Actions on structures

STEP lecture A3
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Objectives

To give an overview of the classification of the actions applied to structures. To define the characteristic value for the most common actions applied to buildings. To present the design situations and the associated values for combined actions.

Summary

In accordance with EC1, this lecture deals with the evaluation of the actions used in EC5 design calculations. Regardless of dynamic effects, the representative values of the actions on buildings depend on their variation with time. These values are established for permanent, imposed, snow and wind actions. Then, the combined value of actions is calculated for the various design situations. A typical example of the calculation of the actions for a frame complements the lecture.

Introduction

For the intended construction work, the designer is first faced with the conceptual design of the structural system. This stage will consider the type of structure and on construction material to be used. The structural design then starts with an analysis of the actions that may be applied to the chosen structure. Account should be taken of direct actions that are the applied external forces as well as the indirect actions that result from imposed deformations (e.g. settlement of supports or dimensional change induced by moisture variations).

Regardless of the construction material, the design requires the evaluation of the actions that may act during the life of the structure. These depend on the structural form, on the type of construction work and on the method of construction. At this stage, it is necessary to consider the nature of the actions or action-effects, i.e. either static or dynamic, to achieve an accurate structural analysis. For example, the quasi-static assumption may not be acceptable in the following cases:

- floors subjected to human or machine-induced vibrations,
- flexible plate-like structures such as suspension-bridge decks that could flutter when subjected to wind velocities above a critical value,
- structures loaded by ground acceleration due to seismic action.

In these cases, a dynamic analysis model should be used to find the action-effects of the force-time history, considering the stiffness, the mass and the damping ratio of structural members. However, the resonant component of the action-effect is small for most structures. Therefore the static calculations are made, and an equivalent dynamic amplification factor applied to the static value of action.

This lecture, therefore, deals with the assessment of direct actions and their combination for static analysis only. These calculations will also need to consider the National Application Documents and current regulations applicable to the country where the structure is constructed.

General concepts

Structural classifications

The design Eurocodes (EC2 to EC7) are based on a calibration of successful traditional design methods. Nevertheless, a mention should be made of the criteria to which the reliability concept of EC1 referred. Regarding human hazard and economic losses, the structural safety and serviceability requirements consider the working life and the design situations of the structures (C.E.B., 1980).

Class	Working life *(years)*	Example
1	1 to 5	Temporary structures
2	25	Replaceable structural elements
3	50	Buildings and common structures
4	100	Bridges or other engineering works

Table 1 Design working life classification.

The working life corresponds to the period for which the structure is to be used for its intended purpose. Table 1 gives a classification of the construction works. In addition, the design situations refer to events that may occur during the working life of the structure. Therefore, the actions are evaluated for the relevant design situations that are classified as:

- *persistent situations* related to the conditions of normal use,
- *transient situations* related to temporary conditions, e.g. during execution,
- *accidental situations* related to exceptional conditions like fire or impact,

Load classification

In addition to the previous classifications, differentiation of the actions has to be considered according to the variation of their magnitude in space and with time. For common design, the actions or action-effects are defined as:

- *permanent actions* (G), e.g. self-weights of the construction works,
- *variable actions* (Q), e.g. imposed actions, snow and wind actions.

Other actions like accidental (A) and seismic (S) actions are outside the scope of this lecture (see STEP lectures A2, B17 and C17).

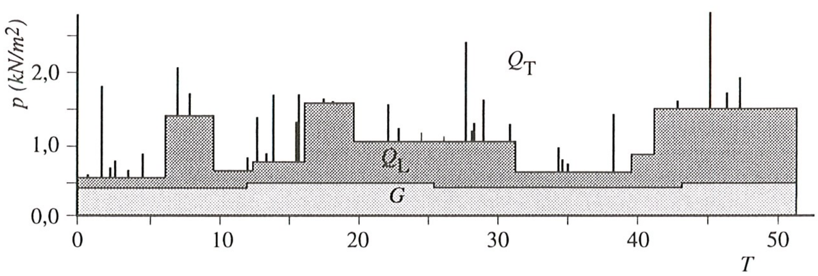

Figure 1 Time-variation of the total applied actions on a floor.

The permanent actions have negligible variation in magnitude with time, except when changes to a construction are made (see Figure 1). For the variable actions (Hendrickson et al, 1987, Rackwitz, 1976), the variations are modelled as a discontinuous process (i.e. snow or wind) or as a process resulting from a sustained part, Q_L, and a transient part, Q_T (i.e. imposed load). For timber which is more time-dependent than other construction materials, the temporal variation of the actions must be emphasised. According to EC5, the design criteria must take into account the load-duration effects. Therefore, the designer must classify the variable actions in relation to the specified load-duration classes (see STEP lecture A2).

In terms of spatial variations, the actions are considered either as fixed or free. Free actions could have any spatial distribution over the structure or part of it. Then, the design is carried out using the worst load arrangements of the free actions.

