Loadings

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Clause 2.1 of EN 1990

> *The construction works must be designed and built in such a way that the loadings that are liable to act on it during its construction and use will not lead to any of the following:*
>
> - *collapse of the whole or part of the works;*
> - *major deformations to an inadmissible degree;*
> - *damage to other parts of the works or to fittings or installed equipment as a result of major deformation of the load-bearing construction;*
> - *damage by an event to an extent disproportionate to the original cause.*

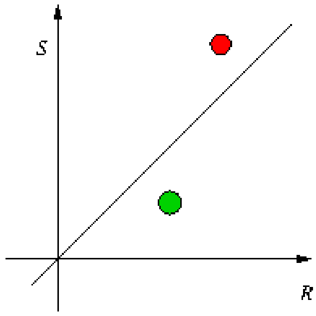
Actions, Effects, Resistances

We want to

- Define *actions* F (both single actions and their combinations).
- Compute the *effects* of the actions S — different aspects of the structural response as determined using a model of the structure.
- Compute the *resistances* R.

so that, in the end, we can assert that $R \geq S$ for every *limit state*.

The line represents $R = S$, the limit state.



$R > S$ means safety, on the contrary if $R < S$ we have an unacceptable situation.

Admissible Stress Design

- ASD was of fundamental importance, being the method used in the first building codes, starting at the beginnining of the XX century.
- ASD is based on checking the stresses in every point of the structure against an *admissible stress*, significantly lower than the rupture (or yield) stress of the material.
- The admissible stress is the rupture/yield stress factored by a *safety factor*, that depends on the material characteristics (e.g., 1.5 for steel, >2 for concrete) and, sometimes, on the type of loading.
- Based on this requirement we can use (we must use) a linear model of the material behaviour. ASD gives just a summary idea of the structural safety because the non linear behaviour is neglected.
- ASD assumes a single safety factor, with no info on actual risks.
- In many cases the failure mechanism is not directly associable with the exceeding of a given stress level.

Uncertainty

A building construction depends on:

- the decision of building it, in a site;
- its design and its construction;
- its testing;

and, later, on

- commissioning;
- service (or use);
- maintenance;
- any extraordinary interventions (to rebuild/modify the functionality);
- eventual demolition.

In every step we have different sources of uncertainties and errors.
Besides, every quantity, pysical or mechanical, can be determined only with a certain amount of approximation.
All in all, we can conclude that, in every moment, the current state of a building is similar, within a certain confidence, to the state of our model of the building.

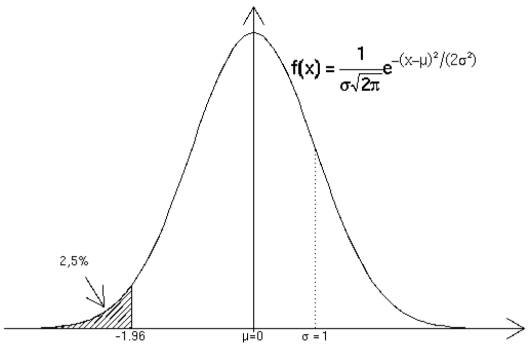
Randomness

We perform a strength test on concrete specimens

- N specimens, formally identical;
- N different rupture strengths measured, $f_i, i = 1, \ldots, N$;
- mean strength is $f_M = \sum f_i/N$;

About 50% of the specimens had a strength inferior to f_M... We must study how the strength varies.
Let's divide the strengths in J intervals and count the number n_j of samples that lie in each interval, and finally plot an histogram of n_j/N (*relative frequencies*) vs the strength intervals.

Randomness



If we ideally grow the number of samples and reduce the width of the intervals the relative frequency becomes a function of the strength, the *density of probability* of the strength.
The density of probability measures how frequent is a particular value in a population.
A common model of the density of probability is the *Gaussian curve* here in the left.

Randomness

We introduced randomness with the example of the strength of concrete, but a description in terms of a density of probability is good also for the different factors of uncertainty that we have listed.
All our discourse on the behaviour of buildings and the events that influence that behaviour *must* be framed in a probabilitic description.
In this sense modern building codes require that all the procedures (design, building, testing, ...) lead to a result that is *safe enough*.
Safe enough means that the probability that an effect is greater than the associated strength is small enough. How small is small enough? We decide in terms to the risks associated with the specific limit state.

What we want

The probabilistic approach, developed around the middle of the 20th century century, aims to evaluate the probability of collapse, P_r and check that it is less than a value, P_r^* (very small) considered acceptable w/r to the risk of the limit state.
To do this you must first know the density of probability of the loads and of the resistance of materials.

What we do

These operations are quite complicated if not impossible in most real situations. For this reason the probabilistic methods are articulated on different levels, with different complexity and different accuracy.

Levels

- Level 4 method (perfect) — the value of P_r is calculated exactly and the value of P_R^* is decided on a rational basis.
- Level 3 method (exact) — the value of P_r is exact, the value of P_r^* is derived from past experiences.
- Level 2 method — P_r is estimated with indirect techniques, P_r^* values are derived from experience and common sense as in the method of level 3.
- Level 1 method (simplified method) — or semi-probabilistic method — it constitutes the method adopted in generalized forms by modern codes.

Semi-probabilistic Method

The method still expressly considers all possibilities of crisis, trying to refer to the actual mechanism that causes the single crisis, but the security, $P_r \leq P_r^*$, is verified indirectly by comparing a reference value R_d of the resistance and a S_d reference value of effects (the d is for design).
These values are calculated trying to take into account

- for R_d, the density of probability of materials' resistances,
- for S_d, the density of probability of the loads,

while all the remaining causes of uncertainty and uncertainty (eg, correspondence between structure and model) are treated with the introduction of appropriate safety coefficients.

Semi-probabilistic Method

To have a sound method, we must define

- the life span of the building, T_s,
- the variable loadings (wind, snow, eartquake, etc) and their probabilty distribution w/r to the life of the building,
- a list of the possible different limit states, differentiated in terms of the associated risk and the associated accetable probabilities of exceeding P_r^*.

Semi-probabilistic Method

Working life

T_s value is chosen according to the type of construction, 10 or 50 years for ordinary constructions, hundreds of years for buildings that have a prevalent interest for the community.

