Actions on structures

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Objective

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 article, 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 [*Table 1 is based upon FprEN 1990: 2022 Table A.1.2(NDP)*]

Table 1: Design working life classification

Working life (years)	Example
≤ 10	Temporary structures [a, b]
25	Agricultural, and similar structures Replaceable structural parts
50	Building structures not covered by another category
100	Monumental building structures

a For structures or parts of structures that can be dismantled in order to be re-used, see 4.5(3).

b For specific temporary structural members, shorter design service lives can apply, see the other Eurocodes.

c For design service life of geotechnical structures, see the relevant part of EN 1997.

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 article (see STEP lectures A2, B17 and C 17).

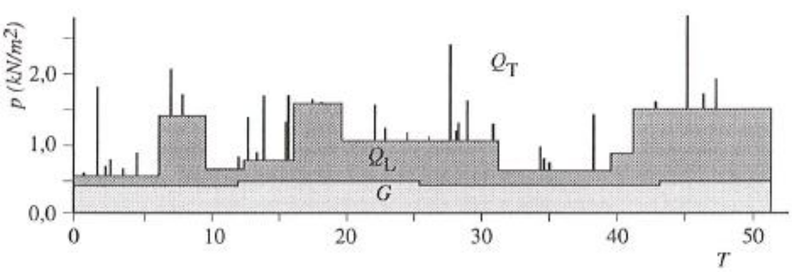


Figure 1: Time-variation of the total applied action on a floor.

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 timedependent 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 Article 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 $G_{k,\mathrm{inf}}$ and upper value $G_{k,sup}$. Assuming a Gaussian distribution for G, these values are given by:

$$G_{k,inf} = G_{mean}(1 - 1{,}64COV);\ G_{k,sup} = G_{mean}(1 + 1{,}64COV) \qquad (1)$$

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:

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

Where COV is the coefficient 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.

Table 2: Categories of Specific Use

Category	Specific Use
A	Areas for domestic and residential activities
B	Public areas (not susceptible to crowding)
C	Public areas where people can congregate (with the exception of areas defined under category A, B, and D)
D	Shopping areas
E	Areas for archive, storage and industrial use
F & G	Garages and vehicle traffic areas (excluding ordinary roads and bridges)
H	Roofs not accessible except for normal maintenance and repair
I	Roofs accessible with occupancy according to categories A to G
K	Roofs accessible for special services, such as classes HC for helicopter landing areas
S	Stairs and landings
T	Terraces and balconies

Table 2 show the various categories of use detailed in EN 1991, but some categories would not normally be considered for timber structures. In common timber buildings, the following classes are considered: -

- A - dwellings,
- B - offices,
- C & D - shops .
- H&I - roofs and
- E - production areas.
- S – Stairs
- T – Terraces and balconies

The resulting variable imposed actions for each sub-category have been tabulated in EN 1991-1-1 Table 6.1 from which the following table of values that we shall use for q_k and Q_k on structures designed and verified for Eurocode compliant timber structures are show in Table 3:-

Table 3: Imposed loading categories

Sub-category	Example	q_k [kN/m^2]	Q_k [kN]
A1	Rooms in residential buildings and houses, including corridors.	2,0	2,0
A2	Bedrooms, wards, dormitories, private bathrooms and toilets in hospitals, hotels, hostels and other institutional residential occupancies.	2,0	2,0
B1	Office areas for general use including corridors other than archive / storage areas (see Category E)	3,0	3,0
B2	Kitchens, communal bathrooms and toilets in hospitals, hotels, hostels and other institutional residential occupancies.	3,0	3,0
C1	Areas with tables, etc. e.g. areas in schools, cafés, restaurants, dining halls, reading rooms, receptions.	3,0	4,0
C2	Areas with fixed seats, e.g. areas in churches, theatres, cinemas, conference rooms, lecture halls, assembly halls, waiting rooms.	4,0	4,0
C3	Areas without obstacles for moving people, e.g. areas in museums, exhibition rooms, etc. and corridors to areas not belonging to categories A1, B1 and C5.	5,0	4,0
C4	Areas with possible physical activities, e.g. dance halls, gymnastic rooms, stages.	5,0	7,0
C5	Areas susceptible to large crowds, e.g. in buildings for public events including corridors like concert halls, sports halls including stands, and railway platforms.	7,5	4,5
D1	Areas in retail shops	4,0	4,0
D2	Areas in department stores	5,0	7,0
E1	Areas susceptible to accumulation of goods, including access areas	7,5	7,0
E2	Industrial use	See notes below	
H	Roofs not accessible except for normal maintenance and repair	0,4	1,0
I	Roofs accessible with occupancy according to categories A to G	See categories A to G	
S1	Stairs and landings to areas belonging to category A1 and B1.	See categories A1 and B1	
S2	Stairs and landings for tribunes without fixed seats that are defined as escape ways.	7,5	3,0
S3	Stairs and landings not belonging to category S1 or S2.	5,0	2,0

