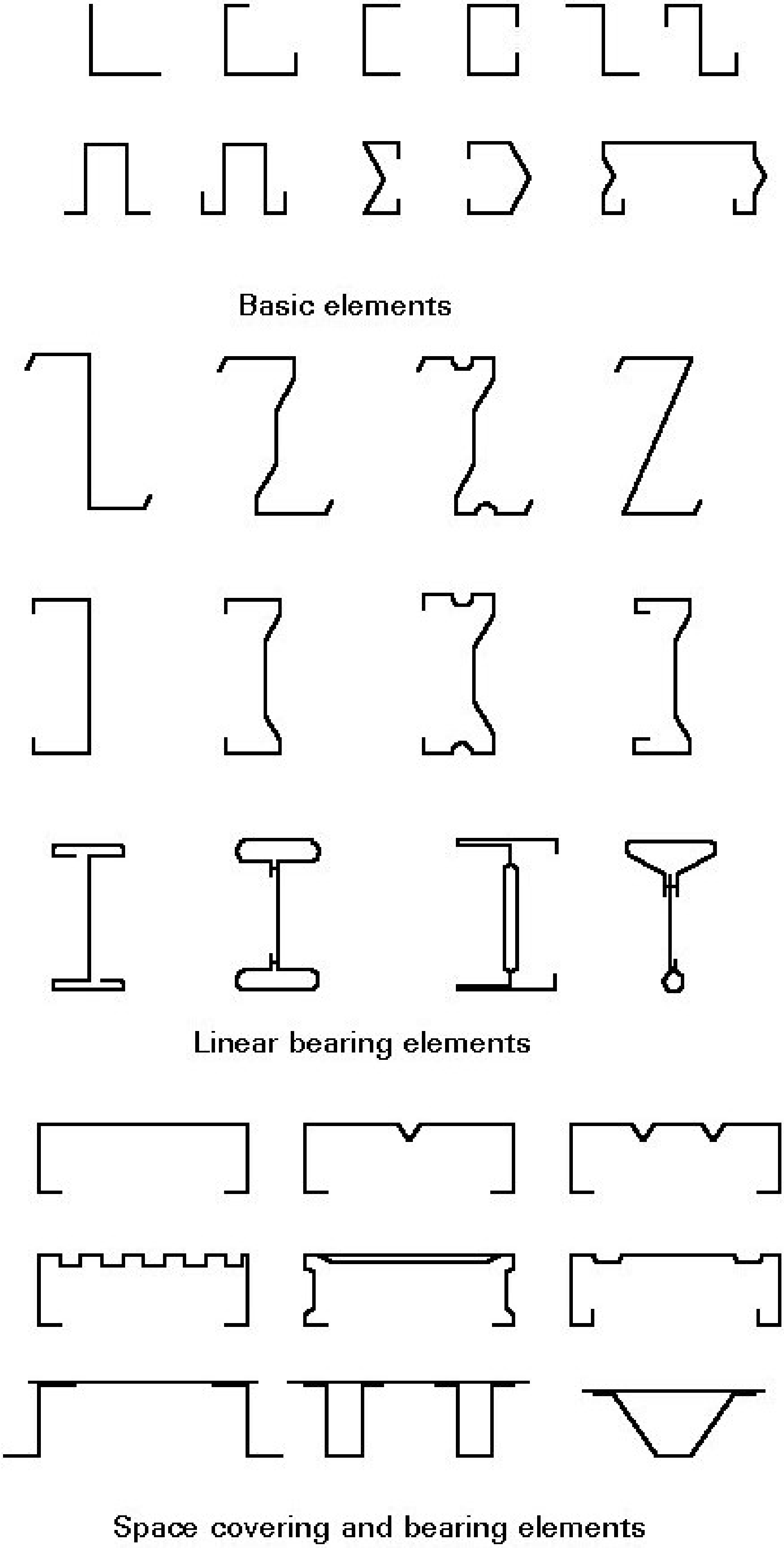



Figure 1 Cold - formed members

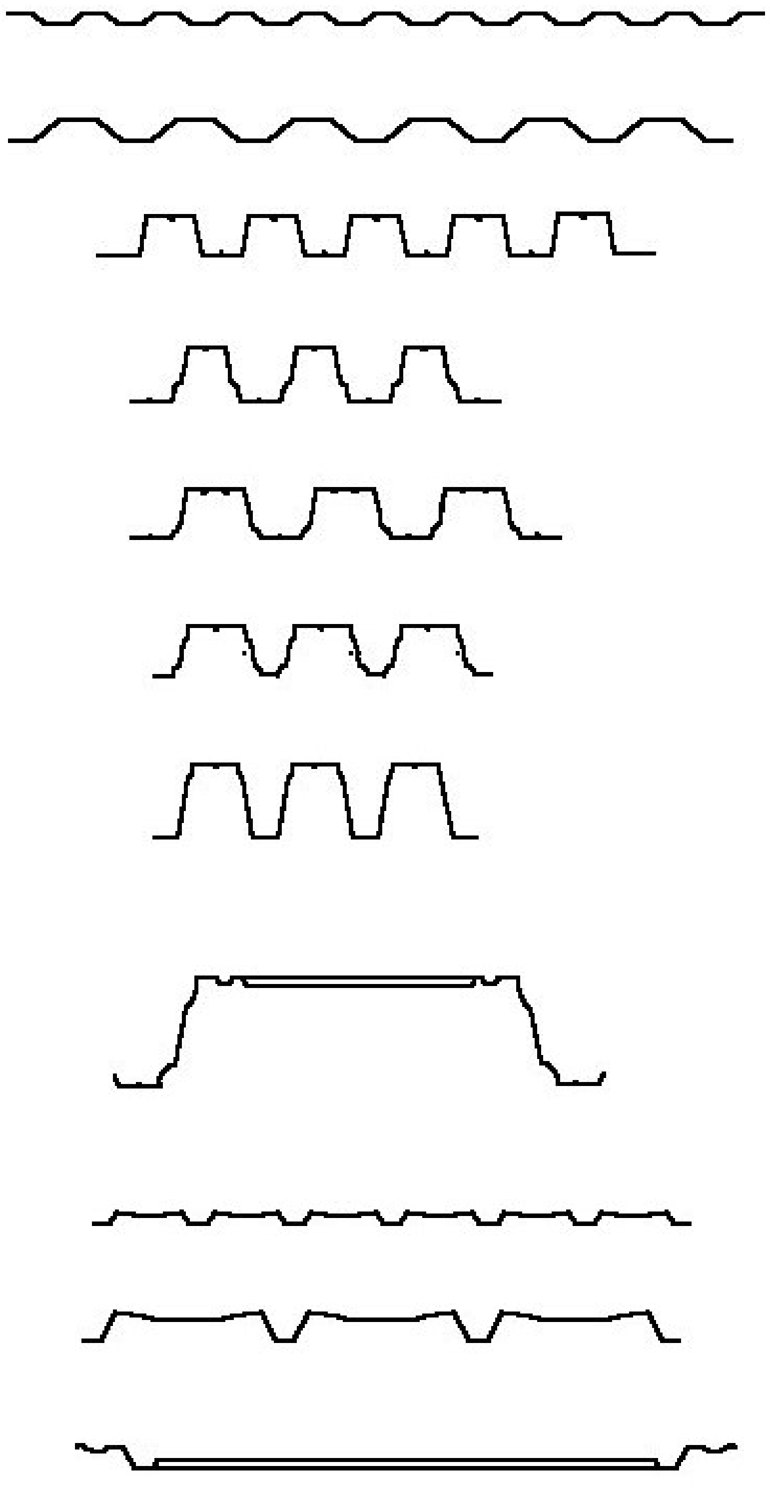


Figure 2 Profiled sheeting

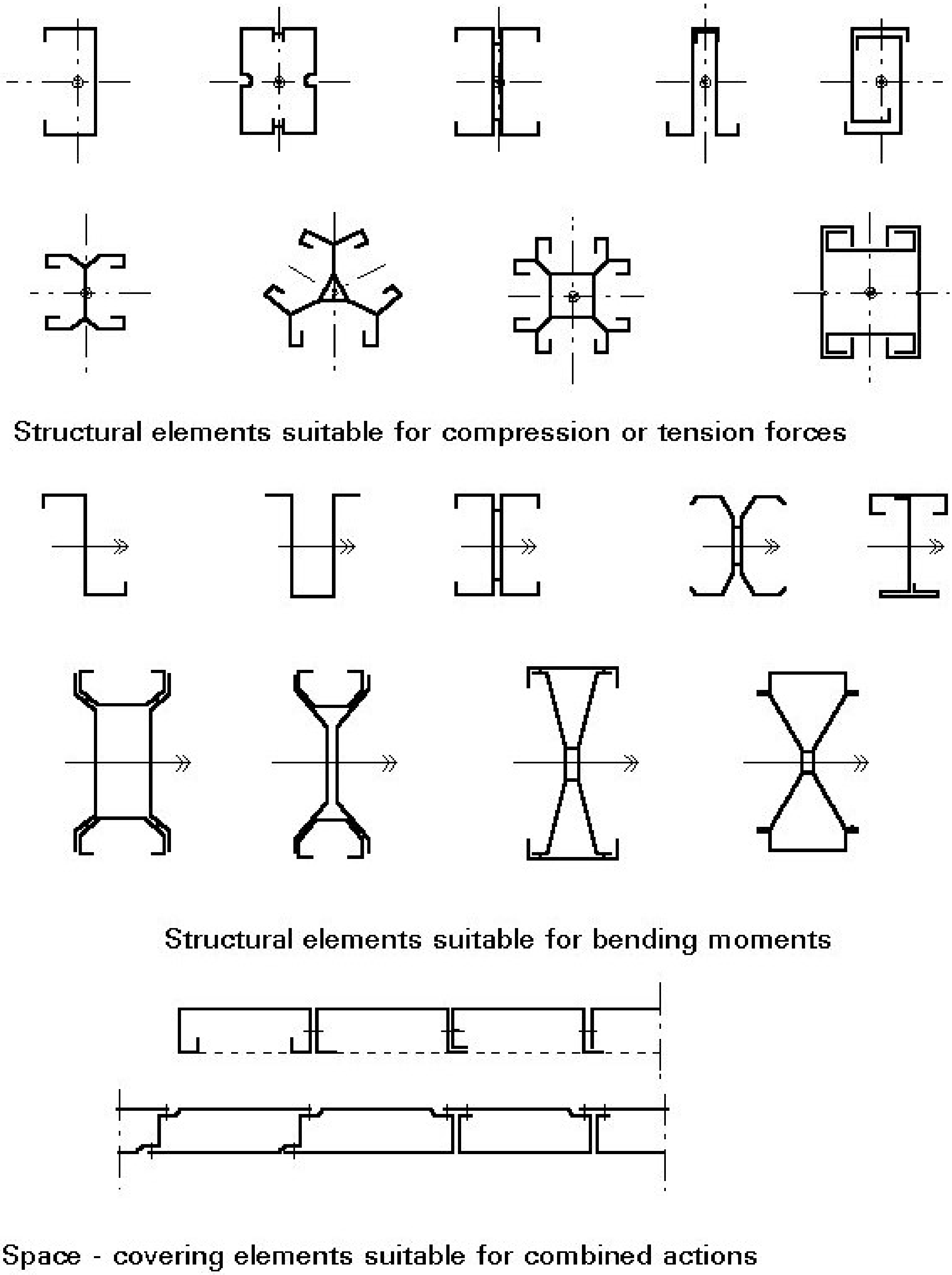



Figure 3 Building elements based on cold-formed sections

A characteristic feature of cold-formed sections is that slender parts in compression are stiffened by folding (intermediate and edge stiffeners), which delays or prevents premature buckling of the compressed zones. This phenomenon is discussed in Section 2.

## 1.2 Applications

The types of products available for use in building structures are:

- linear members, mainly used in the higher range of thickness, as beams for comparatively low loads on small spans (purlins and rails), as columns and vertical supports, and in trusses.
- plane load-bearing members in the lower range of thickness and with load-bearing resistance, are used in cases where a space-covering function under moderate distributed loading is needed, e.g. floors, walls, roofs.

Cold-formed thin-walled building elements are, therefore, mainly used in low-rise and light industrial buildings with small spans, where combination of cold-formed sections and profiled sheeting can be utilised to the best advantage. Stressed skin design of profiled sheeting can also have applications in the more interesting field of space structures such as folded plates or hyperbolic paraboloid shells.

## 1.3 Advantages

The use of cold-formed structural members offers many advantages over construction using more standard steel elements;

- the shape of the section can be optimised to make the best use of the material.
- there is much scope for innovation (in practice this has proved to be very significant).
- cold-formed members combined with sheeting offer economic and reliable solutions which provide a space-covering function and lateral restraint against buckling. Light-weight industrial buildings constructed form cold-formed members and sheeting are an example of the combination of these two effects (Figure 4).

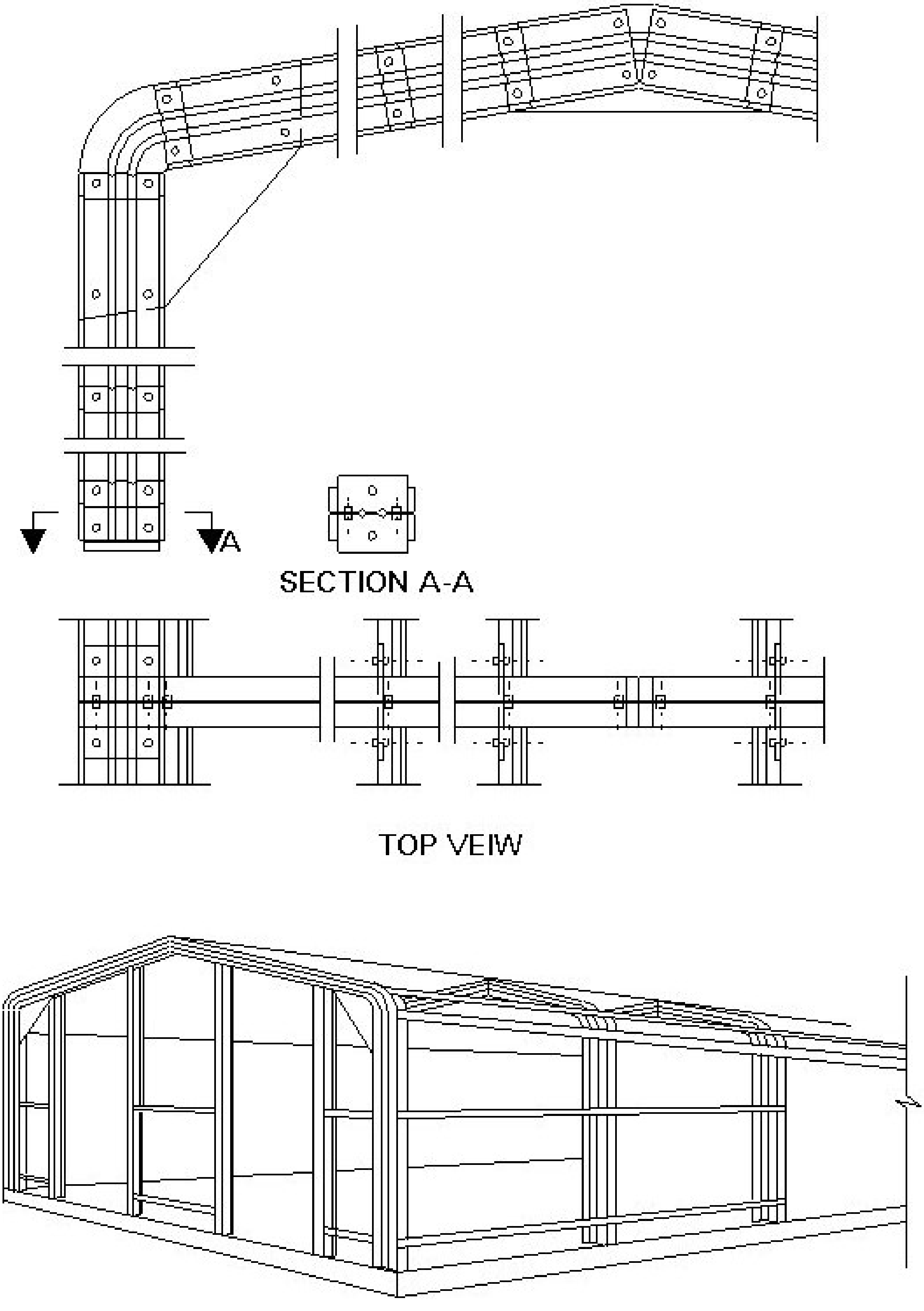



Figure 4 Light-weight industrial building with components of cold-formed members and sheeting

These advantages can, therefore, be generally classified as weight-saving, by optimization of the products with respect to the load-bearing function and constructional demands; and functional performance in terms of space-covering ability.

## 1.4 Manufacture

Cold-formed sections can be manufactured either by folding (Figure 5), press braking (Figure 6), or cold rolling (Figure 7).

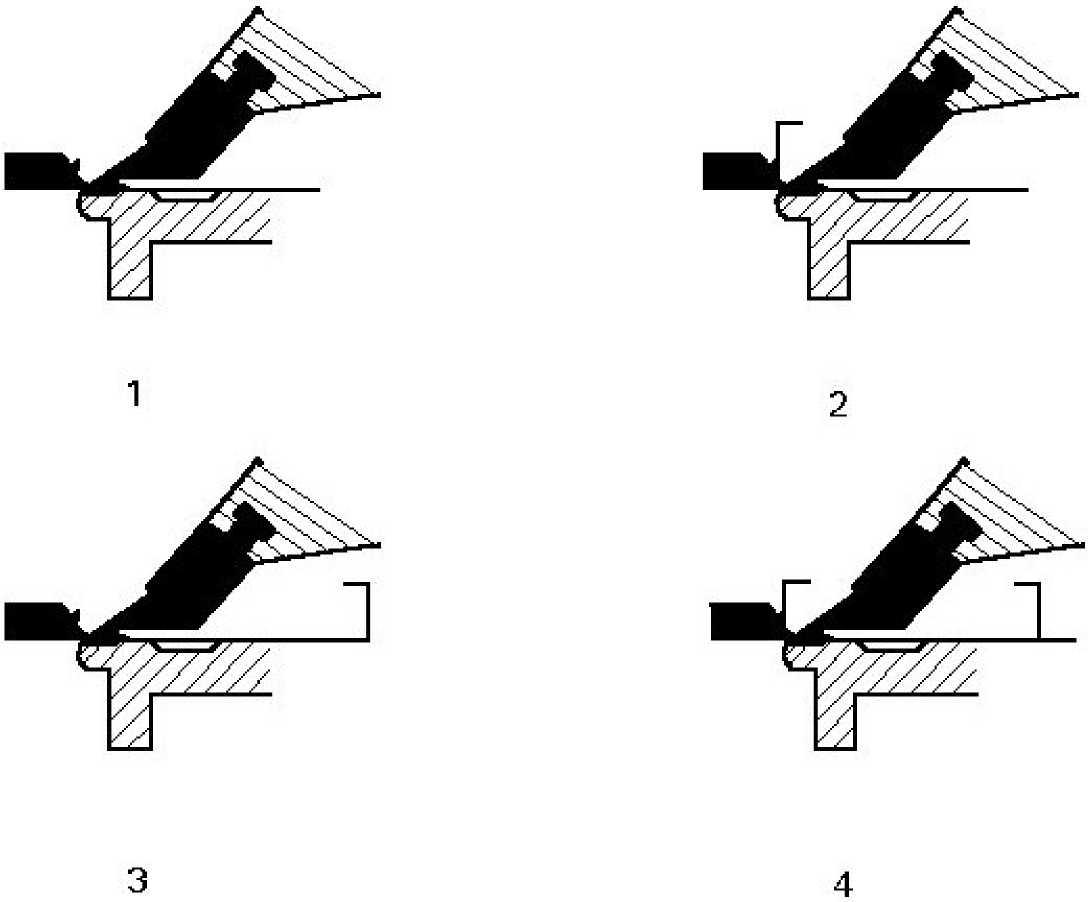



Figure 5 Manufacturing by folding

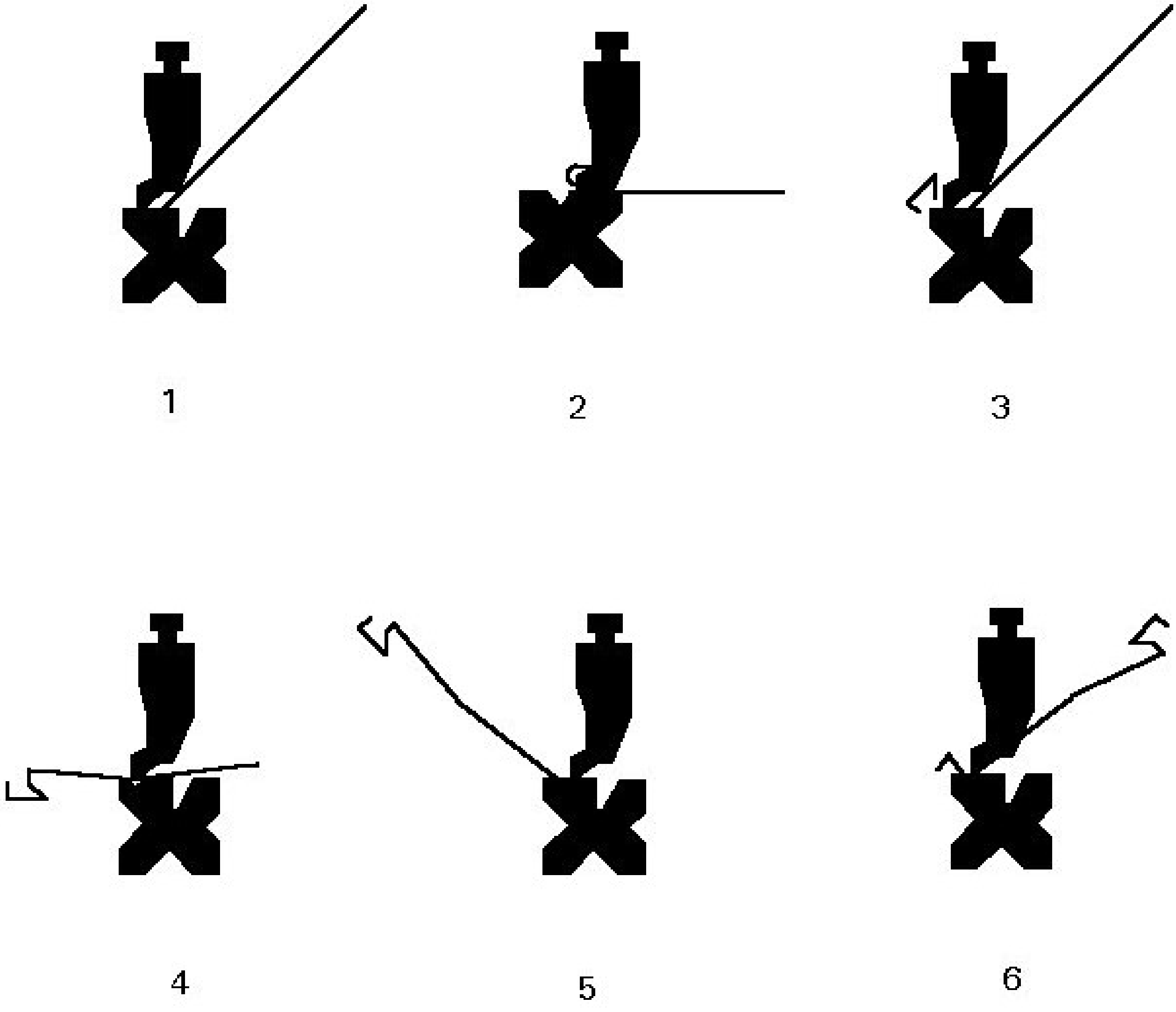



Figure 6 Manufacturing by press braking

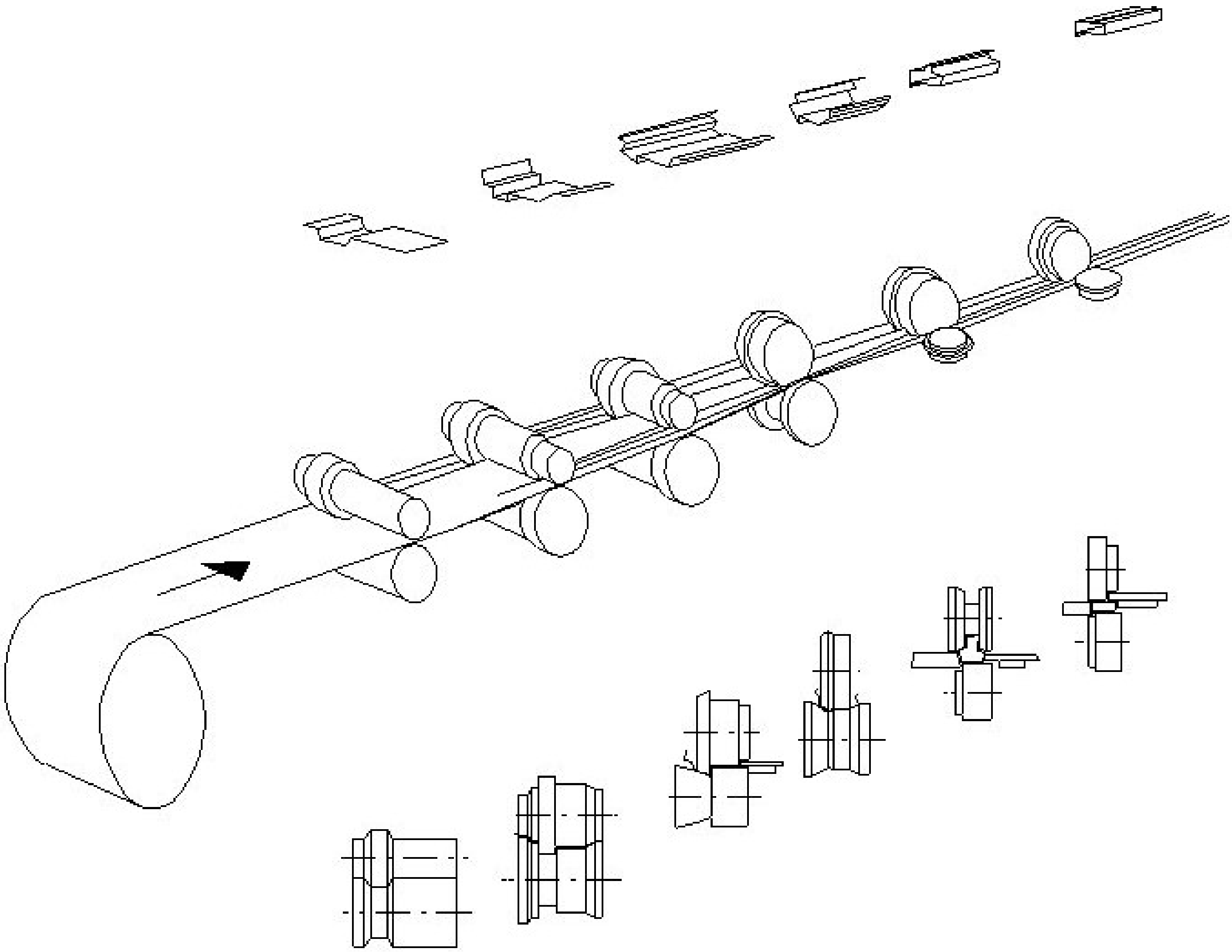


Figure 7 Manufacturing by cold rolling

For small batches of building elements with lengths ≤ 6m (in exceptional cases ≤ 12m), it is normally advantageous to use hydraulic folding or press-braking machines. The effort required to form the shape depends on the sheet thickness, the ductility of the material and the shape of the section, which is limited by the strip width.

These manufacturing methods allow the sections to be shaped for optimum load- bearing resistance, intended purpose and further product processing.

## 1.5 Materials

The type of steel used should be suitable for cold-forming and, if required, for galvanising. For cold-formed sections and sheeting it is preferable to use cold-rolled continuously galvanized steel with yield stresses in the range of 280-320-350N/mm$^2$, and with a total elongation of at least 10% for a 12,5mm wide strip, referred to a gauge length $l_o$ =80mm, and a ratio of ultimate tensile strength to yield stress of at least 1,1.

Under normal conditions, zinc protection Z275 (275g/m$^2$) is sufficient; in more corrosive environments, improved protection using suitable coating systems may be necessary. Continuously applied zinc protective coating systems are generally limited in core thickness to about 3,5mm. For increased material thickness, hot-dip galvanizing and site- or shop-applied top coats may be used.

## 1.6 Effects of Cold Forming

Cold-forming techniques allow the geometrical properties of a shape to be readily varied. It is possible, therefore, to influence the load-bearing behaviour of the element with respect to strength, stiffness and failure modes by, for example, the introduction of intermediate stiffeners or by ensuring adequate width-to-thickness ratios in adjacent flat parts of the section.

As cold forming of the steel sheet involves work hardening effects, the yield stress, the ultimate strength and the ductility are all locally influenced by an amount which depends on the bending radius, the thickness of the sheet, the type of steel and the forming process. The average yield stress of the section then depends on the number of corners and the width of the flat elements. The effect of cold forming on the yield stress is illustrated in Figure 8.

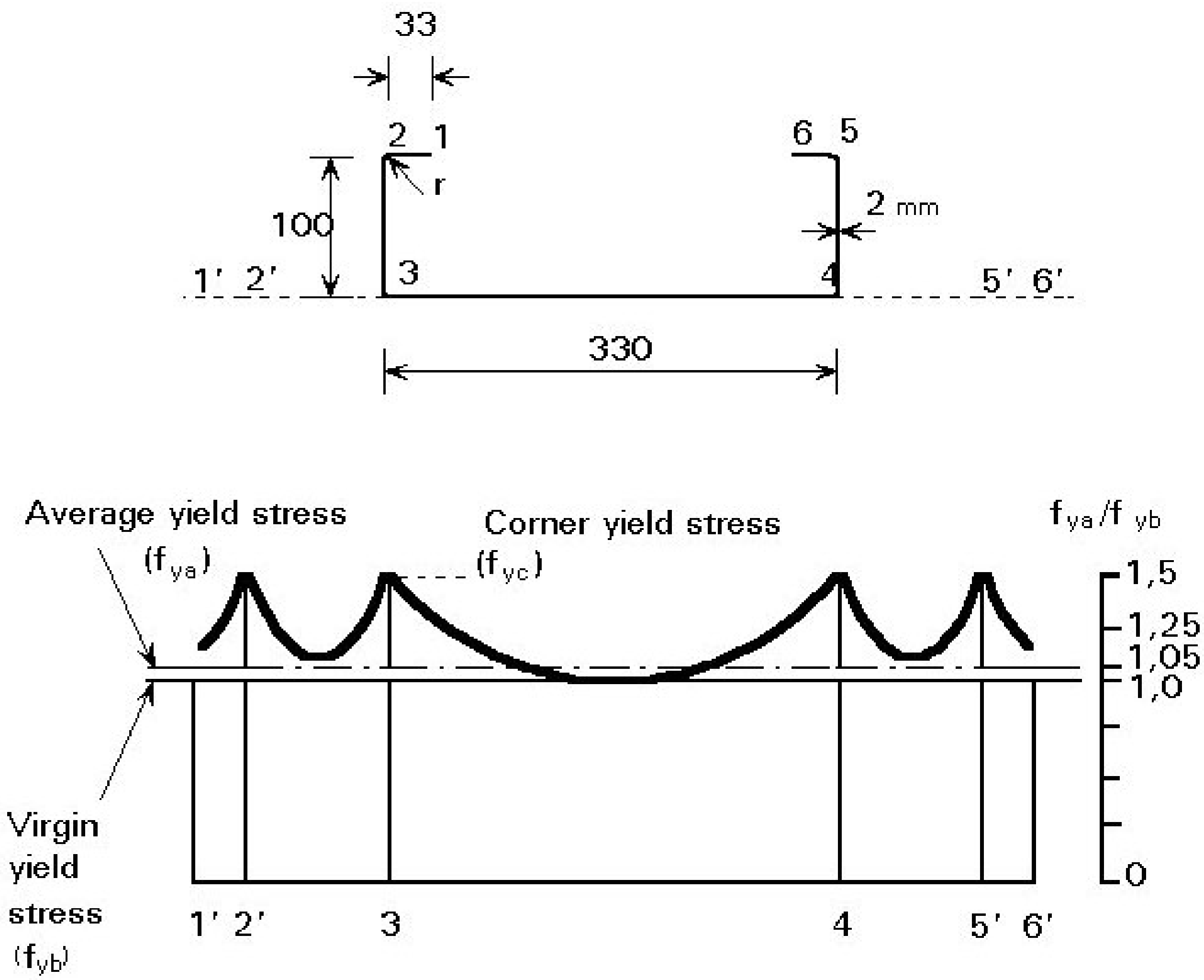


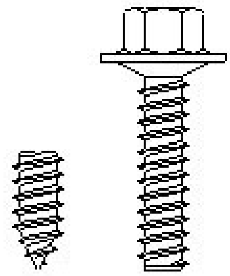

Figure 8 Example of cold - forming effects on the yield stress

The average yield stress can be estimated by approximate expressions given in the appropriate codes. In the example, the average yield stress ratio $f_{ya}/f_{yb} \approx$ 1,05 and the corner yield stress ratio $f_{yc}/f_{yb} \approx$ 1,4.

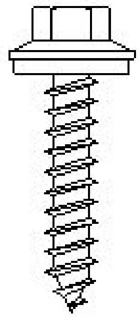
During the cold-forming process varying stretching forces can also induce residual stresses, which can significantly change the load-bearing resistance of a section. Favourable effects can be observed if residual stresses are induced in parts of the section which act in compression and, at the same time, are susceptible to local buckling.

## 1.7 Connections

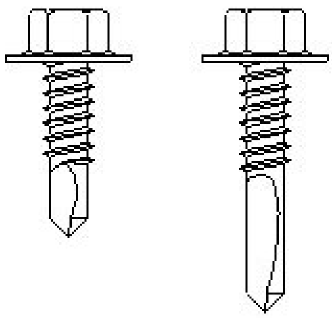
The development of lightweight construction requires the availability of adequate fastening techniques; suitable fasteners are bolts with nuts, blind rivets, self-tapping screws, self-drilling screws and powder actuated fasteners (Figure 9); industrialized production spot welding and adhesives may also be used. In order to use fasteners in building construction, it is necessary to be familiar with the behaviour of the connections and to lay down design criteria for serviceability and stability. Comprehensive experimental and theoretical investigations form the basis of the analytical evaluation of the load-bearing behaviour of the fasteners under static and dynamic loading. Figure 10 shows fields of application and the appropriate failure modes.



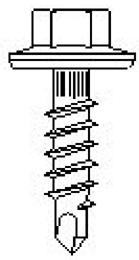
Self-tapping screws
(Sheet to steel substructure)



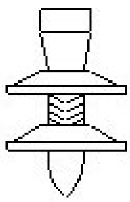
Self-tapping screws
(Sheet to timber substructure)



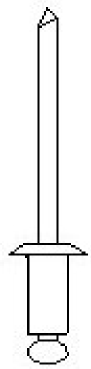
Self-drilling screws
(Sheet to steel substructure)



Self-drilling screws
(Sheet to sheet)



Powder-actuated fasteners
(Sheet to steel substructure)



Blind rivet
(Sheet to sheet)

Figure 9 Non-conventional fasteners for thin-walled elements

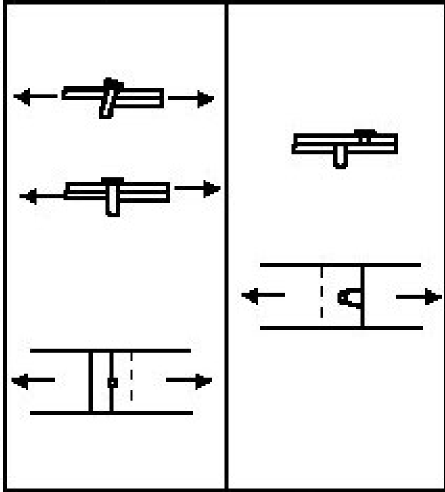
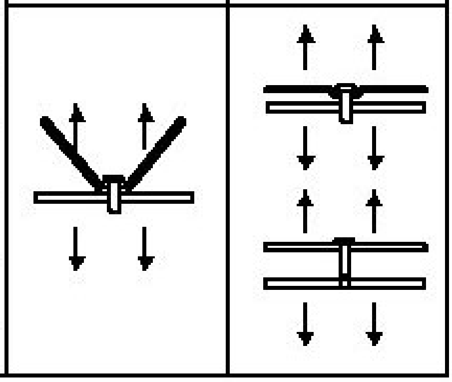
| Type of loading | Type of connection: Thin-to Thin | Type of connection: Thin-to Thick | Type of fastener | Type of failure: Acceptable | Type of failure: Undesirable |
|---|---|---|---|---|---|
| Shear load | X | | Blind rivets |  | |
| | X | | Self-drilling screws | | |
| | X | X | Self-tapping screws | | |
| | X | X | Welds | | |
| | | X | Cartridge fired pins | | |
| | | (X) | Bolts | | |
| | (X) | (X) | Friction-grip bolts | | |
| | (X) | (X) | Adhesives | | |
| Tensile load | X | | Blind rivets |  | |
| | (X) | | Self-drilling screws | | |
| | X | X | Self-tapping screws | | |
| | | X | Cartridge fired pins | | |
| | (X) | (X) | Bolts | | |
| | (X) | (X) | Friction-grip bolts | | |

Figure 10 Type of fasteners and connections; failure modes

Generally, failure modes causing sudden failure of connections should be avoided. Local over-stressing is indicated by large deformations and should be reduced by load transmission to adjacent fasteners.

### 1.8 Codes

Extensive research and product development in the past has led to national design specifications for cold-formed sections and structures in many countries. European Recommendations for the design of cold-formed sections have been developed by the European Convention for Constructional Steelwork [1,2], and form the basis for Part 1.3 of Eurocode 3 "Cold-formed thin-gauge members and sheeting" [3].

## 2. CHARACTERISTIC BEHAVIOUR

### 2.1 General

Compared with conventional steel members, thin-walled structural elements are characterised by:

- relatively high width to thickness ratios.

- unstiffened or incompletely restrained parts of sections.
- singly symmetrical or unsymmetrical shapes.
- geometrical imperfections of the same order as or exceeding the thickness of the section.
- structural imperfections caused by the cold-forming process.

As a consequence, a number of factors must be considered when designing these elements:

- buckling within the range of large deflections.
- effects of local buckling on overall stability.
- combined torsional and flexural buckling.
- shear lag and curling effects.
- effects of varying residual stresses over the section.

Under increasing load, thin-walled structural elements are generally subject to varying non-linear distributions of stress and strain over the cross-section, often in conjunction with substantial out-of-plane deflections. There is also the possibility of different failure modes, particularly for sections with flat parts in compression which are unstiffened, i.e. elastically restrained along one edge only.

The influence of stiffeners on the load-bearing resistance is illustrated in Figure 11, where the mass and nominal force at failure of a hot-rolled profiled HEB240 is compared with different shapes of thin-walled elements. In addition, this example shows the advantage of the space-covering function of thin-walled elements. Another example is given in Figure 12, where the increase in moment resistance due to intermediate flange and web stiffeners is shown.

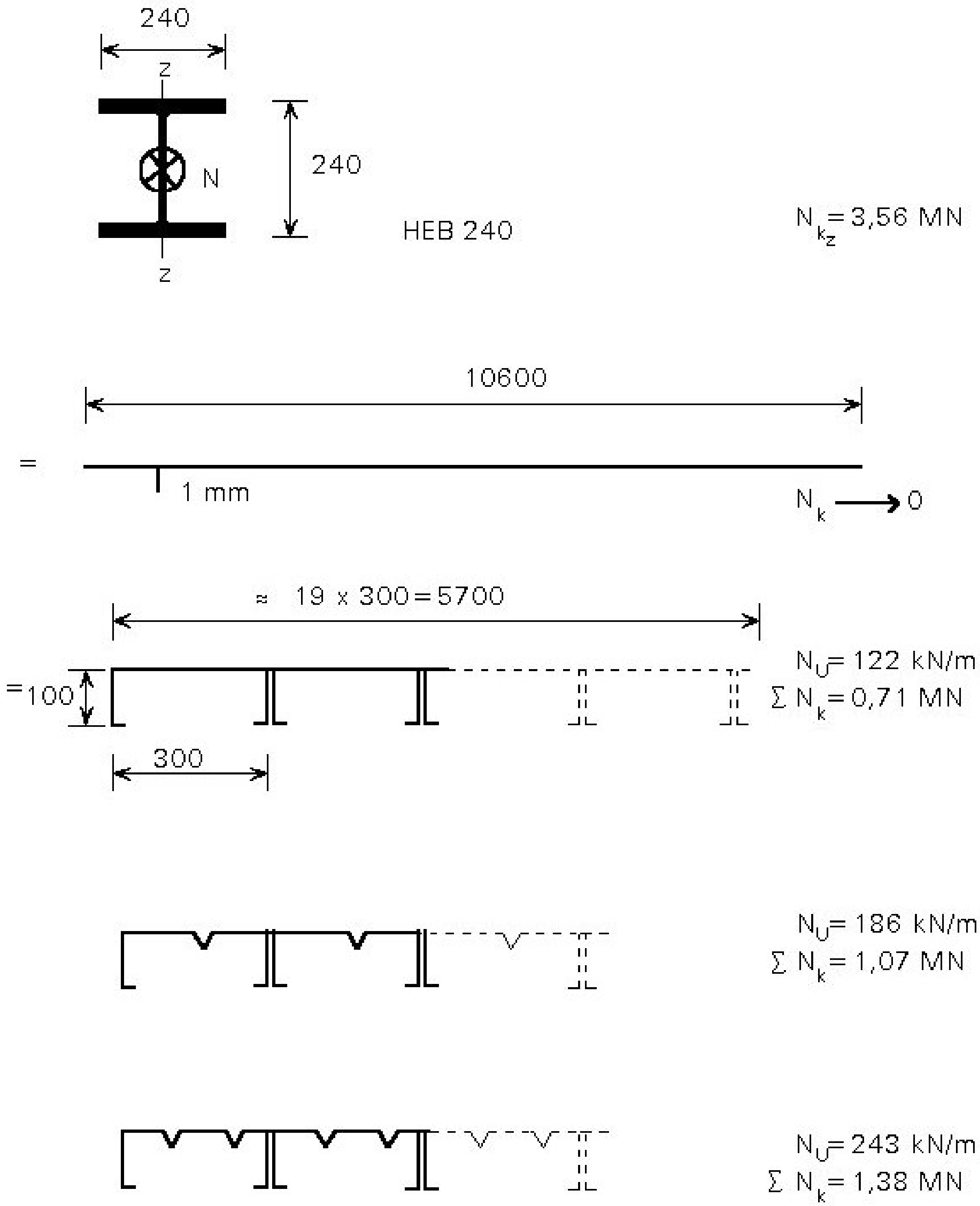



Figure 11 Example: The influence of stiffeners on the load-bearing resistance of thin - walled sections

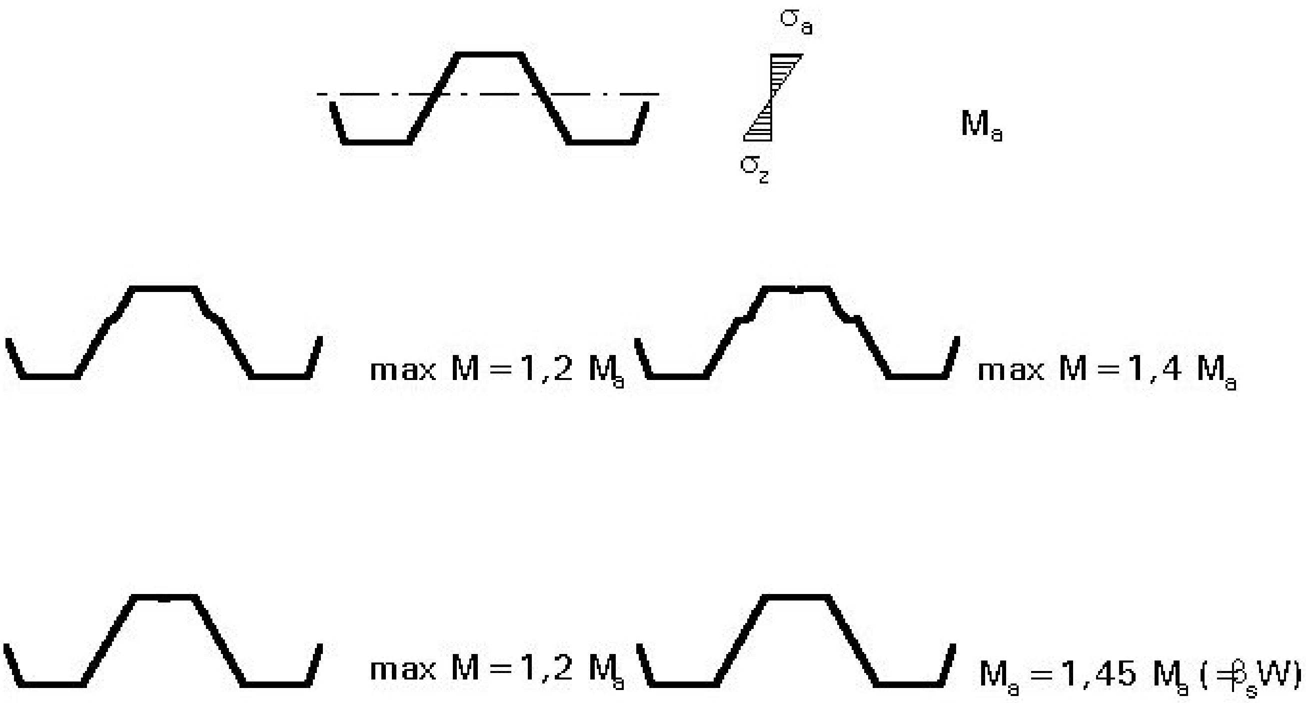



Figure 12 Example:The influence of intermediate stiffeners on the moment resistance of a thin - walled section

It is evident from the above discussion that an accurate analysis of the mode of action is usually extremely complicated, especially when imperfections and plasticity have to be taken into consideration. For practical design there is a need for simplified analytical models which allow an approximate but conservative estimate of the failure load and the behaviour of the structure under service load to be made.

## 3. LOCAL BUCKLING AND THE EFFECTIVE WIDTH CONCEPT

As illustrated above, the effect of local buckling in the compression elements of a section often determines the behaviour and load-bearing resistance. The theoretical solution to this problem, taking into account the post-buckling strength, is not practical for design purposes, for which the effective width design model has been developed.

It is evident from the stress distribution of a simply supported plate strip under normal forces (see Figure 13a) that in the post-buckling range the stresses are concentrated along the plate supports. Thus, the ultimate load can be determined from a uniform stress distribution within an effective width $b_{ef}$, which depends on the critical buckling stress ($\sigma_{cr}$=bifurcation stress) and the yield stress ($f_y$) of the plate material. The expression for $b_{ef}$, given by Von Karman, has been subsequently modified by Winter with provision for unintended geometrical imperfections (see Figure 13b).

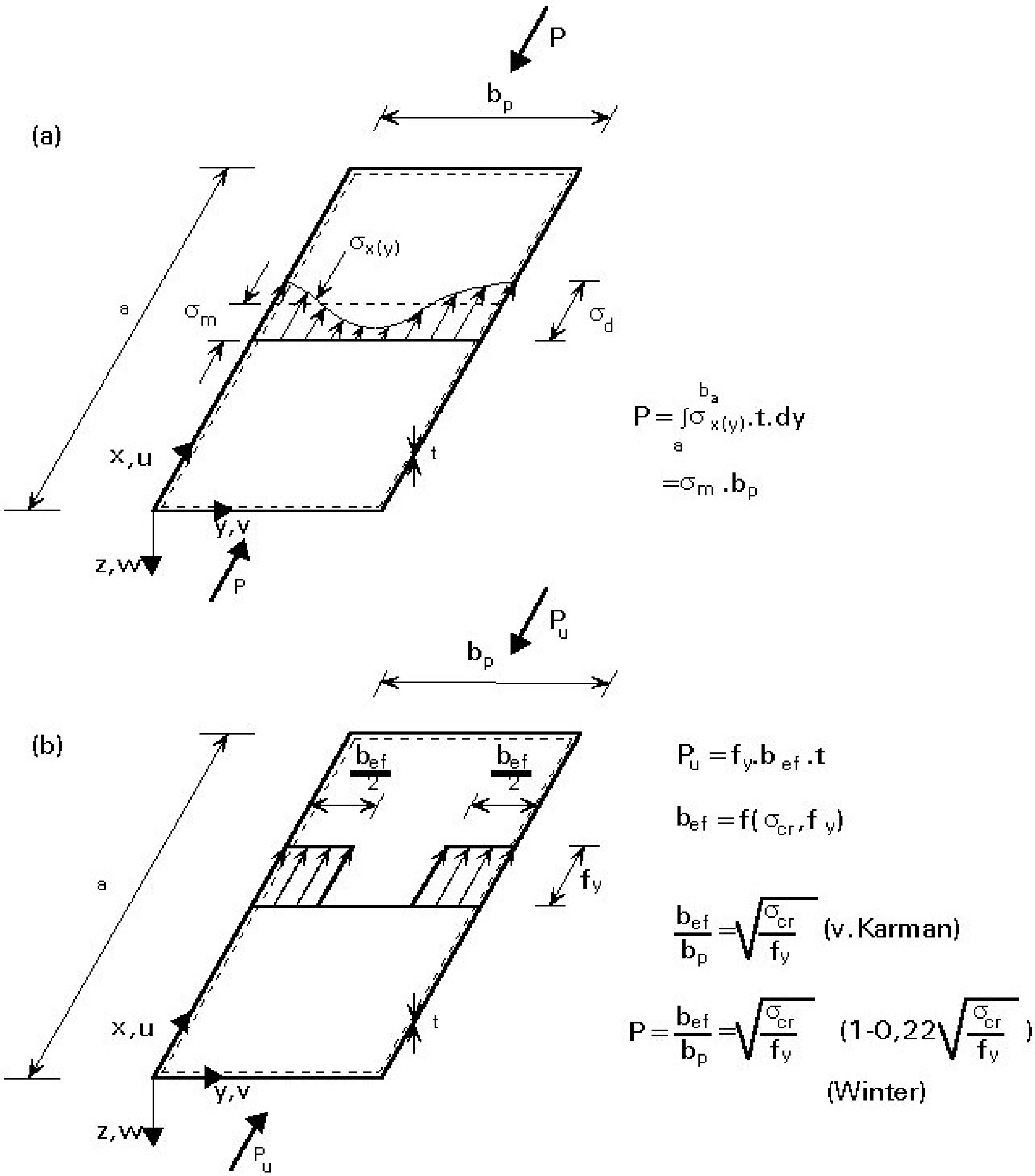



Figure 13 Stress distribution: (a) over a doubly supported plate, and (b) the appropriate effective width approach

The "Winter-Formula"

$$\rho = \frac{b_{ef}}{b_p} = \sqrt{\frac{\sigma_{cr}}{f_y}}\left(1 - 0{,}22\sqrt{\frac{\sigma_{cr}}{f_y}}\right)$$

implies that $b_{ef}=0{,}78\ b_p$, when $\sigma_{cr}=f_y$.

Substituting $\sigma_{cr}$, the relative slenderness $\bar{\lambda}_p$ is given by:

$\bar{\lambda}_p = (1{,}052/\sqrt{k_\sigma})(b_p/t)(\sqrt{f_y/E})$

and

$\rho = (1/\bar{\lambda}_p)(1 - 0{,}22/\bar{\lambda}_p)$

i.e. that $\rho = 1{,}0$ if $\bar{\lambda}_p \leq 0{,}673$.

If the buckling factor $k_\sigma$ for the bifurcation stress is known, the effective width $b_{ef}$ can be calculated; for example, $b_{ef}=b_p$ for a doubly supported plate element under constant normal stress with $k_\sigma=4$, if $b_p/t \leq 1{,}33\ E/f_y$; or for a singly supported plate element with $k_\sigma = 0{,}43$ if $b_p/t \leq 0{,}42\ E/f_y$. Assuming a yield stress $f_y=320 N/mm^2$, the elements are fully effective if $b_p/t \leq 34$ or $b_p/t \leq 11$ respectively.

Where appropriate these reduced effective widths should be taken into account by using the effective values of the section properties, i.e. the effective area ($A_{ef}$), section modulus ($W_{ef}$), and moment of inertia ($I_{ef}$). Appropriate $k_\sigma$ values are given in [1].

### 3.1 Doubly and Singly Supported Elements

Elements of a section are either doubly supported (flanges or webs of trapezoidal sheeting) or singly supported (flanges of U- or L-shaped profiles). Doubly supported elements are much stronger, especially when they also have low b/t ratios; this can be achieved by longitudinal edge stiffeners, (lips, bends folds) and/or by intermediate V, U or trapezoidal shaped stiffeners (see Figures 1, 2). These stiffeners, located in the compression zone, are subjected to normal forces and, working as beam columns on elastic foundations, are prone to buckling. This behaviour gives the basis for a simplified design model where the stiffener and adjacent parts of the flat elements are treated as beams on elastic foundation, with a spring stiffness dependent on the boundary conditions of the element.

