Course: Eurocode 3  
Module 1: Introduction to the design of structural steelwork in accordance with the new Eurocodes

Lecture 2: Introduction to EC1

Summary:

- EC1 provides detailed guidance on the calculation of actions to be used in the design of buildings.
- The principal sources are dead and imposed loads, snow, and wind.
- These are described in separate parts of EC1.
- Dead loads are calculated on the basis of material densities and construction details.
- Imposed loads are related to building usage.
- Snow loads are based on geographical location and roof shape.
- Wind loads are based on location and exposure.
- The distribution of wind pressures on walls and roof slopes depends on the geometry of the building.
- Partial safety factors are used to account for uncertainties in load levels.
- These are modified to account for different combinations of actions.

Pre-requisites:

- None, but a general understanding of the nature of loadings would be useful.

Notes for Tutors:

This material comprises one 60 minute lecture if presenting the material to designers familiar with calculating design loads according to national codes of practice.
### Objectives:
- To describe the structure of EC1 and identify those parts of particular relevance to the design of buildings.
- To outline the principal terminology, symbols and notation used in EC1.
- To describe the procedures for determining design values for actions (dead, imposed, snow and wind loads).
- To explain how combinations of actions are treated and to provide guidance for practical design conditions.

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8. Concluding Summary
1. Scope and structure of EC1

The Eurocodes require structures to be designed to resist the effect of ‘actions’. These are principally loads (direct actions) but also include indirect actions such as imposed deformations (for example foundation settlement) and temperature effects.

EC1 defines how actions on a structure should be determined. The principal exclusions relate to execution (temporary works), appraisal, special (e.g., nuclear), and cases which involve very large deformations. The principal sources of actions on buildings are dead and imposed loads, wind loads and snow loads. These are covered in different parts of EC1 as follows:

- EC1 Part 2.1 - dead and imposed loads
- EC1 Part 2.3 - snow loads
- EC1 Part 2.4 - wind loads

2. Definitions and notation

The principal terms and the equivalent symbols relating to actions are as follows:

- \( A \) Accidental action
- \( G, g \) Permanent action
- \( Q, q \) Variable action
- \( E \) Effect of actions
- \( F \) Force
- \( M \) Moment
- \( N \) Axial force
- \( V \) Shear force
- \( \gamma \) Partial safety factor applied to characteristic values of actions to establish values to be used in design calculations
- \( \psi \) Combination factors used to modify partial safety factors for actions used in combination

EC1 also makes extensive use of subscripts. These can be used to clarify the precise meaning of a symbol. Some common subscripts are as follows:

- \( k \) characteristic
- \( d \) design
- \( \text{sup} \) superior (upper bound)
- \( \text{inf} \) inferior (lower bound)

Specific notation is used for the parts of EC1 dealing with snow and wind loads as follows:

EC1-2-3: Snow loads

- \( s_k \) characteristic snow load at ground level
- \( \mu \) snow load shape coefficient
- \( s \) characteristic snow load on the roof

EC1-2-4: Wind loads
Three different types of action are considered in relation to time:

- **Permanent** actions
  - (G) dead loads

- **Variable** actions
  - (Q) eg imposed, wind, snow* loads

- **Accidental** actions
  - (A) eg fire, impact

*in some locations snow may be regarded as an accidental action

Actions may also be classified in relation to:

- **Spatial** variation
  - fixed (dead loads); free (imposed, snow, wind)

- **Nature** static; dynamic

Characteristic values, denoted by the subscript ‘k’, for different actions can be determined from appropriate parts of EC1. The subscript ‘d’ is used to denote design values. These are obtained by multiplying the characteristic values by a partial safety factor, $\gamma$.

For some cases in which more than one source of loads or actions is considered, the appropriate design loads are combined, using combination factors, $\psi$, to model the probability of such conditions arising in practice.

Subscripts inf and sup are sometimes used to qualify partial safety factors.

The subscript inf is used to denote lower bound values, and should be used where the action has a favourable effect, for example in considering the downwards permanent loads in a wind uplift case.

The subscript sup is used to denote upper bound values, and should be used where the action has an unfavourable effect. This would be the normal condition.

### 3. Eurocode 1 Part 2.1

EC1-2-1 provides detailed guidance on dead and imposed loads, and includes comprehensive data relating to material densities for both construction and stored materials. As with other Eurocodes it is organised in Chapters as follows:

**Chapter 1: General**

This outlines the scope, defines specific terms, and provides a
Chapter 2
Classification of actions
Provides brief guidance on permanent/variable and fixed/free actions

Chapter 3
Design situations
Provides broad comment on some relatively minor aspects of design

Chapter 4
Densities of building materials and stored materials
Tabulates detailed information relating to densities of common materials

Chapter 5
Self weight of construction elements
Provides detailed guidance for calculating self weight.

Chapter 6
Imposed loads on buildings
Provides detailed guidance for determining values of imposed loading for different types of building, including reduction factors for elements supporting large floor areas.

3.1 Dead loads
Dead loads are referred to as ‘self weight’ and are classified as permanent, fixed actions. They are calculated on the basis of material densities (see Table 1) and nominal dimensions.