Representative values of actions

The basic value of an action is *the characteristic value*, denoted G_k or Q_k. Usually, the permanent actions G_k correspond to the nominal value. However, if the structure is sensitive to variation in G or if the coefficient of variation (COV) of G is greater than 10%, two characteristic values are considered, a lower value $G_{k,inf}$ and upper value $G_{k,sup}$. Assuming a Gaussian distribution for G, these values are given by:

$$G_{kinf} = G_{mean}\ (1 - 1{,}64\ COV) \quad ; \quad G_{ksup} = G_{mean}\ (1 + 1{,}64\ COV) \qquad (1)$$

The characteristic variable actions Q_k are related to a given return period of N years, corresponding to a probability of exceedance $p = 1/N$ in a year. According to EC1, the actions Q_k are defined for N=50 years or p=0,02. For other probabilities of exceedance p_N (with $p_N \leq 0{,}2$), the characteristic value Q_N is estimated as:

$$Q_N = Q_k\ \frac{1 - COV\ \frac{\sqrt{6}}{\pi}\ [\ \ln(-\ln(1-p_N)) + 0{,}57722]}{1 + 2{,}5923\ COV} \qquad (2)$$

where COV is the coefficent of variation of Q.

If permitted by National regulations, this relation may be appropriate to define the characteristic value of a variable action:

- from values related to a return period less than 50 years (e.g. snow or wind),
- for structural design with an acceptable higher risk of exceedance (i.e. temporary structures) or, conversely, with a greater safety (p_N<0,02).

In addition, the designer needs to consider other representative values for variable actions given as:

- the combination value ($\psi_0 Q_k$),
- the frequent value ($\psi_1 Q_k$), which is exceeded for 5 percent of the time,
- the quasi-permanent value ($\psi_2 Q_k$), which is related to the time average value.

In practice, the values G_k, Q_k, ($\psi_0 Q_k$) and ($\psi_1 Q_k$) are usually considered when checking the ultimate limit states. For the serviceability limit states, these values are used for the calculations of short-term effects only. The long-term effects (e.g. creep deformations) are assessed considering the values G_k and ($\psi_2 Q_k$) on the loading side, and the deformation factor k_{def} on the material side.

Permanent actions

The permanent actions are due to the self-weight of structural members and the weights of all components to be supported permanently by the members. These dead loads comprise fixed partitions, insulation, cladding or finishes. The estimation of the permanent actions requires knowledge of the structural configuration and the construction materials. The values of the permanent actions are established using the nominal dimensions of the components and the mean weight density of the constituent materials (in kN/m^3). For many building products, the designer should refer to the weights given by the manufacturer.

In order to simplify the calculations, the dead loads due to framing members and lightweight partitions are conveniently defined as uniformly distributed loads over the building area. A reasonable estimate may be obtained by referring to similar structures. The self-weight of the flooring (sheet and joist) or roofing (sheet, rafters and purlins) members ranges usually between 0,25 and 0,45 kN/m^2. For common framing members, the overall weight could be estimated as $g=(15+l)/100$ kN/m^2 where l is the span of the members in metres.

Depending on the weight P of the partition per m^2 of wall area, the partitions may be taken into account as a uniform load equal to 0,75 P per m^2 of floor area. This estimate is used for partitions up to four metres in height if P is less than 1,0 kN/m^2 and less than 40% of the imposed actions.

Imposed actions

The imposed actions in buildings are due to occupancy. They correspond to loads that move by themselves (i.e. people, trucks) and to moveable loads (i.e. furniture, light partitions, stored materials). Distinction is made between the loaded areas according to the intended use. In common buildings, three classes have to be considered: 1- dwellings, offices, shops . . ., 2- roofs and 3-production areas.

Category	Type of use	Example
A	Residential activities	Apartments, bedrooms in hotels
B	Offices	Classrooms, operating rooms in hospital
C	Congregation areas	Assembly halls, theatres, dining rooms
D	Shopping	Areas in warehouses
E	Storage	Archives, storage area of goods

Table 2 Classification of floor areas in buildings.

For production areas, the design is achieved with imposed actions on floors depending on the specific use of the buildings. Otherwise, the values of the imposed actions take into account the density of occupation and the degree of public access to the area. Thus, the first class is subdivided into five categories (Table 2). Roofs are categorized as not accessible except for maintenance or repair (Category H) or as accessible. For accessible roofs, the design is made with the occupancy corresponding to the floor classification.

EC1: Part 2-1

Referring to this classification, the design of a floor or roof takes into account either a uniformly distributed load q_k or a concentrated load Q_k as imposed action. The free load Q_k acts on a square area with a 50 mm side. This load is intended to ensure adequate design of secondary members. It may be also critical on small spans. Table 3 gives the minimum values of these imposed actions as specified in EC1. Reduction coefficients can be applied to these values depending on the floor area and the number of storeys.

According to the load-duration classes of EC5, a medium-term duration is usually considered for the load q_k on areas A to D. This loading is taken as long-term for category E and as short-term for category H. Lastly, the concentrated action Q_k is related to the short-term duration class.

Category		Type of area	q_k (kN/m^2)	Q_k (kN)
Floors, accessible roofs:	A	General	2	2
		Stairs	3	2
		Balconies	4	2
	B	General	3	2
		Stairs,balconies	4	3
	C	General	5	4
		areas with tables	3	4
		areas with fixed seats	4	4
		possibility of concentrations	5	7
	D	Shops	5	4
		Department store	5	7
	E	General	5	7
Non-accessible roofs:	H	slope: $< 20°$	0,75	1,5
		$> 40°$	0	1,5

Table 3 Imposed loads on floors and roofs in buildings

Apart from the previous gravity loads, account may also be taken of horizontal imposed actions on partition walls and barriers. They are short-term actions applied at the height of the hand rails (0,8 to 1,2 m). Table 4 defines the characteristic values of the line action q_k.