The buckling mode and load depend on the effective area and stiffness of the stiffener. If the stiffener has an adequate stiffness, it may be treated as a rigid support for the adjacent flat element; codes of practice gives approximate criteria for assessing this. Depending on the buckling load of the stiffener, an interaction of local and global buckling may occur, as illustrated in Figure 14.

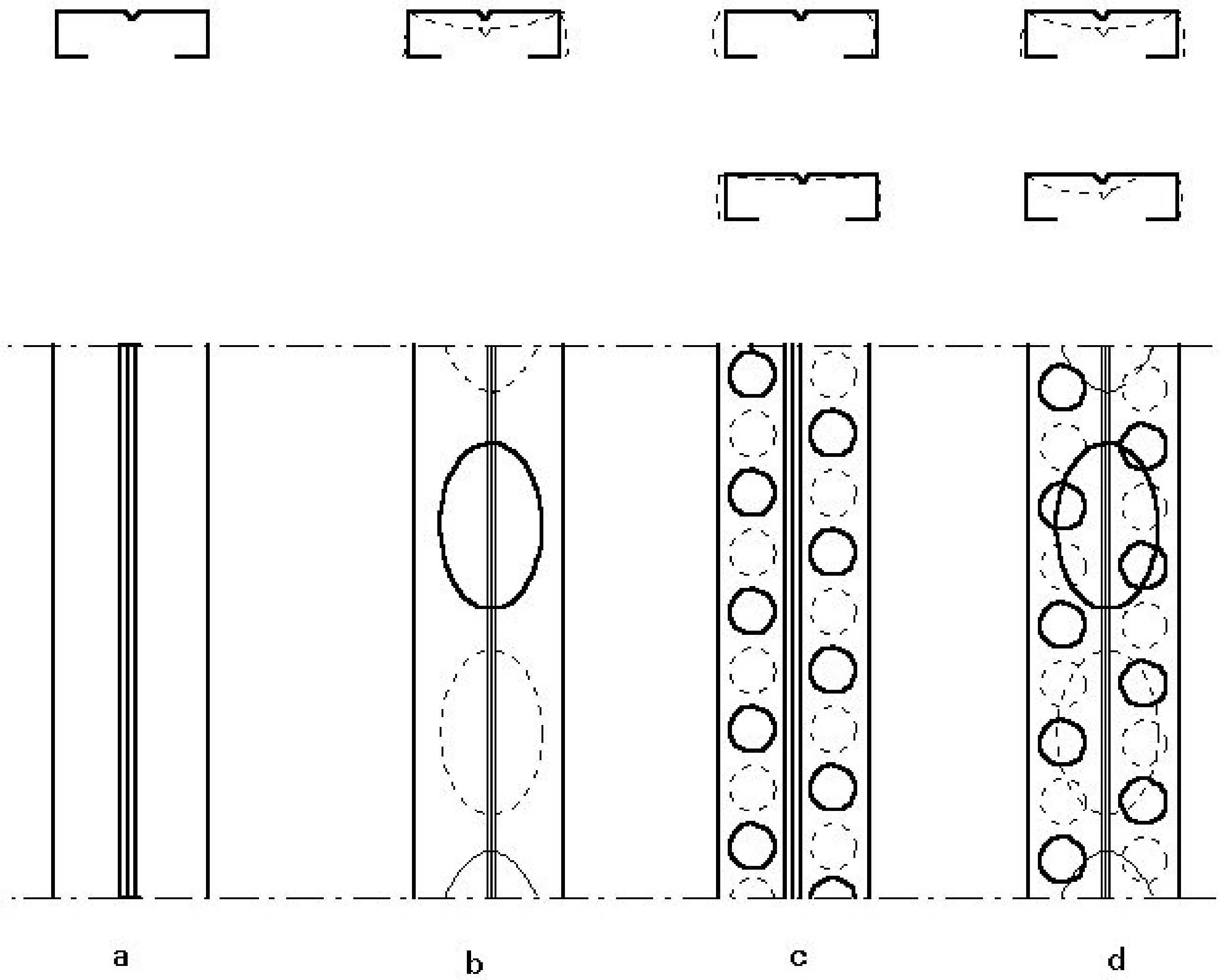



Figure 14 Interaction of local and global buckling, depending on the rigidity of the stiffener and adjacent plate elements

## 3.2 Effective Cross-sections

The first step when analyzing the load-bearing behaviour and estimating the failure load of a cold-formed member is to evaluate the effective width of the compression elements of the section, based on the appropriate stress distribution over the cross-section; the next step is to calculate the geometric properties of the effective section, taking into account the shift of the neutral axis caused by disregarding the ineffective parts of the section. Thereafter the design procedure is the same as for thick-walled sections. In general, the resistance of a thin-walled effective cross-section is limited by the design yield stress at any part of the section, based on an elastic analysis. Deviations from this rule are only permitted in special cases.

In the following, only basic design rules are used in order to explain the design procedure; the interaction of different effects, causing biaxial stress distributions, follows the same principles as for hot-rolled members.

In general terms, the design resistance is based on the value $f_y/\gamma_M$, where $\gamma_M$ is a partial safety factor for resistance (normally $\gamma_M$=1,1).

If the member does not buckle the moment resistance is given by:

$R_M = W_{eff}.f_y/\gamma_M$

where $W_{eff}$ is the section modulus of the effective cross-section. In order to avoid an iterative procedure, the effective portions of the web may be based on $\chi = \sigma_2/\sigma_1$, obtained by assuming the compression flange to be reduced, but the web being fully effective (see Figure 15).

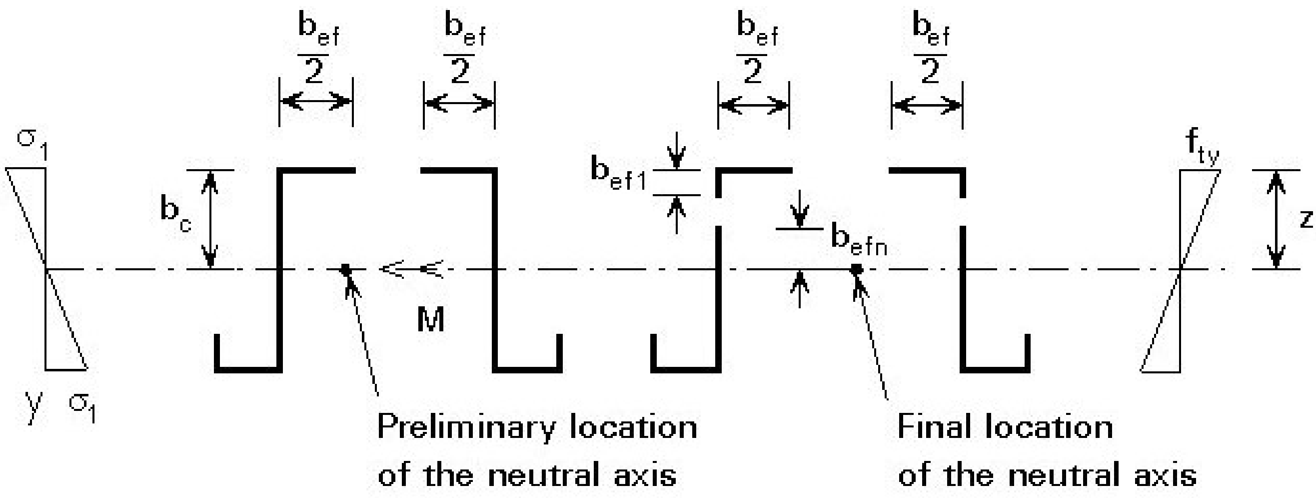



Figure 15 Effective section under bending action

When yielding first occurs on the tension side, the plastic reserves of the tension zone can be utilized until the compression stress reaches $f_y$. This will normally lead to iterative calculations.

If the same section is affected by a normal force acting at the centre of gravity of the cross-section, the effective section has to be determined with respect to compressive stresses in each element. As illustrated in Figure 16, it may happen that the centre of gravity of the effective section moves, causing an additional bending moment (M=Ne). This implies that cross-sections, where the effective neutral axis has shifted, have to be checked for compression and bending.

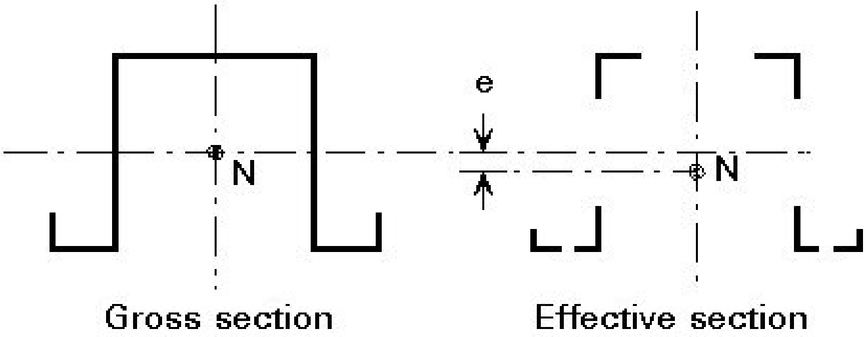



Figure 16 Effective section under action of compression

### 3.3 Web Buckling and Crippling

Web buckling can be caused by compressive bending stresses or by shear stresses above the critical buckling strength. In both cases, the buckling strength depends on the web slenderness ($s_w/t$). For a yield stress of about $f_y$=320N/$mm^2$, webs are prone to buckling if $s_w/t>80$ for pure bending and $s_w/t>60$ for pure shear. However, buckling does not necessarily imply a limit state for the structure, if post-critical equilibrium can be relied on (Figure 17).

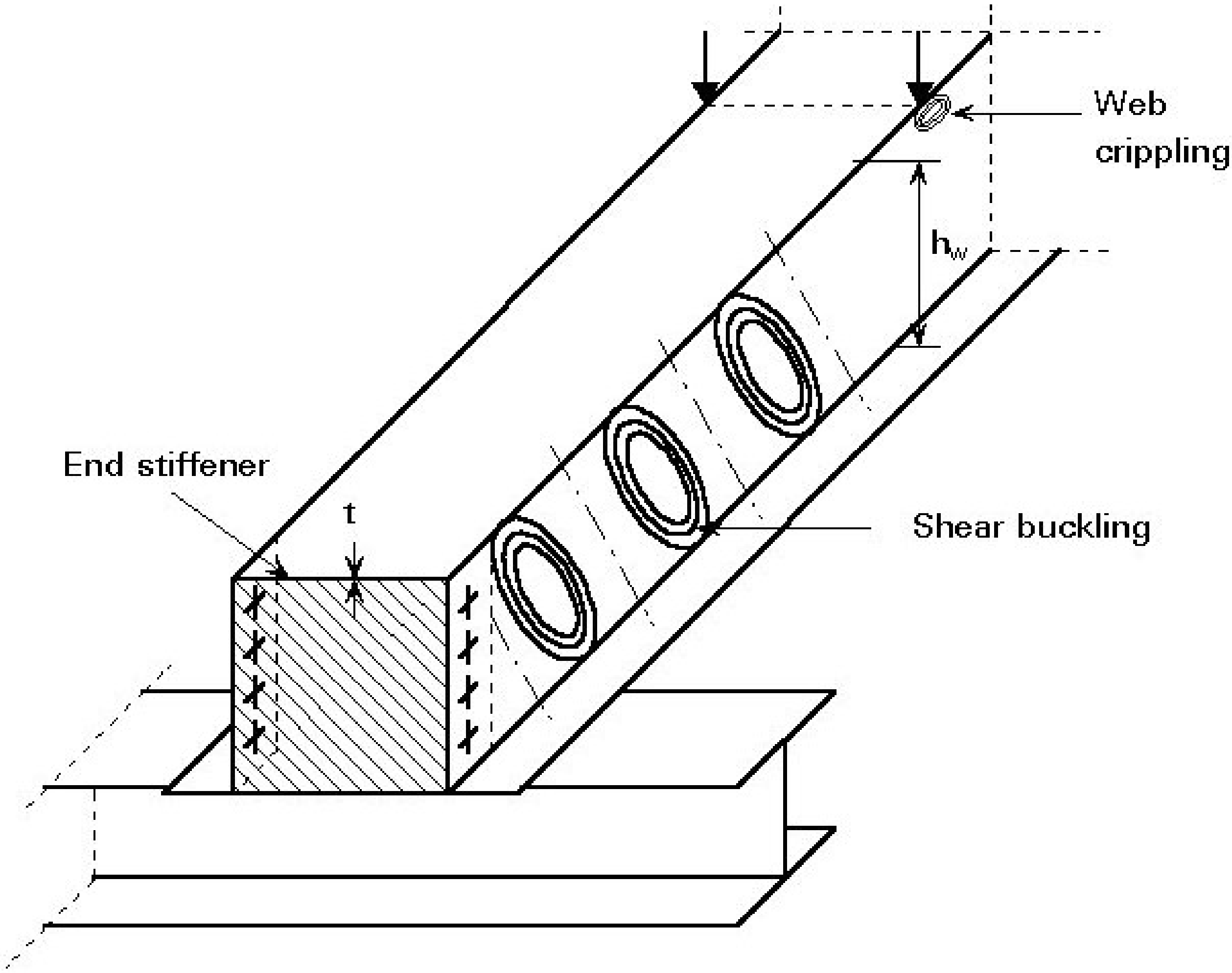
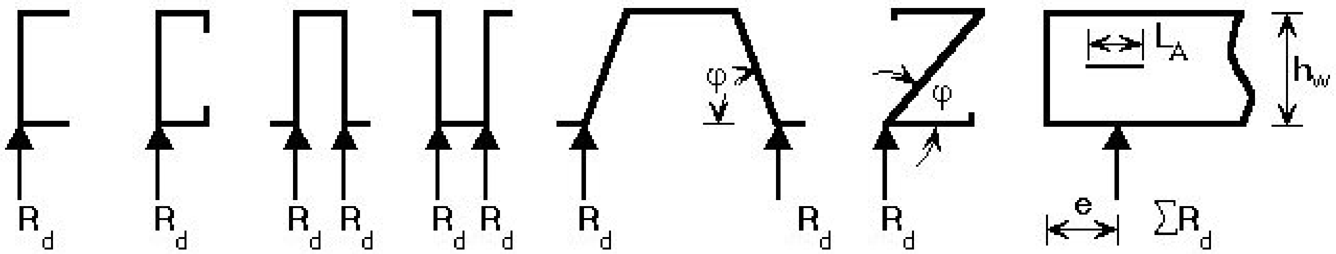
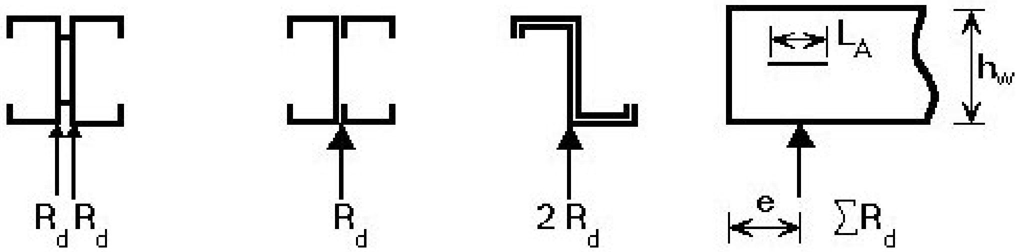



Figure 17 Effect of shear and concentrated loading

Crippling is a phenomenon associated with local loading of high intensity perpendicular to the plane of the web. It is most evident in the case of concentrated loading (Figure 17) or at intermediate supports of continuous beams. It is often more severe than web buckling, since crippling reduces the effective depth of a section and there is no post-critical strength. Depending on the webs' eccentricity relative to the load direction, and on the category of loads (see below), various values for web crippling resistance can be expected (Figure 18).




(a) Webs eccentric to load direction




(b) Webs concentric to load direction

Figure 18 Webs under concentrated loading, prone to web crippling

First category loads include end supports of beams, loads near the ends of a cantilever, and loads applied so close to a support that the distance from the support to the nearest edge of the load, measured parallel to the beam axis, is less than $1{,}5s_w$.

Second category loads include intermediate supports and loads situated more than $1{,}5s_w$ from a support or an end of a cantilever.

It should be noted that expressions given in the codes are semi-empirical.

## 3.4 Lateral-torsional Buckling

Unbraced members in flexure are generally susceptible to lateral-torsional buckling; this type of failure is more likely if the section is subjected to torsion due to the inclination of the main axis relative to the load direction, or if the shear centre of the section is not on the loading axis.

In order to minimise these effects, varieties of Z- and C-sections have been developed (see Figures 19 and 20).

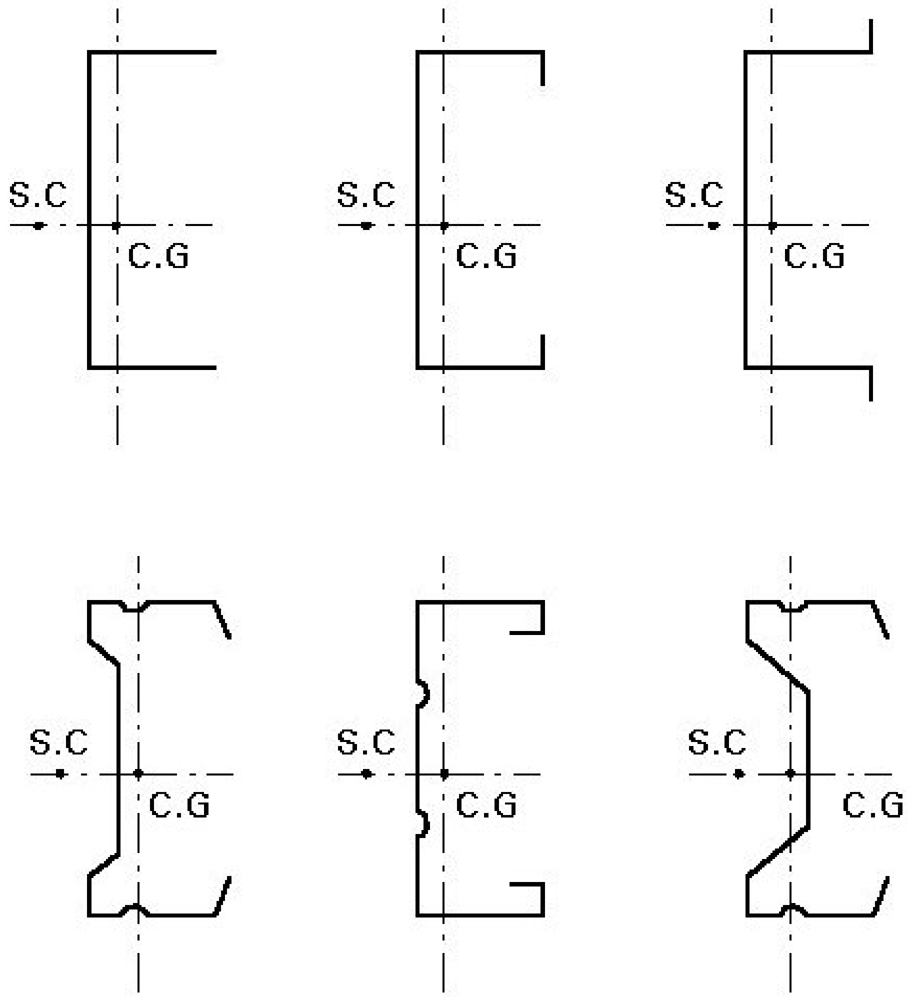



Figure 19 Varieties of C - sections in order to avoid local buckling and to bring the shear centre closer into the section

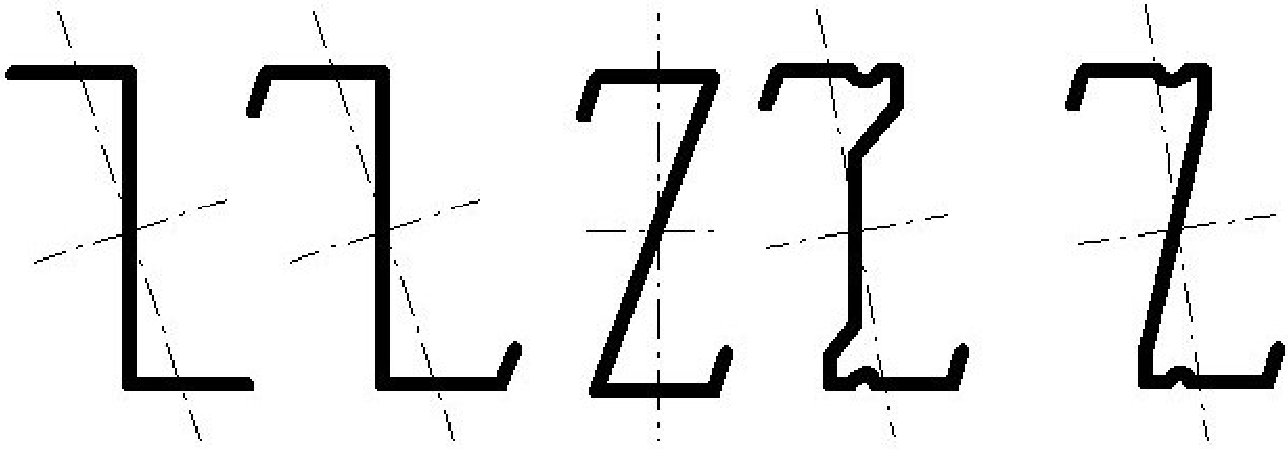


Figure 20 Varieties of Z - sections in order to avoid local buckling and to adjust the inclination of the principle axis

The susceptibility of thin-walled open sections to twisting and lateral-torsional buckling can effectively be neutralized by restraints provided by adjacent building elements, for example, metal sheeting connected to the sections using self drilling or self tapping screws.

In the case where Z-purlins are used for roof structures, the lower flange is normally free to rotate whereas the upper flange is attached to the sheeting. The in-plane stiffness of the sheeting prevents a lateral displacement of the upper flange and the distance between the fasteners and the edges of the section provides the lever arm for torsional restraint. The rotational spring stiffness $C\upsilon$ [Nm/rad] depends on the bending stiffness of the sheeting ($C\upsilon_m$), the distortion of the section ($C\upsilon_p$) and the stiffness of the connection between the sheeting and the purlin ($C\upsilon_A$); the last value must be estimated by tests.

From $1/C\upsilon = 1/C\upsilon_m + 1/C\upsilon_p + 1/C\upsilon_A$

the effective value of $C\upsilon$ can be derived.

The exact analytical solution of the problem of lateral buckling of continuous beams is too complicated for practical use; however, the beam-on-elastic-foundation model can help to solve the problem.

## 3.5 Interaction of Local and Global Buckling

It is obvious that local buckling influences the load-bearing resistance of a section subjected to axial loading. Using the effective width method, the reduced (effective) area $A_{ef}$ has to be taken into account when calculating the slenderness of the column $(l\pi/i_{ef}).(A_{ef}/Ag)^{1/2}$ and when determining the design resistance $N_d=k\ A_{ef}\ f_y/\gamma_M$. The buckling factor k is taken from the relevant European buckling curves (a-d) for the appropriate value of $\bar{\lambda}$. The classification of section types shows that members without end stiffeners should be avoided since the load-bearing resistance is relatively low (see also Lecture 9.2).

# 4. PRACTICAL CONSIDERATIONS

## 4.1 Good Practice Notes

As cold-formed sections are characterised by relatively low sheet thicknesses and/or high width thickness ratios account must be taken of:

- local buckling which can occur in the serviceability state.
- special requirements regarding corrosion protection.
- protection against unacceptable deformations during transport and erection of the structure.

Members and structures should be designed so that:

- deformations in the serviceability state are within acceptable limits with regard to functional requirements.
- preferably symmetrical (double-, single- or point-symmetrical) section shapes are chosen.
- the effective area of the section is as close as possible to the gross area (this can be achieved by the addition of intermediate stiffeners in flat parts of the section under compression).
- joints and connections have sufficient rigidity and rotation capacity.
- local instability phenomena are prevented by adequate stiffeners.
- global instability phenomena such as lateral buckling or increased stresses due to torsion of the section, can be prevented by adequate external restraint (for example, by connecting to building elements such as sheeting or bracing).
- essential load-bearing parts of the structure are protected against impact loads.
- corrosion due to poor detailing, e.g. detailing which allows accumulation of water, is avoided.

## 4.2 Influence of Joint Flexibility

If thin-walled members are connected to each other by mechanical fasteners, the rigidity of the joints is influenced by slip and by local buckling effects in front of the fasteners - the latter may occur if bolts are used in order to transmit relatively high forces; another possible problem is where the rigidity is reduced by large reductions in effective areas within the joint. The flexibility of the joint may influence the distribution and redistribution of bending moments and shear within the structure, and also the calculation of the load-bearing resistance. These effects must be properly investigated - by testing if necessary.

# 5. CONCLUDING SUMMARY

- Cold-formed products are typically used in building construction as light duty beams or columns, or as sheeting.
- Their shape can be optimised to reduce weight and facilitate functional performance.
- They are manufactured by folding, press braking or cold rolling. All of these

processes can result in an increase in yield strength.
- Design of cold-formed sections uses the concepts of effective width, giving effective section properties.
- For beam design maximum moment of resistance, lateral-torsional buckling (if unrestrained), and web buckling and crippling are the principles checks required.

# 6. REFERENCES

[1] European Convention for Constructional Steelwork: "European Recommendations for the Design of Light Gauge Steel Members", Publication 49, ECCS, 1987.

[2] European Convention for Constructional Steelwork: "European Recommendations for the Design of Profiled Sheeting", Publication 40, ECCS, 1983.

[3] Eurocode 3, Part 1.3: "Cold-formed Thin-gauge Members and Sheeting" CEN (in preparation).

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**ESDEP WG 9**

**THIN-WALLED CONSTRUCTION**

# Lecture 9.2: Design Procedures for Columns

### OBJECTIVE/SCOPE

To outline the design procedures required for cold-formed (thin-walled) columns.

### PREREQUISITES

Lecture 7.2: Cross-Section Classification

Lecture 7.3: Local Buckling

Lectures 7.5: Columns

### RELATED LECTURES

Lecture 6.1: Concepts of Stable and Unstable Elastic Equilibrium

Lecture 6.3: Elastic Instability Modes


### SUMMARY

The procedures for the design of thin-walled sections in compression are outlined [1, 2]. This involves the calculation of the effective section properties, determination of related slenderness values, and calculation of the design buckling load. For unsymmetric sections the effective section centroid will not be in the same position as that of the gross section. Bending will also have to be considered.


## 1. INTRODUCTION

In the design of compression members, two phenomena must be distinguished: global buckling which depends on the slenderness of the member, and local buckling which may occur if the b/t ratios of elements of the section are relatively large. The latter occurs in cold-formed members at a loading level lower than the global buckling level. In such a case, an interaction of local and global buckling gives a reduced global buckling load compared to that of a compact section.

The interaction can be simulated by replacing the cross-section with an effective section, taking into account the stress redistribution at each element of the section (see Lecture 9.1). This method allows the calculation of the load-bearing resistance of thin-walled members ("Class IV - sections") in the same way as for compact sections. Axial loading may be assumed if the compressive force is acting at the centroid of the effective cross-section.

## 2. PREPARATION OF DESIGN PROCEDURES

The design procedures outlined in Section 3, require the evaluation of the effective area and slenderness of the section, taking into account such appropriate geometrical properties as b/t ratios, rounding of corners, stiffeners and lips.

**Limits for b/t ratios**

The design rules give limits for b/t ratios as shown in Figure 1. These maximum width-to-thickness ratios depend partly on limited experimental evidence, and partly on experience from manufacturing and handling sections. Wide and flexible elements are prone to mechanical damage; the effective area is small compared to the total area, and buckles at service loads may be visible; some sections with high b/t ratios may, however, perform well and "design by testing" is, therefore, also recommended.

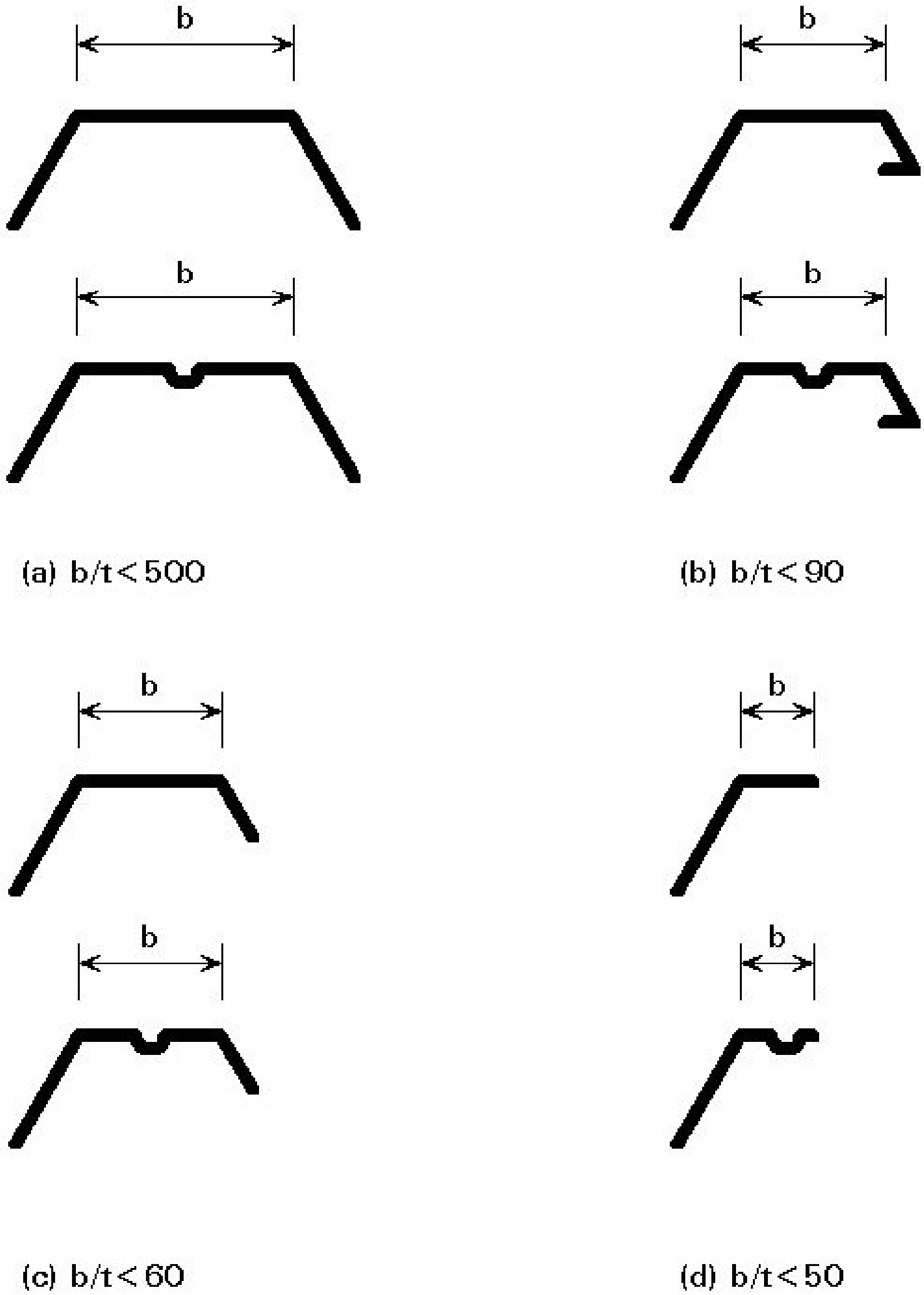



Figure 1 Maximum width to thickness ratios

**Formulae for effective width**

Elements of a section can be doubly supported (e.g. webs or flanges with adequate edge stiffeners); singly supported, e.g. flanges of U- or L-profiles, or elastically supported, e.g. flanges with insufficiently stiff edge stiffeners. For doubly and singly supported elements, the critical buckling stress $f_{cr}$ (bifurcation stress) under uniformly distributed normal stresses or even stress gradients provides the basis for the effective-width concept of the Winter formula (see Lecture 9.1) with the buckling factor $k_\sigma$ referred to the actual system and loading case. Appropriate $k_\sigma$ values can be obtained in the design codes.

*Doubly supported elements*

The effective width ratio of a compression element is as follows:

$\rho = b_{ef} / b_p$ (1)

with $b_{ef}$ = effective width and $b_p$ = total width,

At the ultimate limit state (see Lecture 9.1):

$\rho = (1 - 0,22/\bar{\lambda}_p)/\bar{\lambda}_p \leq 1$ (2)

and $\bar{\lambda}_p = 1,052(b_p/t)\sqrt{(\sigma_1/Ek_\sigma)} \leq 0,673$ (3)

For $\bar{\lambda}_p$ = 0,673 this formula gives $\rho$ = 1,0, i.e. the element is fully effective.

Corresponding values of $\rho$ and $\bar{\lambda}_p$ are illustrated in Figure 2. The effective width is allocated to both sides of the plate element for constant stress in the subcritical state. If non-uniform stress distributions are present, the total effective width is divided into two parts ($b_{ef1}$ and $b_{ef2}$), depending on the stress ratio $\psi = \sigma_2/\sigma_1$, with $\sigma_1 = f_y$ as the maximum compressive stress and $\sigma_2 \geq 0$. In the area of tensile stresses ($b_t$), the section is always taken as fully effective (see Figure 3).

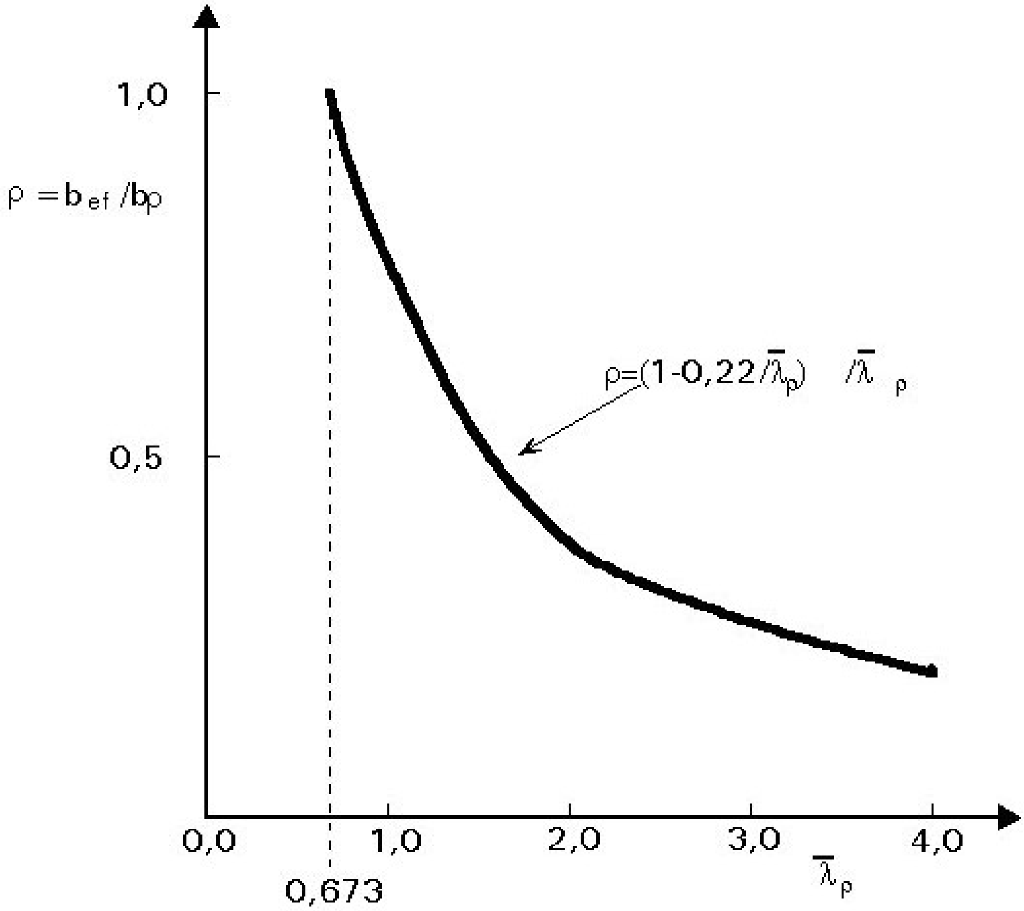



Figure 2 Effective width $b_{ef}$ depending on relative slenderness $\bar{\lambda}$

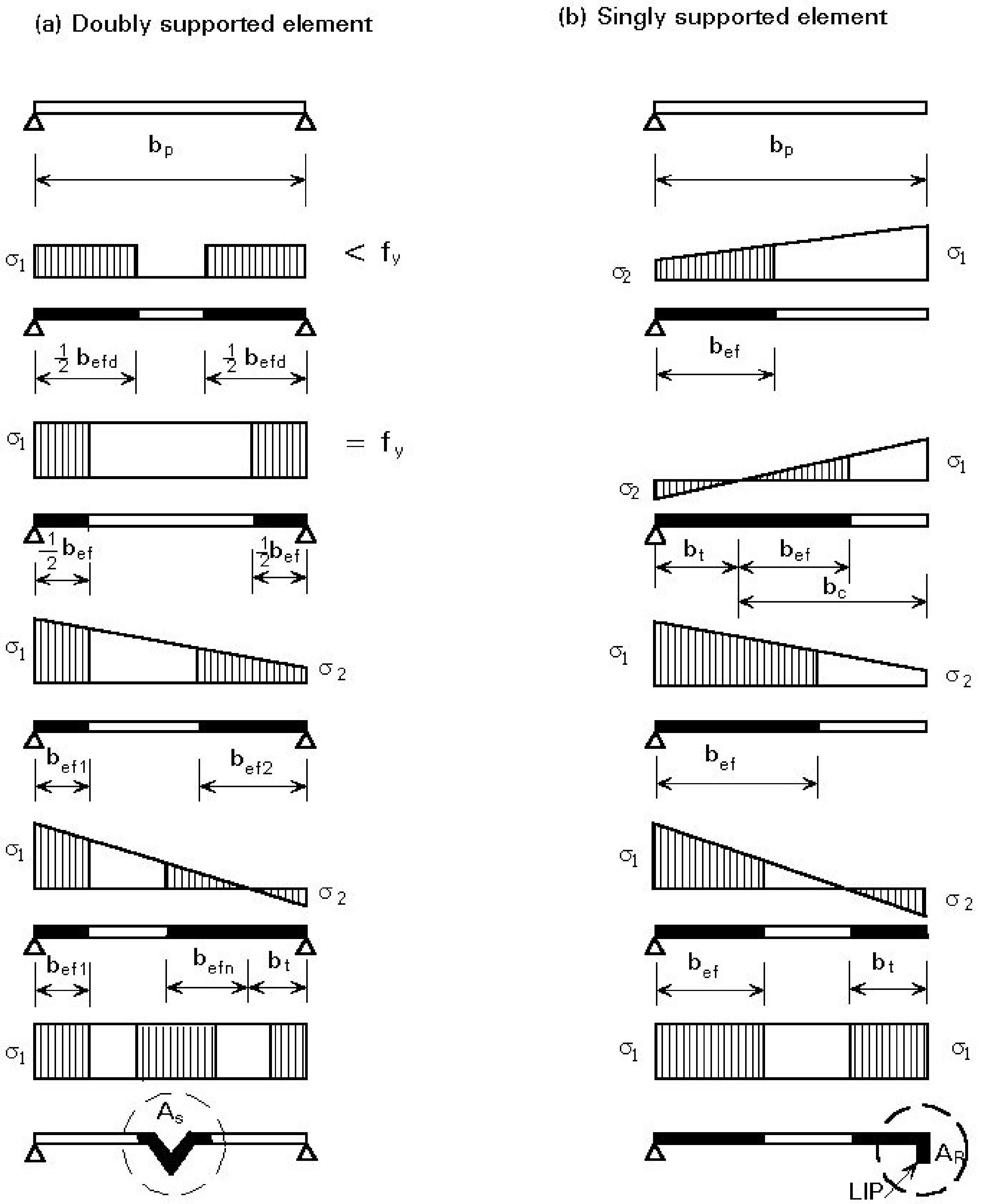



Figure 3 Schematical illustration of the effective width depending on stress distribution and support conditions

At the ultimate limit state, the compressive stress $\sigma_1$ corresponds to the yield strength $f_y$ ($\sigma_1 = f_y$); at serviceability limit state $\sigma_1$ may be taken as equal to $f_y/1.5$.

*Singly supported elements*

For singly supported elements similar solutions can be derived, using appropriate $k\sigma$ values. In this case, however, when calculating the effective width it is important to note whether the maximum compressive stress is located at the supported or unsupported side of the element (Figure 3).

*Validity of the effective width concept*

Comparisons between test results and analytically derived buckling loads of C-shaped sections (Figure 4) and other profiles with different element boundary conditions, have confirmed the practical validity of the design model. One advantage of the effective width concept is that it allows relatively simple methods to be used; it also permits the effect of the section geometry on load bearing resistance to be visualised. This effect can be seen from the values of effective widths referred to different stress distributions and support conditions shown in Figure 3. A practical consequence is that unsupported parts of elements subjected to compressive stresses are ineffective and should be avoided. Their effectiveness can be easily increased by reinforcement of the section by edge stiffeners (lips, bends, folds) and/or intermediate stiffeners. This effect is also qualitatively illustrated in Figure 3.

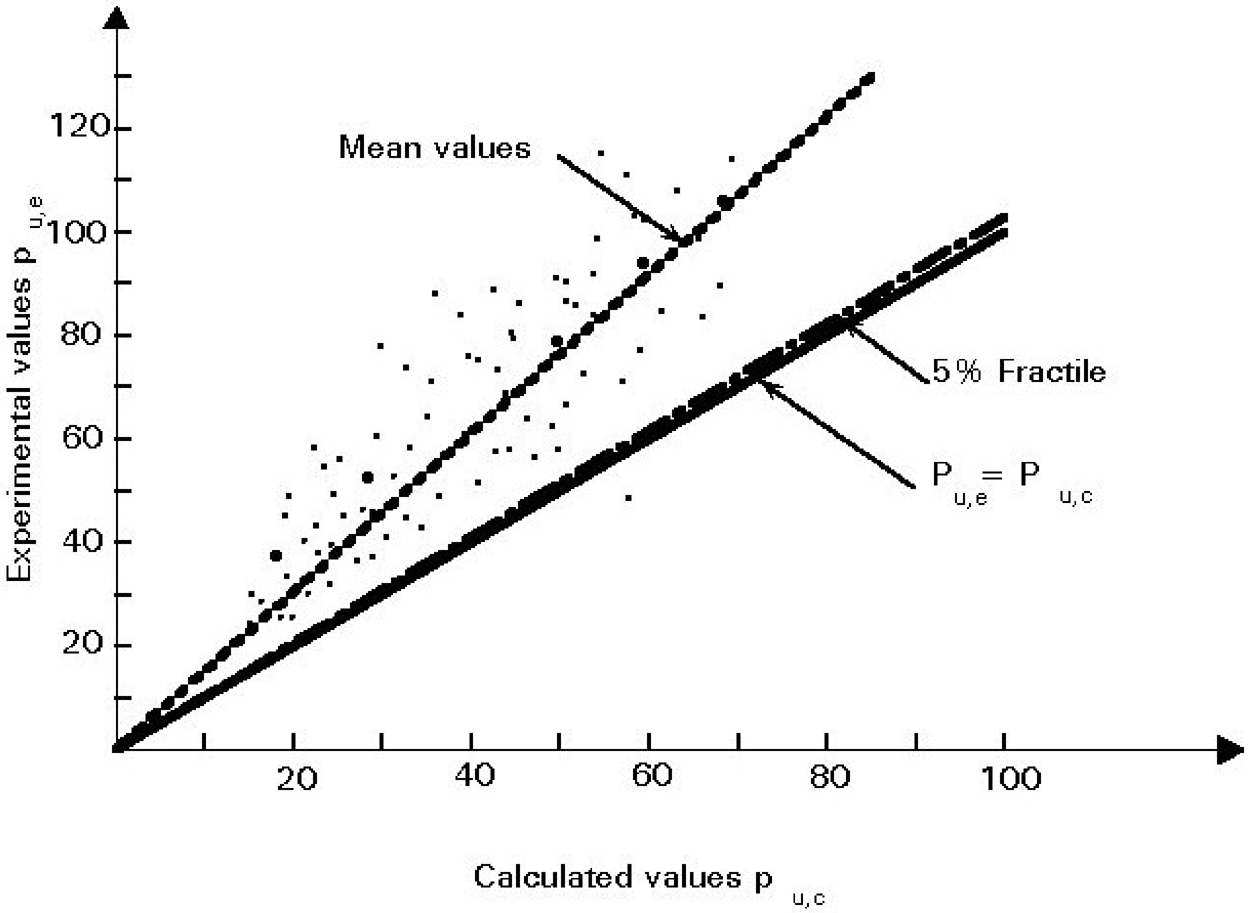
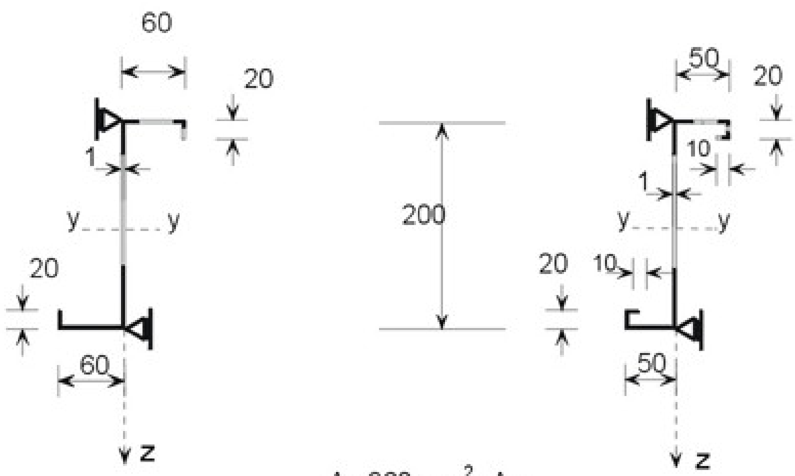



Figure 4 Comparison between test results ($p_{u,e}$) and calculated values ($p_{u,c}$) of ultimate load of c - shaped sections

It can also be seen from Figure 5, which shows the load-bearing resistance of Z-shaped profiles with different types of end stiffeners under bending moments and normal forces respectively, that even small changes of geometrical properties provide increased load-bearing resistances.




$A_1 = 360\ mm^2 = A_2$

| | |
|---|---|
| $M_{y1} = 4{,}15$ kNM | $M_{y2} = 4{,}81$ kNM $= 1{,}16 M_{y1} = 0{,}71 M_{yg}$ |
| $M_{z1} = 1{,}16$ kNM | $M_{z2} = 1{,}32$ kNM $= 1{,}14 M_{z1} = 0{,}92 M_{zg}$ |
| $N_{uz1} = 42{,}1$ kN | $N_{uz2} = 46{,}3$ kN $= 1{,}1 N_{uz1} = 0{,}9 N_{uzg}$ |

(Buckling length = 2500 mm)

Figure 5 Effective width and load bearing resistance of z-sections under bending moments or normal forces ($f_y = 320\ N/mm^2$)

### Treatment of stiffeners and lips

An effective measure to increase the load-bearing resistance and stiffness of thin-walled sections, is to reduce the flat width of elements of a section in compression by intermediate stiffeners, and to provide singly-supported flat parts with edge stiffeners (bends or folds). If the stiffness of the stiffener itself is sufficiently high, it can act as a rigid support to adjacent flat parts (see also Fig. 14 of Lecture 9.1). This means that no collapse of the stiffener, caused either by yielding or instability of the stiffener itself, is allowed to occur before the supported element is itself at the ultimate state. Normally it is impossible to provide such an amount of stiffness which means that an interaction between the adjacent element and the stiffener has to be considered.

Since the analytical solution to the problem is very difficult and impractictical, an approximate solution has been developed based on the component's physical behaviour. In Figure 6, three different buckling modes are illustrated, which represent the following:

- a singly supported strip where a large wave length and free development of the buckling amplitude at the unstiffened side of the strip are expected (Figure 6a).
- the local buckling mode of a doubly supported plate strip where the junction between the strip and the lip remains straight, but where the lip follows the buckling mode (Figure 6b).
- an interaction between the buckling behaviour of the strip and the lip, resulting in a lateral-torsional buckling mode of the lip and adjacent parts of the strip at a wave length which depends on the stiffness of the lip, the b/t ratio and the

restraint to the strip (Figure 6c).

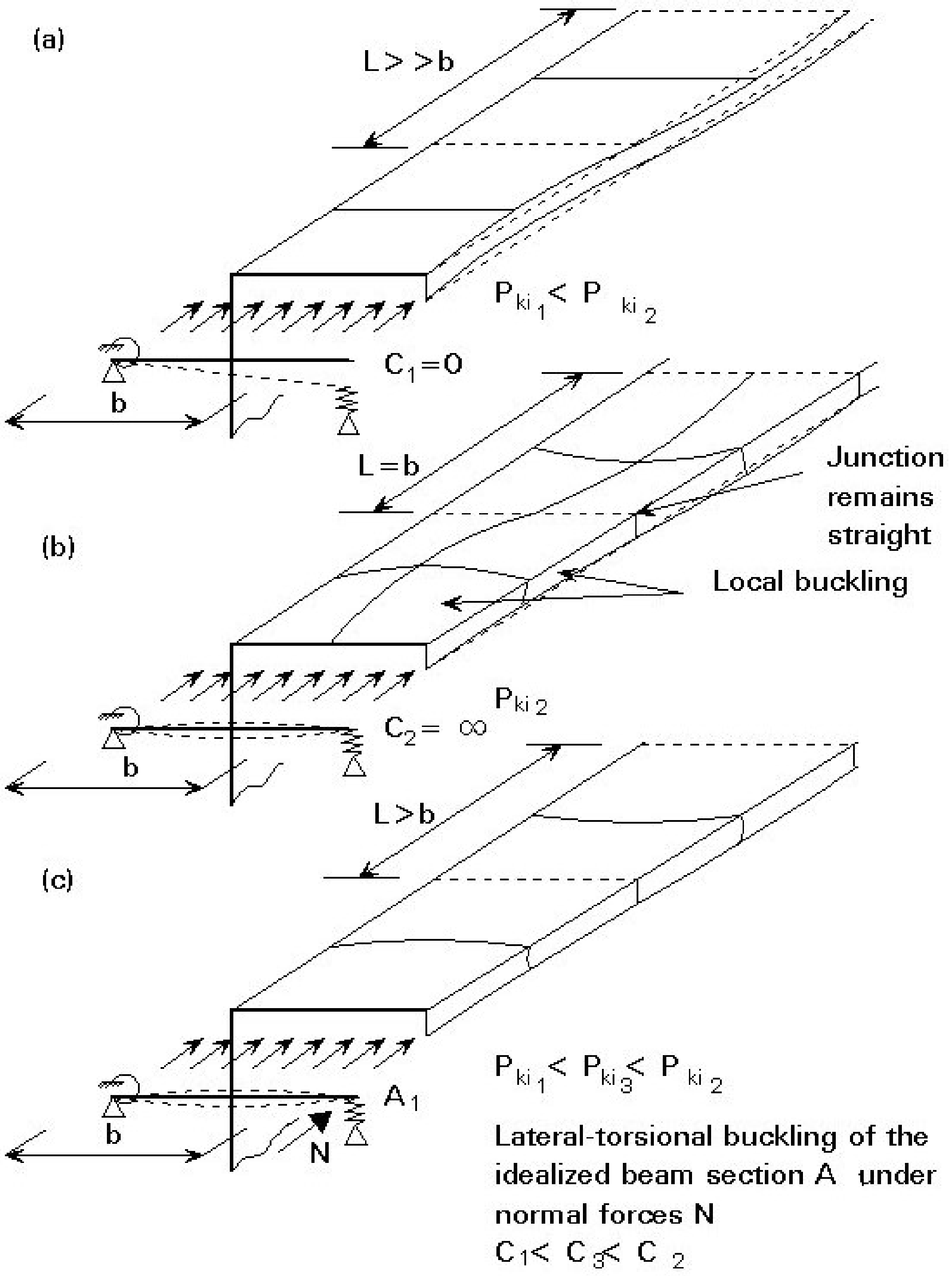
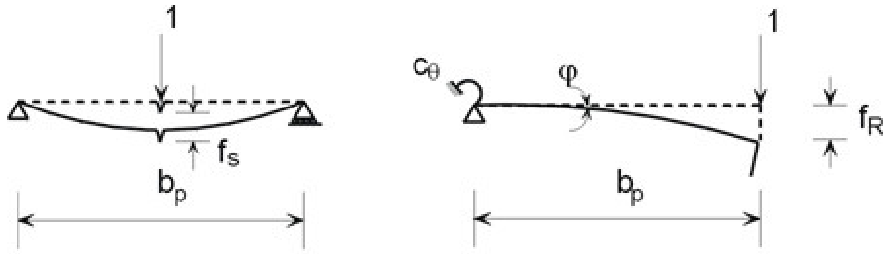



Figure 6 Physical behaviour of a lipped flange under compression and the design model

This behaviour can be simulated by the "beam on elastic foundation" model in which the beam is represented by parts of the lip and the strip, and the elastic foundation by a spring stiffness which represents the restraint to the strip.

