Annex for The Netherlands

Steel Design 1

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Structural basics

bouwen met staal

Colofon / Content

Annex for the Netherlands to *Structural basics* (Steel Design 1)

This annex has been prepared by prof.ir. H.H. Snijder and is based on the original Dutch version of *Stuctural basics*, published in 2011 (and updated in 2012) by Bouwen met Staal as *Krachtswerking* by the same authors. References are made to each NA symbol in *Stuctural basics* and the correspon-ding clause in the Eurocode.
Annexes to *Structural basics* (Steel Design 1) are also available for Belgium, Germany, Luxembourg and Switzerland and can be downloaded free of charge from the website of Bouwen met Staal.

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Eurocodes in The Netherlands

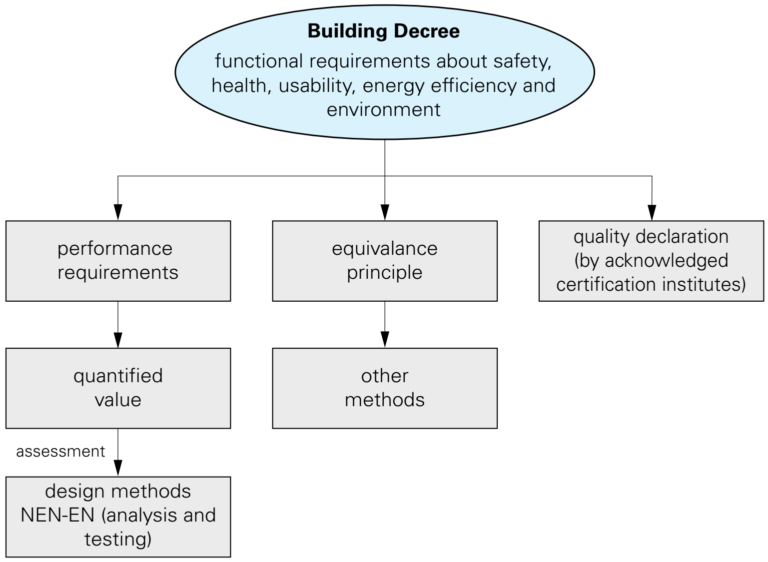
Dutch Building Decree and Eurocodes

The Dutch Building Decree (in Dutch: Bouwbesluit) includes, amongst other things, minimum requirements regarding structures, which every structure in the Netherlands – including those for residential buildings, offices and bridges – has to meet. Fulfilling these requirements is required by law in order to obtain a building permit. In addition to new building structures, renovations are also regulated by the Dutch Building Decree.
The Dutch Building Decree is a General Administrative Order (in Dutch: Algemene maatregel van bestuur) which belongs to the Dutch Housing Law. The first version came into force in 1992. The latest Dutch Building Decree – which came into force on April 1, 2012 – was published in the *Law Gazette* (in Dutch: *Staatsblad*) (2011) 416. The Dutch Building Decree 2012 was last amended on June 22, 2018.

The Dutch Building Decree 2012 only concerns the public law aspects of newly built or existing structures. Private law aspects – such as serviceability requirements concerning deflections, or execution aspects – should always be agreed between the parties involved. The requirements of the Dutch Building Decree are organized into five categories:

– *safety*: preventing or limiting of danger for users of the building or for other parties;
– *health*: preventing or limiting of harmful or inconvenient consequences for users of the building;
– *usability*: facilitating performance of the characteristic activities for the building;
– *energy efficiency*: contributing to an efficient use of energy in the building;
– *environment*: avoiding too much irreversible damage to the environment (soil, air, water) due to the building.

There are also requirements for installations, which are not divided into these categories. The Dutch Building Decree also contains requirements regarding fire safety, construction, and demolition activities.
The requirements of the Dutch Building Decree are formulated as performance requirements. A performance requirement is derived from functional requirements, and is quantified through a limiting value as a minimum requirement. For the assessment of a building structure requirements regarding safety are of particular importance. The Building Decree refers to the Eurocodes for checking the resistance of structures. Apart from performance requirements in the form of limiting values, the Dutch Building Decree also provides so-called determination methods. These determination methods can be used to establish if a building structure or part of a structure meets the limiting values of the performance requirements. The determination methods are provided in codes, for



NL1 Relation scheme Dutch Building Decree.

structures in the Eurocodes. If an application for a building permit is based on, and meets, the requirements of the (Euro)codes designated in the Dutch Building Decree, then the applicant may assume that the building permit will be granted so far as technical requirements are concerned.
To allow innovations the Dutch Building Decree also has a so-called equivalence principle, apart from the system of performance requirements and determination methods. Based on this equivalence principle, building structures which cannot simply be assessed using the determination methods of the Eurocodes can still be designed and built. The applicant should in such cases show by means of other methods – for example tests – that the structure preforms as intended by the prescribed performance requirements.
Recognized quality declarations have a separate status within the Dutch Building Decree. In this case, recognized means that the quality declaration has been prepared and issued by an accredited certification institution. A quality declaration is generally linked to a construction product in a particular application. The quality declaration implicitly demonstrates that the construction product meets the performance requirements, or has an equivalent quality.

Figure NL1 shows schematically the way in which the requirements of the Dutch Building Decree can be met.

The Dutch Building Decree does not provide requirements concerning deformations of a structure. Despite EN 1990 specifying serviceability requirements for elements of a structure through the Dutch National Annex, these requirements are not mandatory according to the Dutch Building Decree. Therefore an assessment of deformations is not necessary in order to obtain a building permit. In practice however, checking deformations is an essential part of the assessment of a structure. The requirements mentioned in the Dutch National Annex to EN 1990 are practically always applied.

denotation	title
CUR-recommendation 25	*Korte ankers in beton. Berekening en uitvoering*, published by CUR, 2000. (Short anchors in concrete. Calculation and execution)
CUR/BmS-report 10	*Kolomvoetplaatverbindingen. Aanbevelingen voor de berekening volgens de Eurocodes*, published by Bouwen met Staal and CUR, 2009. (Column base plate connections. Recommendations for the design according to the Eurocodes)
RMBS 2004	*Richtlijnen voor de toepassing van metalen beplating als schijf-constructie*, published by Bouwen met Staal, 2004. (Guidelines for the application of metal sheeting as membrane structure)
RS 1990	*Reken- en beproevingsmethoden ter bepaling van sterkte en stijfheid van sandwichpanelen*, published by Bouwen met Staal, 1990. (Design and test methods for the determination of strength and stiffness of sandwich panels)
–	*Dwarskrachtverbindingen. Tabellen voor het ontwerp van hoekstaal-, kopplaat- en lipverbindingen volgens Eurocode 3*, published by Bouwen met Staal, 2006. (Shear connections. Tables for the design of beam-to-column connections with angle sections, end plates and fin plates according to Eurocode 3)
–	*Kwaliteitsrichtlijn applicatie brandwerende coating*, published by Bouwen met Staal, 2010 (2e druk). (Quality guideline for the application of fire-resistant coating, 2nd edition)
–	*Momentverbindingen*, published by Bouwen met Staal, 1998. (Moment resistant connections)
–	*Normaalkrachtverbindingen en dwarskrachtverbindingen*, published by Bouwen met Staal, 1998. (Normal and shear force connections)
–	*Trillingen van vloeren door lopen. Richtlijn voor het voorspellen, meten en beoordelen*, published by SBR, 2005. (Vibration of floors due to walking. Guideline for prediction, measurement and assessment)
–	*Vloeren van kanaalplaten met geïntegreerde stalen liggers* (Technisch dossier 2), published by Bouwen met Staal, 2007. (Hollow core floors and integrated steel beams (Technical dossier 2))

NL2 Selection of Dutch guidelines for steel structures.

Dutch guidelines for steel structures

When a designer makes use of national Dutch guidelines, the assessment of a steel structure against the requirements of the Dutch Building Decree should always be based on the equivalence principle, see figure NL1.

Table NL2 shows a selection of Dutch guidelines for steel structures. In general, they are issued by BmS (in Dutch: Bouwen met Staal, Dutch Steel Association), sometimes in collaboration with other institutions. Guidelines for composite steel-concrete structures are, for example, published in collaboration with CUR (in Dutch: Centrum Uitvoering Research en Regelgeving, Centre for Civil Engineering Research, Codes and Specifications).

Annex NL 1

Structural safety

p. 1-7 ***EN 1993, cl. 6.1(1)***

The recommended values for the partial factors for resistance are used.

p. 1-10 (a + b) ***EN 1990, cl. A1.2.2, table A1.1***

The Dutch National Annex gives different ψ factors, see table NL1.1.

p. 1-11 (a) ***EN 1990, cl. A1.3.1, table A1.2(B)***

The reduction factor ξ is: $\xi = 1{,}2/1{,}35 = 0{,}89$.

NL1.1 Values of ψ factors for variable actions.

action	ψ_0	ψ_1	ψ_2
imposed loads in buildings:			
– cat. A: domestic, residential areas	0,4	0,5	0,3
– cat. B: office areas	0,5	0,5	0,3
– cat. C: congregation areas	0,6/04[a]	0,7	0,6
– cat. D: shopping areas	0,4	0,7	0,6
– cat. E: storage areas	1,0	0,9	0,8
– cat. F: traffic area, vehicle weight ≤ 30 kN	0,7	0,7	0,6
– cat. G: traffic area, 30 kN < vehicle weight ≤ 160 kN	0,7	0,5	0,3
– cat. H: roofs	0	0	0
snow loads:	0	0,2	0
wind loads	0	0,2	0
temperature (non-fire) in buildings	0	0,5	0

a. The value 0,6 should be applied for parts of the building which can be loaded heavily by a crowd during exceptional events (escape routes, stairs).

p. 1-11 (b) ***EN 1990, cl. 6.4.3.2(3) and cl. A1.3.1***

Only equations (1.9) and (1.10) with specific values for γ_G and γ_Q should be applied for buildings. For the ultimate limit state regarding internal failure (or occurrence of excessive deformations) of the structure, where the resistance of the structural materials and/or members is critical (STR, see section 1.6.6) and prestress is neglected, the equations (1.9) and (1.10) can be modified into:

$$1{,}35G + \sum_{i\geq 1} 1{,}5\psi_{0,i}Q_{k,i}$$

$$1{,}2G + 1{,}5Q_{k,1} + \sum_{i>1} 1{,}5\psi_{0,i}Q_{k,i}$$

These equations change for a grandstand (CC3, RC3, $\beta = 4{,}3$ and $K_{FI} = 1{,}1$) into:

$$1{,}1\left(1{,}35G + \sum_{i\geq 1} 1{,}5\psi_{0,i}Q_{k,i}\right) = 1{,}5G + \sum_{i\geq 1} 1{,}65\psi_{0,i}Q_{k,i}$$

$$1{,}1\left(1{,}2G + 1{,}5Q_{k,1} + \sum_{i>1} 1{,}5\psi_{0,i}Q_{k,i}\right) = 1{,}3G + 1{,}65Q_{k,1} + \sum_{i>1} 1{,}65\psi_{0,i}Q_{k,i}$$

For an industrial building with one or two floors (CC1, RC1, $\beta = 3{,}3$ and $K_{FI} = 0{,}9$) the fundamental combinations of actions become as follows:

$$0{,}9\left(1{,}35G + \sum_{i\geq 1} 1{,}5\psi_{0,i}Q_{k,i}\right) = 1{,}2G + \sum_{i\geq 1} 1{,}35\psi_{0,i}Q_{k,i}$$

$$0{,}9\left(1{,}2G + 1{,}5Q_{k,1} + \sum_{i>1} 1{,}5\psi_{0,i}Q_{k,i}\right) = 1{,}1G + 1{,}35Q_{k,1} + \sum_{i>1} 1{,}35\psi_{0,i}Q_{k,i}$$

For greenhouses and light structures, or structural components in consequence class CC1, the actions may be reduced by application of a factor K_{FI} = 0,85; see the Dutch National Annex to EN 1990, cl. B.3.3.

p. 1-11 (c) ***EN 1990, cl. 6.4.3.3 and cl. A1.3.2 (table A1.3)***

Equation (1.14) is used in The Netherlands.

p. 1-12 ***EN 1990, cl. A1.4.1***

The characteristic combinations of actions are used for assessing both the vertical deflections of roofs which are not frequently used by persons, and for assessing horizontal deformations. The frequent combinations of actions should be used for assessment of the vertical deflection of floors and roofs which are frequently used by persons.

p. 1-14 (a + b) ***EN 1990, cl. B3.1(1), table B1***

The Dutch National Annex provides a different classification of building types into consequence classes, see table NL1.2.

p. 1-15 (a) ***EN 1990, cl. 6.4.3.2(3) and cl. A1.2(B)***

See remarks to p. 1-11 (b).

p. 1-15 (b) ***Assessment of existing buildings***

The assessment of existing buildings in The Netherlands is regulated by national legislation through the Dutch Building Decree. The structural reliability of existing buildings should only be considered in case of renovation, or on notification by the government.

