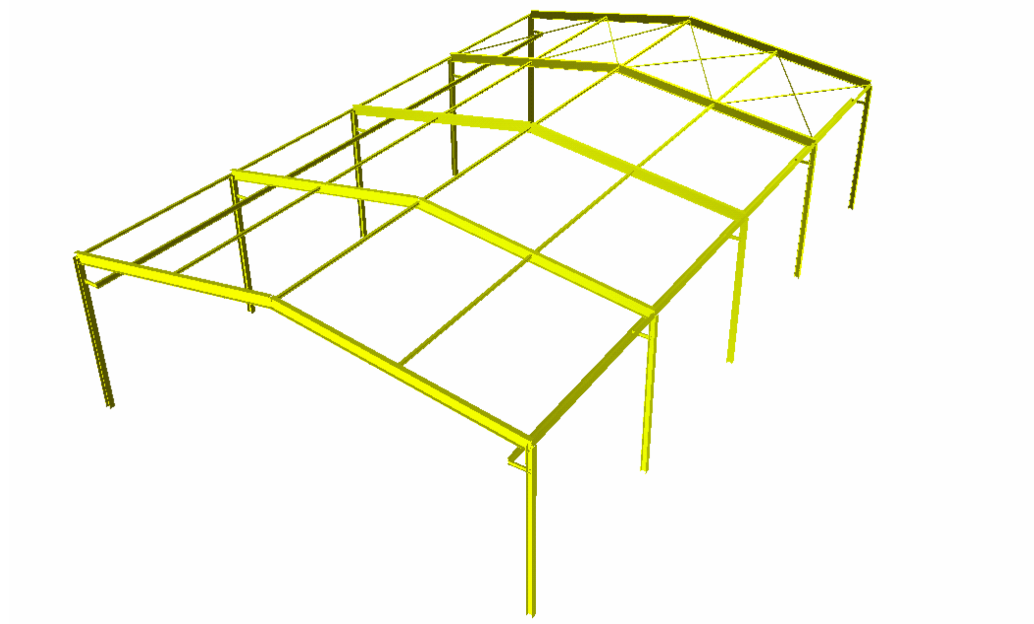
IMPLEMENTATION OF EUROCODES

HANDBOOK 3

ACTION EFFECTS FOR BUILDINGS



Guide to basis of structural reliability and risk engineering related to Eurocodes supplemented by practical examples

LEONARDO DA VINCI PILOT PROJECT CZ/02/B/F/PP-134007

DEVELOPMENT OF SKILLS FACILITATING IMPLEMENTATION OF EUROCODES

Leonardo da Vinci Pilot Project CZ/02/B/F/PP-134007

DEVELOPMENT OF SKILLS FACILITATING IMPLEMENTATION OF EUROCODES

HANDBOOK 3

ACTIONS EFFECTS FOR BUILDINGS

PARTNERSHIP:

The Klokner Institute of the Czech Technical University in Prague (KI CTU), convener, Prof. Milan Holický, Czech Republic

The Czech Chamber of Certified Engineers and Technicians Engaged in Construction (CKAIT), Prof. Alois Materna, Czech Republic,

The Institute for Steel Construction, University of Technology Aachen (RWTH), Prof. Gerhard Sedlacek, Germany

The Spanish Organisation for Scientific Research (IET), Dr. Angel Arteaga, Spain

The University of Pisa (UOP), Prof. Luca Sanpaolesi, Italy

The Netherlands Organisation for Applied Scientific Research (TNO), Prof. Ton Vrouwenvelder, Netherlands

The Institute for Metal Constructions (IMK), Dr. Igor Kovse, Slovenia

The Building Research Establishment (BRE), Prof. Haig Gulvanessian, United Kingdom

Aachen 10.2005

Leonardo da Vinci Pilot Project CZ/02/B/F/PP-134007

DEVELOPMENT OF SKILLS FACILITATING IMPLEMENTATION OF EUROCODES

HANDBOOK 3

ACTION EFFECTS FOR BUILDINGS

Annex **Properties of selected Materials (Pages Annex-1 to Annex-31)**

FOREWORD

The Leonardo da Vinci Pilot Project CZ/02/B/F/PP-134007, "Development of Skills Facilitating Implementation of Structural Eurocodes" addresses the urgent need to implement the new system of European documents related to design of construction works and products. These documents, called Eurocodes, are systematically based on recently developed Council Directive 89/106/EEC "The Construction Products Directive" and its Interpretative Documents ID1 and ID2. Implementation of Eurocodes in each Member State is a demanding task as each country has its own long-term tradition in design and construction.

The project should enable an effective implementation and application of the new methods for designing and verification of buildings and civil engineering works in all the partner countries (CZ, DE, ES, IT, NL, SI, UK) and in other Member States. The need to explain and effectively use the latest principles specified in European standards is apparent from various enterprises, undertakings and public national authorities involved in construction industry and also from universities and colleges. Training materials, manuals and software programmes for education are urgently required.

The submitted Handbook 2 is one of 5 upcoming handbooks intended to provide required manuals and software products for training, education and effective implementation of Eurocodes:

Handbook 1: Basis of Structural Design
Handbook 2: Reliability Backgrounds
Handbook 3: Load Effects for Buildings
Handbook 4: Load Effects for Bridges
Handbook 5: Design of Buildings for Fire Situation

It is expected that the Handbooks will address the following intents in further harmonisation of European construction industry:

- reliability improvement and unification of the process of design;
- development of the single market for products and for construction services;
- new opportunities for the trained primary target groups in the labour market.

The Handbook 3 is focused on the application of Eurocode EN 1990 and the relevant Eurocodes for load assumptions. The following topics are treated in particular:

- definition of permanent and imposed loads
- definition of the climatic actions due to wind, snow and temperature
- description of material properties
- load combination
- examples of concrete buildings, steel buildings and composite buildings

The Annex to the Handbook 3 provides a review of "Material Properties " frequently used in the text. The Handbook 3 is written in a user-friendly way employing only basic mathematical tools. Attached software products accompanying a number of examples enable applications of general rules in practice.

A wide range of potential users of the Handbooks and other training materials includes practising engineers, designers, technicians, experts of public authorities, young people - high school and university students. The target groups come from all territorial regions of the partner countries. However, the dissemination of the project results is foreseen to be spread into all Member States of CEN and other interested countries.

Aachen 10/2005

CHAPTER I - SELF-WEIGHT AND IMPOSED LOADS ON BUILDINGS

Milan Holický

Klokner Institute, Czech Technical University in Prague, Czech Republic

Summary

Permanent loads due to self-weight of construction elements and materials, and variable loads due to imposed loads that are provided in EN 1991, Part 1.1, are discussed and basic guidance for their use in design calculation is given. Practical examples illustrate the application of principles and operational rules for load arrangements and for the determination of reduction factors for imposed loads.

1 INTRODUCTION

1.1 General

General principles for classification of actions on structures including environmental impacts and their modelling in verification of structural reliability are introduced in basic Eurocode EN 1990 [1]. In particular EN 1990 [1] defines characteristic, representative and design values used in design calculation (see also Designers' Handbook [2]). Detailed description of individual types of actions is then given in various Parts of Eurocode EN 1991 [3]. Part 1.1 of EN 1991 [3] covers the following topics:

- densities of construction materials and stored materials;
- self-weight of construction elements;
- variable action due to imposed loads on buildings.

Principles and application rules for self-weight and imposed loads differ and are therefore treated in EN 1991, Part 1.1 [3] independently.

1.2 Background documents

Background documents used to develop EN 1991, Part 1.1 [3] consist of national standards of CEN Member states, International Standard ISO 9194 [4], CIB Reports 115 and 116 [5, 6] (see also recent paper [7]). However, some principles, rules and numerical data provided in these documents are not entirely consistent. Moreover, available statistical data concerning densities, angles of repose and imposed loads are inconclusive [7]. Consequently, for properties of some materials and for some imposed loads intervals instead of distinct values are given in EN 1991, Part 1.1 [3].

2 GENERAL PRINCIPLES AND RULES

2.1 Classification of actions

Considering its variation in time and space, self-weight of a construction element is classified as permanent fixed action while imposed load as variable free action. However, if there is doubt about the permanency of a self-weight, then the load shall be treated as variable imposed load. Generally the imposed load is considered as static load, which may be increased by a dynamic magnification factor (see equation (2.1) in EN 1991 [3]). If an imposed load causes significant acceleration of the structure or structural element, dynamic analysis should be applied in accordance with EN 1990 [1].

2.2 Design situations

In each design situation identified in accordance with EN 1990 [1] (permanent, transition, accidental as well as seismic design situation) the most critical load cases should be considered. For a given structural element and load effect considered the most critical load cases should be determined taking into account the most unfavourable influence area of every single action (see examples in Section 6). This general principle concerns primarily load arrangements of imposed load. However, it may concern also self-weight, in particular when structural and non-structural elements or stored and materials may be removed or added (during transition or permanent design situation).

3 DENSITIES

3.1 Definition of density

The term "density" is in EN 1991, Part 1.1 [3] used for weight per unit volume, area or length. For materials having all three dimensions of the same order of magnitude, the characteristic values of densities are given as weights per unit volume (using the unit kN/m^3). For sheeting (roofing) materials having one dimension of smaller order of magnitude than the other two dimensions the characteristic values of densities are weights per unit area (kN/m^2). For one-dimensional construction elements characteristic values of densities are weights per unit length (kN/m).

Note that in some national and international documents including the International Standard ISO 9194 [4] and EN 1991-4 [8] the term "density" is understood as mass (not weight) per unit volume, area or length. Then its magnitude is given using the unit kg/m^3, kg/m^2 or kg/m and the corresponding numerical values differ from those given in EN 1991 [3]. For example, in accordance with EN 1991 [3] the density of normal weight concrete is 24 kN/m^3, in accordance with ISO 9194 [4] the density is 2400 kg/m^3 (the gravity acceleration 10 m/s^2 is usually assumed).

In general density is random variable, which may have in some cases (e.g. in case when moisture content and degree of consolidation may affect the density) a considerable scatter. In such cases the mean value and variance may be determined using available experimental measurements. The characteristic value of the density is usually defines as the mean. However, when the coefficient of variation is greater than 0,05, then upper and lower characteristic value may be used (see EN 1990 [1]).

3.2 Characteristic values

Normative characteristic values of densities as well as of angles of repose are given in Annex A of EN 1991, Part 1.1 [3]. The term "nominal" is not defined, but the quantities indicated in this document correspond to the mean values, which are usually accepted as the characteristic values. In actual conditions, both density and angle of repose may vary due to differences in local material properties, in quality of workmanship, moisture content, depth of storage, etc. That is one of the reasons why background materials and available statistical data are inconclusive. Consequently, in some cases EN 1991, Part 1.1 [3] indicates for densities and angles of repose expected intervals instead of distinct values. For example, for densities of cement mortar an interval from 19 to 23 kN/m^3 is given.

In special cases, when the variability of self-weight is large (when the coefficient of variation is greater than 0,05) or when it may have significant effect on structural reliability, the lower and upper characteristic values may be considered [1, 2]. In addition the random variability of self-weight is taken into account in design calculation by appropriate partial safety factors (for example 1,35 considered for permanent load for the verification of ultimate limit states).

4 SELF-WEIGHT OF CONSTRUCTION ELEMENTS

Construction elements cover both structural elements (load bearing frames and supporting structures) and non-structural elements (completion and finishing elements including services and machinery fixed permanently to the structure).

Self-weight of construction elements shall be determined considering nominal dimensions (given in design documentation) and characteristic (nominal) values of densities. Upper and lower characteristic values should be considered for densities of materials expected to consolidate during use, e.g. ballast on railway bridges.

5 IMPOSED LOADS ON BUILDINGS

5.1 Classification of loaded areas

Taking into account their specific use loaded areas are classified in Section 6 of EN 1991, Part 1.1 [3] into ten main categories denoted A, B, C, D, E, F, G, H, I and K. Definitions of these area are provided in Tables 6.1, 6.3 and 6.7 in EN 1990 [1]. For example, category A comprises areas for domestic and residential activities like rooms in residential buildings and houses, bedrooms in hospitals and hotels, category B covers office areas.

5.2 Load arrangements and load cases

For the design of a particular horizontal element within one storey the imposed load shall be considered as a free action applied at the most unfavourable part of the influence area of the action effects analysed. Where the loads on other storeys contribute to the resulting load effect, they may be considered as uniformly distributed (fixed) actions (see example 6.2). While this simplification may reduce number of critical load cases, it may lead in some cases (e.g. in case of a simple two bay two storeys frame with unusual topology) to unsafe results and should not be used without appropriate precaution.

The imposed load may be reduced by the reduction factor α_{A} due to the extent of the loaded area A, and by the factor α_n due to the number n of loaded storeys. Factor α_{A} is defined in EN 1991 [3] by equation (6.1):

$$\alpha_{\mathrm{A}} = \psi_0 \times 5/7 + A_0/A \leq 1 \qquad (1)$$

where factor ψ_0 is given in EN 1990 [1], Table A.1.1 (equal to 0,7 for categories A, B, C, D), and the reference area A_0 = 10 m^2. Note that with increasing loaded area A, the factor α_A decreases.

For the design of a particular vertical element (column or wall) loaded from several storeys the total imposed load may be considered in each storey as uniformly distributed. The imposed load acting on a vertical element from several storeys may be reduced by factor α_n due to number n (> 2) of loaded storeys. The factor α_n is given in EN 1991 [3] by equation (6.2):

$$\alpha_n = (2 + (n - 2)\,\psi_0)/n \tag{2}$$

With increasing number of storeys n, the factor α_n decreases (obviously there is no reduction for two storeys).

However, when the characteristic value of an imposed load is reduced by the factor ψ in a combination with other types of variable actions (e.g. with wind and snow), the reduction factor α_n shall not be used for this imposed load.

5.3 Characteristic values

Characteristic values of vertical imposed loads to be applied in these areas are specified in Tables 6.2, 6.4, 6.5, 6.6, 6.8, and 6.9 in EN 1990 [1]. Dynamic assessment of vertical loading due to dancing and rhythmic jumping is not explicitly covered in [1]. Characteristic values of horizontal imposed loads are specified in Table 6.10 for barriers and partition walls having function of barriers, and in the informative Annex in EN 1990 [1] for vehicles barriers.

In the above-mentioned tables the characteristic values of horizontal and vertical imposed loads are often given by ranges or recommended values. To select appropriate values actual conditions and use of loaded area should be taken into account by designers. In some cases the responsible national institution may specify recommended values.

5.4 Movable partitions

Provided that a floor allows a lateral distribution of loads the self-weight of lightweight movable partitions may be considered as an equivalent uniform load which is added to the imposed load. This uniformly distributed load is defined in accordance with the self-weight of the partitions as follows:

- for movable partitions with a self-weight ≤ 1,0 kN/m wall length: q_k = 0,5 kN/m^2,
- for movable partitions with a self-weight ≤ 2,0 kN/m wall length: q_k = 0,8 kN/m^2,
- for movable partitions with a self-weight ≤ 3,0 kN/m wall length: q_k = 1,2 kN/m^2.

For movable partition walls having greater self-weight than 3,0 kN/m, their actual weight, potential location and orientation should be considered.

6 EXAMPLES

6.1 Simple supported cantilever beam

An example of a simply supported cantilevered beam (shown in Figure 1) is used to illustrate the basic principle of determination of the critical load cases. Three independent permanent loads g_1, g_2 and G are indicated in Figure 1.

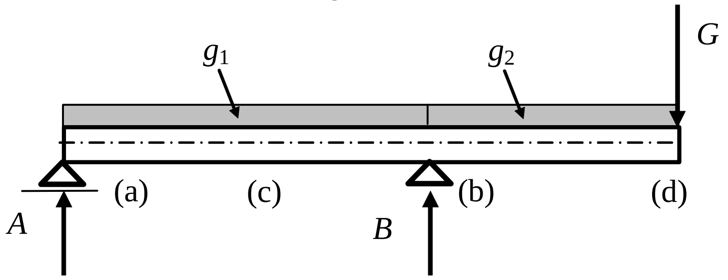


Figure 1. Three independent permanent loads g_1, g_2 and G.

Obviously the permanent load G (self-weight of cladding) is independent of the other permanent loads g_1 and g_2 (self-weight of horizontal structure and floor). These permanent loads g_1 and g_2 are considered as separate actions in order to verify equilibrium limit state. Note that the permanent loads g_1, g_2 due to self-weight of structural and non-structural elements may have the same characteristic value $g_{1,k} = g_{2,k} = g_k$, which may be multiplied by two different load factors $\gamma_{g,1} \neq \gamma_{g,2}$ when the limit state of static equilibrium is verified.

Figure 2 shows two independent imposed loads q_1 and q_2. Obviously these two actions may occur completely independently.

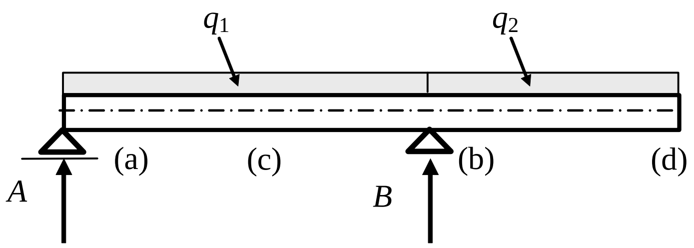


Figure 2. Two independent imposed loads $q_1$and q_2.

For a given location on a structure the most critical load cases should be determined by taking into account the most unfavourable influence area of every single action (which may be assumed to be independent of the remaining actions).

The total self-weight and imposed loads consisting of five single actions g_1, g_2, q_1, q_2 and G indicated in Figure 3 represent the critical load case of the beam for verification of bending resistance and the reaction at the support (b). Two other load cases shown in Figure 4 and 5 shall be used to verify bending moment resistance at the midspan point (c) (Figure 4), and static equilibrium of the beam (reaction A) (Figure 5).

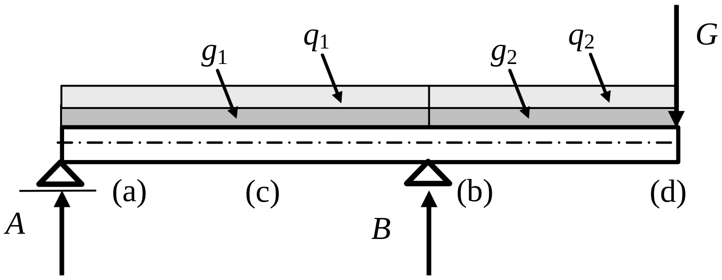


Figure 3. The total self-weight and imposed loads of the cantilevered beam.

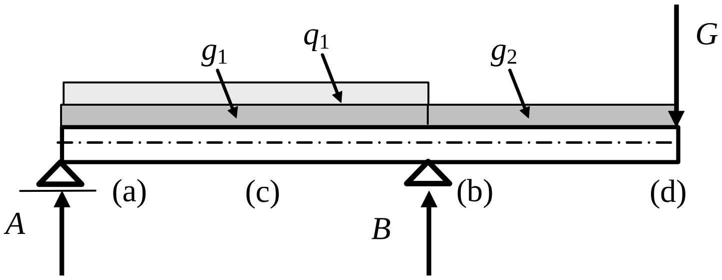


Figure 4. The critical load case to verify bending resistance at the midspan point (c).

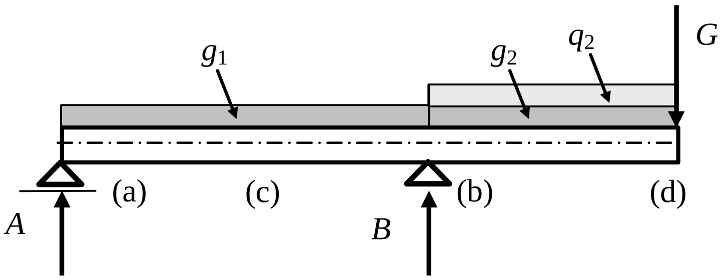


Figure 5. The critical load case to verify bending resistance at point (b) and static equilibrium (reaction A).

6.2 Frame structure

As mentioned in Section 5.2 when elements of multi-storey building are designed, some simplifying rules may be applied. For example when a particular horizontal element within one storey of a multi-storey building is designed, the imposed load at that storey shall be considered as a free action applied at the most unfavourable part of the influence area of the action effects analysed. Where the loads on other storeys contribute to the resulting load effect, they may be considered as uniformly distributed (fixed) actions.

As a practical example, the horizontal beam at the second storey of the frame shown in Figure 6 is considered. If bending resistance at the points (a) and (b) is verified then in accordance with the above rule the imposed load indicated in Figure 6 may be considered instead of a more correct (chessboard type) load arrangement shown in Figure 7.

When bending resistance at point (c) is verified then in accordance with the simplified rule the imposed load at the second floor will be located at the first and second span from the left edge. If bending resistance at point (d) is verified then the imposed load at the second floor will be located at the middle span only.

Note that in accordance with EN 1991 [3] (as already stated in Section 5.2) when designing a column at the first floor loaded by all storeys, the imposed load may be considered in each storey as uniformly distributed.

Depending on the structural element considered, the imposed load q might be reduced using factors α_A and α_n (see Section 5.2). For example when verifying the bending moment of the horizontal beam of the second storey at point (b) (see Figure 5) the imposed load q may be reduced using factor α_A given by equation (1). If the loaded area A = 30 m^2, ψ_0 = 0,7 (categories A, B, C, D) and A_0 = 10 m^2, then α_A is:

$$\alpha_A = \psi_0 \times 5/7 + A_0/A = 0{,}7 \times 5/7 + 10/30 = 0{,}83$$

If a column at the first storey is verified, the imposed load q may be reduced using factor α_n given by equation (2). Considering the number of storeys n = 3 (as indicated in Figures 6 and 7) and again ψ_0 = 0,7 the factor α_n is given as:

$$\alpha_n = (2 + (n - 2)\psi_0)/n = (2 + 0{,}7)/3 = 0{,}9$$

Thus, in this case the imposed load may be reduced by 10%. A much more significant reduction shall be obtained for greater number of loaded storeys.

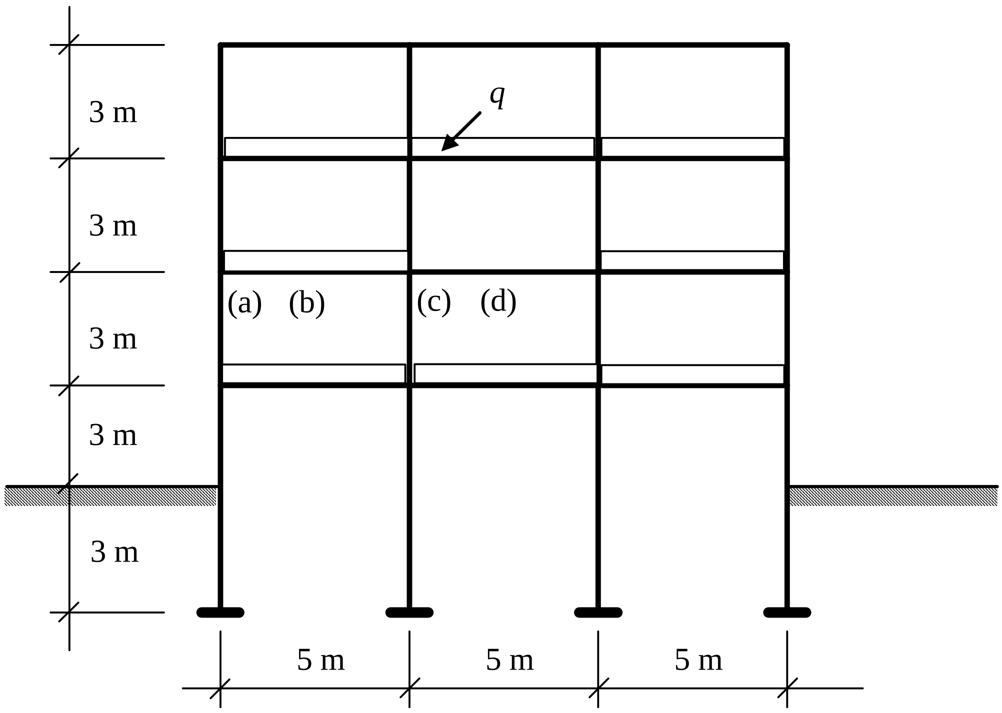


Figure 6. Simplified arrangement of the imposed load for verification of bending resistance at points (a) and (b).

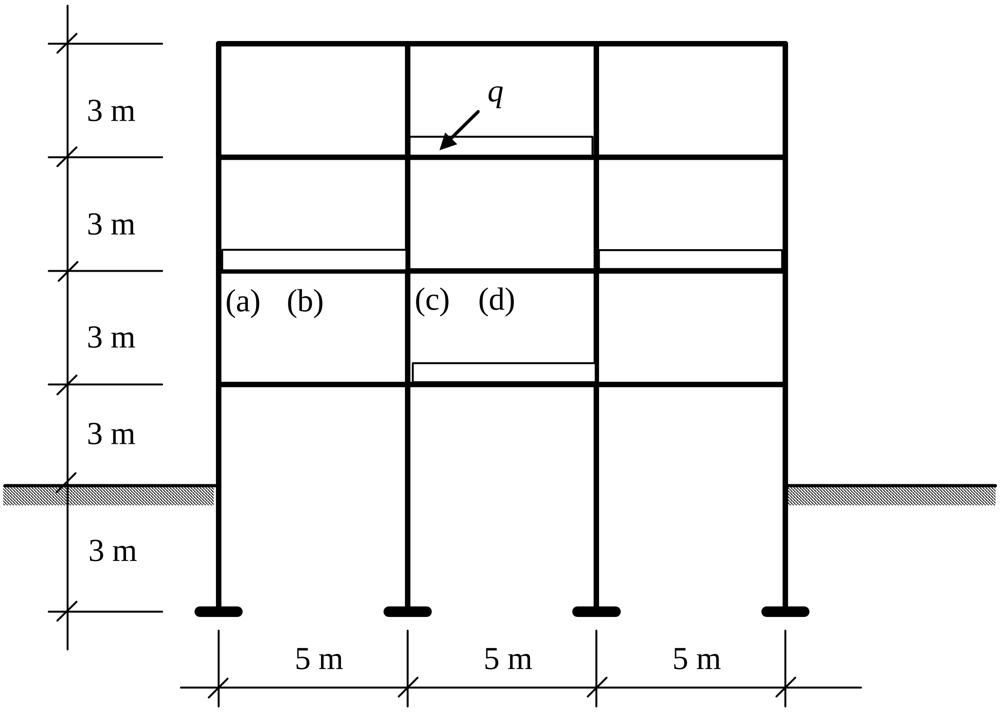


Figure 7. The most critical arrangement of the imposed load for verification of bending resistance at points (a) and (b).

7 PROBABILISTIC MODEL OF SELF-WEIGHT

Self-weight G of structural members may be usually determined as a product of the volume Ω and the density γ:

$$G = \Omega\,\gamma \tag{3}$$

Both, the volume Ω and the density γ, are random variables that may be described by normal distributions [9]. The mean of the volume Ω is approximately equal to the nominal value (as a rule slightly greater), the mean of the density γ is usually well defined by the producer. Informative coefficients of variation are indicated in Table 1; more extensive data are available in [9].

The coefficient of variation V_G of the resulting self-weight may be estimated using approximate expression for coefficients of variation

$$V_G^2 = V_\Omega^2 + V_\gamma^2 \tag{4}$$

Generally the self-weight G is described by the normal distribution.

Table 1. Examples of coefficients of variation (indicative values only).

Material	Coefficient of variation of		
	Ω	γ	G
Steel (rolled)	0,03	0,01	0,031
Concrete (plate 300 mm thick, ordinary)	0,02	0,04	0,045
Masonry unplastered	0,04	0,05	0,080
Timber (sawn beam 200 mm thick, dry)	0,01	0,10	0,100

Data indicated in Table 1 should be considered as informative values only. The coefficients of variation of Ω for concrete and timber depend strongly on the size (increase with decreasing thickness of members) and type of material. Note also that variability of non-structural members may be considerably greater than self-weight of structural members (see also [5]).

8 PROBABILISTIC MODEL OF IMPOSED LOADS

The imposed load Q is usually described by a Gumbel distribution (in [9] also Gamma and exponential distributions are used for sustained and intermittent loads respectively). In general, the total imposed load Q consists of the sustained (long-term) component q and the intermittent (short-term) component p. The sustained load q is always present while the intermittent component p may be absent and in fact may be active only very rarely (for example few days a year only). The parameters of both components including jump rate λ of sustained load, v jump rate of intermittent load and d duration time of intermittent load are indicated in Table 2, which is taken from JCSS materials [9].

Table 2. Parameters of imposed loads in accordance with loading areas.