*Simplified design of stiffeners*

Based on the physical behaviour described above, the design model requires the estimation of an effective section, and the spring stiffness of the "foundation". Then, the ideal critical buckling load of the section ($N_{cr}$) and the reduced ultimate load ($N_u$), depending on the relative slenderness, can be determined. The spring stiffness of an intermediate stiffener mainly depends on the $b_p/t$ ratio of the compressed element and that of an edge stiffener (e.g lips or foldings) on the amount of restraint at the opposite side of the strip. The determination of the spring stiffness is demonstrated in Figure 7.




$$f_s = \frac{\alpha(1-\mu^2)b_p^3}{Et^3}$$

With $\alpha$ =5,taking into account a restraint of the plate strip; otherwise $\alpha$ =4

$$c_s = \frac{1}{f_s}$$

$$f_R = \frac{4(1-\mu^2)b_p^3}{Et^3} + \varphi b_p$$

With $\varphi$=1/$c_\theta$=angle due to the amount of restraint at the support

$$c_R = \frac{1}{f_R}$$

Figure 7 Determination of the spring stiffness of an intermediate and end stiffener respectively

The procedure for the determination of the load-bearing resistance of the compression flange of a Z section is illustrated in Figure 8 where the steps are as follows:

Step 1 The spring stiffness $C_R = 1/f_R$ is determined, taking into account the rotational stiffness at the support due to the adjacent web.

Step 2 Determination of the effective width of the plate element and the lip respectively, assuming a hinged support at the junction

$\Rightarrow \Sigma A_{ef} = b_{ef,1}.t + A_R$ [= $(b_{ef,1} + C_{ef,1}).t$].

Step 3 Having calculated $I_R$, the moment of inertia of the cross-section with area $A_R$ (referred to the axis a-a of $A_R$), the ideal buckling stress $\sigma_{ki,R}$ is given by:

$\sigma_{ki,R} = (2/A_R)\sqrt{(C_R EI_R)}$

representing the bifurcation stress of the beam on elastic foundation.

Step 4 Determination of the related slenderness $\bar{\lambda} = \sqrt{(f_y/\sigma_{ki,R})}$ and evaluation of the reduction factor from a buckling curve (normally curve b), which gives $\sigma_k = \kappa.f$ and the load-bearing resistance of the "beam section" $N_{u,2} = \kappa.f_y.A_R$.

Step 5 or, referred to the yield stress, $N_{u,2} = f_y(\kappa.A_R)$ which means that $A_R$ has to be reduced to a value of $A_{ef,2} = \kappa.A_R$ (equivalent section).

If $\kappa$ is substantially less than 1,0, an iterative process with at least two steps (6 and 7) can improve the load-bearing resistance so that at the end of the iteration $\kappa \approx 1{,}0$ and $N_{u,n} = f_y.A_{ef,}{}^{*}{}_2$. The total load-bearing resistance is then $\Sigma N_u = f_y\,(A_{ef,1} + A_{ef,}{}^{*}{}_2)$.

The effect of an intermediate stiffener can be determined in a similar way. The validity of this model has been confirmed by tests.

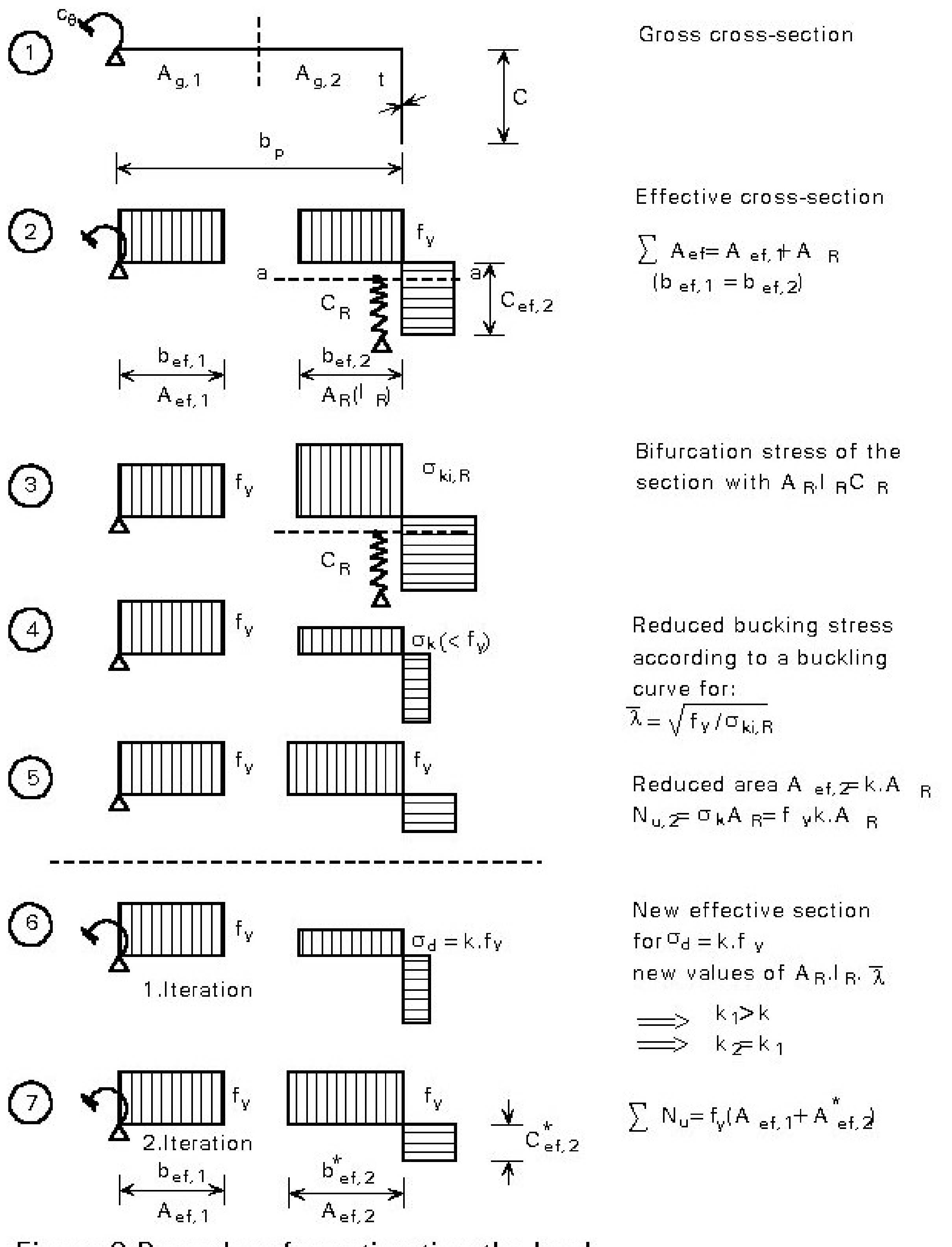



Figure 8 Procedure for estimating the load bearing resistance of a compressed flange of a z-section

### Other considerations

In preparing the design procedure, the enhancement of the yield strength caused by the cold-forming process (see Lecture 9.1), can be taken into account, bearing in mind that roundings of corners (radii) have to be considered in the evaluation of section properties.

# 3. DESIGN OF AXIALLY LOADED COLUMNS

The design procedure for axially loaded thin-walled columns mainly follows the procedure for compact sections, that is: choice of the buckling curve (a-c) with reference to the type of the section; calculation of the section properties ($I_{ef}$,$A_{ef}$) and the slenderness ($\lambda$) of the columns; derivation of the related slenderness, f($\lambda$, $f_y$); and estimation of the buckling factor $\alpha$ and the design buckling load $N_d$. For this procedure the following aspects must be considered:

### Buckling curves and types of sections

Types of sections and related buckling curves (a-c), represented here by imperfection factors $\alpha$ = 0,21 - 0,34 - 0,49, are shown in Figure 9. The more the section is prone to local buckling or to twisting the more the $\alpha$-values increase and the buckling reduction factors decrease. This fact underlines the need to consider in the design the actual type of loading the section undergoes.

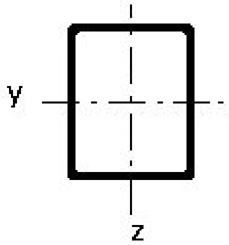
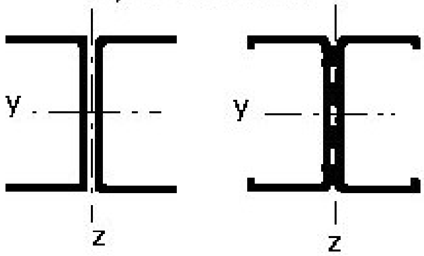
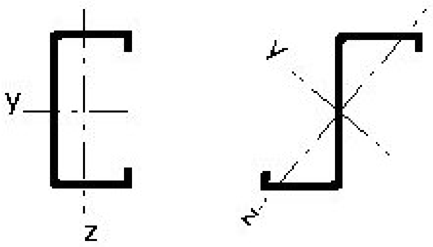
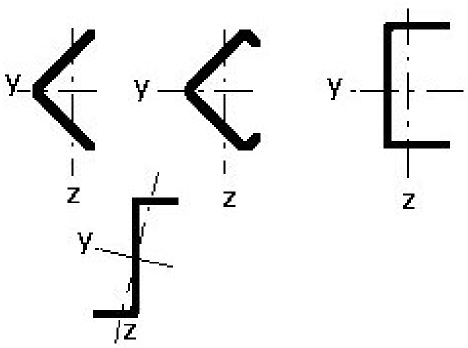
| Type of cross section | | Buckling about | |
|---|---|---|---|
| | | y - y | z - z |
| Hollow sections  | If $f_{yb}$ is used | $\alpha = 0{,}34$ | $\alpha = 0{,}34$ |
| | If $f_{ya}$ is used | $\alpha = 0{,}49$ | $\alpha = 0{,}49$ |
| I,H -Sections  | | $\alpha = 0{,}21$ | $\alpha = 0{,}34$ |
|  | | $\alpha = 0{,}34$ | $\alpha = 0{,}34$ |
|  | | $\alpha = 0{,}49$ | $\alpha = 0{,}49$ |

Figure 9 Types of sections and related buckling curves,expressed by imperfection factors $\alpha$

## Buckling of symmetrical sections

The effective cross-section is calculated on the assumption of constant compressive stresses, acting on the gross cross-section. For symmetrical sections the neutral axis of the effective section is identical with that of the gross cross- section and the member has to be checked for pure compression forces only.

The procedure is then as follows:

- Determination of the necessary section properties (see Figure 10):

  $A_g$, $A_{ef}$, $Q = A_{ef}/A_g$

  $I_{ef}, i_{ef} = \sqrt{(I_{ef}/A_{ef})}$, $\lambda = L/i_{ef}$ referred to the appropriate axis (y,z)

- Determination of related slenderness values:

  $\lambda_1 = \sqrt{(E/f_y)}$

  $\lambda/\lambda_1 = \bar{\lambda}$

- Choice of buckling curve, depending on the type of section:

  $\alpha$-value according to Figure 9.

- Calculation of the curve parameter:

  $\phi = 0,5[1 + \alpha(\bar{\lambda} - 0,2) + \bar{\lambda}^2]$

- Determination of the reduction factor:

  $\kappa = 1/\{\phi + [\phi^2 - \bar{\lambda}^2]^{1/2}\} < 1$ (5)

- Determination of the design value for the buckling load:

  $N_{Rd} = \kappa A_{ef} f_y/\gamma_M$ (6)

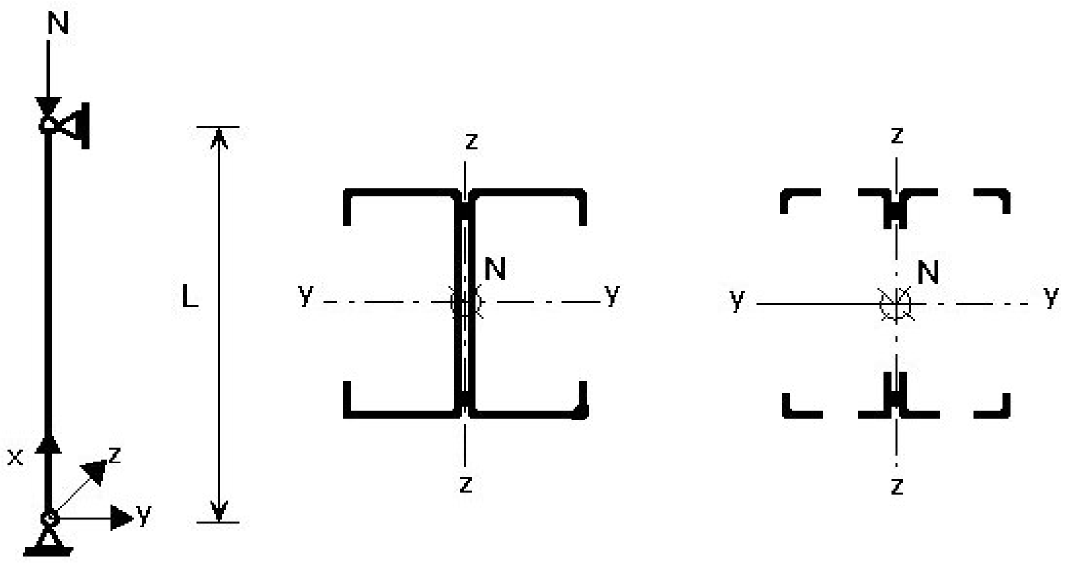



Figure 10 Axial compressed member;symmetrical gross and effective cross-section

**Buckling of unsymmetrical sections**

For unsymmetrical sections (see Figures 11 and 12), the neutral axis of the effective section (if $A_{ef}/A_g < 1$) shifts with respect to that of the gross cross-section. Since concentric compression is defined as the normal force acting at the centroid of the effective section, this case will be only valid if the load is made concentric by constructional arrangements.

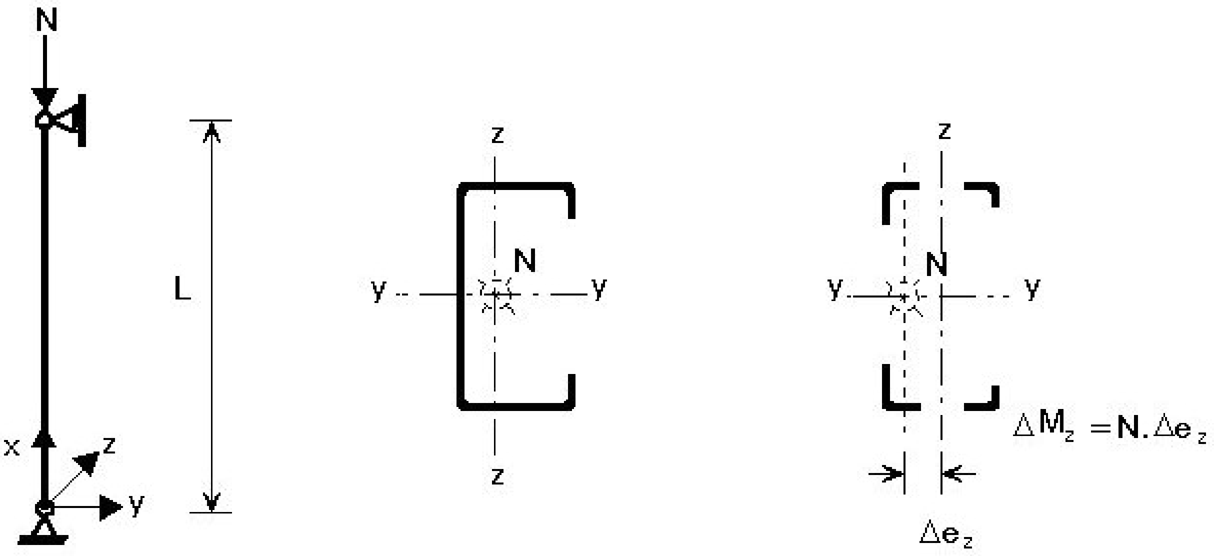



Figure 11 Shift of neutral axis z-z,causing an additional bending moment $\Delta M_z$ ; axial compression with respect to axis y-y

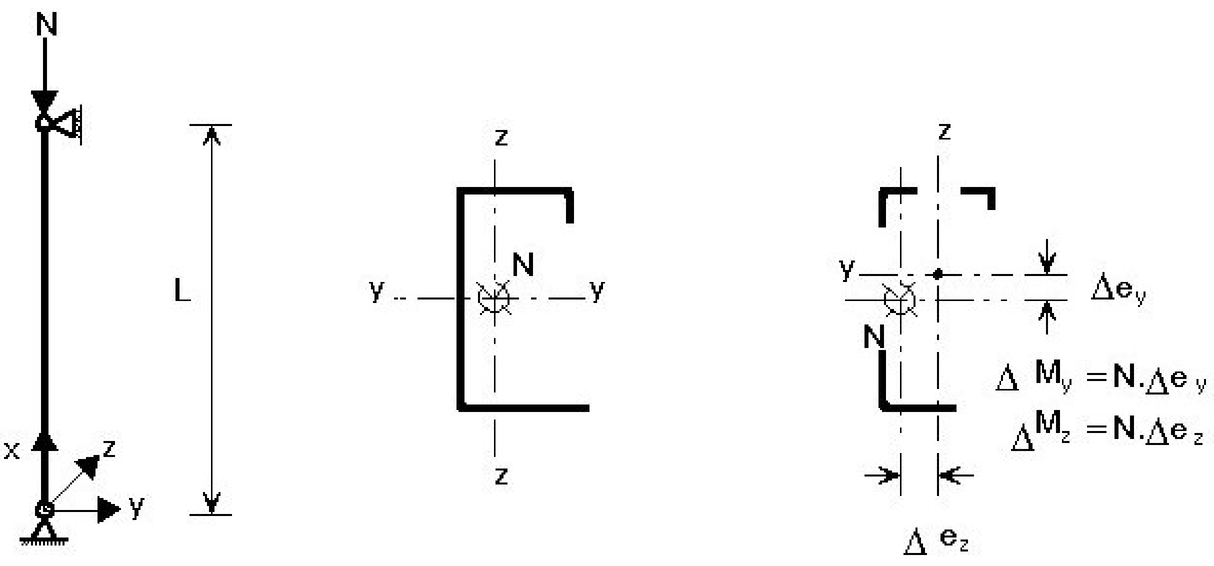



Figure 12 Shift of neutral axis y-y and z-z, causing additional bending moments $\Delta M_y$ and $\Delta M_z$

Normally the shift of the neutral axis will produce an additional bending moment, $M_b$ = N.e, which has to be taken into account in the same way as flexural buckling. The additional moment caused by an interaction of normal forces and external bending moments. In general, all members subjected to combined bending and axial compression must satisfy the following conditions:

$$\frac{N}{N_{Rd,min}} + \frac{M_y + \Delta M_y}{M_{Rd,y}} \cdot (k_y \cdot k_{LT}) + \frac{M_z + \Delta M_z}{M_{Rd,z}} \cdot k_z \leq 1 \quad (7)$$

where:

$\Delta M_y$, $\Delta M_z$ are the additional bending moments due to the shift of the neutral axis.

$M_y$, $M_z$ are the nominal external bending moments according to first order theory.

$M_{Rd,y}$, $M_{Rd,z}$ are the design bending moments referred to the effective cross section.

$k_y$, $k_{LT}$, $k_z$ are enhancement factors to cover second order effects.

Equation (7) covers the case of combined bending, and axial compression with lateral-torsion buckling if $M_{d,y}$ is the design bending moment considering lateral- torsional buckling, and if $k_{LT}$ is the appropriate enhancement factor.

# 4. CONCLUDING SUMMARY

- The design of thin-walled members in compression must take into account the flexibility of such elements by means of an effective width approach leading to a reduction of the total area used to calculate load resistance.
- The load-bearing resistance can be increased by the provision of lips and stiffeners.
- The design of axially loaded thin-walled columns takes into account the general form of the effective area of the section (symmetric or non-symmetric).
- Symmetric sections are checked for pure compression but non-symmetric sections must be checked for axial compression and bending moments.

# 5. REFERENCES

[1] European Convention for Constructional Steelwork: "European Recommendation for the Design of Light Gauge Steel Members", Publication 49, ECCS, 1987.

[2] Eurocode 3, Part 1.3: "Cold-Formed Steel Sheeting and Members" CEN (in preparation).

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**ESDEP WG 9**

**THIN-WALLED CONSTRUCTION**

# Lecture 9.3 Design Procedures for Beams

**OBJECTIVE/SCOPE**

To present design methods for thin-walled flexural members.

**PREREQUISITES:**

Lecture 6.1: Concepts of Stable and Unstable Elastic Equilibrium

Lecture 7.3: Local Buckling

Lecture 9.1: Thin-walled Members and Sheeting

Lecture 9.2: Design Procedures for Columns

**RELATED LECTURES:**

Lecture 6.2: General Criteria for Elastic Stability

**SUMMARY:**

Design methods for thin-walled flexural members are presented which take into account the different kinds of buckling acting in such members and also shear lag. In particular the design of purlins is discussed.

## 1. INTRODUCTION

Thin-walled flexural members are applied for carrying lateral loads such as gravity loads in roofs for example. Their behaviour can be affected by local buckling, shear lag, web crippling, flange curling and lateral buckling.

The effect of local buckling is covered in design by effective widths of the cross-section based on the stress distribution produced by bending moments and axial force.

For shear lag, web crippling and flange curling design rules are given in Eurocode 3, Part 1.3 [1].

Lateral-torsional buckling has to be evaluated similarly to hot-rolled sections taking the effective cross-section values into account.

Thin-walled cold-formed flexural members have their most important application in wall and roof constructions as purlins. The structural connection with the sheeting provides purlins with lateral and torsional restraints at one flange. This connection gives the purlin high additional stiffness compared to that of a free purlin spanning from frame to frame. The values of torsional restraints have to be measured by testing.

## 2. PREPARATION OF DESIGN PROCEDURES

### 2.1 Resistance Moment $M_{Rd}$

The resistance moment $M_{Rd}$ is the ultimate bending moment of a cross-section with pure bending.

#### 2.1.1 Plastic resistance moment

If flexural members are not subjected to twisting or lateral-torsional buckling the resistance moment $M_{Rd}$ may be found utilizing plastic reserves. By iterative calculations the resistance moment $M_{Rd}$ can be determined under the following conditions:

- At the compression side the effective widths due to the given compressive stress distribution in the iteration step are used. Under particular conditions the compression strain may reach a maximum strain higher than $f_y/E$. Otherwise the compression edge may only reach yield stress $f_y$.
- At the tension side the full cross-section can be utilised. Plastic strain may occur. The tension strains are not limited.

Figure 1 shows an example: Plastic strains occur at the tension side beyond the elastic limit until the compression stresses just reach yield stress, too.

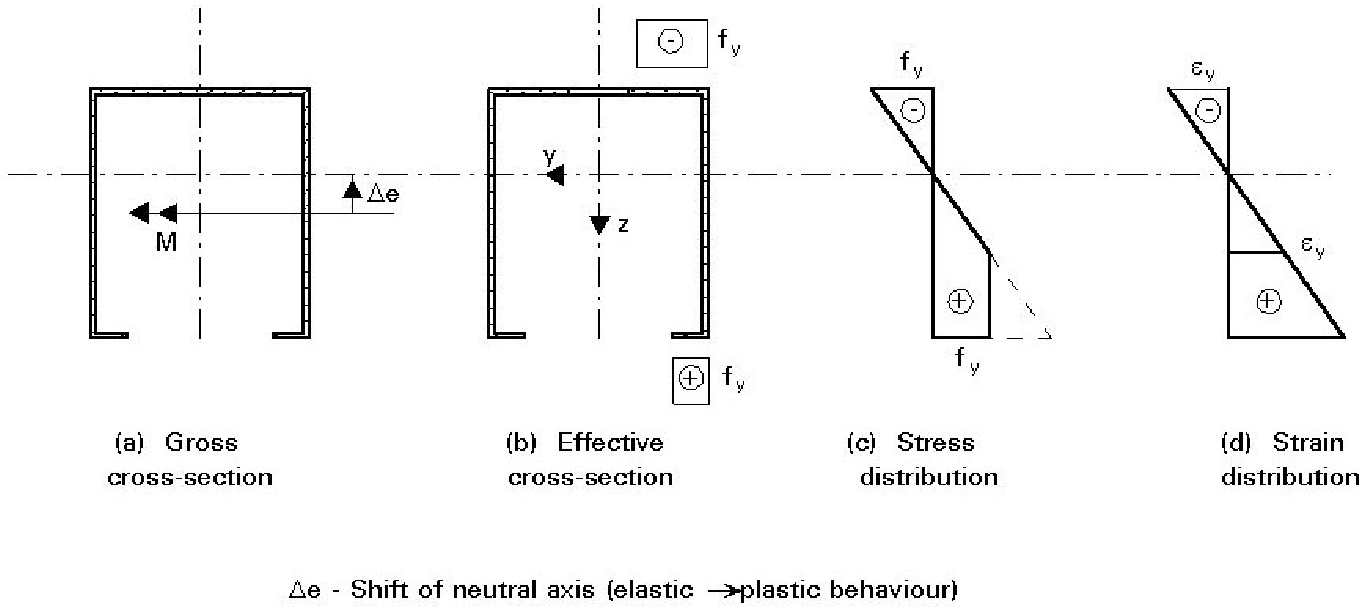



Figure 1 Monosymmetric cross-section with bending moment M : Utilization of plastification in the tension zone

### 2.1.2 $M_{Rd}$ with respect to buckling

The resistance moment $M_{Rd}$ which takes lateral or torsional buckling into account is calculated with the value $k_d$ from the European buckling curves (parameter $\alpha$). This value is a reduction factor for determination of $M_{Rd}$ from the plastic resistance moment $f_y \cdot W_{eff}$ due to the slenderness of the member. The formulae are given below:

$$\bar{\lambda}_{LT}^2 = \frac{W_{eff} \cdot f_y}{M_{cr}}$$

$$\phi_{LT} = 0,5 \cdot [1 + \alpha_d \cdot (\bar{\lambda}_{LT} - 0,2) + \bar{\lambda}_{LT}^2]$$

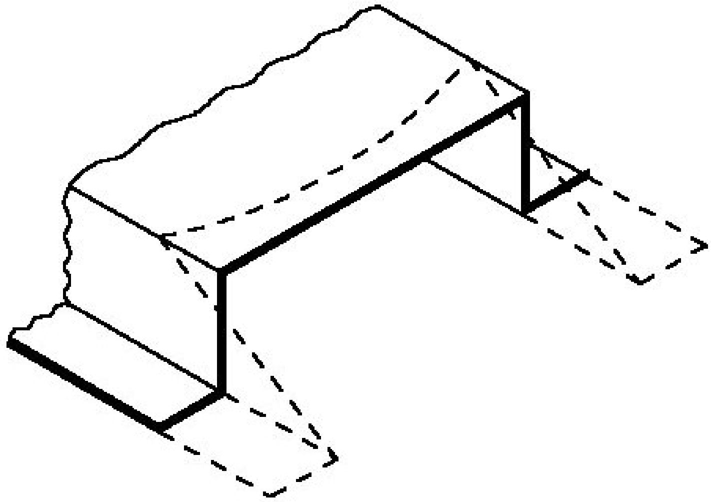
$$\chi_{LT} = \frac{1}{\phi_{LT} + \sqrt{\phi_{LT}^2 - \bar{\lambda}_{LT}^2}} \quad \text{but} \le 1,0$$

$$M_{Rd} = \chi_{LT} \cdot f_y \cdot W_{eff}$$

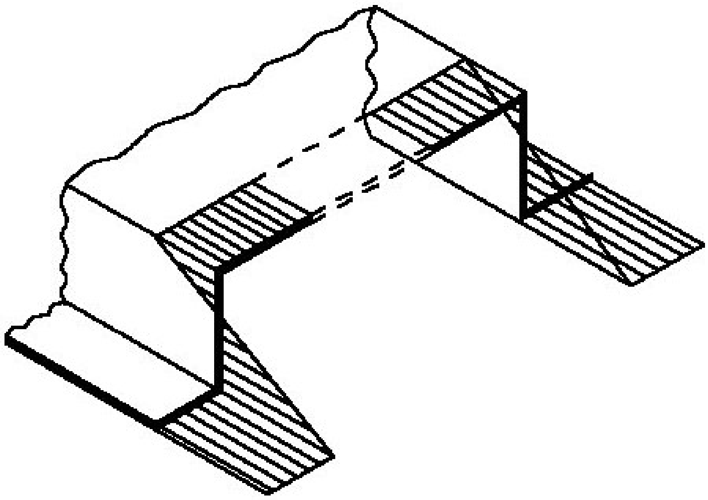
## 2.2 Shear Lag

In wide flanges ($b \geq L/20$) the normal stress distribution due to axial force and bending moment can be affected by shear deformations (see Figure 2). Shear lag appears at locations with large shear stresses, for example at supports. The effective area is calculated similarly to the effective widths due to local buckling by multiplying the gross area with a reduction factor $\psi_s$. The final effective widths which take into account local bucking and shear lag are given by:

$$b_{ef} = \psi_s \cdot \rho \cdot b_p$$



(a) Normal stress distribution across the gross cross-section



(b) Idealised stress distribution across the effective cross-section

Figure 2 Shear lag

Singly supported elements have a further reduction of 15%. The reduction factor $\psi_S$ may be taken from Figure 3 as an approximation.

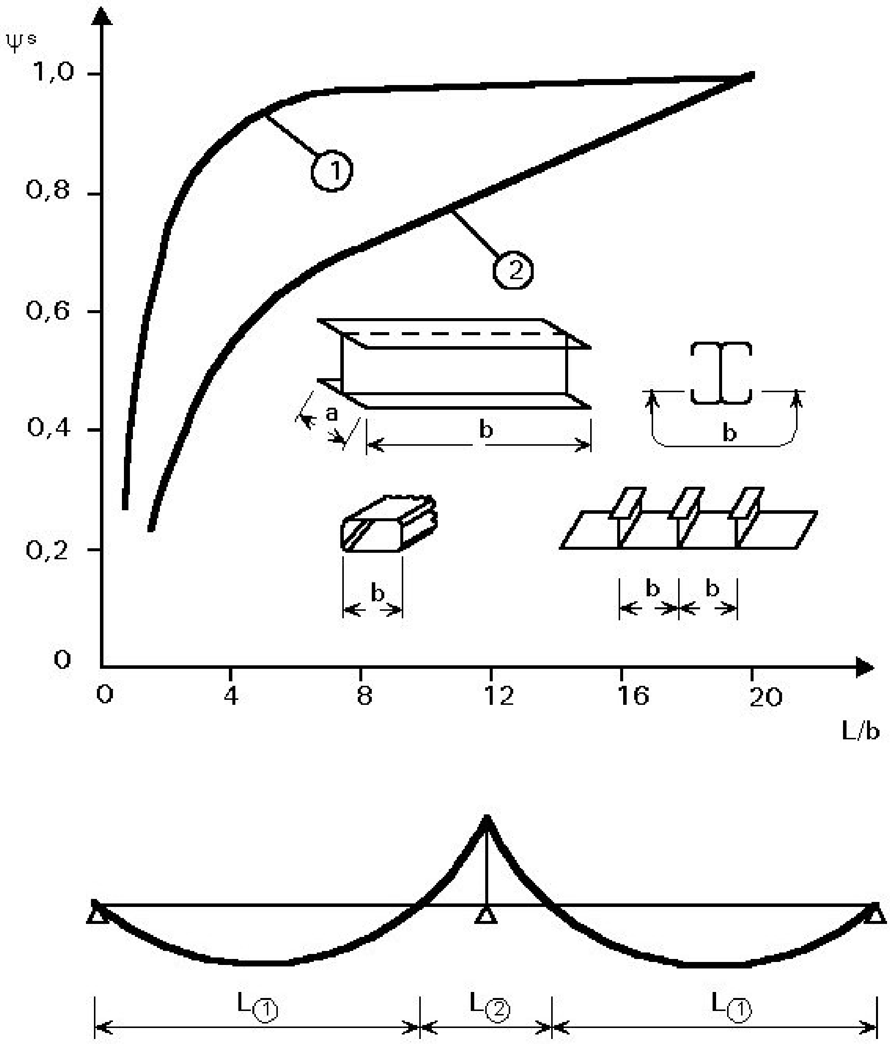



Figure 3 Shear lag reduction factor $\Psi_s$

## 2.3 Flange Curling

Cross-sections with wide flanges or arched profiles subjected to flexure may show an effect which is called "flange curling": The membrane stresses due to the bending moment have to be turned around the curvature of the element or of the deformation which causes lateral loads. Hence the wide flange is bent additionally as shown in Figure 4 and consequently the cross-section stiffness and modulus W decreases. The amplitude of this deformation may be estimated using formulae in Eurocode 3 [1]. The cross-section properties are calculated now on the basis of the cross-section with curved geometry due to flange curling.

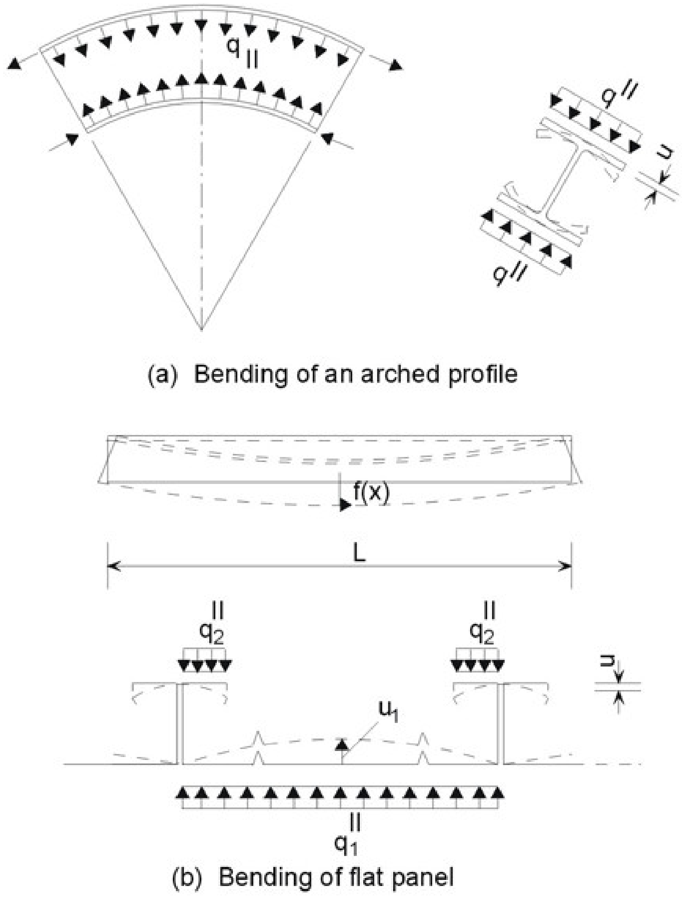



Figure 4 Flange curling:Cross-section deformations by deviation forces $q^{II}$ are proportional to the curvature and the membrane forces in the flanges

Flange curling is a second order effect. In many cases, however, cross-section deformations similar to flange curling occur already due to first order theory. This cross-section distortion may become much bigger than flange curling and should be taken into account using an appropriate theory. For example cassettes without lateral supports at the free flanges show this behaviour.

## 2.4 Lateral and Torsional Restraints

Lateral and torsional restraints may be given to the beam by the adjacent construction. These restraints are found mostly when dealing with purlins which are directly connected to the sheeting of the roof. Figure 5 shows an example.

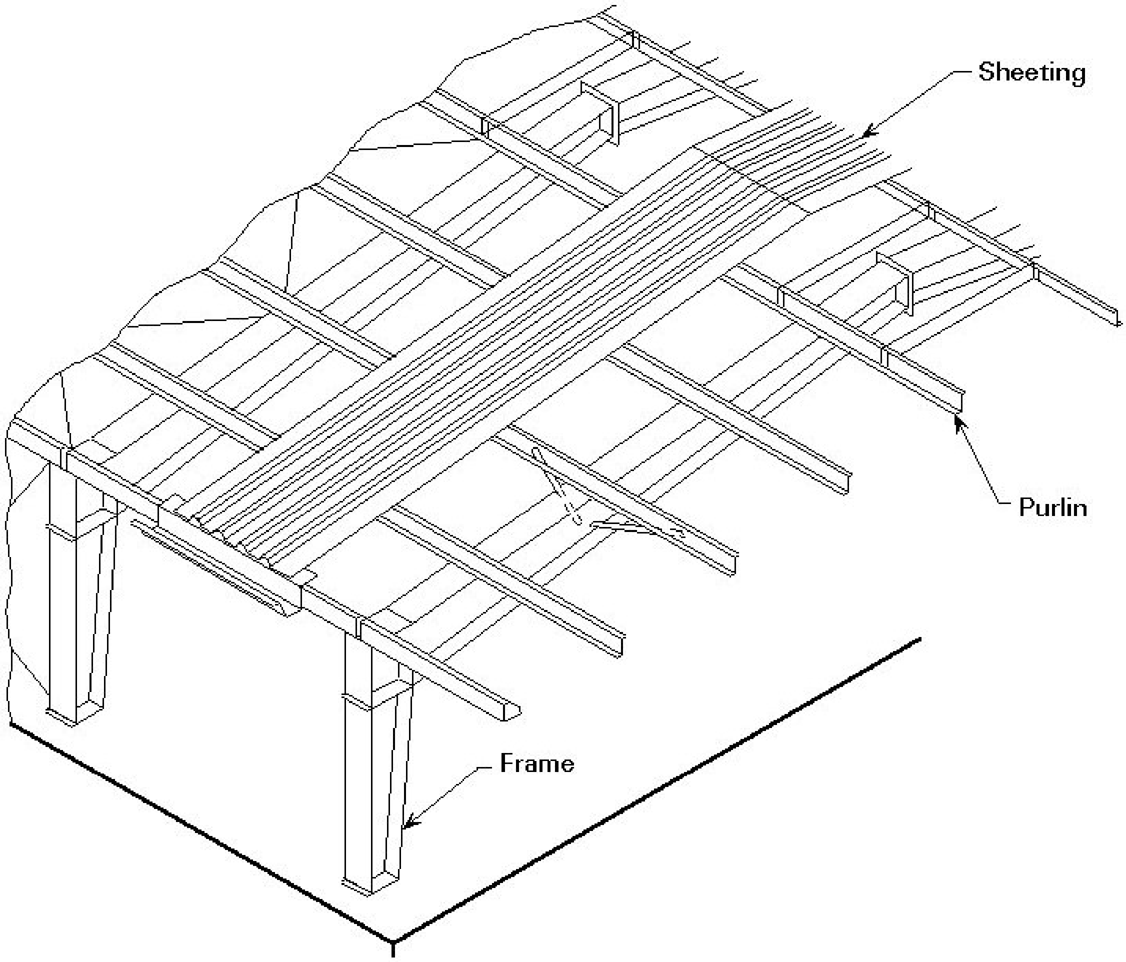
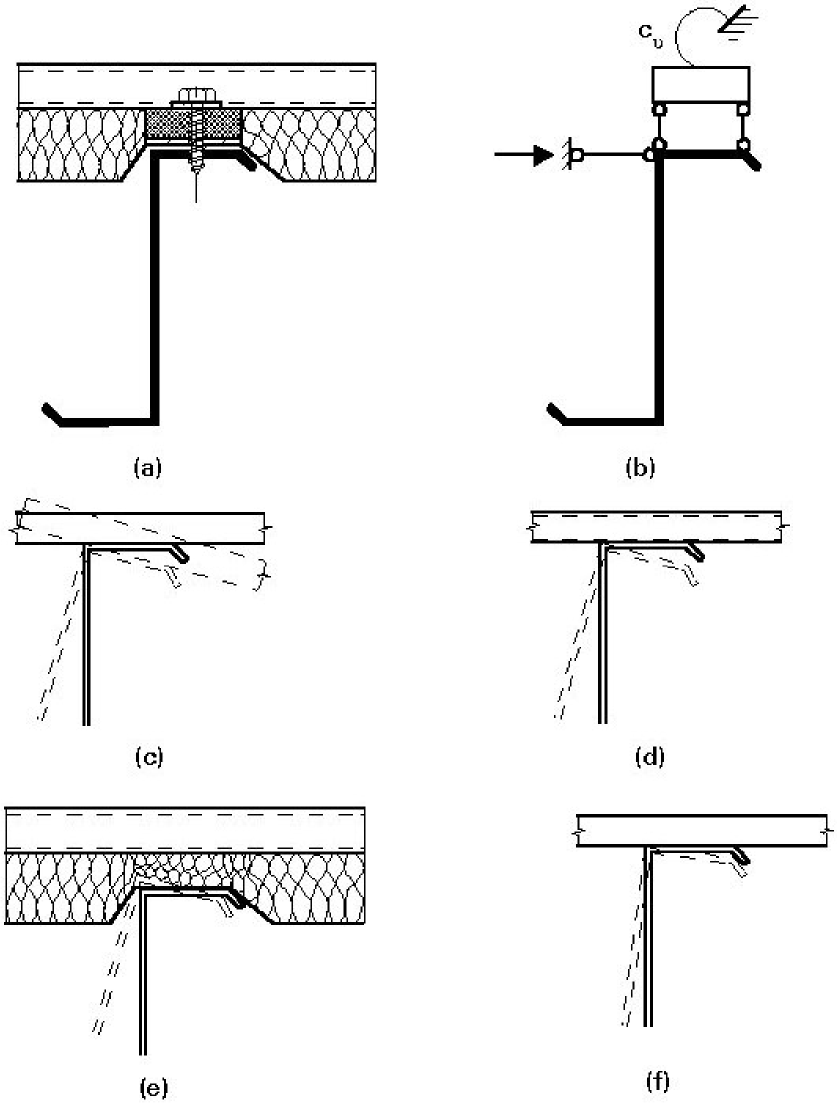



Figure 5 Example for a roof construction : frame-purlin-sheeting

For the design of the beam these restraints may be idealised as a rigid lateral support and a torsional restraint $c_v$ at the flange which is connected to the sheeting (Figure 6).




(a) Real construction
(b) Idealisation
(c) Deformation of sheeting (to be calculated)
(c)-(f) Deformation parts contributing to $c_\upsilon$

(d), (f) Local deformation of sheeting and purlin at connection
(e) Deformation of insulation (d-f to be found by testing)

Figure 6 Torsional restraint $c_\upsilon$

The stiffness $c_\upsilon$ has to be determined partly by testing, partly by calculation. $c_\upsilon$ is a serial combination of the stiffness of sheeting and the local connection. The stiffness of the local connection can be determined with the test setup in Figure 7. Since the deformation $\delta$ involves the web bending, the flange deflection has to be reduced by the cross-section deformation.

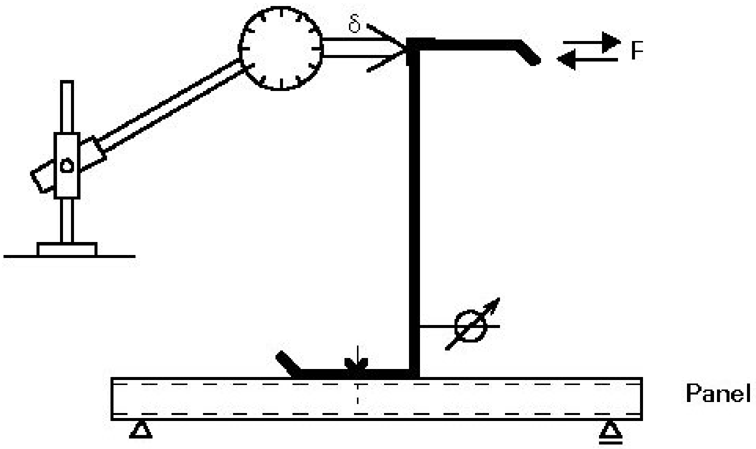
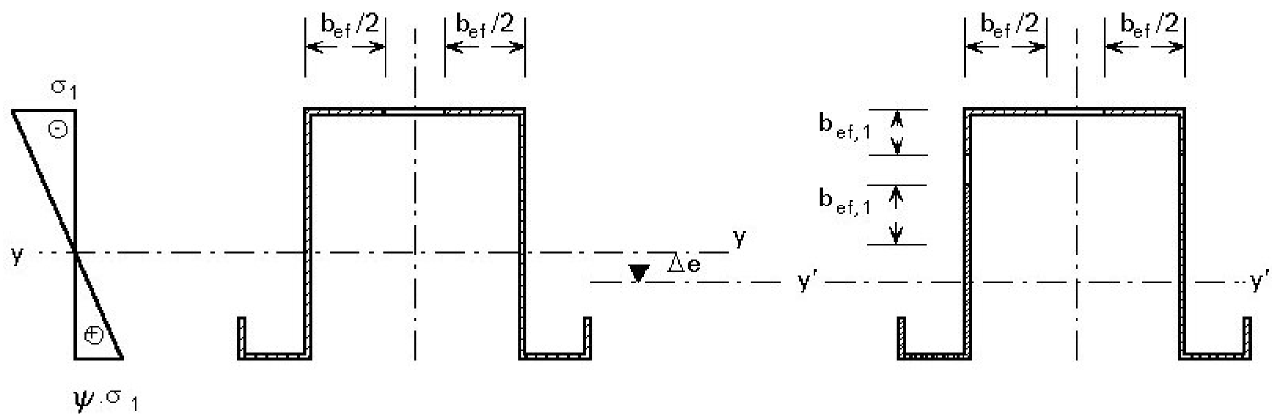



Figure 7 Test set up for determination of the local stiffness of the connection purlin/panel

# 3. DESIGN OF BEAMS

## 3.1 Design of Beams without Lateral-Torsional Buckling

Beams may be stressed by axial force and bending moments. If local buckling has to be considered, effective widths have to be introduced in the cross-section. As a consequence the neutral axis determined on the basis of the gross cross-section may shift (Figure 8). The additional bending moment $\Delta M = N \cdot \Delta e$ has to be considered.




Location of the neutral axis y-y preliminary location
y'-y' final location
Δe shift of neutral axis

Figure 8 Effective cross-section of flexural member

Lateral-torsional bucking can be disregarded if torsion of the cross-section is prevented by the construction.

The design rule below adds separately the different stress resultants related to the yielding load under one stress resultant only. If lateral displacements of the cross-section are not prevented the yielding axial force takes flexure buckling due to both axes into account. The lower value has to be taken in the design rule:

$$\frac{N}{N_{Rd,min}} + \frac{M_y + \Delta M_y}{M_{Rd,y}} \cdot k_y + \frac{M_z + \Delta M_z}{M_{Rd,z}} \cdot k_z \leq 1{,}0$$

where $k_y$ and $k_z$ are coefficients to take into account the interaction of bending and axial force.

For detailed information, see Eurocode 3 [1].

## 3.2 Design of Beams with Lateral-Torsional Buckling

The design rule for beams where lateral-torsional buckling is not prevented is very similar to the case in Section 3.1.

There are only two differences:

- The resistance moment $M_{Rd,y}$ takes account of lateral-torsional buckling.
- The coefficient $k_y$ changes to $k_{Lt}$ and so respects the interaction between bending and axial force in another form.

For detailed information, see Eurocode 3 [1].

### 3.3 Design for Torsion

If the load is applied eccentrically to the shear centre of a beam, torsional effects have to into account. Thin-walled open cross-sections have very small stiffness in respect to torsion. Hence the load-bearing resistance is reduced substantially by torsion so that torsional moment should be avoided in construction.

If there are torsional moments, the warping stresses in the cross-section have to be considered. (Warping stresses arise as follows. Cross-sections with less than three axes of symmetry will generally deform out-of-the-plane under torsional movements. Where these warping displacements are restrained in some way, a system of longitudinal warping stresses will arise.) In design the superposition of stresses due to axial force, bending moments and torsional moments must remain below the limit of yield stress. Additionally the superposition of shear stresses has to be proved. Figure 9 shows the effect of torsion on the stress distribution.

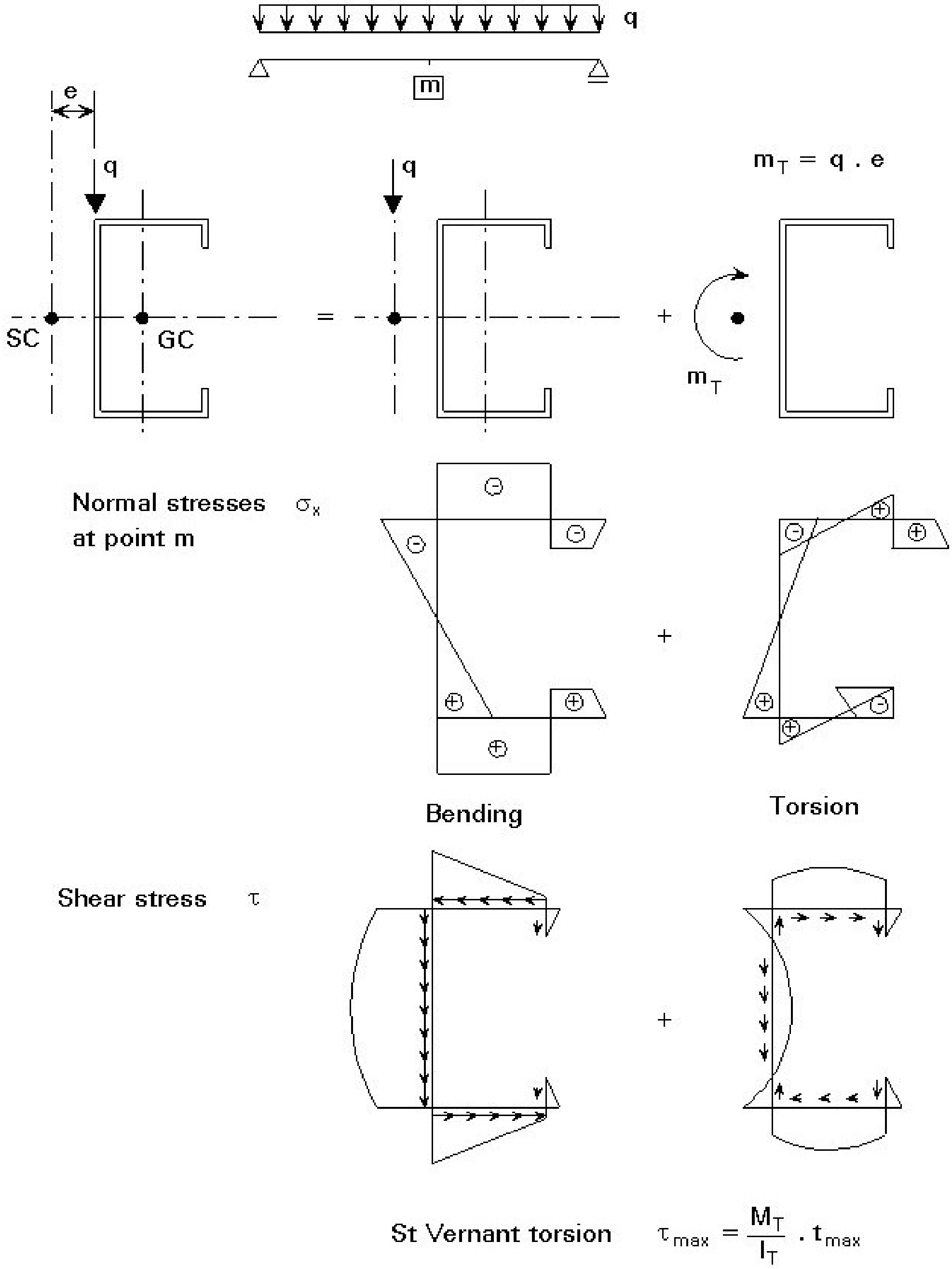



Figure 9 Stress distribution due to bending and torsion

# 4. DESIGN OF PURLINS

## 4.1 Cross-Sections

Purlins represent the major application of cold-formed beams in construction. Several cross-section types have been developed for purlins (Figure 10). The manufacturers' aims are:

- The cross-section shall have full effective widths in the compression zone. This aim is reached by stiffeners or limitation of slenderness.
- Load application shall be as near as possible to the shear centre. For example, sigma-purlins have shear and gravity centres close together and almost directly below the load application point (Figure 11).
- Purlins shall stack easily for transportation. Therefore flanges often have minute differences in widths. At overlaps the connection of two purlins is easily made by taking the second purlin upside down.