Category	A	B	C,D	Public events in C or D
q_k (kN/m)	0,5	1,0	1,5	3,0

Table 4 Horizontal imposed actions on partitions and barriers.

Snow loads

The snow loads are based on measurements of snow depths on the ground and snow density. Depending on the surrounding terrain and the local weather, the specific density of snow varies from 0,1 (fresh snow) to 0,4 (old or wet snow). From a statistical analysis of these records, the characteristic snow load on the ground (s_k) is defined for a return period of 50 years. As they depend on the geographical location and the altitude of the site, the characteristic values s_k are given in the national loading codes. In addition, the designer should also consider local effects that may modify the specified value s_k. For example, significant increase in the snow load on a member can result from snow turning into ice or rain falling on the snow. For structural calculations, the designer has to consider the load arrangements on the roofs such as:

- balanced distributions resulting from uniform snow falls,
- and unbalanced loads due to drifting under windy conditions or snow sliding.

From the analysis of snow falls on the ground, the snow loading is generally treated as a variable action of short-term duration (less than one week). Referring to the horizontal projection of the area, the characteristic value of the roof snow load is calculated as:

$$S_k = \mu_i \; s_k \tag{3}$$

The shape coefficient μ_i takes into account the roof exposure and geometry. Three coefficients μ_i are defined in EC1, depending on the roof slope α (Figure 2).

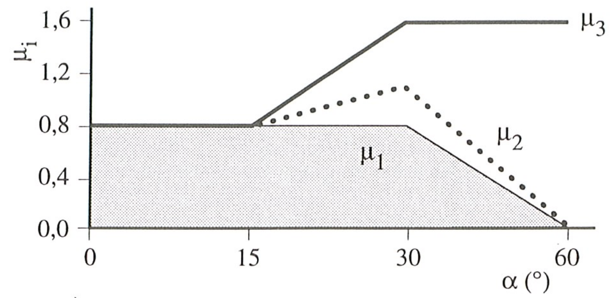

Figure 2 Snow shape coefficients on roofs.

Assuming that the snow could slide off the roof, Figure 3 describes the design patterns S_1 and S_2 for the snow load on pitched (a, b and c) and curved roofs (d).

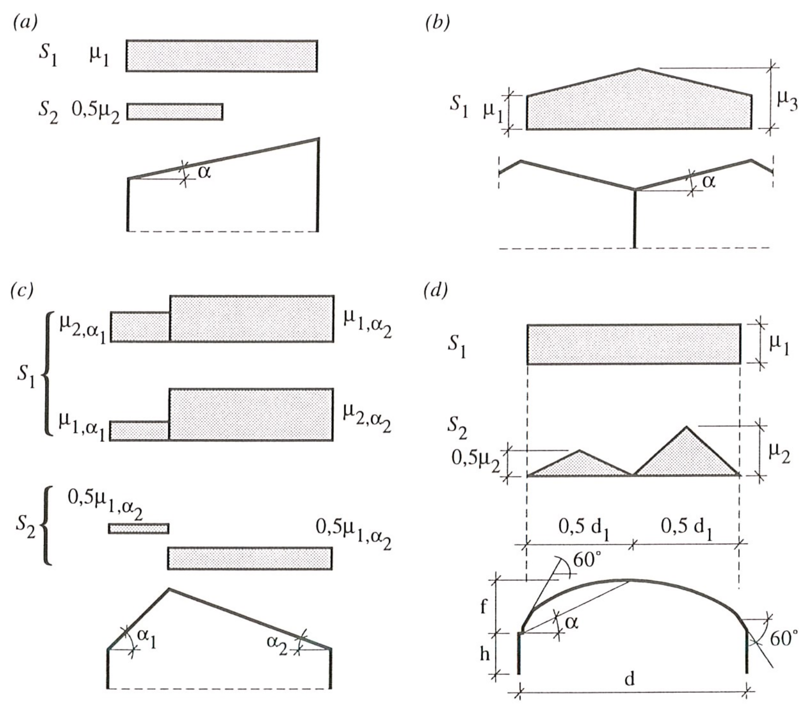

Figure 3 Snow load arrangements on roofs.

In addition, the designer should pay attention to the possible increase in the snow load due to the shape and the location of the structure. For example, the design has to take into account the additional loads due to filling of roof valleys or formation of drifts against walls.

Wind actions

Wind actions fluctuate with time and these variable actions are related to the short-term load duration class. The structural response could be considered as the combination of a quasi-static component and a resonant component. This component could be significant for flexible (e.g. buildings with a height to width ratio greater than 3) and elongated vertical structures. In these cases, detailed wind analysis is required. However, the resonant component is of minor importance for most structures, and wind actions are defined using the simplified method described in this section. The wind actions are represented by static pressures on the surfaces of the structure or by global pressure and friction wind forces (E.C.C.S., 1987).

Wind variations

The design calculations are based on the reference wind velocity v_{ref} and pressure q_{ref}. Referring to a mean return period of 50 years, v_{ref} is defined as the average wind velocity over a ten minutes period at 10 *m* above terrain category II (see Table 5). The geographical location is taken into account using the basic wind velocity $v_{ref,0}$ at sea level given in national wind maps. From this value, v_{ref} and q_{ref} are defined as:

$$v_{ref} \;=\; C_{DIR}\; C_{TEM}\; C_{ALT}\; v_{ref,0} \qquad m/s \tag{4}$$

$$q_{ref} \;=\; 0{,}5\; \rho\; v_{ref}^2 \qquad N/m^2 \tag{5}$$

where C_{DIR} is a factor related to the wind direction (e.g. C_{DIR}=1),
C_{TEM} is a reduction factor for temporary structure,
C_{ALT} is the altitude factor specified in the wind maps,
ρ is the air density taken as 1,25 kg/m^3.