Sub-category	Example	q_k [kN/m^2]	Q_k [kN]
T1	Roof terraces, access balconies, balconies, loggias, etc.	3,0	2,0

Additional notes are provided to these categories in EN 1991.

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 several sub-categories (Table 3). Roofs are categorized as not accessible except for maintenance or repair (Category H) or as accessible with category I. For accessible roofs, the design is made with the occupancy corresponding to the floor classification.

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 EN 1991-1-1 these are [NDP] values so check that they are valid for the country the structure is to be executed within.

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 EN 1995, a medium-term duration is usually considered for the load on areas A to D & I. 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.

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.

Table 4: Horizontal imposed loads on barriers

Category	q_k *(kN/m)*
A, B, C1, and H	0,8
C2, C3, C4, and D	1,0
C5	3,0
E[a]	2,0
I, S, T	As for their adjacent areas A to E

[a] For areas of category E, the horizontal loads depends on the occupancy. Therefore the value of q_k is defined as a minimum value and should be checked for the specific occupancy and actual storage conditions.

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 the loads or actions depend on the geographical location and the altitude of the site, the characteristic values are given in EN 1991-1-3 as [NDP] values and shall be checked for the country in which the structure is to be executed. 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). However in some national regions the snow will last longer refer to the appropriate National Annex that will give guidance on the expected duration and the time period the action may be allocated. Referring to the horizontal projection of the area, the characteristic value of the roof snow load is calculated as:

$$S = \mu_i C_t s_k \qquad (3)$$

μ_i is the snow load shape coefficient

C_t is the thermal coefficient

EN 1991-1-3 gives formulae for the calculation of C_e, the exposure coefficient, together with C_t and μ_i.

The shape coefficient takes into account the roof exposure and geometry. Several coefficients are defined in EN 1991-1-3, depending on the roof slope α (Figure 2).