Table 1. Densities of selected construction materials

<table>
<thead>
<tr>
<th>Materials</th>
<th>Density $\gamma$ [kN/m$^3$]</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>concrete</strong> (see ENV 206)</td>
<td></td>
</tr>
<tr>
<td>Lightweight (varies with density class)</td>
<td>9 - 20</td>
</tr>
<tr>
<td>normal weight</td>
<td>*24</td>
</tr>
<tr>
<td>heavyweight</td>
<td>&gt;28</td>
</tr>
<tr>
<td>reinforced and prestressed concrete; unhardened</td>
<td>+1</td>
</tr>
<tr>
<td>concrete</td>
<td></td>
</tr>
<tr>
<td>cement mortar</td>
<td>19 - 23</td>
</tr>
<tr>
<td>gypsum mortar; lime mortar</td>
<td>12 - 18</td>
</tr>
<tr>
<td>lime-cement mortar</td>
<td>18 - 20</td>
</tr>
<tr>
<td><strong>masonry units</strong> (see prEN 771)</td>
<td></td>
</tr>
<tr>
<td>dense limestone</td>
<td>20 - 29</td>
</tr>
</tbody>
</table>
The weight of fixed machinery, equipment and fittings should be considered as self weight, as may earth and ballast. Note that additional items may need to be included to allow for services and other similar elements which may not be clearly located at structural design stage. Although such loads are not strictly uniformly distributed, it is sufficiently accurate to model them as such. The magnitude of these allowances as equivalent uniformly distributed loads is best estimated on the basis of experience. The weight of fixed partitions may be considered as self weight and represented as an equivalent uniformly distributed load. No guidance is given as to the required magnitude of such an equivalent load intensity, and the designer should use a reasonable approach in making such an estimate. However a minimum
value of 1,0 kN/m² is typically used for offices with normal weight partitions and storey heights. A higher value may be appropriate for heavier and/or taller forms. The sequence shown in figure 1 details the procedures for assessment of the self weight of individual elements of construction such as walls, floors etc. The procedure will be familiar to most practising design engineers, and is based on calculating unit weights per square metre of construction.

![Flow chart for determining dead loads](image)

**Fig. 1** Flow chart for determining dead loads

### 3.2 Imposed loads

Imposed loads on a building are those arising from occupancy and are principally modelled as uniformly distributed loads. Characteristic values for these are tabulated in relation to building use. In special cases concentrated loads may be relevant, and these are also specified.
Imposed loads are generally classified as variable and free. Loads due to moveable partitions and services which may be repositioned are treated as imposed loads. The characteristic values of imposed loads are composed of long-term, medium-term, and short-term components. In practice there is often no need to distinguish between these except where materials are sensitive to time-dependent actions. For example, concrete is subject to creep, and load duration may therefore need to be considered in some aspects of the design of composite structures.

Building occupancy is defined in five categories (A-E) with some subdivisions as shown in Table 2 and corresponding characteristic imposed floor loadings are specified. These are identified in Table 3. In general, roofs with access should be designed for the same level of imposed load as for floors, but garage and vehicle traffic areas are treated separately.

### Table 2 Building occupancy categories

<table>
<thead>
<tr>
<th>Category</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Residential (including hospital wards, hotel bedrooms etc)</td>
</tr>
<tr>
<td>B</td>
<td>Office areas</td>
</tr>
<tr>
<td>C</td>
<td>Assembly areas (subdivided into 5 sections depending on likely density of occupation and crowding)</td>
</tr>
<tr>
<td>D</td>
<td>Shopping</td>
</tr>
<tr>
<td>E</td>
<td>Storage areas</td>
</tr>
</tbody>
</table>

### Table 3 Characteristic imposed floor loadings

<table>
<thead>
<tr>
<th>Loaded areas</th>
<th>$q_k$ [kN/m²]</th>
<th>$Q_k$ [kN]</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Category A</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>- general</td>
<td>2.0</td>
<td>2.0</td>
</tr>
<tr>
<td>- stairs</td>
<td>3.0</td>
<td>2.0</td>
</tr>
<tr>
<td>- balconies</td>
<td>4.0</td>
<td>2.0</td>
</tr>
<tr>
<td><strong>Category B</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>3.0</td>
<td>2.0</td>
</tr>
<tr>
<td><strong>Category C</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>- C1</td>
<td>3.0</td>
<td>4.0</td>
</tr>
<tr>
<td>- C2</td>
<td>4.0</td>
<td>4.0</td>
</tr>
<tr>
<td>- C3</td>
<td>5.0</td>
<td>4.0</td>
</tr>
<tr>
<td>- C4</td>
<td>5.0</td>
<td>7.0</td>
</tr>
<tr>
<td>- C5</td>
<td>5.0</td>
<td>4.0</td>
</tr>
<tr>
<td><strong>Category D</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>- D1</td>
<td>5.0</td>
<td>4.0</td>
</tr>
<tr>
<td>- D2</td>
<td>5.0</td>
<td>7.0</td>
</tr>
</tbody>
</table>
Provision is made for reducing imposed loads on structural elements which are supporting large floor areas. The reductions are applied through a factor, $\alpha$, applied according to simple formulae related to the area supported by a beam, or the number of storeys supported by a column.

For beams: $\alpha_A = 5\psi_0/7 + 10/A$

where $A$ is the loaded area supported by the beam ($m^2$)

For columns: $\alpha_n = (2 + (n - 2)\psi_0)/n$

where $n$ is the number of storeys supported

In both cases $\psi_0$ is determined from EN1-1 and described in section 6.2 below.

Figure 2 shows a flow chart for determining imposed load intensities:

1. Define the category of building and specific use according to Table 2.

2. From Table 3 determine the appropriate value of imposed load intensity $q_k$ ($kN/m^2$). Note that the alternative concentrated load ($Q_k$) is intended only for use in assessing structural performance over a very small area.