Return period

The ΔT return period of an action is the *average* interval between two moments in which the action exceeds a given value or, by reversing the terms, to a given return period we can associate a characteristic value of the action with probability of being exceeded during the unit of time equal to $1/\Delta T$.

Semi-probabilistic Method

Limit states are states beyond which the performance requirements are no longer satisfied.
We have two different types of limit states

Ultimate Limit States associated with the various forms of structural failure or states close to structural failure.

Serviceability Limit State correspond to states beyond which specified criteria for the structure related to its use or function are not longer met.

By virtue of this distinction and the different amounts of direct damage in case of achievement of a limit state, we assign different accettable levels

- ULS, $P_r^* = 10^{-4} \div 10^{-6}$, or $\Delta T = 10 \div 20\, T_s$;
- SLS, $P_r^* = 10^{-1} \div 10^{-3}$, or $\Delta T = 0.2 \div 0.5\, T_s$.

Actions

Actions modify the state of a building.
In first instance actions are forces that require a modification in the state of the structure to be equilibrated (stress characteristics, deformations and deflections).
The designer must identify every significant action, to consider in structural design and structural verification.

Actions

Direct actions
Concentrated or distributed loads (self weigth of the structure or of the building, use of the building, wind pressure, snow, earthquake)

Indirect actions
Support settlements, forced displacements, variations of temperature, initial deformations and long-term deformations (shrinkage, viscosity) etc.

Actions

Static actions
Static actions necessarily require a state of stress to be equilibrated, and hence structural displacements.

Geometric actions
Produce structural displacements but not necessarily a state of stress: support settlements, variation of volume, etc.

Actions

Consider that all actions vary in time and so all structural displacements: the structure is moving in a neighborhood of its equilibrium position and forces of inertia are generated.

Static actions

The rate of variation of the action is so slow that the inertial forces are negligible.

Quasi-static actions

The inertial forces are not negligible but do not influence the structural response, the inertial forces are taken into account with a dynamic amplification factor.

Dynamic actions

The motion of the structure produces inertial forces so large that the structural response is greatly influenced (e.g., resonance)..
The inertial forces must be taken explicitly into account into the analysis, i.e., a *dynamic analysis* must be performed.

Actions

We have classified the action in terms of their short period variability, but we must consider also a different time scale.

Permanent actions ($\boldsymbol{G}$)

Are present for the entire working life of the structure.

Variable actions ($\boldsymbol{Q}$)

Are commonly present, with values that can be significantly different in different moments.

Accidental actions ($\boldsymbol{A}$)

Are rarely present, in case of accidents (collisions, fires, explosions, etc).

The terms ($\boldsymbol{G}$), ($\boldsymbol{Q}$) and ($\boldsymbol{A}$) are used by the EC to denote the different actions.

Actions

Permanent actions ($\boldsymbol{G}$)

Actions present during all the working life of the building, their rate of variation is so amall that, with a good approximation, we can consider them as constant:

- structural self weigth,
- weigth of supported parts of the building (floors, walls, roofs etc),
- soil pressure,
- pretension and postcompression,
- shrinkage, viscosity, etc.

Actions

Variable actions (Q)

Actions that act on the structure or on a structural member with instantaneous values that may be quite different in different times.

Long duration

Are present, with significant intensity, possibly discontinuously, for a significant part of the working life:

- self weigth of non structural parts,
- weight of objects that the structure is designed to support but could be removed.

Short duration

They act during time spans short with respect to the working life

- wind,
- snow,
- occupancy,
- earthquake,
- variation of temperature.

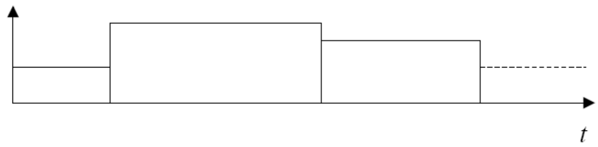
Actions

Permanenti:
(Peso proprio,
Sovraccarichi fissi)

Variabili:

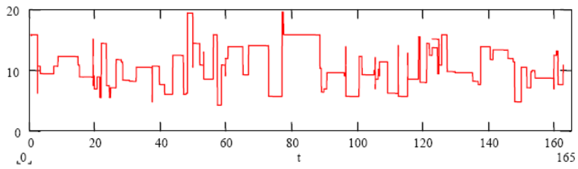
Quasi perm.

(Carico d'uso – arredi –)

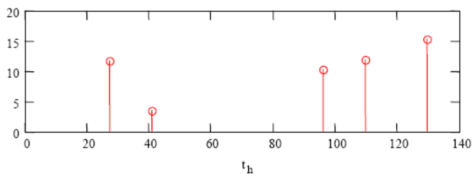


Actions

Rapidamente variabili:
(persone occupanti, vento, neve)



Azioni accidentali o eccezionali:
(esplosioni, urti, sisma)



Actions

Combinations

For every action $\boldsymbol{A}$ we have a characteristic value $\boldsymbol{A}_k$. With reference to the probability distribution of $\boldsymbol{A}$ $\boldsymbol{A}_k$ has a given, low probability of being exceeded in a unit time (tipically 5%).

> *In other words, the characteristic value has a non negligible probability of being exceeded.*

Actions are combined to maximize their effect on each part of the structure, i.e., if in a combination an action decreases the effect, the action enters the combination with a value lower than the characteristic value — on the contrary if the effect is increased, the action enters the combination with a value larger than the characteristic value.

In a combination, one of the variable actions takes the role of leading action and the other enter the combination as accompanying actions, scaled accordingly to reflect the reduced probability of their occurrence at characteristic values.

Actions on residential buildings

- permanent:
  - slabs, beams, joists, columns, structural walls, foundation systems,
  - floors,
  - vertical dividers,
  - roofs,
  - etc etc;
- variabile:
  - people, furniture, facilities and everything needed to the functions of the building,
  - wind, snow, all environmental loads;
- accidental: earthquake, collisions, etc.

For permanent actions Italian code makes a distivtion between structural loads and non-structural loads.