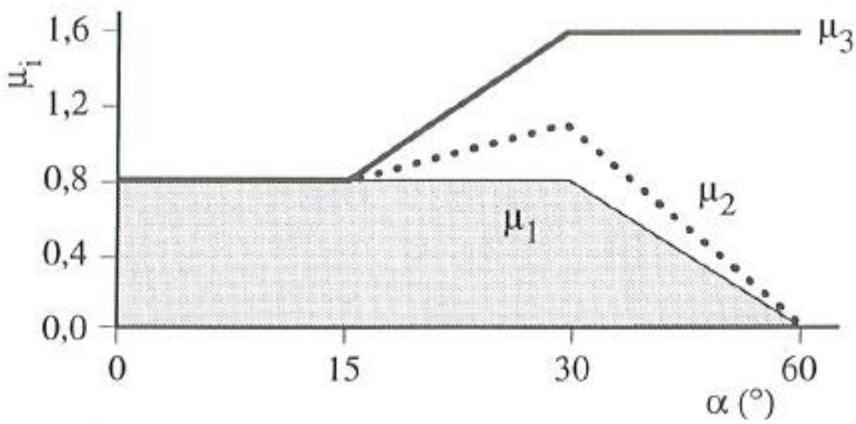


Figure 2: Snow shape coefficients on roofs

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

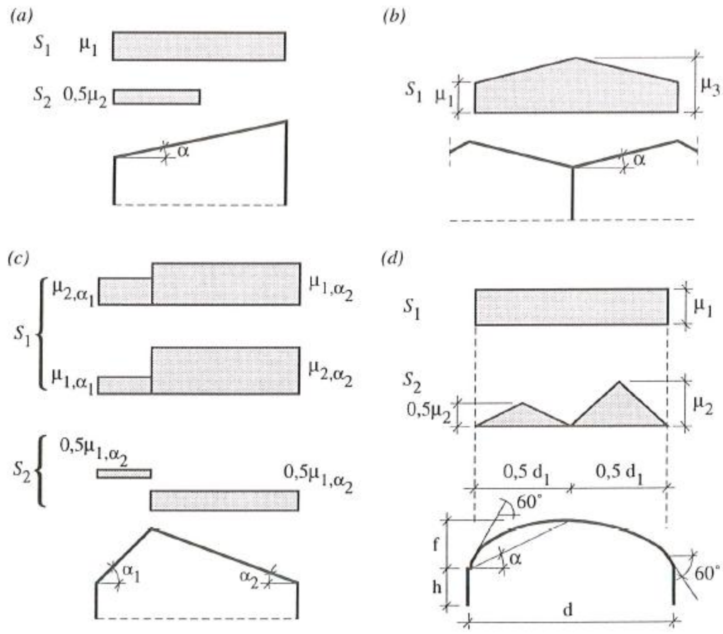


Figure 3: Snow load arrangements on roofs

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 (ECCS 1987-2012).

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:

$$The\ basic\ wind\ velocity;\ v_b = c_{prob} c_{dir} c_{season} c_{alt} v_{b0} \qquad (4)$$

where

- v_{b0} is the fundamental basic or map wind velocity
- c_{alt} is the altitude factor;
- c_{dir} is the direction factor;
- c_{season} is the season factor;
- c_{prob} is the probability factor

The four factors are 1,0 unless the National Annex says differently

The reference basic mean velocity pressure is found from;

$$q_b = \frac{1}{2}\rho {v_b}^2 \quad (5)$$

Where; 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_e of the external building surfaces. Depending on the shape of the building and the crosswind dimension *b*, the in-wind depth is taken as d, and the height of the structure is *h*.

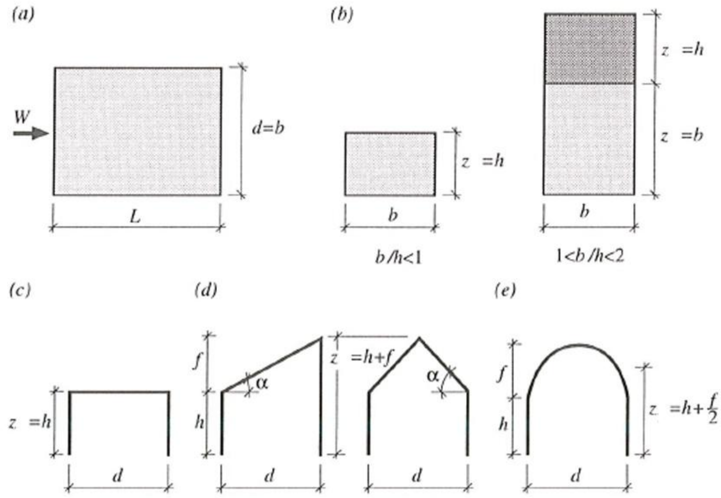


Figure 4: Definition of the reference height z for buildings;

The 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 $c_r(z)$. With the classification and the values given in Table 5, this coefficient is defined by the logarithmic wind profile as:

$$c_r(z) = k_r ln[max(z_0, z_{min})/z_0] \quad (6)$$

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.