NL1.2 Definition of consequence classes.

consequence class	description	examples of buildings and civil engineering works
CC3	• *high* consequences for loss of human life, and/or • *very high* economic, social or environmental consequences	• high-rise buildings (h > 70 m) • grandstands • exhibition halls • concert halls • large public buildings
CC2	• *medium* consequences for loss of human life, or • *considerable* economic, social or environmental consequences	• residential buildings • office buildings • public buildings • industrial buildings (three or more floors)
CC1	• *low* consequences for loss of human life, and/or • *small or negligible* economic, social or environmental consequences	• agricultural buildings • greenhouses • standard one-family houses • industrial buildings (one or two floors)

• *Renovation*. Renovation of existing structures should be carried out according to the Dutch Building Decree and NEN 8700 such that the end result meets the requirements for new structures. According to the Dutch Building Decree, the municipality may deviate from this – and accept a lower level of structural reliability – based on the following three considerations:
– adjustment of the reliability level of an existing building to the reliability level of newly built buildings usually costs more than for newly built buildings;
– the expected remaining working life of an existing building is usually shorter than the design working life (reference period) of a similar newly built building;
– the properties (such as dimensions, materials and resistance) of an existing structure can be determined more or less accurately through measurements.

When assessing the actual structural reliability of an existing structure, the dimensions and material properties specified in documents, such as drawings and specifications used to obtain the building permit, may be used. However, corrosion, which may cause significant loss of cross-sectional area, should be taken into account. When documentation is not available measurements are required to provide a good insight into the existing situation.
• *Notification*. From the day of completion of a newly built structure, the structural reliability of the structure decreases due to degradation by, for example, erosion or corrosion. This degradation can be delayed by sufficiently frequent maintenance of the structure. When appropriate maintenance is not carried out the structural reliability can decrease to such an extent that an unsafe situation occurs. The responsible municipality can interfere by a so-called notification. The actual reliability of an existing structure shall not be lower than the so-called notification level. The notification states that the minimum level of structural reliability is not reached, and that the owner should immediately change the use of the structure concerned. This can be achieved for example by a significant reduction of the actions on the structure, or by immediately ending or changing the activities in the building.

• *NEN 8700* (Beoordeling van de constructieve veiligheid van een bestaand gebouw bij verbouw en afkeuren. Grondslagen), 2015.
English: Assessment of existing structures in case of reconstruction and disapproval. Basic rules.

p. 1-17 ***EN 1990, cl. 2.3(1), table 2.1***

Only four design working life classes for buildings are provided, rather than the five classes provided by EN 1990 (see table NL1.3).

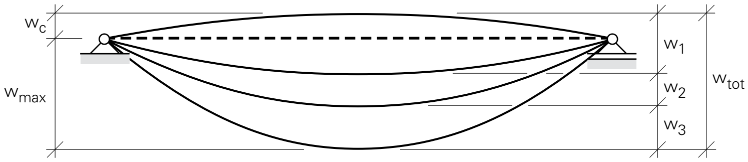
p. 1-22 ***EN 1990, Annexes***

More detailed definitions for the vertical deflections are given in annex A1 (see fig. NL1.4).
Annex B is partially normative (only parts B1 to B3) and annex C only in special cases. Also annex D is normative under the condition that any alternative clauses in the material specific Eurocodes prevail.

NL1.3 Design working life for buildings.

category	design working life (years)	examples
1	5	temporary structures
2	15	agricultural structures and industrial buildings with one or two floors
3	50	buildings and other common structures
4	100	monumental building

NL1.4 Definitions of vertical deflections for a simply supported beam.



w_c precamber in the unloaded structural element

w_1 initial part of the deflection under permanent loads of the relevant combination of actions according to expressions (1.16) to (1.21) determined with short term properties

w_2 long term part of the deflection under permanent loads of the quasi-permanent combinations of actions of expressions (1.20) and (1.21), or: the deflections determined with long term properties minus the deflection determined with short term properties

w_3 additional part of the deflection due to the variable actions of the relevant combinations of actions to expressions (1.16) to (1.21) determined with short term properties

w_{tot} total deflection ($w_{tot} = w_1 + w_2 + w_3$)

w_{max} remaining total deflection taking into account the precamber

Annex NL 2

Actions and deformations

p. 2-6 ***EN 1993-1-1, cl. 6.1(1)***

The recommended values for the partial factors for resistance are used.

p. 2-8 (a) ***EN 1990, Annex A1***

The recommended values for the partial factors for actions of annex A1 are used or alternative values are provided when describing the combinations of actions.

p. 2-8 (b+c) + p. 2-9 ***EN 1990, cl. 6.4.3.2(3)***

Equations (6.10a) and (6.10b) shall be used and equation (6.10) may not be used. This, with a modified factor ξ = 0,89, results in the partial factors of table NL2.1.

p. 2-10 ***EN 1990, cl. 6.4.3.2(3) + cl. A1.1(1)***

The design working life is referred to as the reference period.

During construction, the design working life should be at least equal to the construction time, with a minimum of one year. For a completed building, the design working life is at least the planned period of use. When nothing different is agreed with the client, the design working life should be applied according to table NL2.2.

limit state	design situation or combinations of actions		γ_G unfavourable	γ_G favourable	γ_Q	γ_A
ultimate limit state EQU (set A)	persistent or transient design situation (fundamental combinations)		1,1	0,9	1,5	–
ultimate limit state STR/GEO (set B)	persistent or transient design situation (fundamental combinations)	RC3	1,5 1,3	0,9 0,9	1,65 1,65	– –
		RC2	1,35 1,2	0,9 0,9	1,5 1,5	– –
		RC1	1,2 1,1	0,9 0,9	1,35 1,35	– –
ultimate limit state STR/GEO (set C)	persistent or transient design situation (fundamental combinations)		1,0	1,0	1,3	–
ultimate limit state	accidental design situation		1,0	1,0	1,0	1,0
ultimate limit state	seismic design situation		1,0	1,0	1,0	1,0
serviceability limit state	characteristic, frequent, quasi-permanent combinations		1,0	1,0	1,0	–

NL2.1 Partial factors for actions for several limit states and design situations.

Note that any agreed design working life should not be shorter than the relevant value given in table NL2.2.

For a design working life different from 50 years and for actions other than snow, wind and thermal actions, for example for imposed actions of floors, the characteristic action may, in accordance with the Dutch National Annex to EN 1990, cl. A1.1(2), be adapted as follows:

$$F_t = F_{t0}\left(1 + \frac{1-\psi_0}{9}\ln\frac{t}{t_0}\right) \qquad \text{(NL2.1)}$$

Where:

F_t adapted characteristic value of the variable action for the chosen design working life;

F_{t0} characteristic value of the variable action for a design working life of fifty years;

ψ_0 factor, see *Structural basics 1* (Structural safety), table 1.10;

t chosen design working life;

t_0 design working life of fifty years.

NL2.2 Design working life for buildings.

category	design working life (years)	examples
1	5	temporary structures
2	15	agricultural structures and industrial buildings with one or two floors
3	50	buildings and other common structures
4	100	monumental buildings

Example NL2.1

- *Given:* A steel structure for a retail space for which the client requires a design working life of one-hundred years. The characteristic imposed load on the floor according to EN 1991-1 is $q_k = 4$ kN/m^2, based on a design working life of fifty years.
- *Question:* Determine the adapted imposed load on the floor for a design working life of one-hundred years.
- *Answer:* The multiplication factor depends on the value for ψ_0. For a retail space $\psi_0 = 0{,}4$. The adapted imposed load on the floor follows from equation (NL2.1), where F is replaced by q_k:

$$q_{k,t} = q_{k,t0}\left(1 + \frac{1-\psi_0}{9}\ln\frac{t}{t_0}\right) = 4\cdot\left(1 + \frac{1-0{,}4}{9}\cdot\ln\frac{100}{50}\right) = 4\cdot 1{,}046 = 4{,}2\ \text{kN/m}^2$$

NL2.3 Imposed load.

category of use	description	q_k (kN/m^2)	Q_k (kN)
A	*areas for domestic and residential activities*		
	– floors	1,75	3
	– stairs	2,0	3
	– balconies	2,5	3
B	*office areas*	2,5	3
C	*areas where people may congregate*		
C1	– tables	4,0	7
C2	– fixed seats	4,0	7
C3	– without obstacles for moving people	5,0	7
C4	– physical activities	5,0	7
C5	– large crowds	5,0	7
D	*shopping areas*		
D1	– general retail shops	4,0	7
D2	– department stores	4,0	7
E	*areas for accumulation of goods or industrial use*		
E1	– shops	≥ 5	≥ 7
E1	– libraries	≥ 2,5	≥ 3
E1	– other	≥ 5	≥ 10
E2	– industrial use	≥ 3	≥ 7
	garages and vehicle traffic areas		
F	– light vehicles, lighter than 25 kN	2	10
G	– medium weight vehicles: 25 kN to 120 kN	5	40
	roofs		
H	– not accessible (α = roof slope):		
	$0 \leq \alpha \leq 15^\circ$	1,0	1,5
	$15^\circ \leq \alpha < 20^\circ$	$4 - 0{,}2\alpha$	1,5
	$\alpha \geq 20^\circ$	0	1,5
–	– for areas below ground level (no traffic load)	4	7

NL2.4 Imposed load for access routes of areas.

category of use	description	q_k (kN/m^2)	Q_k (kN)
A	*areas for domestic and residential activities*	2	3
B	*office areas*	3	3
C	*areas where people may congregate*	5	7
D	*shopping areas*	4	4
E	*areas for accumulation of goods or industrial use*		
E1	– libraries	3	3
E1	– other	4	4
E2	– industrial use	4	4

p. 2-14 (a) ***EN 1991-1-1, cl. 6.3.1.2(10)***

The reduction factor α_A for area may not be applied.

p. 2-14 (b) ***EN 1991-1-1, cl. 6.3.1.2(11)***

The reduction factor α_n is applicable in The Netherlands.

p. 2-14 (c) ***EN 1991-1-1, cl. 6.3.1.2***

Characteristic values for uniformly distributed loads q_k and concentrated loads Q_k per category of use are specified in the Dutch National Annex. A (not complete) summary is shown in table NL2.3.

According to the Dutch National Annex to EN 1991-1-1, cl. 6.2.2.2, for libraries the imposed load q_k from table NL2.3 should only be applied to the area between the bookshelves. The average load which should be taken into account can be determined with:

$$q_k = \frac{A_1 \gamma_{bk} h + A_2 p_0}{A_1 + A_2} \qquad \text{(NL2.2)}$$

Where:

A_1 area in m^2 on which the shelves are located;
A_2 residual area in m^2;
γ_{bk} self-weight of the books ($\gamma_{bk} = 6\ kN/m^3$);
p_0 load for the area between the shelves ($p_0 = 2{,}5\ kN/m^2$);
h height of the shelves.

Non-Complementary Information

The Dutch National Annex specifies values for access routes to areas, depending on the category of use (table NL2.4). For access routes in houses, the imposed load Q_k should be applied over an area of maximum $0{,}5 \times 0{,}5\ m^2$.

Example NL2.2

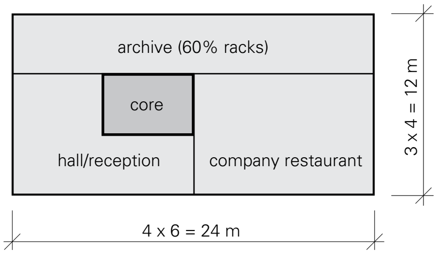
- *Given:* An office building with twenty storeys. At the ground floor an entry is combined with the archive and the company restaurant (fig. NL2.5). The height of each storey is 3,5 m and the free height is 2,8 m. The stability of the building is provided by a core.
- *Question:* What is the imposed load on the ground floor around the core?
- *Answer:* The imposed load on the ground floor depends on the use of the different areas.