3. Determine the area supported by individual beams or the number of storeys supported by individual columns.

4. Determine imposed load reduction factors as appropriate.

5. In calculating the characteristic imposed load on individual elements, multiply by the corresponding reduction factor if it is less than 1.0.
Fig. 2  Flow chart for determining imposed load intensities

4. EC1 Part 2.3 Snow loads

EC1-2-3 provides detailed guidance on the calculation of roof loads due to snow, but excludes the following:
- sites over 1500m altitude
- impact loads associated with snow falling from a higher roof.
- additional wind loads due to ice accretion
- sites where snow is present all year
- lateral loads due to snow drifting
- the effect of heavy rain falling on snow

As with other Eurocodes it is organised in Chapters as follows:

Chapter 1  General
  This outlines the scope, defines specific terms, and provides a notation list

Chapter 2  Classification of actions
  Provides brief guidance on snow as a variable, free action, and in some cases an accidental action

Chapter 3  Design situations
  Provides broad comment of a rather general nature

Chapter 4  Representation of actions
  Provides broad comment on idealisation of snow as a uniformly distributed load

Chapter 5  Load arrangements
  Provides detailed guidance for calculating snow load intensities in relation to the snow load on the ground (Chapter 6) and the shape of the roof (Chapter 7).

Chapter 6  Snow load on the ground
  Provides the basis for determining snow load on the ground with reference to snow maps contained in Annex A.

Chapter 7  Snow load shape coefficients
  Provides detailed guidance on modifications to the uniform snow load applicable to roofs of different shapes.

Annex A  (Informati ve)
  Provides detailed guidance on characteristic snow loads on the ground for different geographical locations. Where appropriate, methods of adjusting these values for altitude are given.

4.1 Determination of snow loads

Snow loads, \( s \), are calculated on the basis of a characteristic load, \( s_k \), corresponding to a uniform depth of snow accumulated under calm
conditions on flat ground. This is modified to account for the shape of the roof and the effect of wind on the distribution of snow. (Note the value of $s$ may be reduced by exposure and thermal coefficients which account for more severe wind conditions, and reduced thermal insulation in the roof but these are generally taken as 1.0). The characteristic values are based on snow maps for individual member states, and relate $s_k$ directly to geographical location. A typical snow map is shown in Figure 3.
Fig. 3 Snow map for Finland

The influence of roof geometry is accounted for by a snow load shape coefficient, $\mu$, related to the roof pitch. Conventional pitched roofs, monopitch, multi-pitch and cylindrical roofs are treated separately. A typical chart giving values of snow load shape coefficient in relation to roof slope is shown in Figure 4 and Table 4. Note that for monopitch roofs two load cases should be considered, one in which the full snow load is applied over the full extent of the roof, and a second in which half the snow load intensity is applied over the most unfavourable half of the roof. The second load case will rarely be critical.

![Diagram of snow load shape coefficient](image)

**Fig. 4. Snow load shape coefficient in relation to slope for duopitch roofs.**

<table>
<thead>
<tr>
<th>Angle of pitch of roof, $\alpha$</th>
<th>$0^\circ \leq \alpha \leq 15^\circ$</th>
<th>$15^\circ \leq \alpha \leq 30^\circ$</th>
<th>$30^\circ \leq \alpha \leq 60^\circ$</th>
<th>$\alpha \geq 60^\circ$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shape coefficient $\mu_1$</td>
<td>0.8</td>
<td>0.8</td>
<td>0.8(60 - $\alpha$)/30</td>
<td>0.0</td>
</tr>
<tr>
<td>Shape coefficient $\mu_2$</td>
<td>0.8</td>
<td>0.8 + 0.6($\alpha$-15)/30</td>
<td>1.1(60-$\alpha$)/30</td>
<td>0.0</td>
</tr>
</tbody>
</table>

**Table 4 Snow load shape coefficient in relation to slope for duopitch roofs.**
Roofs with abrupt changes in height must account for the possibility of snow sliding from an upper level, and this is done by a further shape coefficient. Rules are given to enable the effect of snow overhanging the edge of a roof to be accounted for. These apply to those parts of a roof cantilevered out beyond the walls, but the magnitude of the allowance is independent of the length of overhang.

A flow chart for determining snow loads on roofs is shown in figure 5.

**Figure 5** Flow chart for determining snow loads on roofs

5. EC1 Part 2.4 Wind loads

EC1-2-4 provides detailed guidance on the calculation of wind loads. As with other Eurocodes it is organised in Chapters as follows:

- **Chapter 1** General
  - This outlines the scope, defines specific terms, and provides a notation list

- **Chapter 2** Classification of actions
  - Defines wind as a variable, free action.

- **Chapter 3** Design situations
  - Provides broad comment of a rather general nature

- **Chapter 4** Representation of actions
  - Provides broad comment on idealisation of wind as a
uniform pressure acting normal to the surface of a building

Chapter 5  
Wind pressure on the surface
Provides detailed guidance for calculating wind pressures on building surfaces.

Chapter 6  
Wind forces
Relates wind pressures to global forces

Chapter 7  
Reference wind
Provides detailed guidance on calculation of basic wind pressure by reference to Annex A.

Chapter 8  
Wind parameters
Provides detailed guidance on modifications to reference wind for special conditions

Chapter 9  
Choice of procedures
Defines the conditions for using the simple procedure

Chapter 10  
Aerodynamic coefficients
Provides more detailed procedures for calculating wind pressures on special structures.