As the wind pressure varies with height above the ground, the designer has to consider the reference height z_w of the external building surfaces. Depending on the shape of the building and the crosswind dimension b_w, Figure 4 specifies the reference height for walls and roofs.

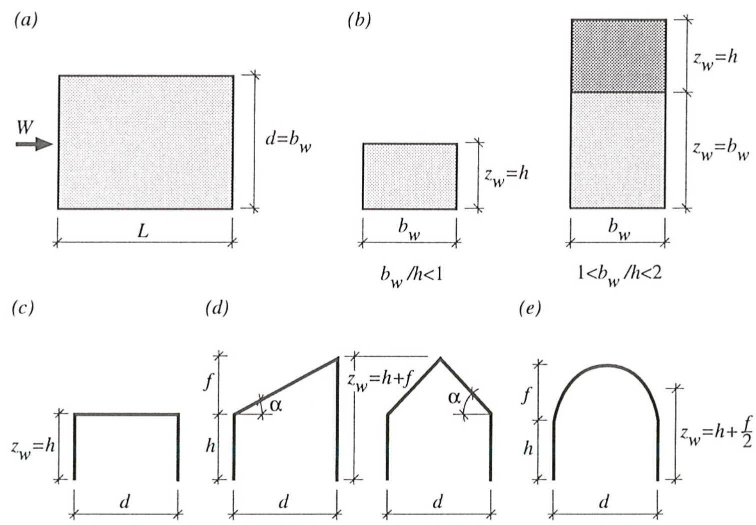

Figure 4 *Definition of the reference height z_w for buildings: plan and crosswind dimension (a) walls (b) flat (c) pitched (d) and vaulted (e) roofs.*

The effect of height and ground roughness on the wind velocity is first considered with the roughness coefficient $c_r(z_w)$. With the classification and the values given in Table 5, this coefficient is defined by the logarithmic wind profile as:

$$c_r\,(z_w) \;=\; K_r\, \ln\,[\,\max(\,z_w\,,\,z_{min})\,/\,z_0] \tag{6}$$

where z_0 is the roughness length,
z_{min} is the height of the ground layer where the wind velocity is

constant,
K_r is the terrain factor.

Category	Terrain	K_r	z_{min} (m)	z_0 (m)
I	- Rough open sea, lakes (fetch upwind >5km) - Smooth flat country without obstacles	0,17	2	0,01
II	- Farmland with hedges, occasional houses or farm structures	0,19	4	0,05
III	- Suburban and industrial areas and forests	0,22	8	0,30
IV	- Urban areas covered with buildings of average height greater than 15m	0,24	16	1,0

Table 5 *Terrain roughness classification and terrain parameters.*

The resultant adjustment for the environmental effects on the wind is then covered by the exposure coefficient C_e. Considering the reference height z_w and the site conditions of the designed structure, the exposure coefficient is determined from:

$$C_e(z_w) = c_r^2(z_w)\ c_t^2 + 7\ K_r\ c_r(z_w)\ c_t \qquad (7)$$

where c_t is the topography coefficient taking into account local terrain variations such as hills or escarpments (e.g. c_t=1).

Pressure coefficients

The pressure coefficients define the wind pressures acting normally to the surfaces of the buildings. The external (C_{pe}) and internal (C_{pi}) pressure coefficients are defined as positive if the wind pressure acts towards the surface. A negative value denotes suction on the walls or uplift of the roofs. The effect of the wind direction θ is taken into account by two separate sets of coefficients considering the windward side as either the gable (θ = 90°) or the long-side (θ = 0 or 180°). The external pressure coefficient also varies with the shape of the structure. In addition, wind tunnel tests have shown that larger pressures occur at the edges and the corners of structures (Lusch, 1964). These observations result in pressure distributions as shown in Figures 5 and 6.

According to EC1, the specified coefficients vary on the structure as specified in the following sections for common shapes of rectangular buildings. These values correspond to the upper value for all wind directions ± 45° from the normal to the side under consideration. Figure 5 gives the coefficient C_{pe} for wall areas greater than 10 m^2 and building dimensions such as: d/h(θ=0°) or L/h(θ=90°)≤1. These pressure distributions relate to the aspect windward dimension e_w, where e_w= min (b_w,2 h). For smaller wall areas, higher values of the pressure coefficient have to be used. On the windward side, the coefficient C_{pe} is reduced to +0,6 for an elongated building area (L/h or d/h) ≥ 4.

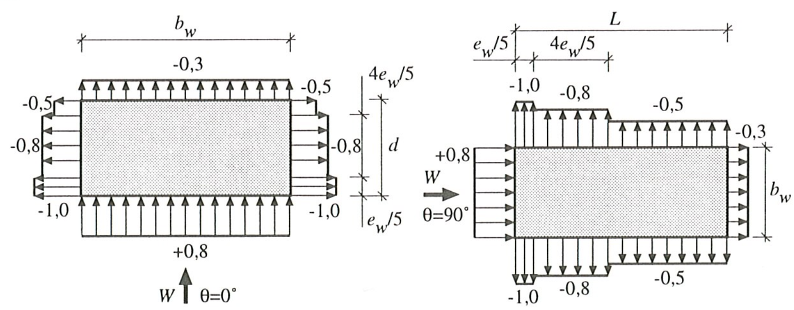

Figure 5 *Pressure coefficients for vertical walls.*

In addition to the wall pressures, the wind actions applied to roofs require special attention as wind uplift may affect the design of the joints. In the case of flat roofs, Figure 6 defines the pressure coefficient for the wind directions $\theta = 0$ or 90°.