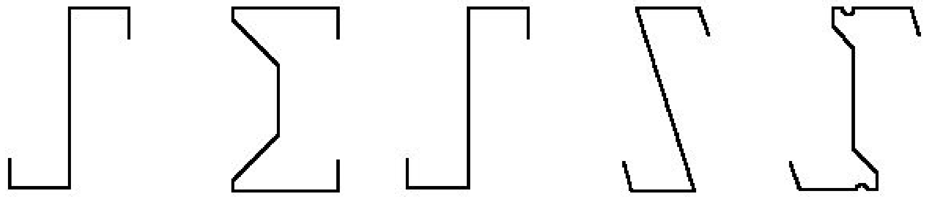


Figure 10 Some types of purlin cross-sections

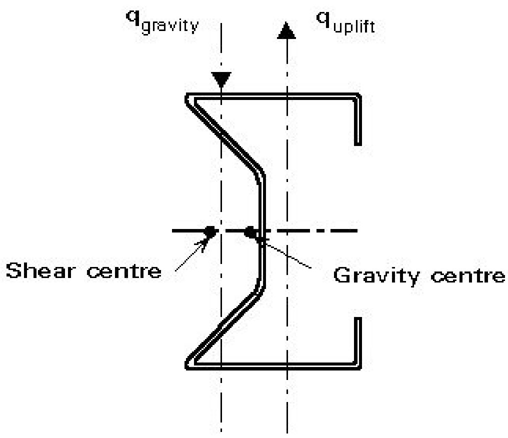



Figure 11 Sigma purlin: shear centre and gravity centre

### 4.2 Purlin Systems

Depending on the supporting construction there are single span purlin systems and multi-span systems. For the continuously connected purlins two systems have been developed (Figure 12):

- In sleeve systems two purlins are connection by a short sleeve element with a fitting cross-section which overlaps both ends of the two purlins.
- In overlap systems one of the purlins overlaps the end of the other purlin and the two purlins are connected directly web to web.

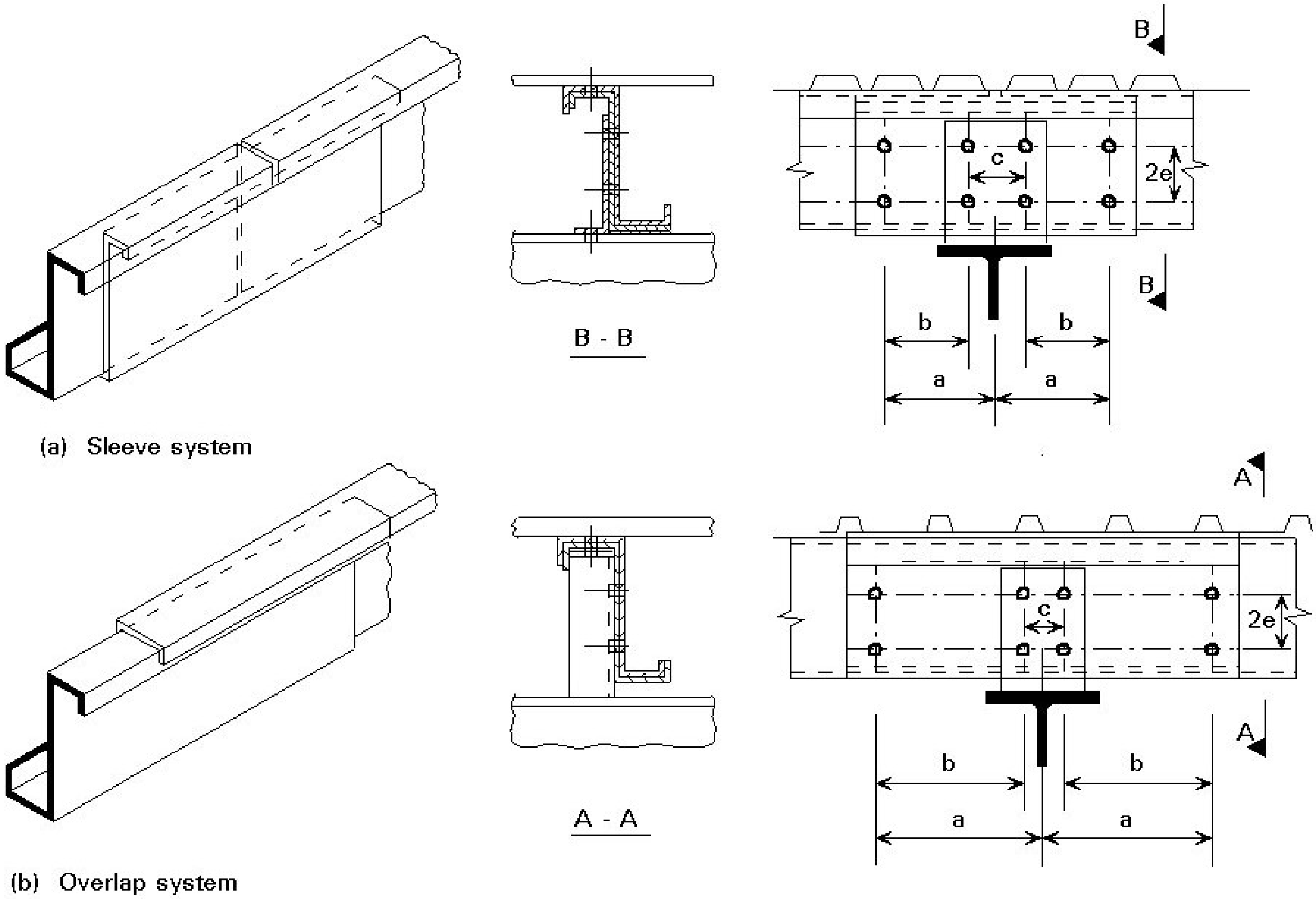



Figure 12 Continuous purlin systems

In both constructions the effect of slip in the screw connectors on the bending moment distribution of the continuous system should be taken into account. Additionally the designer should pay particular attention to the yield load of the screw connections.

### 4.3 Design Models

Since the behaviour of purlins is rather complicated different models have been developed for design. There are two main deformation modes: Bending around the strong axis of the cross-section and torsion. For the calculation of torsion effects and the stability of the free flange two types of design models exist (Figure 13):

- Models which consider the whole cross-section with lateral and torsional restraints and distortion: This model involves the major deformation modes and the correct load factors for bending and torsion but it is necessary to determine warping functions due to torsion.
- Models which consider only the free flange of the purlin as a beam on a lateral elastic foundation: The foundation parameter is given by the torsional restraint of the upper flange and the cross-section distortion. This model is helpful but the major difficulty with it is the definition of which part of the web belongs to the flange. The model is sensitive to this factor.

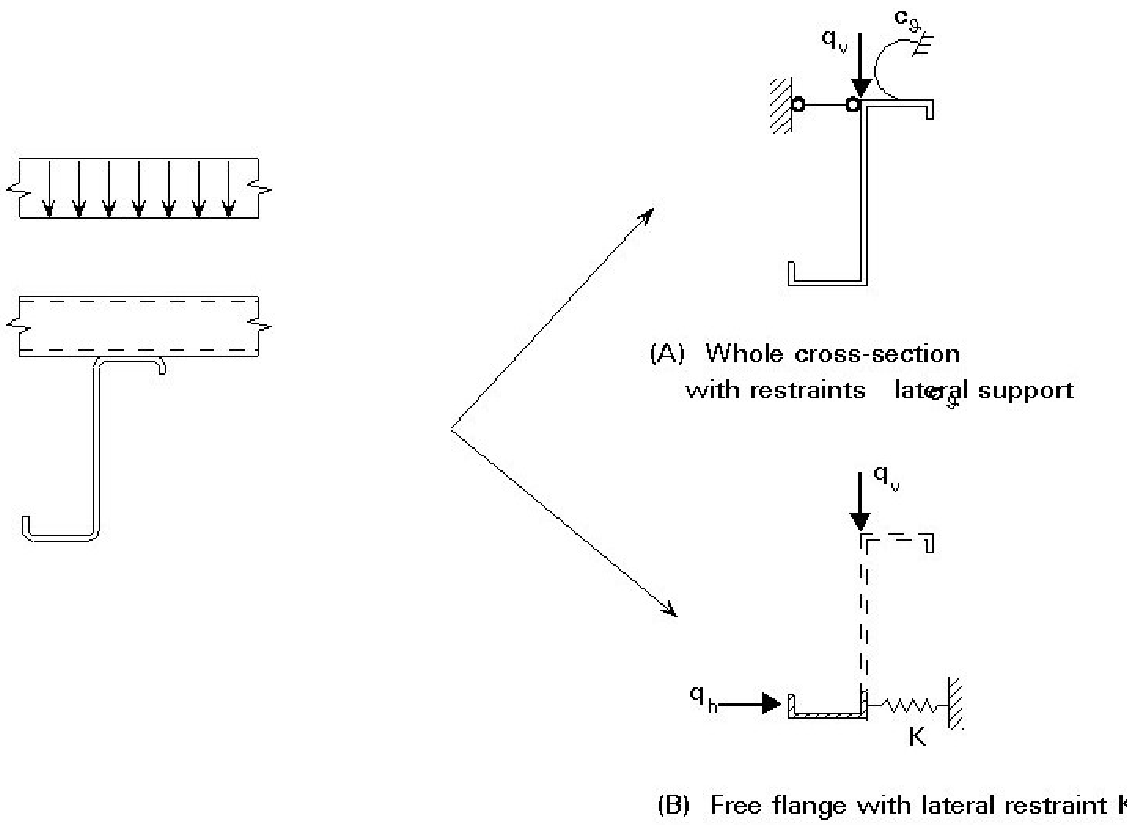



Figure 13 Two design models for the free purlin flange

A model due to the second type has become part of Eurocode 3 Part 1.3 [1]. The stresses due to bending around the strong axis and axial force are determined with the whole cross-section and effective widths. Additional stresses arise because of bending of the free flange around the vertical axis. These stresses are calculated using the system shown in Figure 14. The flange is embedded on elastic foundation K. The foundation modulus can be found using Figure 15. It depends on the torsional restraint at the upper flange and the distortion of the cross-section.

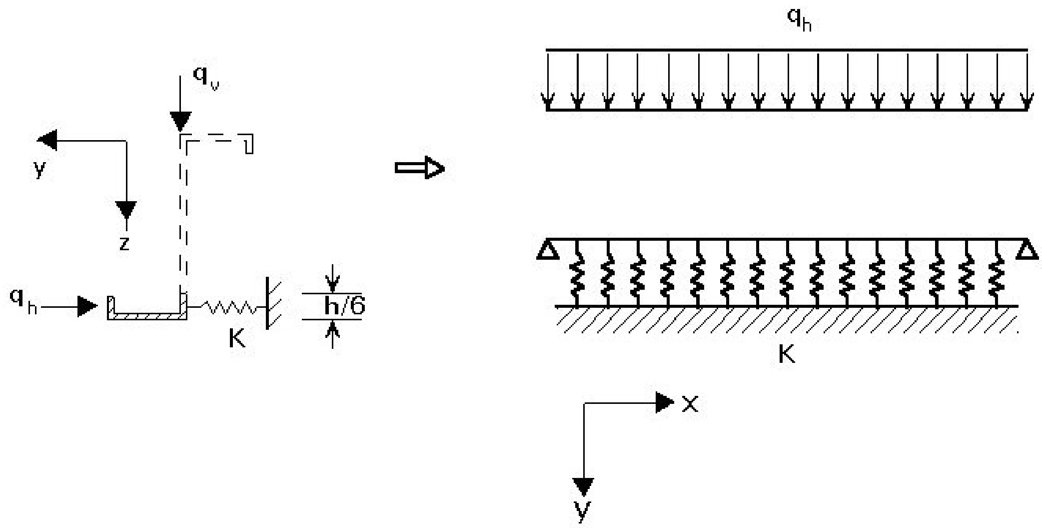
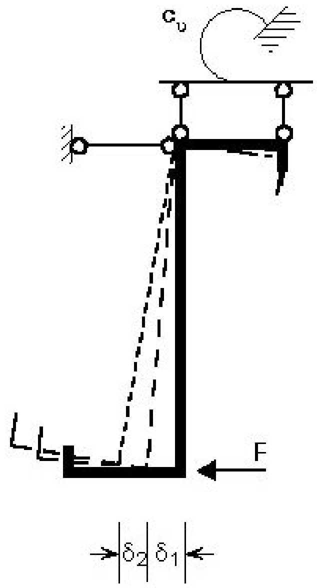



Figure 14 Flange bending of a single span purlin




$$K = \frac{F}{\delta_1 + \delta_2}$$

Figure 15 Modulus of elastic foundation K

$\delta_1$ – $c_\upsilon$

$\delta_2$ – Distortion

The actual stresses are calculated with the following formulae:

Braced flange:

$$\sigma_x = \frac{M_y}{W_{ef}} + \frac{N}{A_{ef}} \leq f_y$$

Free flange:

$$\sigma_x = \frac{N}{A_{ef}} + \frac{M_y}{W_{ef}} \frac{M_z}{W_{fz}} \leq f_y$$

## 4.4 Stability Check

If the free flange of a purlin is under compression a stability check has to be performed. The free flange of single spanned purlins is compressed in cases where there is wind suction only. The free flange of multispan purlins is compressed in the midspan region in the case of windsuction, whereas the support region is compressed under gravity loads. Wind suction is the more severe loading case in respect of stability.

For the stability check the code [1] proposes a w - rule: The stresses which cause instability are amplified by a w - value in the superposition of stresses.

$$w \cdot \left( \frac{M_y}{W_{ef}} + \frac{N_y}{A_{ef}} \right) + \frac{M_z}{W_{fz}} \leq f_y$$

The w - value depends on the slenderness of the compressed free flange.

## 4.5 Design of Special Purlin Systems

### 4.5.1 Single span systems

Under gravity load without axial force the free flange of the purlin is under tension. Bending of the flange may be disregarded. Design takes into account only bending stresses and support reactions. Deflections are checked.

Under uplift loads the whole free flange is compressed (Figure 16). Flange bending is taken into account and the stability has to be checked.

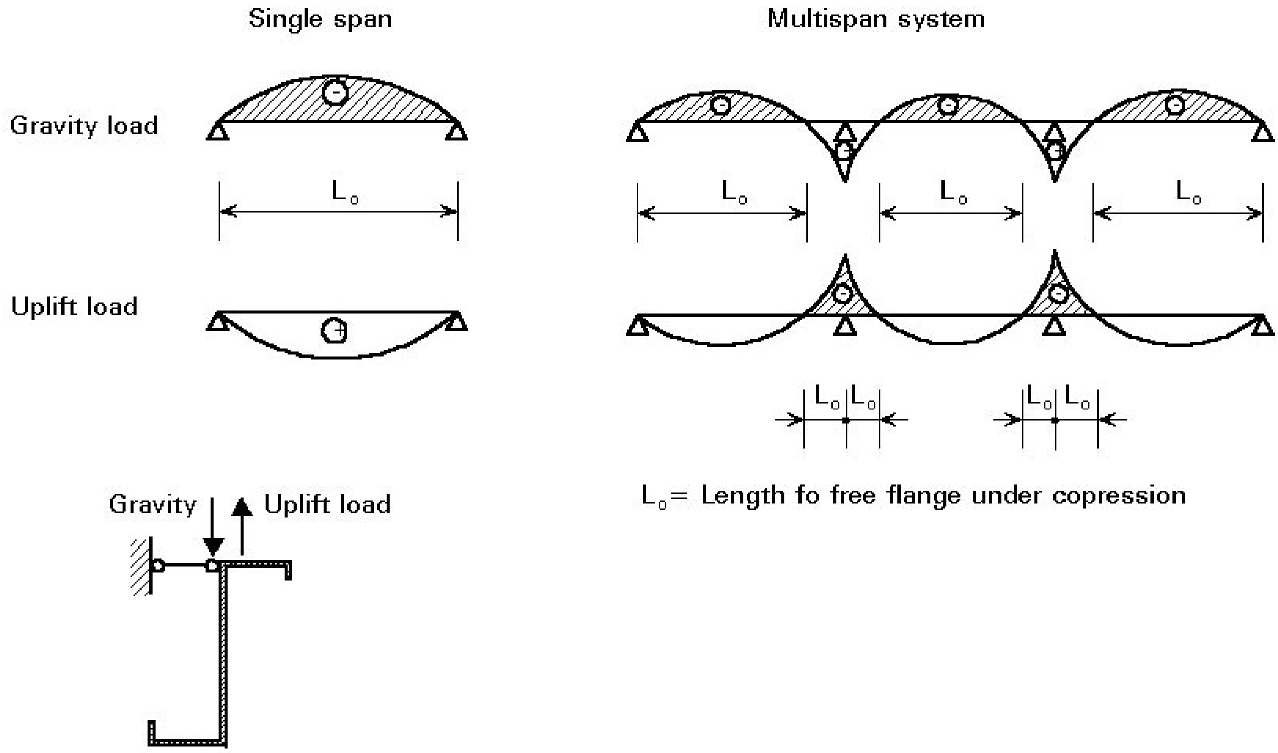



Figure 16 Stability check for the compressed free flange of the purlin

### 4.5.2 Double span systems with continuous cross-sections

In continuous systems (no sleeve or overlap systems) plastic behaviour at the middle support may be taken into account for the ultimate state. This means that with increasing load the bending moment over the support increases until it reaches the moment resistance $M_u$ of the cross-section (Figure 17). Increasing the load further leads to a redistribution of moments. The moment rotation behaviour shows a decrease of moment at the support whereas the midspan moment increases because of equilibrium. The limit state is reached when the midspan moment is equal to the moment resistance of the cross-section.

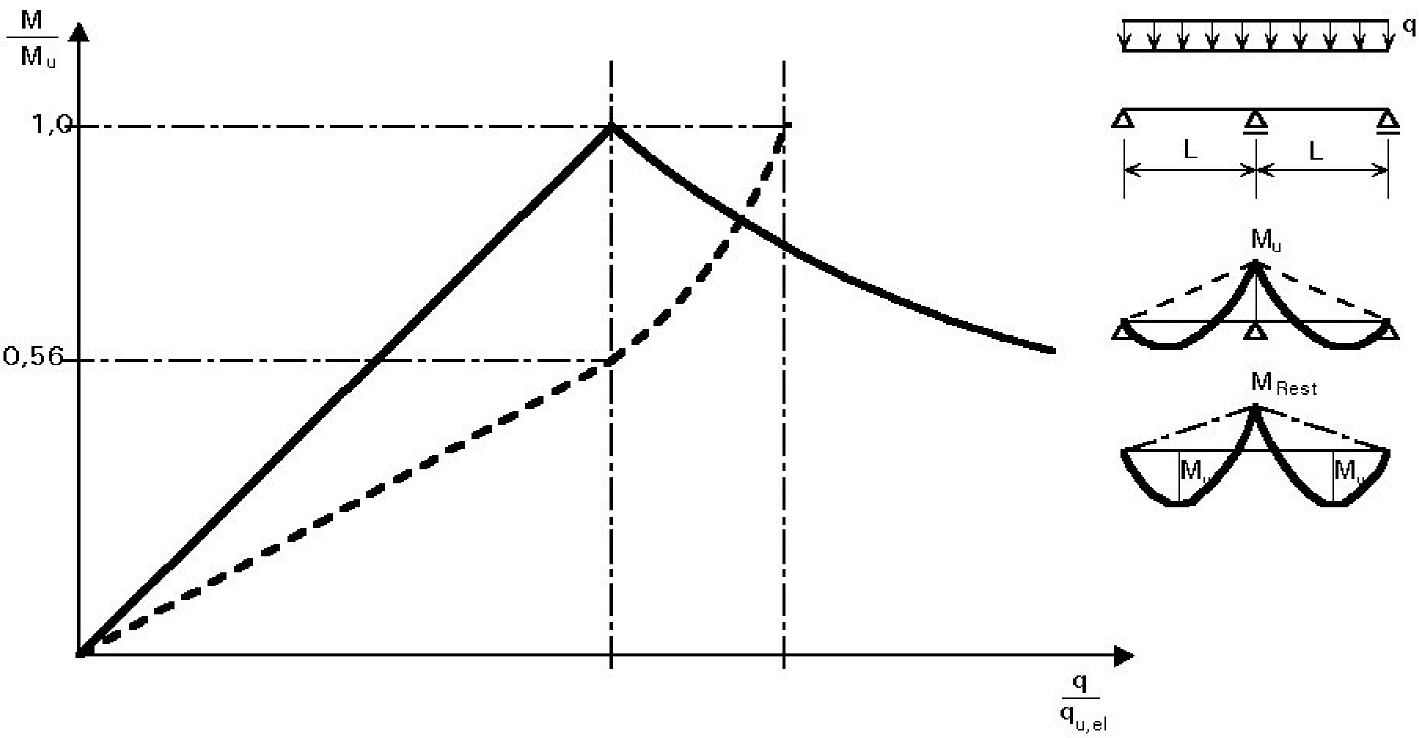



Figure 17 Behaviour of a double span purlin with increasing load

Stability has to be checked additionally.

### 4.5.3 Overlap and sleeve systems

The stiffness of the overlap or sleeve connection has to be found by testing. The moment distribution is calculated using this stiffness. The stresses due to the moment distribution have to be within the stress limits. Particular consideration must be given to end spans, which only benefit from continuity at one end.

Additionally stability of the compressed flanges must be checked and the deflections evaluated. The shear or support failure can be checked by testing.

## 4.6 Further Aspects in Design

Additional to the cross-section check of the purlin some further aspects have to be considered:

- The sheeting of roofs or walls has to carry in-plane loads from the purlins. These in-plane loads have two components (Figure 18):

  - The first component is the component of the external load in the direction parallel to the sheeting. This perpendicular component is carried by the purlin.

  - The second component is the lateral force at the upper flange of purlins with unsymmetrical cross-section.

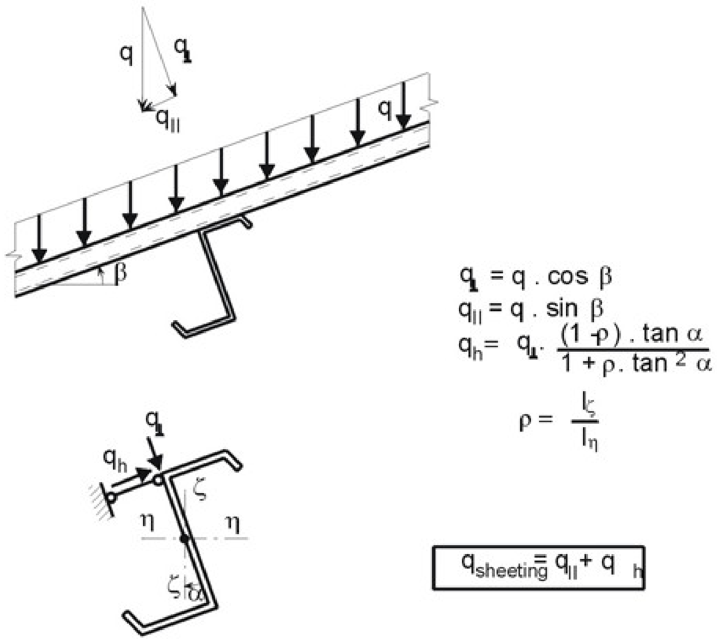



Figure 18 Membrane force in sheeting $q_s$

These forces in the plane of the sheeting have to be carried to the sag bars. Usually this is done using the cleats at the connection of the purlin on the sag bar. The connection has to be checked for this condition.

- Failure at the support of the purlin has to be checked. One failure mode is web-crippling; another failure mode is shear failure near to the support. Interactions between bending moments and support reactions have to be considered.
- Deflections should not exceed span/180 for serviceability.

## 4.7 Design by Testing

Testing is necessary to investigate the properties and behaviour of parts of the construction which cannot be analysed theoretically with the necessary accuracy.

Guidance is given in Part 1.3 of Eurocode 3 [1] concerning the number of tests, and the test set-up.

In the design of purlins several aspects of the construction may be examined by testing:

- Torsional restraint of the purlin by the sheeting (see Section 2.4).
- Maximum moment resistance of the purlin and the moment rotation capacity after plastic hinges form. The maximum bending moment resistance may be found also by calculation (yield stress multiplied by

the section modulus of the effective cross-section), but the moment rotation behaviour can be found only by testing.

- Failure at the support given by an interaction between bending moment and support reaction. In the support test, Figure 19, s is made equal to the distance between points of zero moment.
- Stiffness of the overlapping or sleeved parts of multispan beams (evaluated also by a support test).

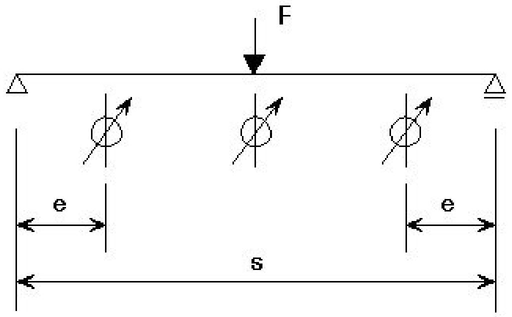



Figure 19 Support test

## 4.8 Some Practical Aspects

There are many constructional details in roof systems for connections of purlins between themselves or with the frames, for the in-plane forces of the sheeting or for the prevention of lateral-torsional buckling. Brief comments are given below.

### 4.8.1 Connection of purlins to frames

In some constructions the purlins are fixed directly to the frame by screws. Other systems use cleats as shown in Figure 20. Cleats shall be designed for the vertical and horizontal forces according to the rules common for steel construction.

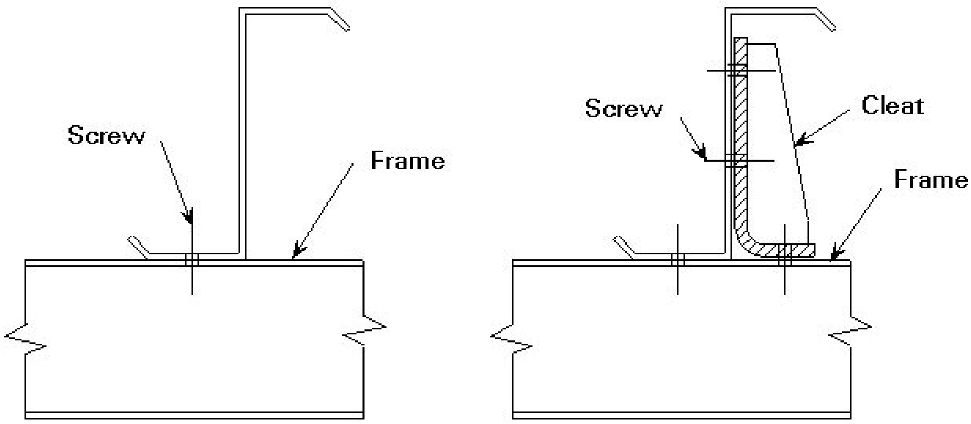



Figure 20 Connections of purlins on the frame

### 4.8.2 In-plane forces in sheeting

The in-plane forces in the sheeting have to be considered in the design. The forces have to be carried to the frames using the cleats at the supports of the purlin or ties between them along the frame (Figure 21). Some roof constructions connect the sheeting of one part of the roof at the ridge with the other part of the sheeting with opposite inclination. The purlins at the ridge have then to be stiffened (Figure 22) and the screws have to be checked.

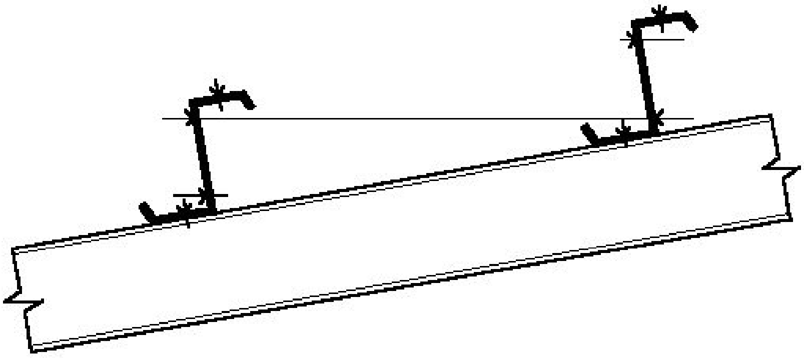


Figure 21 Ties between purlins

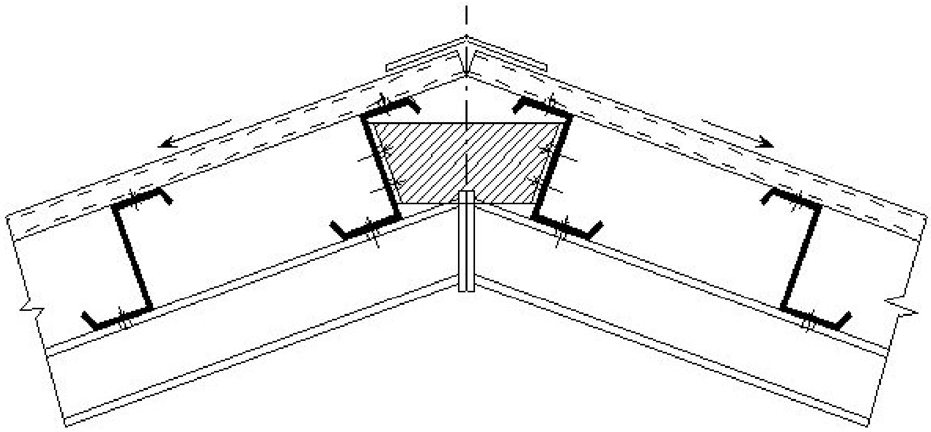


Figure 22 Transverse stiffener at top of the roof

As an alternative to using the membrane action of the sheeting, metal strips can be used to hold the upper flange of the purlins. These strips span over the whole roof. They have to be anchored to the frame.

#### 4.8.3 Prevention of lateral-torsional buckling

If the torsional restraint of the purlin has too low a stiffness to prevent lateral- torsional bucking, additional elements have to be added to the construction to hold the free flange of the purlin. There are two elements in use (Figure 23):

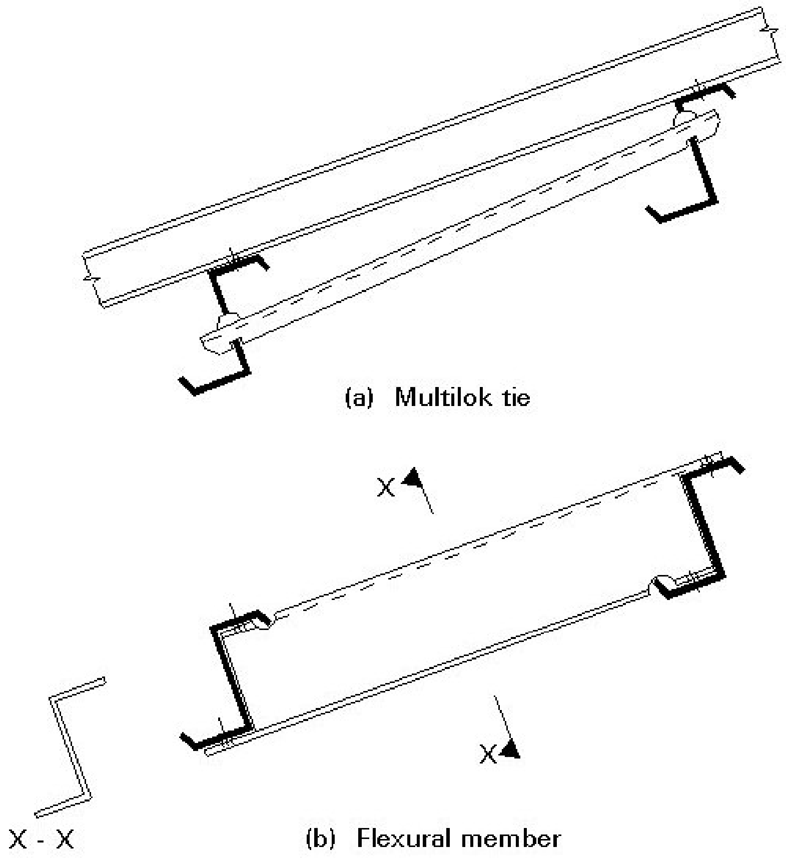



Figure 23 Prevention of lateral - torsional buckling

- **Multilok ties** carry tension forces only and stabilize the free flange. This element is usually used.
- **Flexural members** hold the purlin against twisting and carry compression forces too.

## 5. CONCLUDING SUMMARY

- The effective area of the cross-section is modified by reduction factors to take into account the effect of shear lag, local buckling in compression, and flange curling.
- Eccentricity moments arising from displacement of the effective neutral axis from the nominal neutral axis must be considered in design.
- Where the beam is adequately restrained, no account need be taken of lateral torsional buckling.
- Without adequate restraint, due account must be taken of lateral torsional buckling.
- Purlins represent a major application of cold-formed beams. Several special purlin systems have been developed.
- A particular feature of some purlin systems is the account taken of partial continuity from overlap and sleeve systems.
- Roof and wall systems using cold-formed beams have usually been developed as overall systems. The beams are only fully effective within these systems with appropriate restraint from the other components.

## 6. REFERENCES

[1] Eurocode 3, Part 1.3: "Cold-Formed Steel Sheeting and Members" CEN (in preparation).

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**ESDEP WG 9**

**THIN-WALLED CONSTRUCTION**

# Lecture 9.4: Design Procedures for Sheeting

### OBJECTIVE/SCOPE

To introduce the different types of sheeting and to discuss test requirements and calculation methods for trapezoidal sheeting.

### PREREQUISITES

Lecture 9.1: Thin-Walled Members and Sheeting

Lecture 9.7: Application of Thin-Walled Construction

Worked Example 9.3: Trapezoidal Sheeting

### RELATED LECTURES

Lecture 2.4: Steel Grades and Qualities

### SUMMARY

The lecture illustrates how the different types of sheeting are typically used in roof, floor and wall construction. Product development is discussed and the reasons for "design by testing" explained. Calculation procedures for design of trapezoidal sheeting are given involving checks on bending, shear, web crippling and bending/shear interaction at internal supports.

## 1. INTRODUCTION - TYPES OF SHEETING

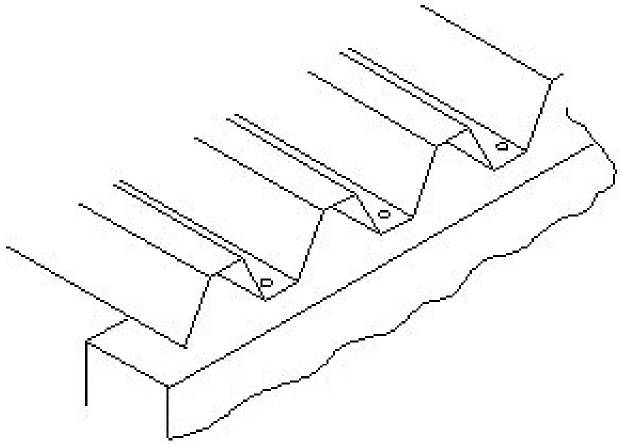
As discussed in Lecture 9.1, cold-formed sheeting can be developed not only to give adequate load bearing resistance but also to satisfy the functional requirements of the design. This aspect is now considered in more detail in relation to the common usage of cold-formed sheeting in roof, wall and floor structures.

**Roof structures**

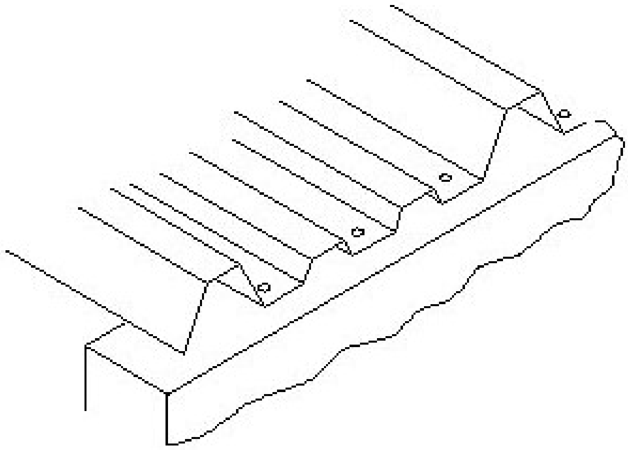
Roof sheeting systems can be used for either "cold" or "warm" roofs as described below:

a) A "cold roof" has an outer waterproof skin with internal insulation if required

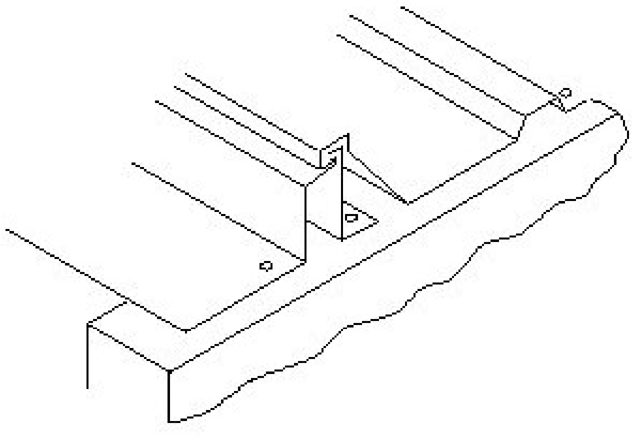
(Figure 1). The main requirement of preventing penetration by rain water leads to shallow profiles with a sequence of wide and narrow flanges; sheets are fixed using fasteners applied to the crests of the corrugations or by means of clips (standing seam profiles). The use of few points of fastening means that the forces at these points are relatively high; for these types of profile spans are relatively small.



(a) Simple trapezoidal profile



(b) Profile with wide, stiffened trough



(c) Standing seam

Figure 1 Outer waterproof skin in roof structures

b) A "warm roof" (Figure 2) includes insulation and water proofing, and is built up using a load-bearing profile, insulation (mineral wool or plastic foam), and an outer layer, e.g. metal skin, as mentioned above.

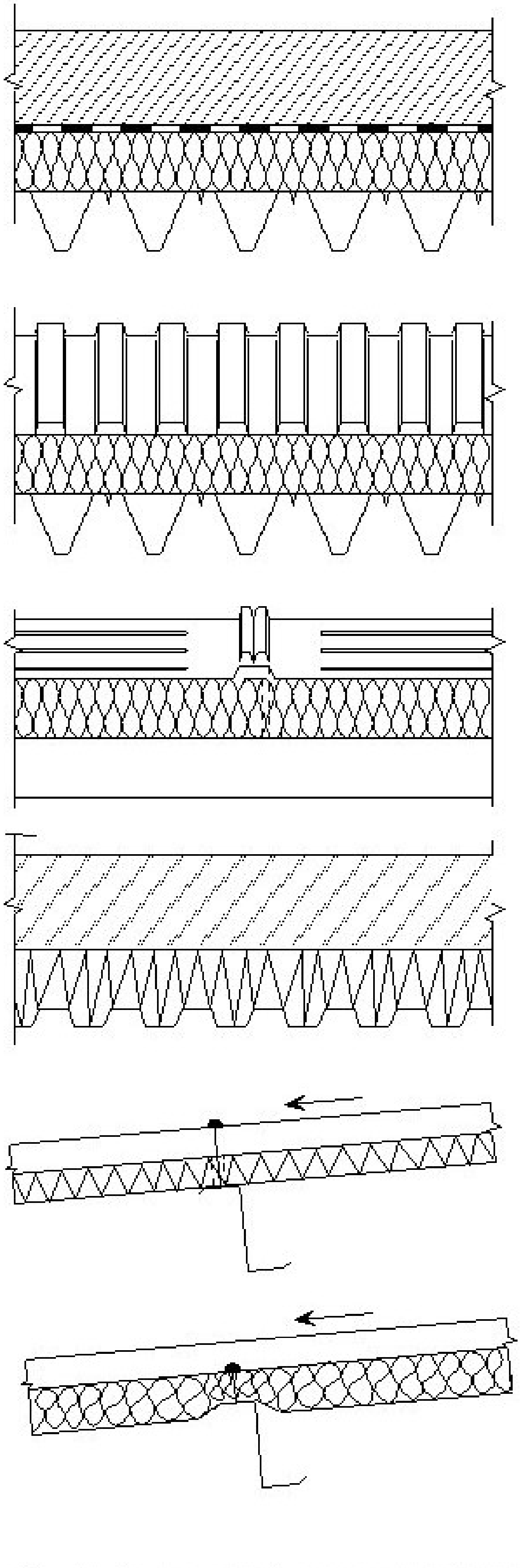


Figure 2 Built-up roof constructions

The load-bearing profiled sheeting in this type of roof normally has the wider flanges turned up in order to provide sufficient support for the insulation.

Fasteners are placed in the bottom of the relatively narrow troughs. In this case, the tendency is towards longer spans, using more complex profiles provided with intermediate stiffeners.

**Wall structures (Figure 3)**

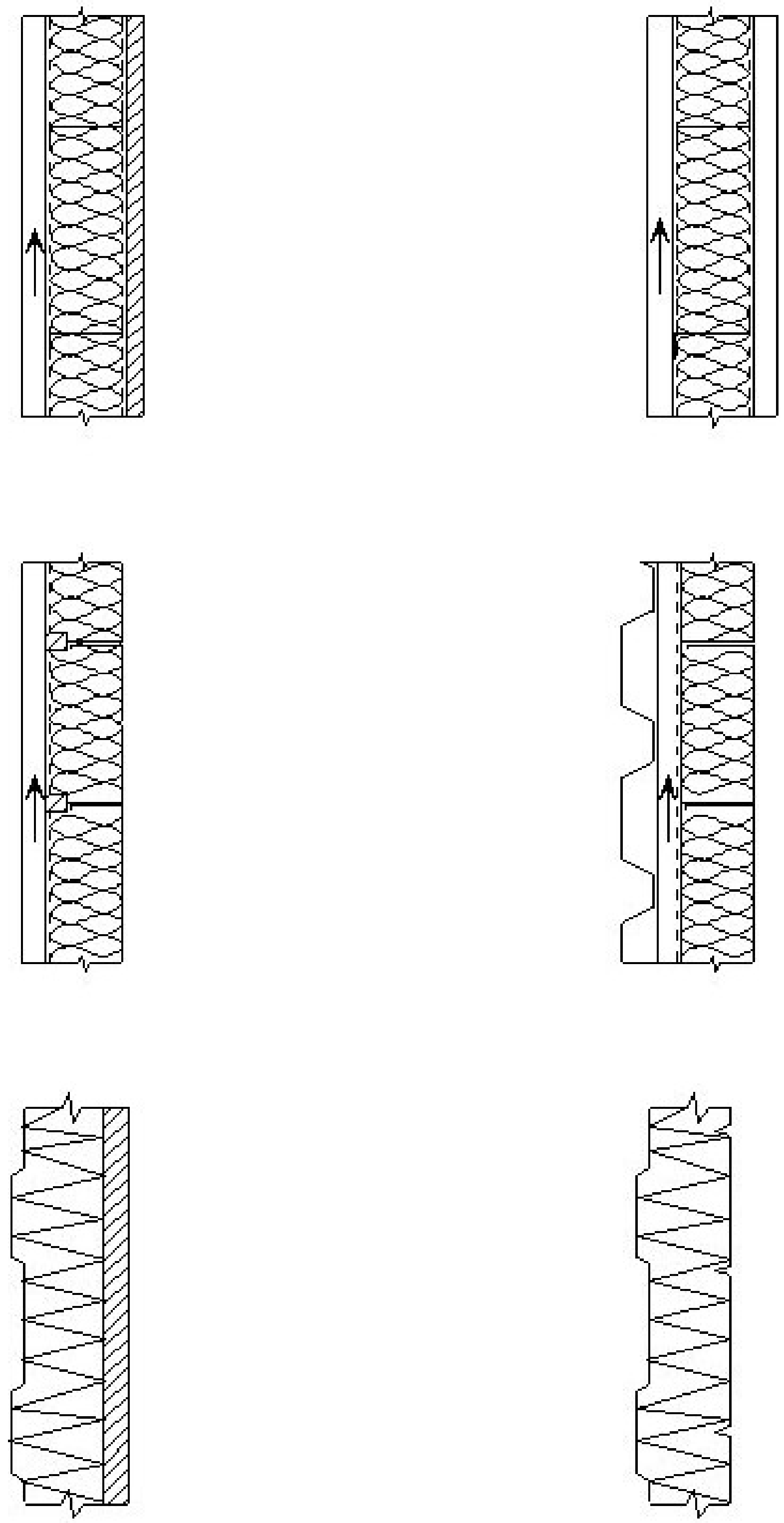


Figure 3 Example of wall structures

Wall structures comprise an outer skin of "architectural" sheeting of relatively small span, and a substructure which transmits the wind loading to the main building structure. The substructure can be a system of wall rails or horizontal deep profiles (cassettes) with integrated insulation. Another solution combines the load-bearing and protecting function in a "sandwich" panel built up by metal profiles of various shapes and a core of polyurethane.

**Floor structures (Figure 4)**

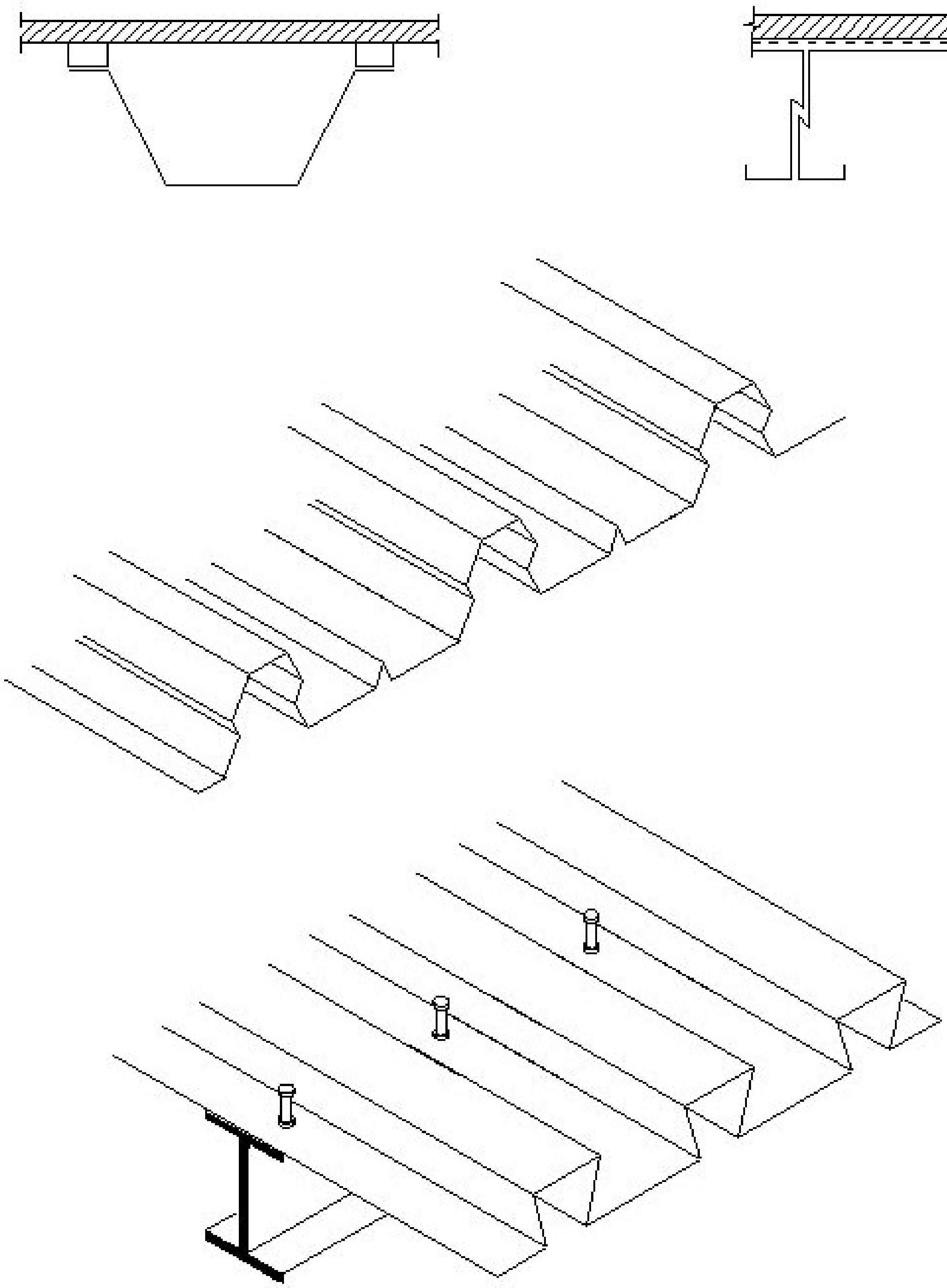


Figure 4 Example of floor structures

Floor structures have sheeting, e.g. trapezoidal sheeting or cassettes, as the load bearing part, either alone or in composite action with other materials such as board or plywood decking or cast in-situ concrete. In the first case, the composite action is provided by adhesives and mechanical fasteners, in the second by means of indentations and/or special shear studs.

Since bending moment resistance is the main requirement, the profiles selected for flooring purposes are similar to those for roof decking.

**Design objectives**

For trapezoidal sheeting with "normal" geometric properties of depth, width, stiffness and sheet thickness, design based on analytical expressions is possible (see Worked Example 3). Testing is required for profiles outside the defined range of geometrical properties and where composite action of other materials acting with the sheeting is to be assessed.

# 2. DESIGN PROCEDURES

Because of the many types of sheeting available and the diverse functional requirements and loading conditions that apply, design is generally based on experimental investigations (except for trapezoidal sheeting where analytical methods can be used). This experimental approach is generally acceptable for mass produced products, where optimization of the shape of the profiles is a competitive need.

During the 1960's, more and more countries established their own production of trapezoidal sheeting and export of these products increased. As a consequence of increased competition and trade, product development quickly led to new types of profiles with intermediate stiffeners, higher material strength and geometrical improvements (see Figure 5). The developments all led to increase of load bearing resistance.

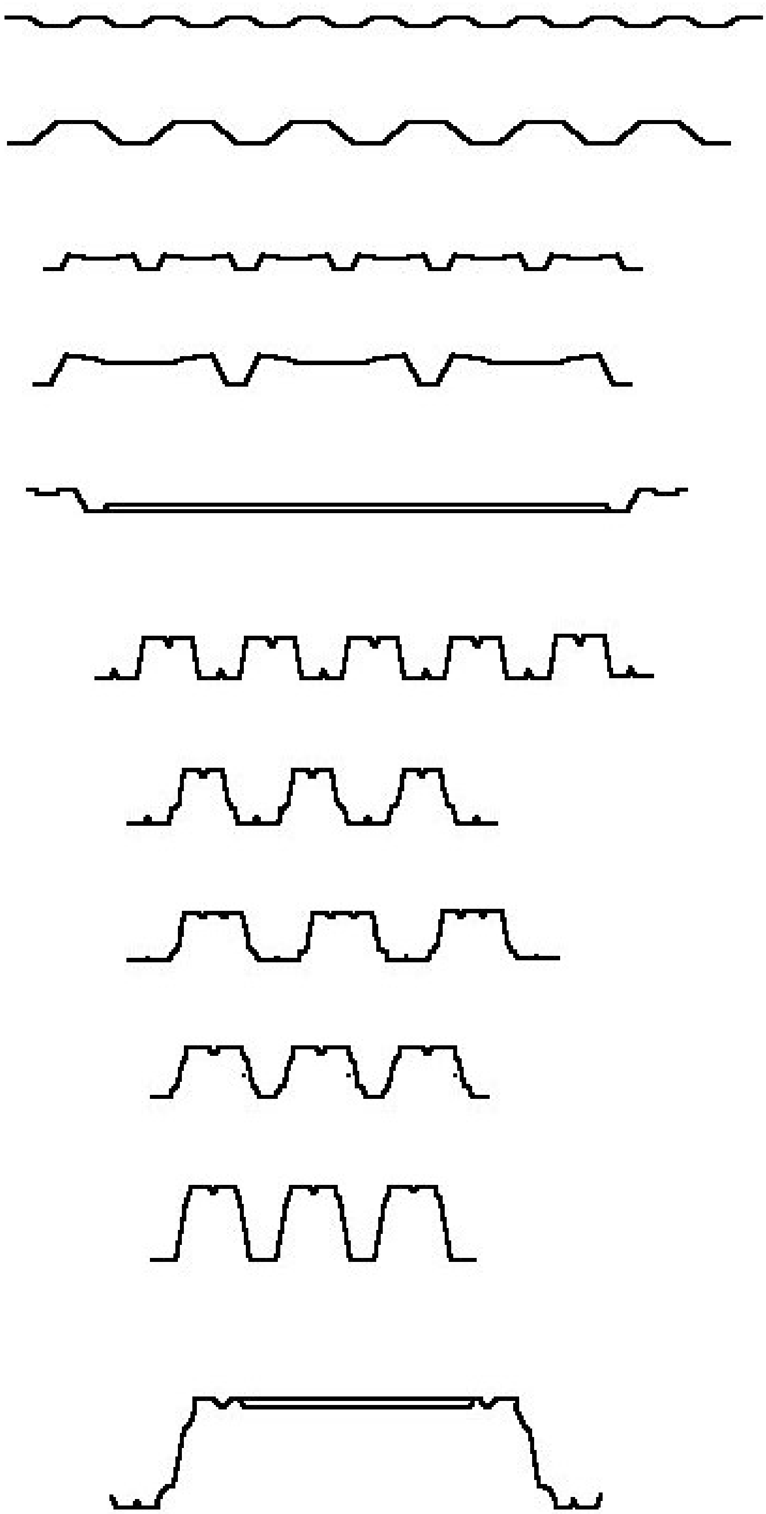


Figure 5 Types of trapezoidal sheeting

The product development, however, was based more on experience of the functional behaviour of the products than on analytical methods. The initial "design by testing", and subsequent growing understanding of the structural behaviour allowed analytical design methods to be developed; theoretical (semi-empirical) design formulae were then created based on the evaluation of test results. This type of interaction of analytical and experimental results occurs whenever special phenomena are responsible for uncertainties in the prediction of design resistance (ultimate limit state) or deformations (serviceability limit state).