Annex A  
(Informative)
Provides detailed guidance on reference wind speeds for different geographical locations.

Annex B  
(Informative)
Provides detailed procedures for assessing the dynamic behaviour of structures

Annex C  
(Informative)
Provides detailed rules for vortex excitation and other aeroelastic effects

5.1 Determination of wind loads

Two approaches are presented in ENV 1991-2-4. The simple method is applicable where structures are unlikely to be susceptible to dynamic excitation. In practice, steel framed buildings less than 200m tall should satisfy this condition, but for very slender or unusual structures, more detailed calculations may be required to justify this. This section covers only the simplified approach and applies to ‘permanent’ structures. In general temporary structures may be designed for lower wind pressures.

Wind loads are treated as quasi-static pressures acting normal to the surface of the building (except in special circumstances such as tangential friction forces due to wind blowing over large flat surfaces). The characteristic wind pressure is the reference mean wind velocity pressure, \( q_{\text{ref}} \), derived from the corresponding velocity \( (v_{\text{ref,0}}) \). The reference wind velocity \( (v_{\text{ref,0}}) \) is determined from wind maps for each member state. These are detailed in Annex A of the code. Exceptionally, the value of \( v_{\text{ref,0}} \) may need to be modified to account for altitude or wind direction for some countries.
Indicative values of the reference wind velocity for all of Europe are shown in Figure 6.

Fig. 6. European wind map (indicative values only).

The reference wind pressure is then determined directly:

Fig. 7.2

cit. 7.1
\[ q_{rel} = 0.5 \rho v_{ref}^2 \]

where \( \rho \) is the density of air, taken as 1.25\( \text{kg/m}^3 \) unless otherwise specified.

This is modified by an exposure coefficient \( c_e(z) \), which takes account of terrain roughness according to Table 5, topography and height above ground. For ‘flat’ terrain the exposure coefficient \( c_e(z) \) can be determined from Figure 7 which relates height and terrain category. Terrain is generally considered ‘flat’ except for locations close to isolated hills and escarpments.

### Table 5. Terrain categories and related parameters

<table>
<thead>
<tr>
<th>Terrain Category</th>
<th>( k_r )</th>
<th>( z_o [\text{m}] )</th>
<th>( z_{\text{min}} [\text{m}] )</th>
<th>( \varepsilon )</th>
</tr>
</thead>
<tbody>
<tr>
<td>I Rough open sea, lakes with at least 5km fetch upwind and smooth flat country without obstacles</td>
<td>0.17</td>
<td>0.01</td>
<td>2 [0.13]</td>
<td></td>
</tr>
<tr>
<td>11 Farmland with boundary hedges, occasional small farm structures, houses or trees</td>
<td>0.19</td>
<td>0.05</td>
<td>4 [0.26]</td>
<td></td>
</tr>
<tr>
<td>11 Suburban or industrial areas and permanent forests</td>
<td>0.22</td>
<td>0.3</td>
<td>8 [0.37]</td>
<td></td>
</tr>
<tr>
<td>IV Urban areas in which at least 15% of the surface is covered with buildings and their average height exceeds 15m</td>
<td>0.24</td>
<td>1</td>
<td>16 [0.46]</td>
<td></td>
</tr>
</tbody>
</table>

Note: The parameters of this table are calibrated to obtain the best fit of available data. The values \( k_r \), \( z_o \) and \( z_{\text{min}} \) are used in 8.2, the value \( \varepsilon \) is used in Annex B (Section 3).
Fig. 7. Exposure coefficient $c_e(z)$ as a function of height $z$ above ground and terrain categories I to IV for ‘flat’ terrain. - $c_1 = 1$

Note that tall buildings may be considered in sections with appropriate height ranges, and wind pressures calculated for each section according to the corresponding ‘reference height’, as shown in fig. 8.

Fig. 8. Reference height $z_e$ as a function of the breadth and actual building height. Wind pressure may be considered separately within the separate zones.

Wind pressure distribution
The net pressure on a surface is the algebraic sum of the internal and...
external pressures. This is indicated in fig. 9.

Fig. 9  Pressure on surfaces of buildings
Wind pressure coefficients $c_{pe}$ and $c_{pi}$ are applied to determine the distribution of external and internal pressures respectively. Values are tabulated for

- vertical walls of rectangular buildings
- flat roofs
- monopitch roofs
- duopitch roofs
- hipped roofs

A typical chart is shown in Fig. 10 and Table 6.
Key for vertical walls.

Fig. 10 External pressure coefficients for vertical walls of rectangular plan buildings.
Table 6 External pressure coefficients for vertical walls of rectangular plan buildings.

<table>
<thead>
<tr>
<th>Zone</th>
<th>A</th>
<th>B, B*</th>
<th>C</th>
<th>D</th>
<th>E</th>
</tr>
</thead>
<tbody>
<tr>
<td>d/h</td>
<td>C_{pe,1}</td>
<td>C_{pe,1}</td>
<td>C_{pe,1}</td>
<td>C_{pe}</td>
<td>C_{pe,1}</td>
</tr>
<tr>
<td>≤ 1</td>
<td>-1,0</td>
<td>-1,3</td>
<td>-0,8</td>
<td>-1,0</td>
<td>-0,5</td>
</tr>
<tr>
<td>≥ 4</td>
<td>-1,0</td>
<td>-1,3</td>
<td>-0,8</td>
<td>-1,0</td>
<td>-0,5</td>
</tr>
</tbody>
</table>