The hall with the reception is an access route to an office (category B), and so: q_k = 3 kN/m^3 and Q_k = 3 kN (table NL2.4). A company restaurant in an office is an area where people may congregate. There are tables, so this area belongs to category C1: q_k = 4 kN/m^2 and Q_k = 7 kN (table NL2.3). The archive is meant for storage of binders, books and other documents in racks (category E1). This area is comparable with a library. The average uniformly distributed load which should be taken into account, according to equation (NL2.2), is:

$$q_k = \frac{A_1\gamma_{bk}h + A_2p_0}{A_1 + A_2} = \frac{(0,6\cdot4\cdot24)\cdot6\cdot2,8 + (0,4\cdot4\cdot24)\cdot2,5}{(0,6\cdot4\cdot24) + (0,4\cdot4\cdot24)} = \frac{968 + 96,0}{96,0} = 11,1 \text{ kN/m}^2$$

For the concentrated load in the archive $Q_k \geq 3$ kN (table NL2.4).

NL2.5 Functional plan of the ground floor of an office building.



p. 2-16 (a) *EN 1991-1-3, cl. 5.2*

$C_e = C_t$ = 1,0 and s_k = 0,7 kN/m^2 should be used. This means the snow load which should be taken into account may be simplified to:

$$s = 0{,}7\mu_i \text{ kN/m}^2 \qquad \text{(NL2.3)}$$

The accompanying values of the snow load are set to $\psi_i\cdot\sigma_k$, with ψ_0 = 0 for the combination value, ψ_1 = 0,2 for the frequent value and ψ_2 = 0 for the quasi-permanent value. This results in the representative values of the snow load for simultaneous occurrence with other variable actions.

p. 2-16 (b) *EN 1991-1-3, cl. 4.3*

Exceptional snow loads on the ground need not be taken into account.

p. 2-16 (c) *EN 1991-1-3, cl. 3.3 + 6*

The snow load arrangement according to figure 2.18b does not need to be considered.

p. 2-17 *EN 1991-1-4, cl. 4.2(1)*

The Netherlands is divided into three wind areas (I, II and III) each with its own fundamental value of the basic wind velocity $v_{b,0}$ (fig. NL2.6).

NL2.6 The three wind areas in the Netherlands and the related fundamental values of the basic wind velocity $v_{b,0}$.

Wind area I: in the province of Noord-Holland all municipalities north of the municipalities of Heemskerk, Uitgeest, Wormerland, Purmerend and Edam-Volendam.

NL2.8 Peak velocity pressure $q_p(z)$ in kN/m^2 for $c_0(z) = 1$.

height (m)	wind area I			wind area II			wind area III	
	coastal	non-urbanized	urbanized	coastal	non-urbanized	urbanized	non-urbanized	urbanized
1	0,93	0,71	0,69	0,78	0,60	0,58	0,49	0,48
2	1,11	0,71	0,69	0,93	0,60	0,58	0,49	0,48
3	1,22	0,71	0,69	1,02	0,60	0,58	0,49	0,48
4	1,30	0,71	0,69	1,09	0,60	0,58	0,49	0,48
5	1,37	0,78	0,69	1,14	0,66	0,58	0,54	0,48
6	1,42	0,84	0,69	1,19	0,71	0,58	0,58	0,48
7	1,47	0,89	0,69	1,23	0,75	0,58	0,62	0,48
8	1,51	0,94	0,73	1,26	0,79	0,62	0,65	0,51
9	1,55	0,98	0,77	1,29	0,82	0,65	0,68	0,53
10	1,58	1,02	0,81	1,32	0,85	0,68	0,70	0,56
15	1,71	1,16	0,96	1,43	0,98	0,80	0,80	0,66
20	1,80	1,27	1,07	1,51	1,07	0,90	0,88	0,74
25	1,88	1,36	1,16	1,57	1,14	0,97	0,94	0,80
30	1,94	1,43	1,23	1,63	1,20	1,03	0,99	0,85
35	2,00	1,50	1,30	1,67	1,25	1,09	1,03	0,89
40	2,04	1,55	1,35	1,71	1,30	1,13	1,07	0,93
45	2,09	1,60	1,40	1,75	1,34	1,17	1,11	0,97
50	2,12	1,65	1,45	1,78	1,38	1,21	1,14	1,00
55	2,16	1,69	1,49	1,81	1,42	1,25	1,17	1,03
60	2,19	1,73	1,53	1,83	1,45	1,28	1,19	1,05
65	2,22	1,76	1,57	1,86	1,48	1,31	1,22	1,08
70	2,25	1,80	1,60	1,88	1,50	1,34	1,24	1,10
75	2,27	1,83	1,63	1,90	1,53	1,37	1,26	1,13
80	2,30	1,86	1,66	1,92	1,55	1,39	1,28	1,15
85	2,32	1,88	1,69	1,94	1,58	1,42	1,30	1,17
90	2,34	1,91	1,72	1,96	1,60	1,44	1,32	1,18
95	2,36	1,93	1,74	1,98	1,62	1,46	1,33	1,20
100	2,38	1,96	1,77	1,99	1,64	1,48	1,35	1,22
110	2,42	2,00	1,81	2,03	1,68	1,52	1,38	1,25
120	2,45	2,04	1,85	2,05	1,71	1,55	1,41	1,28
130	2,48	2,08	1,89	2,08	1,74	1,59	1,44	1,31
140	2,51	2,12	1,93	2,10	1,77	1,62	1,46	1,33
150	2,54	2,15	1,96	2,13	1,80	1,65	1,48	1,35
160	2,56	2,18	2,00	2,15	1,83	1,67	1,50	1,38
170	2,59	2,21	2,03	2,17	1,85	1,70	1,52	1,40
180	2,61	2,24	2,06	2,19	1,88	1,72	1,54	1,42
190	2,63	2,27	2,08	2,20	1,90	1,75	1,56	1,44
200	2,65	2,29	2,11	2,22	1,92	1,77	1,58	1,46

p. 2-18 ***EN 1991-1-4, cl. 4.2(2)***

The directional factor c_{dir} = 1 is used. The seasonal factor c_{season} = 1 is used.

p. 2-19 (a) ***EN 1991-1-4, cl. 4.3.2***

Three terrain categories are defined (table NL2.7)

p. 2-19 (b) ***EN 1991-1-4, cl. 4.3.3***

The procedure to determine $c_o(z)$ according to EN 1991-1-4, Annex A.3 is confirmued.

p. 2-19 (c) ***EN 1991-1-4, cl. 4.5***

The peak velocity pressure values are tabulated depending on the wind area, the terrain category and the height (table NL2.8).

p. 2-19 (d) ***EN 1991-1-4, cl. 4.4***

The recommended value for the turbulence factor is confirmed in the Dutch National Annex.

p. 2-19 (e) ***EN 1991-1-4, cl. 4.5***

The recommended values for the air density is confirmed in the Dutch National Annex.

p. 2-21 ***EN 1991-1-4, cl. 5.2(1) + 7.2.2***

External pressure coefficients c_{pe} for the different zones of vertical façades of buildings with a rectangular plan are shown in table NL2.9.

p. 2-22 ***EN 1991-1-4, cl. 5.3(5) + 7.2.2(3)***

Due to lack of correlation of wind pressures at the windward and leeward sides the resulting force due to wind pressures at the windward and leeward side of buildings may be multiplied by a factor 0,85.

p. 2-26 ***EN 1991-1-4, cl. 6.3.1***

The factors k_p, B^2 and R^2 in the structural factor c_sc_d should be determined according to EN 1991-1-4, annex C. Also c_sc_d = 0,85 should be taken if equation (2.25) yields a lower value than 0,85.
For the peak factor k_p, Annex C refers to EN 1991-1-4, annex B, cl. B2.(3).
The *background response factor* B^2 according to annex C is:

$$B^2 = \frac{1}{1 + \frac{3}{2}\sqrt{\left(\frac{b}{L(z_s)}\right)^2 + \left(\frac{h}{L(z_s)}\right)^2 + \left(\frac{b}{L(z_s)}\cdot\frac{h}{L(z_s)}\right)^2}} \qquad \text{(NL2.4)}$$

The *resonance response factor* R^2 according to Annex C is:

$$R^2 = \frac{\pi^2}{2\delta} S_L K_s \qquad \text{(NL2.5)}$$

NL2.7 Terrain categories and terrain parameters.

terrain category	z_0(m)	z_{min}(m)
0 sea or coastal area	0,005	1
II non-urbanized area	0,2	4
III urbanized area	0,5	7

NL2.9 External pressure coefficients c_{pe} for the different zones of vertical façades of buildings with a rectangular plan.

		h/d	
		≤ 1	5
zone A	$c_{pe,1}$	–1,4	–1,4
	$c_{pe,10}$	–1,2	–1,2
zone B	$c_{pe,1}$	–1,1	–1,1
	$c_{pe,10}$	–0,8	–0,8
zone C	$c_{pe,1}$	–0,5	–0,5
	$c_{pe,10}$	–0,5	–0,5
zone D	$c_{pe,1}$	+1,0	+1,0
	$c_{pe,10}$	+0,8	+0,8
zone E	$c_{pe,1}$	–0,5	–0,7
	$c_{pe,10}$	–0,5	–0,7

Where:

K_s size reduction function – in EN 1991-1-4, cl. C.2(5) referred to as $K_s(n_1)$ – as follows:

$$K_s = \frac{1}{1 + \sqrt{\left(G_y\phi_y\right)^2 + \left(G_z\phi_z\right)^2 + \left(\frac{2}{\pi}G_y\phi_y G_z\phi_z\right)^2}}$$

with (NL2.6)

$$\varphi_y = \frac{c_y b n_1}{v_m(z_s)}; \qquad \varphi_z = \frac{c_z h n_1}{v_m(z_s)}$$

Here, the decay constants c_y and c_z are both equal to 11,5; for buildings with a uniform horizontal mode shape variation and a linear vertical mode shape variation the constant G_y = ½ and the constant G_z = 3/8. Other values of these constants are tabulated in EN 1991-1-4, cl. C.2(6).

p. 2-29 (a) ***EN 1991-1-3, cl. 7***

The action on roofs due to rain water is discussed in the Dutch National Annex to EN 1991-1-3, cl. 7. For action due to rainwater, it is assumed that the ordinary drains are blocked but that the rain water can flow away over the edge of the roof or through emergency drains. The location and the number of emergency drains and the slope of the roof are important. The water level above the emergency drain d_{nd} (m) for a straight free overfall follows from the Dutch National Annex to EN 1991-1-3, cl. 7.2(5) (fig. NL2.10):

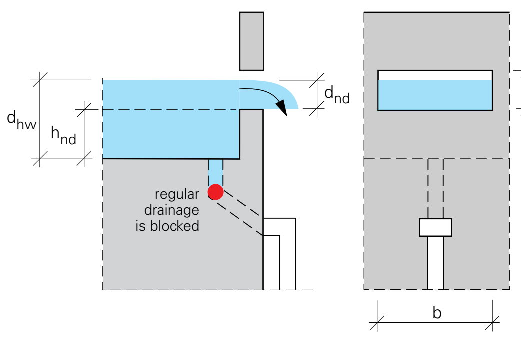
$$d_{nd} = 0{,}70\sqrt[3]{\left(\frac{Q_h}{b}\right)^2} \qquad \text{(NL2.7)}$$

Where:

Q_h flow rate through the emergency drain (m^3/s);

b width of the emergency drain (m).

NL2.10 Dimensions of an emergency drain for a flat roof with a straight free overfal.



Another equation applies for an emergency drain that takes the form of a circular central drain. The flow rate Q_h is determined by multiplying the area of the emergency drain A (m^2) by the rainfall intensity i_r (m/s). The rainfall intensity depends on the reference period; for a reference period of 50 years: $i_r = 0{,}05 \cdot 10^{-3}$ m/s.