Category	A_0 [m^2]	Sustained load q				Intermittent load p			
		μ_q [kN/m^2]	σ_V [kN/m^2]	σ_U [kN/m^2]	$1/\lambda$ [years]	μ_p [kN/m^2]	σ_U [kN/m^2]	$1/v$ [years]	d [days]
Office	20	0,5	0,3	0,6	5	0,2	0,4	0,3	1-3
Lobby	20	0,2	0,15	0,3	10	0,4	0,6	1	1-3
Residence	20	0,3	0,15	0,3	7	0,3	0,4	1	1-3
Hotel rooms	20	0,3	0,05	0,1	10	0,2	0,4	0,1	1-3
Patient room	20	0,4	0,3	0,6	5-10	0,2	0,4	1	1-3
Laboratory	20	0,7	0,4	0,8	5-10				
Libraries	20	1,7	0,5	1	10				
Classroom	100	0,6	0,15	0,4	10	0,5	1,4	0,3	1-5
Stores first floor upper floor	 100 100	 0,9 0,9	 0,6 0,6	 1,6 1,6	 1-5 1-5	 0,4 0,4	 1,1 1,1	 1,0 1,0	 1-14 1-14
Storage	100	3,5	2,5	6,9	0,1-1				
Industrial - light - heavy	 100 100	 1 3	 1 1,5	 2,8 4,1	 5-10 5-10				
Concentration of peoples	20					1,25	2,5	0,02	0,5

The standard deviation of the sustain load q may be determined [9] as

$$\sigma_q^2 = \sigma_V^2 + \sigma_U^2 \frac{A_0}{A} \kappa \tag{5}$$

where σ_V is the standard deviation of the overall load intensity, σ_U is the standard deviation of the random field describing space variation of the load, A_0 denotes the reference area (20 or 100 m^2), A the loaded area and κ is the influence factor depending on structural arrangement including boundary conditions. In common cases the factor κ is within the interval from 1 to

2,4 [9]. Figure 8 shows typical influence lines and corresponding factors κ (κ = 2 is considered in the following example as a representative value).

A relationship similar to equation (5) may be used to determine the standard deviation σ_p of the intermittent load p.

As an example consider an office area for which the characteristic value Q_k = 3 kN/m^2 is recommended in [3]. In accordance with Table 2 the mean values μ_q (for 5 years period) and μ_p (for 1-3 days period) are

$$\mu_{q,5} = 0{,}5 \text{ kN/m}^2,\ \mu_p = 0{,}2 \text{ kN/m}^2 \tag{6}$$

Assuming the factor κ = 2 and the loaded area A = 40 m^2 the standard deviations are as follows

$$\sigma_q = (0{,}30^2 + 0{,}60^2 \times 2 \times 20/40)^{0.5} = 0{,}67 \text{ kN/m}^2,\ \sigma_p = (0{,}40^2 \times 2 \times 20/40)^{0.5} = 0{,}40 \text{ kN/m}^2 \tag{7}$$

Note that the standard deviations are strongly dependent on the factor κ and the loading area A = 40 m^2 loaded area A; if κ = 2 and A = 20 m^2 then

$$\sigma_q = (0{,}30^2 + 0{,}60^2 \times 2 \times 20/20)^{0.5} = 0{,}90 \text{ kN/m}^2,\ \sigma_p = (0{,}40^2 \times 2 \times 20/20)^{0.5} = 0{,}57 \text{ kN/m}^2 \tag{8}$$

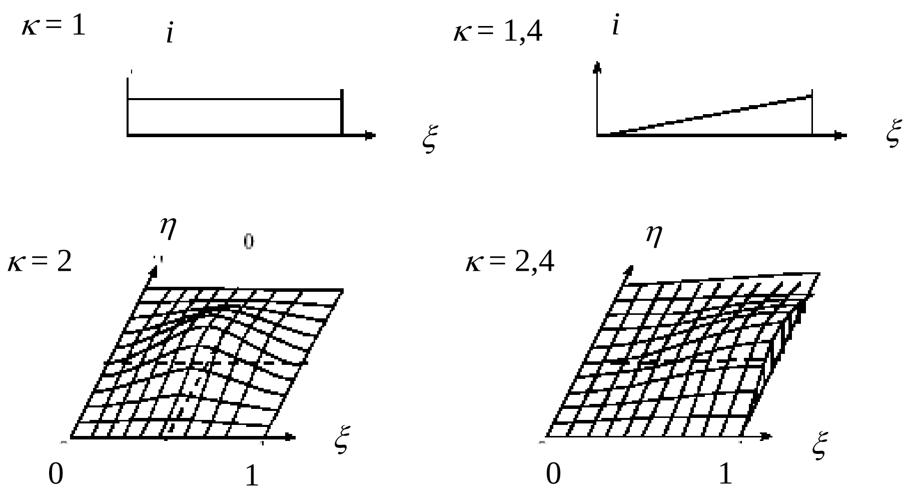


Figure 8. Typical influence lines and corresponding factors κ.

In general, with increasing loaded area A the standard deviations of both load components decrease.

Considering the factor κ = 2 and the loaded area A = 40 m^2 and assuming a Gumbel distribution for the sustained load p, then the 50-year-extremes have the mean $\mu_{p,50}$ and coefficient of variation $V_{p,50}$

$$\mu_{p,50} = 0{,}5 + 0{,}78 \ln (10) \times 0{,}67 = 1{,}70 \text{ kN/m}^2,\ V_{p,50} = 0{,}34 \tag{9}$$

Indicative parameters $\mu_{p,50}/Q_k$ = 0,6 and V_p = 0,35 correspond well to the above data and may be therefore chosen for a first approximation.

9 CONCLUDING REMARKS

The self-weight of structural and non-structural elements is usually considered as permanent action, the imposed load as variable action. In each design situation the most critical load cases shall be identified taking into account unfavourable influence area for a given structural elements and load effects verified (e.g. axial force, bending moment, shear force). The self-weight of construction elements shall be determined considering nominal dimensions and characteristic densities.

Imposed loads are specified for ten basic loaded areas. Factor α_A (due to the extent of loaded area A) and α_n (due to the number n of loaded storeys) may be used to reduce an imposed load. Factors α_A and α_n should not be considered when the characteristic value of this load is already reduced by factor ψ (in combination with other variable loads like wind or snow). Self-weight of movable partitions having actual weight up to 3 kN/m may be considered as equivalent uniform load that is added to the imposed load.

It appears that the permanent load may be described by the normal distribution having the indicative values of the mean and the coefficient of variation given as follows

$$m_G/G_k = 1 \text{ and } V_G = 0{,}1$$

The coefficient of variation may however considerable differ depending on the type of material and size of the member.

The imposed load may be described by Gumbel distribution having the indicative mean and coefficient of variation

$$m_{p,50}/Q_k = 0{,}6 \text{ and } V_p = 0{,}35$$

It should be emphasised that the above parameters of imposed load should be considered as a first approximation only. More accurate data should be specified taking into account particular structural conditions (including loaded area and influence factor).

REFERENCES

[1] EN 1990 Basis of Structural Design. European Committee for Standardisation, CEN/TC 250, April 2002.

[2] H. Gulvanessian, J.-A. Calgaro, M. Holický: Designer's Guide to EN 1990, Eurocode: Basis of Structural Design. Thomas Telford, London, 2002, ISBN: 07277 3011 8, 192 pp.

[3] EN 1991 Actions on Structures, Part 1.1 Densities, Self-weight and Imposed Loads on Buildings. European Committee for Standardisation, CEN/TC 250, April 2002.

[4] ISO 9194 Basis for Design of Structures – Action Due to the Self-weight of Structures, Non-structural Elements and Stored Materials – Density. 1987.

[5] CIB Report 115 Actions on Structures, Self-weight Loads. CIB/W81, 1989.

[6] CIB Report 116 Actions on Structures, Live Loads in Buildings. CIB/W81, 1989.

[7] G. Sedlacek, H. Gulvanessian: Eurocode 1, Basis of Design and Actions on Structures, Part 2.1: Densities, Self-weight, Imposed Loads. In: IABSE Colloquium Basis of Design and Actions on Structures, Background and Application of Eurocode 1. Delft, 1996.

[8] EN 1991 Actions on Structures. Part 1-4: Wind load. European Committee for Standardisation, CEN/TC 250/SC1, working draft, 2003.

[9] JCSS: Probabilistic model code. JCSS working materials, http://www.jcss.ethz.ch/, 2001.

CHAPTER II - SNOW LOAD

Oliver Kraus, Christiane Butz

Institute for Steel Construction, RWTH Aachen, Germany

Summary

This chapter provides an overview of the definition of snow loads given in the Eurocodes. An exemplary determination of the relevant snow loading for a building illustrates the application of prEN 1991-1-3 “Snow loads” [2].

1 DEFINITION OF SNOW LOADING

1.1 General

Eurocode 1, part 1-3: “snow loads” (prEN 1991-1-3 [2]) is based on an extensive research activity which was funded by the European Commission [3]. In the frame of this investigation measured data of hundreds of meteorological stations in Europe were analysed and the characteristic values of the ground snow load were determined by means of extreme value statistics.

Furthermore functions were developed which allow to describe the interdependency between the characteristic value of the ground snow load and the height of the relevant site above sea level. The new European snow load map as well as the relationship between altitude and snow load given in prEN 1991-1-3 are adopted from the final report of this European research project.

According to prEN 1991-1-3 the snow load on the roof is described by the following equation:

$$s = \mu_i \cdot C_e \cdot C_t \cdot s_k \tag{1}$$

where: μ_i roof shape coefficient

C_e exposure coefficient

C_t thermal coefficient

s_k characteristic value of the ground snow load for the relevant altitude

1.2 Snow load map

The Eurocode prEN 1991-1-3 is provided with maps which give the characteristic values of the snow loads on sea level for the relevant European countries. Several snow load maps are available for different climatic regions. These regions and the associated countries are:

Table 1. Climatic regions and corresponding countries as given in [2]

Climatic Region	Associated Countries
Alpine Region	South Germany, Austria, North-West France, North Italy
Central East	Denmark, Germany
Greece	Greece
Iberian Peninsula	Spain, Portugal
Mediterranean Region	Italy, South France
Central West	The Netherlands, Belgium, Luxembourg, France
Sweden, Finland	Sweden, Finland
UK, Republic of Ireland	UK, Ireland
further maps	Czech Republic, Norway, Iceland, Poland

The maps for the several climatic regions are subdivided into snow load zones Z. In addition to the values of the altitude the numbers Z of these zones are the basic input parameters for the determination of the characteristic value of the ground snow load s_k.

1.3 Determination of the snow load on the ground

For each climatic region (see table 1) in [2] an equation for the calculation of the characteristic value of the ground snow load on the relevant altitude is given. Here the equation for the climatic region “Central East” is shown in order to present an impression which basic parameters are needed:

$$s_k = (0{,}264 \cdot Z - 0{,}002) \cdot \left[1 + \left(\frac{A}{256}\right)\right] \; [\mathrm{kN/m^2}] \tag{2}$$

where: Z zone number (depending on the snow load on sea level)
A altitude above sea level [m]

1.4 Exposure coefficient and thermal coefficient

The exposure of a structure or of a roof to wind effects as well as the thermal transfer from a heated room through a non-insulated roof influences the accumulation of the snow. In order to take into account these effects prEN 1991-1-3 introduces the exposure coefficient C_e and the thermal coefficient C_t.

In general the exposure factor is chosen as C_e = 1,0. Only in case of exceptional circumstances where the roof is located either in open terrain or in surroundings which represent shelter the exposure factor should be adjusted. If the building is placed in open terrain the roof is denoted as “windswept” and the exposure coefficient may be reduced to C_e = 0,8. In case the building is sheltered due to dense vegetation or due to adjacent higher buildings the exposure factor should be enhanced to C_e = 1,2.

The thermal coefficient is also set to C_t = 1,0 for the normal situation. Only where roofs of heated buildings are not or poorly insulated (glass roofs / thermal transmittance > 1 W/m²K) it is allowed to use a reduced factor C_t. It is planned to introduce recommendations for these reduction factors into the National Annexes.

1.5 Roof shape coefficients

Low wind velocities are sufficient to blow snow accumulations from a roof or to cause a drift of snow which could lead to a local enhancement of the snow load. Roof shape coefficients are needed for an adjustment of the ground snow load to a snow load on the roof taking into account these effects. Eurocode prEN 1991-1-3 gives a set of roof coefficients for a variety of roof geometries. For some roof shapes several load cases have to be taken into account because different load arrangements (with or without drifted snow) are possible. Then the most unfavourable load situation has to be chosen for the design.

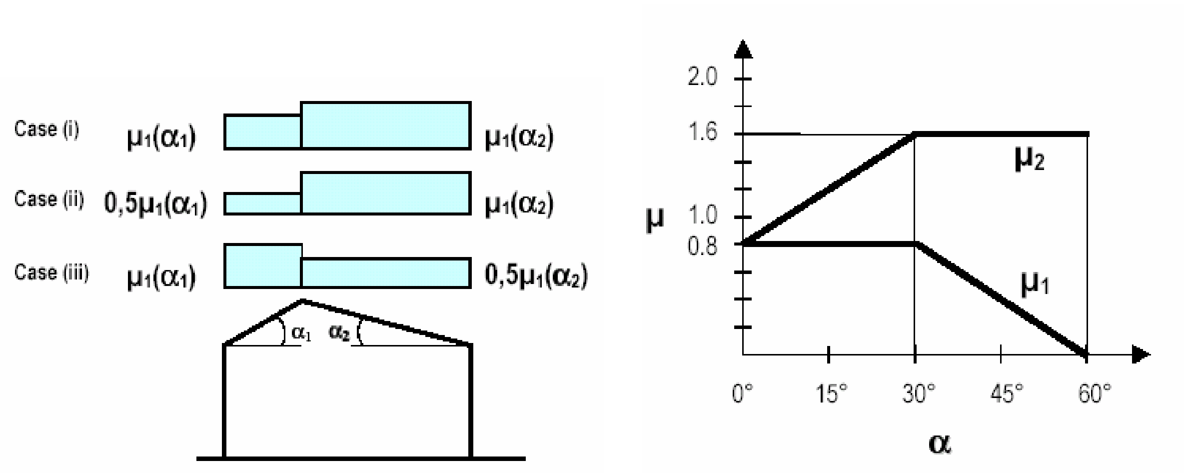


Figure 1. Example for roof shape coefficients depending on the roof angle [2]

1.6 Additional information

In addition to the standard load cases prEN 1991-1-3 gives also definitions for local effects caused by overhanging of snow at the edge of a roof which is not explained here in the presented overview.

Furthermore it is noted here that definitions are also given for accidential snow loads which could be evoked either by exceptional snow falls (e.g., as it happened in North Germany in 1979) or by exceptional snow accumulations causing unfavourable local effects at the roof structure.

It should be pointed out that prEN 1991-1-3 allows to replace normative values of the characteristic ground snow loads by values determined from a statistical analysis of measured data. Of course, this is possible only in case sufficient measured data for the relevant location are available.

2 EXAMPLE

A building with shed roof is given. It is assumed that this building is located in Sweden, snow load zone 2, on a height above sea level of 300 m. The surroundings of the building represent normal conditions, so that the roof can not be denoted as "wind swept" or "wind sheltered". An effective heat insulation is applied on the roof and therefore the thermal coefficient C_t = 1,0 has to be used for the calculation.

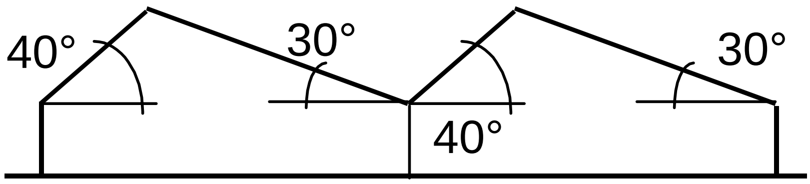


Figure 2. Example for a building with shed roof

For Sweden the following characteristic value for the ground snow load on height 300 m is obtained in case of snow load zone 2:

$$s_k = 0{,}790 \cdot Z + 0{,}375 + \frac{A}{336} = 0{,}790 \cdot 2 + 0{,}375 + \frac{300}{336} = 2{,}85 \text{ kN/m}^2$$

For the determination of roof shape coefficients for shed roofs the more unfavourable case of two load cases has to be applied:

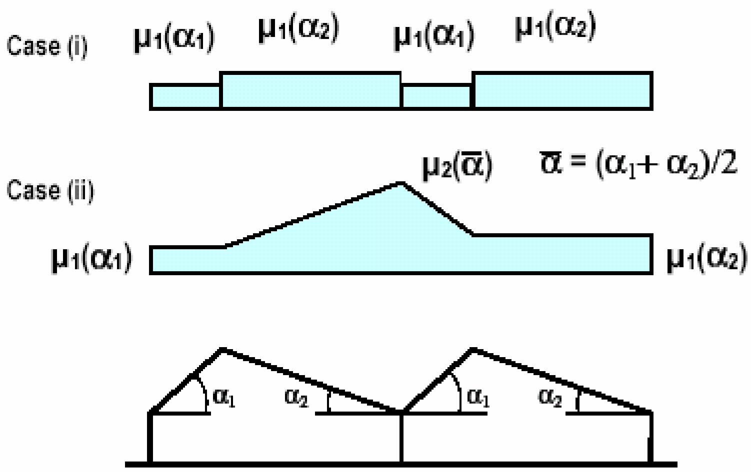


Figure 3. Determination of roof shape coefficients for shed roofs [2]

Here the following values of shape coefficients and snow loads on the roof are obtained, see figure 1:

$$\mu_1\,(\alpha_1) = 0{,}53 \rightarrow s = 2{,}85 \cdot 0{,}53 = 1{,}51 \text{ kN/m}^2$$
$$\mu_1\,(\alpha_2) = 0{,}80 \rightarrow s = 2{,}85 \cdot 0{,}80 = 2{,}28 \text{ kN/m}^2$$
$$\mu_2\,(\overline{\alpha}\,) = 1{,}60 \rightarrow s = 2{,}85 \cdot 1{,}60 = 4{,}56 \text{ kN/m}^2$$

These values lead to the two relevant load cases given in figure 4:

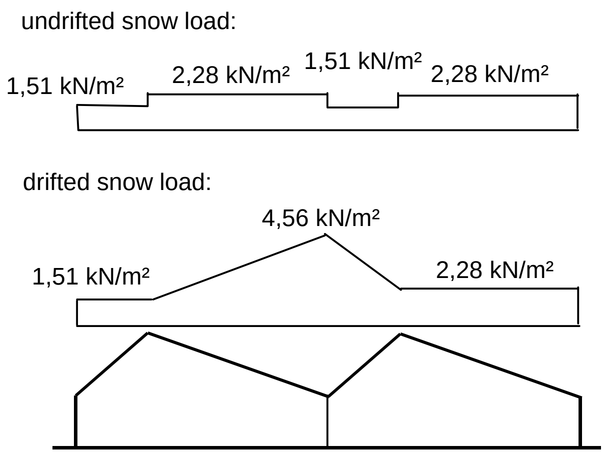


Figure 4. Load arrangements for the case of drifted and undrifted snow on the roof

For the determination of characteristic snow loads and roof shape coefficients see also the excel-sheet ‘snowloads.xls’.

REFERENCES

[1] EN 1990 Basis of structural design. European Committee for Standardisation, CEN/TC 250, April 2002.

[2] prEN 1991-1-3 Actions on structures – General actions – Snow loads. European Committee for Standardisation, CEN/TC 250, April 2002.

[3] L. Sanpaolesi: Scientific support activity in the field of structural stability of civil engineering works – snow loads. European Commission, 1998.

CHAPTER III - WIND ACTIONS

Ton Vrouwenvelder[1], Henri Steenbergen[1], Oliver Kraus[2], Christiane Butz[2]

[1]TNO Building and Construction Research, The Netherlands
[2]Institute for Steel Construction, RWTH Aachen, Germany

Summary

In this chapter the determination of wind effects on structures according to the European draft standard EN 1991-1-4 is explained. The following practical examples show the application of operational rules for the determination of the wind pressure and wind actions on:

- a simple rectangular multi-storey frame building with flat roof
- a simple rectangular building with duopitch roof
- a rectangular tower block
- a glazing panel
- an industrial hall
- a cylindrical tower block
-

The excel-sheet 'windloads.xls' helps to determine characteristic wind velocity and pressure coefficients according to EN 1991-1-4 for buildings.

1 INTRODUCTION

1.1 General

General principles for the classification of actions on structures, including environmental impacts and their modelling in verification of structural reliability, are introduced in Eurocode 0, EN 1990 [2]. In particular EN 1990 [2] defines various representative values (characteristic and design values) used in design calculation (see also Designers' Handbook [3]). Detailed description of individual types of actions is given in various parts of Eurocode 1, EN 1991.

Part 1.4 of EN 1991 [1] covers wind actions and gives rules and values for the following topics:

- design situations;
- nature and classification of wind actions;
- wind velocity and velocity pressure;
- effect of wind on the structure;
- pressure and force coefficients.

For the effect of wind on the structure three types of response are covered. From these three types of response only the quasi-static response will be discussed in this chapter. Dynamic and aeroelastic responses are not covered herein because the calculation of these types of response is not needed for most types of buildings.

1.2 Background documents

Part 1.4 of EN 1991 [1] was initially based on an ISO TC98 document and was developed using inputs from the latest wind engineering practice introduced into national standards in European countries.

1.3 Status

Part 1.4 of EN 1991 [1] is not yet available. Hence, this paper is based on its predecessor prEN 1991-1-4 according to the draft of June 2004.

2 BASIS OF APPLICATION

2.1 Characteristics of prEN 1991-1-4

Part 1.4 of EN 1991 [1] enables the assessment of wind actions for the structural design of buildings and civil engineering structures up to a height of 200 m. The wind actions are given for the whole or parts of the structure, e.g. components, cladding units and their fixings.

Part 1.4 of EN 1991 [1] does not cover all possible aspects of wind actions. Special conditions which are not common for most types of structures, like local thermal effects on the characteristic wind, torsional vibrations, vibrations from transverse wind turbulence, vibrations with more than one relevant fundamental mode shape and some aeroelastic effects are not covered. Wind actions on structures like lattice towers, tall buildings with a central core, cable stayed and suspension bridges, guyed masts and offshore structures are not fully covered. If possible, a reference is made to other more specific codes. Otherwise specialist's advice is needed and wind tunnel experiments might be useful.

The application range of the European wind load standard is much larger than compared to some older national standards. Particularly the specification of wind loads for high-rise buildings and for structures which are susceptible to wind induced vibrations is described in detail.

Table 1. Application range of prEN 1991-1-4

Structure	Limitation of EN 1991-1-4
Buildings	Height: max. 200 m
Viaducts	Span: max. 200 m
Suspension Bridge/Stay Cable Bridge	Particular Investigations
Pedestrian Bridge	Span: max. 30 m

For this broad application range an extensive set of pressure coefficients and force coefficients as well as a sufficiently accurate description of the characteristics of the natural wind must be provided. Consequently these advantages lead to a more complex application of the new European wind load standard compared to older national standards. Due to its comprehensiveness the modern European standard demands more skills to determine the relevant wind loading than the simple rules of some national standards, particularly in case of small, simple structures.

Table 2. Advantages and Disadvantages of prEN 1991-1-4

Comparison of prEN 1991-1-4 with national standards	
+	New extensive set of pressure and force coefficients
+	More accurate description of wind loads
+	Suitable for lightweight structures
+	Additional investigations of experts is in many cases avoidable
-	Application demands more skills than simple standards
-	Difficult identification of the relevant information in case of simple structures

2.2 General principles

According to EN 1990 [2] actions shall/should be classified according to their variation in time, spatial variation, origin and their nature and/or structural response.

Considering their variation in time and space, wind actions are classified as variable fixed actions. It means that the wind actions are not always present and the wind actions have for each considered wind direction fixed distributions along the structure. The classification to the origin of wind actions can be direct as well as indirect: Direct for external surfaces and internal surfaces of open structures; indirect for internal surfaces of enclosed structures.

The last classification of wind actions is done according to their nature and/or structural response. This classification depends on the response of the structure due to wind actions. For wind actions the following responses are covered in Part 1.4 of EN 1991 [1]:

- quasi-static response
- dynamic and aeroelastic response

For structures when the lowest natural frequency is so high that wind actions in resonance with the structure are insignificant, the wind action is called quasi-static. The dynamic response is significant for structures, if the turbulence (or gust effect) of the wind is in resonance with the structure's natural frequency whereas the aeroelastic response occurs if an interaction between the movement of a particular structure and the circumfluent wind flow exists.

Due to these different types of wind loading the European wind load standard is subdivided into two parts. The main part gives information and load assumptions for common structures which are not susceptible to wind induced vibrations, i.e. here the rules for determining the quasi-static wind loading are defined. In the annex to prEN 1991-1-4 rules are given for the determination of wind loading for slender, lightweight structures susceptible to vibrations due to turbulence and for structures susceptible to aeroelastic effects due to vortex shedding (e.g. steel chimneys) and due to galloping and flutter (e.g. bridge decks). Concerning the dynamic response due to turbulence prEN 1991-1-4 covers only the along wind vibration response of a fundamental mode shape with constant sign. Structural response of higher vibration modes can not be taken into account using prEN 1991-1-4.

The following text and examples deal only with topics of the main part of prEN 1991-1-4, i.e. only the quasi-static response will be discussed herein. According to prEN 1991-1-4 the quasi-static response needs to be calculated for all structures, while for most buildings it is not needed to take account of the dynamic and aeroelastic response.

If special considerations are necessary for dynamic and aeroelastic response wind tunnel tests should be performed.

3 WIND VELOCITY AND WIND PRESSURE

3.1 General

One of the main parameters in the determination of wind actions on structures is the characteristic peak velocity pressure q_p. This parameter is in fact the characteristic pressure due to the wind velocity of the undisturbed wind field. The peak wind velocity accounts for the mean wind velocity and a turbulence component. The characteristic peak velocity pressure q_p is influenced by the regional wind climate, local factors (e.g. terrain roughness and orography/terrain topography) and the height above terrain.

3.2 Wind climate

The wind climate for different regions/countries in Europe is described by values related to the characteristic 10 minutes mean wind velocity at 10 m above ground of a terrain with low vegetation (terrain category II). These characteristic values correspond to annual probabilities of exceedence of 0,02 which corresponds to a return period of 50 years. In prEN 1991-1-4 this variable is denoted as the *fundamental value of the basic wind velocity* $v_{b,0}$. Values for the wind climate in different regions/countries are given in the National Annex to prEN 1991-1-4. The *basic wind velocity* v_b in a region in Europe can be determined with the formula:

$$v_b = c_{dir}\, c_{season}\, v_{b,0} \tag{1}$$

where: $v_{b,0}$ = fundamental value of basic wind velocity
v_b = basic wind velocity
c_{dir} = directional factor
c_{season} = seasonal factor

The directional factor c_{dir} accounts for the fact that for particular wind directions the velocity v_b could be decreased, whereas the seasonal factor c_{season} takes into account that in case of temporary structures for particular periods the probability of occurrence of high wind velocities is relatively low. For simplification the directional factor c_{dir} and the seasonal factor c_{season} are in general equal to 1,0. A global overlook of the wind maps for Europe is given in Figure 1 (which is not included in prEN 1991-1-4).

The following relationship exists between the basic velocity and the basic pressure:

$$q_b = \rho/2 \cdot v_b^2 \tag{2}$$

where: ρ = density of air (can be set to 1,25 kg/m^3)

Thus this value represents the mean velocity pressure (averaging interval 10 min.), i.e. the turbulence of the wind is not included, at a reference height of 10 m in open terrain with a return period of 50 years.

The basic value of the velocity pressure has to be transformed into the value at the reference height of the considered structure. Velocity at a relevant height and the gustiness of the wind depend on the terrain roughness. The roughness factor describing the variation of the speed with height has to be determined in order to obtain the mean wind speed at the relevant height:

$$v_m(z) = c_r(z) \cdot c_o(z) \cdot v_b \tag{3}$$

where: $v_m(z)$ = mean velocity
$c_r(z)$ = roughness factor
$c_o(z)$ = orography factor (usually taken as 1,0)

In case of structures that are located on elevations like hills etc. the increase of the velocity can be taken into account by defining a particular orography factor $c_o(z)$. In general this factor is set to 1,0.