MATERIALI	PESO UNITÀ DI VOLUME [kN/m^3]
Calcestruzzi cementizi e malte	
Calcestruzzo ordinario	24,0
Calcestruzzo armato (e/o precompresso)	25,0
Calcestruzzi "leggeri": da determinarsi caso per caso	14,0 ÷ 20,0
Calcestruzzi "pesanti": da determinarsi caso per caso	28,0 ÷ 50,0
Malta di calce	18,0
Malta di cemento	21,0
Calce in polvere	10,0
Cemento in polvere	14,0
Sabbia	17,0
Metalli e leghe	
Acciaio	78,5
Ghisa	72,5
Alluminio	27,0
Materiale lapideo	
Tufo vulcanico	17,0
Calcare compatto	26,0
Calcare tenero	22,0
Gesso	13,0
Granito	27,0
Laterizio (pieno)	18,0
Legnami	
Conifere e pioppo	4,0 ÷ 6,0
Latifoglie (escluso pioppo)	6,0 ÷ 8,0
Sostanze varie	
Acqua dolce (chiara)	9,81
Acqua di mare (chiara)	10,1
Carta	10,0
Vetro	25,0
Per materiali non compresi nella tabella si potrà far riferimento a specifiche indagini sperimentali o a normative di comprovata validità assumendo i valori nominali come valori caratteristici.	

NTC: Tabella 3.1.I

concrete
reinforced concrete
light concrete
heavy concrete
lime mortar
cement mortar
lime powder
cement powder
sand

steel
cast iron
aluminum

volcanic tufa
hard limestone
soft limestone
gipsum
granite
brick masonry (full bricks)

coniferous and poplar
other deciduous trees

water
sea water
paper
glass

Non-structural permanent loads

Loads that are not frequently removed during the normal service of the building, e.g., external envelopes, vertical dividers, floors, plastering works, ceilings and false ceilings, technical installations etc.

We have to take into account that, sometimes, these loads are temporarily removed from the building.

These loads can be assimilated to distributed loads if the horizontal slabs, even if ribbed slabs, have a transverse redistribution capability.

Non-structural permanent loads

Vertical dividers

The self weigth of vertical dividers can be assumed an uniformly distributed load g_{2k}, if the construction of slabs permit a redistribution of the loads.

The g_{2k} here defined depends on the self weigth per unit of length of vertical dividers as follows:

- $G_{2k} \leq 2\,\mathrm{kN\,m^{-1}}$: $g_{2k} = 0.80\,\mathrm{kN\,m^{-2}}$
- $1\,\mathrm{kN\,m^{-1}} < G_{2k} \leq 2\,\mathrm{kN\,m^{-1}}$: $g_{2k} = 0.80\,\mathrm{kN\,m^{-2}}$
- $2\,\mathrm{kN\,m^{-1}} < G_{2k} \leq 3\,\mathrm{kN\,m^{-1}}$: $g_{2k} = 1.20\,\mathrm{kN\,m^{-2}}$
- $3\,\mathrm{kN\,m^{-1}} < G_{2k} \leq 4\,\mathrm{kN\,m^{-1}}$: $g_{2k} = 1.60\,\mathrm{kN\,m^{-2}}$
- $4\,\mathrm{kN\,m^{-1}} < G_{2k} \leq 5\,\mathrm{kN\,m^{-1}}$: $g_{2k} = 2.00\,\mathrm{kN\,m^{-2}}$

Vertical dividers with larger unit weigths must be taken int account individually, with consideration of their actual position.

Variable Loads

Cat.	Ambienti	q_k [kN/m^2]	Q_k [kN]	H_k [kN/m]
A	**Ambienti ad uso residenziale.** Sono compresi in questa categoria i locali di abitazione e relativi servizi, gli alberghi. (ad esclusione delle aree suscettibili di affollamento)	2,00	2,00	1,00
B	**Uffici.**			
	Cat. B1 Uffici non aperti al pubblico	2,00	2,00	1,00
	Cat. B2 Uffici aperti al pubblico	3,00	2,00	1,00
C	**Ambienti suscettibili di affollamento**			
	Cat. C1 Ospedali, ristoranti, caffè, banche, scuole	3,00	2,00	1,00
	Cat. C2 Balconi, ballatoi e scale comuni, sale convegni, cinema, teatri, chiese, tribune con posti fissi	4,00	4,00	2,00
	Cat. C3 Ambienti privi di ostacoli per il libero movimento delle persone, quali musei, sale per esposizioni, stazioni ferroviarie, sale da ballo, palestre, tribune libere, edifici per eventi pubblici, sale da concerto, palazzetti per lo sport e relative tribune	5,00	5,00	3,00
D	**Ambienti ad uso commerciale.**			
	Cat. D1 Negozi	4,00	4,00	2,00
	Cat. D2 Centri commerciali, mercati, grandi magazzini, librerie…	5,00	5,00	2,00

Residential Environment
Apartments and hotel rooms
Office environment
Offices open to general public, Offices not open to general public.
Possibly crowded environments
Hospitals, restaurants, cafes, banks, schools
Terraces, stairs, meeting rooms, cinema, theaters, churches, stands with seatings
Museums etc, train stations, dancing rooms, gyms, stands, public events, concert halls, sports hall.
Commercial environments
Shops
Shopping malls, markets, department stores, libraries/bookshops.

q_k is a distributed load, Q_k a concentrated load and H_k an horizontal, distributed load to be applied to rails and vertical dividers.

E	**Biblioteche, archivi, magazzini e ambienti ad uso industriale**.			
	Cat. E1 Biblioteche, archivi, magazzini, depositi, laboratori manifatturieri	≥ 6,00	6,00	1,00*
	Cat. E2 Ambienti ad uso industriale, da valutarsi caso per caso	—	—	—
F-G	**Rimesse e parcheggi**.			
	Cat. F Rimesse e parcheggi per il transito di automezzi di peso a pieno carico fino a 30 kN	2,50	2 x 10,00	1,00**
	Cat. G Rimesse e parcheggi per transito di automezzi di peso a pieno carico superiore a 30 kN: da valutarsi caso per caso	—	—	—
H	**Coperture e sottotetti**			
	Cat. H1 Coperture e sottotetti accessibili per sola manutenzione	0,50	1,20	1,00
	Cat. H2 Coperture praticabili	secondo categoria di appartenenza		
	Cat. H3 Coperture speciali (impianti, eliporti, altri) da valutarsi caso per caso	—	—	—

* non comprende le azioni orizzontali eventualmente esercitate dai materiali immagazzinati
** per i soli parapetti o partizioni nelle zone pedonali. Le azioni sulle barriere esercitate dagli automezzi dovranno essere valutate caso per caso

Libraries, archives, storage and industrial buildings
Libraries, archives, storage, deposits, workshops
Industrial environments, individual analysis req.
Garages and parking lots
G&PL for vehicles up to 30 kN
for vehicles over 30 kN, individual analysis req.
Roofs and Attics
Accessible only for maintenance
Accessible — according to pertaining category
Special destinations (machineries, heliport, etc) individual analysis req.