The water level at the location of the roof edge or at the emergency drain d_{hw} (m) is, according to the Dutch National Annex to EN 1991-1-3, cl. 7.2(7):

$$d_{hw} = d_{nd} + h_{nd} \qquad \text{(NL2.8)}$$

Where h_{nd} (m) is the height of the emergency drain above the roof (fig. NL2.10).

The deflection due to ponding should, according to the Dutch National Annex to EN 1991-1-1, cl. 7.2(2), be determined iteratively or with analytically derived equations in which the stiffness of the structure is divided by γ_M = 1,3. According to the Dutch National Annex to EN 1991-1-3, cl. 7.2(2), the action due to rain water is then obtained as the sum of the water level above the undeformed roof and the water level due to deflection of the roof, multiplied by the density of water (10 kN/m^3) and by the partial factor for variable actions.

p. 2-29 (b) ***EN 1990, cl. A1.4.2***

The strictest criteria should be satisfied according to:

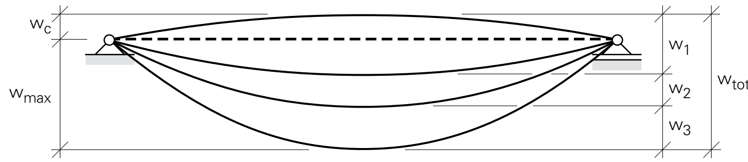
– Dutch National Annex to EN 1990, cl. A1.4.3 and A1.4.4; and
– EN 1992 to EN 1999.

EN 1993-1-1 does not provide serviceability limits in chapter 7 (which deals with serviceability limit states). Therefore, the criteria of the Dutch National Annex to EN 1990, cl. A1.4.3 and A1.4.4, should be used for steel buildings.

Limits for vertical deflections and horizontal displacements are given in the Dutch National Annex to EN 1990, cl. A1.4.3.
The Dutch National Annex to EN 1990, cl. A1.4.3(3) provides requirements for the sum of the deflections w_2 and w_3 and cl. A1.4.3(4) provides requirements for w_{max} (fig. NL2.11). For steel structures at ambient temperatures, the long-term part of the deflection under permanent load is zero: $w_2 = 0$, since shrinkage and creep are of no importance for steel structures. Requirements for w_{max} are only set when the appearance of the structure is important and could be compromised by deflections. The requirements are summarized in table NL2.12. Here, L_{rep} is the span length, or twice the length of a cantilever.

The Dutch National Annex to EN 1990, cl. A1.4.3(7) provides limits for horizontal displacements. The requirements are summarized in table NL2.13. Stricter limits may be necessary if, for example, an overhead crane track is present in a building or if the building has a glass façade.

NL2.11 Definitions of vertical deflections for a simply supported beam.



w_c precamber in the unloaded structural element
w_1 initial part of the deflection under permanent loads of the relevant combinations of actions according to expressions (1.16) to (1.21), see *Structural basics 1* (Structural safety)
w_2 long term part of the deflection under permanent loads
w_3 additional part of the deflection due to the variable actions of the relevant combinations of actions to expressions (1.16) to (1.21), see *Structural basics 1* (Structural safety)
w_{tot} total deflection ($w_{tot} = w_1 + w_2 + w_3$)
w_{max} remaining total deflection taking into account the precamber

NL2.12 Limits for deflections of roofs and floors.

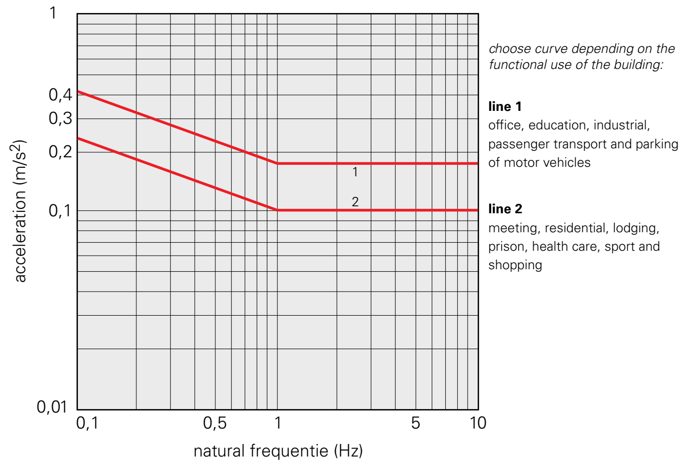
structural part	combination of actions	additional defection w_3	total deflection w_{max}
floors that carry partition walls which are sensitive to cracks	frequent	≤ 0,002L_{rep}	–
	quasi-permanent	–	≤ 0,004L_{rep}
other floors and roofs which are used intensively by people	frequent	≤ 0,003L_{rep}	–
	quasi-permanent	–	≤ 0,004L_{rep}
other roofs	characteristic	≤ 0,004L_{rep}	–
	quasi-permanent	–	≤ 0,004L_{rep}
floor parapets at a height difference	–	≤ 0,0067L_{rep}	–

NL2.13 Limits for horizontal displacements (for characteristic combinations of actions).

number of storeys	building (part)	limit of horizontal displacement
1	industrial building	u ≤ H/150
	other buildings	u ≤ H/300
2 or more	per storey	ui ≤ Hi/300
	whole building	u ≤ H/500
parapet at the location of a height difference	upper edge and baluster together	u ≤ 20 mm

floor type	natural frequency f_e
floors intensively used by people	≥ 3 Hz
floors on which people jump or dance	≥ 5 Hz

NL2.14 Requirements for the natural frequency fe of floors.



NL2.15 Requirements for acceleration due to wind.

p. 2-30 ***EN 1990, cl. A1.4.4***

The serviceability criteria for vibrations are specified in the Dutch National Annex to EN 1990, cl. A1.4.4. The requirements are summarized in table NL2.14.

Floors intensively used by people meet the requirement if the deflection for short-term behaviour and for the quasi-permanent combinations of actions is not more than about 34 mm. This limit does not have to be met if the sum of the characteristic values of the permanent load and ψ_2-times the imposed load is at least 5 kN/m^2 or – in cases where the floor is supported by beams – the total load is 150 kN per beam.

Floors on which people jump or dance meet the requirement if the deflection for short-term behaviour and for the quasi-permanent combinations of actions is not more than about 12 mm.

Wind vibrations in a building are inconvenient for people when they cause excessive accelerations. The Dutch National Annex to EN 1990, cl. A1.4.4(5) states, amongst other things, that for the building functions 'residential' and 'lodging' the acceleration should not be more than 0,25 m/s^2 at a natural frequency of 0,1 Hz, and not more than 0,1 m/s^2 at a natural frequency of 1 Hz. Linear interpolation may be applied for intermediate natural frequencies. For natural frequencies larger than 1 Hz, the acceleration should be limited to 0,1 m/s^2 (line 2 in fig. NL2.15). For, amongst other things, the building functions 'office' and 'education', less strict requirements are acceptable (line 1 in fig. NL2.15). The natural frequency can be determined using EN 1991-1-4, cl. F.2. A check for wind vibrations is not required for buildings with a height less than 20 m and a width not greater than the height.

p. 2-32 ***EN 1991-1-1, Annex A and B***

Annex A and B are informative.

p. 2-33 (a) ***EN 1991-1-3, cl. 3.2(1)***

Exceptional snowfall is not relevant. Only normal conditions occur for which the undrifted and drifted snow load arrangements should be applied according to the code.

p. 2-33 (b) ***EN 1991-1-3, cl. 4.1 + 4.2***

The snow load on the ground is s_k = 0,7 kN/m^2. The accompanying values are specified as $\psi \cdot \sigma_k$ with ψ_0 = 0 for the combination value, ψ_1 = 0,2 for the frequent value and ψ_2 = 0 for the quasi-permanent value.

p. 2-33 (c) ***EN 1991-1-3, cl. 4.3***

Exceptional snow loads on the ground are not relevant.

p. 2-34 (a) ***EN 1991-1-3, cl. 6.3***

Exceptional snowfall is not relevant.
Overhanging snow at the edge of a roof need not be taken into account.

p. 2-34 (b) ***EN 1991-1-3, cl. 7***

For The Netherlands, chapter 7 for actions due to rainwater was added. The action due to rainwater is a free action, and water ponding should be taken into account. The following principles are adopted:

- regular drains are blocked;
- the rain intensity is $0{,}05 \cdot 10^{-3}$ m/s for a reference period of fifty years (for other reference periods, other rain intensities are valid);
- the water is disposed of by flowing over the roof edge or through emergency drains.

The magnitude of the rainwater action is determined based on the water level above the undeformed roof, and the water level due to deflection of the roof resulting from (and causing) ponding. Due to deflection of the structure the water level increases, which leads to larger deflections under the increased load. This iterative effect should be taken into account when determining the magnitude of the rain water action. The deflection should be determined with the action due to the water level above the undeformed roof as the initial action, and the stiffness should be divided by γ_M = 1,3. The Dutch National Annex to EN 1991-1-3 also provides rules for detailing the emergency drains.

p. 2-34 (c) ***EN 1991-1-3, Annexes***

Only case A (normal snow conditions) of Annex A should be applied.
Annex B should not be applied.
The Annexes C, D and E are kept informative.

p. 2-34 (d) ***EN 1991-1-4, 1.5(2)***

For design assisted by testing and measurements, the Dutch National Annex specifies that CUR

Recommendation 103 (Windtunnelonderzoek voor de bepaling van ontwerp-windbelastingen op (hoge) gebouwen en onderdelen ervan) (In English: Wind tunnel research for determining design wind actions on (high-rise) buildings and their components) may be consulted for the determination of wind actions on buildings through wind tunnel investigations.

p. 2-35 ***EN 1991-1-4, cl. 4.5***

Peak velocity pressures are provided in table format, depending on the wind area, the terrain category and the height (see table NL2.8).

p. 2-37 ***EN 1991-1-4, Annexes***

- Annex A, cl. A.1 on illustrations of the upper roughness per terrain category is informative. The clauses A.2 to A.4 are made normative. The clause A.5 on displacement height is not applicable.
- Annex B containing a procedure for determining the structural factor is not applicable.
- Annex C, which provides an alternative procedure for determining the structural factor, is made normative.
- Annex D containing structural factors for different types of structures is kept informative.
- Annex E on vortex shedding and aeroelastic instabilities is kept informative.
- Annex F on the dynamic characteristics of structures is kept informative.

p. 2-44 ***EN 1991-1-4, cl. 4.5***

See remarks to p. 2-19 (c).

p. 2-45 ***EN 1991-1-4, cl. 5.2***

See remarks to p. 2-16 (a).

p. 2-46 (a) ***EN 1991-1-4, cl. 4.2(1)***

See remarks to p. 2-19 (c).

p. 2-46 (b) ***EN 1991-1-4, cl. 4.2(2)***

See remarks to p. 2-18.

p. 2-49 (a) ***EN 1991-1-4, cl. 4.3.3(1) + A3 + 4.4***

The recommended procedure for determining the orography factor according to Annex A3 is followed. The recommended value for the turbulence factor is confirmed as 1,0.

p. 2-49 (b) ***EN 1991-1-4, cl. 4.2***

The recommended values for the directional factor and the seasonal factor are confirmed to be 1,0.

p. 2-51 ***EN 1991-1-4, cl. 5.3(5) + 7.2.2(3)***

See remarks to p. 2-22.

Worked examples for The Netherlands

The three worked examples in section 2.7 are elaborated here, specifically for the Dutch situation.

2.7.1 Actions on a floor in a residential house

- *Given:* A floor in a detached house consists of timber beams with floor panels and a steel IPE 270 beam (see fig. 2.35). A non-load bearing (moveable) internal wall of cellular concrete blocks with a thickness d = 70 mm and a density of γ = 8 kN/m^3 is located on the floor. The floor height is h = 2,4 m.
- *Question:* Determine the actions on the timber beams and on the steel beam at the ultimate limit state, and the corresponding force distribution.
- *Answer:* A detached house (single-family dwelling) belongs in consequence class CC1 and reliability class RC1, see *Structural basics 1* (Structural safety), tables 1.11 and 1.12. The actions on the floor consist of a permanent load part (self-weight g_k) and a variable action part (imposed load q_k).