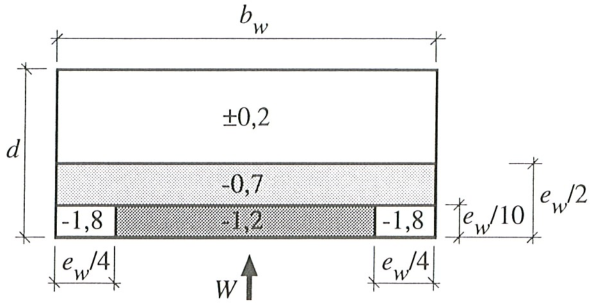

Figure 6 Pressure coefficients for flat roofs.

For windward sloping roof surfaces, the wind actions are pressures or suctions depending on the pitch angle α. Both pressure and suction have to be considered when α varies between 15 and 30° (see Tables 6 and 7).

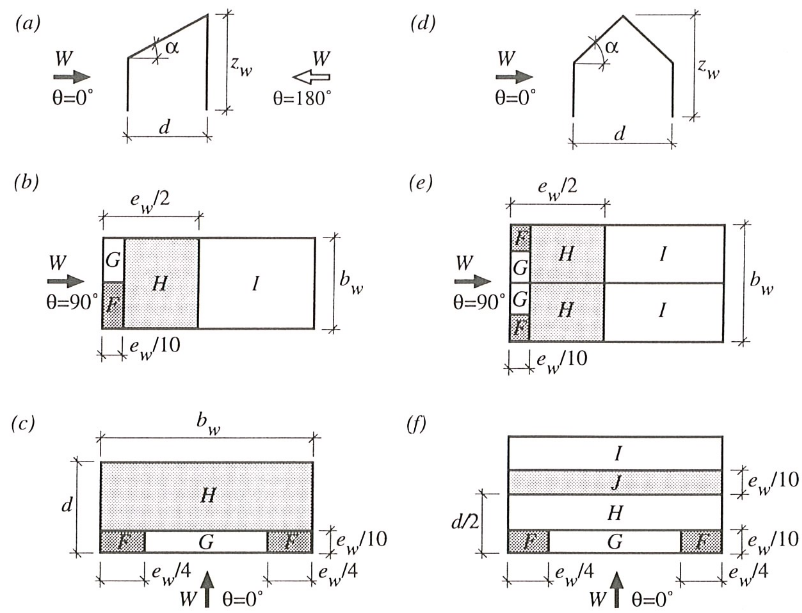

Figure 7 Wind areas on monopitch (a,b,c) and duopitch (d,e,f) roofs for the different wind directions θ.

α	θ = 0°			θ = 180°			θ = 90°			
(°)	F	G	H	F	G	H	F	G	H	I
5	-1,7	-1,2	-0,6	-2,3	-1,3	-0,8	-1,6	-1,8	-0,6	-0,5
15	-0,9 +0,2	-0,8 +0,2	-0,3 +0,2	-2,5	-1,3	-0,9	-1,3	-1,9	-0,8	-0,7
30	-0,5 +0,7	-0,5 +0,7	-0,2 +0,4	-1,1	-0,8	-0,8	-1,2	-1,5	-1,0	-0,8
45	+0,7	+0,7	+0,6	-0,6	-0,5	-0,7	-1,2	-1,4	-1,0	-0,9

Table 6 External pressure coefficients C_{pe} for monopitch roofs

α			$\theta = 0°$					$\theta = 90°$	
(°)	F	G	H	I	J	F	G	H	I
5	-1,7	-1,2	-0,6	-0,3	-0,3	-1,6	-1,3	-0,7	-0,5
15	+0,2	+0,2	+0,2	-0,4	-1,0	-1,3	-1,3	-0,6	-0,5
30	+0,7	+0,7	+0,4	-0,4	-0,5	-1,1	-1,4	-0,8	-0,5
45	+0,7	+0,7	+0,6	-0,2	-0,3	-1,1	-1,4	-0,9	-0,5

Table 7 External pressure coefficients C_{pe} for duopitch roofs.

The presence of openings and the porosity of the external surfaces greatly affects the internal wind pressure in buildings. Considering the influence of the wind direction, the internal pressure coefficient C_{pi} varies with the opening ratio of the windward side. For normal closed buildings with opening windows or doors, the value of C_{pi} is taken either as 0,8 or -0,5 for all the internal surfaces, whichever results in the more severe load case.

Design wind actions

For building design, the wind action effects are generally estimated using the wind pressure distribution on the surfaces. It results from the combination of the external (w_e) and internal (w_i) pressures given by:

$$w_e = q_{ref} \; C_e(z_w) \; C_{pe} \qquad\qquad w_i = q_{ref} \; C_e(z_i) \; C_{pi} \tag{8}$$

where z_i is equal to the reference height of the walls for closed buildings or the mean height of the openings.