Table 5: Terrain categories and parameters [based on EN 1991-1-4]

	Terrain category	z_0 [m]	z_{min} [m]	k_r
0	Sea or coastal area exposed to the open sea	*0,003*	*1*	*0,16*
I	Lakes or flat and horizontal area with negligible vegetation and without obstacles	*0,01*	*1*	*0,17*
II	Area with low vegetation such as grass and isolated obstacles (trees, buildings) with separations of at least 20 obstacle heights	*0,05*	*2*	*0,19*
III	Area with regular cover of vegetation or buildings or with isolated obstacles with separations of maximum 20 obstacle heights	*0,3*	*5*	*0,22*

Terrain category	z_0 [m]	z_{min} [m]	k_r
(such as villages, suburban terrain, permanent forest)			
IV Area in which at least 15 % of the surface is covered with buildings and civil engineering works or forest, and their average height exceeds 15 m	*1,0*	*10*	*0,23*

The terrain types are illustrated with sketches in EN 1991-1-4

Surface and internal pressure coefficients are given in detailed tables of the various Annexes to EN 1991-1-4

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 = 90^0) or the long-side (θ = 0 or 180 ^{0}). 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 ^{0} 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°). These pressure distributions relate to the aspect windward dimension e, where e = min (b, $2h$). 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 (Uh or d/h) 2 4.

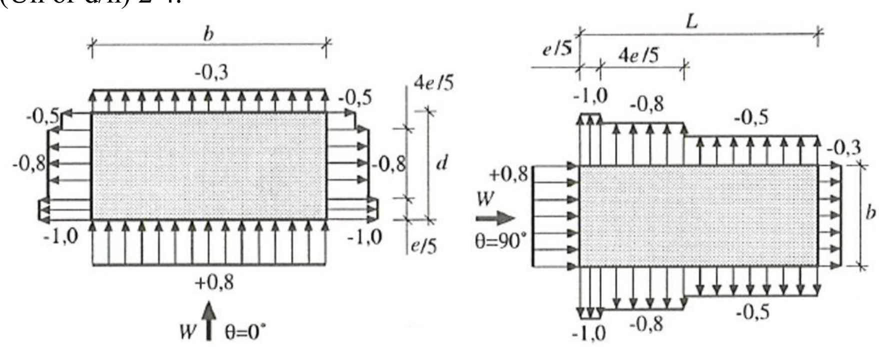


Figure 5: Pressure coefficients for vertical walls

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 θ = 0 or 90 ^{0} .

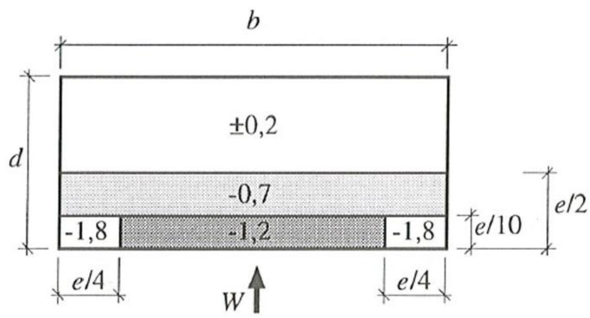


Figure 6: Pressure coefficients for flat roofs.

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 ^{0} (see Tables 6 and 7).

In some tables two pressure coefficients are given as c_{pe1} and c_{pe10}; c_{pe1} is normally used for the design of cladding and c_{pe10} for larger areas such as building facades and whole building and roof surfaces.