Permanent load (g_k)

For the timber floor including beams, the self-weight is 0,30 kN/m^2 (see table 2.9). The steel IPE 270 beam has a self-weight of 0,36 kN/m (such information can be obtained from books of tables). For the non-load bearing internal wall, the self-weight per meter is: $g_k = dh\gamma = 0{,}07 \cdot 2{,}4 \cdot 8 = 1{,}34$ kN/m. This self-weight is larger than 1,0 kN/m and smaller than 2,0 kN/m. The internal wall may be taken into account as a uniform distributed load q_k = 0,8 kN/m^2 (see table 2.13), which should be included in the imposed load. The action due to the non-load bearing wall can then be regarded as a variable free action.
The timber beams each support an average floor area with a width of 0,75 m. The actions due to the timber beams on the steel beam are taken as a uniformly distributed load for reasons of simplicity. The steel beam supports a floor area with a width of 0,5·(3,0 + 4,0) = 3,5 m. The permanent load on the beams is therefore:

timber beam:	$g_k = 0{,}30 \cdot 0{,}75 \quad = 0{,}23$ kN/m
steel beam:	$g_k = 0{,}36 + 0{,}30 \cdot 3{,}5 = 1{,}41$ kN/m

Imposed load (q_k)

The imposed load which should be taken into account is an imposed floor load. A room in a residential house belongs to category of use A (areas for domestic and residential activities) according to EN 1991-1-1, table 6.1. The imposed load on a floor of category A consists of a uniformly distributed load q_k = 1,75 kN/m^2 and a concentrated load Q_k = 3 kN (see table NL2.3). The load due to the self-weight of the wall should be included in the uniformly distributed load: q_k = 1,75 + 0,8 = 2,55 kN/m^2. Which of the two imposed loads is critical for bending in the *timber beams* can be determined based on the bending moments:

uniformly distributed load: $M_{q,k} = \frac{1}{8} q_k L^2 = \frac{1}{8} \cdot (2,55 \cdot 0,75) \cdot 4,0^2 = 3,8 \text{ kNm}$ (critical)

concentrated load: $M_{Q,k} = \frac{1}{4} Q_k L = \frac{1}{4} \cdot 3 \cdot 4,0 = 3,0 \text{ kNm}$

The uniformly distributed load is critical in determining the maximum bending moment.
The shear force in a timber beam is at a maximum when the concentrated load $Q_k = 3$ kN is located near the support of the beam. Which of the two imposed loads is critical for shear in the *timber beams* can be determined based on the shear forces:

uniformly distributed load: $V_{q,k} = \frac{1}{2} q_k L = \frac{1}{2} \cdot (2,55 \cdot 0,75) \cdot 4,0 = 3,8 \text{ kN}$ (critical)

concentrated load: $V_{Q,k} = Q_k = 3,0 \text{ kN}$

The uniformly distributed load is also critical in determining the maximum shear force.

For the *steel beam*, the uniformly distributed load is critical because this load is applied to the whole floor area, compared to the concentrated load which only acts on an area of $0,05 \text{x} 0,05 \text{m}^2$. Therefore, for the steel beam: $q_k = 2,55 \cdot 3,5 = 8,92$ kN/m.

Combinations of actions

The governing moment in the timber beams at the ultimate limit state occurs for a combination of permanent load and variable action: in this case the self-weight and the imposed load. A detached house is a single-family dwelling and belongs to consequence class CC1 with a corresponding reliability class RC1, see *Structural basics 1*, table 1.11 and 1.12. The combinations of actions for persistent or transient design situations (fundamental combinations, set B) are for reliability class RC1 (see table NL2.1 and *Structural basics 1*, equation (1.26) and (1.27)):

$$1,2G + \sum_{i \geq 1} 1,35 \psi_{0,i} Q_{k,i} = 1,2 g_k + 1,35 \psi_0 q_k$$

$$1,1G + 1,35 Q_{k,1} + \sum_{i > 1} 1,35 \psi_{0,i} Q_{k,i} = 1,1 g_k + 1,35 q_k$$

The ψ factor in these equations for a residential house is: $\psi_0 = 0,4$; see *Structural basics 1*, table 1.10. For the critical moment, the following combinations of actions should be considered:

$$M_{Ed} = \frac{1}{8}\left(1,2 g_k\right) L^2 + \frac{1}{8}\left(1,35 \psi_0 q_k\right) L^2$$
$$= \frac{1}{8} \cdot (1,2 \cdot 0,23) \cdot 4,0^2 + \frac{1}{8} \cdot (1,35 \cdot 0,4 \cdot 2,55 \cdot 0,75) \cdot 4,0^2 = 2,6 \text{ kNm}$$

$$M_{Ed} = \frac{1}{8}\left(1,1 g_k\right) L^2 + \frac{1}{8}\left(1,35 q_k\right) L^2$$
$$= \frac{1}{8} \cdot (1,1 \cdot 0,23) \cdot 4,0^2 + \frac{1}{8} \cdot (1,35 \cdot 2,55 \cdot 0,75) \cdot 4,0^2 = 5,7 \text{ kNm} \quad \text{(critical)}$$

For the critical shear force, the following combinations of actions should be considered:

$$V_{Ed} = \frac{1}{2}\left(1{,}2g_k\right)L + \frac{1}{2}\left(1{,}35\psi_0 q_k\right)L$$
$$= \frac{1}{2}\cdot\left(1{,}2\cdot 0{,}23\right)\cdot 4{,}0 + \frac{1}{2}\cdot\left(1{,}35\cdot 0{,}4\cdot 2{,}55\cdot 0{,}75\right)\cdot 4{,}0 = 2{,}6 \text{ kN}$$

$$V_{Ed} = \frac{1}{2}\left(1{,}1g_k\right)L + \frac{1}{2}\left(1{,}35q_k\right)L$$
$$= \frac{1}{2}\cdot\left(1{,}1\cdot 0{,}23\right)\cdot 4{,}0 + \frac{1}{2}\cdot\left(1{,}35\cdot 2{,}55\cdot 0{,}75\right)\cdot 4{,}0 = 5{,}7 \text{ kN} \quad \text{(critical)}$$

For the *steel beam*, the critical combination of actions at the ultimate limit state is:

$$q_{Ed} = 1{,}2g_k + 1{,}35\psi_0 q_k = 1{,}2\cdot 1{,}41 + 1{,}35\cdot 0{,}4\cdot 8{,}92 = 6{,}5 \text{ kN/m}$$

$$q_{Ed} = 1{,}1g_k + 1{,}35q_k = 1{,}1\cdot 1{,}41 + 1{,}35\cdot 8{,}92 = 13{,}6 \text{ kN/m} \quad \text{(critical)}$$

The bending moment is then:

$$M_{Ed} = \frac{1}{8}q_{Ed}L^2 = \frac{1}{8}\cdot 13{,}5\cdot 5{,}25^2 = 46{,}5 \text{ kNm}$$

And the shear force is:

$$V_{Ed} = \frac{1}{2}q_{Ed}L = \frac{1}{2}\cdot 13{,}5\cdot 5{,}25 = 35{,}4 \text{ kN}$$

2.7.2 Actions on a free-standing platform canopy at a station

- *Given:* A free-standing canopy for a bus station in Den Bosch, The Netherlands, having six (unbraced) frames at a centre-to-centre spacing b = 6,5 m in a non-urbanized area. The roof consists of profiled steel sheeting with a 300 mm wide gutter in the centre. The gutter height is 200 mm and the top of the gutter is located at the same height as the roof cladding. A 1,2 m high billboard, which extends 0,3 m above the roof, is located on both sides of the roof. The area between the columns is closed with a glass wall (see fig. 2.36).
- *Question:* What is the governing load on the canopy structure?
- *Answer:* A canopy for a bus station can be regarded as a (part of a) public building and belongs to consequence class CC2 and reliability class RC2, see *Structural basics 1* (Structural safety), table 1.11 and 1.12. A classification in consequence class CC1 would not be correct because during a storm, or other unfavourable weather conditions, the canopy functions as a shelter to persons. A classification in class CC3 would be too strict regarding the open nature of the structure.

The partial factors for actions at the ultimate limit state for reliability class RC2 for persistent or transient design situations (fundamental combinations, set B) can be determined based on table NL2.1:

γ_G = 1,35 (unfavourable), γ_G = 0,9 (favourable) and γ_Q = 1,5
γ_G = 1,20 (unfavourable), γ_G = 0,9 (favourable) and γ_Q = 1,5

The reference period for buildings and regular structures is 50 year (see table NL2.2). The combinations of actions on the canopy include the following six actions:
- permanent load (self-weight);
- imposed load (variable action on the roof);
- wind action;
- action due to rain water;
- snow load;
- accidental actions.

Permanent load (self-weight)
Self-weights must be assumed since the dimensions of the structure and the profiled steel sheeting have not yet been determined. The self-weight of the profiled steel sheeting (including gutter), purlins, and frames of the canopy structure and of the bracings is assumed to be 0,15 kN/m^2, and the self-weight of the billboards along the roof edges is assumed to be 0,10 kN/m.

Imposed load (variable action)
The roof of the canopy is only accessible for maintenance and repair and belongs to category of use H. Table NL2.3 provides values for the uniformly distributed and concentrated loads, depending on the slope. For a slope α = 6,5°, q_k = 1,0 kN/m^2 (with ψ_0 = 0; see *Structural basics 1*, table 1.10) over a maximum area of 10 m^2. For a purlin, with a centre-to-centre spacing of 2,2 m, this leads to a loaded roof area of 2,2x4,55 = 10 m^2. For the trusses, with a centre-to-centre spacing of 6,5 m, a loaded roof area along the roof edge of 6,5x1,54 = 10 m^2 is critical (see fig. 2.37). The concentrated load which shall be taken into account is Q_k = 1,5 kN, acting on an area of 0,1x0,1 m; this load is, in this case, less critical than the uniformly distributed load.

Wind action
The wind action on a structure or structural element may be determined directly using equation (2.22). The wind action may also be determined by vector summation of the wind actions for external pressure, internal pressure and friction according to equation (2.24). For some structural elements (canopy roofs, billboards), the method with force coefficients applies (direct method), and for some other structural elements (glass wall) the method with pressure coefficients should be applied (vector summation). In this example, the wind action is determined as far as possible using the method of vector summation, and the direct method is used to determine an equivalent force coefficient.
The height of the canopy (including billboard) is 5,25 + 1,20 = 6,45 m. This value is smaller than 15 m, which means that the structural factor equals c_sc_d = 1,0.
The force coefficient c_f should be determined per surface exposed to wind according to EN 1991-1-4, cl. 7.1.1(4). Based on figure 2.22, for the glass wall with a height/depth ratio h/d =

6,45/13,2 = 0,5, it can be determined that $c_{pe} = c_{pe,10} = 0{,}8$ at the windward side (zone D) and $c_{pe} = c_{pe,10} = -0{,}5$ at the leeward side (zone E). This leads to a total force coefficient $c_f = 0{,}8 + 0{,}5 = 1{,}3$. Due to lack of correlation between the wind pressures at the windward and leeward sides, the total force coefficient may be multiplied by 0,85, so: $c_f = 1{,}3 \cdot 0{,}85 = 1{,}11$ (see the Dutch National Annex to EN 199191-4, cl. 5.3(5) and 7.2.2(3)). It should be noted that in this example, for determination of the wind action, the depth of the glass wall corresponds with the total width of the canopy.
For the roof of the canopy, EN 1991-1-4, cl. 7.3 (canopy roofs) applies. There is a full degree of blockage (full solidity): $\varphi = 1$. EN 1991-1-4, table 7.7 and figure 7.17 should be applied for determining c_f. The slope α is 6,5°. As conservative value $\alpha = -10°$ is used in this example. The maximum value of the force coefficient is then, according to EN 1991-1-4, table 7.7, for $\varphi = 1$: $c_f = +0{,}4$ and the minimum value $c_f = -1{,}4$. A distinction is made between the following six load cases in figure 2.38:

a. downward load, complete roof loaded: $c_f = 0{,}4$;
b. downward load, left half of the roof loaded: $c_f = 0{,}4$;
c. downward load, right half of the roof loaded: $c_f = 0{,}4$;
d. upward load, complete roof loaded: $c_f = 1{,}4$;
e. upward load, left half of the roof loaded: $c_f = 1{,}4$;
f. upward load, right half of the roof loaded: $c_f = 1{,}4$.