According to EC1, structures are designed for all wind directions taking into account the characteristic value of the wind actions (w_k). They correspond to the net pressure distribution defined as:

$$w_k = w_e - w_i \tag{9}$$

EC1: Part 2-4

For some structures, the wind forces resulting from pressure and friction effects may need to be considered. The pressure force (F_w) is the summation of pressures on the projected structural area normal to the wind. For structures which are sensitive to torsion, the resulting force F_w is assumed to act with an eccentricity $e = b_w/10$. The friction force (F_{fr}) has to be considered in the case of large surfaces swept by the wind (e.g. free standing roofs).

Combination of actions

After the estimation of the actions, the design requires the structural analysis of the action effects. This stage involves the selection of realistic load arrangements for which the structure or the structural components are to be designed. Then, the design values result from the following combinations of the actions. Firstly, at the ultimate limit states, the combination for persistent or transient situations is:

$$\sum_i \gamma_{G,\,i} \, G_{k,\,i} \; + \; 1{,}5 \; Q_{k,\,1} \; + \; \sum_{j>1} 1{,}5 \; \psi_{0,\,j} \; Q_{k,\,j} \tag{10}$$

where $\gamma_{G,i}$ is the partial factor for the permanent loads (see STEP lecture A2).
$Q_{k,1}$ represents the dominant variable action.

Secondly, the combination at the serviceability limit states depends on the action effect being checked considering both:

the characteristic combination: $\sum_{i} G_{k,i} + Q_{k,1} + \sum_{j>1} \psi_{0,j} Q_{k,j}$ (11)

and the quasi-permanent combination: $\sum_{i} G_{k,i} + \sum_{j\geq 1} \psi_{2,j} Q_{k,j}$ (12)

According to EC1, the Ψ factors for buildings are given in Table 8 where Ψ_1-values refer to accidental load combinations.

Actions			Ψ_0	Ψ_1	Ψ_2
Imposed loads	Category	A,B	0,7	0,5	0,3
		C,D	0,7	0,7	0,6
		E	1,0	0,9	0,8
		H	0	0	0
Snow loads			0,6	0,2	0
Wind actions			0,6	0,5	0

Table 8 Ψ factors for variable actions on buildings.

For timber structures, the designer must pay special attention to finding out the critical load cases as they depend on the material load-duration factors. At the ultimate limit states, the combination (10) is related to the use of the k_{mod} factor. For each combination including variable actions, the appropriate k_{mod} factor corresponds to the dominant action $Q_{k,1}$. At serviceability limit states, the combination (11) applies to the calculation of the instantaneous action effects in service. In addition, the combination (12) refers to the calculation of the long-term action effects using the relevant factors k_{def} for the materials and the service class of the structure (Racher and Rouger, 1994). According to EC1 calculations, the k_{def} factors related to the permanent load-duration class have to be used (see STEP lectures A17 and C18).

Considering the different limit states, the combination of the actions is calculated for each critical load case. The designer's judgement could lead him/her to consider a few worse-case load arrangements. These are commonly:

- *(dead + imposed)* for floor members or *(dead + snow)* for roof members,
- *(dead + wind + snow $S_1/2$ or S_2)* (see Figure 3) for the structure.

Uniformly distributed loads usually control the design of members, while unbalanced load cases can induce more critical effects for connections or in some framing systems (i.e. lattice structures).

Example

In the example, the design values of the combined actions are calculated for the frame shown in Figure 8. The building is 48 metres long and the frame spacing s_F is $4{,}8\ m$. Referring to national snow and wind maps, the location of the projected building provides the following characteristics for:

- snow loads on the ground: $s_k = 1{,}5\ kN/m^2$
- reference wind velocity: $v_{ref} = v_{ref,0} = 24\ m/s$.
- terrain classification: ground category III (industrial area).

According to the national regulations, the snow and wind actions are classified in the short-term duration class. As the structure is located at an altitude greater than 500 m a combination of wind and snow shall be considered. The Ψ factors for snow are:

$\Psi_{0,S}=0,67$, $\Psi_{1,S}=0,3$ and $\Psi_{2,S}=0,1$.

The preliminary choice of the designer results in the values of the characteristic permanent loads as:

- self-weight of the frame: $g_{k,1}= 0,70\ kN/m$
- roofing elements: $g_{k,2}= 0,55\ kN/m^2$

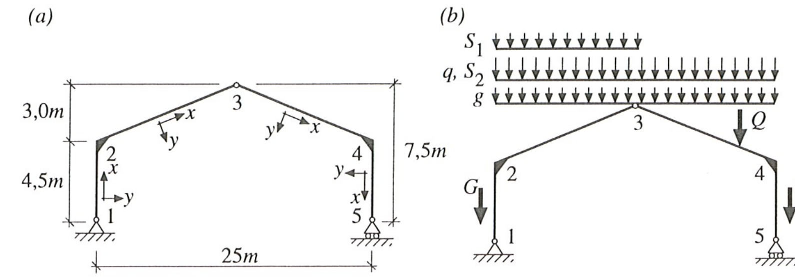

Figure 8 *Geometry of the frame (a) and applied gravity loads (b): permanent (g,G), variable (q, Q) and snow (S_1, S_2) loads.*

Permanent loads

The uniformly distributed load on the horizontal projection of the rafters, due to permanent actions is:

$$g_k = (g_{k,1} + s_F\ g_{k,2})\ \frac{1}{\cos\alpha} = (\ 0{,}7\ +\ 4{,}8\ \cdot\ 0{,}55\ )\ \frac{1}{\cos\ 13{,}5^\circ} = 3{,}43\ kN/m$$

The self-weight of the vertical members results in the load:

$$G_k = h\ g_{k,1} = 4{,}5\ \cdot 0{,}7 = 3{,}15\ kN$$

Table 3

Imposed loads

The design requires only consideration of the imposed loads corresponding to the maintenance of the roof. As the slope of the roof α ($\alpha = 13{,}5°$) is less than 20°, the uniformly distributed and the concentrated imposed loads are:

$$q_k = 4{,}8\ \cdot 0{,}75\ /\ \cos(13{,}5) = 3{,}7\ kN/m \qquad Q_k = 1{,}5\ kN$$

These loads belong to the short-term duration class and they do not act simultaneously with other variable actions.