Snow

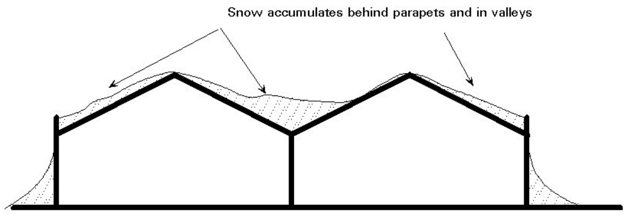


Figure 17 Accumulation of snow

Snow loading is evaluated w/r to the building site (local climate and specific exposure conditions) and the type of roofing.

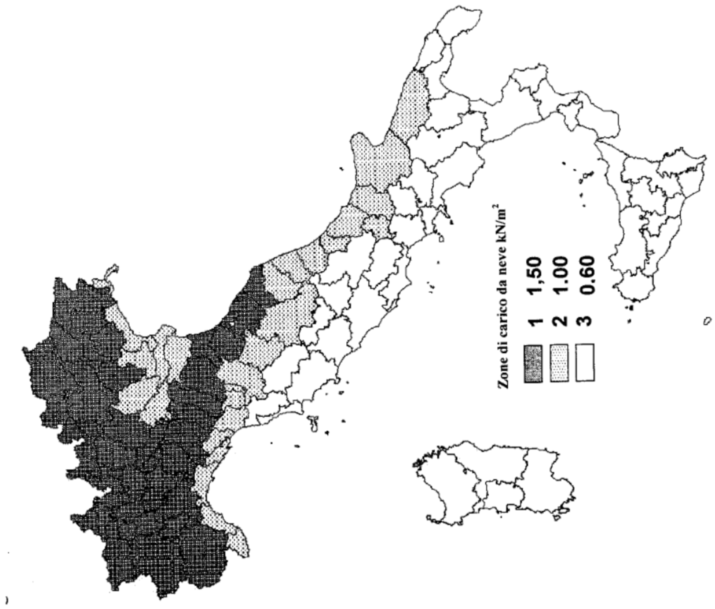
We have to determine the snow load on ground and the influence of the shape of the roof on the accumulation of snow on it.

Snow

$$q_s = \mu_i \cdot q_{sk} \cdot C_E \cdot C_t$$

- q_s is the snow load on the roof
- μ_i is the shape coefficient of the roof
- q_{sk} is the characteristic value of the snow load on the ground, with a return period of 50 years (every 50, in mean, the snow load will be higher)
- C_E is the exposition coefficient
- C_t is a thermic coefficient.

By hypotesis, the load is vertical and is referred to the horizontal projection of the roof.



Snow, q_{sk}

Zone 1 — Alpine

$$q_{sk} = \begin{cases} 1.50\,\mathrm{kN^2\,m^{-1}} & a_s \leq 200\,\mathrm{m} \\ (1.39 + (1 + (\frac{a_s}{728\mathrm{m}})^2))\mathrm{kN^2\,m^{-1}} & a_s > 200\,\mathrm{m} \end{cases}$$

Zone 1 — Mediterranean

$$q_{sk} = \begin{cases} 1.50\,\mathrm{kN^2\,m^{-1}} & a_s \leq 200\,\mathrm{m} \\ (1.35 + (1 + (\frac{a_s}{602\mathrm{m}})^2))\mathrm{kN^2\,m^{-1}} & a_s > 200\,\mathrm{m} \end{cases}$$

Zone 2

$$q_{sk} = \begin{cases} 1.00\,\mathrm{kN^2\,m^{-1}} & a_s \leq 200\,\mathrm{m} \\ (0.85 + (1 + (\frac{a_s}{481\mathrm{m}})^2))\mathrm{kN^2\,m^{-1}} & a_s > 200\,\mathrm{m} \end{cases}$$

Zone 3

$$q_{sk} = \begin{cases} 0.60\,\mathrm{kN^2\,m^{-1}} & a_s \leq 200\,\mathrm{m} \\ (0.51 + (1 + (\frac{a_s}{481\mathrm{m}})^2))\mathrm{kN^2\,m^{-1}} & a_s > 200\,\mathrm{m} \end{cases}$$

a_s is the height over the sea of the construction site,

Snow, C_E and C_t

Topography	Description	C_E
Wind blown	Flat unobstructed zone, exposed on every side, no trees or buildings around	0.9
Normal	The wind does not remove the snow from the roof	1.0
Repaired	Zones where the building is in a depression or is encircled by higher trees/buildings	1.1

The thermal coefficient, in lack of a detailed analysis, can be taken into account as

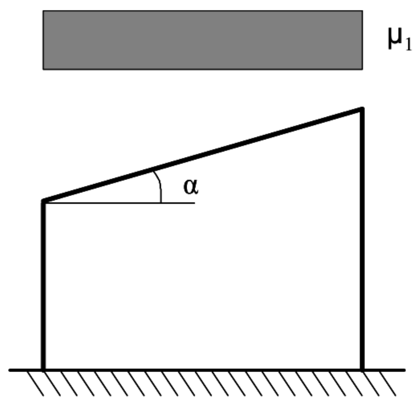
$$C_t = 1.0.$$

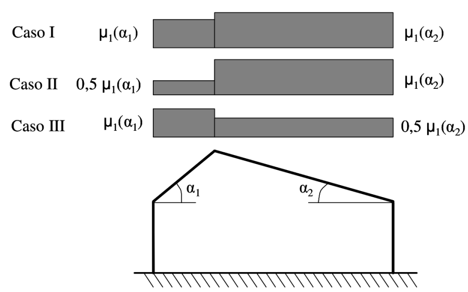
Snow, μ_1

α is the angle between the roof and the horizontal.