For wind from the left, case e is critical for the canopy, and obviously therefore for wind from the right it is case f.

For the billboards along the roof edges, EN 1991-1-4, cl. 7.4.3 provides the force coefficients. The distance from the bottom of the billboards to the ground is greater than h/4, where h is the height of the billboard: 5,25 m > 1,2/4 = 0,3 m. For this case: $c_f = 1{,}80$ shall be applied.
For wind friction, a very rough surface (ripples, ribs, folds) is assumed due to the presence of the gutter and profiled steel sheeting. The friction coefficient, according to table 2.29, is then $c_f = 0{,}04$.
Finally, the peak velocity pressure $q_p(z_e)$ at reference height z_e shall be determined. The reference height is $z_e = 6{,}45$ m. The peak velocity pressure can be determined using table NL2.8. For wind area III (Den Bosch, The Netherlands), non-urbanized, the peak velocity pressure is: $q_p(z_e = 6{,}45) = 0{,}60$ kN/m^2.
The wind action then finally results in the loading arrangement of figure 2.39. For the components of the wind action, similar to equations (2.22) and (2.24), the following equation applies:

$$q_{w,k} = c_s c_d \cdot (\chi_f \text{ or } c_{fr}) \cdot q_p(z_e) \cdot b = 1{,}0 \cdot (c_f \text{ or } c_{fr}) \cdot 0{,}60 \cdot 6{,}5 = 3{,}90 \cdot (c_f \text{ or } c_{fr})$$

For the four wind action components the following values can be obtained:

compression + suction on the glass wall:	$q_{1,w,k} = 3{,}90 \cdot 1{,}11 = 4{,}33$ kN/m
upward compression on the left half:	$q_{2,w,k} = 3{,}90 \cdot 1{,}4 = 5{,}46$ kN/m
compression and suction on the billboards:	$q_{3,w,k} = 3{,}90 \cdot 1{,}8 = 7{,}02$ kN/m
friction along the roof surface:	$q_{4,w,k} = 3{,}90 \cdot 0{,}04 = 0{,}16$ kN/m

Action due to rainwater

Action due to rainwater occurs when the regular drains of the gutter are blocked. The gutter then fills up until the water flows over its edges. The action due to rainwater on one frame is then (see fig. 2.40):

$$F_{water,k} = A_{gutter}\gamma_{water}b = 0{,}3 \cdot 0{,}2 \cdot 10 \cdot 6{,}5 = 3{,}9 \text{ kN}$$

Ponding is not taken into account in the equation above. Ponding is the phenomenon that, due to the deflection of the roof, additional water flows onto the roof and because this extra weight causes a larger deflection yet more water must be carried, see section 2.4.4. For this canopy, there is no risk for ponding based on the design and detailing of the canopy.

Snow load

For persistent and transient design situations, the snow load which should be taken into account in The Netherlands can be determined using equation (NL2.3):

$$s = 0{,}7\mu_i \text{ kN/m}^2$$

Figure 2.18a can be used to determine the snow load shape coefficients for the canopy, where only the roof parts at both sides of the central column are relevant. EN 1991-1-3 does not provide snow load shape coefficients for canopies, and the centre part of figure 2.18a shows the best resemblance to the canopy being considered in this example. Case (i) is the undrifted snow load arrangement and case (ii) the drifted snow load arrangement. For the slope $\alpha = 6{,}5^\circ$, the following coefficients are valid:

$$\mu_1 = 0{,}8$$

$$\mu_2 = 0{,}8 + \frac{0{,}8\alpha}{30} = 0{,}8 + \frac{0{,}8 \cdot 6{,}5}{30} = 0{,}97$$

The fact that the edge of the billboard extends h = 0,3 m above the roof plane is also important. For such a case, figure 2.18b is valid, resulting in the following coefficients:

$$\mu_1 = 0{,}8$$

$$\mu_2 = \frac{\gamma h}{s_k} = \frac{2h}{s_k} = \frac{2 \cdot 0{,}3}{0{,}7} = 0{,}86$$

The density of snow $\gamma = 2$ kN/m^3 is taken from EN 1991-1-3, cl. 6.2. The drift length is $l_s = 2h = 2 \cdot 0{,}3 = 0{,}6$ m, but due to the prescribed limitation $l_s = 5$ m is taken. Superposition leads to the governing snow load shape coefficients for the canopy, as shown in figure 2.41.

Accidental action

Accidental load cases are irrelevant for the canopy regarding its nature.

With these actions, combinations of actions for persistent or transient design situations (fundamental combinations, set B) for the ultimate limit state can be determined. It will then appear that the combination of actions 'permanent load (self-weight) + wind action' is governing for the analysis of this canopy.

2.7.3 Wind action on the façades of an office building

- *Given:* A 70 m high office building with twenty floors and a plan of 24x12 m in Rotterdam, The Netherlands, in a non-urbanized area. The structure consists of a steel frame around a concrete core which provides stability (see example NL2.2 and fig. NL2.5). The self-weight of the building is 78000 kg per meter height. The façades mainly consist of glass and steel. The flat roof has a bitumen cover with a gravel finish.
- *Question:* What is the wind action on the façades of the building for a stability analysis?
- *Answer:* The building is located in Rotterdam, The Netherlands in wind area II with $v_{b,0}$ = 27 m/s (fig. NL2.6). The basic wind velocity at a height of 10 m is then, according to equation (2.10):

$$v_b = c_{dir} \cdot c_{season} \cdot v_{b,0} = 1 \cdot 1 \cdot 27 = 27 \text{ m/s}$$

For a stability analysis of the office building, internal forces are of no importance because they are internally in equilibrium. Due to the rectangular plan of the building, the wind action on the short face is different from the wind on the long face. The wind action on the faces of the office building is determined by the wind force F_w according to equation (2.22):

$$F_w = c_s c_d \cdot c_f \cdot q_p(z_e) \cdot A_{ref}$$

The following quantities are determined in sequence:
- force coefficient c_f (also required for determining the resonance response factor B^2 in the equation for the structural factor);
- structural factor $c_s c_d$;
- peak velocity pressure $q_p(z_e)$;
- reference area A_{ref}.

Force coefficient c_f

The force coefficient c_f follows from equation (2.23) containing the parameters $c_{f,0}$, ψ_r and ψ_λ.
- For *wind on the short face* of the office building d/b = 24/12 = 2. From figure 2.26, it can be determined that $c_{f,0}$ = 1,65. For a building with sharp edges: ψ_r = 1,0.

The slenderness of the building is according to figure 2.27 (case number 2 with ℓ = 70 m ≥ 50 m) the smaller value of $\lambda = 1{,}4\ell/b = 1{,}4 \cdot 70/12 = 8{,}2$ or λ = 70. So the slenderness is λ = 8,2. For a closed building, the solidity ratio is φ = 1,0. From figure 2.28 it can be determined that ψ_λ = 0,68. With these data the force coefficient c_f becomes:

$$c_{f,\text{short face}} = c_{f,0} \psi_r \psi_\lambda = 1{,}65 \cdot 1{,}0 \cdot 0{,}68 = 1{,}12$$

- For *wind on the long face* of the office building d/b = 12/24 = 0,5. From figure 2.26, it can be determined that $c_{f,0}$ = 2,3. Since the building has sharp edges ψ_r = 1,0.

The slenderness of the building is according to figure 2.27 (case number 2 with ℓ = 70 m ≥ 50 m) the smaller value of $\lambda = 1{,}4\ell/b = 1{,}4\cdot 70/24 = 4{,}1$ or λ = 70. The first value is critical. Again, for a closed building, the solidity ratio is φ = 1,0. From figure 2.28 it can be determined that ψ_λ = 0,66. With these data the force coefficient c_f can be obtained as follows:

$$c_{f,\text{long face}} = c_{f,0}\psi_r\psi_\lambda = 2{,}3\cdot 1{,}0\cdot 0{,}66 = 1{,}52$$

Structural factor c_sc_d

For wind on the front surface the structural factor is c_sc_d = 1, since the height of the building is less than four times the depth of the building in the direction of the wind (h = 70 m < 4·24 = 96 m) and the building consists of frames and a core of concrete walls lower than 100 m.

For wind on the long face, the criterion is not met because h = 70 m ≥ 4·12 = 48 m. In this case, c_sc_d shall be determined using equation (2.25). The reference height z_s is (see fig. 2.30): z_s = 0,6h = 0,6·70 = 42 m ≥ z_{min} = 4 m for non-urbanized areas. The roughness length is z_0 = 0,2 m (table NL2.7). The following parameters can now be determined:

– mean wind velocity $v_m(z_s)$;
– background response factor B^2;
– resonance response factor R^2;
– peak factor k_p;
– turbulence intensity $I_v(z_s)$.

- The *mean wind velocity* $v_m(z_s)$ can be determined using equation (2.11). The roughness factor $c_r(z_s)$ is provided by equation (2.12). The orography factor $c_o(z_s)$ for a level surface is equal to 1,0, so:

$$k_r = 0{,}19\left(\frac{z_0}{0{,}05}\right)^{0{,}07} = 0{,}19\cdot\left(\frac{0{,}2}{0{,}05}\right)^{0{,}07} = 0{,}209$$

$$c_r(z_s) = k_r \ln\left(\frac{z_s}{z_0}\right) = 0{,}209\cdot\ln\left(\frac{42}{0{,}2}\right) = 1{,}12$$

$$v_m(z_s) = c_r(z_s)c_0(z_s)v_b = 1{,}12\cdot 1{,}0\cdot 27 = 30{,}2 \text{ m/s}$$

- The *background response factor* B^2 can be determined using equation (NL2.4), where the turbulence length scale $L(z_s)$ is provided by equation (2.30):

$$L(z_s) = L_t\left(\frac{z_s}{z_t}\right)^{\alpha} = 300\cdot\left(\frac{42}{200}\right)^{0{,}67\,+\,0{,}05\cdot\ln 0{,}2} = 119{,}6 \text{ m}$$

$$B^2 = \frac{1}{1 + \frac{3}{2}\sqrt{\left(\frac{b}{L(z_s)}\right)^2 + \left(\frac{h}{L(z_s)}\right)^2 + \left(\frac{b}{L(z_s)}\cdot\frac{h}{L(z_s)}\right)^2}}$$

$$= \frac{1}{1 + \frac{3}{2}\sqrt{\left(\frac{24}{119,6}\right)^2 + \left(\frac{70}{119,6}\right)^2 + \left(\frac{24}{119,6}\cdot\frac{70}{119,6}\right)^2}} = 0,514$$

- The *resonance response factor* R^2 can be determined using equation (NL2.5). First, the logarithmic decrement δ with parameters δ_s, δ_a and δ_d is determined using equation (2.32).