The roughness factor related to a minimum height z_{min} for the calculation is:

$$c_r(z) = k_r \cdot \ln(z/z_0)\ ,\ \text{but } z \geq z_{min} \tag{4}$$

$$k_r = 0{,}19 \cdot (z_0/z_{0,II})^{0,07} \tag{5}$$

where: k_r = terrain factor
z_0 = roughness length
z_{min} = minimum height

Figure 1. Overlook of the European wind map for basic wind velocities $v_{b,0}$ (indicative values only)

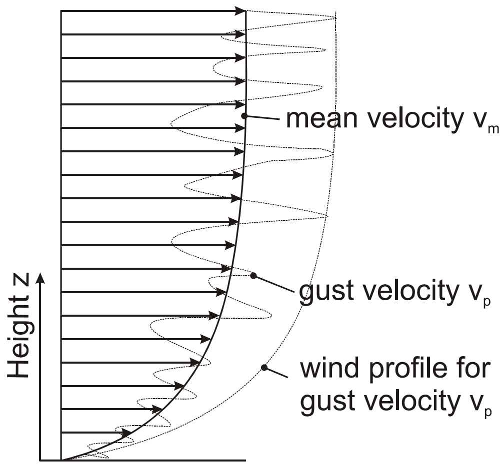


Figure 2. Variation of wind velocity depending on height *z*

The input parameters of the formulae above are defined in dependence of the relevant terrain roughness:

Table 3. Terrain categories

Terrain category	Characteristics of the terrain	z_0 [m]	z_{min} [m]
0	sea or coastal area	0,003	1,0
I	lakes; no obstacles	0,01	1,0
II	low vegetation; isolated obstacles with distances of at least 20 times of obstacle heights	0,05	2,0
III	regular vegetation; forests; suburbs; villages	0,3	5,0
IV	at least 15% of the surface covered with buildings with average height of at least 15 m	1,0	10,0

The gust velocity (or peak velocity) $v_p(z)$ at the reference height of the considered terrain category is calculated with the mean velocity and the gust factor G:

$$v_p(z) = v_m(z) \cdot G \tag{6}$$

where:

$$G = \sqrt{c_e(z)} = \sqrt{1 + 7 \cdot I_v(z)} = \sqrt{1 + 7 \cdot \frac{\sigma_v(z)}{v_m(z)}} = \sqrt{1 + \frac{7 \cdot k_I}{c_o(z) \cdot \ln(z/z_0)}} \quad \textit{with} \quad z \geq z_{min} \tag{7}$$

where: k_I = turbulence factor (usually taken as 1,0)

As indicated above the gust factor represents the square root of the exposure coefficient. Hence the following expression for the gust pressure in the relevant reference height is obtained:

$$q_p(z) = q_b(z) \cdot [c_r(z)]^2 \cdot [c_o(z)]^2 \cdot \left[1 + \frac{7 \cdot k_I}{c_o(z) \cdot \ln(z/z_0)}\right] \tag{8}$$

which is simplified in case of the general assumption of $c_o(z) = k_I = 1{,}0$:

$$\underbrace{q_p(z)}_{\text{peak pressure}} = \underbrace{q_b}_{\text{basic pressure}} \cdot \underbrace{[c_r(z)]}_{\text{wind profile}}{}^2 \cdot \underbrace{\left[1+\frac{7}{\ln(z/z_0)}\right]}_{\text{squared gust factor}} \tag{9}$$

4 WIND PRESSURE FOR DETERMINATION OF QUASI-STATIC RESPONSE

4.1 General

In prEN 1991-1-4 regulations are given not only for the determination of the external wind pressure w_e on the structure's cladding but also for the application of the internal wind pressure w_i in case of openings in the cladding. Both types of wind pressure depend on the geometry of the considered structure. In addition to that the internal pressure varies with the permeability of the building:

$$w_e = q_{b,0} \cdot c_e(z_e) \cdot c_{pe} \tag{10}$$

$$w_i = q_{b,0} \cdot c_e(z_i) \cdot c_{pi} \tag{11}$$

where: w_e = external pressure
w_i = internal pressure
$q_{b,0}$ = basic value of velocity pressure
$c_e(z)$ = exposure factor
$c_{pe}(z)$ = external pressure coefficient
$c_{pi}(z)$ = internal pressure coefficient
z_e; z_i = reference height of the considered building

Both, external and internal wind pressure, are defined as acting orthogonally to the surface of the building.

4.2 Pressure coefficients

Due to the fact that an increase of the gust pressure is related to a decrease of the surface area loaded by the corresponding gust in prEN 1991-1-4 external pressure coefficients are given as a function of the size of the relevant cladding: In some cases the pressure coefficients are decreased for smaller sizes.

For the external wind load as well as for the internal wind load the term "pressure" includes also suction: a positive wind load stand for pressure whereas a negative wind load stands for suction on the surface. If both, internal and external wind pressure, have to be applied, they have to be superposed if their effect is unfavourable for the design.

The pressure coefficient c_{pe} for the external pressure has to be chosen in dependence of the structure's geometry. The pressure coefficient c_{pi} has to be applied depending on the permeability of the structure's surface. If it is not possible to estimate the building's permeability then c_{pi} should be taken as the more onerous of +0,2 and –0,3 (see 7.2.9 (6) NOTE 2).

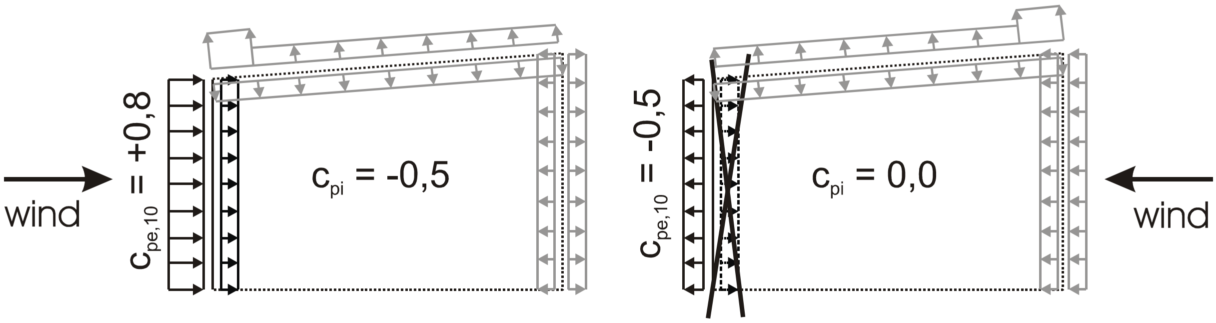


Figure 3. Wind load on cladding:
Superposition of external pressure with unfavourable internal pressure:
wind load on cladding (when cladding windward, c_{pi} is unfavourable - when cladding is leeward, c_{pi} has to be taken as 0,0)

5 DETERMINATION OF THE WIND INDUCED FORCE

5.1 General

The resulting wind force can be determined by integration of the wind pressure over the whole surface or by applying appropriate force coefficients that are given in prEN 1991-1-4 for different kinds of structures. It is noted here, that for many structures force coefficients result into more accurate results than integration of pressure coefficients. The wind force F_w is determined using the following equation:

$$F_w = c_s\, c_d \cdot c_f \cdot q_p(z_e) \cdot A_{ref} \tag{11}$$

where: F_w = wind force
c_s = size factor
c_d = dynamic factor for structures susceptible to wind induced vibrations
c_f = force coefficient
A_{ref} = reference area
z_e = reference height (maximum height of the structure above ground level)

For structures which are not susceptible to turbulence induced vibrations, the quasi-static structural response is crucial. Then the size factor and the dynamic factor are fixed at $c_s\, c_d = 1{,}00$. As mentioned above, only this quasi-static case is treated in this text.

Particularly in case of complex structures like e.g. trusses or in case of structures where the pressure distribution on the surface depends on the Reynolds Number (i.e. the ratio of the inertia force to the frictional force of the flow) as it is for curved shapes like cylinders, spheres, the application of force coefficients instead of integration of pressure coefficients is recommendable. Then the consideration of force coefficients in the calculation leads to a time saving and more accurate determination of wind effects.

5.2 Force coefficients

In prEN 1991-1-4 besides values for particular constructions a distinction is made between force coefficients for elements with rectangular/polygonal shapes and with curved shapes.
For rectangular/polygonal shapes force coefficients are determined by:

$$c_f = c_{f,0} \cdot \Psi_r \cdot \Psi_\lambda \tag{12}$$

where: $c_{f,0}$ = force coefficient for shapes with sharp corners
ψ_r = reduction factor for rounded corners at rectangular structures
ψ_λ = end-effect factor

The ψ_r-value makes allowance for the fact that the wind pressure at rounded corners is lower than at sharp corners that represents a higher obstacle to the flow. In turn it leads to a decreased wind force at structures with rounded corners. Furthermore, independend on the shape of the corner, with the ψ_λ-factor it is taken into account that at the top of structures the resulting wind pressure is lower than the average value at the inner surface. This effect, i.e. this reduction, decreases relatively with higher slenderness of structures.

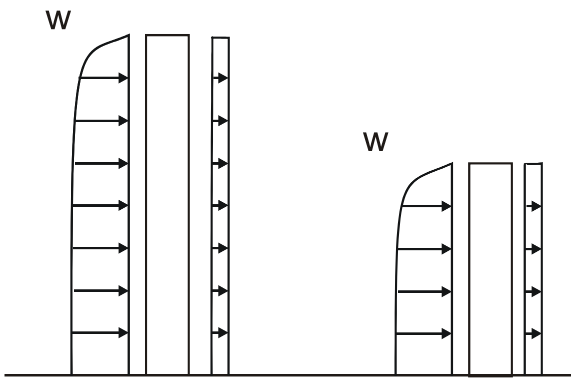


Figure 4. Relative influence of the flow at the edge of the structure on the resulting wind force is for slender structures smaller than for compact structures

The force coefficient for cylinders is defined as:

$$c_f = c_{f,0} \cdot \psi_\lambda \tag{13}$$

where: $c_{f,0}$ = force coefficient for cylinders without free-end flow
ψ_λ = end-effect factor

The force coefficient for spheres has to be determined according to the relevant Reynolds Number $Re = b \cdot v(z_e) / \nu$ (where b = diameter, ν = viscosity of air – $15 \cdot 10^{-6}$ m^2/s).

In addition to the general shapes mentioned above, force coefficients are given for structural elements like:

Table 4: Particular structural elements force coefficients are available for

Structure	c_f depends on
Lattice Structures	End-effect, *Re*-No., solidity ratio*, cross section of members
Flags	End-effect, mass per unit area
Bridges	Ratio height of section/width of section, inclination to the vertical, shape of section, permeability of road restraint system
Sharp edged sections	End-effect

*ratio of impermeable area and overall area of a structure

6 EXAMPLES

6.1 Wind pressure on an industrial hall

The wind pressure is relevant for the design of frames of the following typical structure for an industrial hall and has to be determined:

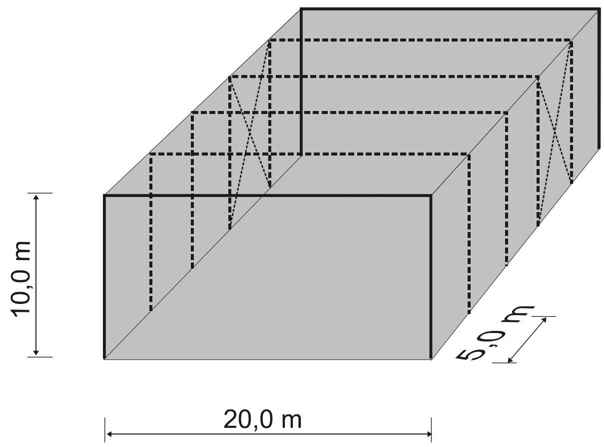


Figure 5. Example: system of an industrial hall

The main structure consists of six frames with a spacing of 5,0 m. The length of the hall is 25,0 m. Each frame is 10,0 m high and has got a bay width of 20,0 m. It is assumed, that in case of a storm there is no opening in the surface of the hall, so that the internal pressure can be neglected, i.e. $c_{pi} = 0{,}0$.

The hall shall be erected in an industrial area in Aachen, Germany corresponding to wind load zone II and related to a basic value of velocity pressure of

$$q_{b,0} = 0{,}39 \text{ kN/m}^2$$

With the input data according to terrain category III (see table 3) the following roughness coefficient is determined with the reference height 10 m:

$$k_r = 0{,}19 \cdot (0{,}3/0{,}05)^{0{,}07} = 0{,}22$$

$$c_r(z) = 0{,}22 \cdot \ln(10{,}0/0{,}3) = 0{,}77$$

Furthermore the exposure factor is:

$$c_e(10{,}0) = 1 + \frac{7{,}0}{1{,}0 \cdot \ln(10{,}0/0{,}3)} = 3{,}00$$

The peak velocity pressure is:

$$q_p(10{,}0) = 0{,}77^2 \cdot 3{,}00 \cdot 0{,}39 = 0{,}69 \text{ kN/m}^2$$

From **Table 7.1** ("External pressure coefficients for vertical walls of rectangular plan buildings") and **Table 7.2** ("External pressure coefficients for flat roofs") of prEN 1991-1-4 the following application of c_{pe}-values are obtained:

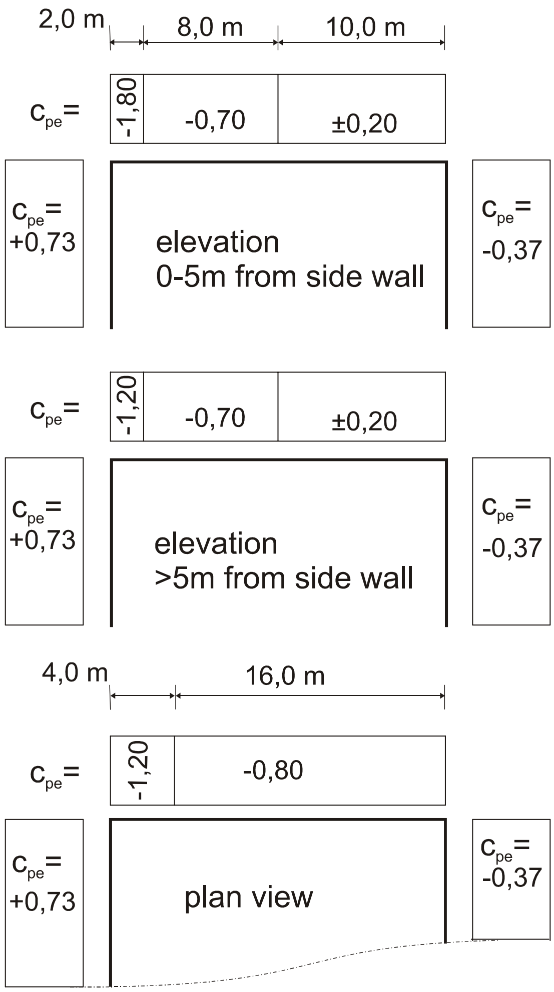


Figure 6. External pressure coefficients according to prEN 1991-1-4

The following characteristic values of the wind loading are obtained from the combination of the pressure coefficients with the peak velocity pressure:

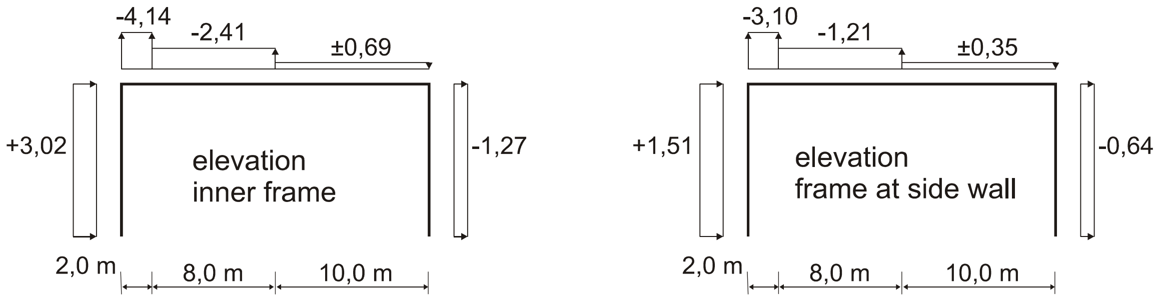


Figure 7. Characteristic values of wind loading in [kN/m] for inner frames (left) and for frames at the side wall (right)

6.2 Wind pressure on a rectangular building with flat roof

A simple rectangular building with flat roof is shown in Figure 6. The dimensions of the building are: height 12 m, width 30 m and depth 15 m. The building is situated in flat terrain of terrain category II. The basic wind velocity $v_{b,0}$ is equal to 26 m/s. The wind forces on the main structure for the wind direction as given in figure 6 will be considered in detail.

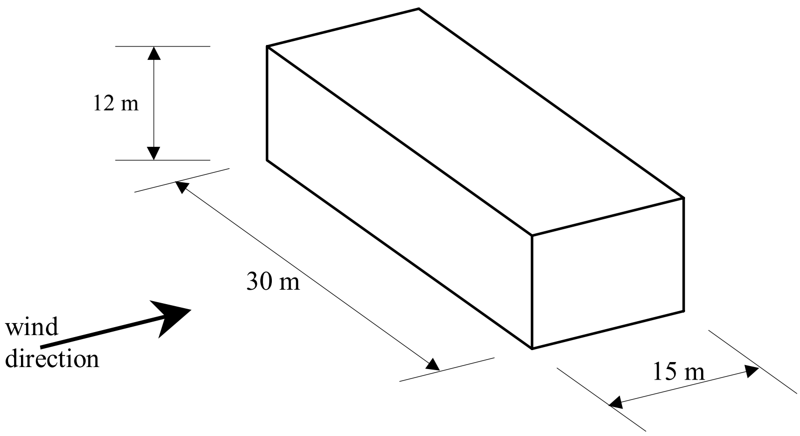


Figure 8. Simple rectangular building with flat roof

The wind forces on the building's main structure in case of a quasi-static response have to be calculated according to section 5.3 of prEN 1991-1-4. The coefficients for the distribution of wind forces on buildings are given as pressure coefficients. The friction forces F_{fr} are neglected. Therefore the resulting wind force, F_w, on the structure is determined with the following equations:

For external pressures: $$F_{w,e} = c_s\, c_d\, \Sigma\, w_e(z)\, A_{ref}(z) \qquad (13)$$

For internal pressures: $$F_{w,i} = \Sigma\, w_i(z)\, A_{ref}(z) \qquad (14)$$

The summation must be carried out vectorial by taking into account the spatial distribution of the wind pressures $w_e(z)$ and $w_i(z)$. Reference is made to clause 5.1 of prEN 1991-1-4 for the calculation of the wind pressures. This results in the following equations for the wind forces:

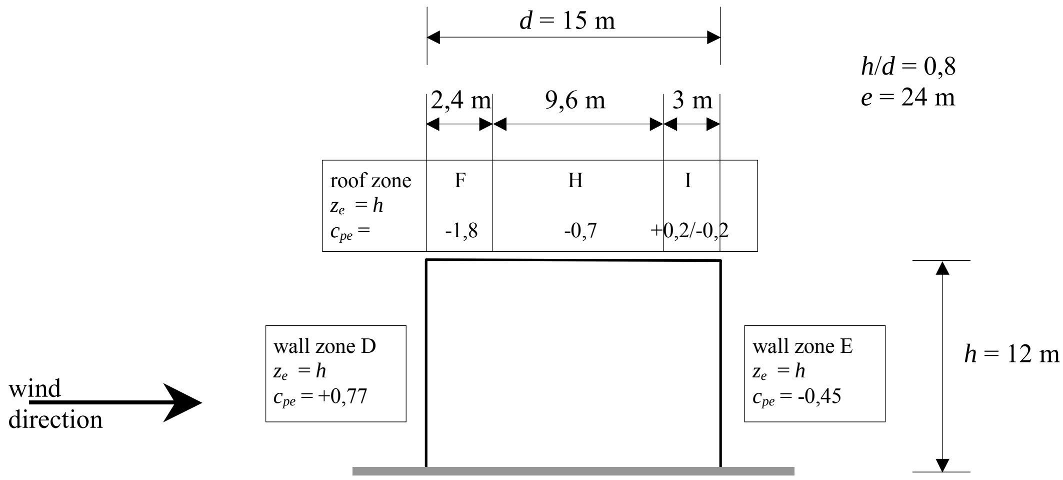
For external pressures: $$F_{w,e} = c_s\, c_d\, \Sigma\, q_p(z_e)\, c_{pe}\, A_{ref}(z) \qquad (15)$$

For internal pressures: $$F_{w,i} = \Sigma\, q_p(z_i)\, c_{pi}\, A_{ref}(z) \qquad (16)$$

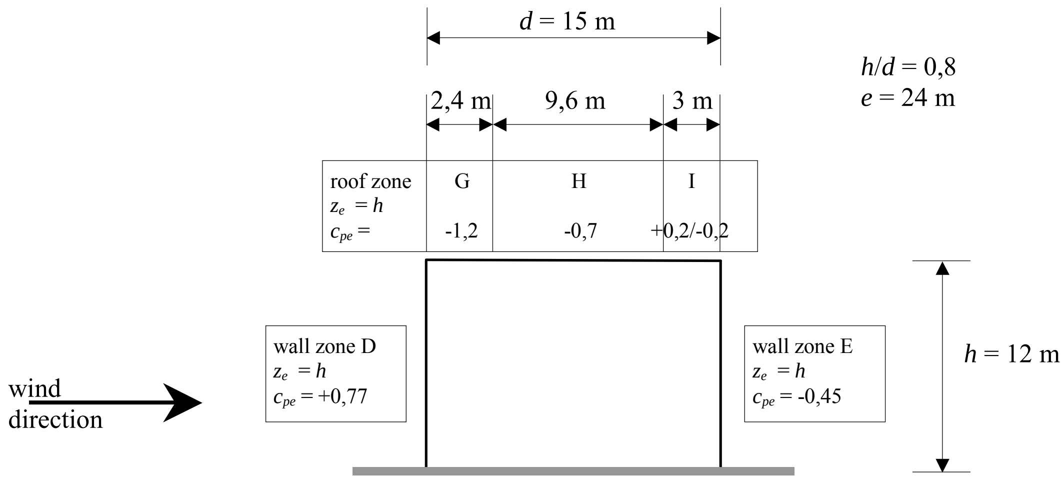
$c_s\, c_d$ is the structural factor. Clause 6.2 (1) a) states that for buildings with a height of less than 15 m the value of $c_s\, c_d$ may be taken as 1 which is conservative.

The pressure coefficients c_{pe} and c_{pi} are given in section 7 of prEN 1991-1-4. In the general clause for buildings it can be seen that the wind forces needs to be calculated in four orthogonal wind directions perpendicular to the side walls, because the pressure coefficients represent the most unfavourable values in a range of wind directions. Also it is given that the pressure coefficient $c_{pe} = c_{pe,10}$ because the loaded area A for the main structure is larger than 10 m^2.

The external pressure coefficients and accompanying reference height can now be determined for the walls and the flat roof. The results are given in Figure 9 for the wind direction given in Figure 8.



a) Cross section with distance to side wall ≤ $e/4$ = 6 m



b) Cross section with distance to both side walls > $e/4$ = 6 m

Figure 9. External pressure coefficients and accompanying reference heights

For the determination of the resulting force on the main structure the lack of correlation between the windward and leeward side (which means that the peak wind pressures do not appear at the same time) may be taken into account. For this example ($h/d = 0,8 < 1$) the resulting force from the external pressures on the walls may be multiplied by 0,85 for the verification of the overall stability.

The internal pressure coefficient and the accompanying reference height are determined in the next step. Assuming uniformly distributed permeability the ratio μ is 0,74 (see chapter 3.2). The results are given in Figure 10. For the reference height the conservative value $z_i = z_e$ is chosen.

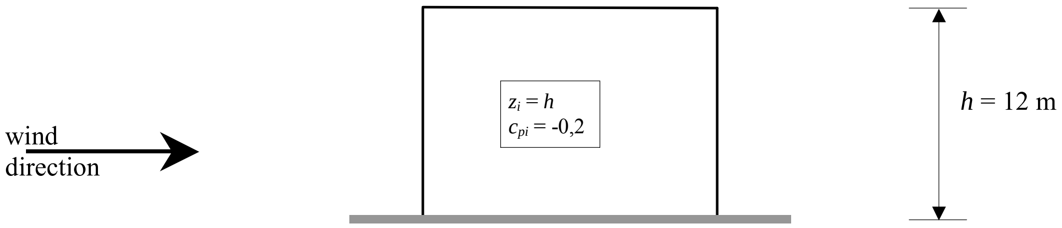


Figure 10. Internal pressure coefficients and accompanying reference height

For all the pressure coefficients the reference height is taken equal to $z_e = z_i$ = 12 m. The characteristic peak velocity pressure is q_p(z = 12 m) for flat terrain of terrain category II with $v_{b,0}$ = 26 m/s and is equal to q_p = 1043 N/m^2.

If the distance between the frames is equal to 7,5 m, the distributed forces on the frame are according to Figure 11.

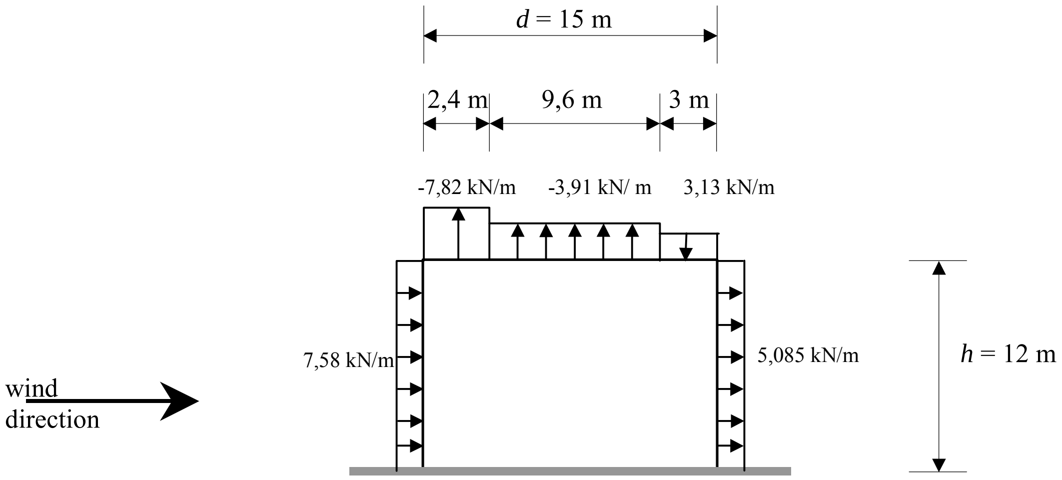


Figure 11. Wind force on the central frames (not influenced by roof zone F)

6.3 Simple rectangular building with duopitch roof

A simple rectangular building with duopitch roof is shown in Figure 12. The dimensions of the building are: Ridge height 6 m, gutter height 2 m, width 30 m and depth 15 m. The building is situated in flat terrain of terrain category II in an area where the basic wind velocity $v_{b,0}$ is equal to 26 m/s. For this building only the pressure coefficients for the wind direction as given in figure 12 will be considered in detail.

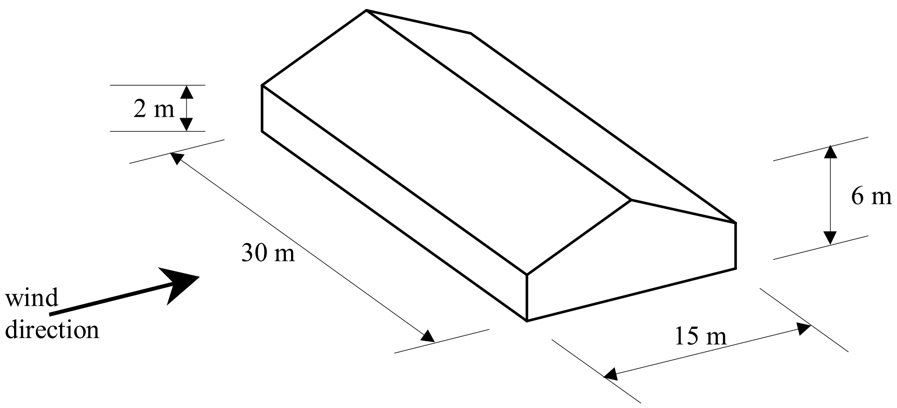


Figure 12. Simple rectangular building with duopitch roof

The procedure for the determination of the wind forces on the main structure (frames) of the building with duopitch roof is nearly equal to the previous example. Only the pressure coefficients and reference heights differ.

The external pressure coefficients and accompanying reference height are presented in Figure 13.

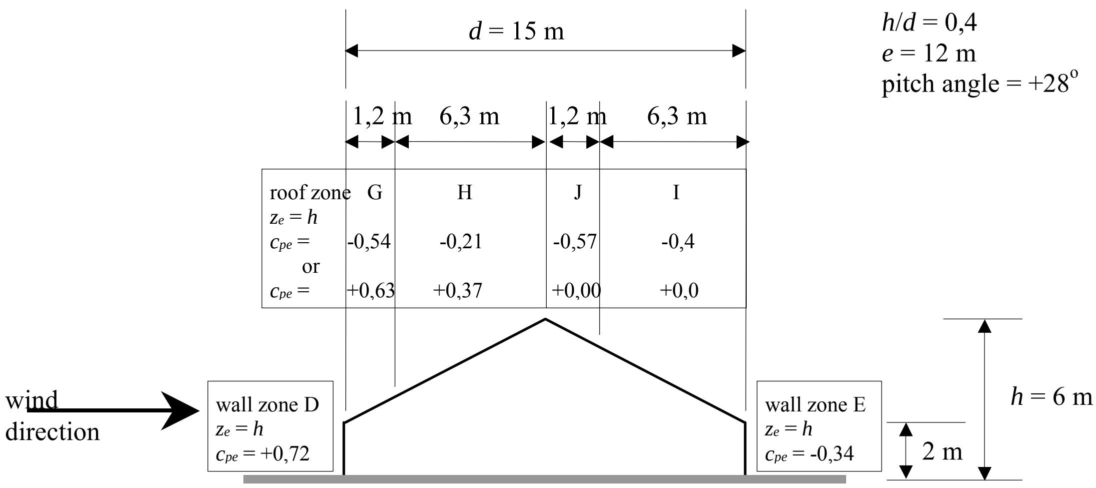


Figure 13. Pressure coefficients and accompanying reference heights
(distance to side wall > 3 m)

The reference height for the wall is equal to the height of the building. For the pressure coefficients on the roof two sets of c_{pe} values are given. Combinations with partial suction and overpressure for the roof zones G and H need not be taken into account.

The internal pressure coefficient and accompanying reference height should be determined in the next step. For buildings with uniformly distributed permeability and $d/h = 2{,}5$ the internal pressure coefficient should be taken as $c_{pi} = -0{,}35$ or $c_{pi} = +0{,}25$.

For the given wind direction four loadcases due to wind should be considered. These are the two load case due to the external wind pressure (suction on first pitch or overpressure on first pitch) each combined with the two load cases due to internal pressure (suction or overpressure).

6.4 Wind force on a cylindrical tower block

The wind force which acts on the foundation of a tower block with circular plan has to be determined.