Shape coefficient	$0 \leq \alpha \leq 30$	$30 \leq \alpha \leq 60$	$60 \leq \alpha$
μ	0.8	$0.8 \frac{60-\alpha}{30}$	0

These values are for a single-sided or two-sided roof. More complex roofs must be studied in more detail.





Snow, Lombardy high plains

Zone 1 — $a_s \approx 250\,\mathrm{m}$

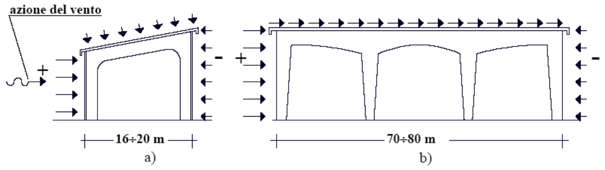
$$q_s = 1.35 + (1 + \left(\frac{250}{602}\right)^2 = 1.583 \approx 1.60\mathrm{kN/m^2}$$

Exposition: normal, $C_E = 1.0$
Roof: almost flat, $\alpha < 30 \rightarrow \mu_1 = 0.8$
The characteristic value of the snow load *on the roof* is hence

$$q_{sk} = 1.60 \cdot 0.8 \cdot 1.0 \cdot 1.0 = 1.28\mathrm{kN/m^2}$$

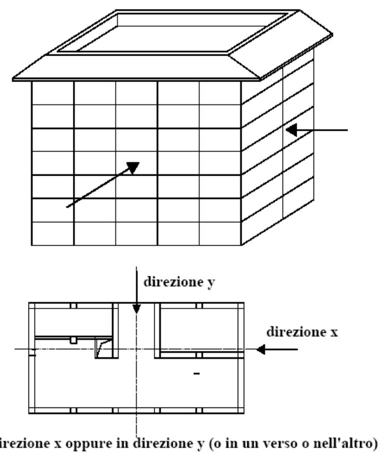
Wind

Wind exert a pressure (a depression) on the building surfaces.
Conventionally pression is positve and depression negative.



For large extension roofs, we have to take into account also the tangential force exerted by the wind.
The wind direction is defined as horizontal and the dynamic effects are regarded as negligible — this is not true for unusual constructions, particularly high or slender or flexible — in such circumstances the following won't apply.

Wind



Wind

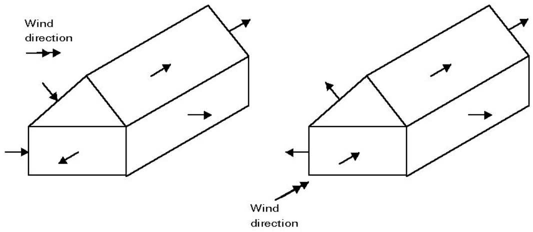


Figure 13 Effect of wind direction on pressure distribution

Wind

Local effects of wind on roofs

Wind pressure

A constant air flux produces, on an orthogonal surface, a pressure

$$p = \frac{1}{2} C \rho v^2$$

where $\rho = 12.5\,\mathrm{kN/m}^3$ is the mass density of the air, v is the velocity and C is an aerodynamic coefficient.
Our study of wind begins with the *reference velocity* of the air flux, for us v_b is the characteristic value of the wind velocity at 10 m over the ground (category II, see further) with a return period of 50 years.

$$v_b = \begin{cases} v_{b,0} & a_s \leq a_0 \\ v_{b,0} + k_a(a_s - a_0) & a_0 \leq a_s \leq 1500\,\mathrm{m} \end{cases}$$

$v_{b,0}, a_0, k_a$ are parameters defined in terms of the site location, a_s is the elevation of the construction site.
For $a_s > 1500$m a study of local conditions is required.

Wind

Tabella 3.3.I - *Valori dei parametri* $v_{b,0}$, a_0, k_a

Zona	Descrizione	$v_{b,0}$ [m/s]	a_0 [m]	k_a [1/s]
1	Valle d'Aosta, Piemonte, Lombardia, Trentino Alto Adige, Veneto, Friuli Venezia Giulia (con l'eccezione della provincia di Trieste)	25	1000	0,010
2	Emilia Romagna	25	750	0,015
3	Toscana, Marche, Umbria, Lazio, Abruzzo, Molise, Puglia, Campania, Basilicata, Calabria (esclusa la provincia di Reggio Calabria)	27	500	0,020
4	Sicilia e provincia di Reggio Calabria	28	500	0,020
5	Sardegna (zona a oriente della retta congiungente Capo Teulada con l'Isola di Maddalena)	28	750	0,015
6	Sardegna (zona a occidente della retta congiungente Capo Teulada con l'Isola di Maddalena)	28	500	0,020
7	Liguria	28	1000	0,015
8	Provincia di Trieste	30	1500	0,010
9	Isole (con l'eccezione di Sicilia e Sardegna) e mare aperto	31	500	0,020

Wind

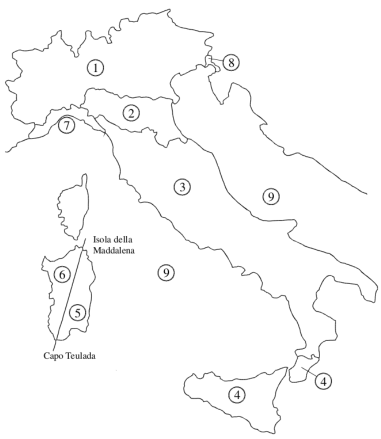


Figura 3.3.1 – *Mappa delle zone in cui è suddiviso il territorio italiano*

Wind, eq. static forces

Wind static forces are pressures and depressions normal to the external and internal surfaces of the buildings.
Wind action on a building element is the worst combination of the pressures acting on the external side and the internal side.
In the case of large extension buildings, the tangential wind force has to be taken into account.
The total action on a building is the resultant of the actions on individual elements.
The directions to take into account are both the directions of the principal axes of the building.

Wind pressure

$$p = q_b \cdot c_e \cdot c_p \cdot c_d$$

- q_b reference pressure, see previous slides,
- c_e exposure coefficient,
- c_p aerodynamic coefficient, depends on the construction geometry and the wind direction,
- c_d dynamic coefficient, takes into account the spatial variability of the wind and the interaction with structural vibrations.