For a structure in concrete (core) and steel (frames): $\delta_s = 0{,}08$, see table 2.31. For determining δ_a, equation (2.33) is used. The fundamental frequency n_1 may be assumed as $n_1 = 46/h = 46/70 = 0{,}657$ Hz. The equivalent mass is $m_e = 78000$ kg/m. This leads to δ_a being:

$$\delta_a = \frac{c_{f,\text{longface}}\,\rho b v_m(z_s)}{2n_1 m_e} = \frac{1,52\cdot 1,25\cdot 24\cdot 30,2}{2\cdot 0,657\cdot 78000} = 0,013$$

The building has no special provisions (such as mass dampers or a water tank on the top floor) to limit the damping, so $\delta_d = 0$. The logarithmic decrement δ is then:

$$\delta = \delta_s + \delta_a + \delta_d = 0{,}08 + 0{,}013 + 0 = 0{,}093$$

The dimensionless wind power spectral density function S_L follows from equation (2.34):

$$f_L = \frac{n_1 L(z_s)}{v_m(z_s)} = \frac{0,657\cdot 119,6}{30,2} = 2,60$$

$$S_L = \frac{6,8 f_L}{\left(1 + 10,2 f_L\right)^{5/3}} = \frac{6,8\cdot 2,60}{\left(1 + 10,2\cdot 2,60\right)^{5/3}} = 0,0705$$

The size reduction function K_s can be determined using equation (NL2.6):

$$\varphi_y = \frac{c_y b n_1}{v_m(z_s)} = \frac{11,5\cdot 24\cdot 0,657}{30,2} = 6,00 \quad \text{and} \quad \varphi_z = \frac{c_z h n_1}{v_m(z_s)} = \frac{11,5\cdot 70\cdot 0,657}{30,2} = 17,5$$

$$G_y = \frac{1}{2} = 0,5 \quad \text{and} \quad G_z = \frac{3}{8} = 0,375$$

$$K_s = \frac{1}{1 + \sqrt{\left(G_y\phi_y\right)^2 + \left(G_z\phi_z\right)^2 + \left(\frac{2}{\pi}G_y\phi_y G_z\phi_z\right)^2}}$$

$$= \frac{1}{1 + \sqrt{\left(0,5\cdot 6,00\right)^2 + \left(0,375\cdot 17,5\right)^2 + \left(\frac{2}{\pi}\cdot 0,5\cdot 6,00\cdot 0,375\cdot 17,5\right)^2}} = 0,0646$$

With these data, the resonance response factor can be determined:

$$R^2 = \frac{\pi^2}{2\delta} S_L K_s = \frac{\pi^2}{2 \cdot 0,093} \cdot 0,0705 \cdot 0,0646 = 0,242$$

- The *peak factor* follows from the equations (2.26) and (2.27):

$$\nu = n_1 \sqrt{\frac{R^2}{B^2 + R^2}} = 0,657 \cdot \sqrt{\frac{0,242}{0,514 + 0,242}} = 0,372$$

$$k_p = \sqrt{2\ln(\nu T)} + \frac{0,6}{\sqrt{2\ln(\nu T)}} = \sqrt{2 \cdot \ln(0,372 \cdot 600)} + \frac{0,6}{\sqrt{2 \cdot \ln(0,372 \cdot 600)}} = 3,471$$

- The *turbulence intensity* $I_v(z_s)$ is provided by equation (2.28), where $c_0(z_s) = 1,0$ should be applied for the orography factor and $k_I = 1,0$ should be applied for the turbulence factor:

$$I_v(z_s) = \frac{k_I}{c_o(z_s) \ln \frac{z_s}{z_0}} = \frac{1,0}{1,0 \cdot \ln \frac{42}{0,2}} = 0,187$$

Finally, the structural factor can be determined with the data above:

$$c_s c_d = \frac{1 + 2k_p I_v(z_s)\sqrt{B^2 + R^2}}{1 + 7I_v(z_s)} = \frac{1 + 2 \cdot 3,471 \cdot 0,187 \cdot \sqrt{0,514 + 0,242}}{1 + 7 \cdot 0,187} = 0,922$$

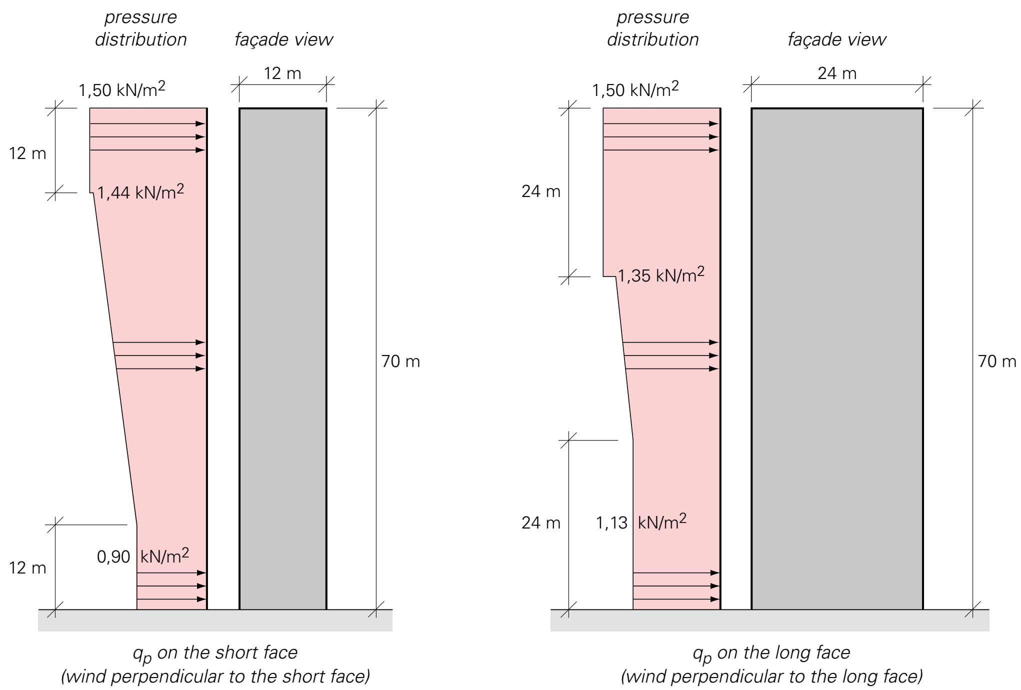
Peak velocity pressure $q_p(z_e)$ at reference height z_e

For this office building the height is greater than twice the width: h = 70 m > 2b = 2·24 = 48 m. Therefore, for determining the reference height, the distribution of figure 2.21c applies. The peak velocity pressure depends on the height; the values are provided in table NL2.8.
For wind on the lower part of the building, up to a height above the ground equal to the width of the building, the peak velocity pressure is:

long face: $q_p(z_e = b = 24) = 1,13$ kN/m^2 for the lower 24 m
short face: $q_p(z_e = b = 12) = 0,90$ kN/m^2 for the lower 12 m

For wind on the top part of the building, up to a height h – b:

long face: $q_p(z_e = h = 70) = 1,50$ kN/m^2 from the top to h – b = 70 – 24 = 46 m height
short face: $q_p(z_e = h = 70) = 1,50$ kN/m^2 from the top to h – b = 70 – 12 = 58 m height.



NL2.16 Distribution of the peak velocity pressure qp(ze) over the height of the short and long faces.

For the height in between, the peak velocity pressure is in accordance with the distribution shown in figure 2.21c. The resulting distribution of the peak velocity pressure on both faces is schematically shown in figure NL2.16.
Asymmetrical actions, that for example lead to a pressure distribution which causes torsion (see fig. 2.25), should also be taken into account although they are not considered in this example.

Reference area A_{ref}

For a rectangular building, the reference area A_{ref} is equal to the area of the façade loaded by wind, or the area of a strip with height h_{strip} (see fig. 2.21c) of that façade loaded by the peak velocity pressure at the height of that strip.

Pressure and friction coefficients c_{pe} and c_{fr}

For a stability analysis, the overall pressure coefficients $c_{pe,10}$ are important. For wind on the short face the height to width ratio is h/d = 70/24 = 2,9 and the values follow from figure 2.22 by linear interpolation. For the windward side (zone D) $c_{pe,10}$ = +0,8 is obtained and for the leeward side (zone E) $c_{pe,10}$ = –0,6, giving a total coefficient of 1,4 (not including friction). Due to the lack of correlation between the wind pressures on the windward and leeward sides, the total coefficient may be multiplied by 0,85 resulting in 1,4·0,85 = 1,19.

For wind on the long face h/d = 70/12 = 5,8 > 5. Figure 2.22 may not be applied in this case. The wind action should be directly determined using EN 1991-1-4, cl. 7.6, where the force coefficient $c_{f,long\ face}$ = 1,52 (determined earlier) should be applied. This force coefficient takes friction into account. When, for reasons of comparison, figure 2.22 is applied, $c_{pe,10}$ = +0,8 is obtained for the windward side (zone D) and $c_{pe,10}$ = –0,7 for the leeward side (zone E), giving a total coefficient of 1,5 (not including friction). Again, due to the lack of correlation of the wind pressures at the windward and leeward sides, the total coefficient may be multiplied by 0,85 resulting in 1,5·0,85 = 1,28. This is lower than the value for the force coefficient of $c_{f,long\ face}$ = 1,52 mentioned earlier. This is partly because the pressure coefficients for h/d = 5 are assumed while for this building h/d = 5,8. It is also because friction is already included in the force coefficient c_f. The other zones (side faces) are not relevant for a stability analysis.

For wind the friction coefficient follows from table 2.29, and in the context of this example, with steel and glass façades (smooth surface), c_{fr} = 0,02.
Friction forces only need to be taken into account on faces parallel with the wind direction, with a distance to the roof edges or the corners at the windward side larger than the smaller value of 2b or 4h. Here, 2b is critical. For the wind on the long face: 2b = 2·24 = 48 m. This is larger than the building dimension in the wind direction. For the wind on the short face: 2b = 2·12 = 24 m. This is exactly equal to the building dimension in the wind direction. In both cases, friction does not need to be taken into account.

Wind forces F_w
With the values obtained, it is possible to determine all required wind forces on the faces of the building for a stability analysis. These wind actions are not developed further here.

Modelling

3 Annex NL

No annex required for this chapter

Annex NL 4 Analysis

p. 4-5 ***EN 1993-1-1, cl. 5.2.1(3)***

For both elastic and plastic analysis the same boundary is used: $\alpha_{cr} \geq 10$. This then is the criterion which can be used to classify frames as sway or non-sway:

$\alpha_{cr} \geq 10 \quad \Rightarrow \quad$ non-sway
$\alpha_{cr} < 10 \quad \Rightarrow \quad$ sway

p. 4-8 ***EN 1993-1-1, cl. 5.3.2(3)***

Table 5.1 should not be applied. Instead, the Dutch National Annex provides the local bow imperfections in the form of an equation:

$$e_0 = \alpha\left(\bar{\lambda} - 0,2\right)\frac{M_{c,Rd}}{N_{c,Rd}}$$

Where:

a imperfection factor, depending on the buckling curve to be applied for the member;
$\bar{\lambda}$ relative slenderness of the member;
$M_{c,Rd}$ design value of the moment resistance of the cross-section;
$N_{c,Rd}$ design value of the compression resistance of the cross-section.

p. 4-9 ***EN 1993-1-1, cl. 5.3.2(11)***

No restrictions regarding the application of this method.

p. 4-10 ***EN 1993-1-1, cl. 5.3.2(11)***

No restrictions regarding the application of this method.

p. 4-11 ***EN 1993-1-1, cl. 5.2.2(8)***

In the first-order analysis, the global initial sway imperfections of EN 1993-1-1, cl. 5.3.2(3) should be taken into account. Also second-order effects should be taken into account for beams and joints. Therefore, it is referred to design rules for the modification of the force distribution in frames. EN 1993-1-1, cl. 5.2.2(8) assumes implicitly that the frame is a sway frame, however second-order effects may also influence the force distribution in beams and joints of non-sway frames. Therefore, the Dutch National Annex provides rules for the modification of the force distribution for non-sway frames.

p. 4-36 ***EN 1993-1-1, cl. 5.2.2(8)***

EN 1993-1-1, cl. 5.2.2(8) holds for sway frames and may be applied if:

– in the analysis of the internal forces according to first-order theory global initial sway imperfections are taken into account according to EN 1993-1-1, cl. 5.3.2(3);
– the effect of using the sway buckling length on the force distribution in beams and their connections to columns is taken into account according to EN 1993-1-1, cl. 5.2.2(9) to 5.2.2(12) of the Dutch National Annex.

• Clause 5.2.2(9) of the Dutch National Annex to EN 1993-1-1 specifies the way in which the force distribution needs to be modified for beams and their connections in sway frames.
• Clause 5.2.2(10) of the Dutch National Annex to EN 1993-1-1 is similar to cl. 5.2.2(8) of EN 1993-1-1 but describes the equivalent column method for non-sway frames. The method may be applied if:
– in the analysis of the internal forces according to first-order theory local initial bow imperfections are taken into account according to EN 1993-1-1, cl. 5.3.2(3);
– the effect of using the non-sway buckling length on the force distribution in beams and their connections to columns is taken into account according to EN 1993-1-1, cl. 5.2.2(11) of the Dutch National Annex.
• Clause 5.2.2(11) of the Dutch National Annex to EN 1993-1-1 specifies requirements for which the force distribution in non-sway frames does not need to be modified.
• If these requirements in clause 5.2.2(11) are not met, clause 5.2.2(12) of the Dutch National Annex to EN 1993-1-1 specifies the way in which the force distribution needs to be modified for beams and their connections in non-sway frames.

p. 4-37 ***EN 1993-1-1, cl. 5.3.2(11)***

No restrictions regarding the application of this method.

p. 4-39 ***EN 1993-1-1, cl. 5.3.4***

The recommended value of k = 0,5 is adopted.