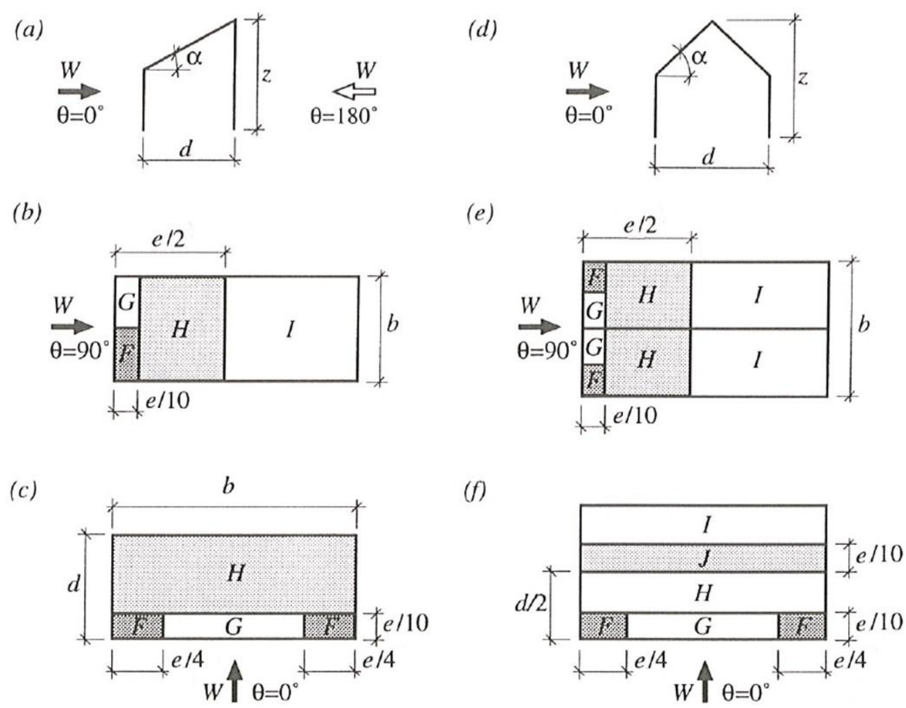


Figure 7: Wind areas on monopitch (a, b, c) and duopitch (d, e, f) roofs for different wind directions θ.

External pressure coefficients for monopitch roofs for θ = 0° and θ = 180°

Table 6: Based on FprEN 1991-1-4

Wind direction	Pitch angle α	F		G		H	
		c_{pe10}	c_{pe1}	c_{pe10}	c_{pe1}	c_{pe10}	c_{pe1}
θ = 180°	−45°	-0,6	-1,3	−0,5	-1,0	-0,7	-1,0
	−30°	-1,1	-2,3	-0,8	-1,5	-0,8	-1,0
	−15°	-2,5	-2,8	-1,3	-2,0	-0,9	-1,2
	−5°	-2,3	-2,5	-1,3	-2,0	-0,8	-1,2

Wind direction	Pitch angle α	F		G		H	
		c_{pe10}	c_{pe1}	c_{pe10}	c_{pe1}	c_{pe10}	c_{pe1}
θ = 0°	5°	-1,7	-2,5	-1,2	-2,0	-0,6	-1,2
		0,0		0,0		0,0	
	15°	-0,9	-2,0	-0,8	-1,5	-0,3	-1,0
		+0,2		+0,2		+0,2	
	30°	-0,5	-1,5	-0,5	-1,5	-0,2	-1,0
		+0,7		+0,7		+0,4	
	45°	0,0		0,0		0,0	
		+0,7		+0,7		+0,6	

Table 7: Based on Table in FprEN 1991-1-4

Pitch angle α	F_{up}		F_{low}		G		H		I	
	c_{pe10}	c_{pe1}	c_{pe10}	c_{pe1}	c_{pe10}	c_{pe1}	c_{pe10}	c_{pe1}	c_{pe10}	c_{pe1}
5°	2,1	2,6	2,1	2,4	1,8	2,0	0,6	1,2	0,2	1,2
15°	2,4	2,9	1,6	2,4	1,9	2,5	0,8	1,2	0,7	1,2
30°	2,1	2,9	1,3	2,0	1,5	2,0	1,0	1,3	0,8	1,2
45°	1,5	2,4	1,3	2,0	1,4	2,0	1,0	1,3	0,9	1,2

Table 8: Based on Table in FprEN 1991-1-4

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 (F_e) and internal (F_i) pressures given by:

$$w_e = q_{ref} c_{(z)} c_{pe} \qquad w_i = q_{ref} c_{(zi)} c_{pi} \tag{7}$$

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:

$$w_k = w_e - w_i \tag{8}$$

EC1: 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/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:

$$\sum_i \gamma_{G,i} G_{k,i} + 1{,}5 Q_{k,1} + \sum_{j>1} \psi_{0,j} Q_{k,j} \qquad (9)$$

where $\gamma_{G,i}$ is the partial factor for the permanent loads (see STEP Article A2). $Q_{k,l}$ 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: $\sum_i G_{k,i} + Q_{k,1} + \sum_{j>1} \psi_{0,j} Q_k$ (10)

and the quasi-permanent combination: $\sum_i G_{k,i} + \sum_{j\geq 1} \psi_{2,j} Q_{k,j}$ (11)