Thousands of tests - as described below - have been carried out and evaluated, and design formulae have been developed which adequately predict the load bearing resistance of trapezoidal sheeting. However it is necessary to restrict design values to a relatively low level so that the design load resistance is somewhat lower than the actual resistance. This low level is necessary because a wide range of geometrical properties has to be covered by the design rules and, depending on the geometrical shape, different types of failure can occur.

Calculation procedures for trapezoidal sheeting are described in Section 3 below:

**Types of testing (Figure 6)**

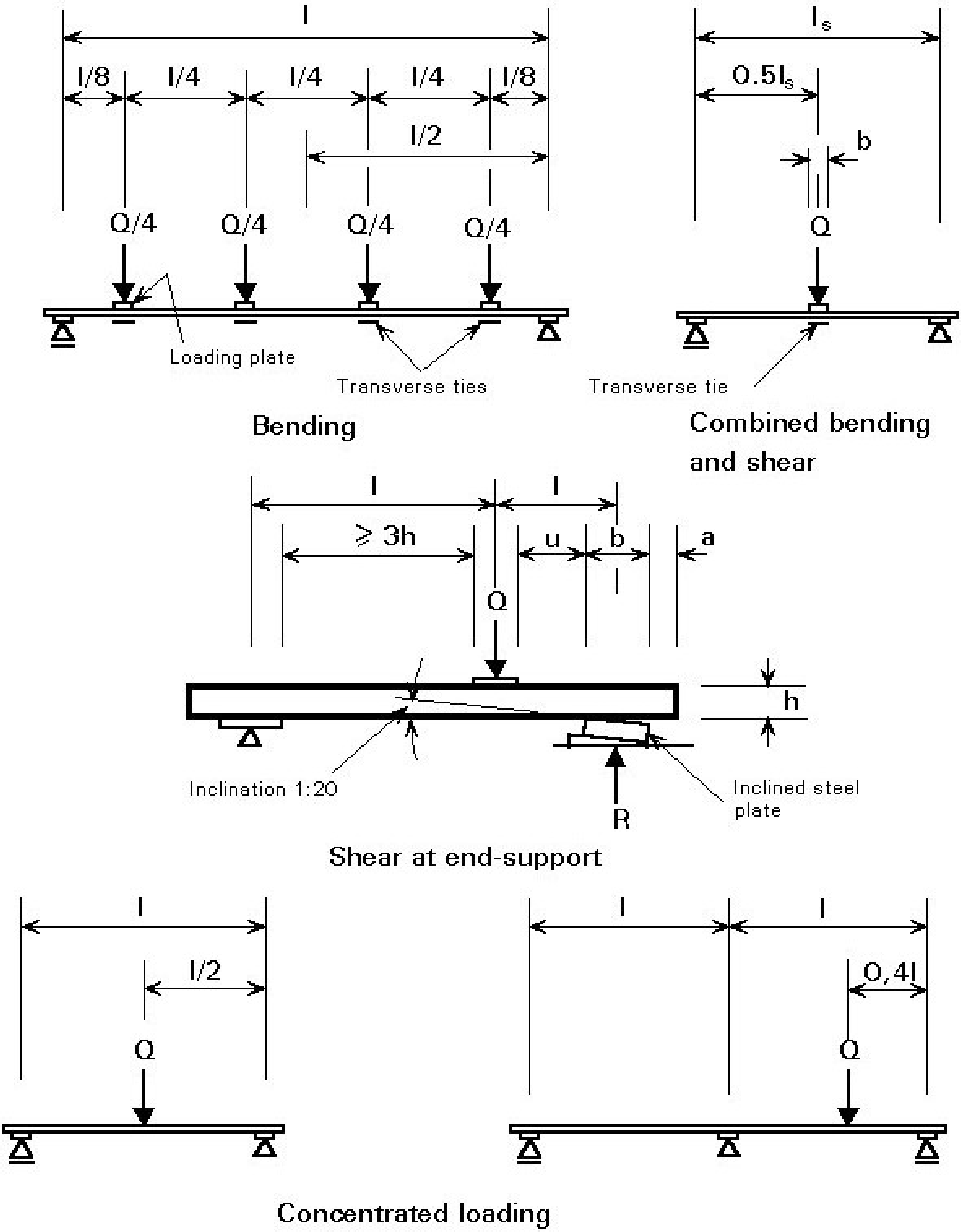



Figure 6 Types of testing for trapezoidal sheeting (examples)

Testing may be required for optimization purposes or due to lack of appropriate analytical design methods; if necessary, tests may be carried out to ascertain the following:

- bending resistance and bending stiffness, estimated by "single span beam tests".

- combined bending and shear or crippling resistance, estimated by either "intermediate support tests" or "two-span beam test" (allowing for possible moment redistribution).
- shear resistance at the end support, estimated by "end support tests".
- resistance to concentrated loads both during and after erection, representing "walkability".

Normally tests are performed under both gravity, e.g. dead weight and snow load, and uplift loading, i.e. wind suction; in addition, if the sheeting is unsymmetrical the loads are placed in two positions, depending on the application. Practical support and loading conditions are taken into account, which implies, for example, that for uplift loading the resistance of the fasteners and connections at the end and interior supports must be measured. More detailed information is presented in Part 1.3 of Eurocode 3 [1].

The tests should simulate the real behaviour of the sheeting under practical conditions. It is important that the testing equipment and the testing procedures are kept simple in order to achieve reliable and comparable results.

The rate of load application should usually be such that the stresses can be treated as quasi-static and be approximately equal in a test series. Furthermore an adequate number of load steps should be used in building up the load.

The load-bearing tests must be accompanied by standard tensile tests, using test specimens taken from the plane parts of the profiles. The load-bearing test results are then corrected with respect to the actual values of the core thickness ($t_f$), yield strength ($f_{yt}$) and the specified values ($t$, $f_y$).

The corrected test result is derived from the actual test result as follows:

$R_n = R_t(f_f/f_{yt})^{\alpha} (t/t_t)^{\beta}$

$\beta = 1$ for $t \geq t_t$

$\beta = 2$ for $t < t_t$

$\alpha = 0$ for $f_y \geq f_{yt}$

$\alpha = 0,5$ for $f_y < f_{yt}$ if failure is caused by local buckling, otherwise $\alpha = 1$

Using this procedure, the test results are transformed into values, with reference to nominal or specified values of sheet thickness and yield strength.

## 3. CALCULATION PROCEDURES FOR TRAPEZOIDAL SHEETING

When designing sheeting the following checks should be carried out:

- bending resistance.
- shear resistance.
- concentrated load resistance (crippling resistance).

- interaction of bending and shear and/or crippling.
- stiffness of the sheeting.

Design formulae for the above have been developed using the notation given in Figure 7.

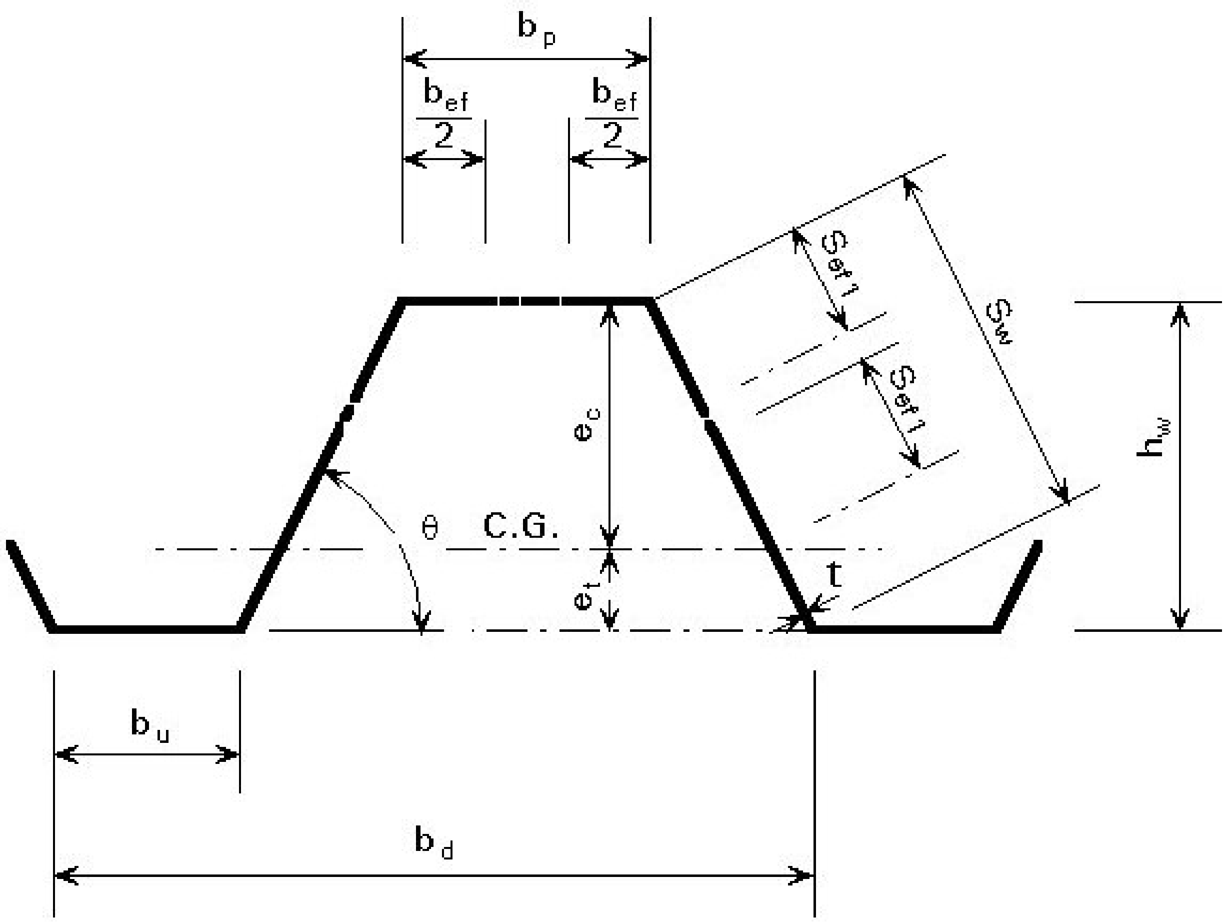



Figure 7 Notation for trapezoidal sheeting

### 3.1 Calculation Procedures for Bending

Step 1: Check the section geometry complies with the appropriate limits, i.e. $b/t \leq 500$, $s_w/t \leq 500$ (otherwise design by testing)

Step 2: Check whether rounding of corners can be ignored, i.e. $r/t \leq 5$, $r/b_p \leq 0{,}15$ (otherwise use the section properties from Figure 8)

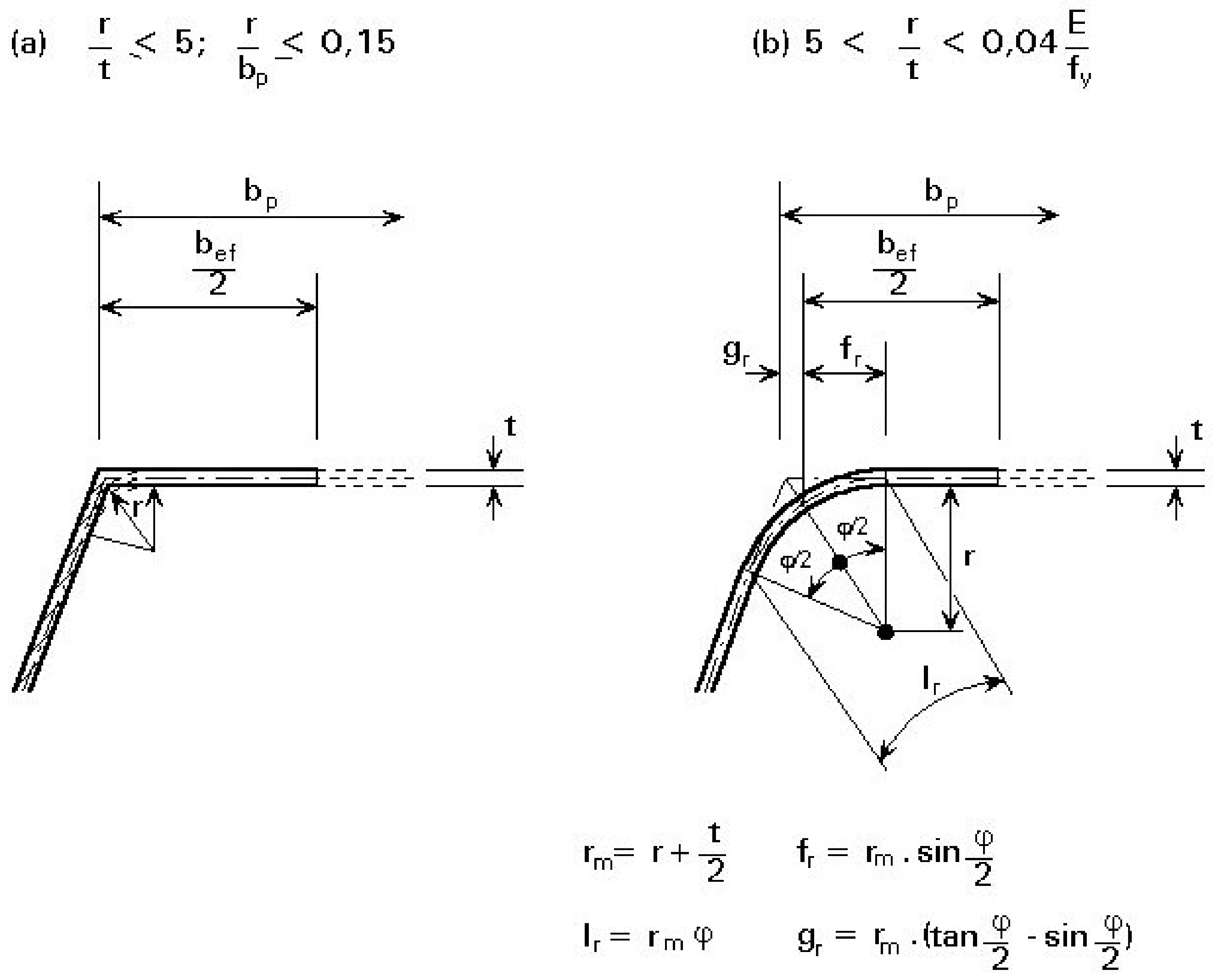



Figure 8 Geometrical treatment of rounding of corners

Step 3: Check the effect of flange curling (see 3.1.2)

Step 4: Check the effect of shear lag (see 3.1.3)

Step 5: Calculate the section values of the gross cross-section ($A_g$, $W_g$, $I_g$)

Step 6: Calculate the effect of intermediate stiffeners in flanges and webs (see 3.1.4)

Step 7: Calculate the section values of the effective cross-section ($A_{ef}$, $W_{ef}$, $I_{ef}$) at ultimate limit state (for $f_y$) and serviceability limit state (for $\sigma_c < f_y$)

Step 8: Determine the moment resistance $M_{c,Rd} = f_y . W_{ef}$. Plasticity in the tension zone, (see 3.1.5) may also be taken into account. Determine the bending stiffness ($EI_{ef}$) at serviceability limit state.

The following sections 3.1.1 - 3.1.5 are only intended as an explanation; design formulae and procedures should be taken from the codes.

### 3.1.1 Effective portions of the web

Normally it is assumed that the bending resistance ($M_{c,Rd}$) at the ultimate limit state can be calculated by assuming the stress in the compression zone is at yield (see Figure 11a). The effective width $b_{ef}$ of the compression flange is calculated in the usual way. With a reduced compression flange (see Lecture 9.1), and the web treated as fully effective, the depth $e_c$ to the approximate position of the neutral axis is calculated. The effective portions of the compression zone of the web are then positioned as shown in Figure 7, having lengths given by:

$$S_{ef,1} = 0,76 \,.\, t\sqrt{\frac{E}{\sigma_c}}$$

$$S_{ef,2} = 1,5\, S_{ef,1}$$

where $\sigma_c$ is the compressive stress at the flange level. The ultimate moment of resistance $M_c$ may then be calculated for the doubly reduced cross-section with $e_c$ and $e_t$ referred to the approximate neutral axis.

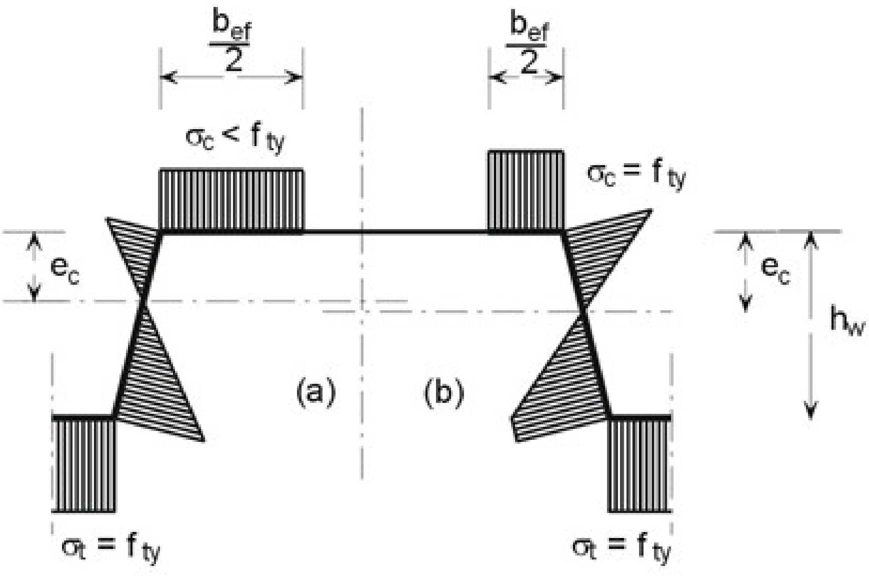



Figure 11 Stress distributions of ultimate states when $e_c > h_w/2$

### 3.1.2 Effect of flange curling

Due to the curvature of the sheeting, flanges of trapezoidal sheeting with high b/t ratios are prone to an inward deflection towards the neutral plane, caused by a radial component of tensile or compressive bending stresses. Normally this effect has only to be considered if the $b_p/t$ ratio is more than $250(s_w/b_p)$, where $s_w$ is the width of the web and $b_p$ the width of the flange. Approximate formulae for calculating the effect of flange curling, which in principle reduces the moment resistance, are given in the codes.

### 3.1.3 Effect of shear lag

Shear lag is associated with wide flanges with relatively short span lengths ($L/b_p \leq 20$). Owing to the action of in-plane shear strain in the flanges, the longitudinal displacements in the parts of the flange remote from the webs lag behind those nearer the webs. As the stress distributions, due to shear lag, have similarities to those of local buckling, an effective width approach can be applied. Normally this phenomenon can be ignored for trapezoidal sheeting. For other cases appropriate design rules are given in the codes.

### 3.1.4 Effect of intermediate stiffeners in flanges and webs

The design of profiles with stiffeners is not within the scope of this lecture, nor is it explicitly illustrated in Part 1.3 of Eurocode 3 [1]; such advanced profiles with intermediate stiffeners in flanges and/or webs are normally developed by manufacturers who will supply load resistance information derived from testing programmes. Intermediate stiffeners, as shown in Figure 9, can substantially increase the load-bearing resistance with respect to bending, shear and crippling as well as the stiffness (see Lecture 9.1). The basic idea is to reduce the flange width (b) and the web height (h) by means of supporting springs perpendicular to the plane elements (flanges and webs), where the spring stiffness depends on the flexibility of the elements and their boundary conditions (see Figure 10). The interaction of the elements, however, implies an iterative design procedure, resulting in increased effective section properties, compared to those of plane elements without stiffeners.

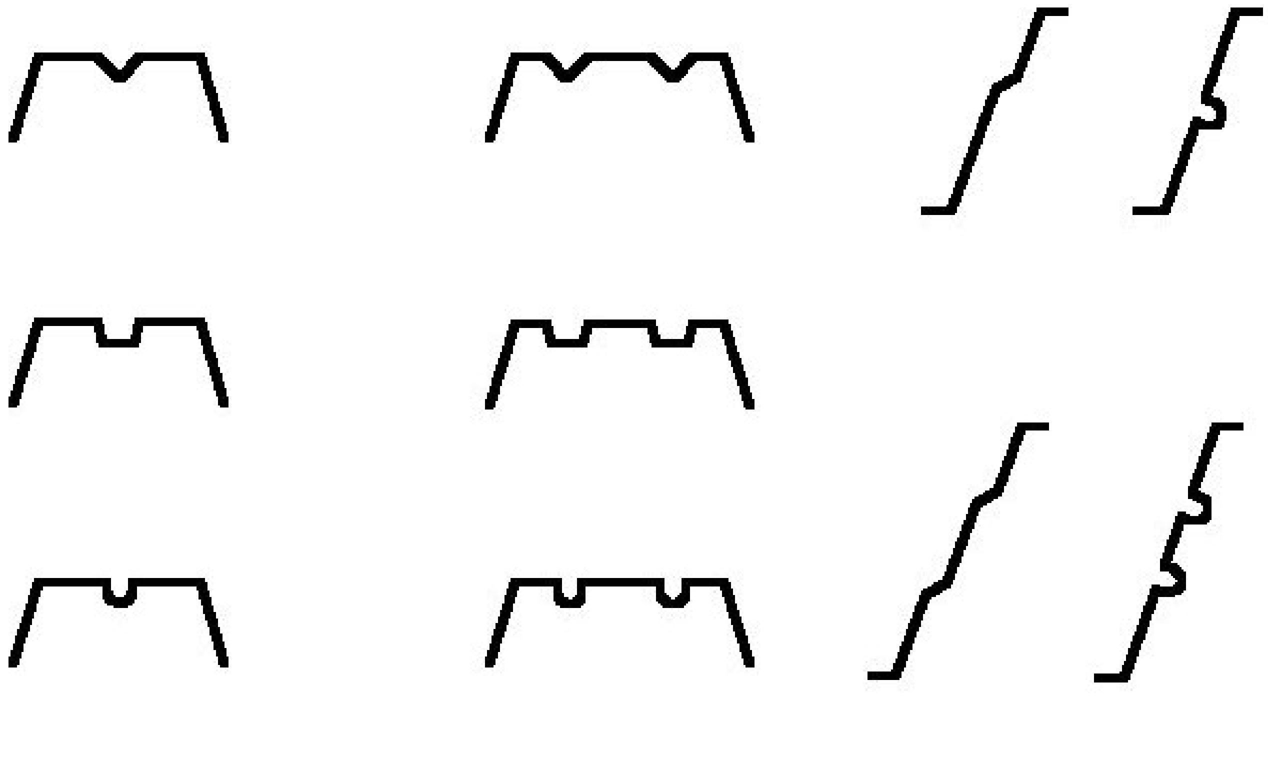


Figure 9 Types of intermediate stiffeners

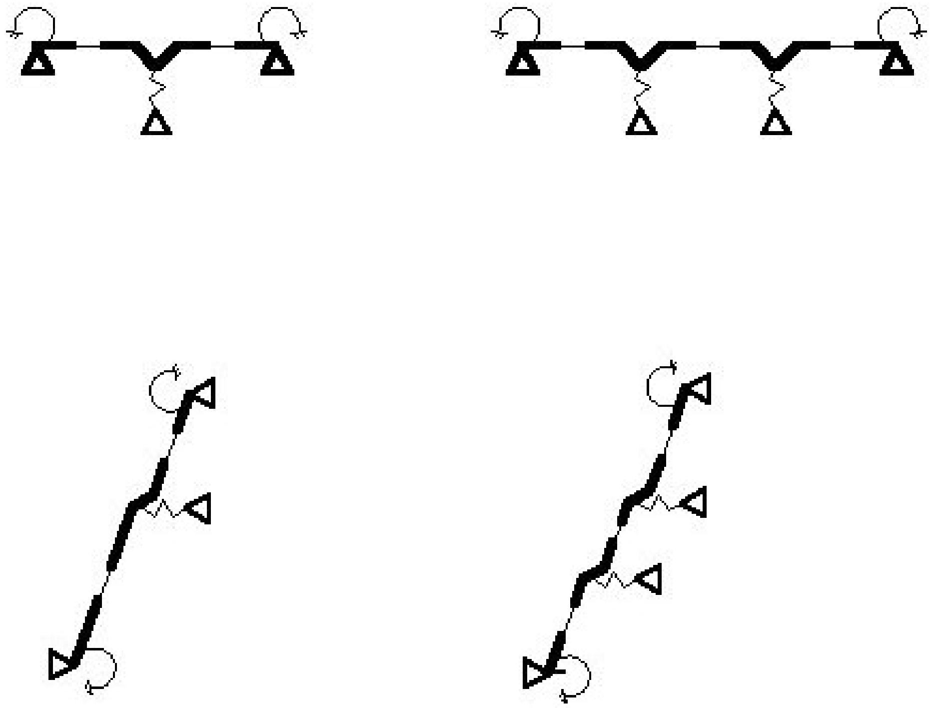


Figure 10 Modelling of action of stiffeners

### 3.1.5 Effect of plasticity in the tension zone

There are two cases of neutral axis position that should be considered:

If the neutral axis of the effective section is located closer to the compression flange than the tension flange, then tensile yield will occur first and plasticity of the tension zone may generally be utilized. According to Figure 11b, the equilibrium of a section under bending is given by:

$$M_{p,Rd} = \int_{A_{ef}} z_i \sigma_c . dA / \gamma_M \qquad (1)$$

and the position of the neutral axis can be derived from the equation:

$$\int_{A_{ef}} \sigma . dA = 0 \qquad (2)$$

If the neutral axis is located closer to the tension flange, then compression yielding occurs first. No plasticity is permitted. Hence, for a linear stress distribution (Figure 11a) without plasticity and for max $\sigma_c \leq f_y$, Equation (1) can be written as:

$$M_{c,Rd} = \sigma_c . W_{ef} / \gamma_M \quad (3)$$

where $W_{ef}$ is the section modulus of the effective cross-section.

Where plasticity in the tension zone of the effective cross-section occurs, the location of the neutral axis has to be determined by an iterative process.

### 3.2 Calculation Procedures for Shear

The maximum shear stress in the web of trapezoidal sheeting is as follows:

- for compact sections, $\tau_w = f_y/\sqrt{3}$
- for webs prone to web buckling, the buckling stress $\tau_{sp} = f(\bar{\lambda}_w)$, where $\bar{\lambda}_w$ is the related slenderness of the web expressed as:

  $\bar{\lambda}_w = 0,346\ (s_w/t)\ \sqrt{(f_y/E)}$ (4)

It may be assumed that the shear stresses are uniformly distributed along the web, so that the design shear resistance along the web is given by the following:

$V_{w,Rd} = \tau_{sp} \cdot s_w \cdot t/\gamma_M$ (5)

where:

$\tau_{sp}$ is taken from a buckling curve for $\bar{\lambda}_w > 0,8$, or if $\bar{\lambda}_w \leq 0,8$, is equal to the maximum value of $0,58\ f_y$.

$S_w$ is the distance between the points of intersection of the system lines of the web and flanges.

t is the core thickness of the section.

If the web is provided with intermediate stiffeners, the design strength is increased.

### 3.3 Calculation Procedures for Web Crippling

This phenomenon, which is similar to that of shear buckling, is related to the stability of the web under concentrated loading (see also Lecture 9.1). It is, however more severe with respect to the load-bearing resistance of the sheeting since the post-critical bearing reserve is quickly exhausted if buckling occurs. This is especially true if the concentrated loading is accompanied by shear and bending stresses, as is usually the case. Formulae for the design resistance ($R_d$) are based on test results.

Among other parameters, the design resistance depends on the width of the sheeting support, i.e. the bearing length on the substructure. One means of avoiding the web crippling effects is to provide the sheeting with special support cleats, so that the support reaction is transmitted from the sheeting to the substructure by tension forces instead of compression.

### 3.4 Calculation Procedures for the Interaction of Bending and Support Reactions

The load resistance of continuous sheeting greatly depends on its behaviour in the region of the intermediate support (see also Section 3.5) where the maximum bending moment occurs; the design resistance in this area must, therefore, also be checked. Interaction formulae which have been derived from a large number of test results (see Figure 12), show that interaction need not be taken into account if the actual support reaction or concentrated load is less than 25% of the design load; in this case the bending resistance can be fully utilized. In practice, the load ratio will often be above this limit, requiring a reduction in the bending resistance, as follows:

$$\frac{M_{sd}}{M_{Rd}} + \frac{V_{sd}}{V_{Rd}} \leq 1,25 \qquad (6)$$

where:

$$0,25 < \frac{V_{sd}}{V_{Rd}} \leq 1,0 \qquad (7)$$

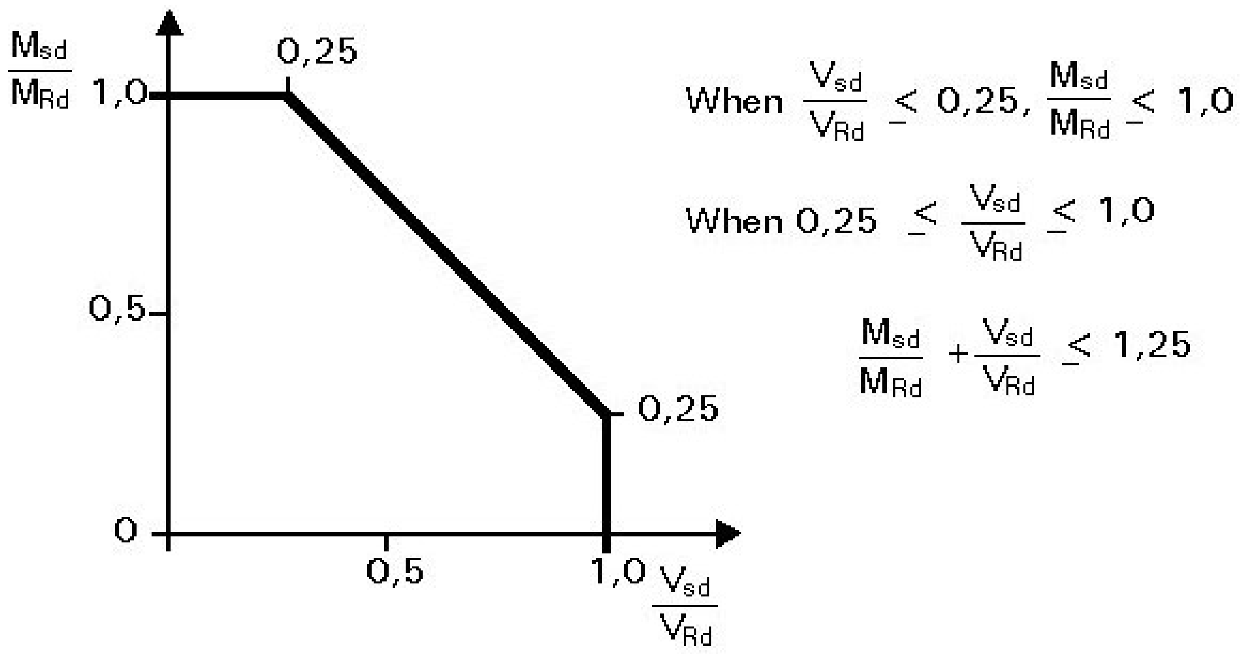



Figure 12 Interaction diagram for values of moment and reaction at internal support

### 3.5 Calculation Procedures for Developing Moment Redistribution

Continuous beams of compact (thick-walled) sections may be designed, as a rule, according to plastic hinge theory allowing moment redistribution by rotation of plastic hinges. For thin-walled sections with adequate rotational capacity the same method can be used; as a rule, however, the plastic capacity is limited by buckling phenomena and only part of the full plastic moment can be used for the moment redistribution. On the other hand, the rotational capacity provided by the "buckling hinges" may be sufficient for a new equilibrium state of the continuous beam to arise after buckling at the support has occurred (Figure 13). The moment redistribution must be investigated by tests, in which the rotational capacity with respect to the geometrical properties can be quantified.

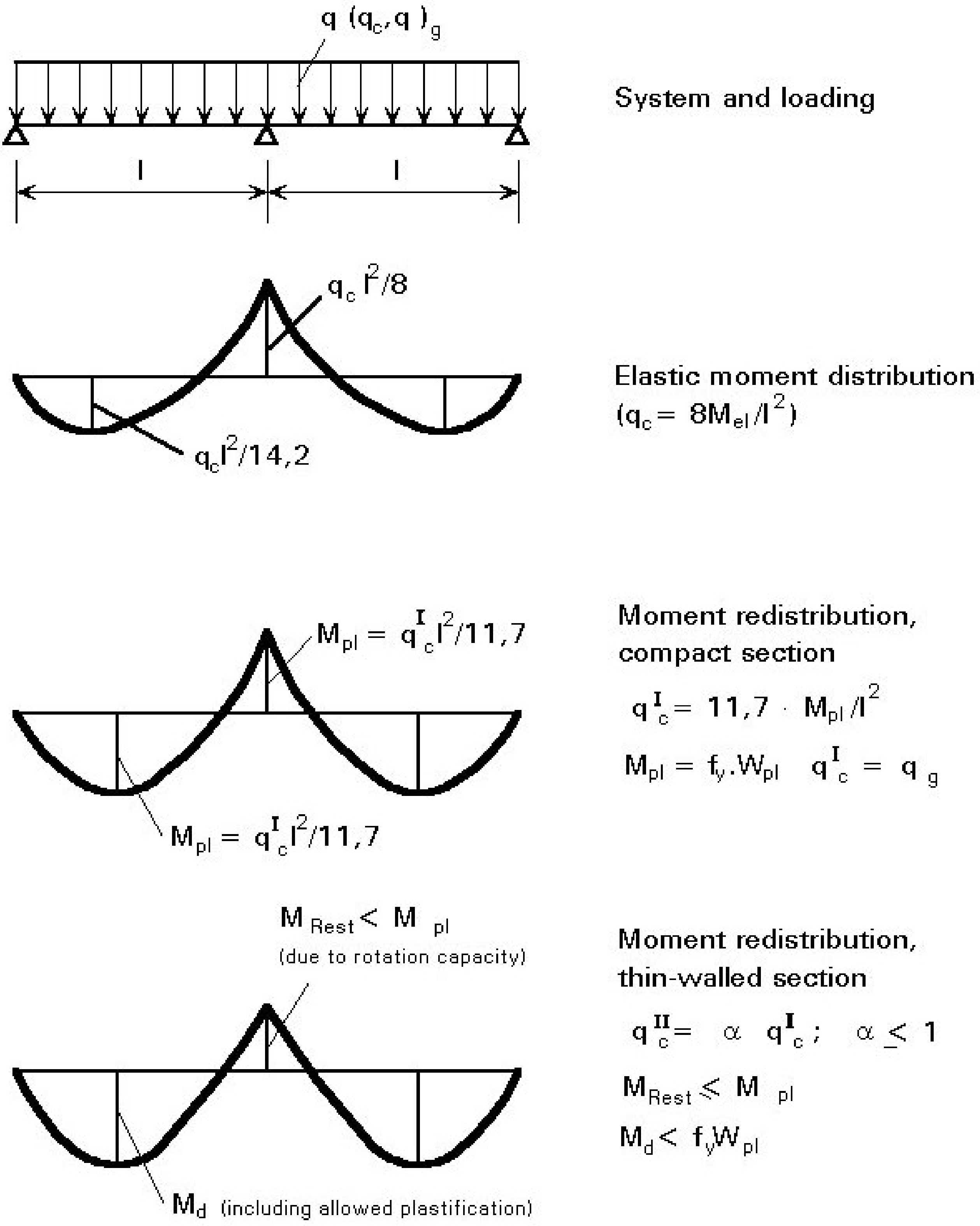



Figure 13 Moment redistribution for two-span beam of compact and thin-walled type respectively

## 3.6 Calculation Procedures for Estimation of the Bending Stiffness

Knowledge of the bending stiffness is important for calculating deflections at the serviceability limit state. As the section properties depend on the effective area, which is a function of the actual stresses, it is necessary to relate the moment of inertia to the appropriate stress level ($\sigma_c < f_y$).

# 4. CONCLUDING SUMMARY

- Sheeting is typically used in roof (cold and warm), wall and floor construction.
- Product development, based on experience of functional behaviour, resulted initially in "design by testing" rather than using analytical methods.
- Analytical methods were subsequently developed for trapezoidal sheeting which were based on experimental data. These methods involve checking bending, shear, web crippling and the interaction of these effects at internal supports.

# 5. REFERENCES

[1] Eurocode 3, Part 1.3: "Cold Formed Steel Sheeting and Members" CEN (in preparation).

# 6. ADDITIONAL READING

1. European Convention for Constructional Steelwork, "European Recommendations for the Design of Profiled Sheeting", Publication40, ECCS, 1984.
2. European Convention for Constructional Steelwork, "European Recommendations for Good Practice in Steel Cladding and Roofing", Publication 34, ECCS, 1983.
3. Höglund, T., "Design of Trapezoidal Sheeting provided with Stiffeners in the Flanges and Webs", Swedish Council for Building Research, Document D28: 1980.

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**ESDEP WG 9**

**THIN-WALLED CONSTRUCTION**

# Lecture 9.5 Stressed Skin Design

## OBJECTIVES/SCOPE

To introduce the concept of stressed skin design and to discuss the practical applications of this method.

## PREREQUISITES

Lecture 7.11: Frames

Lecture 9.1: Thin-Walled Members and Sheeting

## RELATED LECTURES

Lecture 9.4: Design Procedures for Sheeting

## RELATED WORKED EXAMPLES

Worked Example 9.1: Stressed Skin Design

## SUMMARY

This lecture explains the contribution that panels of roofing, flooring and walls make to the resistance and stiffness of frameworks by virtue of their resistance and stiffness in shear ("shear diaphragms"). Procedures and tables for the calculation of the resistance and flexibility of diaphragms are given. The practical applications of stressed skin design are also discussed.

## NOTATION

a = length of diaphragm in a direction perpendicular to the corrugations (mm)

A = cross-section area of longitudinal edge member ($mm^2$)

b = depth of diaphragm in a direction parallel to the corrugations (mm)

c = overall shear flexibility of a diaphragm (mm/kN)

d = pitch of corrugations (mm)

E = modulus of elasticity of steel (205 $kN/mm^2$)

$f_y$ = yield strength of steel in sheeting ($kN/mm^2$)

$F_p$ = design shear resistance of individual sheet/purlin fastener (kN) (see Table1)

$F_s$ = design shear resistance of individual seam fastener (kN) (see Table 1)

$F_{sc}$ = design shear resistance of individual sheet/shear connector fastener (kN) (see Table 1)

h = height of profile (mm)

k = frame flexibility (mm/kN)

K = sheeting constant (see Tables 4 and 5)

l = width of corrugation crest (mm)

L = span of diaphragm between braced frames (mm)

n = number of panels in the length of the diaphragm assembly

$n_b$ = number of sheet lengths within depth of diaphragm

$n_f$ = number of sheet/purlin fasteners per sheet width

$n_p$ = 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

$n^1{}_{sc}$ = number of sheet/shear connector fasteners per intermediate rafter

$n_{sh}$ = number of sheet widths per panel

p = pitch of sheet/purlin fasteners (mm)

q = distributed shear load on diaphragm (kN/mm)

$s_p$ = slip per sheet/purlin fastener per unit load (mm/kN) (see Table1)

$s_s$ = slip per seam fastener per unit load (mm/kN) (see Table1)

$s_{sc}$ = slip per sheet/shear connector fastener per unit load (mm/kN) (see Table1)

t = net sheet thickness, excluding galvanising and coating (mm)

V = applied shear force on diaphragm (kN)

V* = design shear resistance of diaphragm (kN)

$V_{cr}$ = shear force on diaphragm to cause overall shear buckling (kN)

$V_R$ = resistance associated with a given failure mode or ultimate load (kN)

$\alpha_1$, $\alpha_2$, $\alpha_3$ = factors to allow for intermediate purlins (see Table 3)

$\alpha_4$ = factor to allow for number of sheet lengths.

For the case considered $\alpha_4 = (1 + 0,3n_b)$

$ß_1,ß_2$ = factors to allow for the number of sheet/purlin fasteners per sheet width (see Table 2)

$ß_3$ = distance between outermost fasteners across the sheet width divided by sheet width.

For sheeting (seam fasteners in the crests) $ß_3 = \frac{n_f - 1}{n_f}$

For decking (seam fasteners in the troughs) $ß_3 = 1,0$

$\Delta$ = midspan deflection of a panel assembly (mm)

$\upsilon$ = Poisson's ratio for steel (0,3)

# 1. INTRODUCTION - DESIGN PRINCIPLES

## 1.1 Diaphragm Action

It has long been recognised that a building framework is considerably strengthened and stiffened once the roof, floors and walls have been added. Frame stresses and deflections calculated on the basis of the bare frame are usually quite different from the real values. By taking the cladding into account, the actual behaviour of the building can be predicted and usually worthwhile savings may be made in the costs of the frames.

The contribution that panels of roofing, flooring and side cladding make to the resistance and stiffness of frameworks is by virtue of their resistance and stiffness in shear, i.e. the resistance of rectangular panels to being deformed into parallelograms. Hence such panels are known as "shear diaphragms" or simply "diaphragms". In the United States, the design method which takes this effect into account is called "diaphragm design" whereas in Europe it is called "stressed skin design".

Profiled steel sheeting used as roof sheeting or decking, floor decking or side cladding, is very effective as a shear diaphragm. Provided it is positively attached to the secondary members and main frames by mechanical fasteners or welding, it is extremely reliable and predictable and may be confidently used as a structural component. Moreover, it has been verified by many full scale tests and proven by practical experience of many buildings designed on this basis.

The principles of stressed skin design may be illustrated with reference to flat-roofed or pitched-roof buildings. In a flat-roofed building subjected to side load (Figure 1) each of the roof panels acts as a diaphragm taking load back to the gable ends which are stiffened in their own planes by bracing or sheeting.

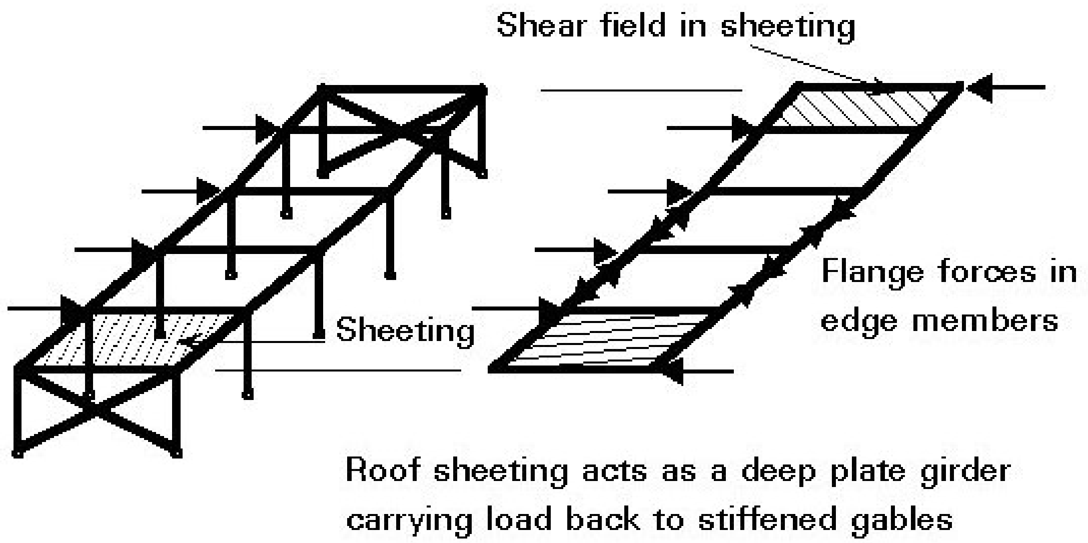


Figure 1 Diaphragm action in a flat roofed building

In a pitched-roof building (Figure 2) under vertical or side load, there is a component of load down the roof slope so that the roof diaphragms tend to prevent the building from spreading or swaying. The flatter the roof pitch, the less effective the diaphragms are in resisting vertical load, but the more effective they are in resisting horizontal load.

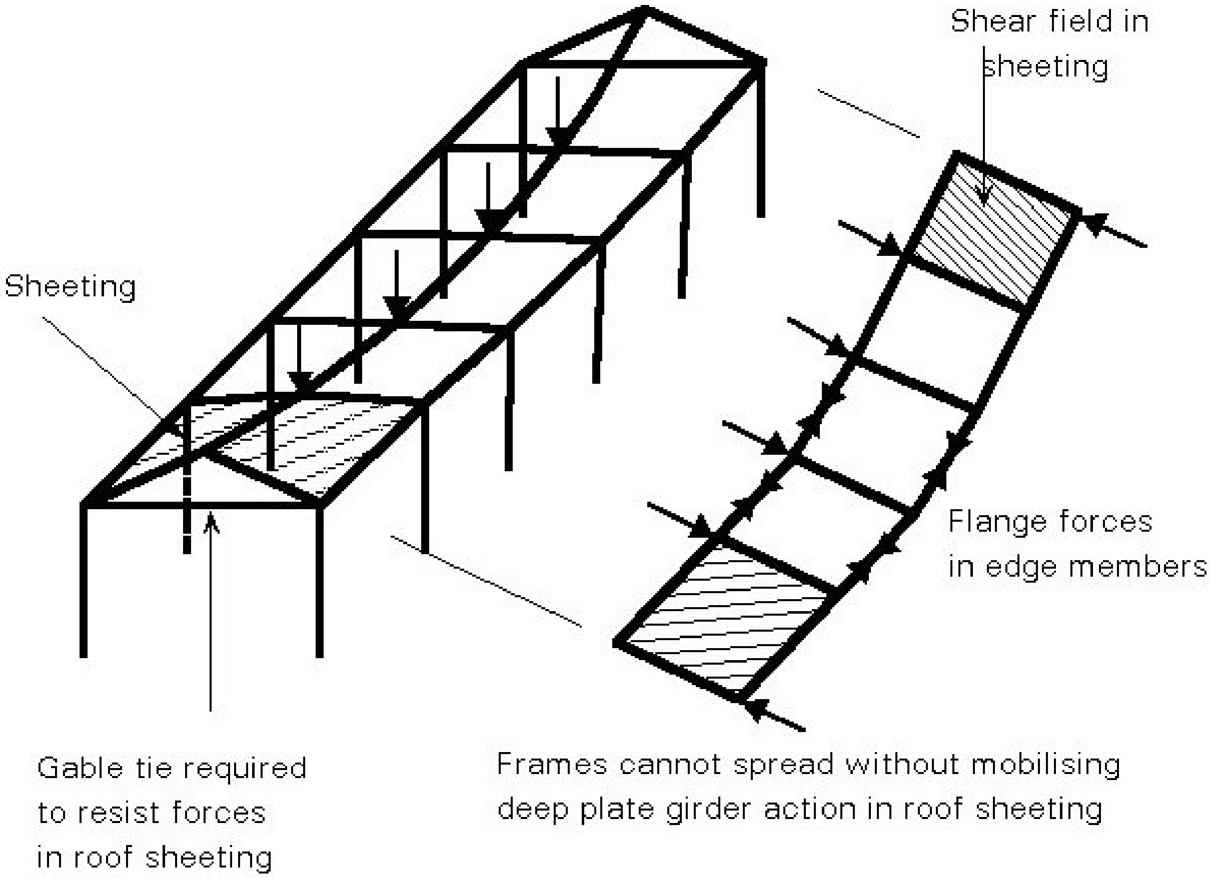


Figure 2 Diaphragm action in a pitched
roof building

The sheeting in Figures 1 and 2 acts in the roof such that the roof behaves like a deep plate girder. Under in-plane load, the end gables take the reactions, the sheeting acts as a web and takes the shear, and the edge members act as flanges and take the axial tension and compression. In no case does the sheeting help the frames to resist bending out of the plane of the sheeting.

## 1.2 Suitable Forms of Construction

If the frames of Figure 1 are pin-jointed, then the horizontal loads are resisted entirely by stressed skin action. In this case the structure must be adequately braced during erection and the sheeting panels must not be removed without proper consideration.

If the frames of Figure 1 have rigid joints, then the horizontal loads are shared between the frames and the diaphragms. In this case it is good practice for the frames alone to be designed to carry the full characteristic load without collapse, and for the completed stressed skin building to be designed to carry the full design load. The diaphragms then effectively provide the required load factor.

Stressed skin design should be used predominantly in low-rise buildings where the roof and floors can behave as a deep plate girder as shown in Figure 1. It should be noted that diaphragm action will always occur in a building,whether or not it is taken into account in design.

## 1.3 Benefits, Conditions and Restrictions

### Benefits

Some of the benefits of stressed skin design are as follows:

a. Calculated frame stresses and deflections are usually much less than in the bare frame.

b. Calculated and observed stresses and deflections agree, so the design is more realistic.

c. Bracing in the plane of the roof is eliminated or frame sizes are reduced.

d. Frame details are standardised.

e. The method is particularly effective where lateral loads act only on one or two frames, e.g. cross surge from light overhead cranes.

f. By taking diaphragm action into account the actual forces on the cladding and fasteners can be calculated.

### Conditions

In order for steel sheeting to act as a diaphragm the following conditions must be met:

a. End gables must be braced or sheeted.

b. Edge members must be provided to panels and these members and their connections must be designed to carry the flange forces.

c. Sheeting must be fastened to members with positive connections such as self drilling screws, cartridge fired pins or welding.

d. Seams between sheets must be fastened with positive connections.

e. Suitable structural connections must be provided to transmit diaphragm forces into the main framework.

f. It is recommended that the shear stress in the sheets be less than 25% of the ordinary bending stress in the sheets, so that if the sheets are corroded they will fail in bending long before the stressed skin building is endangered.

g. It is recommended that roof light openings should be less than 3% of the relevant roof area unless detailed calculations are made, in which case up to 15% may be allowed.

**Restrictions**

Buildings designed on stressed skin principles should normally be umbrella type structures rather than structures which carry fixed loads. In order to ensure the safety of the building at all times, the following restrictions should be placed on design:

a. Most of the load on the building should be applied via the sheeting itself, e.g. self weight, snow load, wind load.

b. If the sheeting is removed, most of the load will also be removed.

c. Sheeting should not be used for helping to resist other fixed loads, e.g.mechanical plant.

d. Sheeting must be regarded as a structural member and so must not be removed without proper consideration.

e. The calculations and drawings should clearly draw attention to the fact that the building is designed by stressed skin methods.

## 1.4 Types of Diaphragm

Sheeting may span perpendicular to the length of the building (Figure 3) or parallel to the length of the building (Figure 4). Whenever possible each panel of sheeting should be fastened on all four edge members since this gives the greatest diaphragm resistance and stiffness. If all members are not at the same level, "shear connectors" as shown in Figure 5 may be used to provide fastening on all four sides. If this is not possible, diaphragms may be fastened to purlins on two edges only provided that the end panels are fastened on their third side to the end gables. If sheeting is fastened only to the purlins, then the purlin/rafter connections at the intermediate rafters must be adequate to introduce the loads at these rafters into the diaphragm.