Figure 2

Snow loads

For a slope α less than 15°, the shape coefficients μ of the snow are defined as:

$$\mu = \mu_{1,\alpha} = \mu_{2,\alpha} = 0{,}8$$

The design considers two characteristic snow loads on the horizontal projection of the structure:

- the symmetrical snow load S_I: $S_{I,k} = (\mu\ s_k)\ s_F = 0{,}8\ \cdot 1{,}5\ \cdot 4{,}8 = 5{,}76\ kN/m$
- the snow on half the frame S_{II}: $S_{II,k} = (\ 0{,}5\ \mu\ s_k\ )\ s_F = 2{,}88\ kN/m$

Wind actions

The value of the reference wind pressure is:

$$q_{ref} = 0{,}5\ \rho\ v_{ref}^2 = 0{,}5\ \cdot\ 1{,}25\ \cdot\ 24^2 = 0{,}36\ kN/m^2$$

Considering the frame geometry, the reference heights for the walls (z_w= 4,5 m) and the roof (z_w= 7,5 m) are less than the groundlayer height z_{min}= 8 m. Therefore, the roughness and exposure coefficients are constant for all the external and internal surfaces.

Equation (6)

$$c_r(z_w) = K_r \ln\left[\frac{z_{min}}{z_0}\right] = 0{,}22 \cdot \ln\left[\frac{8}{0{,}3}\right] = 0{,}722$$

Equation (7)

$$C_e = C_e(z_w) = C_e(z_i) = c_r^2(z_w)\, c_t^2 + 7\, K_r\, c_r(z_w)\, c_t$$
$$= 0{,}722 \cdot 1\,(0{,}722 \cdot 1 + 7 \cdot 0{,}22) = 1{,}63$$

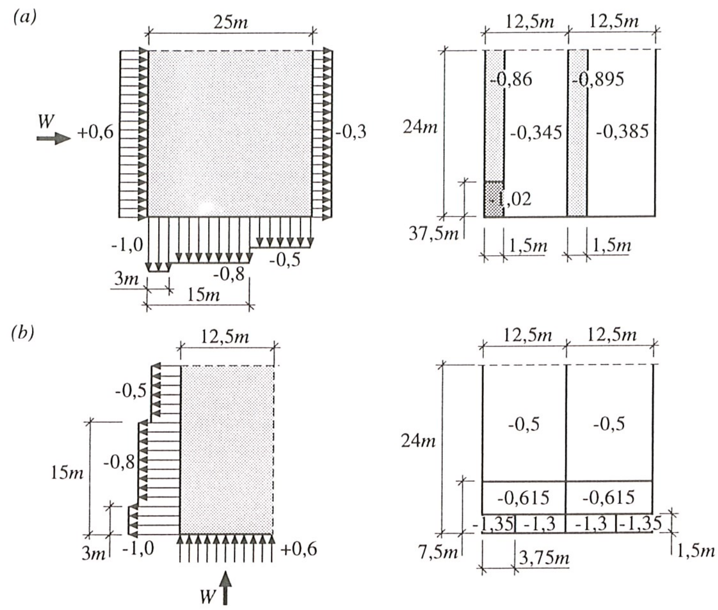

Figure 9 External pressure coefficients for θ =0° (a) and θ =90° (b).

The distribution of the external pressure coefficients (Figure 9) is defined with the aspect dimension e_w that takes the value of 15 m for all wind directions. For the roof, these coefficients are calculated by interpolation between the values of same sign given for 5° and 15°.

Equations (8) and (9)

The characteristic wind actions are obtained as:

$$w_k = q_{ref}\, C_e\, (C_{pe} - C_{pi})\, s_F = 2{,}824\,(C_{pe} - C_{pi}) \qquad kN/m$$

The wind effects on the frame result from a constant internal pressure (C_{pi}=+0,8 or -0,5) combined with the external pressures for each wind direction. The design of the frame considers three distributions resulting from the wind acting on the gable ($w_{1,k}$) or on the long side ($w_{2,k}$ and $w_{3,k}$). Figure 10 shows the wind actions for the frames in the middle of the building.