Annex NL 5

Analysis methods

No annex required for this chapter

Assessment by code checking

6 Annex NL

p. 6-9 ***EN 1990, cl. A1.4.2***

The applicable serviceability limits for a structure are not prescribed by the Dutch Building Decree and should therefore be agreed between the client and structural engineer for a given project. However they may deviate from the serviceability limits of EN 1990, annex A1.4, cl. A1.4.2.

p. 6-28 (a + b) ***EN 1993-1-1, cl. 6.1(1)***

The recommended value γ_{M0} = 1,00 is accepted.

p. 6-29 ***EN 1993-1-1, cl. 6.2.6***

h_w is the height of the web between the flanges ($h_w = h - 2_{tf}$). Other countries may use another definition of h_w.

p. 6-30 ***EN 1993-1-1, cl. 6.2.6***

The shear zone A_v according to equation (6.27) is used. Other countries may use a (slightly) different definition of A_v.

p. 6-31 (a + b) ***EN 1993-1-1, cl. 6.1(1)***

The recommended value γ_{M0} = 1,00 is accepted.

p. 6-34 ***EN 1993-1-1, cl. 6.1(1)***

The recommended value γ_{M0} = 1,00 is accepted.

p. 6-41 (a + b) ***EN 1993-1-1, cl. 6.1(1)***

The recommended value γ_{M0} = 1,00 is accepted.

p. 6-42 ***EN 1990, cl. A1.4.2***

For material independent serviceability limits the Dutch National Annex to EN 1990 refers in cl. A1.4.2 to:

- clause A1.4.3 for vertical deflections of beams and floors and horizontal displacements of buildings and storeys;
- clause A1.4.4 for vibrations.

p. 6-43 ***EN 1990, cl. A1.2.2, table A1.1***

For snow loads in The Netherlands ψ_2 = 0 is used.

p. 6-44 ***EN 1993-1-8, cl. 3.1.1 (3)***

The Dutch National Annex allows the following bolt classes to be used in the Netherlands: 4.6, 5.6, 6.8, 8.8, 10.9.

Annex NL 7

Resistance of cross-sections

p. 7-2 ***EN 1993-1-1, cl. 6.1(1)***

The recommended values γ_{M0} = 1,00, γ_{M1} = 1,00 and γ_{M2} = 1,25 are accepted.

For structures which are not covered by the codes EN 1993-1 (buildings) to EN 1993-6, the partial factors for bridges from EN 1993-2 shall be used.

p. 7-8 ***EN 1993-1-1, cl. 6.1(1)***

The recommended value γ_{M0} = 1,00 is accepted.

p. 7-10 ***EN 1993-1-1, cl. 6.1(1)***

The recommended value γ_{M0} = 1,00 is accepted.

p. 7-12 ***EN 1993-1-1, cl. 6.1(1)***

The recommended value γ_{M0} = 1,00 is accepted.

p. 7-14 (a) ***EN 1993-1-1, cl. 6.2.6(3)***

The recommended values for A_w in are used. Some countries use different values of A_w, although there's no nationale choice allowed.

p. 7-14 (b) ***EN 1993-1-1, cl. 5.1(2)***

η = 1,2 for steel grades up to and including S460, and η = 1,0 for steel grades with higher strengths.

p. 7-16 ***EN 1993-1-1, cl. 6.1(1)***

The recommended value γ_{M0} = 1,00 is accepted.

p. 7-18 ***EN 1993-1-1, cl. 6.2.8***

As an addition to the design rules for the combination of bending and shear the Dutch National Annex provides design rules based on the former Dutch building codes as non-contradictory complementary information for cases currently not covered by EN 1993-1-1, cl. 6.2.8. These additional rules allow the reduced bending moment resistance allowing for shear to be determined for:

– I-sections in class 1 and 2 with identical flanges in bending about the minor axis;
– square and rectangular hollow sections with cross-sections in class 1 or 2 in bending about the major and minor axes.

p. 7-21 ***EN 1993-1-1, cl. 6.1(1)***

The recommended value γ_{M0} = 1,00 is accepted.

p. 7-22 ***EN 1993-1-1, cl. 6.2.9***

No additionale design rules for bending and axial force.

p. 7-26 (a + b) ***EN 1993-1-1, cl. 6.1(1)***

The recommended value γ_{M0} = 1,00 is accepted.

p. 7-27 (a + b) ***EN 1993-1-1, cl. 6.1(1)***

The recommended value γ_{M0} = 1,00 is accepted.

p. 7-28 (a) ***EN 1993-1-1, cl. 6.2.10***

The Dutch National Annex provides design rules for the interaction of:

– mono-axial bending about the major axis, shear and axial force for I-, square and rectangular sections in class 1 and 2;
– mono-axial bending about the minor axis, shear and axial force for I-, square and rectangular sections in class 1 and 2;
– mono-axial bending, shear and axial force for circular hollow sections in class 1 and 2;
– biaxial bending, shear and axial force for I-, circular, square and rectangular sections in class 1 and 2.

Here only a selection of the additional design rules is provided to cover the following three cases:

– mono-axial bending about the major axis, shear and axial force for I-sections in class 1 and 2;
– mono-axial bending about the minor axis, shear and axial force for I-sections in class 1 and 2;
– biaxial bending, shear and axial force for I-sections in class 1 and 2.

- Mono-axial bending about the major (y-)axis, shear and axial force for I-sections in class 1 and 2:

$$\frac{M_{y,Ed}}{M_{y,V,Rd}} + \frac{\dfrac{N_{Ed}}{N_{Vz,Rd}} - \dfrac{a_2}{2}}{1 - \dfrac{a_2}{2}} \leq 1{,}0 \tag{NL7.1}$$

with:

$$N_{Vz,Rd} = \frac{N_{pl,Rd} - \rho A_v f_y}{\gamma_{M0}}$$

$$a_2 = a_1(1 - \rho) \quad \text{with} \quad a_1 = \frac{A - 2bt_f}{A} \leq 0{,}5 \tag{NL7.2}$$

$M_{y,V,Rd}$ follows from equation (7.40), where A_v should be used for A_w and with ρ according to equation (7.37).

- Mono-axial bending about the minor (z-)axis, shear and axial force for I-sections in class 1 and 2:

$$\frac{M_{z,Ed}}{M_{z,V,Rd}} + \left(\frac{\frac{N_{Ed}}{N_{Vy,Rd}} - a_1}{1 - a_1} \right)^2 \leq 1{,}0 \qquad \text{(NL7.3)}$$

with:

$$M_{z,V,Rd} = \frac{q_y M_{pl,z,Rd}}{\gamma_{M0}}$$

$$N_{Vy,Rd} = \frac{N_{pl,Rd} - 2\left(1 - q_y\right) b t_f f_y}{\gamma_{M0}} \qquad \text{(NL7.4)}$$

$$q_y = 1{,}03\sqrt{1 - \left(\frac{V_{y,Ed}}{V_{pl,y,Rd}}\right)^2}$$

Where:

$V_{y,Ed}$ design shear force in the direction of the y-axis;
$V_{pl,y,Rd}$ design plastic shear resistance according to equation (7.18) with $A_v = 2bt_f$;
a_1 factor according to equation (NL7.2).

- Biaxial bending, shear and axial force for I-sections in class 1 and 2:

$$\beta_0 \left(\frac{M_{y,Ed}}{M_{y,N,V,Rd}} \right)^{\alpha_1} + \beta_1 \left(\frac{M_{z,Ed}}{M_{z,N,V,Rd}} \right)^{\alpha_2} \leq 1{,}0 \qquad \text{(NL7.5)}$$

with:

$$M_{y,N,V,Rd} = M_{y,V,Rd} \frac{1 - \frac{N_{Ed}}{N_{Vz,Rd}}}{1 - \frac{a_2}{2}}$$

$$M_{z,N,V,Rd} = M_{z,V,Rd} \left(1 - \left(\frac{\frac{N_{Ed}}{N_{Vy,Rd}} - a_1}{1 - a_1} \right)^2 \right) \qquad \text{(NL7.6)}$$

Where:

$M_{y,V,Rd}$ reduced design moment resistance allowing for shear according to equation (7.40) where A_v should be used for A_w;

$N_{Vz,Rd}$ reduced design axial resistance allowing for shear according to equation (NL7.2);

$M_{z,V,Rd}$ reduced design moment resistance allowing for shear according to equation (NL7.3);

$N_{Vy,Rd}$ reduced design axial resistance allowing for shear according to equation (NL7.4).

The following values for the coefficients α and β in equation (NL7.5) may be applied as a conservative approach:

$$a_1 = a_1 = b_0 = b_1 = 1,0 \qquad \text{(NL7.7)}$$

For b > 0,3h, the following coefficients may be applied and they will lead to a less conservative result:

$$\alpha_1 = \alpha_2 = 1,6 - \frac{\dfrac{N_{Ed}}{N_{c,Rd}}}{2\ln\left(\dfrac{N_{Ed}}{N_{c,Rd}}\right)} \quad \text{and} \quad \beta_0 = \beta_1 = 1,0 \qquad \text{(NL7.8)}$$

Example NL7.1

- *Given*. The pin-ended IPE 300 façade column in steel grade S235 of example 7.9, which is loaded by an axial force N_{Ed} = 305,5 kN, a bending moment M_{Ed} = 75 kNm and a shear force V_{Ed} = 250 kN in the critical cross-section at mid span.
- *Question*. Check the resistance of the cross-section at mid span using the equations provided by the Dutch National Annex to EN 1993-1-1, cl. 6.2.10.
- *Answer*. The Dutch National Annex to EN 1993-1-1, cl. 6.2.10, provides an explicit check for the combination of bending moment, shear and axial force using the design rules of equations (NL7.1) and (NL7.2):

$$\rho = \left(\frac{2V_{Ed}}{V_{pl,Rd}} - 1\right)^2 = \left(2\cdot\frac{250}{348} - 1\right)^2 = 0,19$$

$$M_{y,V,Rd} = \frac{\left(W_{pl,y} - \dfrac{\rho A_v^2}{4t_w}\right)f_y}{\gamma_{M0}} = \frac{\left(628\cdot10^3 - \dfrac{0,19\cdot2568^2}{4\cdot7,1}\right)\cdot235\cdot10^{-6}}{1,00} = 137 \text{ kNm}$$

$$N_{Vz,Rd} = \frac{N_{pl,Rd} - \rho A_v f_y}{\gamma_{M0}} = \frac{1265 - 0,19\cdot2568\cdot235\cdot10^{-3}}{1,00} = 1150 \text{ kN}$$

$$a_1 = \frac{A - 2bt_f}{A} = \frac{5381 - 2 \cdot 150 \cdot 10,7}{5381} = 0,40 \leq 0,5$$

$$a_2 = a_1(1 - \rho) = 0,40 \cdot (1 - 0,19) = 0,32$$

$$\frac{M_{y,Ed}}{M_{y,V,Rd}} + \frac{\frac{N_{Ed}}{N_{Vz,Rd}} - \frac{a_2}{2}}{1 - \frac{a_2}{2}} = \frac{75}{137} + \frac{\frac{305,5}{1150} - \frac{0,32}{2}}{1 - \frac{0,32}{2}} = 0,55 + 0,13 = 0,68 \leq 1,0 \text{ (OK)}$$

This check is more conservative than the reduced yield stress and reduced web thickness approaches of example 7.9.

p. 7-28 (b) ***EN 1993-1-1, cl. 6.1(1)***

The recommended value $\gamma_{M0} = 1,00$ is accepted.

p. 7-40 ***EN 1993-1-1, cl. 6.1(1)***

The recommended value $\gamma_{M0} = 1,00$ is accepted.

p. 7-43 ***EN 1993-1-1, cl. 6.1(1)***

The recommended value $\gamma_{M0} = 1,00$ is accepted.

p. 7-44 ***EN 1993-1-1, cl. 6.1(1)***

The recommended value $\gamma_{M0} = 1,00$ is accepted.