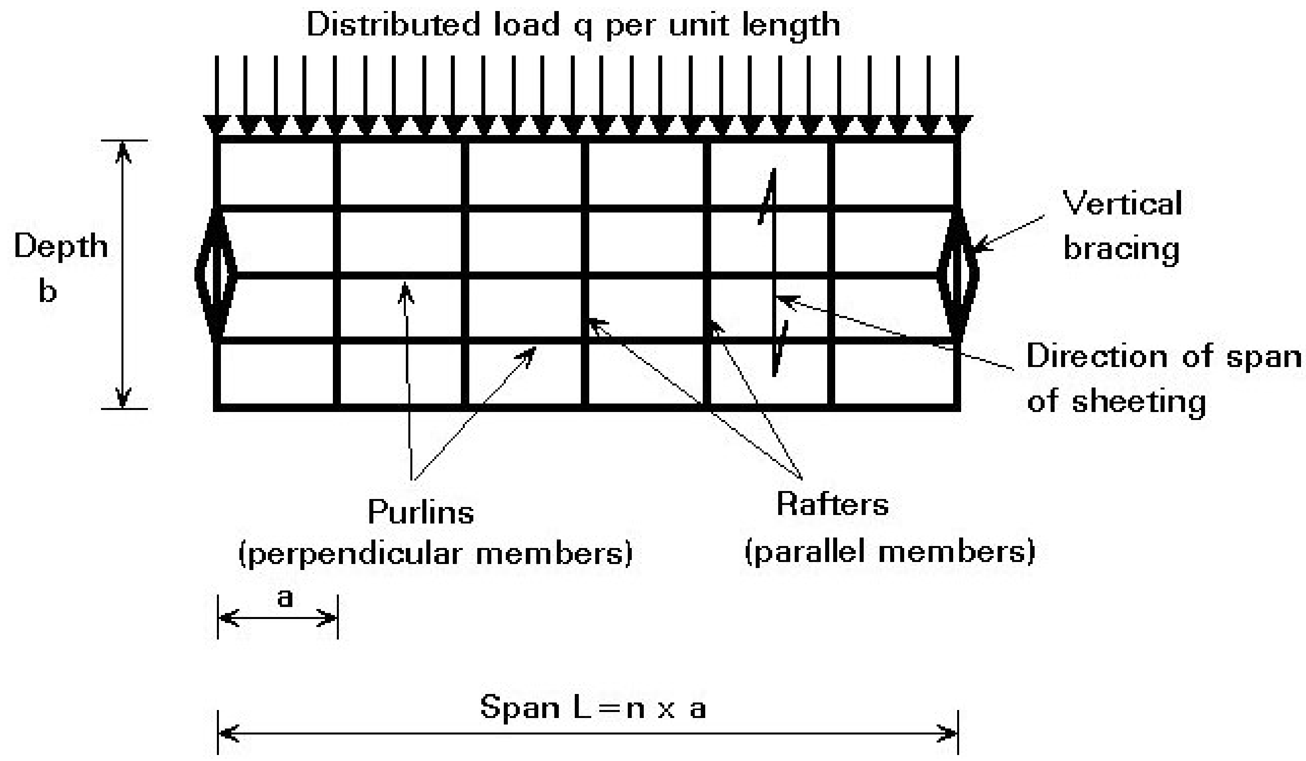


Figure 3 Sheeting spanning perpendicular
to the length of the building

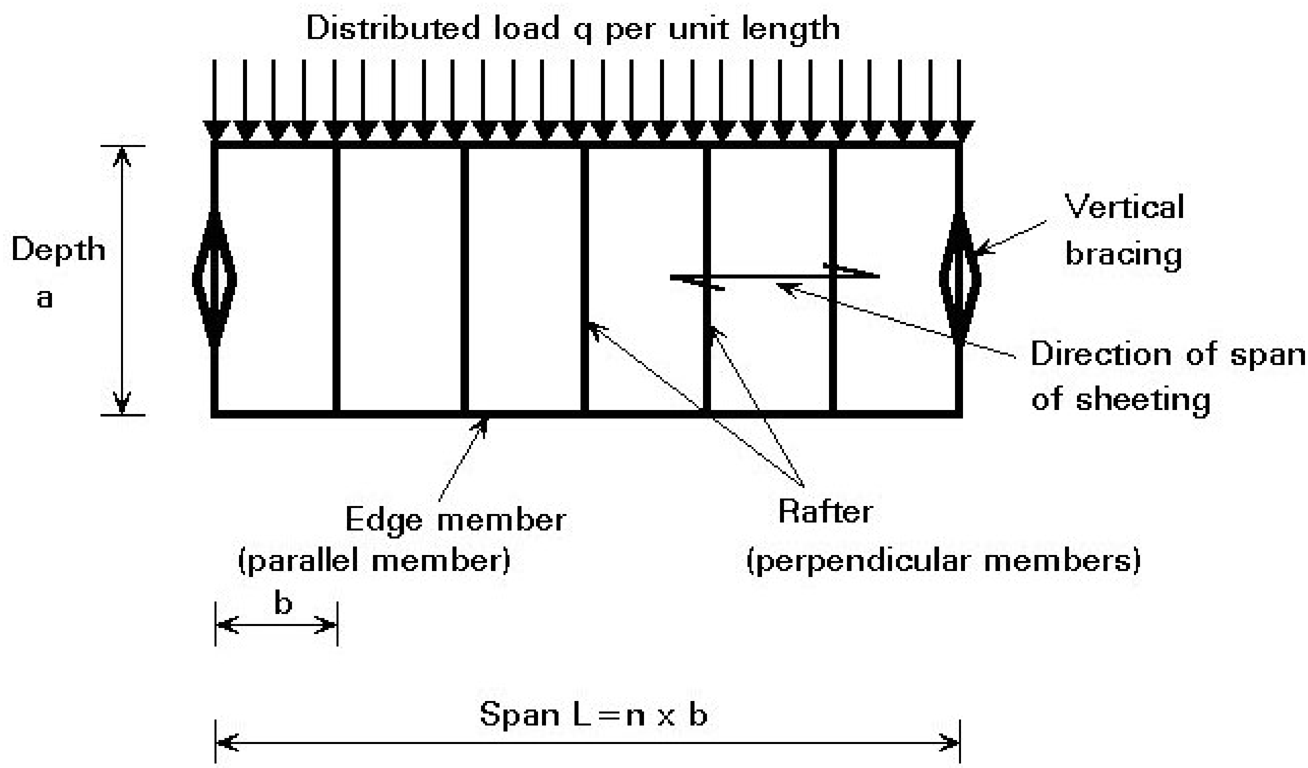


Figure 4 Sheeting spanning parallel
to the length of the building

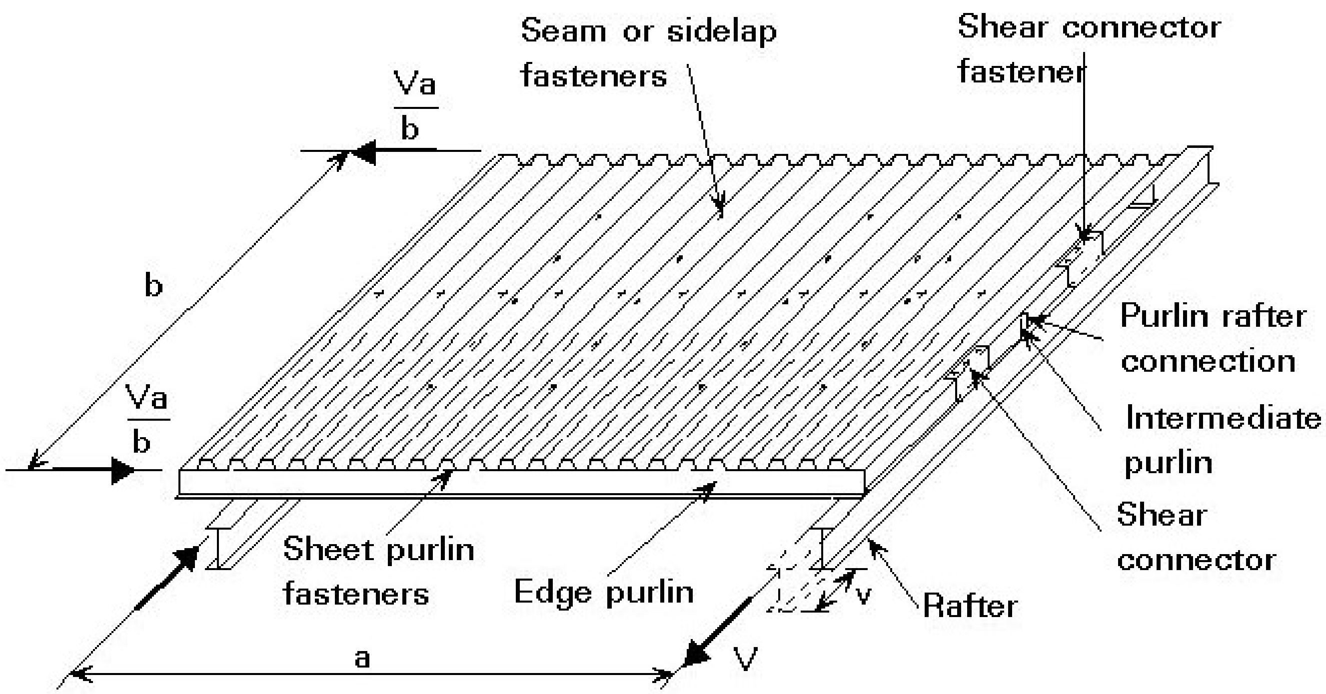


Figure 5 Typical diaphragm panel

The typical diaphragm panel shown in Figure 5 is for sheeting spanning perpendicular to the length of the building. In calculating the shear resistance and flexibility of a panel, the design expressions refer to the direction parallel to the corrugations. For sheeting spanning parallel to the length of the building, a modification to the design expressions must be made. This modification is not considered in this lecture.

# 2. RESISTANCE OF SHEAR DIAPHRAGMS

## 2.1 Principles

For a typical panel attached on all four sides as in Figure 5, the diaphragm resistance $V_R$ in the direction of the load V depends on the failure resistance of:

a. a line of seam fasteners

or

b. a line of shear connector fasteners.

These two failure modes, being ductile, are taken as the design criteria. Any other failure mode, being less ductile, is required to have a considerably greater resistance than the lesser of the above calculated values. Such other modes include failure at the sheet/purlin fasteners, failure of the sheeting due to shear buckling, end collapse of the sheeting profile and failure of the edge members under tension or compression. Because of the low level of shear stress in the sheeting, it is not normally necessary to take diaphragm action into account when designing sheeting for its primary function in bending.

For a typical panel attached to purlins on two edges only (Figure 5 without the shear connectors) an additional design criterion is the tearing resistance of the end sheet/purlin fasteners in the sheeting in an intermediate panel. This case is not considered in this lecture.

## 2.2 Design Expressions

It is not possible in one lecture to derive and explain the design expressions used, but see [1]; instead, the design expressions are presented in this section, and a guide to their use is given in the worked example.

Important note: in the following expressions, design values are used throughout, so that there is no further need to incorporate a material factor.

For a panel attached on all four sides, the expressions for diaphragm resistance are as follows:

Seam resistance

$$V_R = (n_s F_s + \frac{\beta_1}{\beta_3} n_p F_p) \quad (1)$$

Shear connector fastener resistance (at end gables)

$$V_R = (n_{sc} F_{sc}) \quad (2)$$

Shear connector fastener resistance (at internal rafters)

$$(qa) = (n^1_{sc} F_{sc}) \quad (3)$$

In an assembly of panels, Figure 6, V = ½ qa (n-1) so it can be determined whether case (2) or case (3) is more critical. The design shear resistance of the diaphragm V* is then the lesser of the values given by case (1), case (2) or case (3) above.

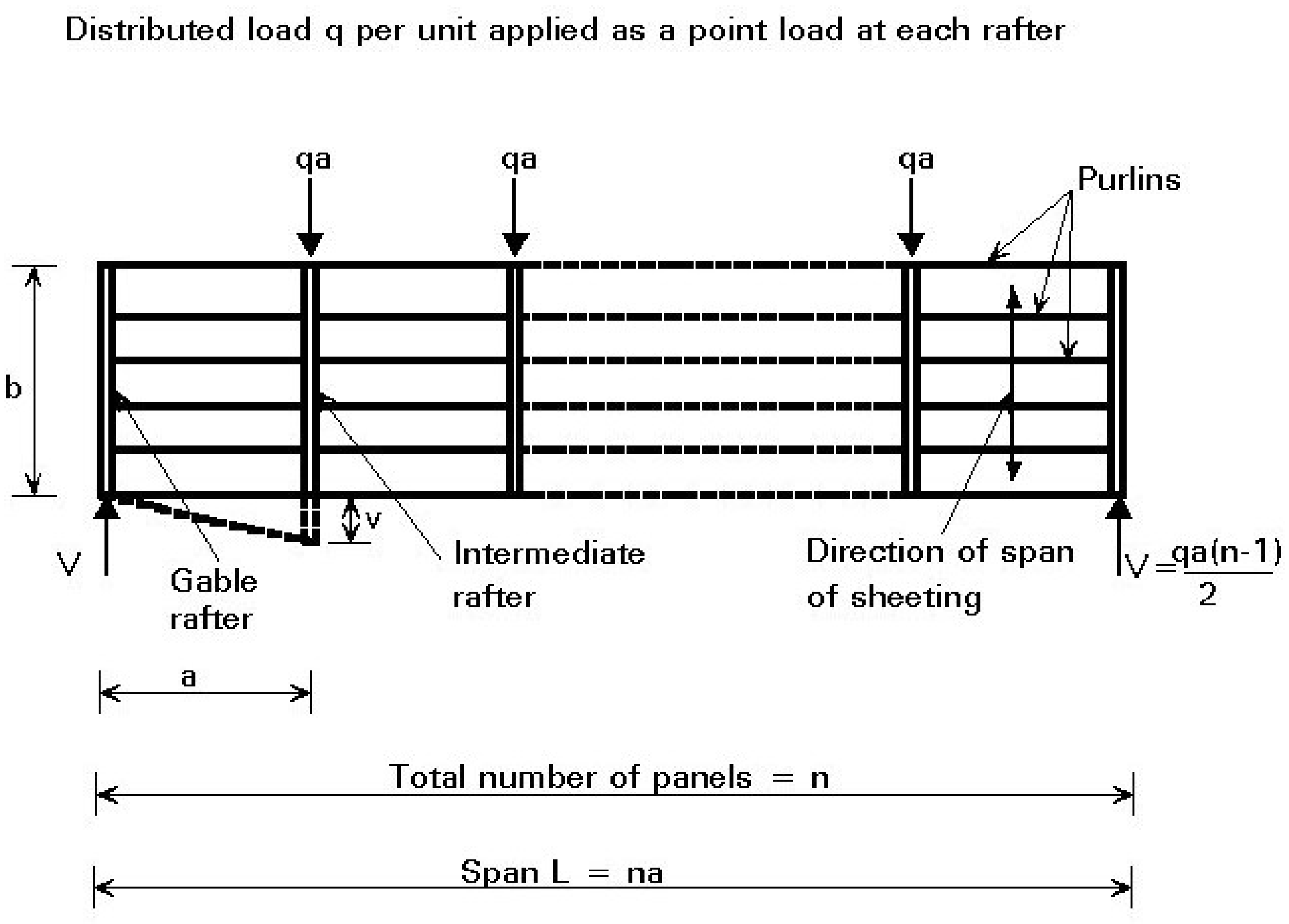


Figure 6 Diaphragm roof - sheeting perpendicular to length of building

In order to avoid the possibility of failure in the sheet/purlin fasteners which may be subject to combined load under wind uplift and shear, and to prying action by the sheeting, a 40% reserve of safety is allowed. It should be checked that

$$0,6bF_p / (p.\alpha_3) \geq V^* \qquad (4)$$

In order to avoid shear buckling of the sheeting, which is a sudden mode of failure, a 25% reserve of safety is allowed. The design expression is given in [1].

In order to avoid gross distortion or collapse of the profile at the end of the sheeting, see [2] and [3], the following limitations on shear force in a panel should be observed:

Every corrugation fastened: $0,9t^{1,5}\ b\ f_y/d^{0,5} \geq V^*$ (5)

Alternate corrugations fastened: $0,3t^{1,5}\ b\ f_y/d^{0,5} \geq V^*$ (6)

In order to avoid failure of the edge members and their connections, especially buckling of the compression flange, a 25% safety reserve is allowed. Referring to Figure 6, the maximum load in an edge member may be taken as $(qL^2.\alpha_3)/8b$.

## 3. FLEXIBILITY OF SHEAR DIAPHRAGMS

## 3.1 Principles

The shear flexibility of a diaphragm, e.g. the panel in Figure 5, is the shear deflection per unit shear load in a direction parallel to the corrugations.

It is therefore the value $\frac{v}{V}$ in Figure 5, or more generally the value of c (mm/kN) shown in Figure 7.

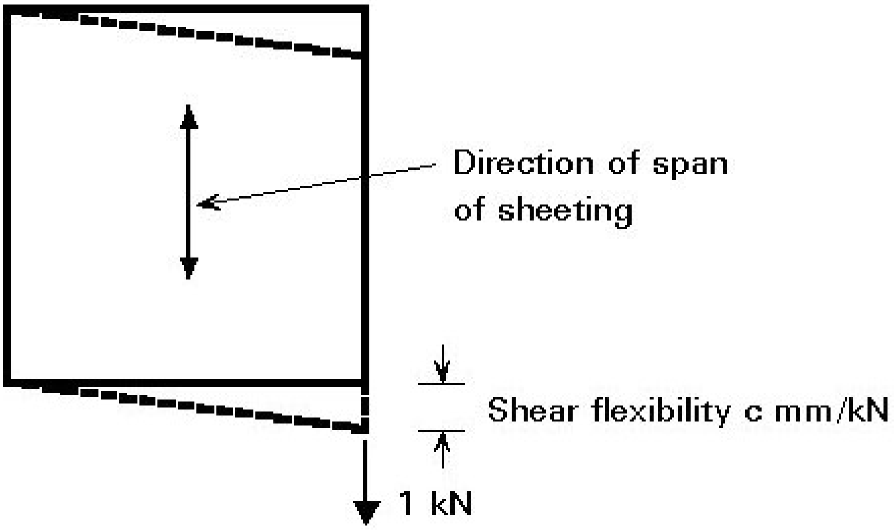


Figure 7 Definition of Shear flexibility

The total shear flexibility of a panel of profiled steel sheeting is the sum of the separate component flexibilities due to the following:

a. profile distortion ($c_{1.1}$)

b. shear strain in the sheet ($c_{1.2}$)

c. slip in the sheet/purlin fasteners ($c_{2.1}$)

d. slip in the seam fasteners ($c_{2.2}$)

e. slip in the sheet/shear connector fasteners ($c_{2.3}$)

f. purlin/rafter connections (in the case of the sheet fastened to the purlins only)

g. axial strain in the longitudinal edge members ($c_3$)

Generally, profile distortion (a) is the largest component flexibility and it is influenced greatly by the sheet thickness, size of profile and especially whether the sheeting is fastened in every corrugation or alternate corrugations. The latter case is much more flexible than the former. Slip in the seam fasteners (d) is often an important component flexibility.

### 3.2 Design Expressions

The design expressions for the various component flexibilities of a panel attached on all four sides are given below. The derivations are given in [1] and a guide to their use is given in the Worked Example 9.1.

a. profile distortion $c_{1.1} = (ad^{2,5}\ \alpha_1\alpha_4 K)/(Et^{2,5}\ b^2)$ (7)

b. shear strain $c_{1.2} = \{2a\ \alpha_2(1 + \nu)[1 + 2h/d]\}/Etb$ (8)

c. sheet/purlin fasteners $c_{2.1} = (2\ as_p\ p\ \alpha_3)/b^2$ (9)

d. seam fasteners $c_{2.2} = \dfrac{2\, s_s\, s_p\, (n_{sh} - 1)}{2 n_s s_p + \beta_1 n_p s_s}$ (10)

e. shear connector fasteners $c_{2.3} = \dfrac{4\,(n+1)\, s_{sc}}{n^2\, n_{sc}^{1}}$ (11)

f. axial strain $c_3 = (n^2\ a^3\ \alpha_3)/(4{,}8\ EAb^2)$ (12)

**Notes** The sheeting constant K can take the value $K_1$ for sheeting fastened in every corrugation (Table 4) or $K_2$ for sheeting fastened in alternate corrugations (Table 5).

The sum of the component shear flexibilities gives the total shear flexibility c of the panel. The midspan deflection of the typical panel assembly, shown in Figure 6, is given by $\Delta = (n^2/8)\ c\ (qa)$.

## 4. APPLICATION OF STRESSED SKIN DESIGN

### 4.1 Shear Diaphragms Alone

If the frames of the flat roofed building in Figure 1 are pinjointed, the roof diaphragm carries all the side loads. The arrangement is as shown in Figure 6. The design criterion for resistance is the end panel, and the design criterion for flexibility is the deflection at midspan. Both of these values must be checked as shown in the Worked Example 9.1.

### 4.2 Shear Diaphragms with Rigid Frames

If the frames of Figure 1 are rigid jointed, the frame flexibility may be defined by k mm/kN as shown in Figure 8. The relative flexibility of the diaphragms to the frames is given by $\psi = c/k$ and the distribution of load between the diaphragms and the frames may be shown to depend on $\psi$, on the number of panels in the length of the building, and on the position of the frame in the building. Table 6 gives the reductions to be applied to the sidesway moments for a small range of values of $\psi$. The application of this table is shown in the Worked Example 9.1.

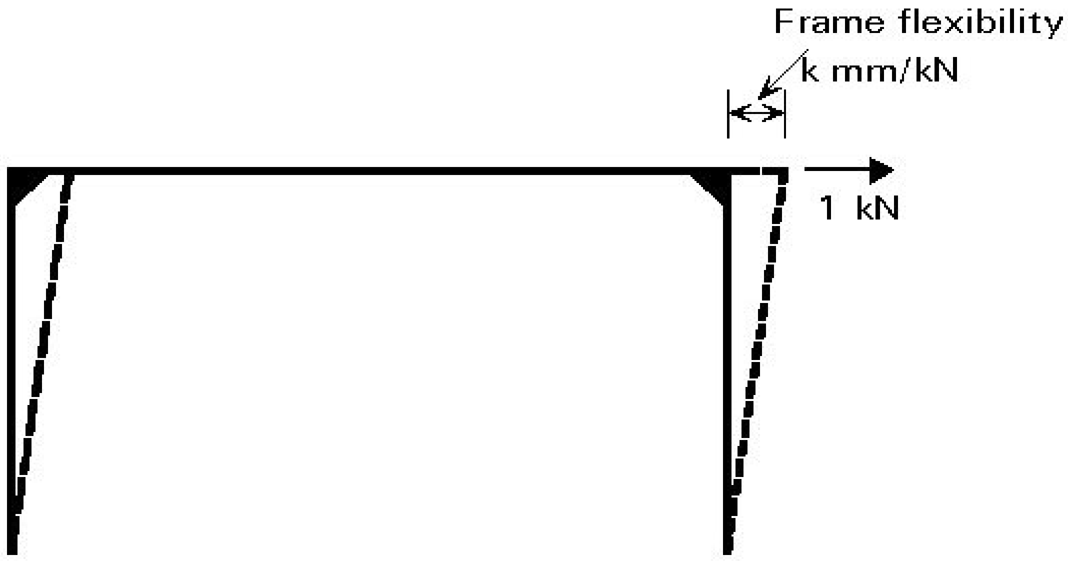


Figure 8 Definition of frame flexibility(k)

### 4.3 Complex Diaphragms

In schools, libraries and similar buildings the flat roof may consist of a number of diaphragms in different directions and at different levels (Figure 9). Each diaphragm must be braced in the end frames, or if one end cannot be braced (a "cantilever" diaphragm) the other three sides must be braced to prevent body rotation of the roof.

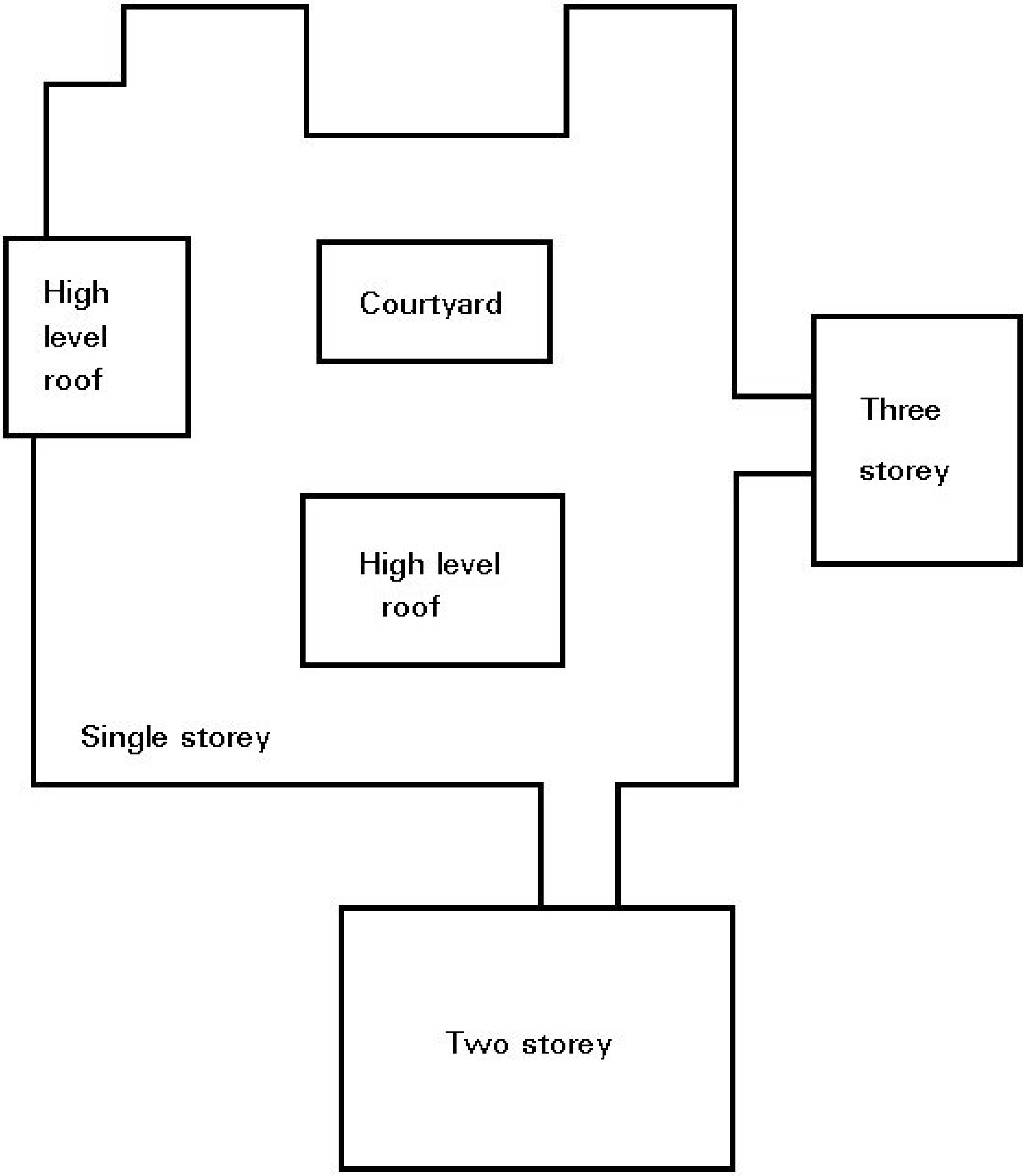


Figure 9 Complex diaphragms

This method of construction has been used in many buildings and it eliminates the need for horizontal bracing in the roof.

### 4.4 Openings in Diaphragms

If a roof has roof lights, particularly if they are in a continuous line, this has the effect of weakening the diaphragm and making it more flexible. Generally, openings should be avoided if possible in the end panels where the shear is greatest. If openings are small and staggered, it is recommended that openings up to 3% of the panel area may be permitted without special calculation. Above this amount, openings up to 15% of the panel area may be allowed if calculations are made for the effect as given in [1].

### 4.5 Diaphragm Bracing

In addition to resisting side load on a building, roof and floor diaphragms may be used to provide horizontal bracing for loads on the end gable of a building, lateral support to the main beams or trusses, and bracing to the eaves of a building. In such cases it is generally only necessary to carry out the calculation for diaphragm resistance and not for diaphragm flexibility.

**End gable bracing**

Load on the end gable, as shown in Figure 10, is usually considered to be taken on the depth of two diaphragms. Vertical bracing must be provided in the side walls. For the case shown, the maximum shear per unit depth in the diaphragms occurs at the ends and is equal to $\frac{qL}{4b}$ kN/mm. If the decking is fastened on all four sides, this shear flow is equal in the x and y directions and the fasteners throughout should be checked to ensure that they are adequate to take this shear.

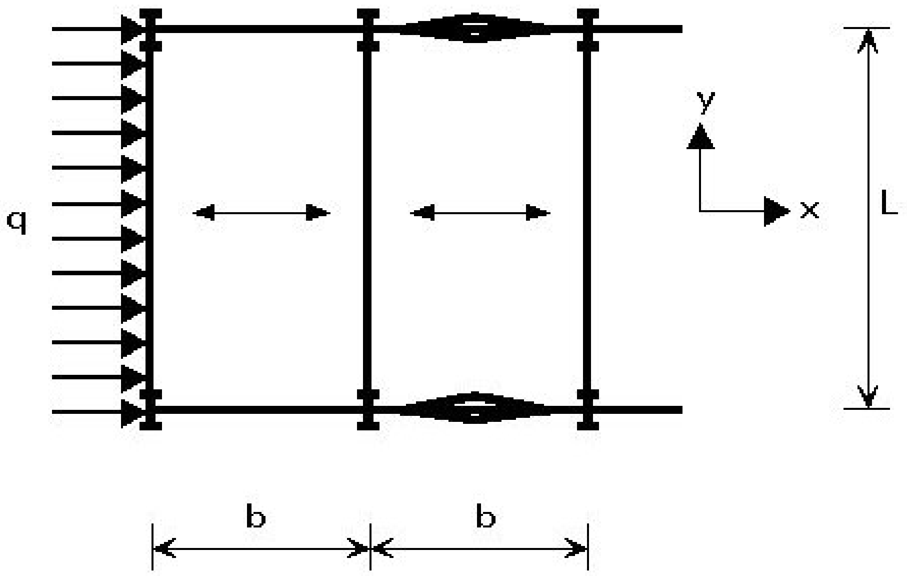


Figure 10 Diaphragm bracing to end gables

**Lateral bracing to beams**

If the decking is supported by main beams or trusses, then the decking may be considered to give lateral support as shown in Figure 11. If the force in the compression flange of the beam is P, then codes of practice specify that the lateral force to be resisted is some 3% times P, distributed along the length of the beam. For a diaphragm of depth b, the maximum shear per unit depth is $\frac{0{,}015P}{b}$, acting in the x and y directions and the fasteners should be checked to ensure that they can take this shear.

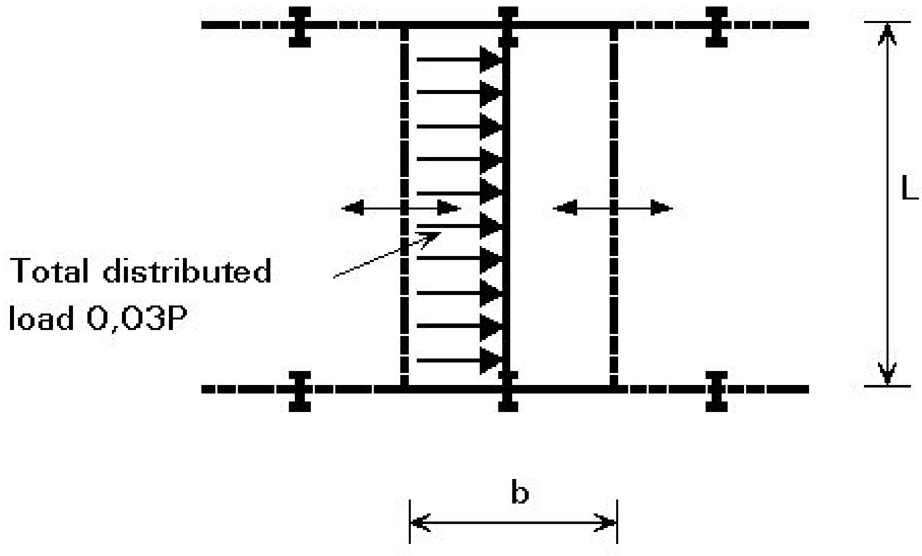


Figure 11 Lateral bracing to beam

It should be noted that if the same sheeting is required to provide bracing to both the gables and the main beams, then the fasteners should be adequate to take the sum of the shears.

**Eaves bracing**

In pitched roof frames, the bottom two purlins are sometimes cross-braced together in order to provide resistance to any horizontal eaves forces between the frames. This function can easily be performed by the sheeting acting as a diaphragm between the bottom one or two purlin spacings.

### 4.6 Simplified Design Method

For common sheeting and decking profiles, fixed in accordance with normal practice, simplified design tables have been calculated by computer for a wide range of diaphragm sizes. These design tables, given in [1], give the shear resistance and deflection of the diaphragms. Although they represent standard diaphragms, the results may be sufficiently accurate for particular cases.

## 5. CONCLUDING SUMMARY

- Stressed skin structures use the cladding to resist lateral load by diaphragm action.
- Stressed skin design is used predominantly in low-rise buildings where the roof and floors behave as deep plate girders.
- For profiled steel sheeting, attached on all 4 sides, the diaphragm resistance depends on the failure strength of a line of seam fasteners or of a line of shear connector fasteners.
- Stressed skin design can be used as the sole horizontal bracing element or in conjunction with rigid frames. Holes in the diaphragm are permitted without justifying calculations, as long as they are small, staggered and less than 3% of the panel area.

## 6. REFERENCES

[1] Davies J.M. and Bryan E.R. "Manual of Stressed Skin Diaphragm Design" Granada Publishing Ltd, London 1982.

[2] Davies J.M. and Fisher J. "End Failures in Stressed Skin Diaphragms" Proceedings Institution of Civil Engineers, Part 2, March 1987.

[3] European Convention for Constructional Steelwork. "European Recommendations for the Stressed Skin Design of Steel Structures," Publication 19, ECCS, 1978.

## 7. ADDITIONAL READING

1. Davies, J. M. "A General Solution for the Shear Flexibility of Profiled Sheets. I: Development and verification of the method. II: Applications of the method". Thin Walled Structures, Vol. 4, 1986 pp 41-68 and 151-161.
2. Maass G. "Stahltrapezprofile: Konstruktion und Berechnung Werner-Verlag GmbH, Düsseldorf 1985.
3. Baehre R. und Wolfram R. "Zur Schubfeldberechnung von Trapezprofilen" Stahlbau 6, 1986.

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

| | Washer type | Overall diameter<br><br>mm | Design resistance kN per mm thickness of sheet | Slip<br><br>mm/kN |
|---|---|---|---|---|
| Screws | Steel | 6,3<br><br>5,5 | 6,0<br><br>5,0 | 0,15 |
| | Neoprene | 6,3<br><br>5,5 | 5,0<br><br>4,0 | 0,35 |
| Fired Pins | | 3,7 - 4,5 | 5,0 | 0,10 |

(2) Seam fasteners (no washers)

| | Overall diameter<br><br>mm | Design resistance kN per mm thickness of sheet | Slip<br><br>mm/kN |
|---|---|---|---|
| Screws | 4,1 - 4,8 | 2,5 | 0,25 |
| Steel or Monel blind rivets | 4,8 | 2,8 | 0,30 |

**Table 1 Resistance and slip values of fasteners**

| Total number of fasteners per sheet width | Factor $\beta_1$ | | Factor $\beta_2$ |
|---|---|---|---|

| $n_f$ | 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 |

### Table 2 Factors to allow for the number of sheet/purlin fasteners per sheet width

| Total number of purlins per panel (or per sheet length for $\alpha_1$) | Correction factors | | |
|---|---|---|---|
| $n_p$ | $\alpha_1$ | $\alpha_2$ | $\alpha_3$ |
| 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 |

### Table 3 Factors to allow for the effect of intermediate purlins

| | l/d<br>h/d | 0,1 | 0,2 | 0,3 | 0,4 | 0,5 | 0,6 | 0,7 | 0,8 | 0,9 |
|---|---|---|---|---|---|---|---|---|---|---|
| $\theta$ = 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 | | | |
| $\theta$ = 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 | 0,431 | | |
| | 0,5 | 0,356 | 0,372 | 0,315 | 0,270 | 0,364 | 0,725 | 0,931 | | |
| | 0,6 | 0,448 | 0,420 | 0,326 | 0,303 | 0,512 | | | | |
| θ = 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 | | | | | |
| θ = 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 | | | | | | |

## Table 4 Sample values of $K_1$ for fasteners in every trough ($15° \leq \theta \leq 30°$)

| | l/d<br>h/d | 0.1 | 0,2 | 0,3 | 0,4 | 0,5 | 0,6 | 0,7 | 0,8 | 0,9 |
|---|---|---|---|---|---|---|---|---|---|---|
| θ = 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 | | | |
| θ = 20° | 0,1 | 0,096 | 0,144 | 0,190 | 0,232 | 0,273 | 0,315 | 0,368 | 0,459 | 0,680 |
| | 0,2 | 0,339 | 0,472 | 0,597 | 0,716 | 0,832 | 0,966 | 1,177 | 1,659 | |
| | 0,3 | 0,743 | 0,978 | 1,204 | 1,416 | 1,633 | 1,927 | 2,481 | | |
| | 0,4 | 1,317 | 1,673 | 2,009 | 2,325 | 2,679 | 3,246 | 3,840 | | |
| | 0,5 | 2,075 | 2,559 | 3,011 | 3,436 | 3,993 | 4,969 | | | |
| | 0,6 | 3,006 | 3,625 | 4,194 | 4,752 | 5,588 | | | | |
| θ = 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 | | | | | |
| θ = 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 | | | | | | |

### Table 5 Sample values of $K_2$ for fasteners in alternate troughs ($15^\circ \leq \theta \leq 30^\circ$)

| No, of frames in building | Frame number | VALUES OF RELATIVE FLEXIBILITY $\psi$ | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | 0,25 | 0,30 | 0,35 | 0,40 | 0,45 | 0,50 | 0,60 | 0,70 | 0,80 | 0,90 | 1,00 | 1,50 |
| 3 | 2 | 0,111 | 0,130 | 0,149 | 0,167 | 0,184 | 0,200 | 0,231 | 0,259 | 0,286 | 0,310 | 0,333 | 0,429 |
| 4 | 2 | 0,200 | 0,231 | 0,259 | 0,286 | 0,310 | 0,333 | 0,375 | 0,412 | 0,444 | 0,474 | 0,500 | 0,600 |
| 5 | 2 | 0,265 | 0,301 | 0,333 | 0,362 | 0,388 | 0,412 | 0,454 | 0,490 | 0,521 | 0,548 | 0,571 | 0,659 |
| | 3 | 0,347 | 0,392 | 0,432 | 0,468 | 0,500 | 0,529 | 0,580 | 0,622 | 0,658 | 0,688 | 0,714 | 0,805 |
| 6 | 2 | 0,310 | 0,347 | 0,379 | 0,407 | 0,432 | 0,455 | 0,494 | 0,526 | 0,554 | 0,579 | 0,600 | 0,677 |
| | 3 | 0,448 | 0,497 | 0,540 | 0,576 | 0,608 | 0,636 | 0,684 | 0,721 | 0,752 | 0,778 | 0,800 | 0,871 |
| 7 | 2 | 0,340 | 0,375 | 0,406 | 0,432 | 0,456 | 0,477 | 0,513 | 0,543 | 0,569 | 0,591 | 0,611 | 0,683 |
| | 3 | 0,515 | 0,563 | 0,604 | 0,638 | 0,667 | 0,692 | 0,734 | 0,767 | 0,793 | 0,815 | 0,833 | 0,892 |
| | 4 | 0,569 | 0,620 | 0,663 | 0,698 | 0,728 | 0,754 | 0,795 | 0,827 | 0,852 | 0,873 | 0,889 | 0,938 |
| 8 | 2 | 0,359 | 0,393 | 0,421 | 0,447 | 0,469 | 0,488 | 0,522 | 0,551 | 0,575 | 0,597 | 0,615 | 0,685 |
| | 3 | 0,558 | 0,603 | 0,641 | 0,672 | 0,698 | 0,721 | 0,758 | 0,787 | 0,811 | 0,830 | 0,846 | 0,898 |
| | 4 | 0,646 | 0,695 | 0,734 | 0,765 | 0,792 | 0,814 | 0,849 | 0,875 | 0,895 | 0,911 | 0,923 | 0,959 |
| 9 | 2 | 0,371 | 0,403 | 0,430 | 0,454 | 0,475 | 0,494 | 0,527 | 0,554 | 0,578 | 0,599 | 0,617 | 0,686 |
| | 3 | 0,585 | 0,627 | 0,662 | 0,690 | 0,715 | 0,733 | 9,770 | 0,796 | 0,818 | 0,836 | 0,851 | 0,901 |
| | 4 | 0,695 | 0,739 | 0,774 | 0,802 | 0,825 | 0,844 | 0,874 | 0,896 | 0,913 | 0,926 | 0,936 | 0,966 |
| | 5 | 0,729 | 0,773 | 0,808 | 0,835 | 0,857 | 0,875 | 0,903 | 0,923 | 0,938 | 0,949 | 0,957 | 0,981 |
| 10 | 2 | 0,379 | 0,409 | 0,436 | 0,458 | 0,479 | 0,497 | 0,529 | 0,556 | 0,579 | 0,599 | 0,618 | 0,686 |
| | 3 | 0,602 | 0,641 | 0,673 | 0,700 | 0,723 | 0,743 | 0,775 | 0,800 | 0,821 | 0,838 | 0,853 | 0,901 |
| | 4 | 0,725 | 0,766 | 0,797 | 0,822 | 0,843 | 0,860 | 0,886 | 0,906 | 0,920 | 0,932 | 0,941 | 0,968 |
| | 5 | 0,780 | 0,820 | 0,850 | 0,873 | 0,891 | 0,904 | 0,929 | 0,944 | 0,956 | 0,964 | 0,971 | 0,987 |

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

### Table 6 Reduction factor on sway forces and moments for each frame in a clad building - all frames loaded, $0{,}25 < \psi < 1{,}50$

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**ESDEP WG 9**

**THIN WALLED CONSTRUCTION**

# Lecture 9.6: Connections in Thin-Walled Construction

### OBJECTIVE/SCOPE

To provide general information on problems related to the design of connections when thin-walled members or elements are used.

### PRE-REQUISITES

Lecture 3.4: Welding Processes

Lecture 11.1.2: Introduction to Connection Design

Lecture 11.3.1: Connections with Non-Preloaded Bolts

### RELATED LECTURES:

Lecture 11.5: Simple Connections for Buildings

Lectures 14.1.1: Single Storey Buildings: Introduction and Primary Structure

Lecture 14.1.2: Single Storey Buildings: Envelope and Secondary Structure

### SUMMARY

Practical treatment of the main aspects of connection design in thin-walled steel sections is given, including: Types of fasteners and connections, structural and non-structural requirements, factors affecting the forces on the connection as well as the distribution of forces in the connection, and failure modes of fasteners.

## 1. INTRODUCTION

Connections play an important role in structural design. They influence significantly the performance of the structure as well as its cost. A proper selection and design of frame connections may make a substantial contribution to the competitiveness of steelwork. This fact was recently recognised by Eurocodes 3 [1] which introduced realistic connection models and gave basic specifications to account for connection response in design. With reference to lightweight structures comprising cold-formed steel sections, the connections may play a substantial role in the behaviour of certain typical forms of construction, such as for example, in systems designed by the stressed skin design method presented in Lecture 9.5.

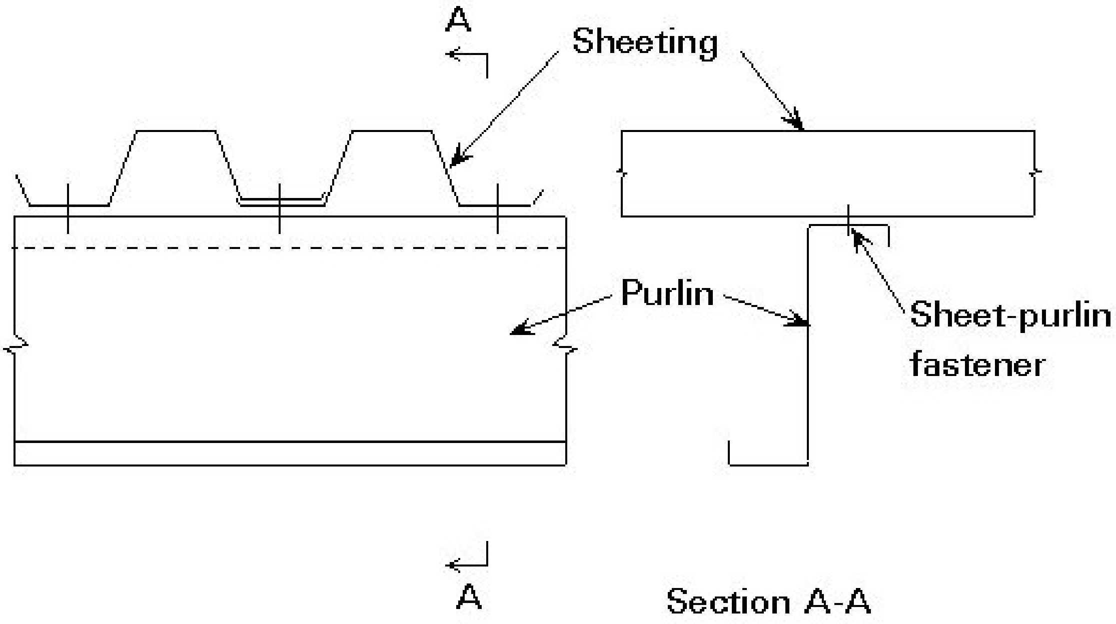
A variety of joining methods between cold-formed sections is available; most of them are specific to thin steel, though fasteners generally used for hot rolled sections, i.e. for thicker plates, may also be used provided that differences in behaviour are taken into account.

This lecture has the main purpose of:

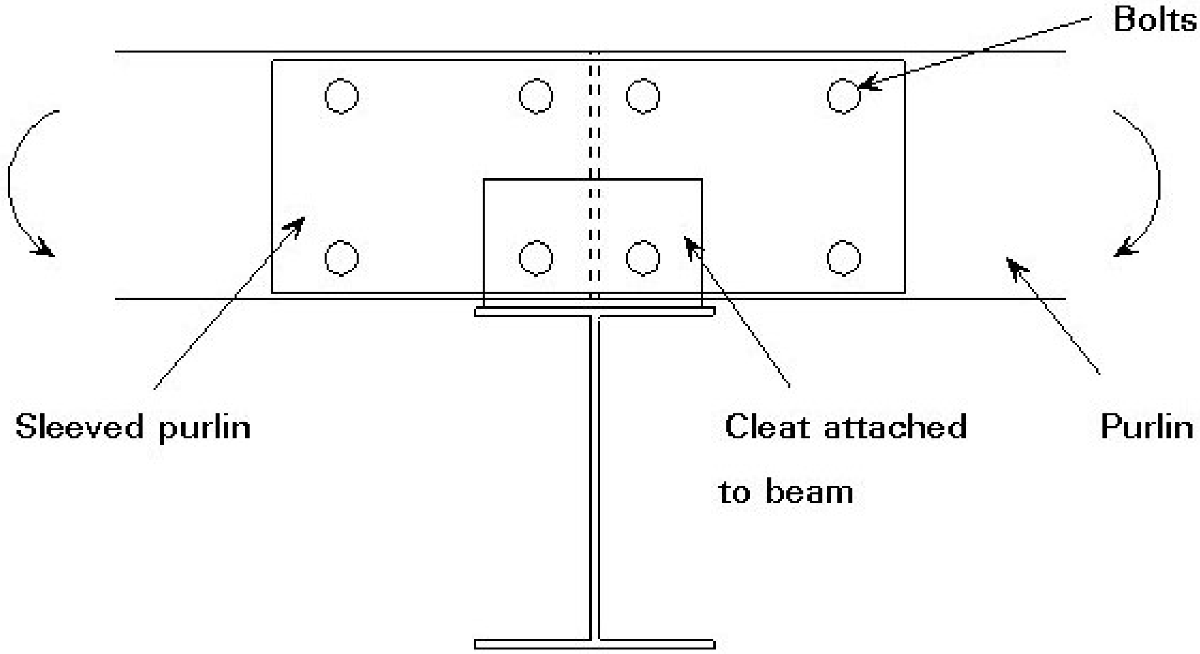
- providing a survey of the most used fastening methods.
- illustrating structural and non-structural requirements.
- presenting the general criteria and procedures for design.

## 2. CONNECTION TYPES

A relatively large number of connections of different types are needed in building construction using cold-formed steel sections. An attempt to identify the main connection types encountered in these structural systems is illustrated in Figure 1. Connections between sheeting and members are of great importance for frames designed by the stressed skin design method, while connections using sleeved or overlapping purlins are typical of lightweight roofing. Increasingly, thin steel sections are used in building frames and beam to column and beam to beam connections have to be designed.

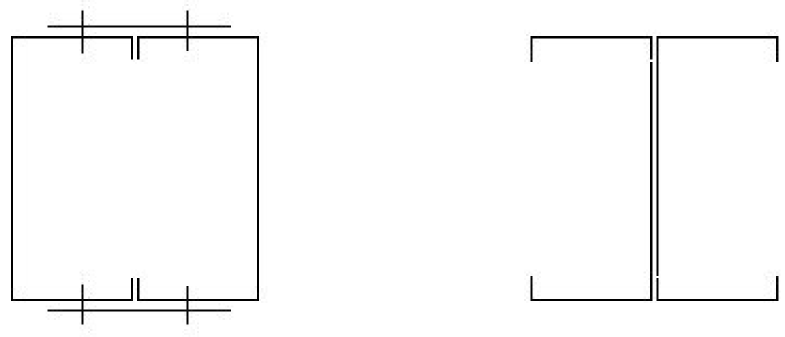



(a) Sheet-purlin fasteners

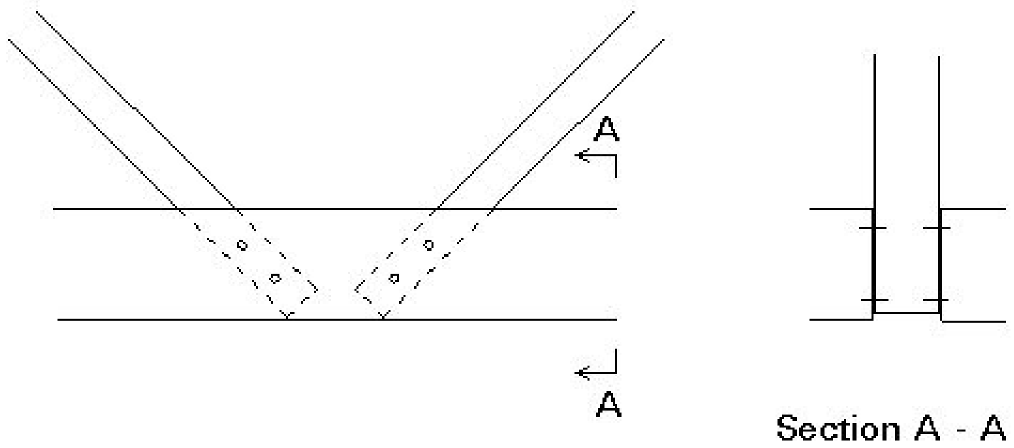



(b) Purlin-purlin attachment by sleeved or overlapping purlin

Figure 1 a-b Examples of connection between cold-formed steel members and sheeting



(c) Built-up members




(d) Truss or panel connection

Figure 1 c-d Examples of connection between cold-formed steel members and sheeting

The important aspects of joining cold-formed sections and/or sheeting are:

- reduced bearing resistance of thin steel.
- avoidance of local buckling.
- connections often made from one side only (as in roof sheeting).

## 3. TYPES OF FASTENERS

Two main categories of fastening may be identified:

1. Fastening by means of mechanical fasteners (bolts, screws ...).
2. Fastening by means of welding.

Both categories will be reviewed briefly in the following sections.

## 3.1 Mechanical Fasteners

Different types of mechanical fasteners as well as their general field of application are presented in Table 1. The guidelines of the manufacturers of the fasteners will provide information concerning how to treat attachments to "thin" and "thick" steel sections.

General information about the use of each type of fastener is given below, in order to provide background for proper selection and use.

**Bolts with nuts**

Bolts with nuts are threaded fasteners which are assembled in preformed holes through the elements to be joined. Thin members will necessitate the use of fully threaded bolts.

For thin-walled sections the bolt diameter range is usually from 5 to 16mm: the preferred bolt Classes are 8.8 or 10.9. High strength slip resistance bolts are not recommended for total thicknesses less than 8mm due to loss of preload in the bolts due to the creep of the zinc layer.

**Screws**

Two main types of screws can be distinguished:

a. self tapping screws: thread forming screws and thread cutting screws;

b. self drilling screws.

Most of the screws will be combined with washers to improve the load-bearing resistance of the fastening and/or to make the fastening self-sealing.

Some types are available with plastic heads or plastic caps for additional corrosion resistance and colour matching.

Figure 2 shows the thread-types for thread forming screws: type A is used for fastening thin to thin sheets, type B for fixings to steel elements of a thickness greater than 2mm, type C for fixings to thin steel elements of a thickness up to 4mm.

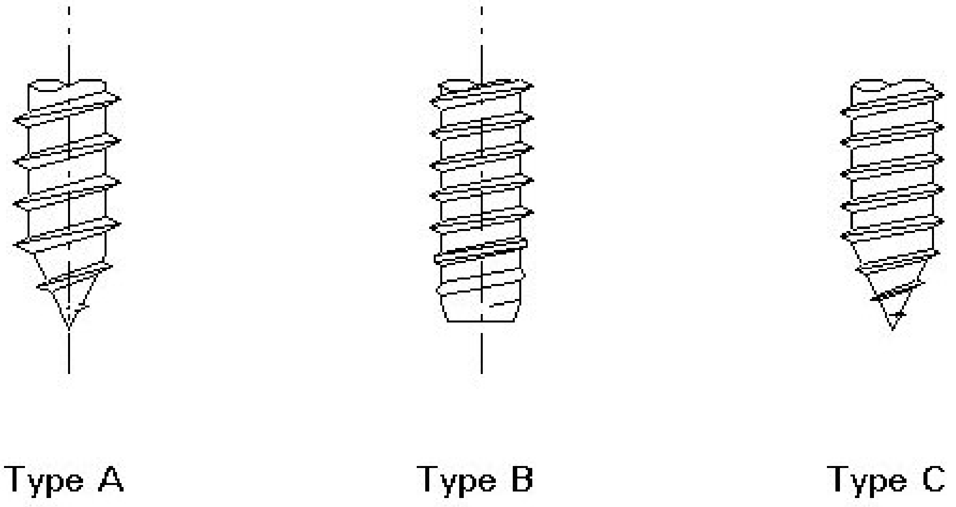



Figure 2 Thread-types for thread forming screws

Thread forming screws normally are fabricated from carbon steel (plated with zinc for corrosion protection and lubrication) or stainless steel (plated with zinc only for lubrication).