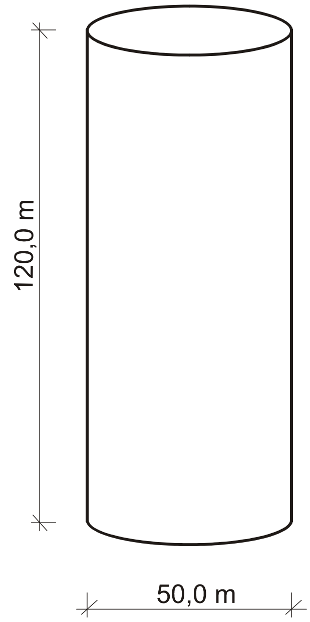


Figure 14. Example: tower block with circular plan

It is assumed that the building will be erected in Dresden, Germany corresponding to wind zone III, so that a basic value of the velocity pressure

$$q_{b,0} = 0{,}47 \text{ kN/m}^2$$

has to be applied.

The tower will be located in a suburb, which belongs to the terrain category II. For the reference height z_e = 120,0 m the following roughness coefficient ist obtained:

$$k_r = 0{,}19 \cdot (0{,}3/0{,}05)^{0{,}07} = 0{,}22$$

$$c_r(z) = 0{,}22 \cdot \ln(120{,}0/0{,}3) = 1{,}32$$

With table 3 the exposure factor is

$$c_e(120{,}0) = 1 + \frac{1{,}0}{1{,}0 \cdot \ln(120{,}0/0{,}3)} = 1{,}17$$

resulting into a peak velocity pressure of

$$q_p(120) = 1{,}32^2 \cdot 1{,}17 \cdot 0{,}47 = 0{,}96 \text{ kN/m}^2$$

For the calculation of the relevant force coefficient the Reynolds-Number has to be determined. The wind speed corresponding to the peak velocity pressure is:

$$v = (2 \cdot q_p(z) / \rho)^{1/2} = (2 \cdot 0{,}96 \cdot 1000 / 1{,}25)^{1/2} = 39 \text{ m/s}$$

resulting into a Reynolds-Number of

$$Re = 50\text{m} \cdot 39\text{m/s} / (15 \cdot 10^{-6}\text{m}^2\text{/s}) = 13 \cdot 10^7 \text{ [-]}$$

In addition to the Reynolds-Number the surface roughness has to be known. It is assumed that it consists of a glass façade. From table 7.13 of prEN 1991-1-4 the equivalent roughness is obtained:

$$k = 0{,}0015 \text{ mm}$$

With the Reynolds-Number and with the ratio

$$k/b = 3 \cdot 10^{-8}$$

from figure 7.28 of prEN 1991-1-4 the basic value of the force coefficient is:

$$c_{f,0} = 0{,}68$$

The reference area for the cylindrical structures is

$$A_{ref} = l \cdot b = 120 \cdot 50 = 6000 \text{ m}^2$$

In order to calculate the end-effect factor according to table 7.16 of prEN 1991-1-4 the smallest of the values $\lambda = 0{,}7 \cdot l/b = 1{,}68$ and $\lambda = 70$ has to be chosen as the effective slenderness. It serves as an input value of figure 7.36 of prEN 1991-1-4. From that diagram the following end-effect factor is derived:

$$\psi_\lambda = 0{,}65$$

Then the resulting characteristic value of the wind induced force on the foundation of the tower block is:

$$F_w = 0{,}96 \cdot 0{,}65 \cdot 6000 = 3744 \text{ kN}$$

6.5 Wind pressure on a rectangular tower block

A simple rectangular high-rise building with flat roof is shown in Figure 15. The dimensions of the building are: Height 55 m, width 20 m and depth 15 m. The building is situated in flat terrain of terrain category II in an area where the basic wind velocity $v_{b,0}$ is equal to 26 m/s. For this building the external wind pressure on the walls for the wind direction as given in figure 15 are considered in detail.

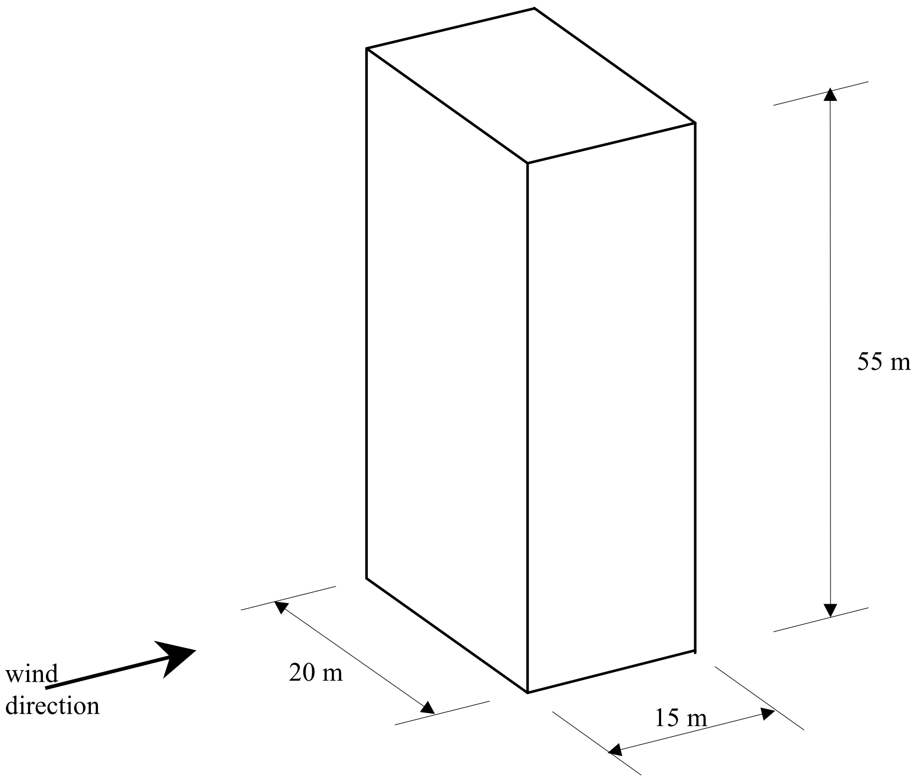


Figure 15. Simple rectangular high-rise building with flat roof

The wind forces on the main structure of a building in case of a quasi-static response need to be calculated according to section 5 of Part 1.4 of EN 1991 [1]. This results in the following equations for the wind forces (see example simple rectangular building with flat roof):

For external pressures: $$F_{w,e} = c_s\, c_d\, \Sigma\, q_p(z_e)\, c_{pe}\, A_{ref}(z) \quad (17)$$

For internal pressures: $$F_{w,i} = \Sigma\, q_p(z_i)\, c_{pi}\, A_{ref}(z) \quad (18)$$

$c_s c_d$ is the structural factor. According to section 6.2 the structural factor should be derived from section 6.3 and Annex B. For a multi-storey steel building (see figure B.4.1 in Annex B.4) the factor $c_s c_d$ is equal to 1,0 and gives a conservative approximation for the factor $c_s c_d$.

The external pressure coefficients and accompanying reference height can now be determined. The results are given in Figure 16 for the wind direction given in Figure 15. The resulting wind pressures are given in Figure 17.

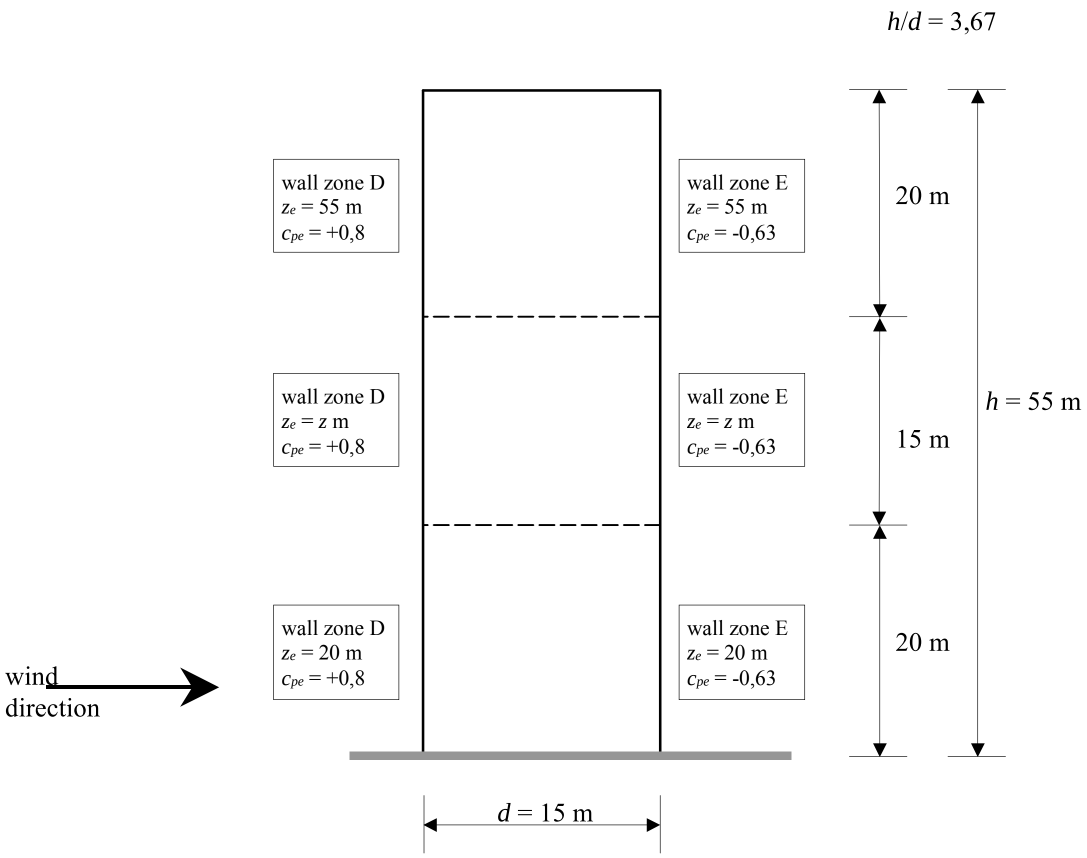


Figure 16. Pressure coefficients and accompanying reference heights

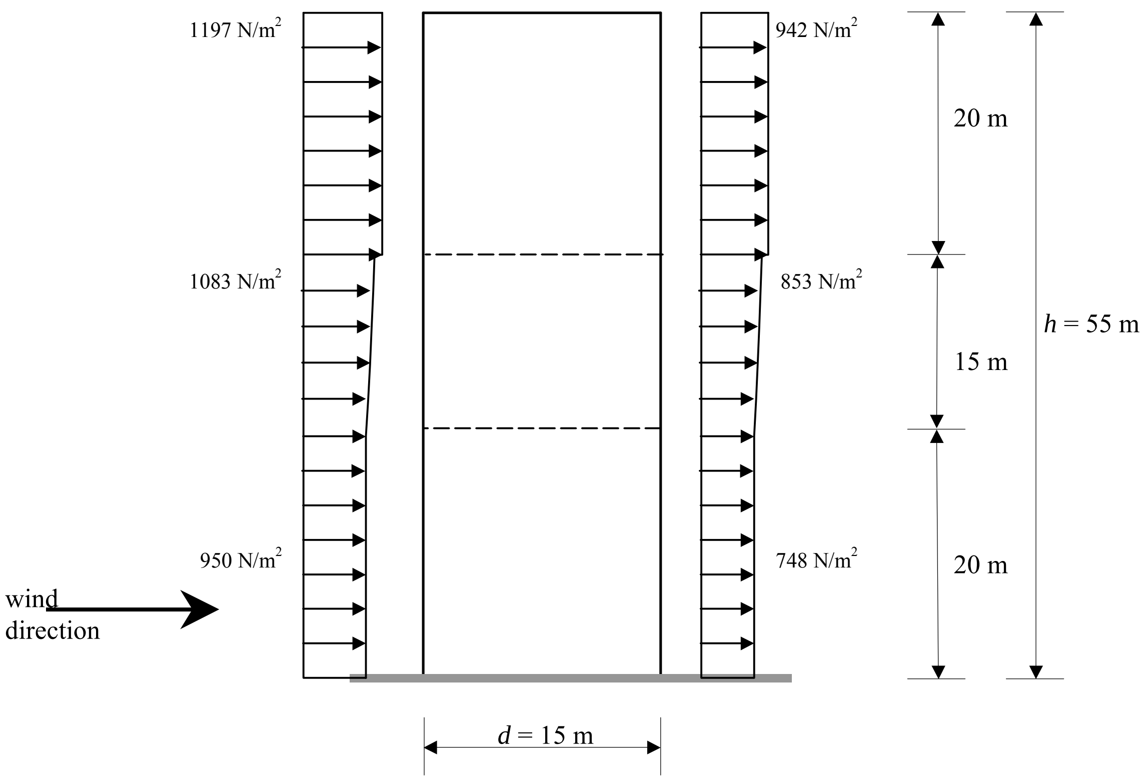


Figure 17. Distribution of the wind pressure on walls

6.6 Glazing panel

A glazing panel of a high-rise building is shown in Figure 17. The dimensions of the glazing panel are: Height 1,5 m and width 2 m. The glazing panel is part of the high building of the previous example.

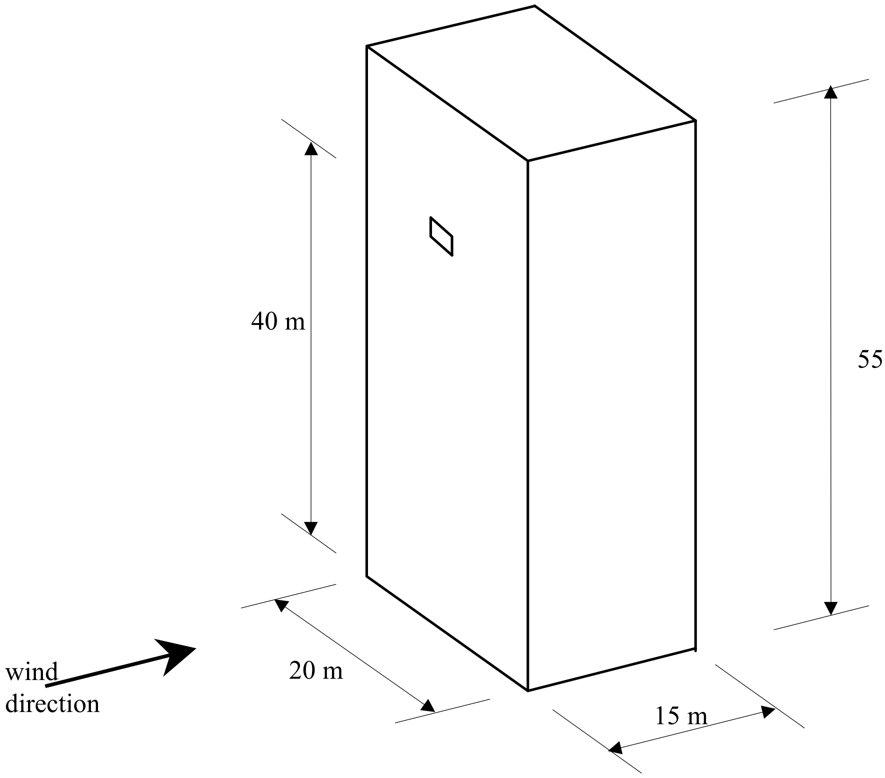


Figure 18. Glazing panel in a high building

According to Part 1.4 of EN 1991 [1] the wind actions on the glazing panel should be calculated as wind pressures according to section 5.1 of the wind code. Therefore the resulting wind actions on the glazing panel should be determined with the following equations:

For external pressures: $$w_e = q_p(z_e)\ c_{pe} \tag{19}$$

For internal pressures: $$w_i = q_p(z_i)\ c_{pi} \tag{20}$$

The pressure coefficients and accompanying reference height can now be determined according to section 7 from Part 1.4 of EN 1991 [1]. It follows that $c_{pe,10}$ = +0,8 and $c_{pe,1}$ = +1,0. This means the external pressure coefficient should be taken as: $c_{pe} = c_{pe,1} - (c_{pe,1} - c_{pe,10})\ {}^{10}\log A = 1{,}0 - 0{,}2\ {}^{10}\log 3 = +0{,}90$ with $z_e = h = 55$ m. The internal pressure coefficient should be taken as -0,4 or +0,3. The accompanying reference height should be taken equal to the mean height of the level considered. In this example this will be taken equal to: $z_i = 40$ m. The representative wind pressure on the glazing panel is equal to:

$$w = q_p(z_e)\ c_{pe} - q_p(z_i)\ c_{pi} = 1496 \cdot 0{,}90 + 1395 \cdot 0{,}40 = 1904 \text{ N/m}^2 \tag{21}$$

REFERENCES

[1] prEN 1991-1-4 Actions on Structures - Part 1-4: General Actions – Wind. European Committee for Standardisation, December 2003.

[2] EN 1990 Basis of structural design. European Committee for Standardization, CEN/TC 250, 2002.

[3] H. Gulvanessian, J.-A. Calgaro, M. Holický: Designer's Guide to EN 1990, Eurocode: Basis of Structural Design. Thomas Telford, London, 2002, ISBN: 07277 3011 8, 192 pp.

CHAPTER IV - THERMAL ACTIONS ON BUILDINGS

Luca Sanpaolesi, Stefano Colombini

Department of Structural Engineering, University of Pisa, Italy

Summary

In general thermal variations cause deformations in single structural elements as well as in the overall structure itself. If the structure is hyperstatic, a further consequence of thermal variations is the emergence of coactive stress states. Therefore, the effects of thermal variations may involve aspects of a structure's functionality as well as its safety. This chapter provides some instruments for evaluating the effects of thermal actions on buildings.

1 INTRODUCTION

1.1 Background documents

The background of this chapter is the regulations EN-1991-1-5 "Eurocode 1: Actions on structures – Part 1.5: General actions – Thermal Actions"

1.2 General principles

The problem of determining a given building's structural response to thermal actions is to be tackled first of all by defining suitable numerical models able to rationally reproduce as faithfully as possible the actual behaviour of thermal fields in structures. Of course, the thermal actions themselves must be accurately determined, as even the most precise model loses all meaning if accurate values of the actions affecting a structure are unavailable. In the case of thermal actions, such determinations are made via their statistic characterisation.

Any given instantaneous thermal field $T(y, z)$ acting in any given section can be decomposed into four separate components, as follows:

- a uniform temperature component;
- a component varying linearly around the *z-z* axis;
- a component varying linearly around the *y-y* axis;
- a self-equilibrating non-linear temperature distribution.

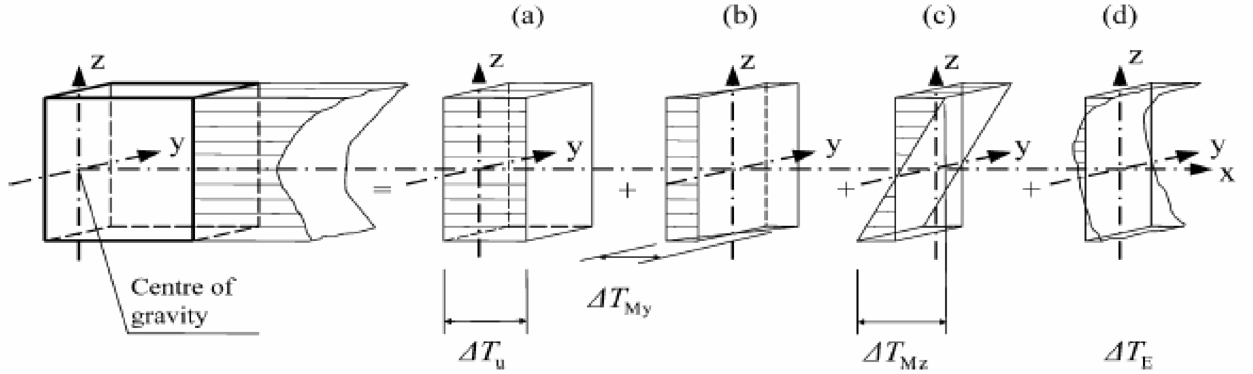


Figure 1. Diagrammatic representation of constituent components of a temperature profile.

Of the four thermal field components, the self-equilibrating one causes the least effects on a structure.

Strictly speaking, the components of the thermal actions at any given instant ‘t’ are given by the difference between the values at that instant and the corresponding values occurring at the initial instant, meaning the instant in which the structure is first placed under loads (e.g., the removal of formworks), shores and supports used during the stages of construction). According to the provisions of Annex A of Eurocode 1991-1-5, lacking more precise indications it can be assumed that T_0 = 10 °C.

2 EVALUATION OF THERMAL ACTIONS

The deformation values and the degree of consequent stresses induced by the thermal actions, above and beyond those due to the value of the actions themselves, are dependent on the geometry of the element considered and the physical properties of the materials employed in its construction. Clearly, if the structure contains materials with different values of the linear thermal expansion coefficient, this must be adequately accounted for in carrying out calculations.

The magnitude of the thermal actions and their distribution throughout the single elements of the structure are a function of numerous parameters, some quite difficult to interpret numerically. There are wide-scale parameters correlated with the climate of the geographical location of the construction and the consequent seasonal temperature variations. Then there are highly aleatory parameters, such as the presence of perturbations, which influence air temperatures and solar radiation, often with fluctuations on a daily scale or, in any event, over relatively short periods of time. Lastly, there are parameters strictly linked to the conditions of the particular building in question: the presence of other nearby structures that act as solar radiation screens, the building’s orientation, its total mass (and consequent thermal inertia), the properties of its finishings (i.e., their degree of solar energy absorption and thermal isolation) and the characteristics of the interior’s heating, air conditioning and ventilation.

Thermal actions must be considered to be variable and indirect actions. Regulations furnish characteristic values whose probability of being exceeded is 0,02, which is equivalent to a return period of 50 years. The fundamental quantities on which thermal actions are based are the extreme air temperatures value, that is, the maximum and minimum, in the shade at the building site. Such values are furnished by the National Meteorological Service of each member state. The shade air temperature is measured by a device known as a “Stevenson Screen”, which is simply a thermometer set in a white painted wooden box with louvered sides. The reason for shrouding the thermometer is to shield it from radiation by the sun, ground and surrounding objects during the day, prevent heat loss by radiation during the night, and finally protect it from precipitation, while at the same time, the louvering allows air to pass freely about it. Eurocode 1991-1-5 does not include maps for extreme temperature determinations: such task is left up to the National Meteorological Services. The document *ENV Thermal Actions* furnishes some indicative maps of the extreme temperatures for some CEN countries; those for Italy have been enclosed herein by way of example.

Figure 2. Map of minima shade air temperatures in Italy.

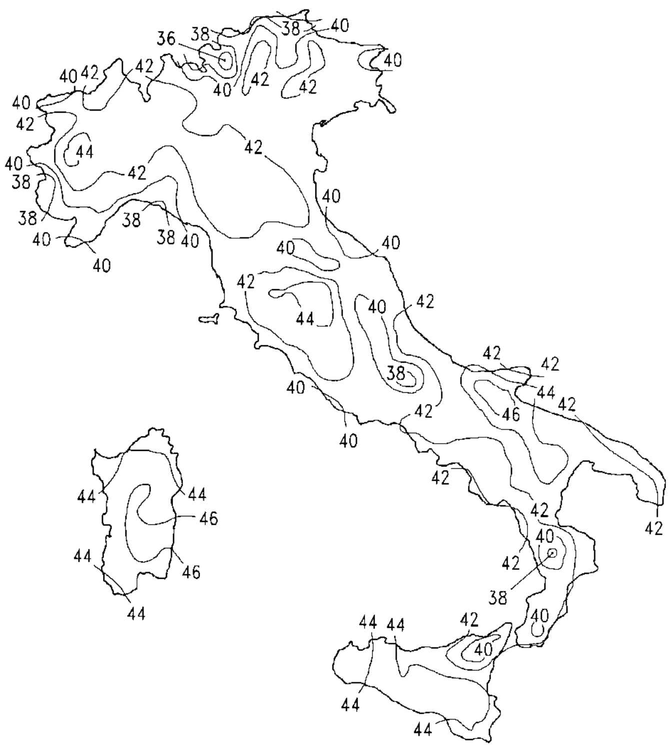


Figure 3. Map of maxima shade air temperatures in Italy.

In order to deal with shade air temperature values (maxima, $T_{max,p}$, or minima, $T_{min,p}$) with a probability other than 0,02, the followings relationships (based on a type I probability distribution of the extreme values) can be used:

$$T_{max,p} = T_{max} \cdot \{k_1 - k_2 \cdot \ln[-\ln(1-p)]\} \tag{1}$$

$$T_{min,p} = T_{min} \cdot \{k_3 + k_4 \cdot \ln[-\ln(1-p)]\} \tag{2}$$

It moreover follows that:

$$k_1 = \frac{u \cdot c}{u \cdot c + 3{,}902} \tag{3}$$

$$k_2 = \frac{1}{u \cdot c + 3{,}902} \tag{4}$$

$$k_3 = \frac{u \cdot c}{u \cdot c - 3{,}902} \tag{5}$$

$$k_4 = \frac{1}{u \cdot c - 3{,}902} \tag{6}$$

Where the parameters 'u' and 'c' are functions of the mean 'm' and the standard deviation 'σ' of the type I extreme value distribution:

for the maximum value
$$u = m - \frac{0{,}57722}{c} \tag{7}$$

$$c = \frac{1{,}2825}{\sigma} \tag{8}$$

for the minimum value
$$u = m + \frac{0{,}57722}{c} \tag{9}$$

$$c = \frac{1{,}2825}{\sigma} \tag{10}$$

Lacking more precise data, the following values are recommended:

$$k_1 = 0{,}781 \tag{11}$$

$$k_2 = 0{,}056 \tag{12}$$

$$k_3 = 0{,}393 \tag{13}$$

$$k_4 = -\ 0{,}156 \tag{14}$$

The effects of the thermal actions must be determined for each and every design situation deemed to be relevant either to the projected work's functionality or its safety. In the event that the structures are not exposed to significant daily or seasonal temperature variations, or variations caused by activities within the building, the effects of short-term thermal actions can be neglected in the structural analysis. In essence, all thermal actions can thus be attributed to either climatic effects or operations in the building's interior. With regard to the first, climatic effects must be determined by considering variations in the shade air temperature and changes in solar radiation, and thereby defining a conventional temperature, as set forth in the following. With regard to the second effects, the influence of activities carried out in the building's interior (technological or industrial processes) must be evaluated according to its specific design characteristics and specifications.

Following the foregoing breakdown of the components of thermal effects, the thermal actions on structural elements (whether their cause is climatic or processes executed in the building) must be specified considering the following fundamental quantities:

- a uniform temperature component, ΔT_{υ}, given by the difference between the mean temperature T of an element and its conventional initial temperature T_0;
- a component of linearly variable temperature, given by the difference, ΔT_{M}, between the temperatures of the external and internal surfaces of a straight section;
- a temperature difference, ΔT_{Π}, between different parts of the structure, given by the difference between the mean temperatures of the parts in question.

Moreover, if the local effects of thermal actions are consistent and significant, they must also be considered in addition to the aforesaid contributions ΔT_{υ}, ΔT_{M}, ΔT_{Π}.

The uniform component of temperature ΔT_{υ} of any given structural element is calculated as the difference between the mean temperature T of the element during the season under study and the temperature T_0 at the initial instant: $\Delta T_{\upsilon} = T - T_0$. The first step is to determine the value of T. This is calculated as the value of the mean winter or summer temperature of the structural element in question by adopting a specific profile that defines the temperature distribution throughout the element's thickness. If the internal (T_{in}) and external (T_{out}) conditions are sufficiently similar, a simplified procedure can be adopted, and the mean temperature value used for T:

$$T = \frac{T_{\mathrm{out}} + T_{\mathrm{in}}}{2} \tag{15}$$

The following step consists in determining the quantities T_{out} and T_{in}. In this regard, Eurocode 1 provides three tables (two different values of T_{out} are distinguished for two parts of the structure, one above ground level and one below). It should moreover be noted that in the following tables the values of T_{out} for the summer season are a function of both the building's orientation and the thermal absorption characteristics of its external surfaces. Obviously, the maximum values are reached on horizontal surfaces and those facing south or southwest, while the minima (which are equal to approximately one half the maximum values) are found on the surfaces facing north.

Table 1. Indicative temperature values for interiors

Season	*Temperature* T_{in}
Summer	T_1
Winter	T_2

Table 2. Indicative values of T_{out} for buildings above ground level.

<table>
<tr><th>Season</th><th colspan="2">Significant factor</th><th>Temperature T_{out}</th></tr>
<tr><td rowspan="3">Summer</td><td rowspan="3">Surface absorption properties, colour-dependent</td><td>0,5
(very light surface)</td><td>$T_{max} + T_3$</td></tr>
<tr><td>0,7
(light or coloured surface)</td><td>$T_{max} + T_4$</td></tr>
<tr><td>0,9
(dark surface)</td><td>$T_{max} + T_5$</td></tr>
<tr><td colspan="3">Winter</td><td>T_{min}</td></tr>
</table>

Table 3. Indicative values of T_{out} for buildings below ground level.

Season	*Depth below ground level*	*Temperature* T_{out}
Summer	Less than 1 m More than 1 m	T_6 T_7
Winter	Less than 1m More than 1 m	T_8 T_9

The T_{in} and T_{out} values in the preceding tables are specified in °C. Moreover, with regard to such values:

- T_1 values are specified in the National Annexes, though lacking more precise indications, a value of 20 °C can be assumed.
- T_2 values are also specified in the National Annexes, though lacking more precise indications a value of 25 °C can be assumed.
- The maximum and minimum values of the shade air temperatures, T_{max}, and T_{min}, and the effects of solar radiations, T_3, T_4 and T_5, will be specified in the National Annexes.