Wind, tangential forces

$$p_f = q_b \cdot c_e \cdot c_f$$

where c_f is the friction coefficient, depending on the superficial roughness of the roof.

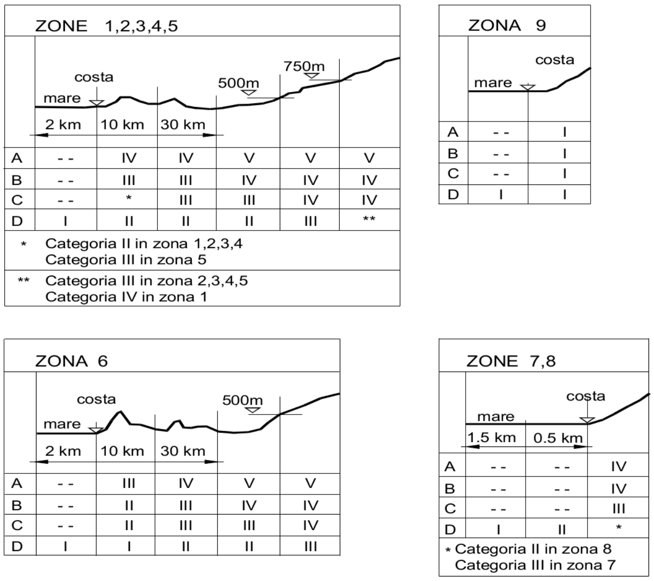
Wind, c_e

c_e depends on z, height above the ground, the site topography and the exposition category of the site. For $z \leq 200\,\mathrm{m}$

$$c_e = \begin{cases} k_r^2 c_t \log(z/z_0) \cdot (7 + c_t \log(z/z_0)) & z \geq z_{\min} \\ c_e(z_{\min}) & z < z_{\min} \end{cases}$$

where c_t is a topographic coefficient and the other parameters are

Categoria di esposizione del sito	k_r	z_0 [m]	z_{min} [m]
I	0,17	0,01	2
II	0,19	0,05	4
III	0,20	0,10	5
IV	0,22	0,30	8
V	0,23	0,70	12



ZONE 1,2,3,4,5

A	- -	IV	IV	V	V	V
B	- -	III	III	IV	IV	IV
C	- -	*	III	III	IV	IV
D	I	II	II	II	III	**

* Categoria II in zona 1,2,3,4
Categoria III in zona 5

** Categoria III in zona 2,3,4,5
Categoria IV in zona 1

ZONA 9

A	- -	I
B	- -	I
C	- -	I
D	I	I

ZONA 6

A	- -	III	IV	V	V
B	- -	II	III	IV	IV
C	- -	II	III	III	IV
D	I	I	II	II	III

ZONE 7,8

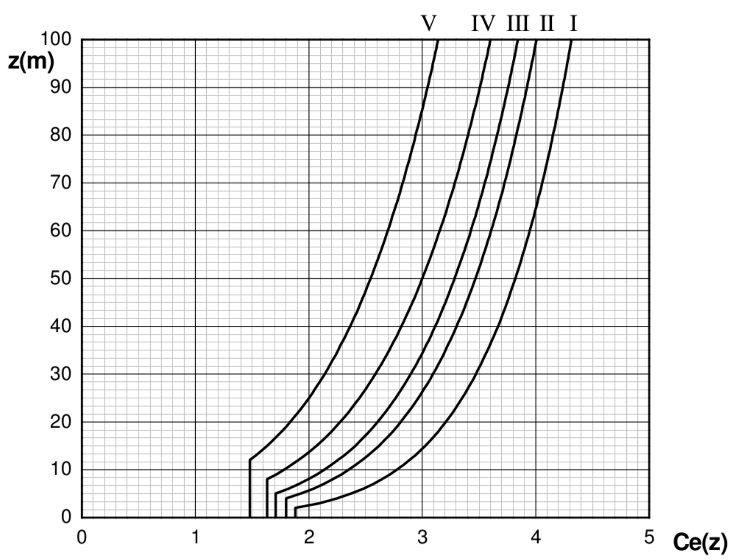
A	- -	- -	IV
B	- -	- -	IV
C	- -	- -	III
D	I	II	*

* Categoria II in zona 8
Categoria III in zona 7

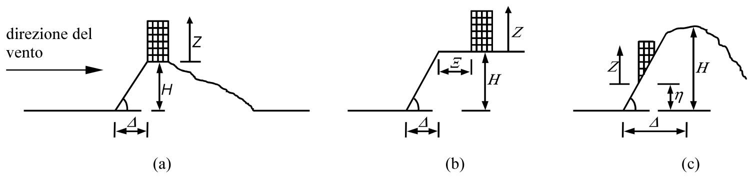
Table with exposition categories, depending on the climate zone 1–9, the distance from the coastline and the site elevation. For zones 1, 2, 3, 4, 5, 6 the exposition category within 40 km from the coast doesn't depend on the elevation of the site.

Wind, terrain roughness

Terrain roughness class	Description
A	Urban areas, 15% of the surface covered by building whose mean height is over 15 m.
B	Urban areas (not in class A), suburban areas, industrial areas and wooded areas.
C	Areas with sparse obstacles.
D	Areas with no obstacles.

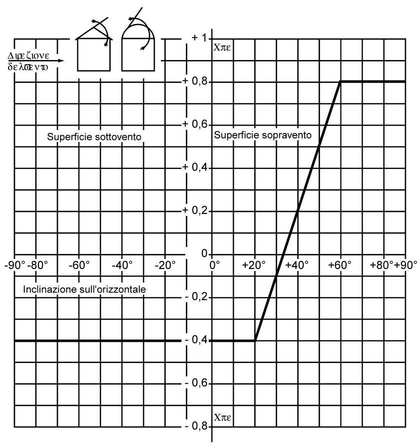


Wind, c_t



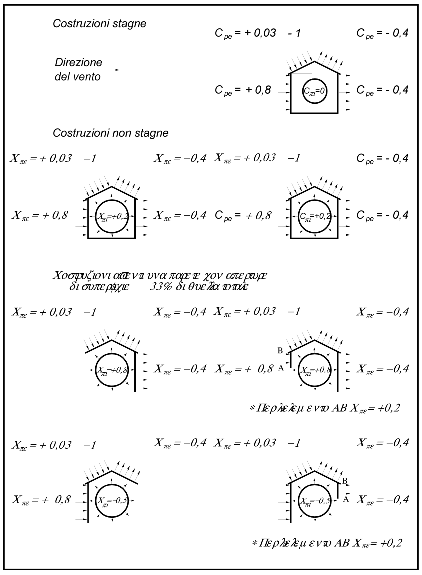
Usually $c_t = 1.0$

Wind, c_p



Wind, c_p

The internal pressure coefficient is $c_p = 0$ for airtight buildings, $c + p = \pm 0.2$ for non-airtight buildings, $c_p = +0.8$ for buildings with large ($> 1/3$ of the total surface) openings when upwind, $c_p = -0.5$ if downwind or parallel to wind and finally $c_{pe} + c_{pi} = \pm 1.2$ when the openings are on two opposite surfaces.