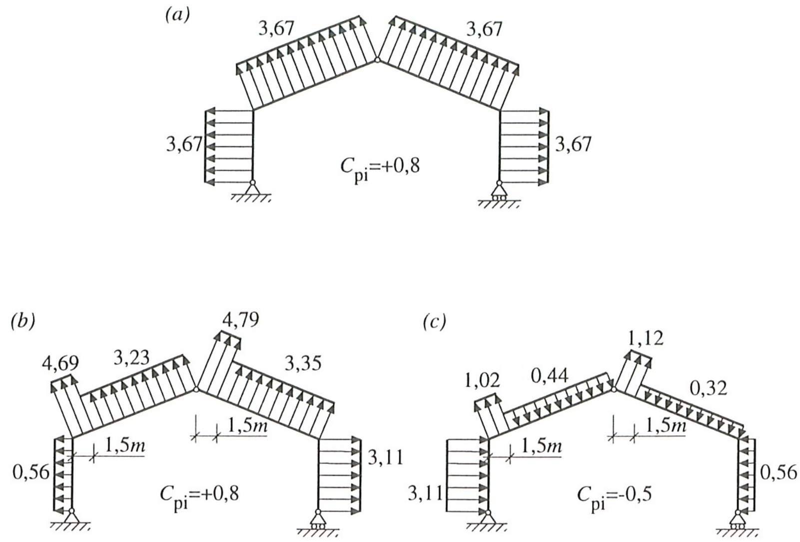

Figure 10 *Distributions of the wind actions on the frame (kN/m): (a) $w_{I,k}$, (b) $w_{2,k}$ and (c) $w_{3,k}$.*

Combination of actions: ultimate limit states

Depending on the effect being checked, the design of the frame refers to the load combinations with one variable action:

$C1$: $1{,}35\ (g_k + G_k)$

$C2$: $1{,}35\ (g_k + G_k) + 1{,}5\ (q_k \text{ or } Q_k)$

$C3$: $1{,}35\ (g_k + G_k) + 1{,}5\ S_{I,k}$

$C4$: $(g_k + G_k) + 1{,}5\ w_{i,k}$

and the combinations of snow and wind actions:

$$C5:\ 1{,}35\ (g_k + G_k) + 1{,}5\ w_{i,k} + 1{,}5\ \psi_{0,S}\ \left[\ \frac{S_{I,k}}{2}\ \text{or}\ S_{II,k}\ \right]$$

$$C6:\ 1{,}35\ (g_k + G_k) + 1{,}5\ \left[\ \frac{S_{I,k}}{2}\ \text{or}\ S_{II,k}\ \right] + 1{,}5\ \psi_{0,W}\ w_{i,k}$$

where $\Psi_{0,S}$ and $\Psi_{0,W}$ are the combination factors associated with snow and wind.

With the prescribed k_{mod} factors, the combination C1 can be critical if the permanent loads represent more than 70% of the total loads. In this example, the first two combinations as well as the combinations of snow and wind do not cause critical effects. In practice, the design of the frame depends on the design of the moment-resisting joint (2 or 4) which is achieved using load case C3. This case also gives the critical combination for the members in combined bending and compression. The combination C4 defines the worst reversal forces due to wind uplift: bending and tension in the members, and tension in the hinges.

Preliminary design values of forces and moments are given in Table 9.

Section	1		2 (column)			3	
Combination	C3	C4	C3	C4	C4	C3	C4
Wind actions		$w_{2,k}$		$w_{1,k}$	$w_{2,k}$		$w_{2,k}$
N (kN)	171	-22,1	166	-26	-25,3	134	-41,1
V (kN)	138	-16,3	138	-25,8	-20,1	32,3	-9,9
M (kN.m)	0	0	-622	60,2	82	0	0

Table 9 *Design values of forces and moments at the ultimate limit states.*

Combination of actions: serviceability limit states
As snow is the main variable action, the instantaneous effects of the actions are calculated from the combinations:

$C7$: $(g_k + G_k) + S_{I,k}$
$C8$: $(g_k + G_k) + S_{II,k} + \psi_{0,w}\, w_{i,k} = (g_k + G_k) + S_{II,k} + 0{,}6\, w_{i,k}$
Depending on the shape and the span of the frame, the limitation for the horizontal deflection of the column is checked using either the combination C7 or C8. The combination C7 gives the maximum value of the vertical deflection in section3.

In addition, the calculation of the long-term effects such as creep deformations refers to the quasi-permanent combinations:
$C9$: $(g_k + G_k) + \psi_{2,s}\, S_{I,k} = (g_k + G_k) + 0{,}1\, S_{I,k}$
$C10$: $(g_k + G_k) + \psi_{2,s}\, S_{II,k} + \psi_{2,w}\, w_{i,k} = (g_k + G_k) + 0{,}1\, S_{II,k}$

To calculate the final deflections, it is therefore necessary to consider the combinations:

- (C7,C9) for the vertical displacements,
- (C7,C9) or (C8,C10) whichever causes the greater horizontal displacements.

References

C.E.B. (1980). Structural safety. Bulletins d'information N°127 and 128, Brussels, Belgium.

E.C.C.S. (1987). Recommendations for calculating the effects of wind on constructions. European Convention for Constructional Steelwork, Technical committee 12, Report N°52, Brussels, Belgium.

Hendrickson, E.M., Ellingwood, B. and Murphy, J. (1987). Limit state probabilities for wood structural members. ASCE, J. of Structural Engineering, USA, Vol. 113 N°1, p. 88-106.

Lusch (1964). Wind tunnel investigations on buildings with rectangular base and with flat and duo-pitched roofs. Report of Bauforschung N°41, Germany.

Racher, P. and Rouger, F. (1994). Serviceability limit states - A proposal for updating Eurocode 5 with respect to Eurocode 1. In: CIB W18A-27th Meeting, Sydney, Australia, July 1994.

Rackwitz, R. (1976). Practical probabilistic approach to design. C.E.B., Bulletin d'information N° 112, Brussels, Belgium.