The values of the shade air temperature should be modified as a function of the building site's altitude above sea level. In the event that more exact information is lacking, the air shade temperature values can be corrected for altitude as follows: for every 100 m above sea level, subtract 0,5 °C from the minimum temperature value and 1,0 °C from the maximum. For instance, for regions lying between latitudes 45°N and 55°N, the following values are recommended:

$T_3 = 0$ °C
$T_4 = 2$ °C
$T_5 = 4$ °C
For surfaces facing northeast

or:

$T_3 = 18$ °C
$T_4 = 30$ °C
$T_5 = 42$ °C
For horizontal surfaces and those facing southwest

- The values of T_6, T_7 and T_8, and T_9 may be specified in the National Annexes. Once again for regions at latitudes between 45° N and 55° N, the following values are recommended:

$$T_6 = 8\ °C$$

$$T_7 = 5\ °C$$

$$T_8 = -5\ °C$$

$$T_9 = -3\ °C$$

The values of the materials' linear expansion coefficients are fundamental to performing structural analyses to determine the effects of thermal actions. For the materials usually employed in civilian buildings, the coefficient values (taken from the table in Annex C) are as follows.

Material	α_T (x 10^{-6} x °C^{-1})
Aluminium, aluminium alloys	24
Stainless Steel	16
Structural steel	12
Concrete (except as specified below)	10
Concrete with light aggregates	7
Masonry	6-10
Wood, parallel to its fibers	5
Wood, orthogonal to its fibers	30-70

Finally, in composite structures and reinforced concrete, the linear thermal expansion coefficient of the steel can be assumed to be equal to that of the concrete.

3 EXAMPLE

We analyze a regular steel framework located in Pisa (Italy), that forms three 5,0 m bays (with an overall plane surface of 15,0 m) and two floors, 3,0 m in height (for a total of 6,0 m), as represented in Figure 4.

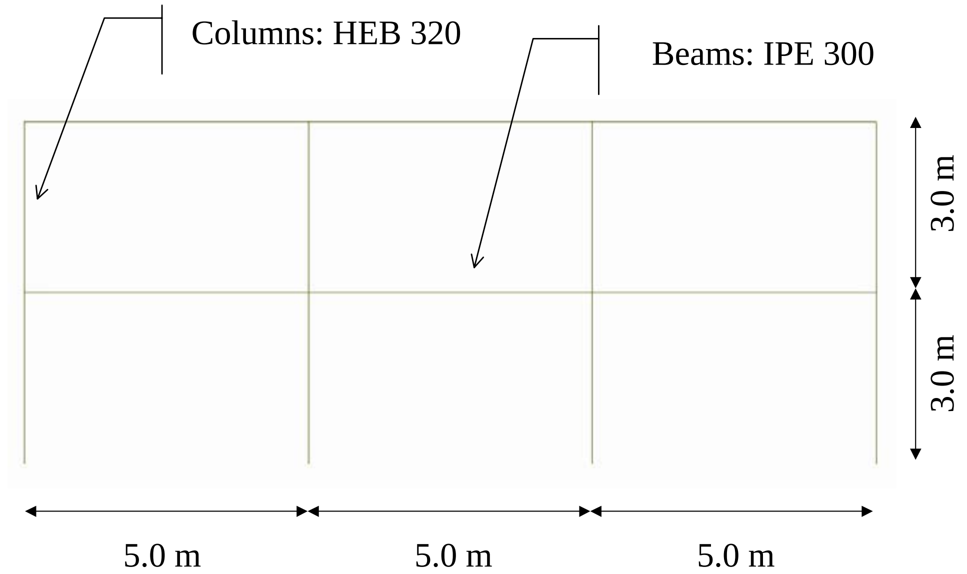


Figure 4. Example of a steel frame

The following structural elements go to make up the framework:
- beams (spanning 5,0 m): IPE 300;
- columns: HEB 320.

We consider three different uniform temperature components:
- Case 1: heating of every structural element (beams and columns) of the structure (summer season);
- Case 2: cooling of every structural element (beams and columns) of the structure (winter season);
- Case 3: heating of the external beams and columns (figure).

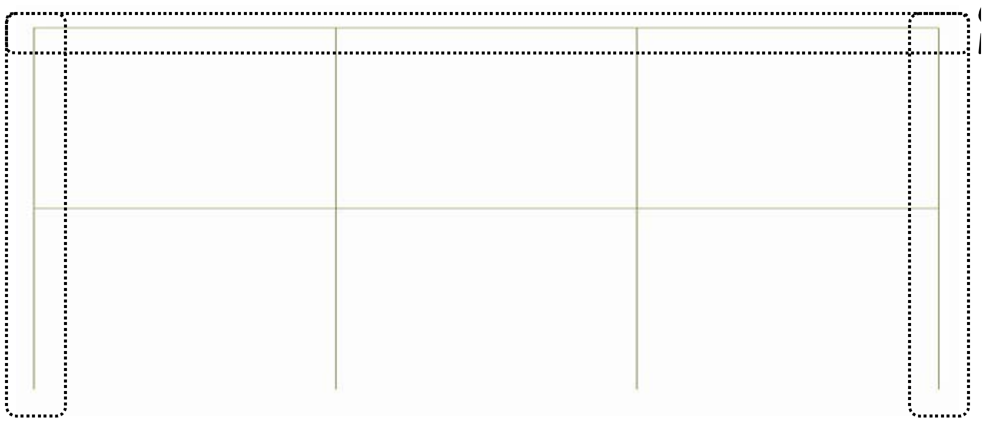


Figure 5. Elements subjected to heating of thermal load case 3.

$$\Delta T_{\upsilon} = T\text{-}T_0 = T - 10\ °\text{C}$$

T: mean temperature of the element during the season under study.

$$T = \frac{T_{\text{out}} + T_{\text{in}}}{2}$$

T_{in} values:
Summer: T_1 = 20 °C
Winter: T_2 = 25 °C

T_{out} values:
Summer (light or coloured surface): $T_{\text{max}} + T_4$ = 40 °C + 30 °C = 70 °C
Winter: T_{min} = - 9 °C

$$T_{\text{summer}} = \frac{T_{\text{out}(\text{summer})} + T_{\text{in}(\text{summer})}}{2} = \frac{70°C + 20°C}{2} = 45°\text{C}$$

$$T_{\text{winter}} = \frac{T_{\text{out}(\text{winter})} + T_{\text{in}(\text{winter})}}{2} = \frac{-9°C + 25°C}{2} = 7°\text{C}$$

$\Delta T_{\upsilon\ (\text{summer})} = T_{\text{summer}}\text{-}T_0$ = 45 °C – 10 °C = 35 °C   (for Load Case 1 and Load Case 3);

$\Delta T_{\upsilon\ (\text{winter})} = T_{\text{winter}}\text{-}T_0$ = 7 °C – 10 °C = - 3 °C   (for Load Case 2).

The following figures report the stress diagrams for the three load cases.

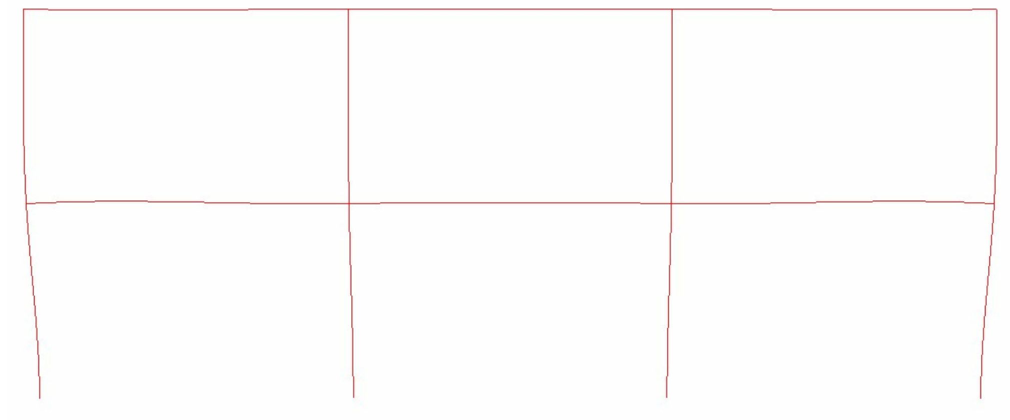


Figure 6. Deformed shape for thermal load case 1.

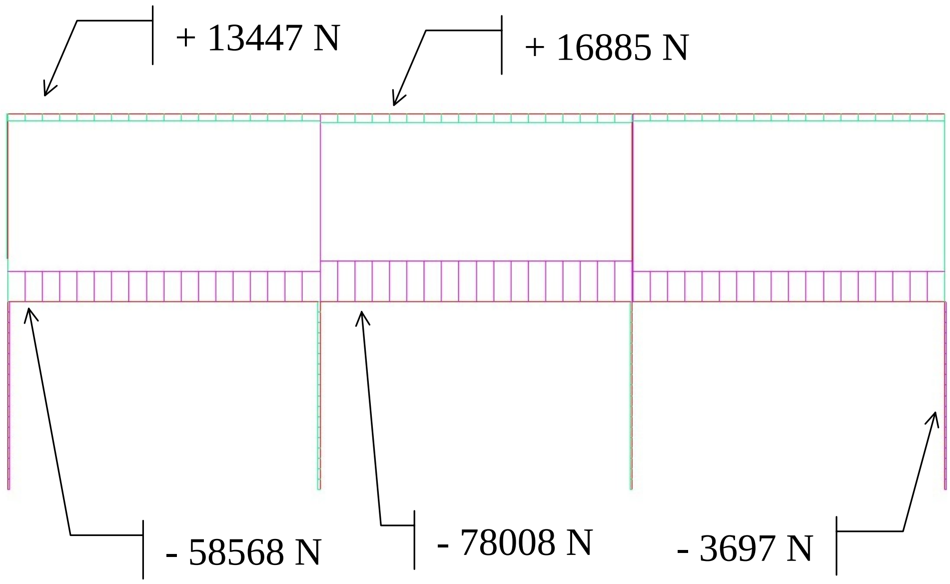


Figure 7. Compressive stress diagram for thermal load case 1

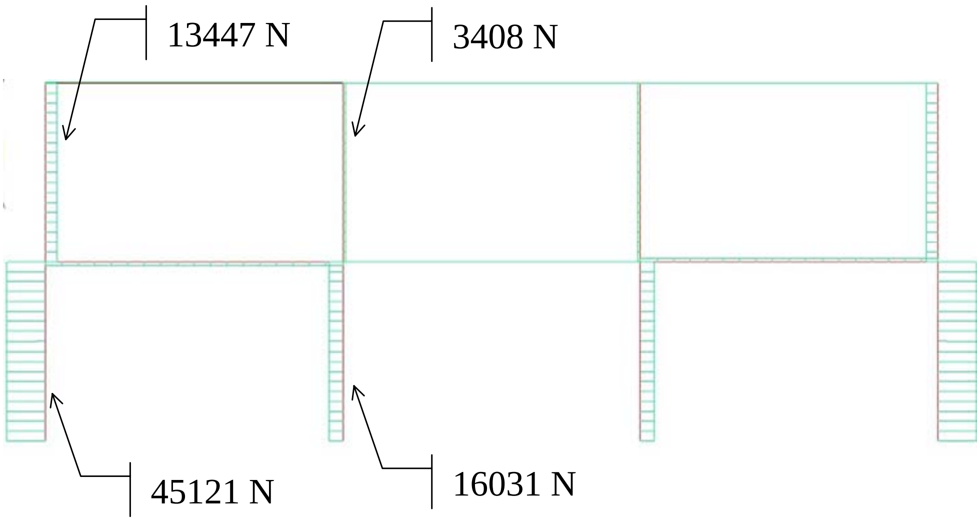


Figure 8. Shear diagram for thermal load case 1.

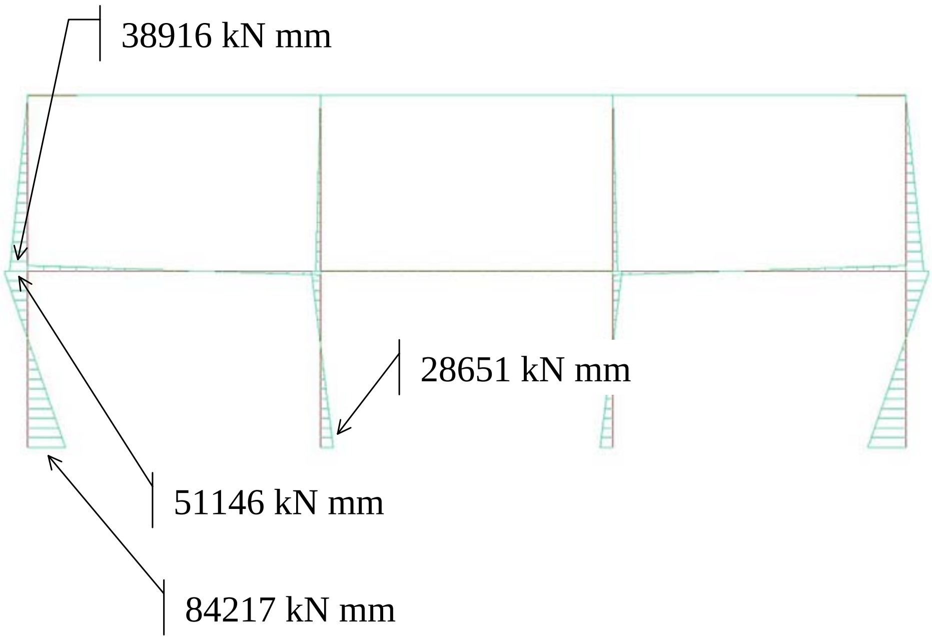


Figure 9. Bending moment diagram for thermal load case 1.

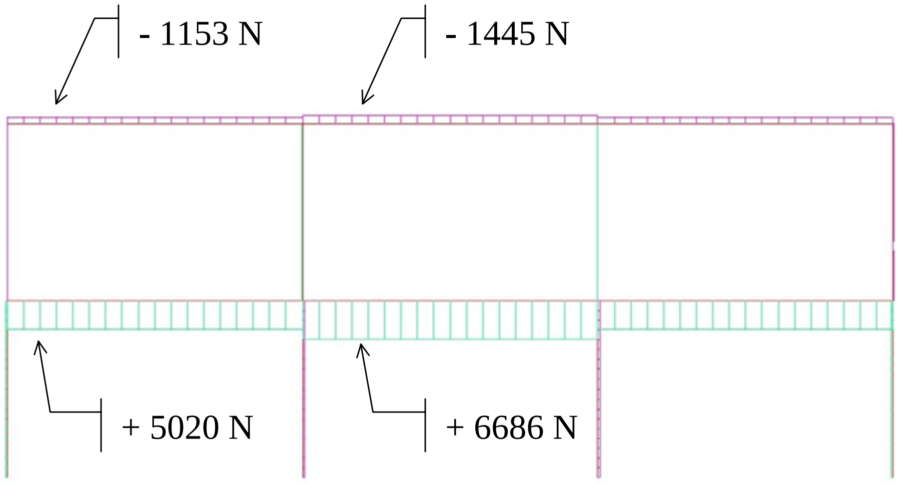


Figure 10. Axial force diagram for thermal load case 2.

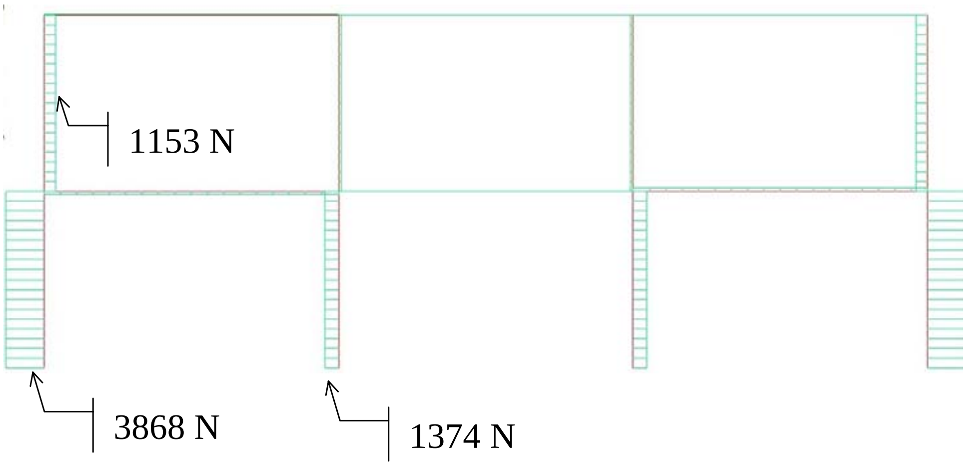


Figure 11. Shear diagram for thermal load case 2

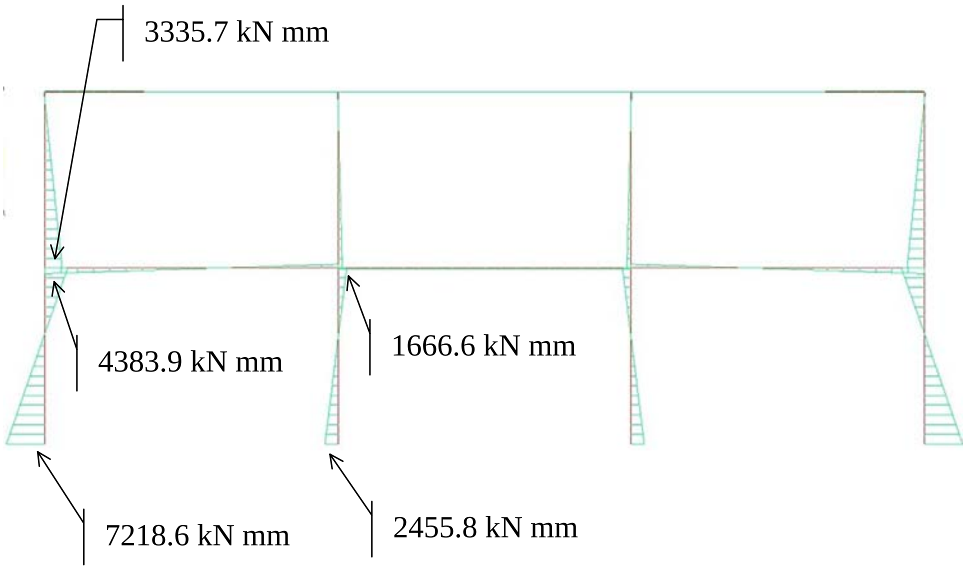


Figure 12. Bending moment diagram for thermal load case 2.

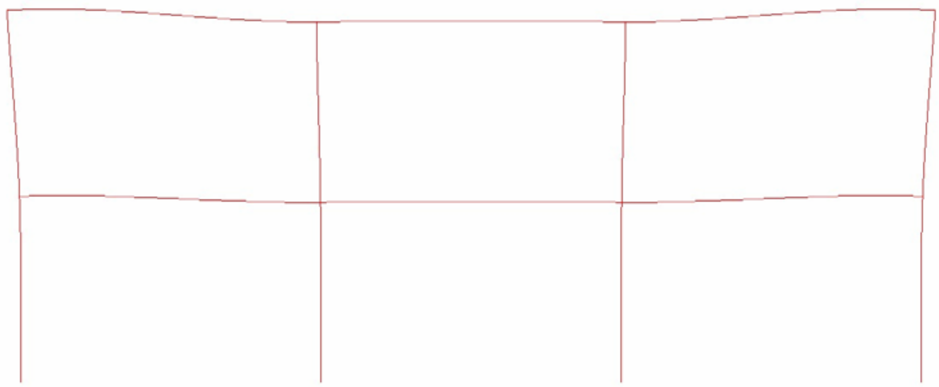


Figure 13. Deformed shape for thermal load case 3

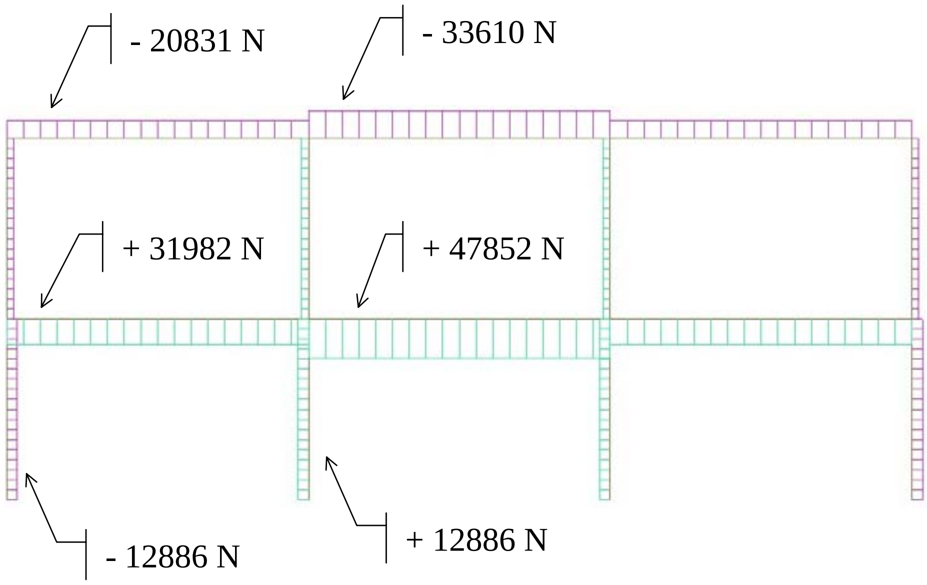


Figure 14. Axial force diagram, thermal load case 3.

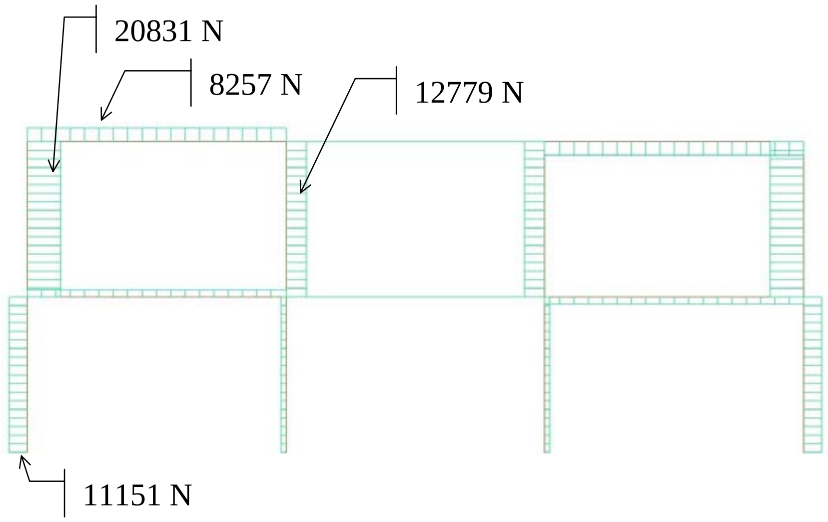


Figure 15. Shear force, thermal load case 3.

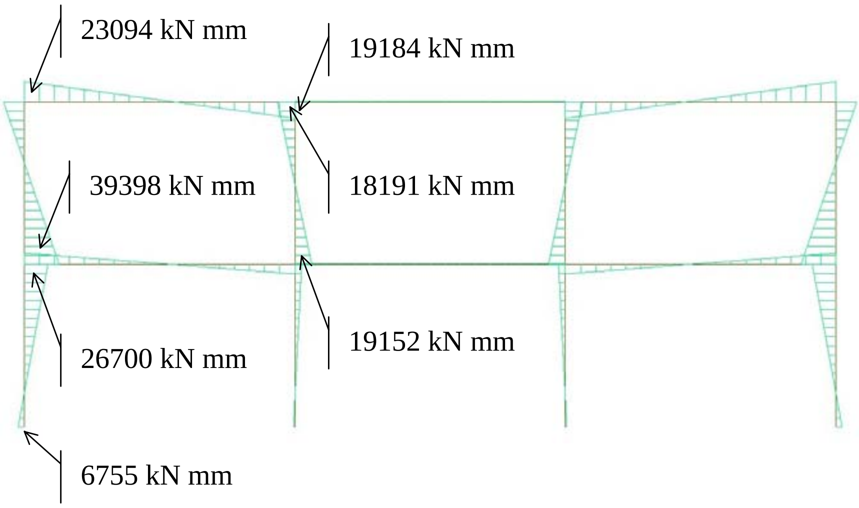


Figure 16. Bending moment, thermal load case 3.

CHAPTER V - ACCIDENTAL ACTIONS ON BUILDINGS

Ton Vrouwenvelder

Delft University of Technology/TNO Bouw, The Netherlands

Summary

The accidental actions covered by Part 1.7 of EN 1991 are discussed and guidance for their application in design is given. A short summary is presented of the relevant clauses, in particular with respect to impact, explosion and measures against so called unidentified actions. After the presentation of the clauses examples are given in order to get some idea of the design procedure and the design consequences.

1. INTRODUCTION

1.1 General

General principles for classification of actions on structures, including accidental actions and their modelling in verification of structural reliability, are introduced in EN 1990 Basis of Design. In particular EN 1990 defines the various design values and combination rules to be used in the design calculations. A detailed description of individual actions is then given in various parts of Eurocode 1, EN 1991. The Part 1.7 of EN 1991 covers accidental actions and gives rules and values for impact loads due to road, train and ship traffic and loads due to internal explosions

The impact loads are mainly concerned with bridges, but also impact on buildings is covered. Explosions concern explosions in buildings and in tunnels. It should be kept in mind that the loads in the main text are rather conventional. More advanced models are presented in Annex C. Apart from design values and other detailed information for the loads mentioned above, the document also gives guidelines how to handle accidental loads in general. In many cases structural measures alone cannot be considered as very efficient.

For buildings Annex A is of special interest. In this annex deemed to satisfy rules are presented in order to guarantee some minimum robustness against identified as well as unidentified causes of structural damage.

1.2 Background Documents

Part 1.7 of EN 1991 is partly based on the requirements put forward in the Eurocode on traffic loads (ENV 1991-3) and some ISO-documents. For the more theoretical parts use has been made of prenormalisation work performed in IABSE [1] and CIB [2]. Specific backgrounds information can be found in [3] and [4]. As far as Annex A is concerned reference is made to [5].

2. BASIS OF APPLICATIONS

Design for accidental situations in the Eurocodes is implemented to avoid structural catastrophes. As a consequence, design for accidental design situations needs to be included mainly for structures for which a collapse may cause particularly large consequences in terms of injury to humans, damage to the environment or economic losses for the society. In practice this means that Eurocode 1, Part 1.7 accepts the principle of safety differentiation. In fact three different safety categories are distinguished:

- category 1: limited consequences
- category 2: medium consequences
- category 3: large consequences

For these categories different methods of analysis and different levels of reliability are accepted.

Another part of the philosophy is that local failure due to accidental actions is accepted provided that it does not lead to overall failure. This distinguishing between component failure and system failure is mandatory to allow a systematic discrimination between design for variable actions and accidental actions.

A result of the acceptance of local failure (which in most cases may be identified as a component failure) provided that it does not lead to a system failure, is that redundancy and non-linear effects both regarding material behaviour and geometry play a much larger role in design to mitigate accidental actions than variable actions. The same is true for a design which allows large energy absorption.

In order to reduce the risk involved in accidental type of load one might, as basic strategies, consider probability reducing as well as consequence reducing measures, including contingency plans in the event of an accident. Risk reducing measures should be given high priority in design for accidental actions, and also be taken into account in design. Design with respect to accidental actions may therefore pursue one or more as appropriate of the following strategies, which may be mixed in the same design:

1. *preventing the action* occurring or reducing the probability and/or magnitude of the action to a reasonable level. (The limited effect of this strategy must be recognised; it depends on factors which, over the life span of the structure, are normally outside the control of the structural design process)
2. *protecting the structure* against the action (e.g. by traffic bollards)
3. *designing* in such a way that neither the whole structure nor an important part thereof will collapse if a local failure (single element failure) should occur
4. *designing key elements,* on which the structure would be particularly reliant, with special care, and in relevant cases for appropriate accidental actions
5. *applying prescriptive design/detailing rules* which provide in normal circumstances an acceptably robust structure (e. g. tri-orthogonal tying for resistance to explosions, or minimum level of ductility of structural elements subject to impact). For prescriptive rules Part 1.7 refers to the relevant ENV 1992 to ENV 1999.

The design philosophy necessitates that accidental actions are treated in a special manner with respect to load factors and load combinations. Partial load factors to be applied in analysis according to strategy no. 3 are defined in Eurocode, Basis of Design, to be 1,0 for all loads (permanent, variable and accidental) with the following qualification in: "Combinations for accidental design situations either involve an explicit accidental action A (e.g. fire or impact) or refer to a situation after an accidental event (A = 0)". After an accidental event the structure will normally not have the required strength in persistent and transient design situations and will have to be strengthened for a possible continued application. In temporary phases there may be reasons for a relaxation of the requirements e.g. by allowing wind or wave loads for shorter return periods to be applied in the analysis after an accidental event.

3. DESIGN FOR IMPACT AND EXPLOSION LOADS

3.1 Impact from vehicles

In the case of hard impact, design values for the horizontal actions due to impact on vertical structural elements (e.g. columns, walls) in the vicinity of various types of internal or external roads may be obtained from Table 3.1. The forces F_{dx} and F_{dy} denote respectively the forces in the driving direction and perpendicular to it. There is no need to consider them simultaneously. The collision forces are supposed to act at 1,25 m above the level of the driving surface (0,5 m for cars). The force application area may be taken as 0,25 m (height) by 1,50 m (width) or the member width, whichever is the smallest.