Wind, c_d

Usually $c_d = 1$

Wind in Milan

$a_s \approx 100\,\mathrm{m}$, $H = 125\,\mathrm{m}$ Zone 1, $v_{b0} = 25\,\mathrm{m/s}$, $a_0 = 1000\,\mathrm{m}$, $k_a = 0.010\,1/\mathrm{s}$.
It is $v_b = v_{b0} = 25\,\mathrm{m/s}$ and $q_b = 0.5 \times 1.25 v_b^2 = 390.625\,\mathrm{N/m^2}$.
Hypotizing a downtown position, the roughness zone is A, the site is far from the coast but below 500 m of elevation, hence exposure zone V and $k_r = 0.23$, $z_0 = 0.70\,\mathrm{m}$ and $z_{\min} = 12\,\mathrm{m}$.
Using $c_t = c + d = 1$

z(m)	k_r	z_o	z_{min}	c_t	c_e	q_{ref}	$c_e \cdot q_{ref}$
12	0,23	0,7	12	1	1,48	390,62	578,12
16	0,23	0,7	12	1	1,68	390,62	656,24
25	0,23	0,7	12	1	2	390,62	781,24
50	0,23	0,7	12	1	2,54	390,62	992,17
75	0,23	0,7	12	1	2,89	390,62	1128,89
100	0,23	0,7	12	1	3,14	390,62	1226,55
125	0,23	0,7	12	1	3,34	390,62	1304,67

Wind in Trieste/Trst

Zone 8, $v_{b0} = 31\,\mathrm{m/s}$, $a_0 = 1500\,\mathrm{m}$, $k_a = 0.012\,1/\mathrm{s}$.
It is $v_b = v_{b0} = 25\,\mathrm{m/s}$ and $q_b = 0.5 \times 1.25 v_b^2 = 600.62\,\mathrm{N/m^2}$.
Hypotizing a downtown position, the roughness zone is A and the site is on the coast, hence exposure zone V and $k_r = 0.22$, $z_0 = 0.30\,\mathrm{m}$ and $z_{\min} = 8\,\mathrm{m}$.
Using $c_t = c + d = 1$

z(m)	k_r	z_o	z_{min}	c_t	c_e	q_{ref}	$c_e \cdot q_{ref}$
8	0,22	0,3	8	1	1,63	600,62	979,01
16	0,22	0,3	8	1	2,11	600,62	1267,31
25	0,22	0,3	8	1	2,45	600,62	1471,52
50	0,22	0,3	8	1	3,00	600,62	1801,86
75	0,22	0,3	8	1	3,35	600,62	2012,08
100	0,22	0,3	8	1	3,60	600,62	2162,23
125	0,22	0,3	8	1	3,80	600,62	2282,36

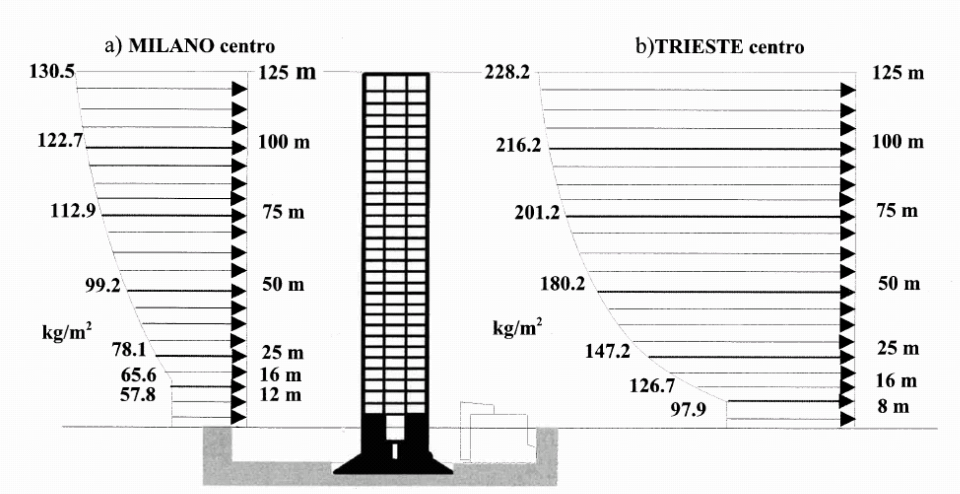


Fig. 2.25 - Profilo altimetrico della pressione del vento sul grattacielo Pirelli a Milano e pressioni a Trieste

Frost and defrost

Combination of loadings

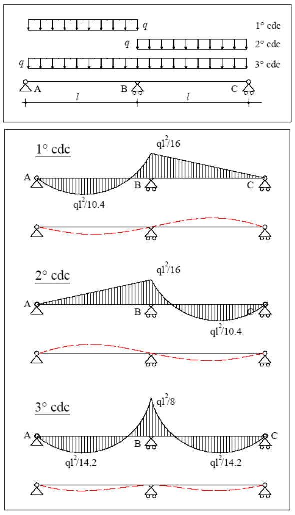
Continuous beam on 3 supports

Our beam has equal spans of length L and is loaded by a live load of intensity q.

In the first and in the second case just one of the spans is loaded, in the third case both spans are loaded.