Figure 3 shows some examples of thread and point-of-thread cutting screws. Thread cutting screws have threads of machine screw diameter-pitch combinations with a blunt point, and tapered entering threads have one or more cutting edges and chip cavities.

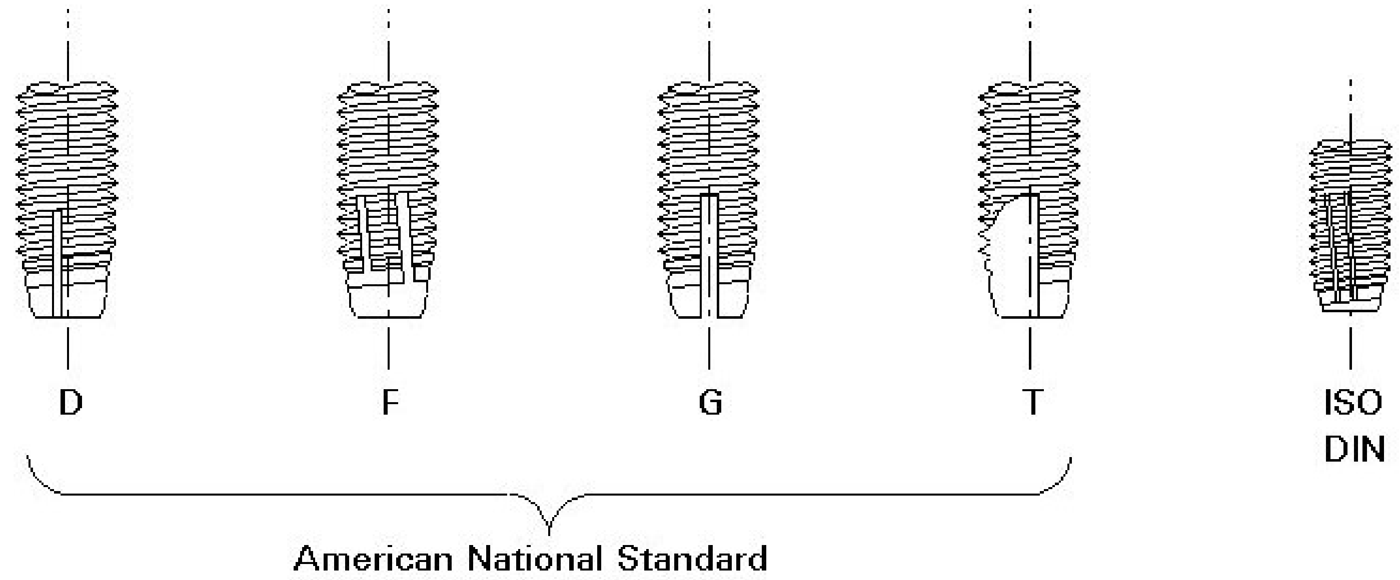



Figure 3 Examples of thread and point - of - thread cutting screws

Thread cutting screws are used for fastening to thicker metal elements. Resistance to loosening is normally not so high for thread cutting screws as for thread forming screws. Thread cutting screws are fabricated from carbon steel case hardened and normally plated with zinc for corrosion and lubrication. Self drilling screws drill their own hole and form their mating thread in one operation. Figure 4 shows two examples of self-drilling screws. Self drilling screws are normally fabricated with carbon steel heat treated (plated with zinc) or with stainless steel (with carbon steel drill point).

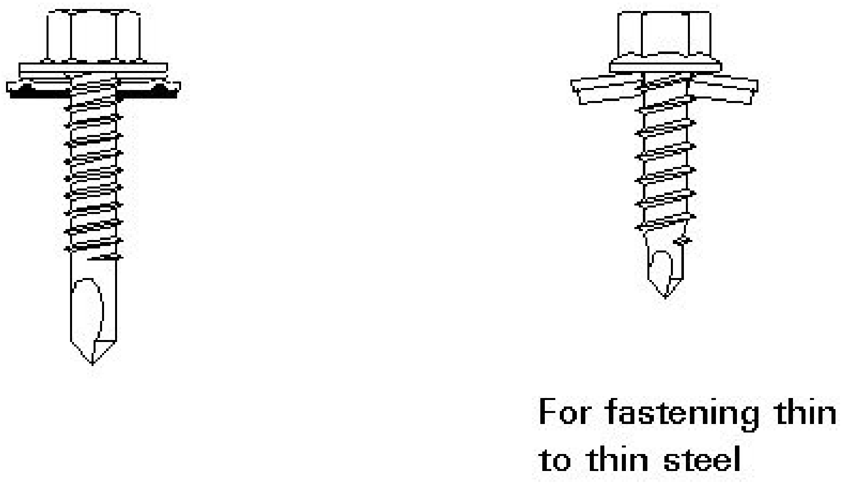



Figure 4 Examples of self-drilling screws

**Blind rivets**

A blind rivet is a mechanical fastener capable of joining work-pieces together where access to the assembly is limited to one side only. They are installed in pre-drilled holes and are used for thin to thin fastenings. Blind rivets are available in aluminium alloys, monel (nickel-copper alloy), carbon steel, stainless steel and copper alloy.

Figure 5 shows different types of blind rivets.

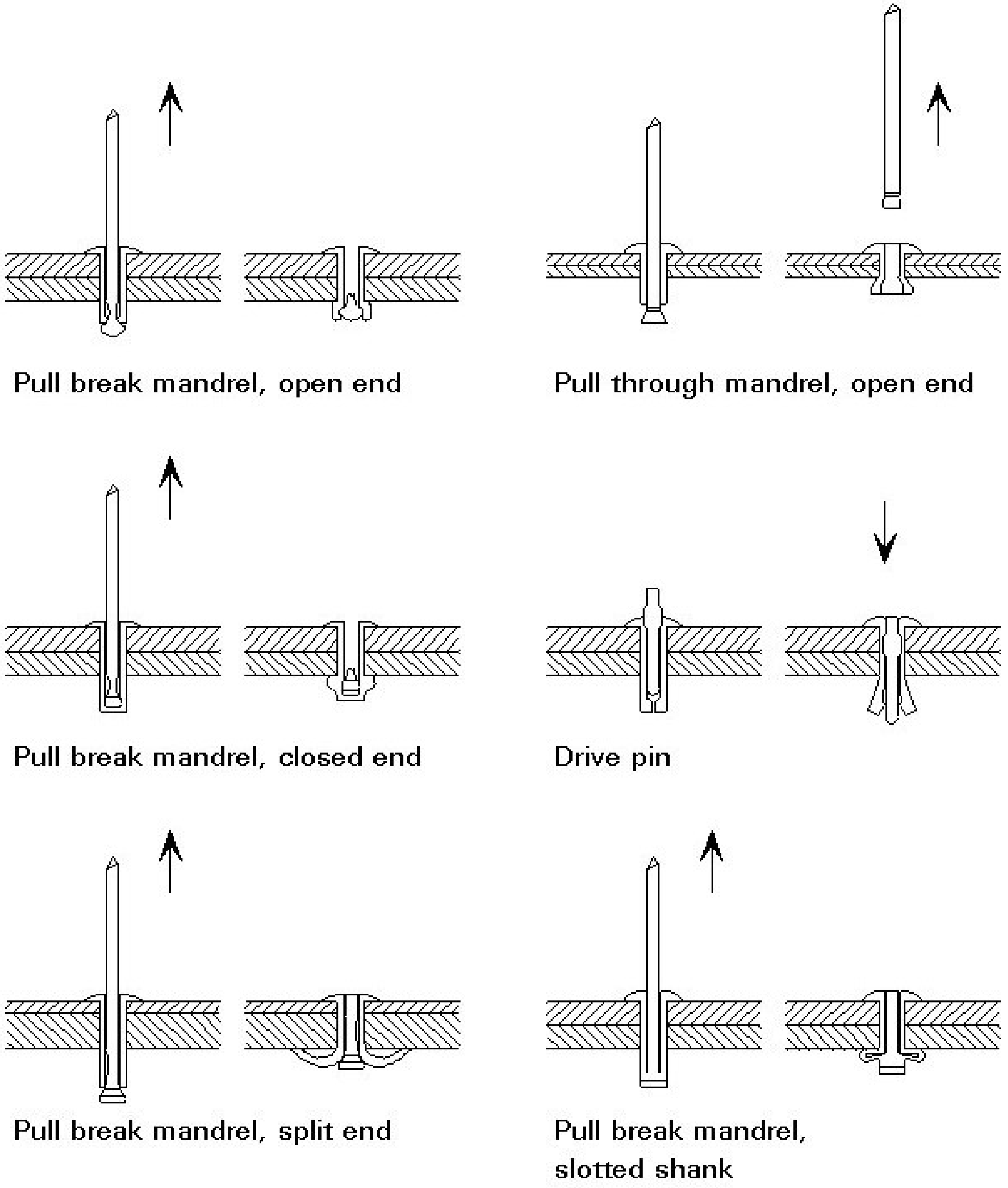



Figure 5 Different types of blind rivets

## Shot fired pins

Shot fired pins are fasteners driven through the element to be fastened into the base metal structure. Depending on the type of driving energy they can be grouped as:

- powder actuated fasteners which are placed with tools which use cartridges filled with propellant which will be ignited.
- air driven fasteners which are placed with tools that act on compressed air.

Figure 6 shows examples of shot fired pins.

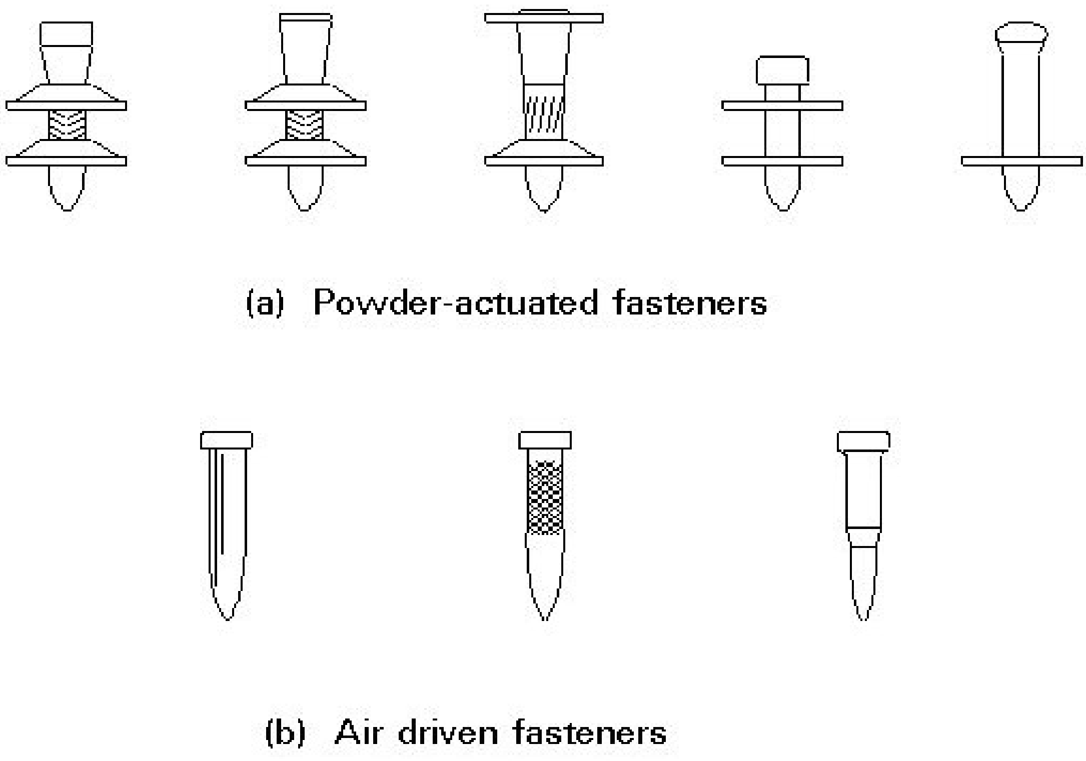



Figure 6 Examples of shot fired pins

**Seam locking**

Seam locking (see Table 1) in structural application will be mainly used as longitudinal connection between adjacent roof sheets.

## 3.2 Welds

In lightweight construction resistance welding is generally used besides more conventional arc welding techniques (Electrode, Gas Metal Arc, Tungsten Inert Gas Welding).

Main types of resistance welding are:

- spot welding.
- seam welding.
- projection welding.

These techniques are illustrated in Figure 7.

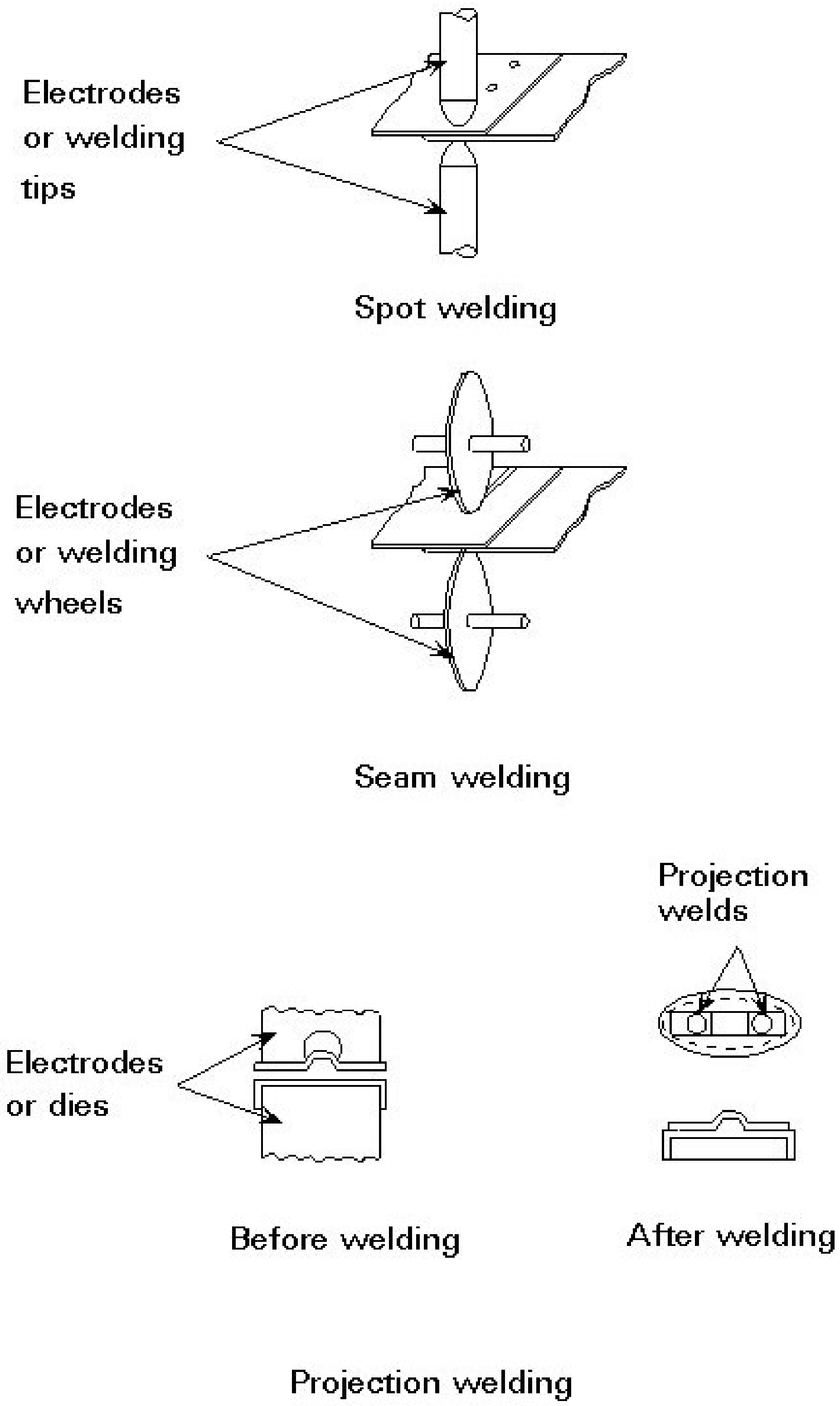



Figure 7 Spot,seam and projection welding processes

Basically resistance welding involves a co-ordinated application of electric current and mechanical pressure of the proper magnitude and duration and a proper surface of the steel sheet. Resistance welding is also possible for zinc coated material, but the welding parameters differ from those for uncoated material.

Both types of welding (arc and resistance welding) can be used for connecting either thin to thin elements or thin to thick elements.

# 4. CONNECTION DESIGN

## 4.1 General Requirements

Structural and non-structural requirements should be considered for an effective and reliable design of connections. The former will mainly be accounted for in sizing and checking the connection, as well as when defining the most appropriate details, whilst the latter should be referred to when selecting the most appropriate fastening type for the specific case. A list of the most important non-structural requirements is provided in Table 2.

Structural requirements can be summarised by the main features the connection behaviour must fulfil, i.e. stiffness, strength and deformation capacity (see Figure 8), described as follows:

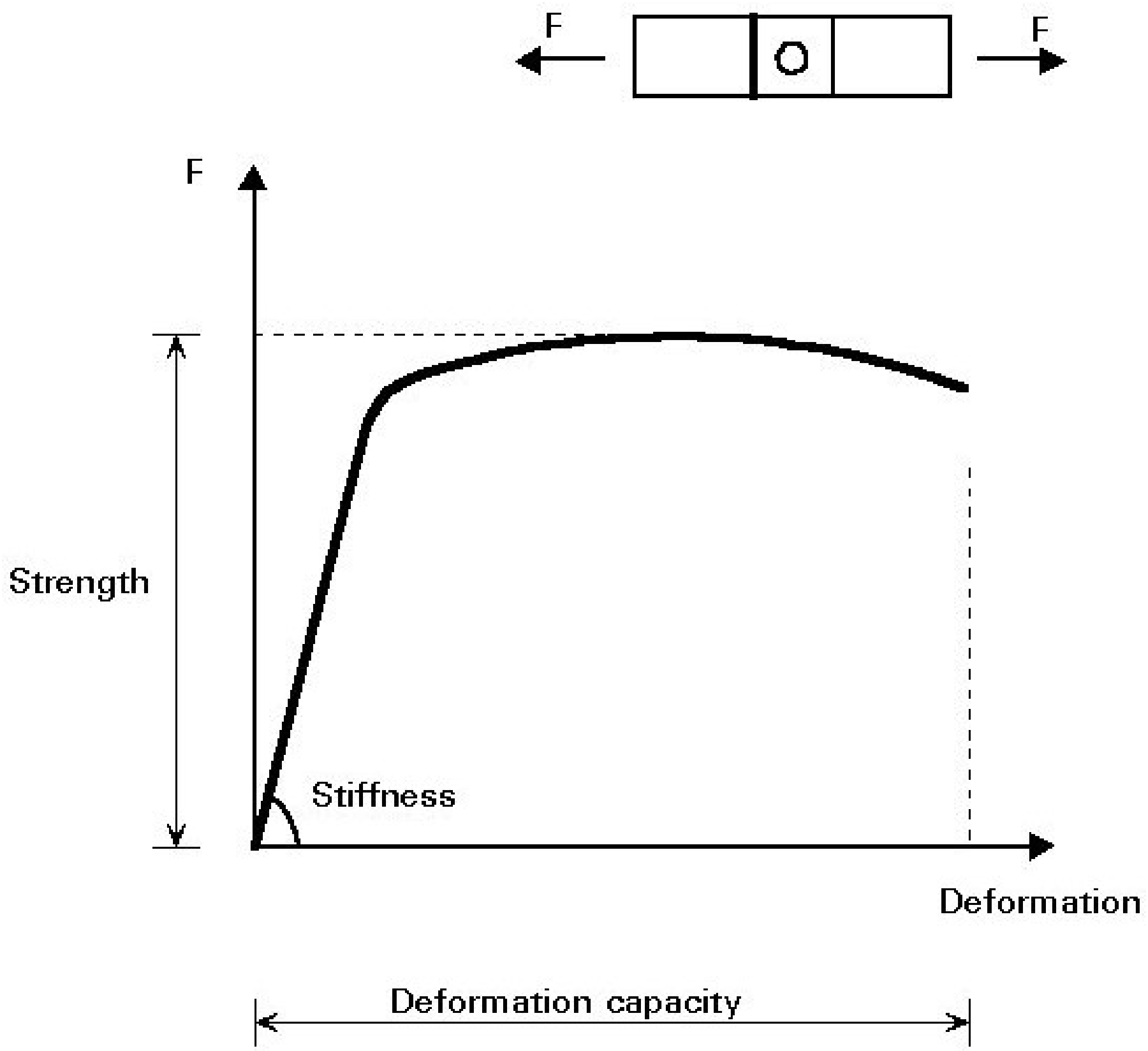



Figure 8 Main features of connection response

a. **Stiffness**

The stiffness of a connection is important because it determines the stiffness and hence deflection of the whole structure or of its components. Moreover the stiffness of the connections will influence the force distribution within the structure. Especially when the connection is a part of a bracing structure, then the stiffer the connection the lower the bracing force will be.

Special systems are available where cold-formed sections interlock to form a connection with a good bending and shear stiffness.

b. **Strength**

Connection strength ensures the capability of resisting forces and moments determined by the analysis of the structure subject to the combinations of actions related to the ultimate limit state condition.

The strength of the connection mainly depends on:

1. the type of fasteners, and

2. the properties of the connected elements (thickness, yield stress).

A reliable assessment of the strength can be achieved in many cases only by testing. However, Eurocode 3: Part 1.3 [1] provides formulae to determine shear and tension resistance of most common fastener types, together with the range of applicability.

Connections between thin elements, e.g. trapezoidal sheeting, are sensitive to repeated loads when they are working in tension. Eurocode 3: Part 1.3 covers this case also by increasing the $\gamma_M$ factor in presence of dynamic loads comparable to wind load.

c. **Deformation capacity**

Deformation capacity is required in order to allow local redistribution of forces without detrimental effects. Otherwise brittle fracture might be caused by local overloading. A proper detailing and fastener selection is vital in order to ensure sufficient deformation capacity to the connection.

The main modes of failure for different types of fasteners are presented briefly in Section 4.3.

## 4.2 Forces in the Connections

Forces and moments, due to the response of the whole structure to design loads, are resisted by the connection through shear and tension forces induced by the individual fasteners.

Basically each fastener will be subject to forces which depend on:

- forces and moments applied to the connection.
- stiffness of the jointed elements.
- stiffness and deformation capacity of the fastenings.

It is useful also to distinguish between:

- Primary forces - forces which are directly caused by the load.
- Secondary forces - forces which are indirectly caused by the load and

which may be neglected in the presence of sufficient deformation capacity in the fastening.

Two types of connections are now considered in more detail, in order to highlight several aspects related to the force distribution between connected members.

a. **Connections in thin-walled sections**

· Consider two similar sections attached together so that, in order to develop their combined strength, the connections are loaded in shear (see Figure 9): The maximum shear force in the fastenings occurs at the ends of the span and is calculated from the formula:

$$S_A = a\frac{VAy}{I}$$

where

$S_A$ is the sum of shear forces in both fastenings in a cross-section A.

a is the distance between the fasteners in the span direction.

A is the area of one section.

V is the vertical shear force at the support.

y is the distance of the centroid of the area of one section to the neutral axis of the composite beam.

I is the moment of inertia of the combined sections.

The calculation method shown gives an upper limit to the shear force in the fasteners. In reality some slip in the fasteners will occur. This causes a smaller section modulus and moment of inertia of the composite beam leading to slight increase in deflections.

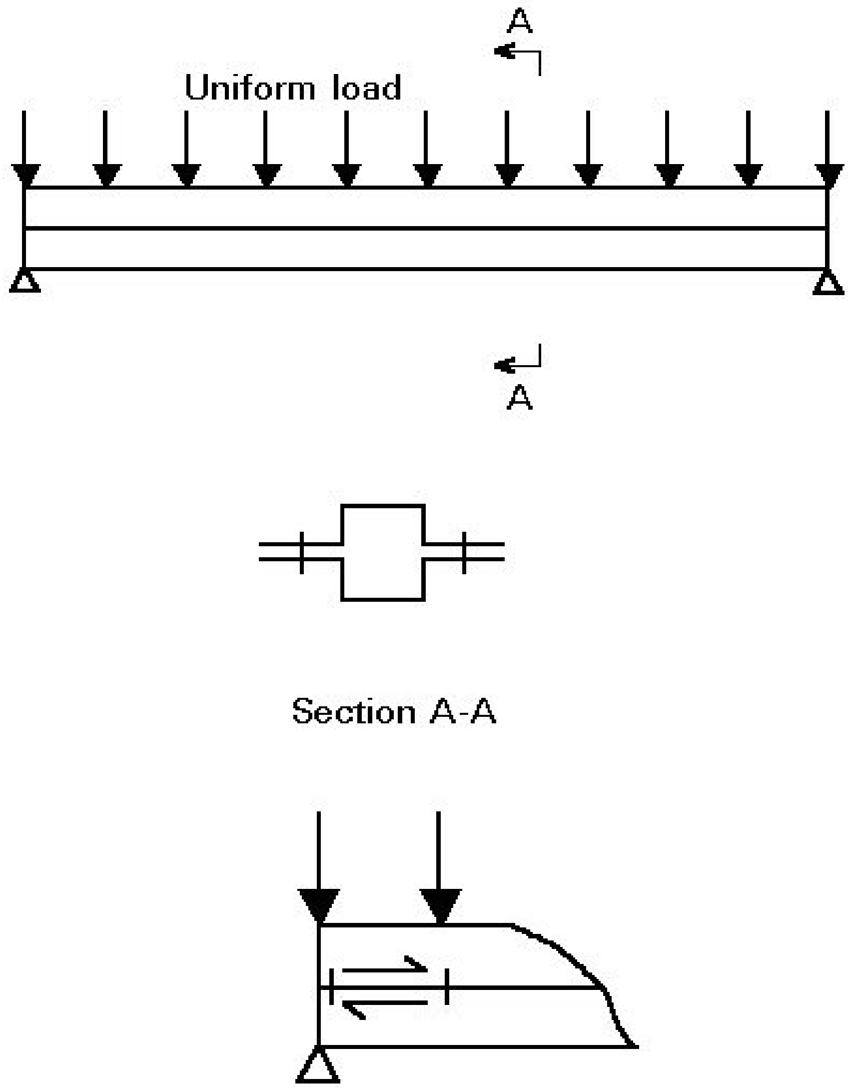



Figure 9 Shear forces in the connectors between two sections acting in combination

Consider next an I-beam made from single C-sections as shown in Figure 10. Because each C-section would twist if not connected, tension forces occur between the C-sections when connected. The tension force T in the upper bolts can be determined knowing the shear centre of the C-sections.

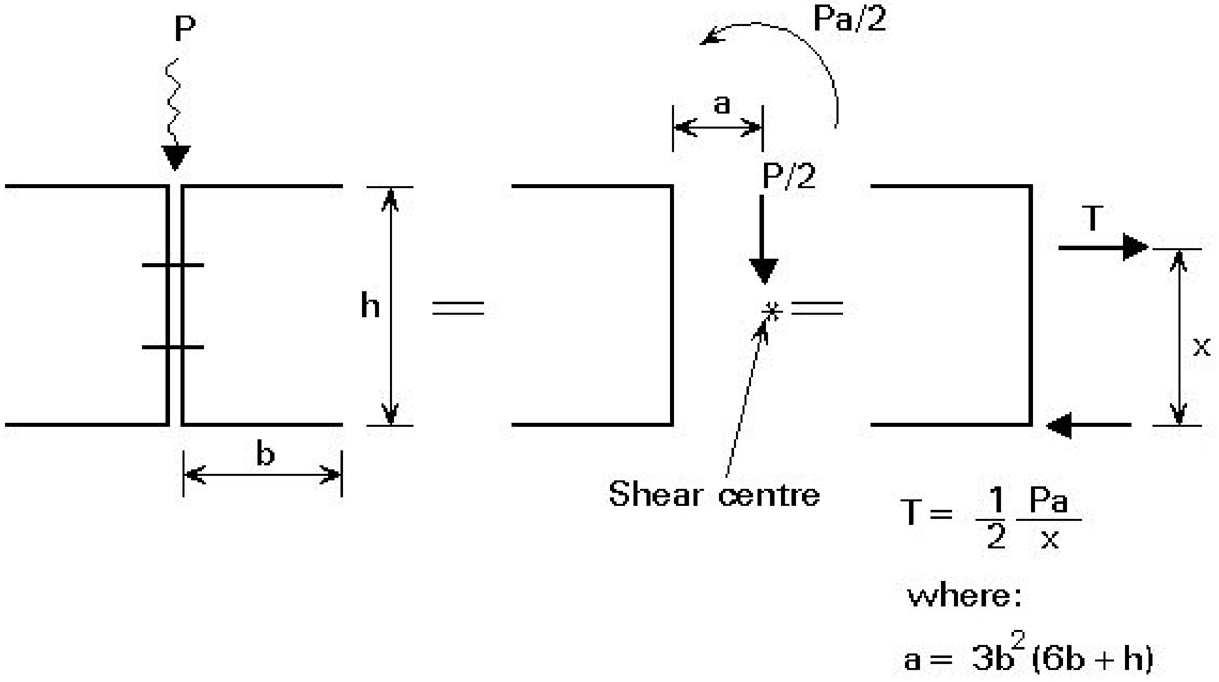



Figure 10 Forces in connection between two C - sections

Secondary forces in connections: Care should be taken, by suitable detailing, that second order effects caused by deformation of thin-walled sections will not generate extra forces in the fastenings.

b. **Connections in profiled sheeting**

It is convenient to discuss these connections by referring to types of forces they should resist:

**Shear forces:**

· The dead weight of steel sheets in wall or facade elements.

· Diaphragm action, when the diaphragm is used deliberately in the absence of a wind bracing, or to provide lateral support for beams or columns.

· Variation of the temperature of the steel sheets; with sufficient deformation capacity the shear forces will be small and may be neglected.

· Rotation of the eccentric fastened sheet ends and the membrane-action of the sheet (see Figure 11), in the presence of sufficient deformation capacity the fastening will not fail.

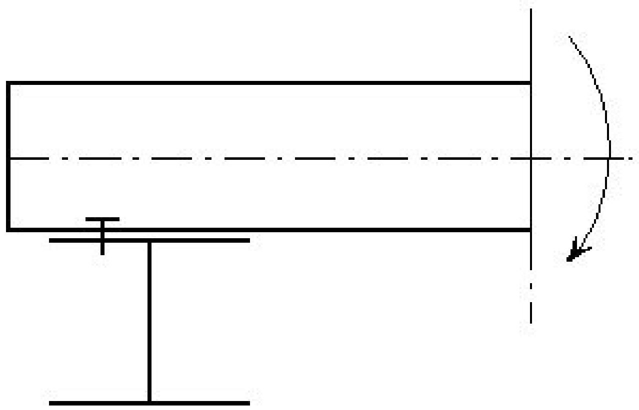


Figure 11 Forces in eccentrically fastened sheet

· Diaphragm action which is not used structurally. It may occur when a sheeting or cladding is only used as an outer skin; it is then necessary for the cladding to follow the deformations of the sub-structure; this is possible when the diaphragm (especially the fastenings) possesses sufficient deformation capacity.

**Tension forces:**

Tension forces will be caused mainly by loads perpendicular to the plane of the steel sheets. For the determination of the required resistance and stiffness of the sheets a simply supported static system is assumed. In reality the sheets are to some extent restrained at the supports; but for the design of the sheets it is safe to neglect the restraining effect.

Overstress can arise in fasteners due to bending of the steel sheet over the supports as in Figure 11. The bending causes an accidental restraining moment of the steel sheets, which generates an extra tension force in the fastener which is known as a prying-force. The magnitude of the prying force depends on:

· the stiffness of the sheets in relation to the span.

· the flexibility of the sheets near the fastener.

· the diameter of the head of the fastener or the diameter and stiffness of the washer.

· the distance between the fastener and the contact points A or B.

· the torsional rigidity of the support.

When sufficient deformation capacity is available the required rotation can take place and design can be based on the reaction ignoring these effects.

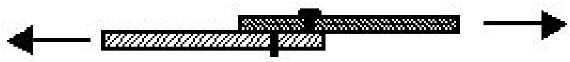
## 4.3 Failure Modes of Connections

Strength and deformation capacity of connections depend substantially on the failure mode of the fastenings. These modes are reviewed below.

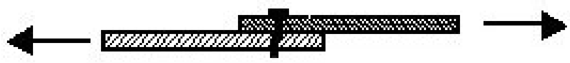
### 4.3.1 Mechanical fasteners

a. **Fasteners Loaded in Shear**

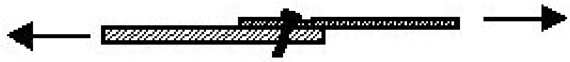
Several failure modes can occur which are illustrated in Figure 12.



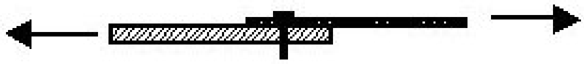
(a) Fastener shear failure



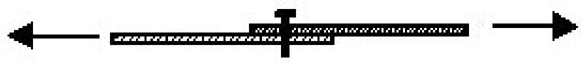
(b) Crushing of fastener



(c) Tilting and pull-out of fastener (inclination failure)

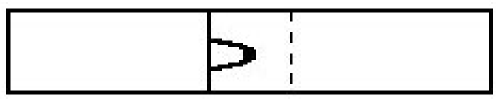


Yield of thinner sheet only



Yield of both sheets

(d) Yield in bearing (bearing failure)



(e) End failure

Figure 12 Failure modes for fastenings subject to shear forces

· Shear Failure (Figure 12a)

Shear failure may occur when the sheet is thick with reference to the fastener diameter, or when an unsuitable fastener is used. This is a relatively brittle form of failure and is not preferred.

· Crushing of the Fastener (Figure 12b)

Crushing may occur with hollow fasteners, and in combination with tilting and yield in bearing.

· Tilting and Pull-out of Fasteners: inclination failure (Figure 12c)

It is the normal mode of failure in thin sheet to thin sheet fastening in which the threads or the site formed rivet heads pull out of the lower sheet. It may occur in combination with yield of both sheets in bearing, and in conjunction with considerable sheet distortion.

· Yield in Bearing: bearing failure (Figure 12d)

Two cases may be encountered: yield may occur only in the thinner sheet or in both the connected sheets. It is the most ductile mode of failure.

· End Failure (Figure 12e)

This failure may occur only when recommended end distances are not achieved.

· Failure at the Net Cross-Section

Failure by fracture of the net cross-section may occur if the tensile resistance of the steel sheet is less than the shear resistance of the fastener.

b. **Fasteners Loaded in Tension**

Several failure modes can occur which are illustrated in Figure 13.

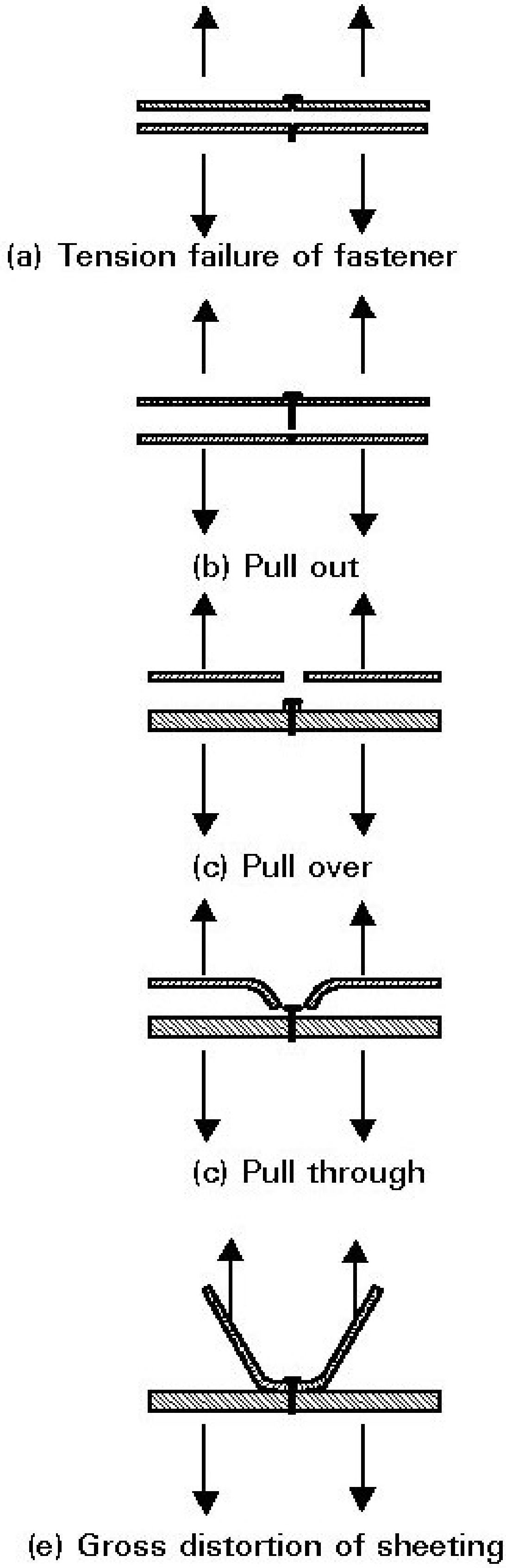



Figure 13 Failure modes of mechanical fastenings loaded in tension

· Tension failure of the fastener (Figure 13a)

Tension failure may occur when the sheet is thick with reference to the fastener, or when an unsuitable fastener is used.

· Pull Out (Figure 13b)

It may occur when the support member is insufficiently thick, or when there is insufficient anchorage of fastener.

· Pull Over (Figure 13c)

It may occur when the head of the fastener is too small.

· Pull Through (Figure 13d)

This mode of failure involves bending of the sheet locally and can be accompanied by washer distortion.

· Gross Distortion of Sheeting (Figure 13e)

Permanent and gross distortion of the sheeting profile may be considered a failure mode, and occurs when the fastener is attached to wide unstiffened sheets.

#### 4.3.2 Failure modes of welded attachments

In thin-walled structures the welded fastenings (fillet and spot welds) should be designed in such a way that the fastening will be loaded in shear.

For fillet welds the weld cross-section should be such that the strength of the fastening is governed by the thickness of the sheet. The failure modes can then be:

- tearing or shearing near the fillet weld.
- failure of the net section.

For spot welds the following failure modes can appear:

- shear of the spot weld itself, which occur with less deformation.
- tearing and bearing at the contour of the weld.
- end failure, when the end distance is relatively short.
- failure of the net section.

### 4.4 Applications

#### 4.4.1 Fastening of outer profiled sheeting to cassettes

Figure 14 shows the detailing of a typical wall cladding. A diagonal pattern of the blind rivets, is chosen because a horizontal pattern would mean that only a few cassettes were loaded. This pattern would lead to over-loading of the fasteners of the relevant cassettes compared to the bearing resistance of the fastenings.

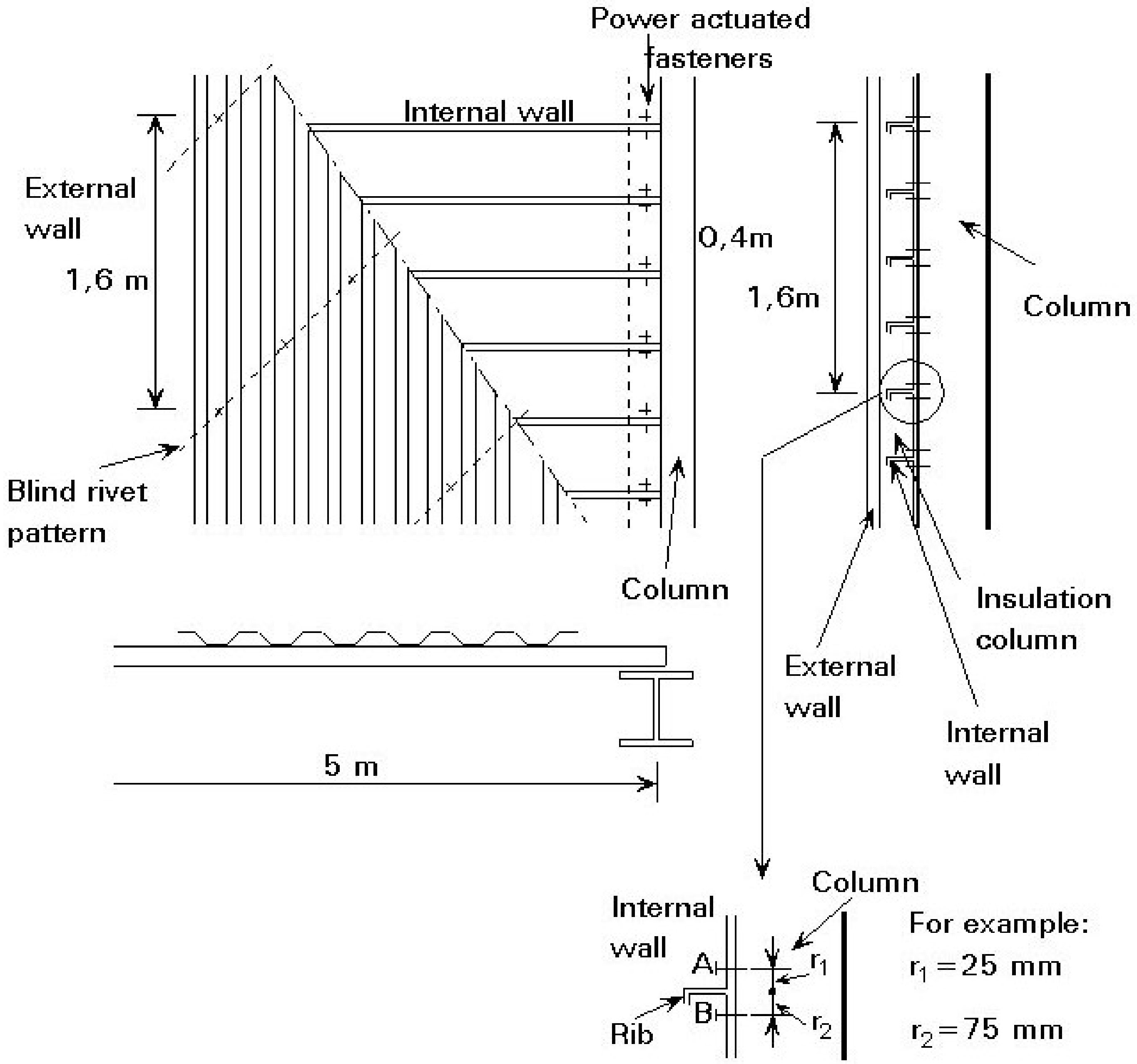



Figure 14 Detailing of a wall cladding with profiled sheeting and cassettes

Furthermore the fastener force at B in the cassettes at distance $r_2$ will become negligible in comparison with the fastener force at A at distance $r_1$. This force differences is caused by the differences in deformations. This means that during design only the resistance of fastener A has to be taken into account.

### 4.4.2 Fastening of outer profiled sheeting to inner profiled sheeting via Z-sections

Figure 15 shows the principle of the build-up of a wall comprising two skins of profiled with Z-sections in between. As a result of relatively high stiffness of the Z-sections prying forces will occur in the structure as shown by forces $k_1$ and $k_2$. They will lead to high forces in the fasteners (forces $N_1$ and $N_2$). The strength of fastenings between Z-section and sheeting (see Section 4.1.b) will often be much lower than the forces $N_1$ or $N_2$.

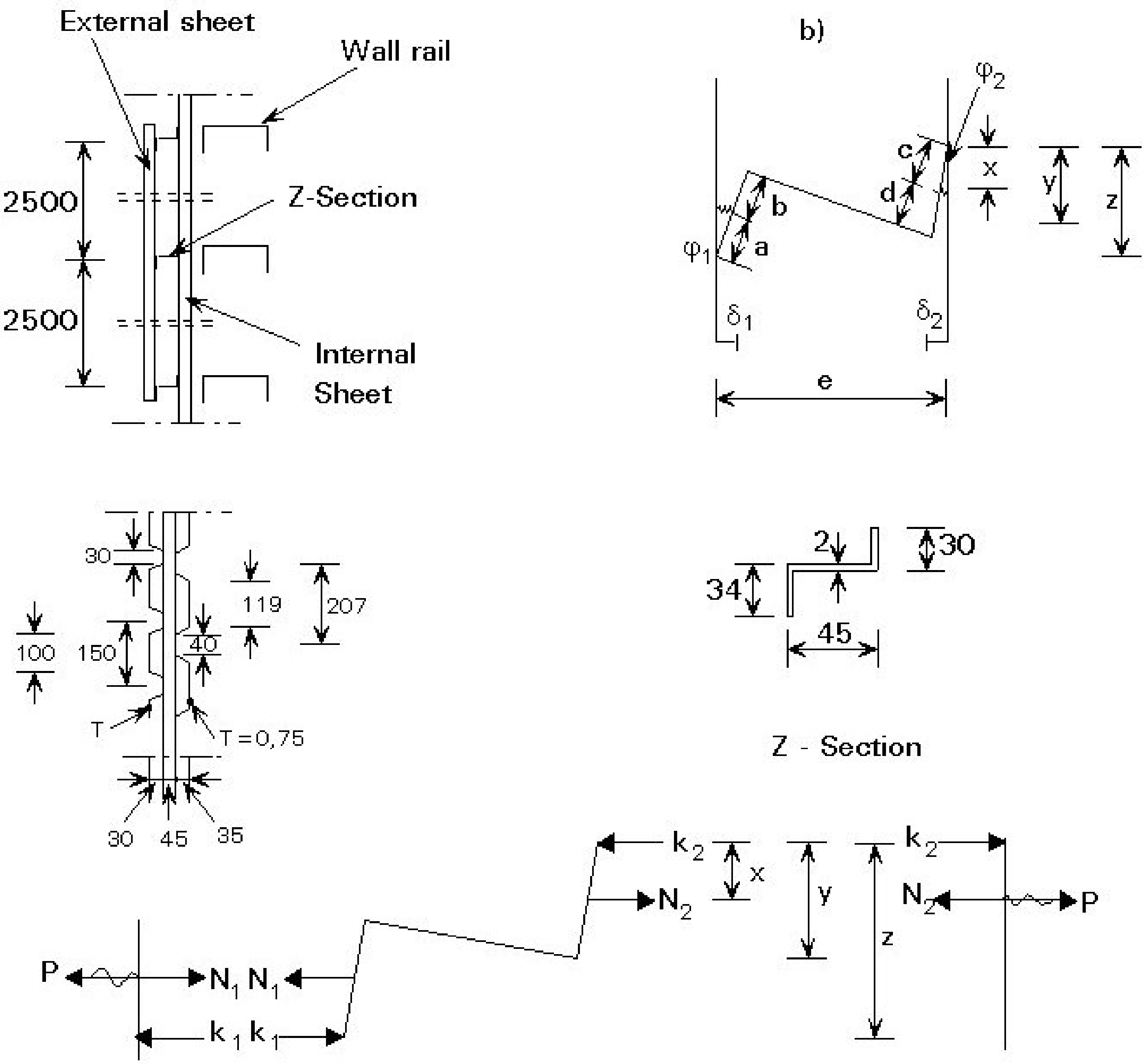



Figure 15 Build-up of a double skin sheeting wall with z-sections in between and scheme of internal forces.

A symmetric loaded connection (by choosing a hat-section instead of a Z-section) will prevent prying forces occurring.

# 5. CONCLUDING SUMMARY

- A wide variety of fasteners is used in thin walled construction, including: bolts with nuts, self-tapping screws, self-drilling and self-tapping screws,

blind rivets, shot fired pins, seam locking systems, spot welding, seam welding and projection welding.
- Structural and non-structural requirements are both important in selecting the most appropriate fastening system.
- Structural design of fasteners needs to consider stiffness, strength and deformation capacity.
- A variety of failure modes occur for fasteners loaded in shear or tension. Several of these arise from the type of fastening, e.g. crushing of hollow fasteners, or the thinness of the material being fastened, e.g. pull through of a threaded fastener.

# 6. REFERENCES

[1] Eurocode 3: Part 1.3: "Cold-Formed Thin Gauge Members and Sheeting" (in preparation).

# 7. ADDITIONAL READING

1. European Convention for Constructional Steelwork: "The Design and Testing of Connections in Steel Sheeting and Sections", Publication2.1, ECCS, May 1983.
2. European Convention for Constructional Steelwork: "Mechanical Fasteners for use in Steel Sheeting and Sections", Publication 4.2, ECCS, June 1983.

**TABLE 1** Global survey of application field for mechanical fasteners

| Thin to Thick Steel | Steel to Wood | Thin to Thin Steel | Fastener | Remark |
|---|---|---|---|---|
| X | | X | | Bolts M5 - M16 diameter |
| X | | | | Self tapping screw 6,3 diameter with washer ≥ 16mm diameter and 1mm thick with elastomer. |
| | X | X | | Hexagon head screw 6,3 diameter or 6,5 with washer ≥ 16mm diameter and 1mm thick with elastomer. |
| X | | X | | Self drilling screw with diameters:<br><br>4,22 or 4,8mm |

| | | | | |
|---|---|---|---|---|
| | | | | 5,5mm<br><br>6,3mm |
| X | | | | Thread cutting screw 8mm diameter with washer ≥ 16mm diameter and 1mm thick with or without elastomer |
| | | X | | Blind rivets with diameters:<br><br>4,0mm, 4,8mm, 6,4mm |
| X | | | | Shot fired pins |
| | | X | | Seam locking |

| |
|---|
| **TABLE 2** Requirements for connections in thin-walled structures |
| Structural requirements:<br><br>1. Strength<br>2. Stiffness<br>3. Deformation capacity |
| Non-structural requirements:<br><br>1. Economic aspects, such as:<br><br>a. total number of fastenings which have to be made.<br><br>b. skill required.<br><br>c. ability to be dismantled.<br><br>d. design life.<br><br>e. installed costs of the fastening. The cost factors are:<br><br>· individual fastener cost.<br><br>· direct labour cost.<br><br>· indirect labour cost. |

· application tools cost.

· maintenance cost.

2. Durability, which depends on:

   a. chemical aggressiveness of the environment.

   b. possible galvanic corrosion.

   c. stress corrosion (can be important with elevated temperatures and aggressive chemical environments).

3. Watertightness

4. Aesthetics

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**ESDEP WG 9**

**THIN-WALLED CONSTRUCTION**

# Lecture 9.7: Application of Thin-Walled Construction

### OBJECTIVE/SCOPE

To present the range of available cold-formed, light gauge products and to illustrate their range of application.

### PREREQUISITES

Lecture 4A.1: General Corrosion

Lecture 5.1: Introduction to Computer Aided Design and Manufacture

Lecture 9.1: Thin-Walled Members and Sheeting

### RELATED LECTURES

Lecture 9.2: Design Procedures for Columns

Lecture 9.3: Design Procedures for Beams

Lecture 9.4: Design Procedures for Sheeting

Lecture 9.5: Stressed Skin Design

Lecture 9.6: Connections in Thin-Walled Construction

### SUMMARY

The range of cold-formed products currently available is described and the particular features offered by products produced in this way are explained. Current uses of such products, often acting to provide several functions, are discussed.

## 1. INTRODUCTION

The general trend in recent years towards lighter and more economic forms of steel construction has led to increases in the use of cold-formed, light-gauge elements. These elements may be either members or flat products.

Members such as purlins, sheeting rails and main framing components may be used in place of heavier and more costly hot-rolled products or substituted for alternative materials such as timber. Flat products such as wall cladding, roof sheeting and floor decking are often used most effectively by combining them with other materials.

The main advantages in using cold-formed products are:

i. A very wide range of components are available through the flexibility provided by the forming process.

ii. Good corrosion resistance through the use of previously coated material.

iii. Good quality surface finish, available in a variety of different types.

iv. Ability to provide high levels of thermal and acoustic insulation when used in conjunction with suitable insulating materials.

v. Availability of relatively simple fastening methods, several of which may be used on site.

vi. High ratios of load-bearing resistance to structural weight.

vii. Permits the use of considerable off-site prefabrication.

Cold-formed products are usually produced from the original coils of thin sheet material by either:

- Roll-forming in which the coil is progressively unwound and various shapes made in a continuous process by passing the sheet through a series of rolls that progressively develop the required shape, Figure 1, or
- Folding in which pre-cut lengths are bent to form a suitable structural element.

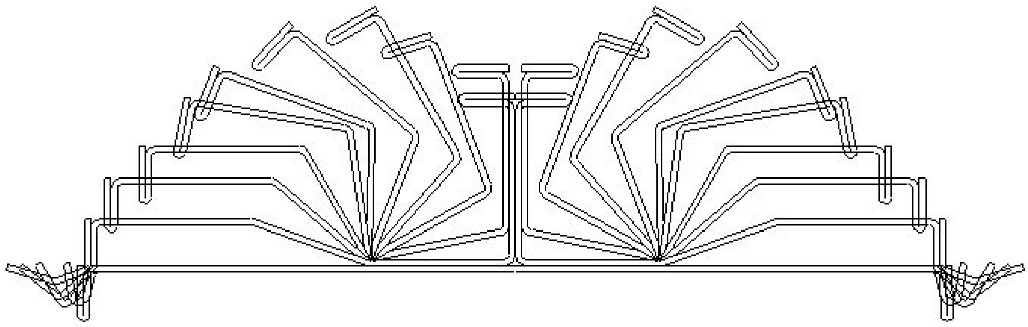


Figure 1 Roll-forming

Whilst the first of these methods is suitable for large volume production, e.g. sheeting for cladding, the second is more appropriate for low volume production of items that need to be tailored for specific applications, e.g. flashings used to seal the joins between components.