Table 1. Horizontal static equivalent design forces due to impact on supporting substructures of structures over roadways

Type of road	Type of vehicle	Force $F_{d,x}$ [kN]	Force $F_{d,y}$ [kN]
Motorway	Truck	1000	500
Country road	Truck	750	375
Urban area	Truck	500	250
Courtyards/garages	Passengers cars only	50	25
Courtyards/garages	Trucks	150	75

In addition to the values in Table 1 the code specifies more advanced models for nonlinear and dynamic analysis in an informative annex. For impact loads the reader is referred to Designers' Handbook No. 4 about bridge design.

3.2 Loads due to explosions

Key elements of a structure should be designed to withstand the effects of an internal natural gas explosion, using a nominal equivalent static pressure is given by:

$$p_d = 3 + p_v \tag{3.1}$$

or

$$p_d = 3 + 0,5\, p_v + 0,04/(A_v/V)^2 \tag{3.2}$$

whichever is the greater, where p_v is the uniformly distributed static pressure in kN/m^2 at which venting components will fail, A_v is the area of venting components and V is the volume of room. The explosive pressure acts effectively simultaneously on all of the bounding surfaces of the room. The expressions are valid for rooms up to a volume of 1000 m^3 and venting areas over volume rations of 0,05 m^{-1} $\leq A_v / V \leq 0$, 15 m^{-1}.

An important issue is further raised in clause D.XXX. It states that the peak pressures in the main text may be considered as having a load duration of 0,2 s. The point is that in reality the peak will generally be larger, but the duration is shorter. So combining the loads from the above equations with a duration of 0,2 s seems to be a reasonable approximation.

3.3 Design example of a column in a building for an explosion

Consider a living compartment in a multi-storey flat building. Let the floor dimensions of the compartment be 8 x 14 m and let the height be 3 m. The two small walls (the facades) are made of glass and other light materials and can be considered as venting area. These walls have no load bearing function in the structure. The two long walls are concrete walls; these walls are responsible for carrying down the vertical loads as well as the lateral stability of the structure. This means that the volume V and the area of venting components A_v for this case are given by:

$$A_v = 2 \text{ x } 8 \text{ x } 3 = 48 \text{ m}^2$$

$$V = 3 \text{ x } 8 \text{ x } 14 = 336 \text{ m}^3$$

So the parameter A_v / V can be calculated as:

$$A_v / V = 48 / 336 = 0{,}144 \text{ m}^{-1}$$

As V is less then 1000 m^3 and A_v / V is well within the limits of 0,05 m^{-1} and 0,15 m^{-1} it is allowed to use the loads given in the code. The collapse pressure of the venting panels p_v is estimated as 3 kN/m^2. Note that these panels normally can resist the design wind load of 1,5 kN/m^2. The equivalent static pressure for the internal natural gas explosion is given by:

$$p_{Ed} = 3 + p_v = 3 + 3 = 6 \text{ kN/m}^2$$

or $$p_{Ed} = 3 + p_v/2 + 0{,}04/(A_v/V)^2 = 3 + 1{,}5 + 0{,}04 / 0{,}144^2 = 3 + 1{,}5 + 2{,}0 = 6{,}5 \text{ kN/m}^2$$

This means that we have to deal with the latter.

The load arrangement for the explosion pressures is presented in Figure 1. According to Eurocode EN 1990, Basis of Design, these pressures have to be combined with the self-weight of the structure and the quasi-permanent values of the variables loads. Let us consider the design consequences for the various structural elements.

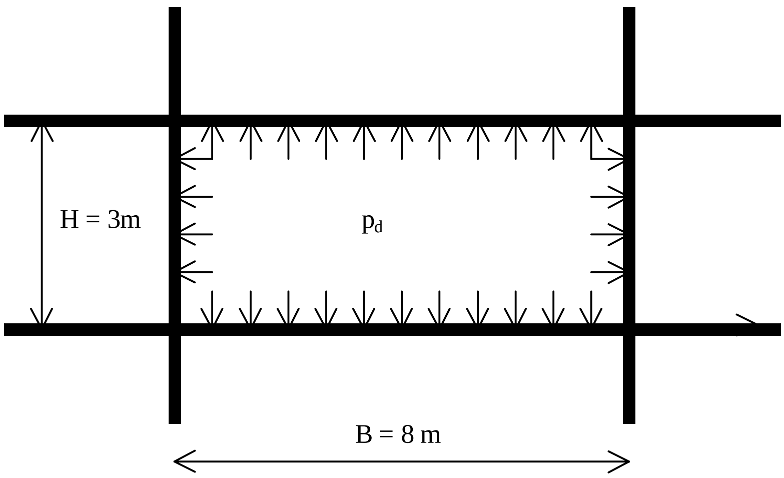


Figure 1. Load arrangement for the explosion load

Bottom floor

Let us start with the bottom floor of the compartment. Let the self-weight be 3 kN/m^2 and the live load 2 kN/m^2. This means that the design load for the explosion is given by:

$$p_{da} = p_{SW} + p_E + \psi_{1LL}\, p_{LL} = 3{,}00 + 6{,}50 + 0{,}5 \cdot 2{,}00 = 10{,}50 \text{ kN/m}^2$$

The design for normal conditions is given by:

$$p_{\mathrm{d}} = \gamma_{\mathrm{G}}\, \xi\, p_{\mathrm{SW}} + \gamma_{\mathrm{Q}}\, p_{\mathrm{LL}} = 0{,}85 \cdot 1{,}35 \cdot 3{,}00 + 1{,}5 \cdot 2{,}00 = 6{,}4 \text{ kN/m}^2$$

We should keep in mind that for accidental actions there is no need to use a partial factor on the resistance side. So for comparison we could increase the design load for normal conditions by a factor of 1,2. The result could be conceived as the resistance of the structure against accidental loads, if it designed for normal loads only:

$$p_{\mathrm{Rd}} = 1{,}2 \cdot 6{,}4 = 7{,}7 \text{ kN/m}^2$$

So a floor designed for normal conditions only should be about 30 % too light. It is now time to remember the clause in Annex B of EC1 Part 1.7, mentioned in section 4. If we take into account the increase in short duration of the load we may increase the load bearing capacity by a factor φ_{d} given by (see Annex 1):

$$\varphi_{\mathrm{d}} = 1 + \sqrt{\frac{p_{\mathrm{SW}}}{p_{\mathrm{Rd}}}}\,\sqrt{\frac{2u_{\max}}{g(\Delta t)^2}}$$

where Δt = 0,2 s is the load duration, g = 10 m/s^2 is the acceleration of the gravity field and $u_{\max}$ is the design value for the midspan deflection at collapse. This value of course depends on the ductility properties of the floor slab and in particular of the connections with the rest of the structure. It is beyond the scope of this paper to discuss the details of that assessment, but assume that $u_{\max}$ = 0,20 m is considered as being a defendable design value. In that case the resistance against explosion loading can be assessed as:

$$p_{\mathrm{REd}} = \varphi_{\mathrm{d}}\, p_{\mathrm{Rd}} = [1 + \sqrt{\frac{3}{7{,}7}}\sqrt{\frac{2 \cdot 0{,}20}{10 \cdot (0{,}2)^2}}\ ] \cdot 7{,}7 = 12{,}5 \text{ kN/m}^2$$

So the bottom floor system is okay in this case.

Upper floor

Let us next consider the upper floor. Note that the upper floor for one explosion could be the bottom floor for the next one. The design load for the explosion in that case is given by (upward value positive!):

$$p_{\mathrm{da}} = p_{\mathrm{SW}} + p_{\mathrm{E}} + \gamma_{\mathrm{Q}}\, \psi\, p_{\mathrm{LL}} = -\ 3{,}00 + 6{,}50\ + 0 = 3{,}50 \text{ kN/m}^2$$

So the load is only half the load on the bottom floor, but will give larger problems anyway. The point is that the load is in the opposite direction of the normal dead and live load. This means that the normal resistance may simply be close to zero. What we need is top reinforcement in the field and bottom reinforcement above the supports. The required resistance can be found by solving p_{Rd} from:

$$\varphi_{\mathrm{d}}\ p_{\mathrm{Rd}} = [1 + \sqrt{\frac{p_{\mathrm{SW}}}{p_{\mathrm{Rd}}}}\sqrt{\frac{2u_{\max}}{g(\Delta t)^2}}\ ]\ p_{\mathrm{Rd}} = 3{,}50$$

Using again p_{SW} = 3 kN/m^2, Δt = 0,2 s, g=10 m/s^2 we arrive at p_{Rd} = 1,5 kN/m^2. This would require about 25 % of the reinforcement for normal conditions on the opposite side.

An important additional point to consider is the reaction force at the support. Note that the floor could be lifted from its supports, especially in the upper two stories of the building where the normal forces in the walls are small. In this respect edge walls are even more vulnerable. The uplifting may change the static system for one thing and lead to different load effects, but it may also lead to freestanding walls. We will come back to that in the next paragraph. If the floor to wall connection can resist the lift force, one should make sure that the also the wall itself is designed for it.

Walls

Finally we have to consider the walls. Assume the wall to be clamped in on both sides. The bending moment in the wall is then given by:

$$m = 1/16\ p\ H^2 = 1/16 \cdot 6{,}5 \cdot 3^2 = 4 \text{ kNm/m}$$

If there is no normal force acting in the wall this would require a central reinforcement of about 0,1%. The corresponding bending capacity can be estimated as:

$$m_p = \omega\ 0{,}4\ d^2\ f_y = 0{,}001 \cdot 0{,}4 \cdot 0{,}2^2 \cdot 300000 = 5 \text{ kNm/m}$$

Normally, of course normal forces are present. Leaving detailed calculations as being out of the scope of this document, the following scheme looks realistic. If the explosion is on a top floor apartment and there is an adequate connection between roof slab and top wall, we will have a tensile force in the wall, requiring some additional reinforcement. In our example the tensile force would be (p_E – 2 p_{SW}) B/2 = (6,5-2x3) * 4 = 2 kN/m for a middle column and (p_E – p_{SW}) B/2 = (6,5-3) * 4 = 14 kN/m for an edge column. If the explosion is on the one but top story, we usually have no resulting axial force and the above mentioned reinforcement will do. Going further down, there will probably be a resulting axial compression force and the reinforcement could be diminished ore even left out completely.

4. ROBUSTNESS OF BUILDINGS (Annex A of EN 1991-1-7)

4.1 Background

The rules drafted for Annex A of EN1991-1-7 were developed from the UK Codes of Practice and regulatory requirements introduced in the early 70s following the partial collapse of a block of flats in east London caused by a gas explosion. The rules have changed little over the intervening years. They aim to provide a minimum level of building robustness as a means of safeguarding buildings against a disproportionate extent of collapse following local damage being sustained from an accidental event.

The rules have proved satisfactory over the past 3 decades. Their efficacy was dramatically demonstrated during the IRA bomb attacks that occurred in the City of London in 1992 and 1993. Although the rules were not intended to safeguard buildings against terrorist attack, the damage sustained by those buildings close to the seat of the explosions that were designed to meet the regulatory requirement relating to disproportionate collapse was found to be far less compared with other buildings that were subjected to a similar level of abuse.

The Annex A also provides an example of how the consequences of building failure may be classified into “Consequences Classes” corresponding to reliability levels of robustness. These classes are:

Consequences class	Example structures
class 1	low rise buildings where only few people are present
class 2, lower group	most buildings up to 4 stories
class 3, upper group	most buildings up to 15 stories
class 4	high rise building, grand stands etc.

Member states are invited to provide rules for a national classification system.

4.2 Summary of design rules in Annex A

In this section the rules will be summarised for every consequences class. Note that for class 1 there are no special considerations and for class 3 a risk analysis is recommended. The depth of the risk analysis is up to the local authorities. A distinction is made between framed structures and load-bearing wall construction.

4.2.1. Design Rules for Class 2, Lower Group, Framed structures:

Horizontal ties should be provided around the perimeter of each floor (and roof) and internally in two right angle directions to tie the columns to the structure (Figure 2)**.** Each tie, including its end connections, should be capable of sustaining the following force in [kN]:

internal ties: $T_{\mathrm{i}} = 0{,}8\ (g_{\mathrm{k}} + \Psi q_{\mathrm{k}})\ s\ L$ (but > 75kN) (4.1)

perimeter ties: $T_p = 0{,}4\ (g_{\mathrm{k}} + \Psi q_{\mathrm{k}})\ s\ L$ (but > 75kN). (4.2)

In here g_{k} and q_{k} are the characteristic values in [kN/m^2] of the self weight and imposed load respectively; Ψ is the combination factor, s [m] is the spacing of ties and L [m] is the span in the direction of the tie, both in m.

Edge columns should be anchored with ties capable of sustaining a tensile load equal to 1% of the vertical design load carried by the column at that level.

4.2.2 Rules for Class 2, Lower group, Load-bearing wall construction:

A cellular form of construction should be adopted to facilitate interaction of all components including an appropriate means of anchoring the floor to the walls.

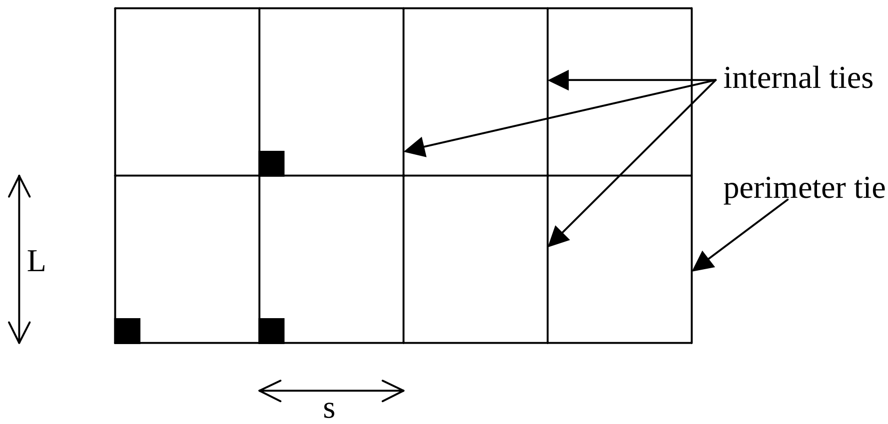


Figure 2. Example of effective horizontal tying of a framed office building.

4.2.3 Rules for Class 2 - Upper Group, Framed structures:

Horizontal ties as above; in addition one of the following measures:

- Effective vertical ties: Columns and walls should be capable of resisting an accidental design tensile force equal to the largest design permanent and variable load reaction applied to the column from any story.
- Ensuring that upon the notional removal of a supporting column, beam or any nominal section of load-bearing wall, the damage does not exceed 15% of the floor in each of 2 adjacent storeys. The nominal length of load-bearing wall construction referred to above should be taken as a length not exceeding 2,25H;

for an external masonry, timber or steel stud wall, the length measured between vertical lateral supports.

- Key elements designed for an accidental design action A_d = 34 kN/m^2.

4.2.4 Rules for Class 2 - Upper Group, Load-bearing wall construction.

Rules for horizontal ties similar to those for framed buildings except that the design tensile load in the ties shall be as follows:

For internal ties $$T_i = \frac{F_t(g_k + \psi \cdot q_k)}{7{,}5}\frac{z}{5} \text{ kN/m but} > F_t \qquad (4.3)$$

For perimeter ties $$T_p = F_t \qquad (4.4)$$

Where F_t = (20 + 4 n) kN with a maximum of 60 kN, where n represents the number of storeys; g, q and Ψ have the same meaning as before, and z = 5 h or the length of the tie in [m], whichever is smallest.

In vertical direction of the building the following expression is presented:

For vertical tie $$T_v = \frac{34A}{8000}\left(\frac{h}{t}\right)^2 \text{ N} \qquad (4.5)$$

but at least 100 kN/m times the length of the wall.

In this formula A is the load bearing area of the wall, h is the story height and t is the wall thickness. Load bearing wall construction may be considered effective vertical ties if (in the case of masonry) their thickness is at least 150 mm and the height of the wall h < 20 t, where t is wall thickness.

4.3 Example structures

4.3 1 Framed structure, Consequences class 2, Upper Group

Consider a 5 storey building with story height h = 3,6 m. Let the span be L = 7,2 m and the span distance s = 6 m. The loads are q_k = g_k = 4 kN/m^2 and Ψ=1,0. In that case the required internal tie force may be calculated as:

$$T_i = 0{,}8\ \{4+4\}\ (6 \text{ x } 7{,}2) = 276 \text{ kN} > 75 \text{ kN}$$

For Steel quality FeB 500 this force corresponds to a steel area A = 550 mm^2 or 2 ø18 mm. The perimeter tie is simply half the value. Note that in continuous beams this amount of reinforcement usually is already present as upper reinforcement anyway. For the vertical tying force we find:

$$T_v = (4 + 4)\ (6 \text{ x } 7{,}2) = 350 \text{ kN/column}$$

This corresponds to A = 700 mm^2 or 3 ø18 mm.

4.3 2 Load bearing wall type of structure, Consequences class 2, Upper Group

For the same starting points we get F_b = min (60, 40) = 40 and z = 5 h = 12 m and from there for the internal and perimeter tie forces:

$$T_i = 60\ \frac{4+4}{7{,}5}\frac{12}{5} = 110 \text{ kN/m}$$

$$T_p = 40 \text{ kN/m}$$

The vertical tying force is given by:

$$T_v = \frac{34 \cdot 0{,}2}{8}\left(\frac{3{,}6}{0{,}2}\right)^2 = 300 \text{ kN/m}$$

For many countries this may lead to more reinforcement then usual for these type of structural elements.

REFERENCES

[1] Ole Damgaard Larsen: Ship Collision with bridges. Structural Engineering Documents, The interaction between Vessel traffic and Bridge structures.

[2] Actions on structures impact. CIB Report, Publication 167, CIB, Rotterdam, 1992.

[3] T. Vrouwenvelder: Stochastic modelling of extreme action events in structural engineering. Probabilistic Engineering Mechanics 15, 109-117, 2000.

[4] Project Team ENV 1991-2-7: Accidental actions due to impact and explosions. Background document to ENV 1991-2-7.

[5] Safety in tall buildings and other buildings with large occupancy by the Institution of Structural Engineers.

[6] Stufib Studiecel, Incasseringsvermogen van bouwconstructies (Robustness fo building structures, in Dutch). Delft, October 2004.

Annex 1

Consider a spring mass system with a mass m and a rigid plastic spring with yield value F_y. Let the system be loaded by a load $F>F_y$ during a period of time Δt. The velocity of the mass achieved during this time interval is equal to:

$$v = (F - F_y)\, \Delta t / m$$

The corresponding kinetic energy of the mass is then equal to:

$$E = 0{,}5\, m\, v^2 = 0{,}5\, (F-F_y)^2\, \Delta t^2 / m$$

By equating this energy to the plastic energy dissipation, that is we put $E = F\, \Delta u$, we may find the increase in plastic deformation Δu.

$$\Delta u = 0{,}5\, (F - F_y)^2\, \Delta t^2 / m\, F_y$$

As $mg = F_{SW}$ we may also write:

$$\Delta u = 0{,}5\, (F-F_y)^2\, g\, \Delta t^2 / F_{SW}\, F_y$$

Finally we may rewrite this formula in the following way:

$$F = F_y\, (1 + \sqrt{\frac{F_{SW}}{F_{Rd}}}\, \sqrt{\frac{2u_{max}}{g(\Delta t)^2}}\,)$$

For the slab structure in section 5.1 we have replaced the forces F by the distributed loads p.

CHAPTER VI - EXAMPLES OF CONCRETE BUILDINGS

Luca Sanpaolesi, Stefano Colombini

Department of Structural Engineering, University of Pisa, Italy

Summary

This chapter examines two buildings with reinforced concrete structures. The first is typical of modern housing, while the second is a representative type of industrial warehouse. After describing the buildings' geometries and constituent materials, we shall determine the design loads and relevant combinations. Particular attention shall be devoted to the appropriate choice of load combinations as a function of the stress characteristics to be maximized in the various structural elements.

1 BACKGROUND DOCUMENTS

Background material for this chapter can be found in regulations EN 1990 "Basis of design" and EN-1991 "Eurocode 1: Actions on structures".

2 HOUSING BUILDING

2.1 Description of the structure

The construction in question is a apartment building located in the urban area of Pisa (Italy). Structurally, it is made up of a regular framework of reinforced-concrete beams and columns. The framework is formed by parallel, plane main frames, distanced 4 m one from the other, which are connected by secondary beams. The main frames form seven 5 m bays (with an overall plane surface of 35 m) and five floors, 3 m in height (for a total of 15 m), as represented in Figures 1 and 2.

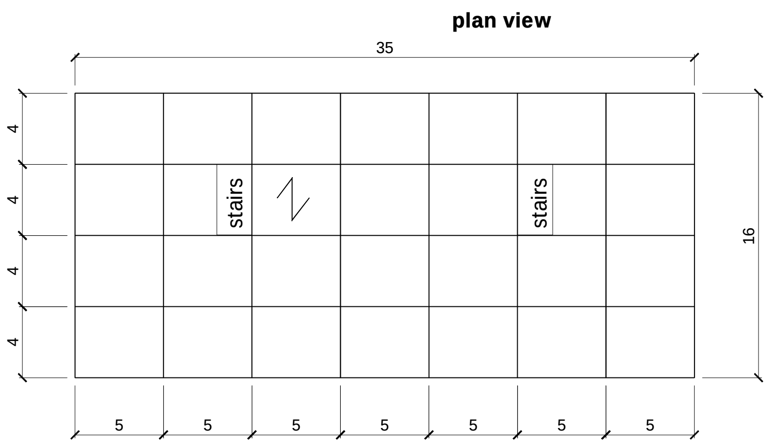


Figure 1. Plane view of house building.

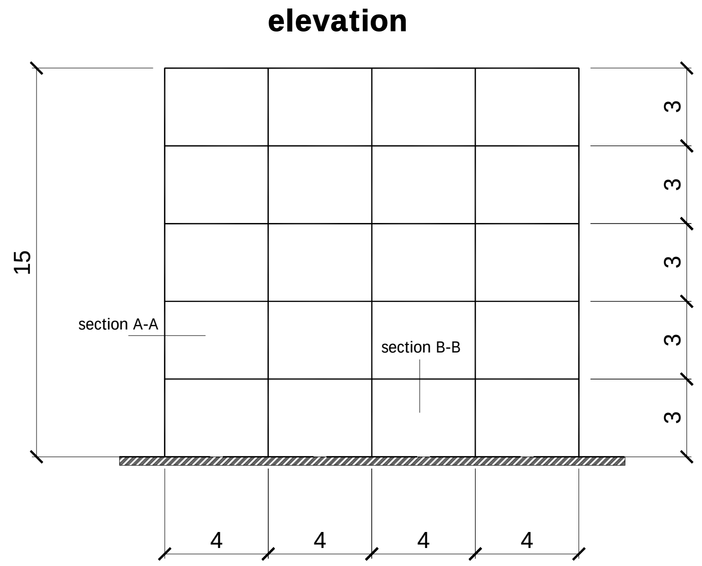


Figure 2. House building elevation.

The following structural elements go to make up the framework :

- main beams (spanning 5 m): rectangular cross section, 300 x 500 mm (Figure 3);
- columns: rectangular cross section, 300 x 400 mm (Figure 3);
- secondary beams (spanning 4 m): rectangular cross section, 400 x 300 mm.

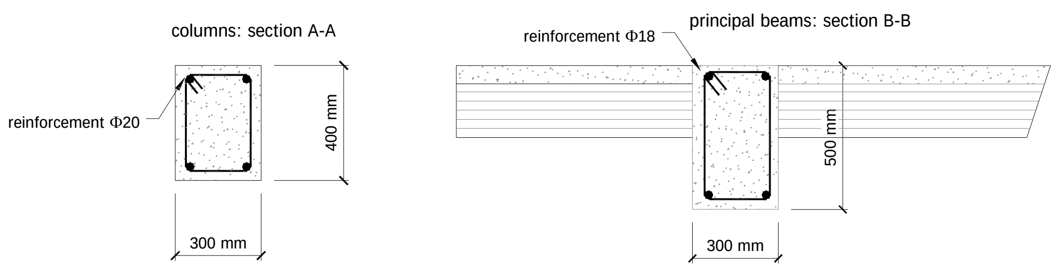


Figure 3. Columns and main beams of framework

The floor, spanning 4,0 m, is composed of beam joists with a concrete base finished on the intrados with a continuous surface coating. The joists are linked to interposed brickwork, and the finishing cast performed on site during construction results in a perfectly single-block floor (Figure 4). The floor is 250 mm thick (of which 50 mm are accounted for by the concrete slab) and an in-service weight of 3,25 kN/m^2.

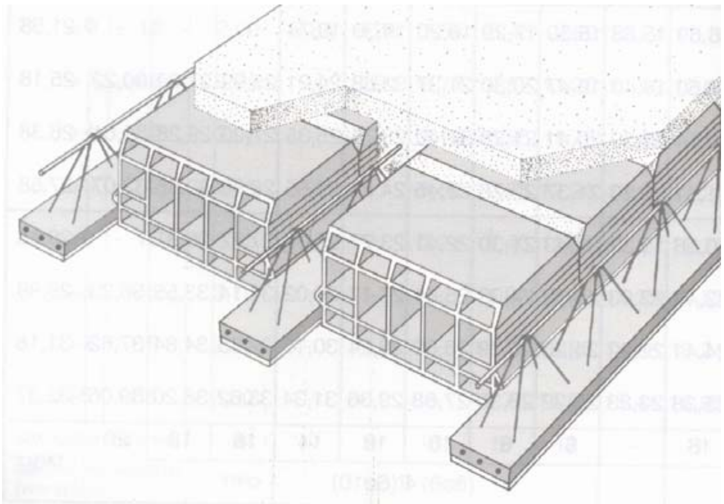


Figure 4. Floor structure

The following hypotheses were made in conducting the modeling and analysis:
- the material constitutive laws are assumed to be linear-elastic; second order geometric effects are moreover considered negligible (linear elastic analysis);
- the floors are considered to possess infinite stiffness within their plane;
- the foundation plate is also assumed to have infinite stiffness (this enables neglecting any ground-structure interaction phenomena).

2.2 Materials

Class C 30/37 concrete (E_c = 33 kN/mm^2) with normal compaction and inert filler materials were employed. The steel used for reinforcement is characterized by a linear elastic modulus, E_s = 200 kN/mm^2.

2.3 Definition of the design loads

2.3.1 Self weight and dead load

The self-weight of the floor is 3,25 kN/m^2.

The weights of the structural elements were determined on the basis of their geometries and the density values indicated in EN 1991, Part 1.1.

For the normal-weight concrete, guidelines specify a density of 24 kN/m^3, which is to be increased by 1 kN/m^3 when a significant percentage of normal reinforcement is present, so that for normal reinforced-concrete elements, the resulting density value is 25 kN/m^3.

- Main beams:
  Straight section: 300 mm x 500 mm = 0,15 m^2.
  Weight per unit length: g_{beam-1} = 0,15 m^2 x 25 kN/m^3 = 3,75 kN/m.
- Secondary beams:
  Straight section: 400 mm x 300 mm = 0,12 m^2.
  Weight per unit length: g_{beam-2} = 0,12 m^2 x 25 kN/m^3 = 3,00 kN/m.
- Columns:
  Straight section: 300 mm x 400 mm = 0,12 m^2.
  Weight per unit length: g_{col} = 0,12 m^2 x 25 kN/m^3 = 3,00 kN/m.
  Column weight per storey height = 3,00 kN/m x 3,0 m = 9,00 kN

Regarding the permanent loads, inner walls and partitions are considered to have a self-weight of 3,0 kN/m (that is, per unit length of wall), which involves a uniformly distributed load equal to q_k = 1,2 kN/m^2. Moreover, a uniformly distributed load of q_{k-2} = 1,1 kN/m^2 was also considered to represents the overall combined weight of the flooring, insulation, footings, piping and the intrados plastering. Therefore, in all, the permanent load is given by:

$$g_{per} = 3{,}25 \text{ kN/m}^2 + 1{,}2 \text{ kN/m}^2 + 1{,}1 \text{ kN/m}^2 = 5{,}55 \text{ kN/m}^2.$$

2.3.2 Imposed load

The building is destined for use as civil dwellings and therefore falls into usage category A, according to the classification set forth in EN 1991, Part 1.1

The imposed loads on the floors are therefore: 2,0 kN/m^2.

The roof is considered to be accessible and subjected to the same imposed load as the underlying floors; it is also classified in category A, and thus characterized by the same imposed load value.

2.3.3 Snow loads

The characteristic ground snow-load value for the area in question (Italy zone 2) is: s_k = 0.8 kN/m2.

The roofing is a horizontal plane. Therefore its shape coefficient takes on the value, μ_I= 0,8.

Assuming C_t = 1 and U_s = 1 (i.e., lacking any particular effects on the snow cover due to heat transmission from the building's interior and considering normal wind exposure), the snow load on the roof becomes:

$$s = \mu_1 \text{ x } U_s \text{ x } C_t \text{ x } s_k = 0{,}8 \text{ x } 1{,}0 \text{ x } 1{,}0 \text{ x } 0{,}8 \text{ kN/m}^2 = 0{,}64 \text{ kN/m}^2.$$

2.3.4 Wind loads

The basic wind velocity for the area in question is 27 m/s - about 97 km/h - (the average value measured over 10 minutes under standard exposure conditions, and having an annual probability of being exceeded of 0,02, which corresponds to a return period of 50 years).