According

Table 9: ψ factors based on Table A.17 (NDP) of EN 1990

Action	ψ_0	ψ_1	ψ_2
Imposed loads in buildings (see EN 1991-1-1):			
Category A: domestic, residential areas	0,7	0,5	0,3
Category B: office areas	0,7	0,5	0,3
Category C: congregation areas	0,7	0,7	0,6
Category D: shopping areas	0,7	0,7	0,6
Category E: storage areas	1,0	0,9	0,8
Category	0,7	0	0
Construction loads (see EN 1991-1-6)	1,0	-	-
Snow loads on buildings (see EN 1991-1-3):			
• Finland, Iceland, Norway, Sweden;	0,7	0,5	0,2
• remainder of CEN Member States, for sites located at altitude H > 1 000 m asl;	0,7	0,5	0,2
• remainder of CEN Member States, for sites located at altitude H ≤ 1 000 m asl.	0,5	0,2	0
Wind actions on buildings (see EN 1991-1-4)	0,6	0,2	0

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 CConsidering 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 Sl/2 or S2) 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: $v_{ref} = 24$ 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:

$$\psi_0 = 0,5 \qquad \psi_1 = 0,2 \qquad \psi_2 = 0$$

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

- self-weight of the frame: *0, 70 kN/m*
- roofing elements: 0,55 kN/m^2

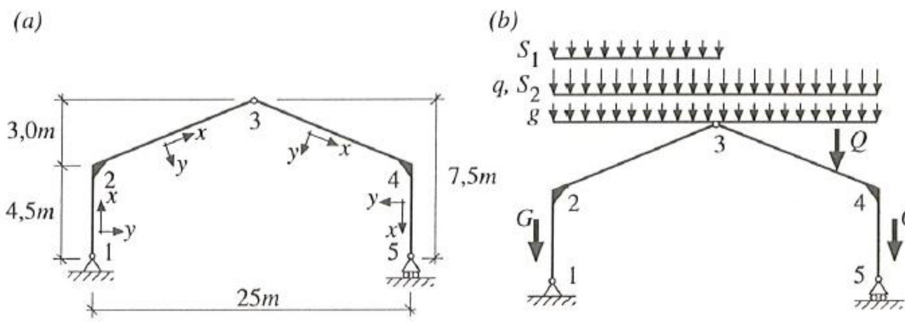


Figure 8: Geometry of the frame (a); applied gravity loads (b)

Permanent loads

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

$g_k = (g_{k,1} + s_F g_{k,2})$ / cos α = (0,7 + 4,8 · 0,55)/cos 13,5° = 3,43 *kN/m*

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

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

Imposed loads

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

q_k = 4,8 · 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.

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_1: $s_{1,k} = (\mu\ s_k)\ s_F$ = 0,8 · 1,5 · 4,8 = 5,76 *kN/m*
- the snow on half the frame S_2: $s_{1,k}$ = (055 · $\mu\ s_k$) 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,= 4,5 m) and the roof (z,= 7,5 m)are less than the ground layer height z = 8 m. Therefore, the roughness and exposure coefficients are constant for all the external and internal surfaces.