For buildings without internal partitions internal pressure coefficients are related to the opening ratio, \( \mu \), which is defined as:

\[
\mu = \frac{\text{Total area of openings at the leeward and wind parallel sides}}{\text{Total area of openings at the windward, leeward and wind parallel sides}}
\]

For an homogeneous distribution of openings for an approximately square building, \( c_{pi} \) may be taken as -0.25.
For closed buildings with internal partitions, the worst case may be
assumed:
\[ c_{pi} = 0.8, \text{ or } c_{pi} = -0.5 \]
Special consideration is given to canopy roofs, buildings with more than one skin, solid boundary walls, fences and signboards, and special structures.
The process for determining wind loads on a building is summarised in fig. 11.
Mean wind velocity: \( v_{ref} \) – (wind ‘maps’) – exceptionally modified for altitude or direction

Mean pressure \( q_{ref} = 0.5 \rho v_{ref}^2 \) (\( \rho = 1.25 \))

Reference height \( z \)

Pressure coefficients
External pressure: \( c_{pe} \) – Tables 10.2.1-5 for walls and roofs of various shapes
Internal pressure: \( c_{pi} \) calculated in relation to \( \mu \), ratio and distribution of openings
Repeat for alternative wind directions

Terrain category I-IV

Exposure coefficient \( c_e(z_e) \)

External pressure \( \text{We} = q_{ref} c_e(z_e) c_{pe} \)

Internal pressure \( \text{Wi} = q_{ref} c_e(z) c_{pi} \)

Net pressure on surface = \( \text{We} +/- \text{Wi} \)
Fig. 11. Flow chart for calculating wind pressures (for buildings not susceptible to dynamic excitation - in practice this applies to 'normal buildings less than 200m high).

6. Load cases
6.1 Design values of loads

Design values are obtained from characteristic values multiplied by appropriate partial safety factors. These account for accidental overloading and inevitable inaccuracies in the analysis. They also implicitly incorporate the global factor of safety necessary in structural design. The magnitude of the partial safety factor depends on the limit state under consideration and the particular type of action. These are detailed in Table 7 which includes three 'classes' for consideration. Class A refers to loss of static equilibrium and should be used, for example, where consideration of overall stability is considered. Class B refers to failure of structure or structural elements and is the condition most likely to apply. Class C refers to cases associated with failure of the ground.

The basic values of partial safety factors to be used in design for Class B - failure of structure or structural elements are summarised in Table 7 for both ultimate (Class B) and serviceability limit states.

Table 7. Basic values of partial safety factors - Class B (failure of structure or structural elements)

<table>
<thead>
<tr>
<th></th>
<th>ULS</th>
<th>SLS</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Q</td>
<td>G</td>
</tr>
<tr>
<td>Unfavourable</td>
<td>1,5</td>
<td>1,35</td>
</tr>
<tr>
<td>effect</td>
<td>(1,35)*</td>
<td></td>
</tr>
<tr>
<td>Favourable</td>
<td>0,0</td>
<td>1,0</td>
</tr>
<tr>
<td>effect</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

*Note that two values are quoted for variable loads. The appropriate value depends on whether a single variable load, or a combination of, for example, imposed and wind loads, is considered, and the simplified method for dealing with load combinations is adopted. This is discussed in more detail below. Note also that the characteristic values of all permanent actions from one source are multiplied by 1,35 if the total effect is unfavourable or by 1,0 if it is favourable. This implies, for example, that for a continuous beam the self weight of the floor should be factored by the same factor for all spans, with no allowance for pattern loading due to permanent actions.

For consideration of Class A (loss of equilibrium) the following partial safety factors apply:

Unfavourable
Permanent $\gamma = 1.1$

Variable $\gamma = 1.5$

Favourable

Permanent $\gamma = 0.9$

Variable $\gamma = 0.0$

6.2 Combination values

When a load combination includes more than one variable load (for example imposed loads and wind loads) the partial safety factors relating to the variable load components change, and each variable load, except the one which has the greatest effect, is multiplied by a combination factor, $\psi$. If it is not clear which variable load has the greatest effect, all combinations should be considered.

The value of the combination factor to use depends on the circumstances, the load type, and the use of the building. For 'normal' circumstances (other than those concerning accidental or seismic situations), the appropriate factor is designated $\psi_0$. The values of $\psi_0$ are as follows:

- Imposed loads for all buildings except storage: $0.7$
- Imposed loads for storage buildings: $1.0$
- Snow loads: $0.6$
- Wind loads: $0.6$

Potentially this gives rise to a very large number of possible load combinations, each of which would require separate consideration and analysis. Fortunately, a simplified approach is allowed for conditions known from previous experience to be critical, and this should be satisfactory for most building designs. The simplified approach is described below.