Whether the wind direction is parallel to the principal frames, or orthogonal to them, the area of the walls directly affected by the wind's action (16,0 m in the first case, 35,0 m in the second) is in any event greater than the height of the building (15,0 m). Therefore, as per the provisions of EN 1991 1-4, the reference height, z_e, is assumed to coincide with the height of the building (z_e = 15,0 m), and a constant uniform distribution is taken for the pressures and depressions on the vertical walls: $q_p(z) = q_p(z_e)$ (Figure 5).

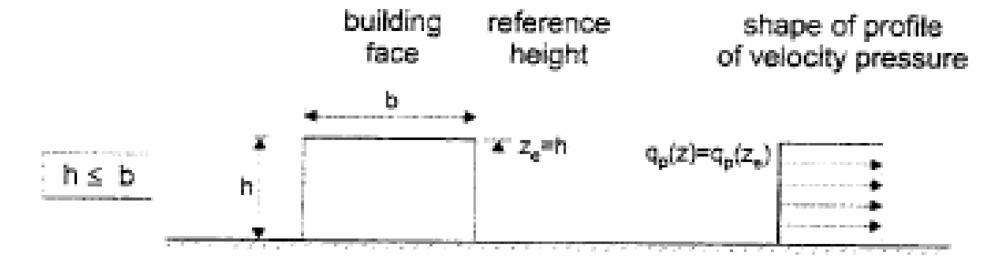


Figure 5. Wind pressure distribution.

We thus have:

$$q_p(z_e) = [1 + 7 \cdot I_v(z_e)] \cdot \frac{1}{2} \cdot \rho \cdot v_m^2(z_e)$$

in which:

$$v_m(z_e) = c_r(z_e) \cdot c_o(z_e) \cdot v_b$$

where v_b = 27 m/s, c_r (z_e) is the roughness factor and c_o (z_e) indicates the orography factor (which we can assume to be equal to 1, considering a level site). ρ represents the air density in the shade, which is generally a function of the site's altitude above sea level, the temperature and the pressure. The recommended value in the regulations is $\rho = 1{,}25$ kg/m^3.

The quantity I_v (z_e) represents the degree of turbulence, calculated at altitude z_e, and defined by the expressions:

$$I_v(z_e) = \begin{cases} \dfrac{k_1}{c_o(z_e) \cdot \ln\left(\dfrac{z}{z_0}\right)} & z_{min} \leq z \\ I_v(z_{min}) & z < z_{min} \end{cases}$$

where k_1 indicates the turbulence factor, whose recommended value is 1. As the building is situated in an urban area, the ground on which it is built is to be classified as category IV (according to the guidelines set forth in EN 1991-1-4), whence the resulting values, z_0 = 1.0 m and z_{min} = 10 m. Because $z_e > z_{min}$, it follows that:

$$I_v(z_e) = \frac{k_1}{1{,}0 \cdot \ln\left(\dfrac{z_e}{z_0}\right)} = \frac{1{,}0}{\ln\left(\dfrac{15{,}0}{1{,}0}\right)} = 0{,}369$$

The roughness factor is instead calculated via the expression:

$$c_r(z_e) = k_r \cdot \ln\left(\frac{z_e}{z_0}\right)$$

with:

$$k_r = 0{,}19 \cdot \left(\frac{z_0}{z_{0,II}}\right)^{0{,}07}$$

(where $z_{0,II}$ = 0,05 m).

We thus have:

$$c_{\mathrm{r}}(z_{\mathrm{e}}) = 0{,}19 \cdot \left(\frac{z_0}{z_{0,\mathrm{II}}} \right)^{0{,}07} \cdot \ln\left(\frac{z_{\mathrm{e}}}{z_0} \right) =$$

$$= 0{,}19 \cdot \left(\frac{1{,}0}{0{,}05} \right)^{0{,}07} \cdot \ln\left(\frac{15{,}0}{1{,}0} \right) = 0{,}634$$

whence it follows that:

$$v_{\mathrm{m}}(z_{\mathrm{e}}) = c_{\mathrm{r}}(z_{\mathrm{e}}) \cdot v_{\mathrm{b}} = 0{,}634 \cdot 27 = 17{,}12 \text{ m/s}$$

and

$$q_{\mathrm{p}}(z_{\mathrm{e}}) = [1 + 7 \cdot 0{,}369] \cdot \frac{1}{2} \cdot 1{,}25 \cdot (17{,}12)^2 = 668{,}9 \text{ N/m}^2$$

The wind pressure exerted on the building's external surfaces takes on the value:

$$w_{\mathrm{e}} = q_{\mathrm{p}}(z_{\mathrm{e}}) \cdot c_{\mathrm{pe}}$$

Thus, the resulting actions on surface areas A_{ref} are:

$$F_{\mathrm{w}} = c_{\mathrm{s}} c_{\mathrm{d}} \cdot \sum_{\text{surfaces}} (w_{\mathrm{e}} \cdot A_{\mathrm{ref}})$$

in which $c_s c_d$ represents the structural factor. For buildings of height less than or at most equal to 15 m, as in the current case, it can be assumed that $c_s c_d = 1{,}0$.

Now, regarding calculation of the external shape coefficients, c_{pe}, in the event that the wind direction is normal to the planes of the principal frames (Figure 6), we distinguish the zones A, B, D, E, F, G and H, whose extensions are determined as a function of the quantity "e":

$$e = \min\{b; 2h\} = \min\{35{,}0; 30{,}0\} = 30{,}0 \text{ m}$$

Regarding the building under exam, it holds that e > d (as d = 16,0 m), and the ratio *h*/*d* turns out to be 15/16 = 0,94 ≈ 1.0 From regulation tables therefore, we obtain:

surface A: c_{pe} = - 1.2 ⇒ $q_{pe}(z_e)$ x c_{pe} = 668,9 N/m^2 x (-1,2) = -802,7 N/m^2.
surface B: c_{pe} = - 0.8 ⇒ $q_{pe}(z_e)$ x c_{pe} = 668,9 N/m^2 x (-0,8) = -535,1 N/m^2.
surface D: c_{pe} = + 0.8 ⇒ $q_{pe}(z_e)$ x c_{pe} = 668,9 N/m^2 x (+ 0,8) = + 535,1 N/m^2.
surface E: c_{pe} = - 0.5 ⇒ $q_{pe}(z_e)$ x c_{pe} = 668,9 N/m^2 x (-0,5) = -334,5 N/m^2.
surface F: c_{pe} = - 1.8 ⇒ $q_{pe}(z_e)$ x c_{pe} = 668,9 N/m^2 x (-1,8) = -1204,0 N/m^2.
surface G: c_{pe} = - 1.2 ⇒ $q_{pe}(z_e)$ x c_{pe} = 668,9 N/m^2 x (-1,2) = -802,7 N/m^2.
surface H: c_{pe} = - 0.7 ⇒ $q_{pe}(z_e)$ x c_{pe} = 668,9 N/m^2 x (-0,7) = -468,2 N/m^2.

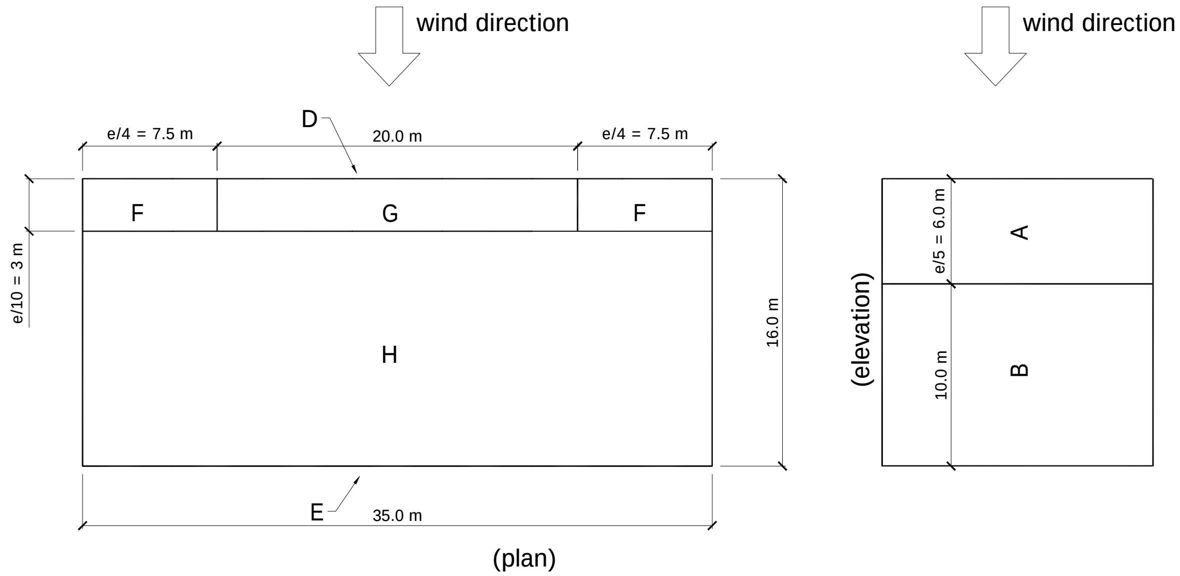


Figure 6. Wind direction orthogonal to the wall with greatest surface area.

In the event that the wind direction is parallel to the planes of the principal frames (Figure 7), the zones A, B, C, D, E, F, G and H determined by quantity "e" are instead:

$$e = \min\{b; 2h\} = \min\{16{,}0; 30{,}0\} = 16{,}0 \text{ m}$$

It still holds that $e < d$ (as d = 35,0 m), and the resulting ratio, h/d = 15/35 = 0,40. Regulation tables then furnish the following:

surface A: $c_{pe} = -\ 1{,}2$ $\Rightarrow q_{pe}(z_e)$ x c_{pe} = 668,9 N/m^2 x (-1,2) = -802,7 N/m^2.
surface B: $c_{pe} = -\ 0{,}8$ $\Rightarrow q_{pe}(z_e)$ x c_{pe} = 668,9 N/m^2 x (-0,8) = -535,1 N/m^2.
surface C: $c_{pe} = -\ 0{,}5$ $\Rightarrow q_{pe}(z_e)$ x c_{pe} = 668,9 N/m^2 x (-0,5) = -334,5 N/m^2.
surface D: $c_{pe} = +\ 0{,}8$ $\Rightarrow q_{pe}(z_e)$ x c_{pe} = 668,9 N/m^2 x (+ 0,8) = + 535,1 N/m^2.
surface E: $c_{pe} = -\ 0{,}4$ $\Rightarrow q_{pe}(z_e)$ x c_{pe} = 668,9 N/m^2 x (-0,4) = -267,6 N/m^2.
surface F: $c_{pe} = -\ 1{,}8$ $\Rightarrow q_{pe}(z_e)$ x c_{pe} = 668,9 N/m^2 x (-1,8) = -1204,0 N/m^2.
surface G: $c_{pe} = -\ 1{,}2$ $\Rightarrow q_{pe}(z_e)$ x c_{pe} = 668,9 N/m^2 x (-1,2) = -802,7 N/m^2.
surface H: $c_{pe} = -\ 0{,}7$ $\Rightarrow q_{pe}(z_e)$ x c_{pe} = 668,9 N/m^2 x (-0,7) = -468,2 N/m^2.
surface I: $c_{pe} = -\ 0{,}2$ $\Rightarrow q_{pe}(z_e)$ x c_{pe} = 668,9 N/m^2 x (-0,2) = -133,8 N/m^2.

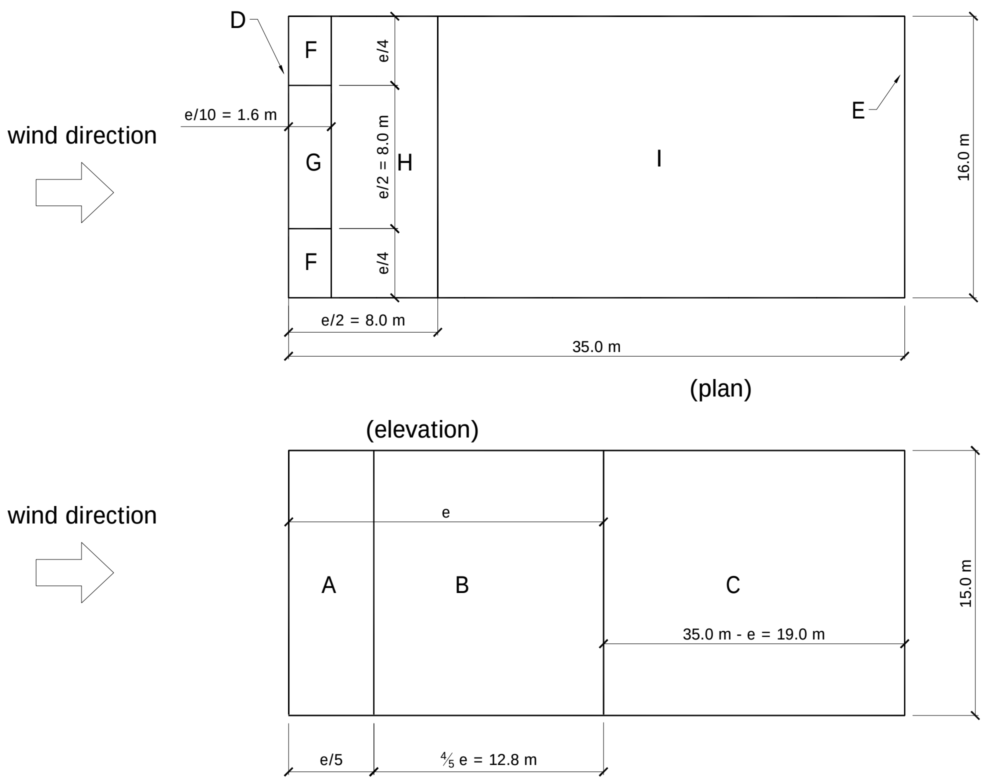


Figure 7. Wind direction parallel to the wall with greatest surface area.

2.4 Combined loads and structural analysis

As per the provisions of EN 1990, we consider the following rule for combining the actions relative to ULS:

$$\sum_j \gamma_{G,j} \cdot G_{k,j} + \gamma_{Q,1} \cdot Q_{k,1} + \sum_{i>1} \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_{k,i}$$

Regarding SLS, it is instead necessary to consider the following combination formulas:

characteristic combination: $\sum_j G_{k,j} + Q_{k,1} + \sum_{i>1} \psi_{0,i} \cdot Q_{k,i}$

frequent combination: $\sum_{j} G_{k,j} + \psi_{1,1} \cdot Q_{k,1} + \sum_{i>1} \psi_{2,i} \cdot Q_{k,i}$

quasi-permanent combination: $\sum_{j} G_{k,j} + \sum_{i \geq 1} \psi_{2,i} \cdot Q_{k,i}$

The building in question falls into usage category A; therefore, the combination coefficients for the imposed loads take on the values:

$$\psi_0 = 0{,}7 \quad \psi_1 = 0{,}5 \quad \psi_2 = 0{,}3$$

For snow (altitude above sea level less than 1000 m), we have:

$$\psi_0 = 0{,}5 \quad \psi_1 = 0{,}2 \quad \psi_2 = 0{,}0$$

While for the wind, they are:

$$\psi_0 = 0{,}6 \quad \psi_1 = 0{,}2 \quad \psi_2 = 0{,}0$$

Moreover, we have:

$$\gamma_G = 1{,}35 \text{ (or 1,00 where unfavorable)}$$

$$\gamma_{Q,1} = 1{,}50 \text{ (or 0,00 where unfavorable)}$$

$$\gamma_{Q,i} = 1{,}50 \text{ (or 0,00 where unfavorable)}$$

The foregoing general expressions are rendered specific on a case by case basis by assuming the main variable action to be due to either the imposed load, the wind (considered to blow alternately along the building's two principal directions) or snow.

Indicating the relevant parameters:

- G: the self and permanent weights (including $g_{beam\text{-}1} = 3{,}75$ kN/m, $g_{beam\text{-}2} = 3{,}00$ kN/m, $g_{col} = 3{,}00$ kN/m and $g_{per} = 5{,}55$ kN/m^2);
- Q: the imposed loads (equal to 2,0 kN/m^2);
- Q: the roof snow load (= s = 0,64 kN/m^2);
- Q_{w1}, Q_{w2}: the load due to the wind (respectively, in the directions orthogonal and parallel to the 35,0 m-wide wall);

For ULS, we thus have the followings relations:

- main variable action: imposed load

$$1{,}35\ [1{,}0]\ G + 1{,}5\ [0{,}0]\ Q + 1{,}50\ [0{,}0]\ (0{,}6\ Q_{w,is} + 0{,}5\ Q_s)$$

- main variable action: wind

$$1{,}35\ [1{,}0]\ G + 1{,}5\ [0{,}0]\ Q_{w,i} + 1{,}50\ [0{,}0]\ (0{,}7\ Q_s + 0{,}5\ Q_s)$$

- main variable action: snow

$$1{,}35\ [1{,}0]\ G + 1{,}5\ [0{,}0]\ Q_s + 1{,}50\ [0{,}0]\ (0{,}7\ Q_s + 0{,}6\ Q_{w,is})$$

It should be noted that as the roof is accessible, with an imposed load value greater than that of the snow load, the load combinations that will certainly have the greatest effect are those that account for the variable load on the floors, while excluding the presence of snow (not also that these two actions are not cumulative on the same surface).

Moreover, it is worthwhile pointing out that, as we are working in the linear field, a direct linear proportionality exists between the causes (the design loads on the structure) and the effects (the stresses on various elements). For this reason, it is possible to load the structure with the considered actions using their characteristic values, barring however subsequent combination of the effects produced. Indicating E as any given effect (which may be a stress characteristic - bending moment, normal stress – ore even a generic component of displacement or deformation), for ULS, it thus follows that:

$$\sum_{j} \gamma_{G,j} \cdot E_{Gk,j} + \gamma_{Q,1} \cdot E_{Qk,1} + \sum_{i>1} \gamma_{Q,i} \cdot \psi_{0,i} \cdot E_{Qk,i}$$

For the sake of simplicity, the following includes only a few load combinations (with the respective analysis results), chosen as particularly significant and best suited to exemplifying and clarifying the issues at hand. It should also be pointed out that the two stairwells present in the structure have been omitted in the FEM model (figure 8 shows two different views of the structural model of the building). The model is composed of “beam” elements, representing both the beams and columns, and “shell” elements, representing the floors.

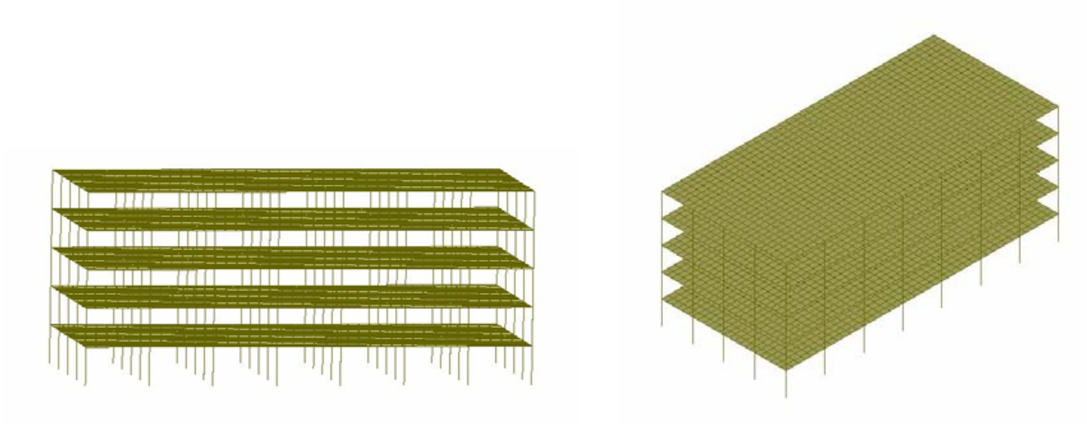


Figure 8. Views of the structural model of the building

I) In order to maximize the compressive stress at the base of the columns, it is necessary to distribute the imposed loads on all the competent surfaces of the column in question. However, the quantity of the imposed load can be legitimately reduced by multiplying by the factor α_n, less than 1, according to the relation:

$$\alpha_n = \frac{2+(n-2)\cdot \psi_0}{n} = \frac{2+(5-2)\cdot 0{,}7}{5} = \frac{2+2{,}1}{5} = 0{,}82$$

(where n is the number of planes on which the imposed load acts). Thus, in this case the imposed load can be reduced by 18%.

By considering the imposed load in its entirety, that is to say, omitting parameter α_n, we obtain the normal stress diagrams illustrated in Figure 9, which moreover include, by way of example, the numerical values of the compressive stress for two columns in the outer row for three particular load combinations out of all the various possibilities. The values reported respectively refer to the following combinations of actions:

Comb (1) = 1,35 x (permanent loads and self-weight) + 1,5 x (imposed load);

Comb (2) = 1,0 x (permanent loads and self-weight) + 1,5 x (wind direction parallel to the larger wall);

Comb (3) = 1,35 x (permanent loads and self-weight) + 1,5 x (wind direction parallel to the larger wall) + 1,5 x 0,7 x (imposed load).

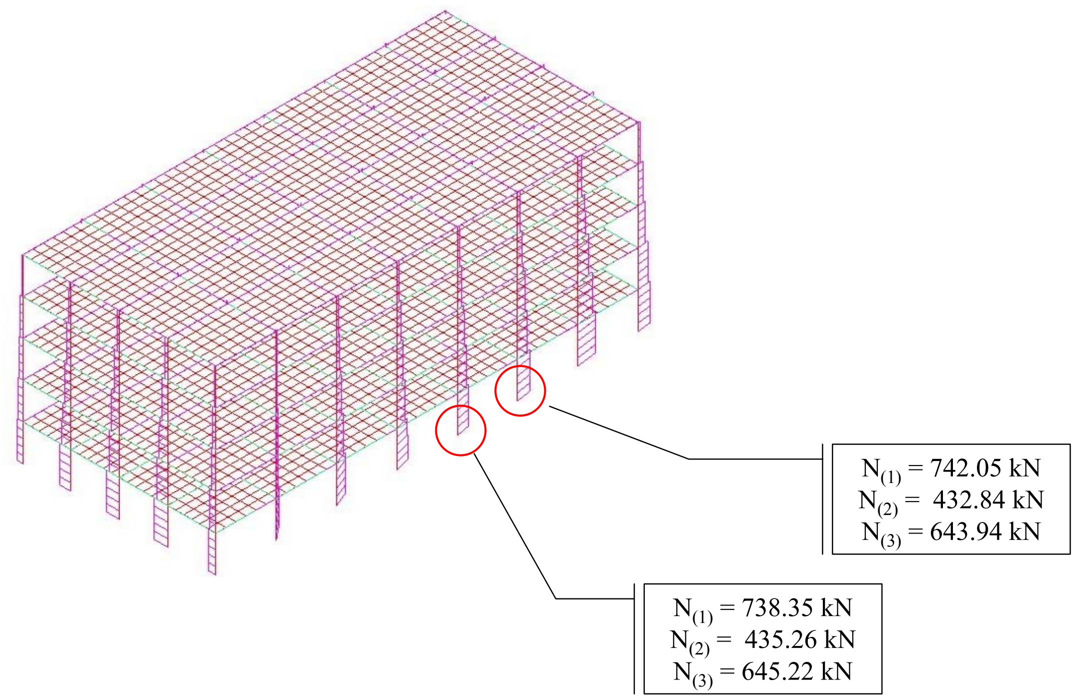


Figure 9. Compressive stress diagrams, with specific reference to the base sections of two outer columns.

II) Maximizing the positive bending moment in the beams following rigorous procedures requires distributing the imposed loads in a "checkerboard" arrangement, according to the theory of influence lines (for example, Figure 10 indicates the proper distribution of loads in order to maximize the positive bending moment in section 'A' at the midpoint of a beam in any given principal frame). A simplified arrangement is however permissible, and can be obtained by considering the imposed loads on all the planes other than the one in question to be a fixed constant, rather than variable (Figure 11).

The imposed load can moreover be reduced by multiplying by the factor α_A (<1), which can be determined via the following relation:

$$\alpha_A = \frac{5}{7}\cdot\psi_0 + \frac{A_0}{A} = \frac{5}{7}\cdot 0{,}7 + \frac{10}{A} = 0{,}5 + \frac{10}{A}$$

where A indicates the area affected by the load.

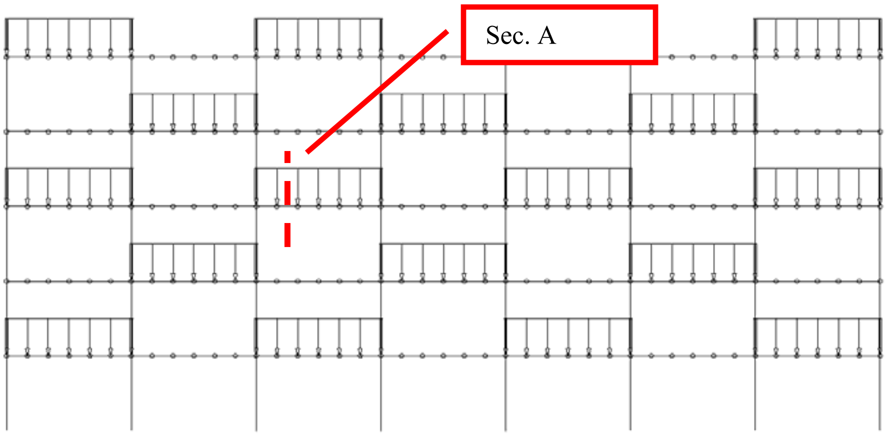


Figure 10. Rigorous distribution of the imposed loads for maximizing the positive bending moment in section A (indicated).

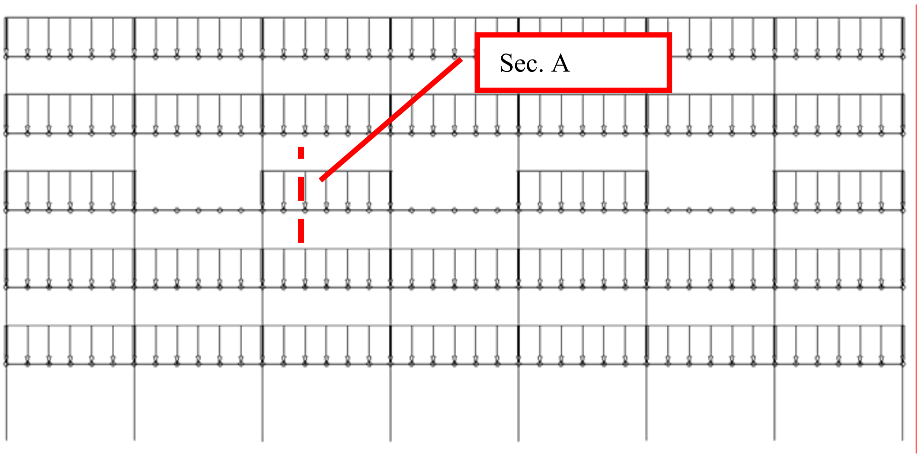


Figure 11. Simplified distribution of the imposed loads for maximizing the positive bending moment in section A (indicated).

Figure 13 shows the envelop diagram for the bending moment with various load combinations for the principal frame indicated in Figure 12 (two numerical values for the previously indicated section 'A' have been included).

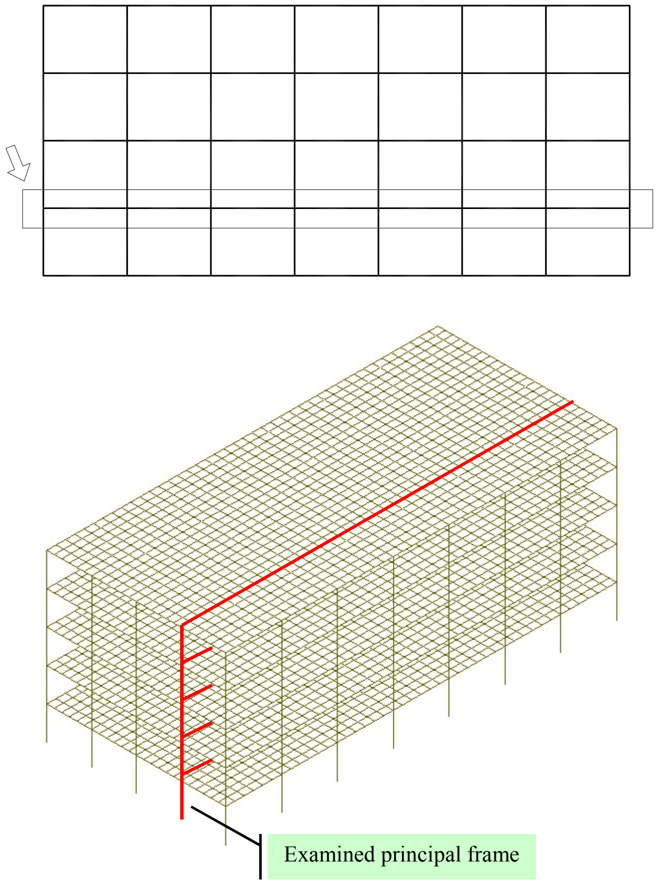


Figure 12. Plane view of the structural scheme - principal frame examined in figure 13.