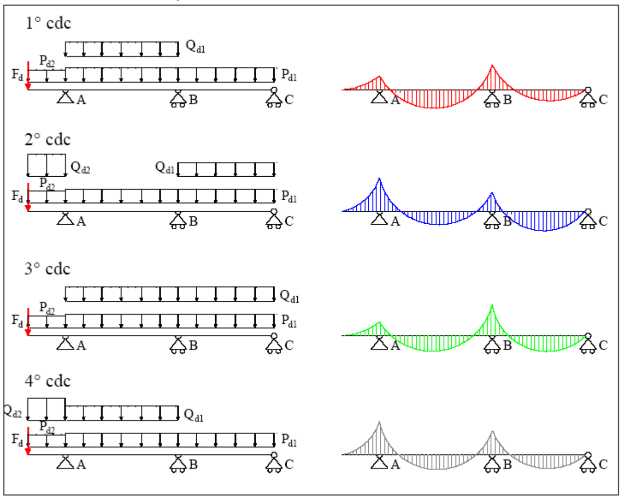
You may think that the 3rd load case is the most severe.

This is not true... It is true that the maximum bending moment, negative on the central support, happens for the 3rd load case but the maximum value of the bending moment for the midspans happens for the other two load cases.



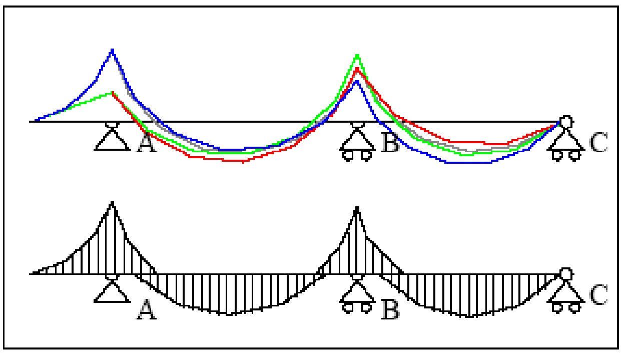
Combination of loadings

Another example:

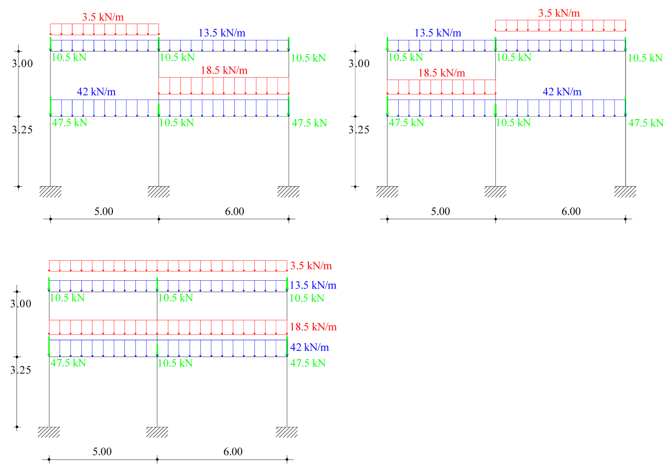


Combination of loadings

To design a beam working in flexure we need an *envelope* of the bending moments — for every section we need the minimum negative and the maximum positive bending moment values:



Combination of loadings



Combination of loadings

In load combinations for different limit states we have always the permanent loads with different combination factors and the live loads, also with different combination factors, but with different combination values.
One of the variable loads has the role of the leading load, while the others enter the combination with accompanying values.
The accompanying values are:

the combination value $\psi_0 Q_k$, the characteristic value multiplied by the coefficient ψ_0 ($\psi_0 \leq 1$), it is used for the verification of ultimate limit states and irreversible serviceability limit states;

the frequent value $\psi_1 Q_k$ used for the verification of ultimate limit states involving accidental actions and for verifications of reversible serviceability limit states,

the quasi-permanent value $\psi_2 Q_k$, used for the verification of ultimate limit states involving accidental actions and for the verification of reversible serviceability limit states. Quasi- permanent values are also used for the calculation of long-term effects.

Action	ψ_0	ψ_1	ψ_2
Imposed loads in buildings, category (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 F: traffic area, vehicle weight ≤ 30kN	0,7	0,7	0,6
Category G: traffic area, 30kN < vehicle weight ≤ 160kN	0,7	0,5	0,3
Category H: roofs	0	0	0
Snow loads on buildings (see EN 1991-1-3)			
– Finland, Iceland, Norway, Sweden	0,70	0,50	0,20
– Remainder of CEN Member States, for sites located at altitude H > 1000 m a.s.l.	0,70	0,50	0,20
– Remainder of CEN Member States, for sites located at altitude H ≤ 1000 m a.s.l.	0,50	0,20	0
Wind loads on buildings (see EN 1991-1-4)	0,6	0,2	0
Temperature (non-fire) in buildings (see EN 1991-1-5)	0,6	0,5	0
Note: The ψ values may set by the National annex.			

Combination of loadings

Combinations for different limit states

Fundamental combination for ultimate limit states

$$\gamma_{G1} G_1 + \gamma_{G2} G_2 + \gamma_P P + \gamma_{Q1} Q_{k1} + +\gamma_{Q2} \psi_{02} Q_{k2} + \dots$$

frequent combination for irreversible SLS

$$G_1 + G_2 + P + Q_{k1} + +\psi_{02} Q_{k2} + \dots$$

frequent combination reversible SLS

$$G_1 + G_2 + P + \psi_{11} Q_{k1} + +\psi_{22} Q_{k2} + \dots$$

quasi-permanent for long term effects

$$G_1 + G_2 + P + \psi_{21} Q_{k1} + +\psi_{22} Q_{k2} + \dots$$

Accidental

$$A_d + G_1 + G_2 + P + \psi_{21} Q_{k1} + +\psi_{22} Q_{k2} + \dots$$

Combination of loadings

In the previuos combinations the $+$ sign has an extended meaning of combination of actions.
The partial coefficient applicable to structural permanent loads has two possible combination values, $\gamma_{G1} = 1.0$ or $\gamma_{G1} = 1.3$ and combination means that we use the partial coefficient that gives the most important effect for the quantity regarded.
For non structural permanent loads it is $\gamma_{G1} = 0.0$ or $\gamma_{G1} = 1.5$.
For variable loads it is $\gamma_{Q1} = 0.0$ or $\gamma_{Q1} = 1.5$ or some loads we must take into account that the load can act in the reverse direction,
Eventually the reduction factors for secondary variable loads have been already shown.