6.3 Simplified treatments

EC1 includes a simplified treatment for building structures in normal design situations. This eliminates the use of the combination ($\psi$) factors and uses modified partial load factors. These expressions imply a single permanent action, generally comprising the dead load, $G_k$. The dead load will be combined with appropriate variable actions, typically those due to imposed, snow and wind loads. For simple floor and roof structures the predominant actions are gravity loads (dead and imposed for floors; dead and snow for roofs), but for framed structures the additional influence of wind loads must of course be considered in combination. Thus typical load combinations for cases where the actions are all detrimental are as follows:

- Serviceability Limit State (for Class B - failure of structure or structural elements):
  - Dead + imposed (or snow): $G_k + Q_k$
  - Dead + imposed (or snow) + wind: $G_k + 0.9 \sum Q_k$
Ultimate Limit State (for Class B - failure of structure or structural elements)
Dead + imposed (or snow): \(1.35G_k + 1.5Q_k\)
Dead + imposed (or snow) + wind: \(1.35G_k + 1.35\sum Q_k\)

In some circumstances, certain actions may have a beneficial effect. For example, dead loads may assist in resisting wind or overturning, and imposed loads on the central span of a continuous beam can relieve bending in the adjacent spans. Under these circumstances, the lower value (inferior) of the partial safety factor should be applied to the beneficial action. In practice, for Class B conditions, imposed loads which are beneficial are simply ignored \(\gamma_{\text{inf}} = 0\), whilst for dead loads resisting wind effects, a partial factor of 1.0 should be used.

7. Worked Examples

7.1 Dead loads
The floor construction of a commercial office building consists of:
- 50mm sand/cement screed
- 150mm normal weight reinforced concrete slab
- 12mm plaster (gypsum mortar) ceiling

Densities for each of these materials can be found from EC1-2-1 Table 4.1. These can be used to determine the dead load intensity \((\text{kN/m}^2)\) as indicated in Table below.

<table>
<thead>
<tr>
<th>Component</th>
<th>Density (kN/m(^3))</th>
<th>Thickness (m)</th>
<th>Load intensity = density (\times) thickness ((\text{kN/m}^2))</th>
</tr>
</thead>
<tbody>
<tr>
<td>Screed</td>
<td>20 (19 - 23)</td>
<td>0.05</td>
<td>(20 \times 0.05 = 1.0)</td>
</tr>
<tr>
<td>Slab</td>
<td>24</td>
<td>0.15</td>
<td>(24 \times 0.15 = 3.6)</td>
</tr>
<tr>
<td>Plaster</td>
<td>15 (12 - 18)</td>
<td>0.012</td>
<td>(15 \times 0.012 = 0.2)</td>
</tr>
</tbody>
</table>

Including allowances for partitions and services, the calculations may be set out as follows:
- Screed: \(20 \times 0.05 = 1.0\ \text{kN/m}^2\)
- Slab: \(24 \times 0.15 = 3.6\)
- Plaster: \(15 \times 0.012 = 0.2\)
- Partitions: \(1.0\)
- Services: \(0.3\)

Total characteristc load: \(6.1\ \text{kN/m}^2\)

7.2 Imposed loads
Consider the case of a commercial office building. From EC1-2-1 Table 6.1, the occupancy relates to category B (offices) and from Table 6.2 the corresponding imposed floor load intensity is 3.0 \text{kN/m}^2. For beams and columns attracting load from wide areas, this value can be reduced.
Consider a main floor beam spanning 10m and supporting secondary beams which span 8m. The total area, A, supported by the main beam is therefore:

\[ A = 10.0 \times 8.0 = 80 \text{ m}^2 \]

The corresponding reduction factor \( \alpha \) is then given by:

\[ \alpha_A = \frac{5 \psi_o}{7} + \frac{10}{A} \]

For imposed floor loads (excluding storage areas) \( \psi_o = 0.7 \)

\[ \alpha_A = \frac{5 \times 0.7}{7} + \frac{10}{80} = 0.5 + 0.125 = 0.625 \]

The imposed load to the beam can be reduced by this factor.

If the beam is subject to the dead load intensity specified in 7.1 above the total beam load is as follows:

Dead load \[ 6.1 \times 8.0 \times 10.0 = 488 \text{ kN} \]

Imposed load \[ 0.625 \times 3.0 \times 8.0 \times 10.0 = 150 \text{ kN} \]

For typical ultimate limit state calculations these would be combined using appropriate partial safety factors (1.35 for dead loads and 1.5 for imposed loads)

Design load to beam (ULS) = \[ 1.35 \times 488 + 1.5 \times 150 = 659 + 225 = 884 \text{ kN} \]

For typical serviceability limit state calculations these would be combined using appropriate partial safety factors (1.0 for dead loads and 1.0 for imposed loads)

Design load to beam (SLS) = \[ 488 + 150 = 638 \text{ kN} \]

For columns: \( \alpha_n = \frac{2 + (n -2)\psi_o}{n} \)

where \( n \) is the number of storeys supported

Thus for a column supporting 4 storeys:

\[ \alpha_n = \frac{2 + (4 -2) \times 0.7}{4} = 0.85 \]

If the column supports two of the beams detailed in the previous example at each storey, the total dead and imposed column loads are as follows:

Dead load \[ = 4 \times 488 = 1952 \text{ kN} \]

Imposed load \[ = 4 \times 0.85 \times 3.0 \times 8.0 \times 10.0 = 816 \text{ kN} \]

For typical ultimate limit state calculations these would be combined using appropriate partial safety factors (1.35 for dead loads and 1.5 for imposed loads)

Design load to column (ULS) = \[ 1.35 \times 1952 + 1.5 \times 816 = 2635 + 1224 = 3859 \text{ kN} \]
7.3 Snow loads

A building in Helsinki has a 24° duo pitched roof. The characteristic snow load on the ground $s_k$, is 2.5 kN/m². The snow load shape coefficients for the roof are:

$$
\mu_1 = 0.8; \quad \mu_2 = 0.8 + 0.6 \frac{(24 - 15)}{30} \\
= 0.8 + 0.2 \frac{3}{30} \\
= 0.8 + 0.02 \\
= 0.82
$$

Possible load arrangements:

$W_1 = 0.82 \times 2.5 = 2.05$  
$W_2 = 0.8 \times 2.5 = 2.0$

$W_1 = 0.5 \times 0.8 = 1.0$  
$W_2 = 0$

Consider the same roof as above but with unequal slopes of 24° and 36°.

$$
\mu_{13} = 0.8 \frac{(60 - 36)}{30} \\
= 0.64
$$

$$
\mu_{24} = 0.98
$$

Possible load arrangements:

$W_1 = 0.88 \times 2.5 = 2.2$  
$W_2 = 0.8 \times 2.5 = 2.0$

$W_1 = 0.5 \times 0.64 \times 2.5 = 0.8$  
$W_2 = 0$

$W_1 = 0.64 \times 2.5 = 1.6$  
$W_2 = 0.98 \times 2.5 = 2.45$

$W_1 = 0$  
$W_2 = 0.5 \times 0.8 \times 2.5 = 1.0$

Notes:

Even for a flat roof there is a reduction factor of 0.8. In most cases it may be conservatively assumed that maximum snow load intensity is applied uniformly across the full width of the roof.
7.4 Wind loads

Consider a building in suburban Athens, 30m high and with plan dimensions 20m × 20m. The building is 'closed' with internal partitions.

Mean wind velocity $v_{\text{ref}} = 36$ m/s

Mean wind pressure $q_{\text{ref}} = 0,5 \times 1,25 \times 36^2 = 0,81 \text{ kN/m}^2$

Reference height: consider the building in two zones $ze = 20$ m and $ze = 30$ m.

Terrain category III ($kt = 0,22$; $ze(m) = 0,3$; $zmin(m) = 8$; $e = 0,37$)

Exposure coefficients

$c_{e(20)} = 2,25$; $c_{e(30)} = 2,55$

Pressure coefficients:

Internal pressure coefficient $c_{pi} = 0,8$ or -0,5

External pressure coefficients:

$d/h = 20/30 = 0,67$

$e = \text{smaller of } b (= 20) \text{ and } 2h (= 60), \text{ so } e = 20$

$d = e \text{ so consider wind pressure on side walls of building as uniform (i.e., pressures in zones B, B' and C are all equal) - except for leading corner (zone A).}$

<table>
<thead>
<tr>
<th></th>
<th>A</th>
<th>B/B*</th>
<th>D</th>
<th>E</th>
</tr>
</thead>
<tbody>
<tr>
<td>$C_{pe10}$</td>
<td>-1,0</td>
<td>-0,6</td>
<td>+0,8</td>
<td>-0,3</td>
</tr>
<tr>
<td>$C_{pe1}$</td>
<td>-1,0</td>
<td>-1,0</td>
<td>+1,0</td>
<td>-0,3</td>
</tr>
</tbody>
</table>

External wind pressure:

Up to 20m:

A: $w_{e(2)} = 0,81 \times 2,25 \times (-1,0) = -1,82 \text{ kN/m}^2$

B: $w_{e(2)} = 0,81 \times 2,25 \times (-0,8) = -1,46 \text{ kN/m}^2$

D: $w_{e(2)} = 0,81 \times 2,25 \times (+0,8) = +1,46 \text{ kN/m}^2$

E: $w_{e(2)} = 0,81 \times 2,25 \times (-0,3) = -0,55 \text{ kN/m}^2$

From 20m to 30m

A: $w_{e(3)} = 0,81 \times 2,55 \times (-1,0) = -2,07 \text{ kN/m}^2$

B: $w_{e(3)} = 0,81 \times 2,55 \times (-0,8) = -1,65 \text{ kN/m}^2$

D: $w_{e(3)} = 0,81 \times 2,55 \times (+0,8) = +1,65 \text{ kN/m}^2$

E: $w_{e(3)} = 0,81 \times 2,55 \times (-0,3) = -0,62 \text{ kN/m}^2$

Internal wind pressure

Up to 20m:

Positive $w_{i(20)} = 0,81 \times 2,25 \times = +1,46 \text{ kN/m}^2$
RAKENNUSTEKNIKKA
Olli Ilveskoski

(+) 0,8

Negative

\[ w_{(20)} = 0.81 \times 2.25 \times (-0.3) = -0.55 \text{ kN/m}^2 \]

From 20m to 30m

Positive

\[ w_{(30)} = 0.81 \times 2.55 \times (+0.8) = +1.65 \text{ kN/m}^2 \]

Negative

\[ w_{(30)} = 0.81 \times 2.55 \times (-0.3) = -0.62 \text{ kN/m}^2 \]

Net wind loads are calculated by combining the internal and external wind pressures algebraically. In doing so it is important to be consistent in relation to wind direction, and direction of internal pressure. Note that for rectangular (not square) buildings it will be necessary to consider wind in two orthogonal directions.