$c(z) = k_r\ ln(z_{min}/z_0) = 0{,}22 \cdot ln(8/0{,}3) = 0{,}722$

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

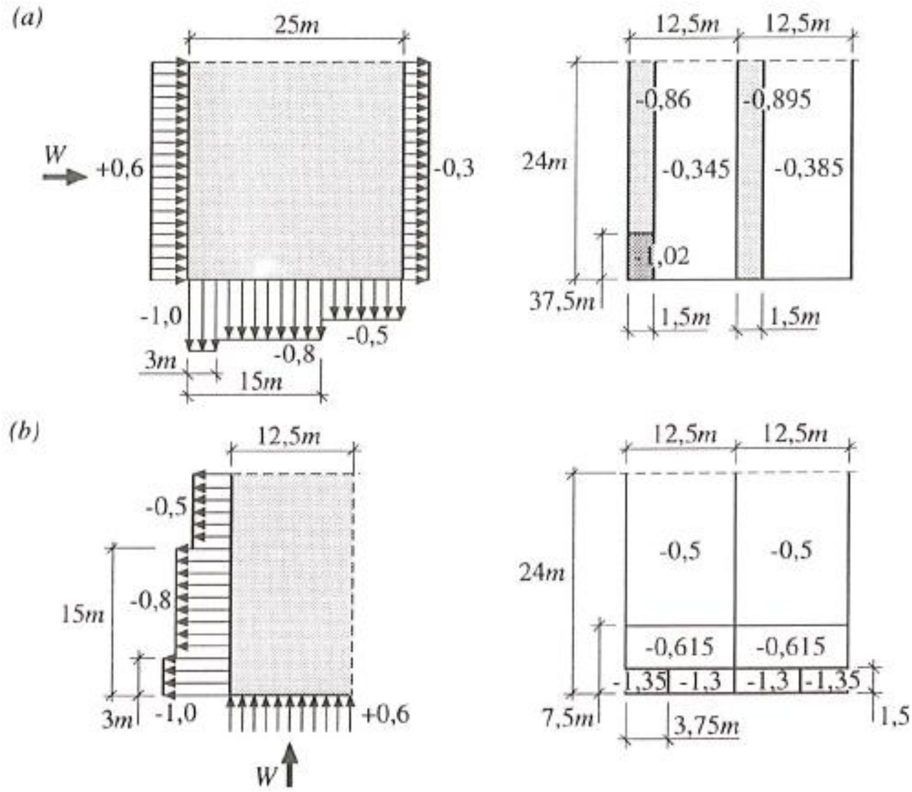


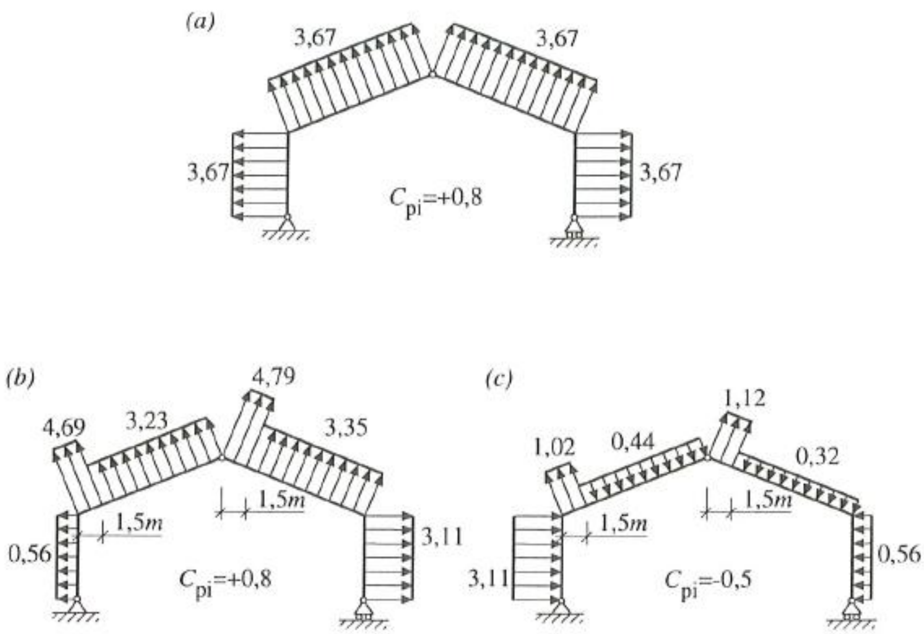
Figure 11: : External pressure coefficients for θ = 0° (a) and θ =90° (b)

The distribution of the external pressure coefficients (Figure 9) is defined with the aspect dimension e, 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°.

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 or on the long side. Figure 10 shows the wind actions for the frames in the middle of the building.



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:

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

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

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.

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