## 1.1 Available Products

### 1.1.1 Profiled sheeting

Profiled sheeting is available in a wide range of geometries and may be adapted for several different uses.

In single or double skin roof sheeting, Figure 2, the steel sheet is normally used as the external profile. Its main function is to ensure the security of the roofing. The main function of the internal sheet in a double skin roof is to support the insulation, although by incorporating suitable perforations it may assist in improving the acoustic properties of the building. Double skins may use the same profile for both sheets - in which case the roofing is generally called "parallel texture", or it can use a different profile internally, thereby replacing the purlins. In this latter case the arrangement is termed "crossed texture".

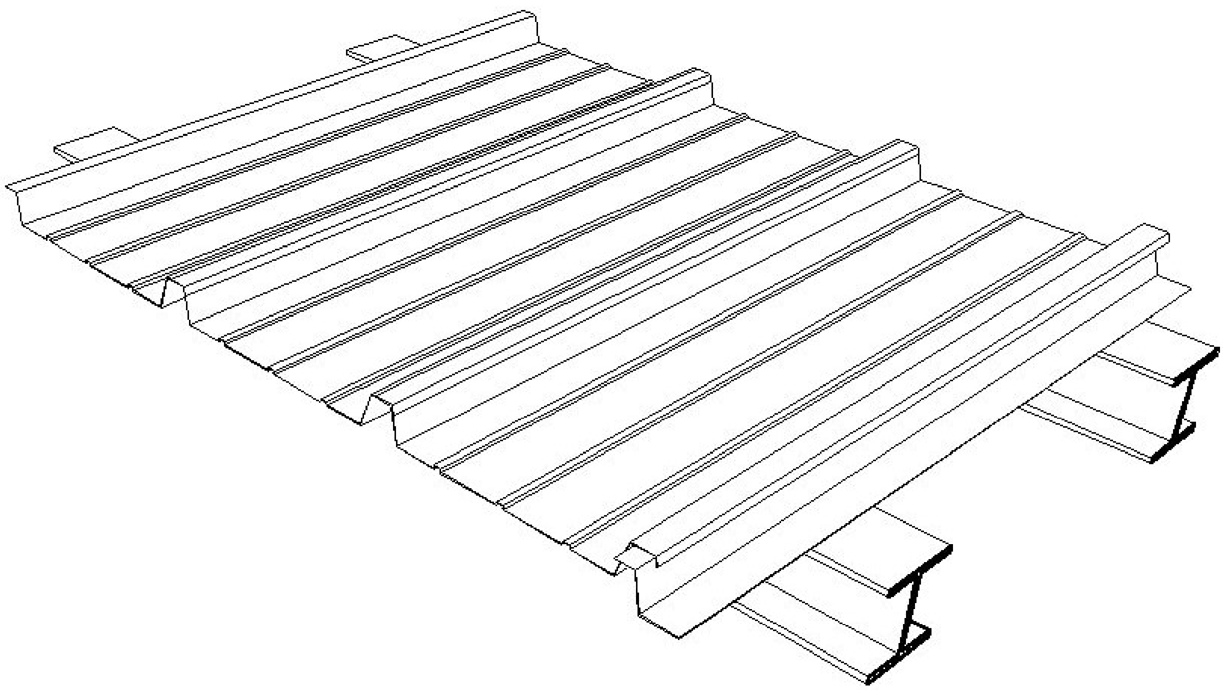


Figure 2a Roof sheeting

For flat roofs, Figure 2b, the steel profile has, in addition to supporting its own dead weight, the function of carrying the insulation and the maintenance and climatic (snow and wind) loads.

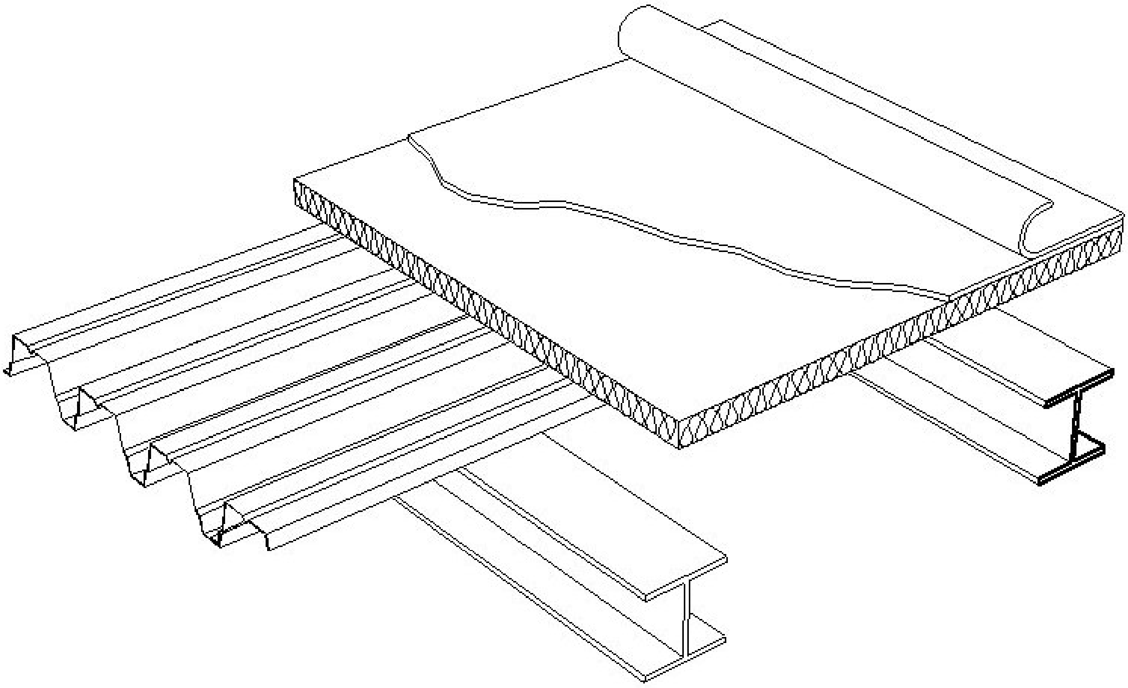


Figure 2b Flat roof decking

In such arrangements it is important to ensure that the insulation can safely span the troughs between adjacent upper flanges of the corrugations.

For vertical walls, the outer skin of either a single or double wall-cladding, Figure 3, has to ensure air tightness and water tightness of the vertical walls, whilst at the same time providing an attractive visual aspect to the building. The internal skin is then effectively a structural tray or cassette, spanning horizontally between columns that provides a flat internal wall and supports the insulation.

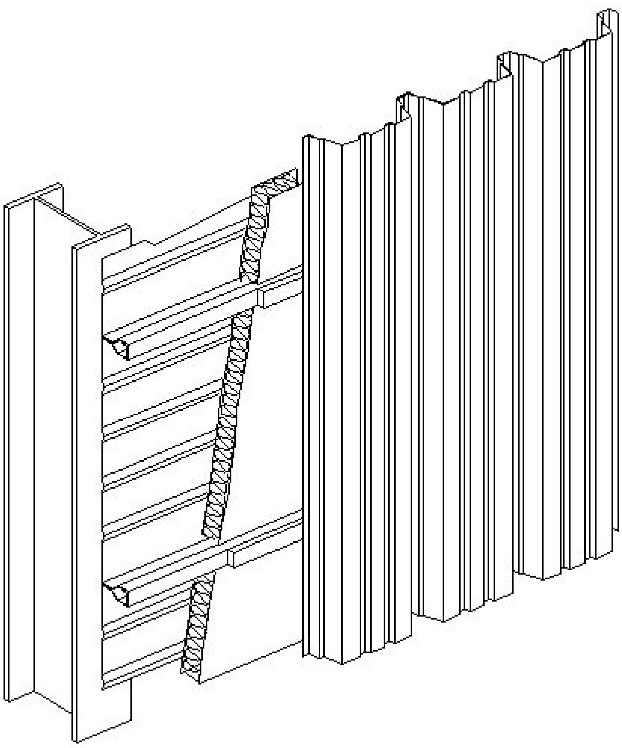


Figure 3 Vertical wall construction

Trapezoidal sheeting may also be used for flooring, e.g. in pre-fabricated houses.

Sheeting may also be used as permanent shuttering to concrete, in which case it may be designed merely to support the weight of the wet concrete and any construction loading. A more efficient arrangement is, however, to use the steel sheeting in the final condition also to produce composite action in the floor as shown in Figure 4.

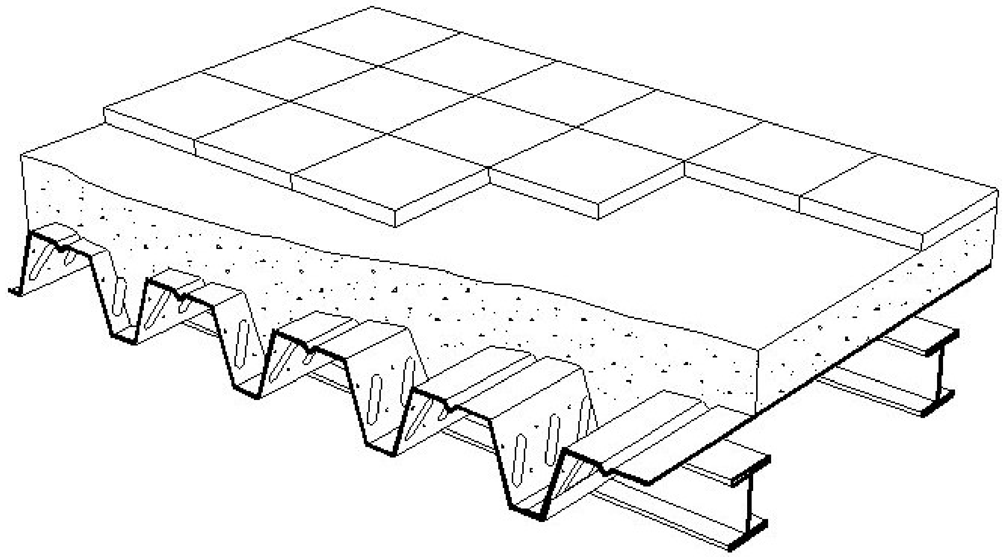


Figure 4 Profiled sheeting in composite floor

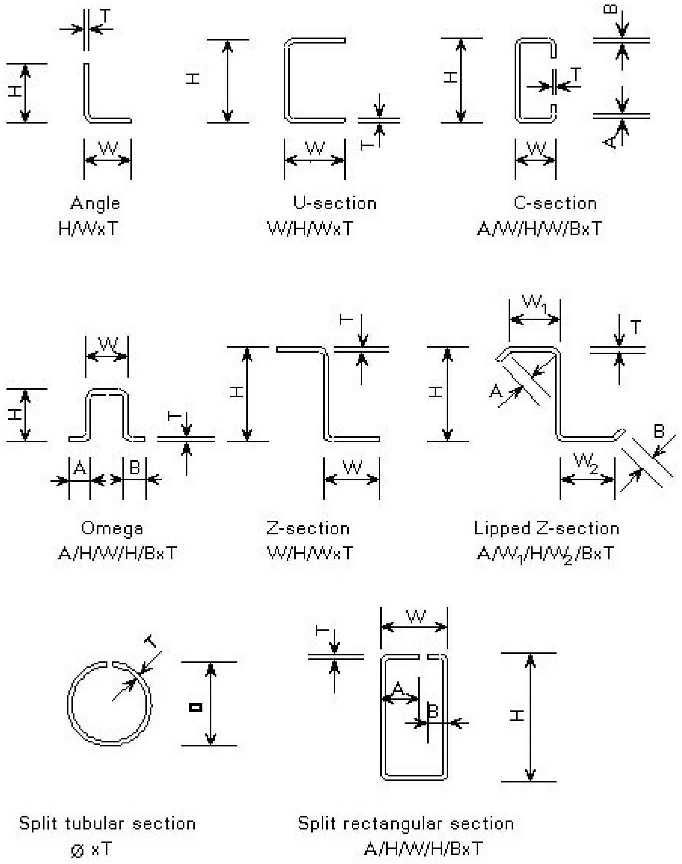
Because precoated sheeting is used for these flooring arrangements, the lower face may be used to provide a sufficiently visually attractive ceiling in some cases.

Since many steel cladding and roofing profiles are also available in equivalent geometrical shapes manufactured from translucent material, it is normally quite straightforward to create areas of natural lighting in buildings using this arrangement.

In order to properly fix cladding and roofing panels it is necessary to employ a range of smaller components at the various junctions. These products are termed flashings and will frequently need to be provided in a range of shapes and sizes. The products may sometimes also fulfil additional functions, e.g. to act as gutters. The folding operation is ideally suited to the production of this multiplicity of different items.

### 1.1.2 Members

Cold-formed members are available in a very wide range of shapes and sizes. Since they are normally formed from a single sheet, the cross-section should be such as not to require additional jointing, e.g. an I-section can only be formed by joining two channels back to back at the web. Figure 5 provides some idea of the range of products currently available.




| Ranges of available dimensions of common cold-formed sections | | | | |
|---|---|---|---|---|
| Section type | Dimension ranges (mm) | | | Remarks |
| | Steel core thickness | Section height H | Flange width W | |
| Angles | 1,5 - 10,0 | 15 - 225 | 15 - 225 | |
| U-section | 1,5 - 10,0 | 20 - 400 | 15 - 160 | |
| C-section | 1,5 - 6,0 | 20 - 400 | 40 - 160 | |
| Omega | 1,5 - 5,0 | 20 - 100 | 20 - 100 | |
| Z-section | 2,0 - 6,0 | 30 - 80 | 20 - 30 | |
| Lipped Z-section | 1,5 - 4,0 | 120 - 300 | 50 - 100 | As purlin in roofs |
| Split tubular section | 0,8 - 4,0 | ø16 - ø80 | 50 - 100 | Diameter is instead of height |
| Split rectangular section | 1,5 - 4,0 | 12 - 140 | 12 - 140 | |

Figure 5 Examples of common cold-formed sections

### 1.1.3 Sandwich panels

In recent years the use of prefabricated sandwich panels, (Figure 6) formed from two metal sheets with a foam core, has increased considerably. Such an arrangement, which may be installed very rapidly, is particularly suitable as a means of providing thermal insulation at the same time as the basic weather shield. It consists of two metal faces bonded to an internal layer of rigid foam. Such panels may be installed very quickly thus saving time on site.

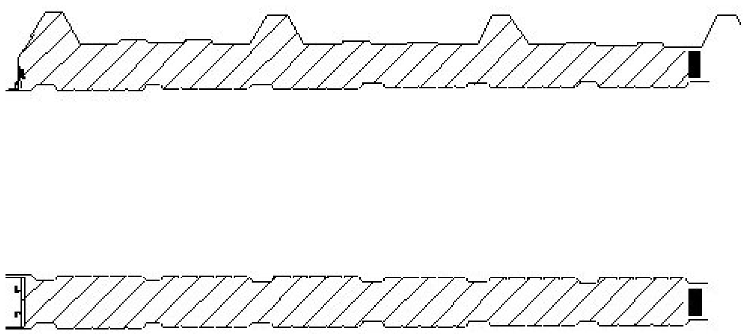


Figure 6 Prefabricated sandwich panels (examples)

## 2. COMPOSITE CONSTRUCTION

### 2.1 Cold-Formed Sections and Sheeting

It is commonplace to use the opportunity offered by the flexibility inherent in the forming processes to arrange for cold-formed products to act in conjunction with other components. One of the most common examples is the use of Z purlins, Figure 7, acting in association with roof sheeting to provide horizontal diaphragm action which helps to stabilise the building. In addition the sheeting acts to provide torsional restraint that improves the load-carrying resistance of the purlins themselves. The concept of deliberately relying on diaphragm action of sheeting has led to the concept of "stressed skin action" in buildings in which a significant contribution is made by the cladding. Such an arrangement can substantially reduce the need for bracings.

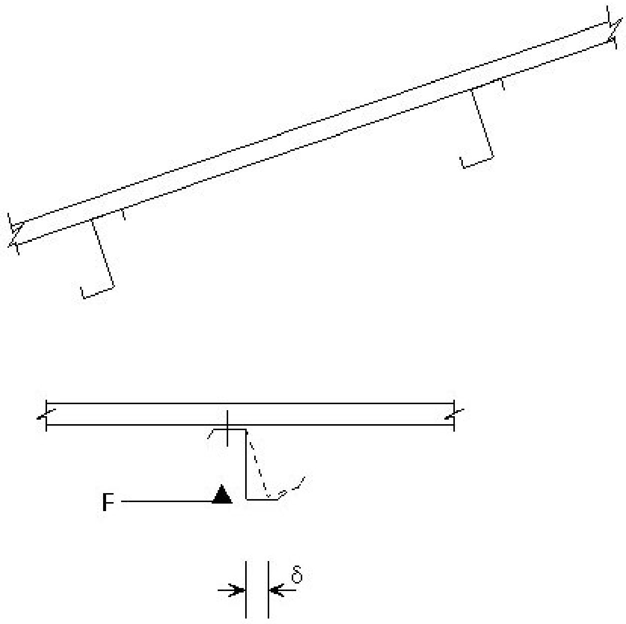



Figure 7 Use of Z purlins

### 2.2 Profiled Sheeting and Concrete

In the construction of multi-storey steel framed buildings, the use of composite metal deck flooring has increased significantly in recent years. Metal decking, typically spanning about 3 metres between secondary beams, may be laid rapidly and, when secured to the top flanges of the beams using shot fired steel pins, provides both a working platform and a protection to the operatives further down in the building. Concrete may be pumped or supplied by skips using a crane to the floors, which are normally reinforced with a light prefabricated mesh to control any cracking that might occur as a result of shrinkage during hardening of the concrete. Composite action with the metal decking is ensured through the use either of the decking profile itself or through indentations on part of the sheeting that provides a shear key arrangement preventing relative movement between the hardened concrete and the metal decking. Shear studs may be welded through the decking onto the top flanges of the beams so as to provide composite action with the primary beams spanning between columns in the frame.

### 2.3 Fasteners

Various proprietary types of fasteners, including spot welding, may be used with light gauge products. Figure 8, which illustrates the main types of mechanical fasteners, shows:

- Bolts with nuts.
- Shot fired pins.
- Screws:

  - self tapping.
  - self drilling.
  - special types for translucent sheeting.

- Blind rivets.
- Crimping of the seams to provide a mechanical interlock between adjacent sheets.
- Welding may be either:

  - Arc welding.
  - Resistance welding.
  - Weld-brazing.

This last process is relatively new and is specifically designed to avoid damage to galvanised or pre-painted coatings. It works by introducing another metal that melts more easily than those to be joined and, using a blowpipe flame, results in a quickly made, tight and resistant joint.

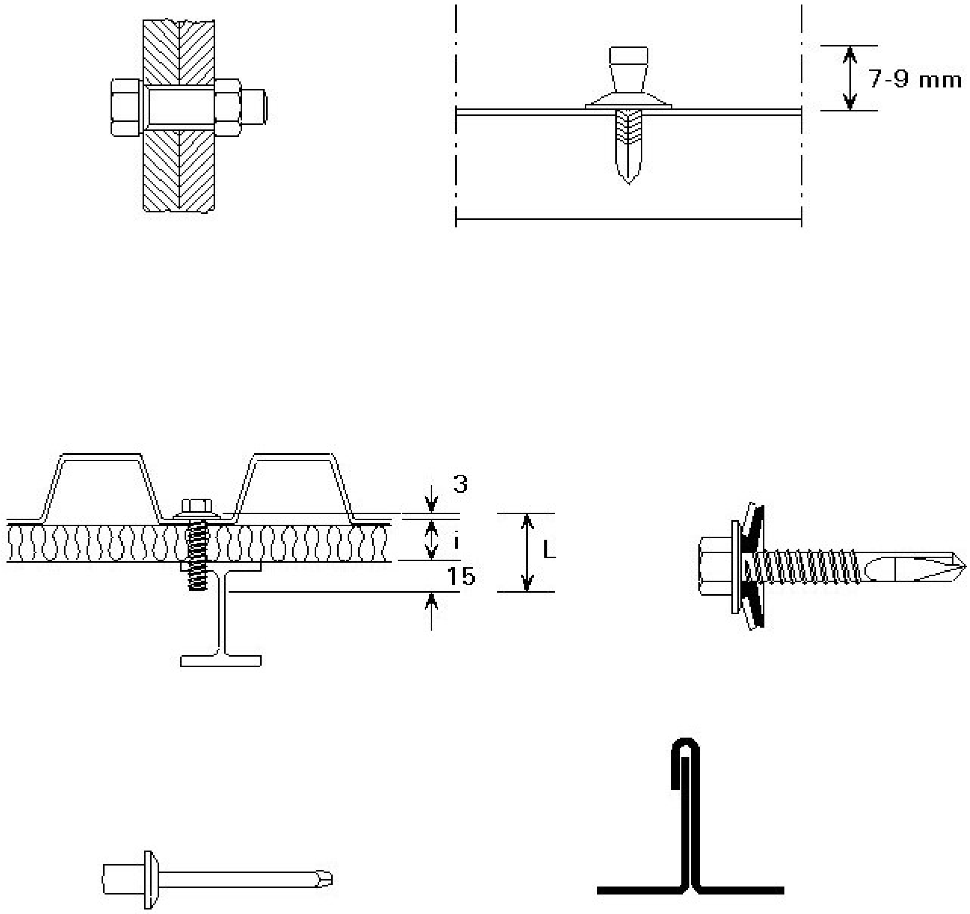



Figure 8 Examples of mechanical fasteners

## 3. PHYSICAL CHARACTERISTICS

Thermal insulation is often required for walls and may be provided by any of rockwool, glass-wool, chipboard, polystyrene, cellular glass or expanded perlite. Since insulated panels are proprietary products, relevant manufacturers literature should be consulted as a way of ascertaining specific properties and recommendations on usage. Of particular concern are dimensional stability, voluminal mass, heat conductivity coefficient, resistance to compression, resistance to bending, reaction to fire, resistance to water vapour, dew point, ability to act as a heat bridge, and fixing arrangements.

### 3.1 Acoustics

The need for adequate sound insulation is an increasingly stringent requirement for buildings. The two characteristics required for limiting the transmission of noise are absorption and insulation. It is often possible to combine the dual functions of thermal and acoustic insulation in the one type of composite panel. Particular ways to improve acoustic insulation are to increase the thickness of the profile and/or the density of the insulation or to add an additional sheet between the two skins.

### 3.2 Fire Resistance

Because light gauge sheets are so thin, they do not posses much inherent fire resistance if exposed directly to elevated temperatures. However, properly insulated double-wall cladding can attain a fire resistance of at least 90 minutes. Support roofing and composite flooring may well reach at least 60 minutes without additional protection.

### 3.3 Condensation

A steel wall, especially when not insulated, may be exposed to condensation since its coefficient of thermal conductivity is rather high. However, several anti-condensation coatings have been developed. These coatings are applied using an industrial process to the inside of the wall. They largely prevent subsequent condensation. One such example is "Grafo Therm". It is a water based mixture including porous components with large specific surfaces suitable for absorbing water. It appears in the form of a coating containing very fine granules of a light grey shade which have a pleasing appearance.

### 3.4 Durability

Profiled sheeting is always pre-coated and is obtainable in thicknesses between 0,3mm and 4 mm and widths of up to 1.500 mm. Typically steel strengths are 320N/mm$^2$ to 350N/mm$^2$, with values up to 550 N/mm$^2$ presently available. Galvanising is the normal form of pre-coating. This coating provides cathodic protection to surfaces where the steel is uncoated, e.g. either accidentally or at edges formed by shearing or drilling, Figure 9. The zinc film may be painted in order to improve further the corrosion resistance, as well as to enhance appearance. Specially produced galvanised and pre-painted systems provide very significant corrosion resistance due to the synergy that exists between the two processes. For members it is also possible to apply paint after roll forming, such as polyester powders by the electrostatic process. In all cases coatings are available in a large range of colours.

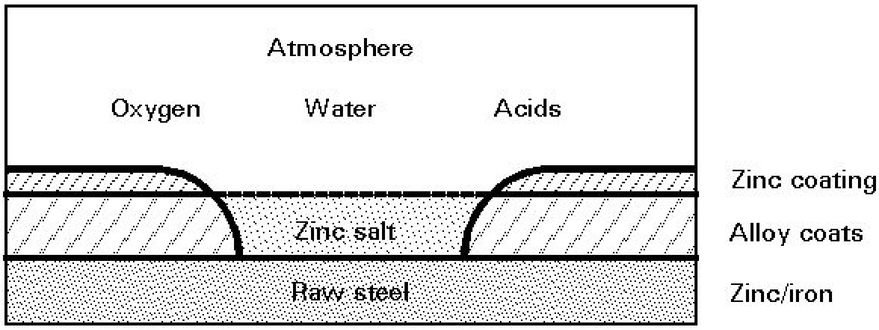



Figure 9 Exposure of galvanised sheeting to the atmosphere.

## 4. USE IN SERVICE

Advances in manufacturing technology enable the range of products and the features that may be included directly in the forming process to increase continually. Clearly clever utilisation of this facility will lead to a progressive reduction in expensive site operations. Figure 10 shows how the termination of the section may now be integrated into the roll-forming process.

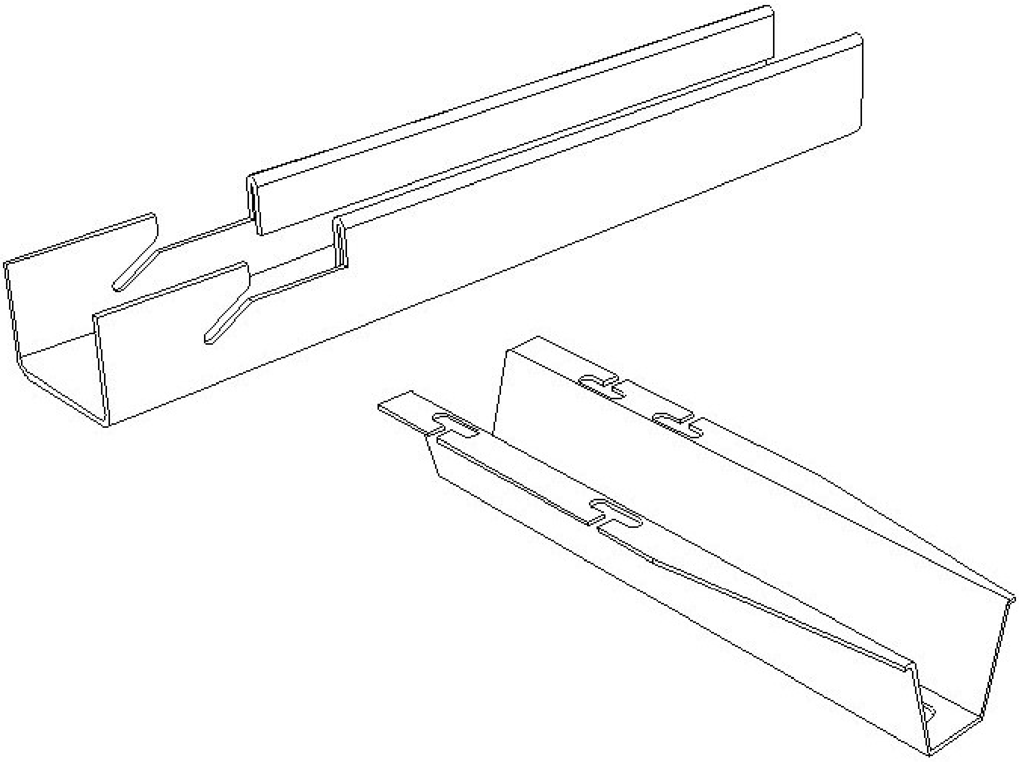


Figure 10 Termination of section integrated into the cold-rolling process (example)

Cold-forming products are normally so light in weight that easy handling is possible. Since most can be handled manually, fast and easy erection is the norm. For transportation, sections and sheeting can normally be nested, thus requiring little space and permitting unloading in quantity. Use of precoated material ensures corrosion protection during transport and erection.

## 5. TYPES OF LIGHT-WEIGHT STRUCTURES

Early applications of cold-formed thin-walled steel sections were restricted to situations where weight saving was important. With the advance in the raw material itself and the manufacturing processes, the range of actual and potential use is virtually unlimited.

### 5.1 Industrial Buildings

Trusses of the types shown in Figure 11 may be found in industrial and storage buildings. The main chords are usually channel sections joined back to back. The web members are normally single channels. A high degree of lateral stability may be provided by using suitably wide chord members. Pre-galvanised high yield steel is normally used, with joints being made by simple bolting. Clear spans of up to 50 metres are possible.

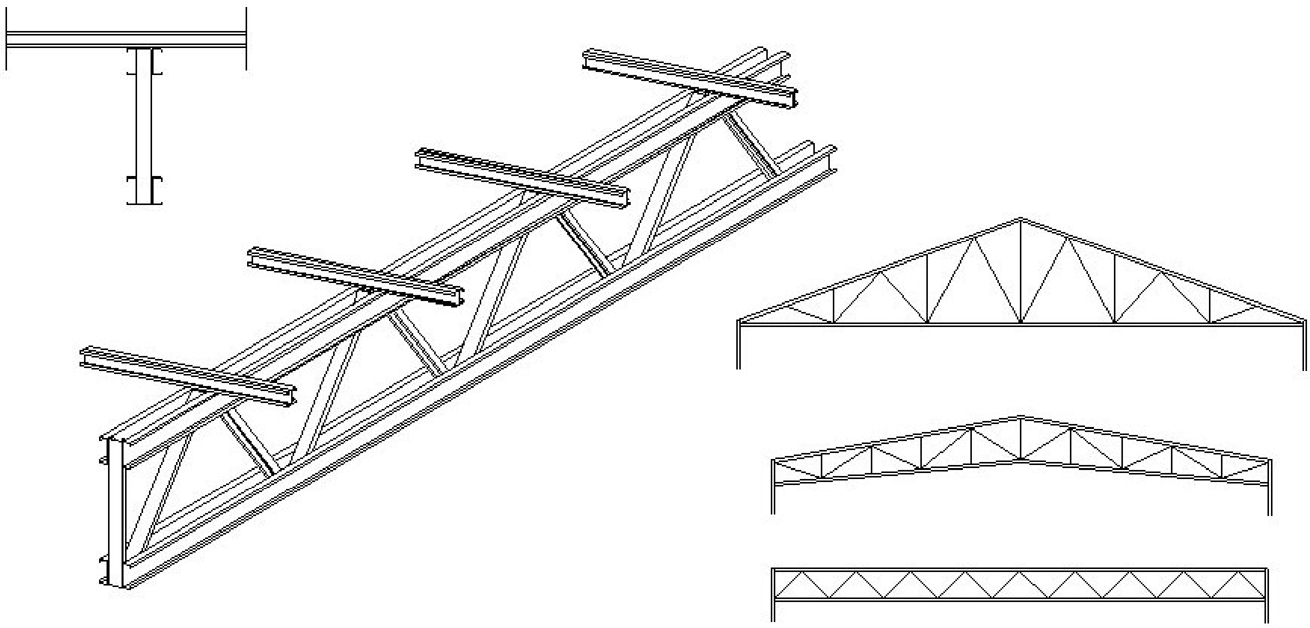


Figure 11 Examples of trusses

A parallel development is the use of lattice portal frames, with both the rafters and the columns being lattice members. Spans of up to 60 metres are possible.

One of the more traditional applications has been in purlins spanning between the heavier main frames in portal frame buildings. For smaller frames, the columns and rafters themselves may be suitable cold-formed sections, as indicated in Figure 12.

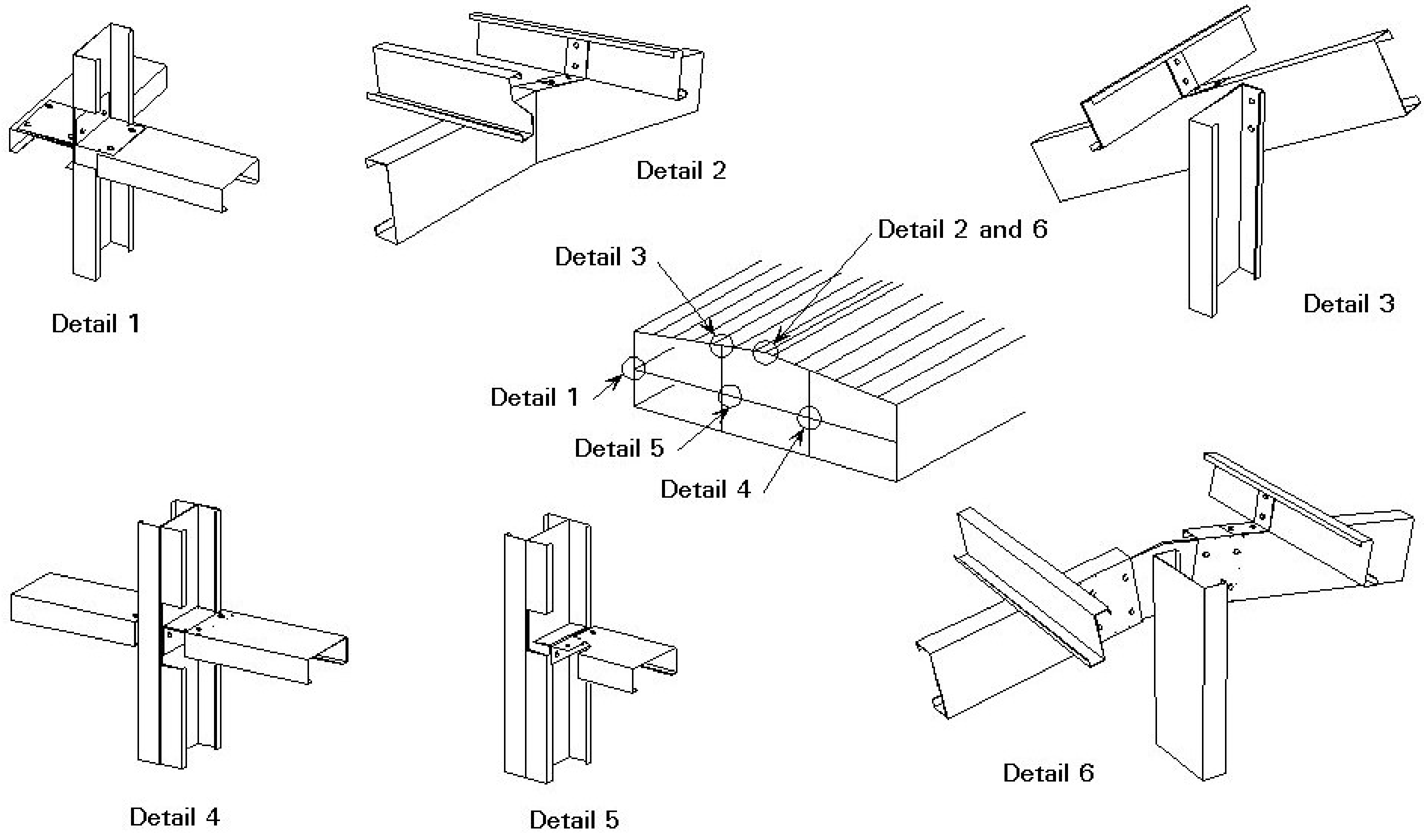



Figure 12 Cold-rolled sections used as columns, rafters and purlins

The insulation of mezzanine floors for storage or as a means of creating office space is a particularly suitable application for cold-rolled sections. Their use as columns and beams permits a lightweight construction using pre-galvanised sections and straightforward fastening.

The availability of cold-formed sections up to 500 mm in depth permits their use as main framing members, either in multi-storey braced frame construction or in portal frames. For the latter, spans up to 25 m are possible. In office buildings concrete filling of columns to provide fire protection is possible, whilst powder painting eliminates the need for final site painting. Members may have all the holes required for fixings automatically produced during the roll-forming process.

One of the largest uses of cold-formed material is in purlins Figure 13, spanning between 4 and 15 metres. Sections of heights between 100 and 300 mm are used and a number of proprietary systems are available. These systems include concepts such as purlin sleeves and overlapping, Figure 14, in order to obtain optimum structural performance. Because manufacturers have often undertaken extensive testing and other development work themselves, design information in support of particular systems is normally available. This information substantially reduces both the labour and complexity of the design process.

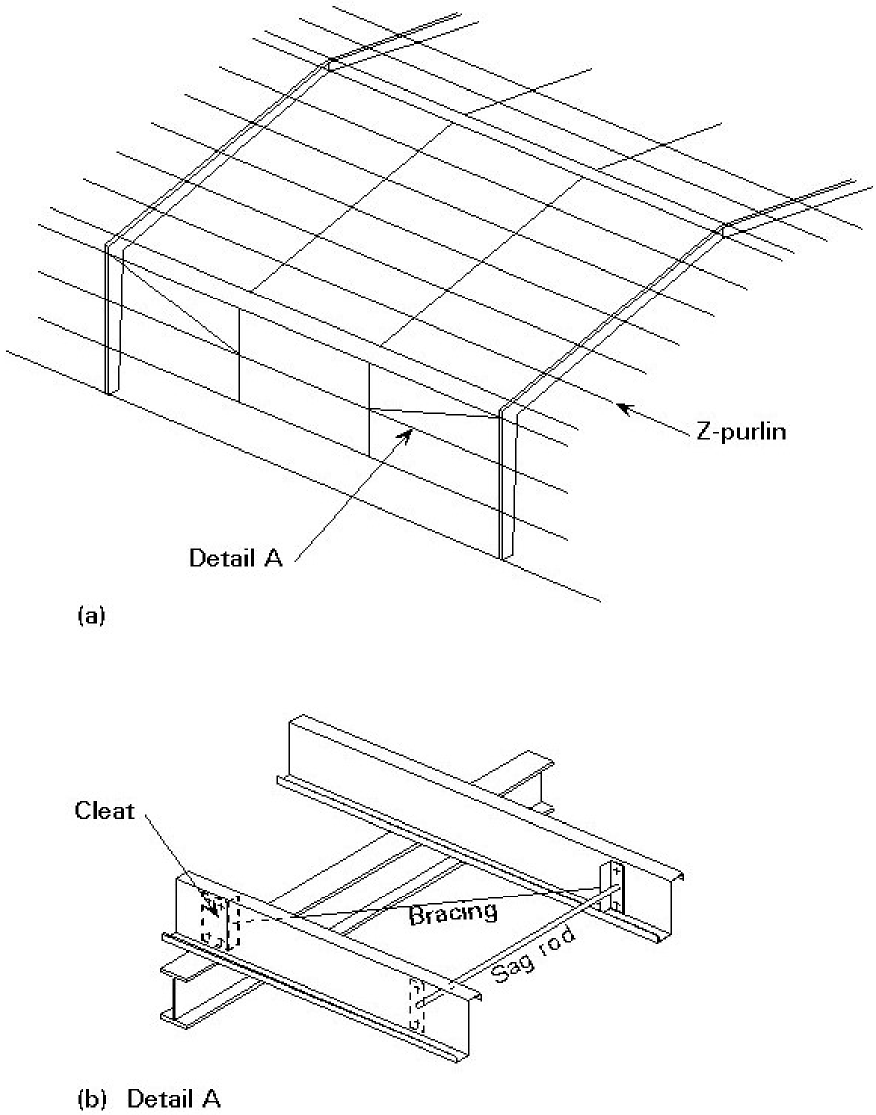
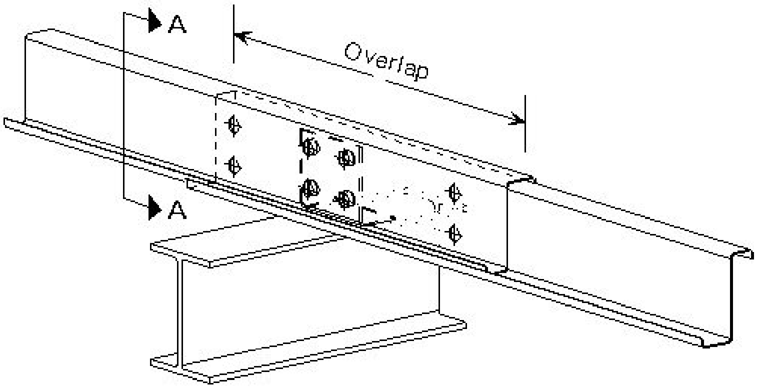
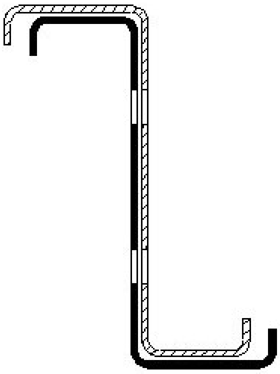



Figure 13 Use of cold-formed sections as purlins




(a)



(b) Section A-A

Figure 14 Overlapping cold-rolled purlins

Trussed rafters are also used in greenhouse construction.

### 5.2 Housing

Light gauge sections are appropriate for use as the steel frame of housing, Figure 15.

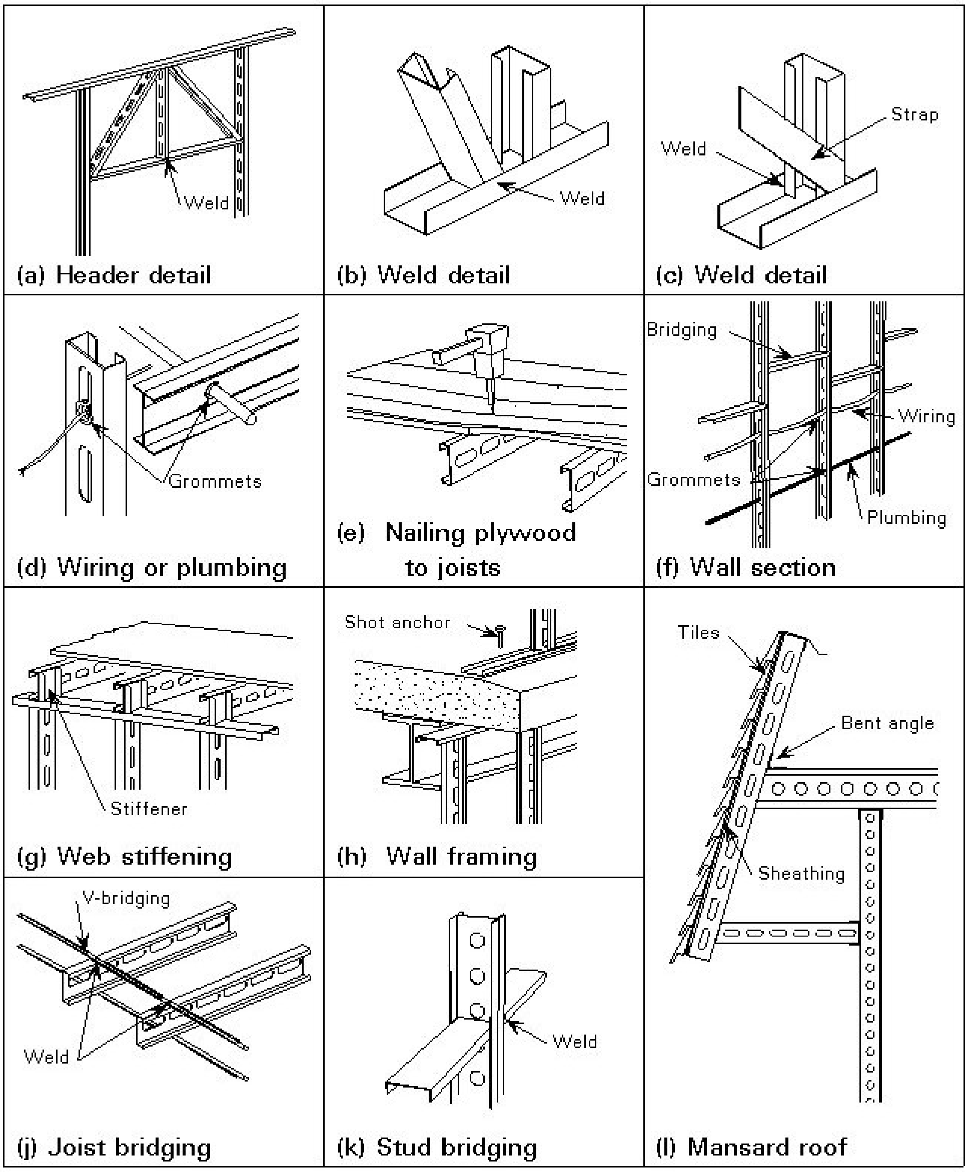



Figure 15 Cold-rolled sections in housing

Two types are available:

- A skeleton completely assembled by bolting on site, Figure 16.
- A skeleton based on the use of steel panels preassembled in the works.

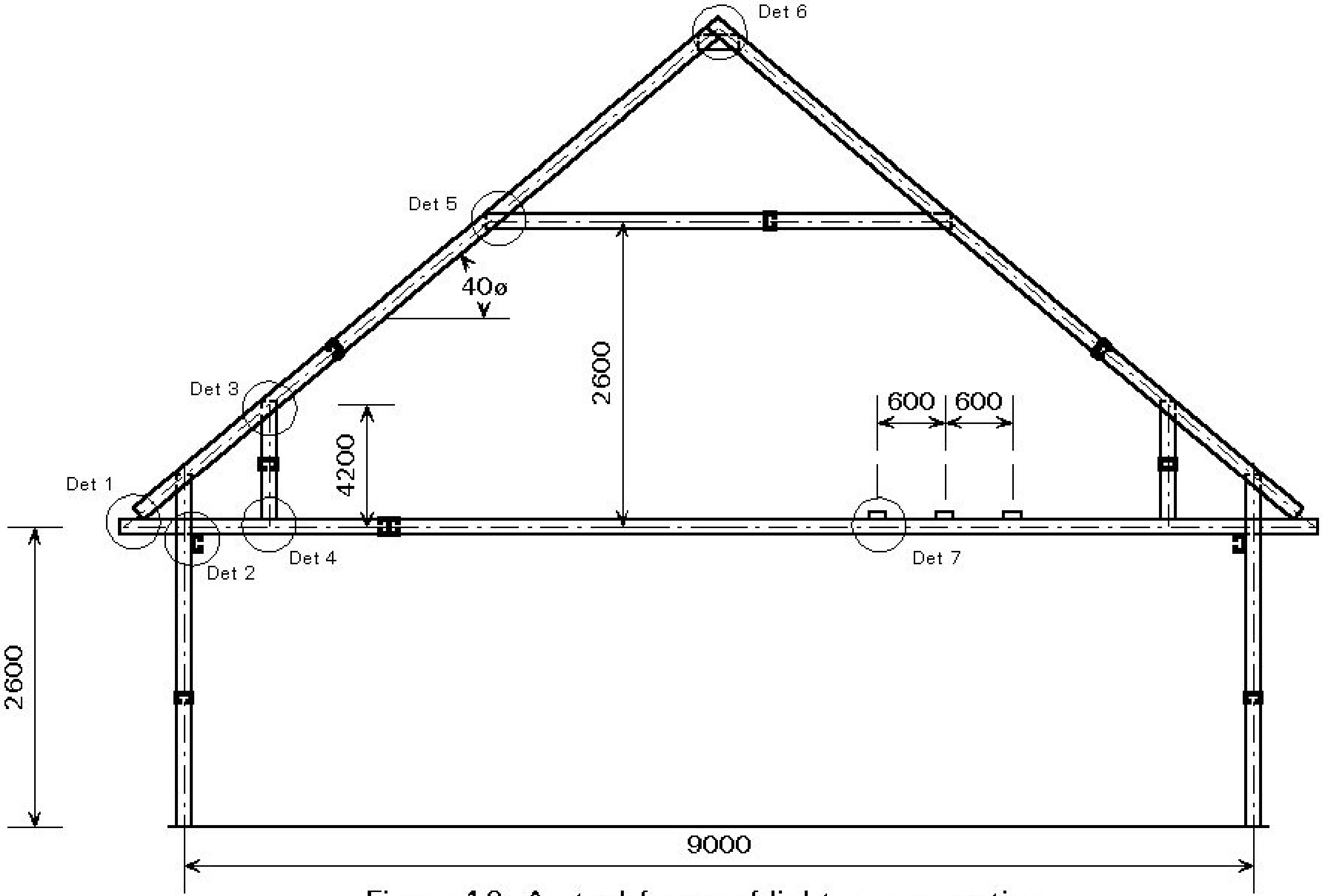


Figure 16 A steel frame of light gauge sections assembled by bolting on site

Using the second principle, it is possible with quite large panels, e.g. 3mx12m to introduce a large degree of pre-fabrication into the structure. All sections are pregalvanised and contain all the holes necessary for wiring, plumbing, etc.

## 5.3 Temporary Accommodation

Modular units for houses, offices, construction site accommodation, etc., may conveniently be produced using cold-formed sections and flat products.

## 5.4 Storage

Storage racking systems, Figure 17, may conveniently be made from cold-formed components, with the forming process being used to produce not simply the most appropriate shapes but also to introduce slots and holes that facilitate rapid assembly and demounting.

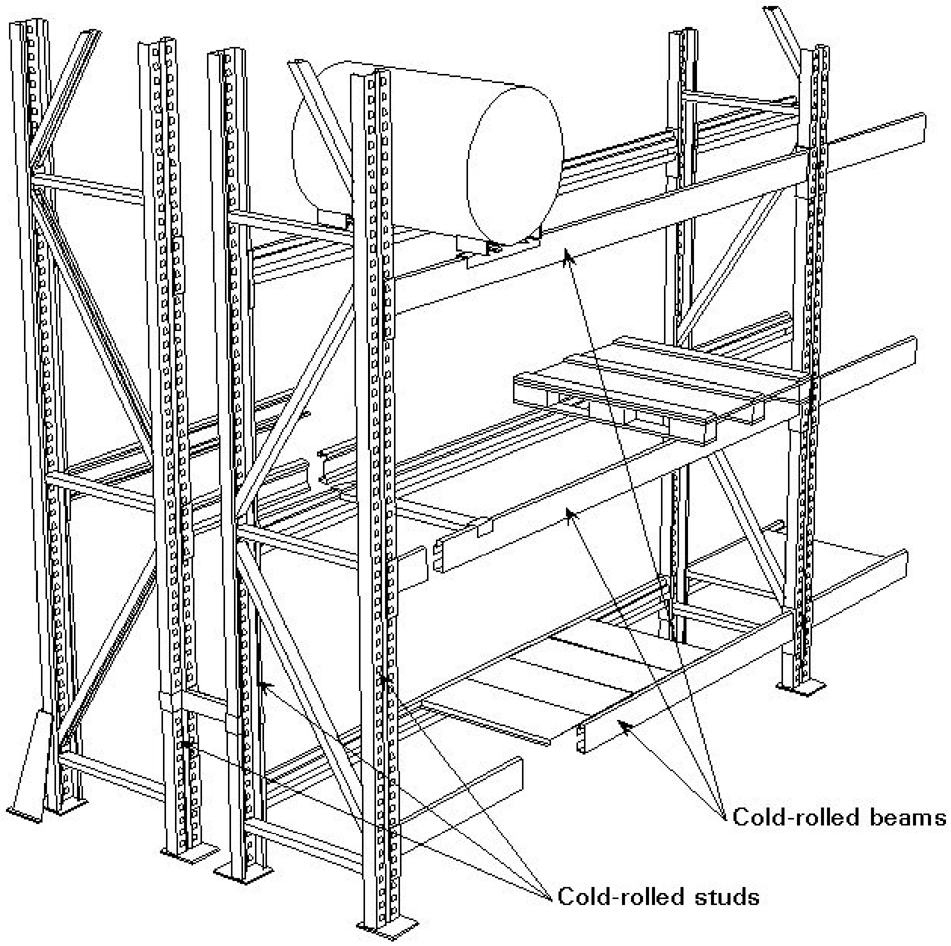



Figure 17 Storage racking system

## 6. CONCLUDING SUMMARY

- Cold-formed products provide the designer with a very wide range of items, capable of being used in a variety of different ways.
- Very efficient structural elements may be produced by combining different types of cold-formed element or by using such elements in association with other materials.
- Use of precoated sheeting, together with the incorporation of as many features as possible in the factory production process drastically reduces the need for expensive site operations.
- Corrosion protection, thermal and acoustic insulation and good visual appearance are all readily achievable.
- The use of this material permits several simple techniques of site jointing to be employed.

## 7. ADDITIONAL READING


1. European Convention for Constructional Steelwork: "European Recommendations for the Design of Light Gauge Steel Members", Publication 49, ECCS, 1987.
2. European Convention for Constructional Steelwork: "European Recommendations for the Design of Profiled Sheeting", Publication 40, ECCS, 1983.
3. Eurocode 3, Part 1.3: "Cold-formed Thin-gauge Members and Sheeting" (in preparation).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