7.5 Load combinations

7.5.1 3-span continuous beam (ULS for failure of structure)

For consideration of failure of structural elements partial safety factors are determined from Table 9.2. For a three span continuous beam, a number of conditions must be considered. These correspond to different combinations of loading on the three spans. If these are designated \( W_1 \), \( W_2 \) and \( W_3 \) and the beams each span 10m, are at 8m centres, and are subject to loadings as calculated in 7.1-2 above) the load combinations to be considered are as follows:

Maximum bending in span 1:

\[ W_1 = 1.35 \, G_k + 1.5 \, Q_k; \quad W_2 = 1.35 \, G_k; \quad W_3 = 1.35 \, G_k + 1.5 \, Q_k \]

\( W_1 = 884 \text{ kN} \)

\( W_2 = 1.35 \times 488 = 659 \text{ kN} \)

Maximum bending in span 2:

\[ W_1 = 1.35 \, G_k; \quad W_2 = 1.35 \, G_k + 1.5 \, Q_k; \quad W_3 = 1.35 \, G_k \]

\( W_1 = W_3 = 659 \text{ kN} \)

\( W_2 = 884 \text{ kN} \)

For maximum loads in the supporting internal columns, the maximum load would be considered for all spans, ie \( W_1 = W_2 = W_3 = 884 \text{ kN} \)
7.5.2 2-storey building - single bay with balcony (ULS - loss of equilibrium)

For consideration of loss of equilibrium partial safety factors are determined from Table 9.1.

For a 2-storey single bay building with cantilever, a number of conditions must be considered. These correspond to different combinations of dead, imposed and wind loading, including different patterns of vertical load on the span and cantilever at first floor and roof. If the loads are designated \( W_{r1}, W_{r2} \) (for the roof span and cantilever), \( W_{f1}, W_{f2} \) (for the floor span and cantilever), and \( W_h \) is the horizontal load due to wind, the load combinations to be considered are as follows:

- \( \psi_0 = 0.6 \) (snow load); \( \psi_0 = 0.6 \) (wind loads); \( \psi_0 = 0.7 \) (floor imposed loads)
- \( W_{r2} = 1.35 G_{kr} + 1.35 Q_{kr} \)
- \( W_{f1} = 1.0 G_{kf} \)
- \( W_{f2} = 1.35 G_{kf} \)

Considering imposed roof loads dominant:
- \( W_{r1} = 1.0 G_{kr} \)
- \( W_{r2} = 1.35 G_{kr} + 1.5 Q_{kr} \)
- \( W_{f1} = 1.0 G_{kf} \)
- \( W_{f2} = 1.35 G_{kf} + \psi_0 \) (floor) \((1.50 Q_{kf})\)
  \[ = 1.35 G_{kf} + 0.7 \times 1.50 Q_{kf} \]
  \[ = 1.35 G_{kf} + 1.05 Q_{kf} \]
- \( W_h = \psi_0 \) (wind) \((1.5 W_k)\)
  \[ = 0.6 \times 1.5 W_k \]
  \[ = 0.9 W_k \]
Considering imposed floor loads dominant:

\[ W_{r1} = 1.0 \ G_{kr} \]
\[ W_{r2} = 1.35 \ G_{kr} + \psi_0 \ (snow) \ (1.50 \ Q_{kr}) \]
\[ = 1.35 \ G_{kd} + 0.6 \times 1.50 \ Q_{kr} \]
\[ = 1.35 \ G_{kd} + 0.9 \ Q_{kr} \]
\[ W_{f1} = 1.0 \ G_{kf} \]
\[ W_{f2} = 1.35 \ G_{kf} + 1.5 \ Q_{kf} \]
\[ W_h = \psi_0 \ (wind) \ (1.5 \ W_k) \]
\[ = 0.6 \times 1.5 \ W_k \]
\[ = 0.9 \ W_k \]

Considering wind loads dominant

\[ W_{r1} = 1.0 \ G_{kr} \]
\[ W_{r2} = 1.35 \ G_{kr} + \psi_0 \ (snow) \ (1.50 \ Q_{kr}) \]
\[ = 1.35 \ G_{kd} + 0.6 \times 1.50 \ Q_{kr} \]
\[ = 1.35 \ G_{kd} + 0.9 \ Q_{kr} \]
\[ W_{f1} = 1.0 \ G_{kf} \]
\[ W_{f2} = 1.35 \ G_{kf} + \psi_0 \ (floor) \ (1.5 \ Q_{kf}) \]
\[ = 1.35 \ G_{kf} + 0.7 \times 1.50 \ Q_{kf} \]
\[ = 1.35 \ G_{kf} + 1.05 \ Q_{kf} \]
\[ W_h = 1.5 \ W_k \]

Considering all variable loads in combination using the simplified approach:

\[ W_{r1} = 1.0 \ G_{kr} \]
\[ W_{r2} = 1.35 \ G_{kr} + 1.35 \ Q_{kr} \]
\[ W_{f1} = 1.0 \ G_{kf} \]
\[ W_{f2} = 1.35 \ G_{kf} + 1.35 \ Q_{kf} \]
\[ W_h = 1.35 \ G_k \]

8. Concluding Summary

- The principal actions to be considered in the design of buildings are dead and imposed loads, snow, and wind.
- Dead loads are calculated on the basis of material densities specified in EC1-2-1 and construction details.
- Imposed loads are defined in EC1-2-1 in relation to building usage, classified as residential, office, assembly (subdivided according to likely density of occupation), shopping, and storage.
- Calculation procedures for snow loads are detailed in EC1-2-2 and are based on geographical location and roof shape.
- Calculation procedures for wind loads are detailed in EC1-2-3 and are based on.
- For most buildings wind loads can be assumed to be determined from quasi-static pressures determined according to location, exposure and building height.
• Internal wind pressures depend on the distribution of openings in walls.
• The distribution of external wind pressures on walls and roof slopes depends on the geometry of the building.
• Partial safety factors are used to account for uncertainties in load levels; values depend on load type, the limit state considered, and the design condition (structural failure, loss of equilibrium or failure of the ground).
• Partial safety factors are modified to account for different combinations of actions, but a simplified approach can be adopted.