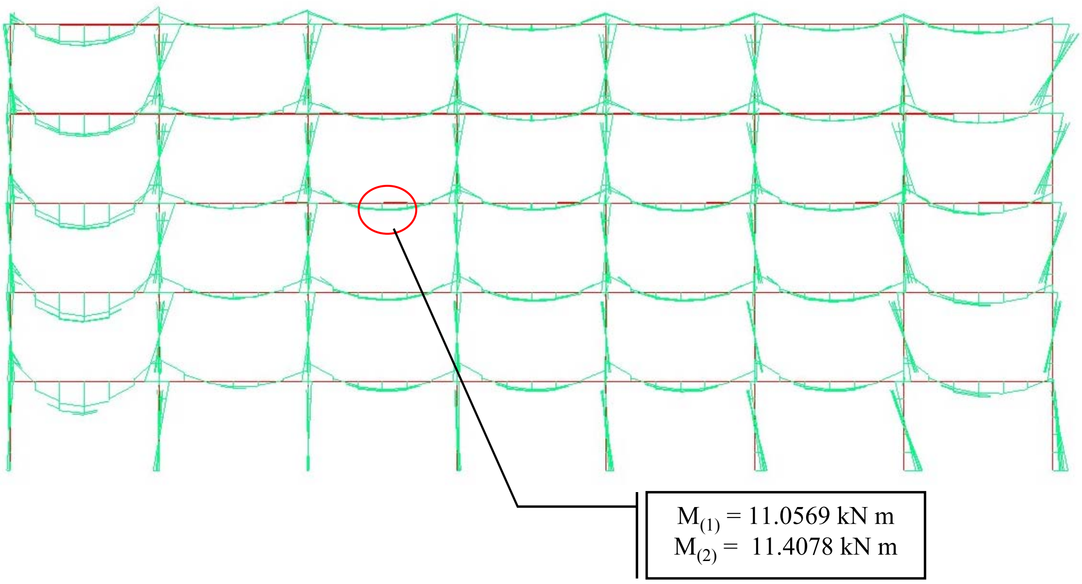


Figure 13. Envelope diagram of bending moments for the principal frame indicated in figure 12.

Figure 13 shows the numerical values of the bending moments obtained in the section indicated for the following load combinations:

Comb (1) = 1,35 x (permanent load/self-weight) + 1,5 x (imposed load - simplified load distribution);

Comb (2) = 1,35 x (permanent load/self-weight) + 1,5 x (imposed load - rigorous checkerboard load distribution).

Note that the bending moments in the two cases differ by only about 3%, which reveals how the simplified application of imposed loads provided for by Eurocodes 1 is absolutely reasonable and yields, in any event, very precise results.

• The same procedure can be used to determine all the other stress characteristics (bending moment, shear, normal stress, torsion). By way of example, figure 14 refers to a suitable distribution of the variable loads on floors for determining the minimum bending moment in section B.

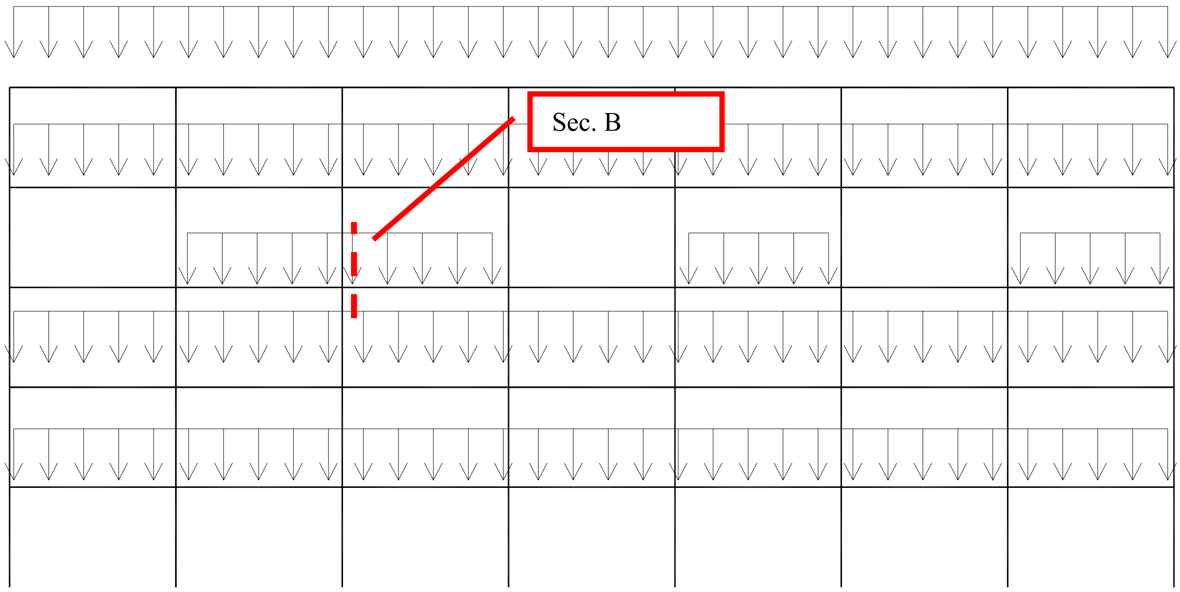


Figure 14. Distribution of variable loads on the floors for minimizing the negative bending moment in joint section B.

3 INDUSTRIAL BUILDING

3.1 Description of the building

The building in question is a large warehouse, with a rectangular, 64,5 m x 32,0 m, floor plan, located in Pisa (Italy). There are seven longitudinal bays, 9,0857 m each, with three rows of columns set 15,50 m apart. The intermediate floors are inserted into the four vertices of the rectangle and supported by columns. Figures 15-17 show various views of the overall building and its dimensions (in cm).

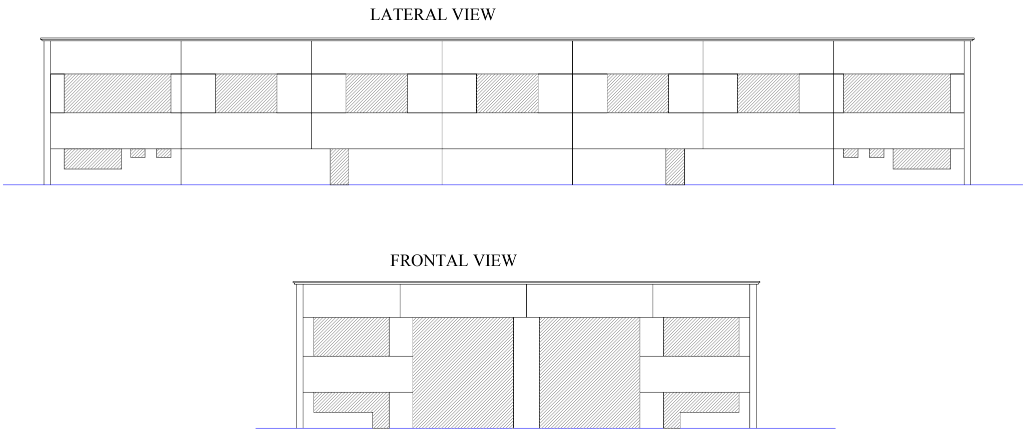


Figure 15. Elevation of industrial building

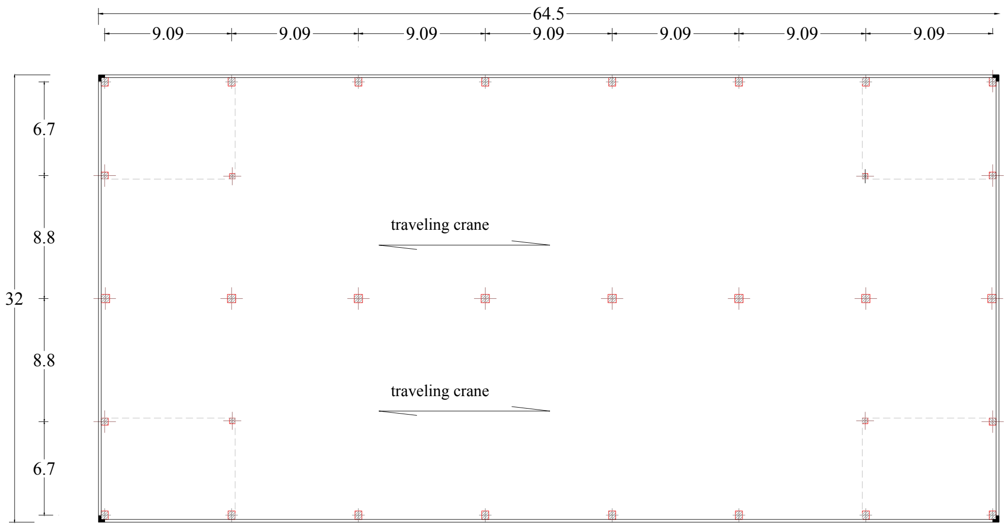


Figure 16. Plane view of industrial building

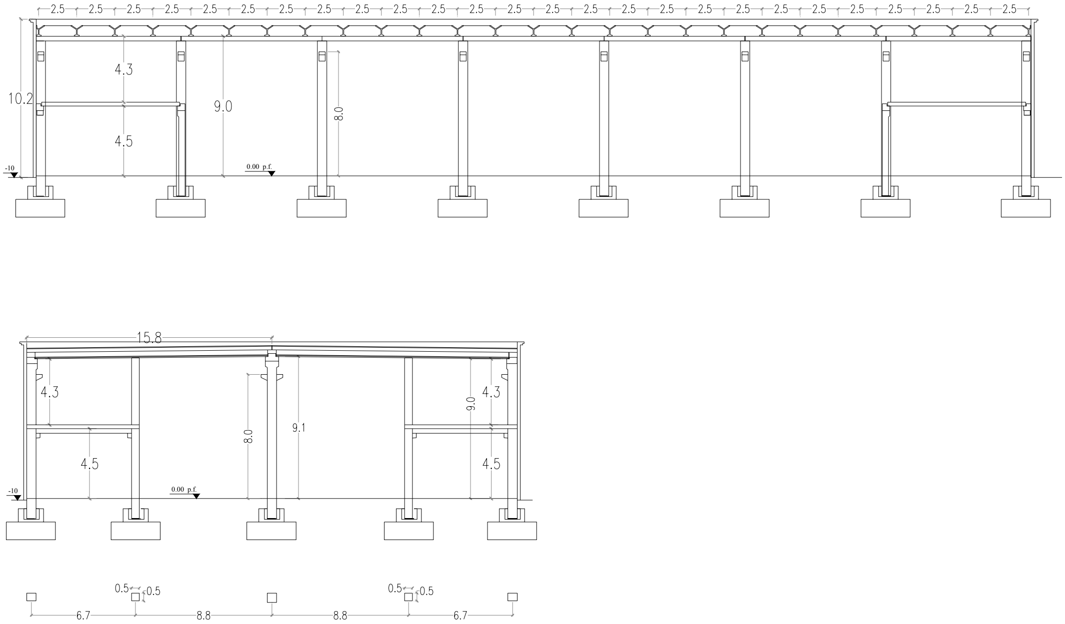


Figure 17. Cross-sectional views of industrial building

The roofing beams, spanning 15,50 m with a simple abutment static scheme, are made up of prefabricated “Y” section members (figure 18). The design calls for the two longitudinal aisles to bear traveling cranes (each with a maximum load capacity of 10^4 kg). The roofing panels are set directly on the Y beams.

3.2 Materials

The building elements caste on site (i.e., the foundation substructures) were constructed with normal weight concrete, class C 30/37 (E_c = 33 kN / mm^2) and normal inert filler materials,. The prefabricated elements were instead caste with a higher class concrete: C 40/50. The reinforcement steel is characterized by a linear elastic modulus, E_s = 200 kN/mm^2.

3.3 Definition of the design loads

3.3.1 Self-weight and permanent loads

The self-weights of the structure have been determined by assuming the reinforced concrete to have a density of 25 kN/m^3.

Roofing “Y” beams:
Area of the straight beam section = 1021 cm^2 (= 0,1021 m^2)
Self-weight of the beams per linear meter:
$p_{\text{beam-Y}}$ = 25 kN/m^3 x 0,1021 m^2 = 2,552 kN/m

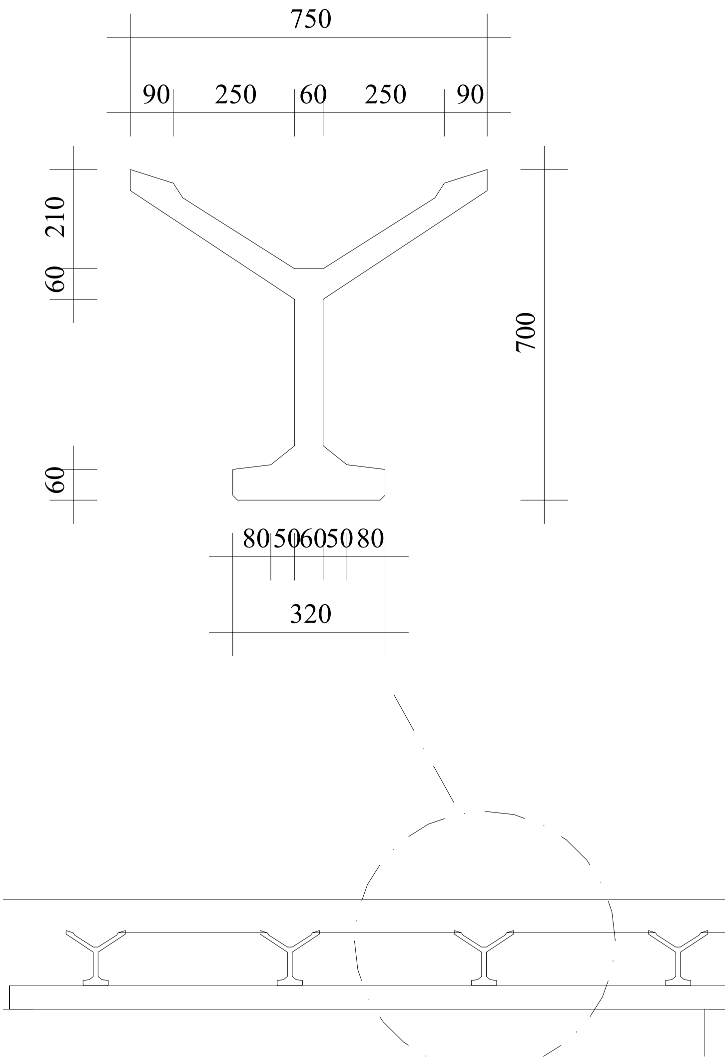


Figure 18. Roofing “Y” beams (units: mm)

Central columns:

Each column in the center row (Figure 19) has a volume of 3,78 m^3 (also considering the enlarged sections sustaining the crane rails) and a self-weight of 94,55 kN.

Lateral columns:

Each column in the side rows (Figure 20) has a volume of 3,06 m^3 (also considering the enlarged sections sustaining the crane rails) and a self-weight of 76,6 kN.

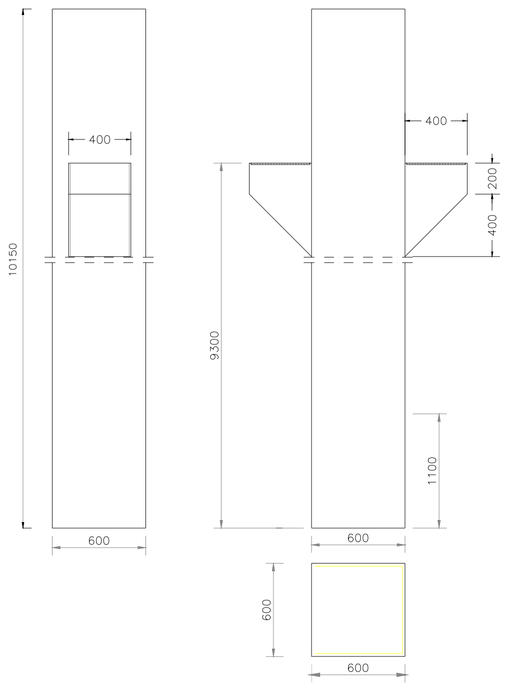


Figure 19. Central columns

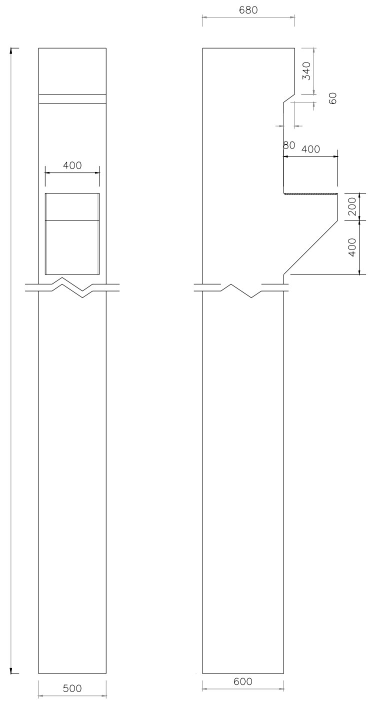


Figure 20. Lateral columns

Lateral longitudinal beams:

The straight section of these beams has a cross-sectional area of 0,23 m^2 (Figure 21); therefore their weight per linear meter is 25 kN/m^3 x 0,23 m^2 =5,75 kN/m.

Central longitudinal beams:

The longitudinal beams connecting the columns in the central row (spanning 9,0857 m, equal to the length of the longitudinal beams in correspondence to the columns of the outer row) have a straight cross section of 0,333 m^2 (Figure 22), and therefore weigh 25 kN/m^3 x 0,333 m^2 = 832,5 kN/m.

Lastly, to complete the list of the permanent loads acting, we consider the roofing panels to have a weight, p_{roof} = 500 N/m^2.

3.3.2 Snow load

The characteristic value of the ground snow load for the area in question is s_k= 0,8 kN/m^2. Since the roof is a horizontal plane, the shape coefficient takes on the value 0,8 and, therefore, assuming $c_t = c_e$= 1, we have a roof snow load of:
s = 0,8 x 1,0 x 1,0 x 0,8 kN/m^2 = 0,64 kN/m^2.

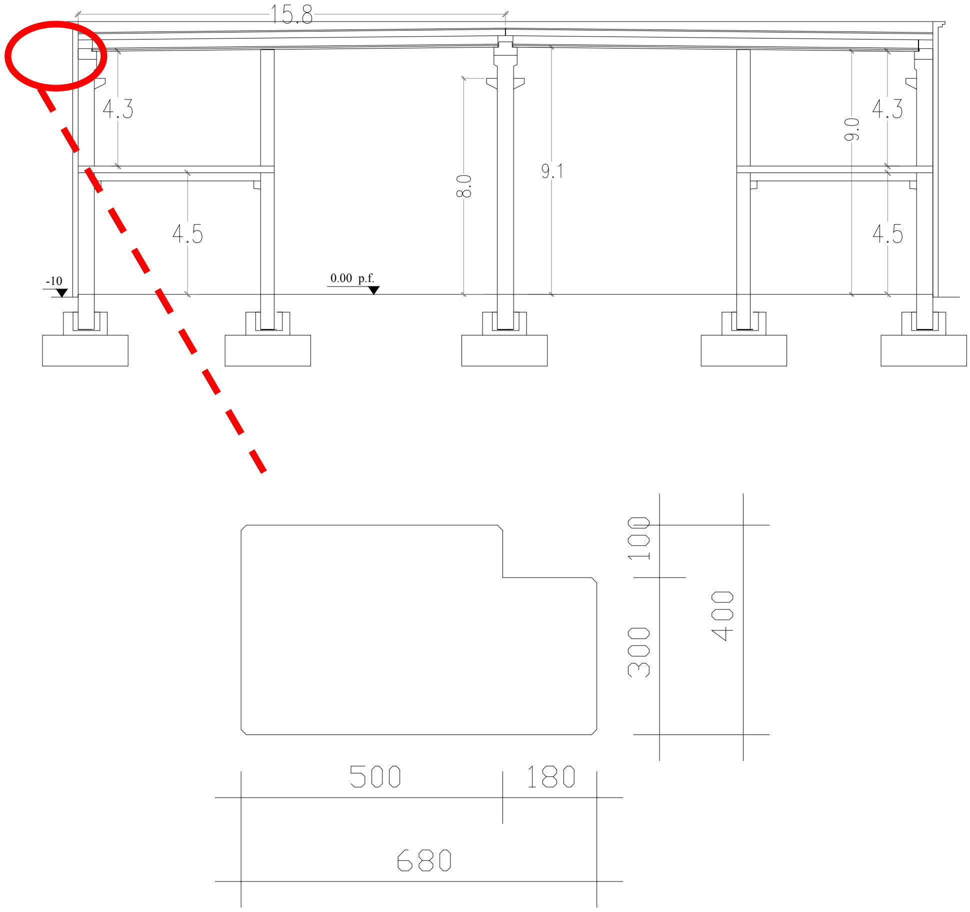


Figure 21. Lateral longitudinal beams (units: m and mm)

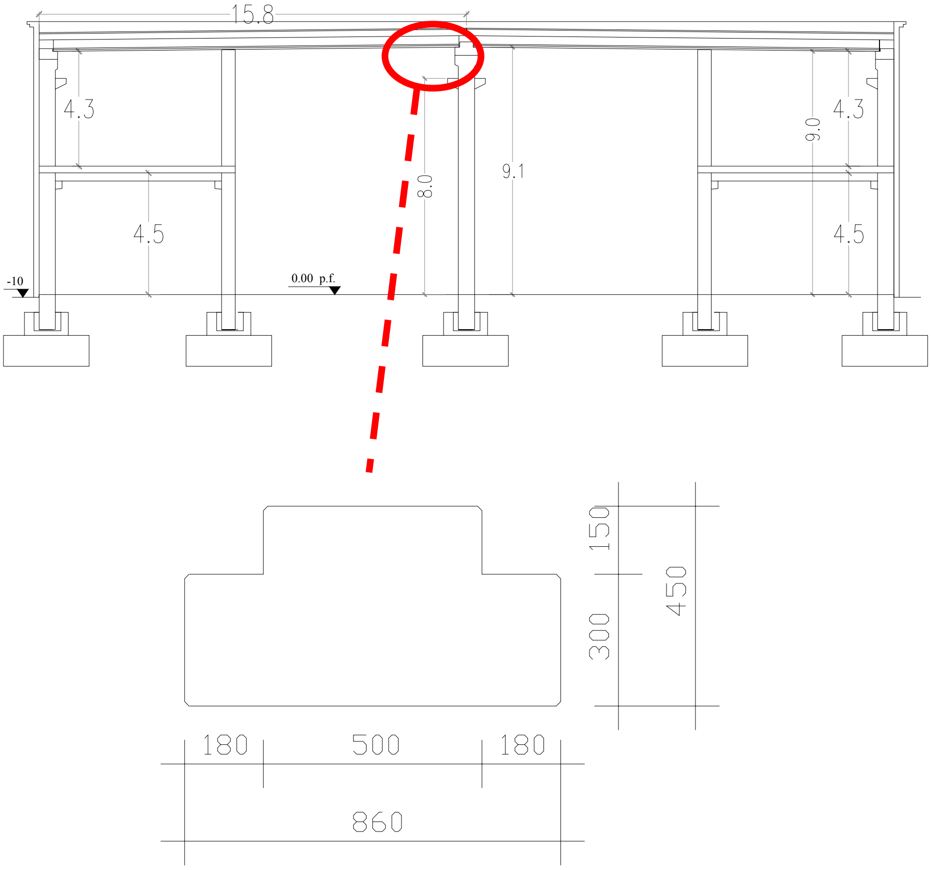


Figure 22. Central longitudinal beams (units: m and mm)

3.3.3 Wind Load

The basic wind velocity is the same as that previously calculated for the housing building (as it is located in the same area), which is 27 m/s. In this case, however, the wind exposure class is different: in fact the building is assumed to be in category III (that is, according to EN 1991-1-4, *"an area with regular cover of vegetation or buildings or with isolated obstacles with reparation of maximum 20 obstacle heights (such as villages, suburban terrain, permanent forest)"*, for which we have parameters $z_0 = 0{,}3$ m, $z_{min} = 5{,}0$ m.

Regardless of the direction of the wind, the width of the walls directly involved (that is, those orthogonal to such direction) are greater than the height of the building (respectively, 32,0 m > h = 10,2 m and 64,5 m > h = 10,2 m). Therefore, as per EN 1991-1-4, we may assume an equivalent reference height that coincides with the building's height (z_e = 10,2 m), and a uniform distribution of the pressures and depressions on the vertical walls. Calculations yield the following values:

$$I_v(z_e) = \frac{k_1}{1{,}0 \cdot \ln\left(\frac{z_e}{z_0}\right)} = \frac{1{,}0}{1{,}0 \cdot \ln\left(\frac{10{,}2}{0{,}3}\right)} = 0{,}283$$

$$c_r(z_e) = k_r \cdot \ln\left(\frac{z_e}{z_0}\right) = 0{,}19 \cdot \left(\frac{z_0}{z_{0,II}}\right)^{0{,}07} \cdot \ln\left(\frac{z_e}{z_0}\right) = 0{,}19 \cdot \left(\frac{0{,}3}{0{,}05}\right)^{0{,}07} \cdot \ln\left(\frac{10{,}2}{0{,}3}\right)$$
$$= 0{,}19 \cdot 1{,}134 \cdot 3{,}526 = 0{,}759$$

$$v_m(z_e) = c_r(z_e) \cdot c_o(z_e) \cdot v_b = 0{,}759 \cdot 27 = 20{,}5 \text{ m/s}$$

$$q_p(z_e) = [1 + 7 \cdot I_v(z_e)] \cdot \frac{1}{2} \cdot \rho \cdot v_m^2(z_e) = (1 + 7 \cdot 0{,}283) \cdot \frac{1}{2} \cdot 1{,}25 \cdot (20{,}5)^2 = 783 \text{ N/m}^2$$

Whether the wind direction is oriented normal to the building's longitudinal axis, or parallel to it, we obtain

$$e = \min\{b; 2h\} = 2h = 20{,}4 \text{ m}$$

Surfaces A, B, C, D, E, F, G, H and I can thus be distinguished on the roof, each of which is characterized by different values of the shape coefficient (although the buildings and dimensions at play are quite different, the distribution of the roofing zones is analogous to that indicated in figures 6 and 7), specifically:

surface A: $c_{pe} = -1{,}2$ $\Rightarrow q_{pe}(z_e) \text{ x } c_{pe} = 783 \text{ N/m}^2 \text{ x } (-1{,}2) = -944{,}2 \text{ N/m}^2$.
surface B: $c_{pe} = -0{,}8$ $\Rightarrow q_{pe}(z_e) \text{ x } c_{pe} = 783 \text{ N/m}^2 \text{ x } (-0{,}8) = -629{,}4 \text{ N/m}^2$.
surface C: $c_{pe} = -0{,}5$ $\Rightarrow q_{pe}(z_e) \text{ x } c_{pe} = 783 \text{ N/m}^2 \text{ x } (-0{,}5) = -393{,}4 \text{ N/m}^2$.
surface D: $c_{pe} = +0{,}7$ $\Rightarrow q_{pe}(z_e) \text{ x } c_{pe} = 783 \text{ N/m}^2 \text{ x } (+0{,}7) = +550{,}8 \text{ N/m}^2$.
surface E: $c_{pe} = -0{,}3$ $\Rightarrow q_{pe}(z_e) \text{ x } c_{pe} = 783 \text{ N/m}^2 \text{ x } (-0{,}3) = -236{,}0 \text{ N/m}^2$.
surface F: $c_{pe} = -1{,}8$ $\Rightarrow q_{pe}(z_e) \text{ x } c_{pe} = 783 \text{ N/m}^2 \text{ x } (-1{,}8) = -1416{,}2 \text{ N/m}^2$.
surface G: $c_{pe} = -1{,}2$ $\Rightarrow q_{pe}(z_e) \text{ x } c_{pe} = 783 \text{ N/m}^2 \text{ x } (-1{,}2) = -944{,}2 \text{ N/m}^2$.
surface H: $c_{pe} = -0{,}7$ $\Rightarrow q_{pe}(z_e) \text{ x } c_{pe} = 783 \text{ N/m}^2 \text{ x } (-0{,}7) = -550{,}8 \text{ N/m}^2$.
surface I: $c_{pe} = -0{,}2$ $\Rightarrow q_{pe}(z_e) \text{ x } c_{pe} = 783 \text{ N/m}^2 \text{ x } (-0{,}2) = -157{,}4 \text{ N/m}^2$.

3.3.4 Crane loads

In order to account for dynamic effects, the actions consequent to the cranes and their operation are formulated as: $F_k = \varphi_i F$. The general relation contains the following quantities:

- φ_1 = effects of vibration on the crane structure due to the lifting of loads off the ground (to be applied to the crane's self-weight);
- φ_2 = dynamic effects of transfer of the lifting force from the ground to the crane (to be applied to the load lifted);
- φ_4 = dynamic effects induced during the crane's movement over the rails or tracks (to be applied to both the crane's self-weight and the load lifted).

Regarding φ_1, it is calculated as:

$$\varphi_1 = 1 \pm a = 1{,}1$$

where $1+ a = 1+ 0{,}1$ is the maximum pulsation value.

Instead, in order to calculate φ_2 , it is necessary to know the class of the loading device and the velocity at which the load is raised. Let us suppose the cranes in question to be class HC3, with a lifting speed of v_h = 5,5 m/min (equivalent to 5,5/60 = 0,092 m/s). Now, each single crane has a maximum capacity of 100 kN. Given that

$$\varphi_2 = \varphi_{2,\min} + \beta_2 \cdot v_h ,$$

we now need to include the lifting velocity in m/s.

The values set forth by ENV 1991-1-5 for class HC3 devices are β_2 = 0,51 and φ_2 = 1,15; thus we have:

$$\varphi_2 = \varphi_{2,\min} + \beta_2 \cdot v_h = 1{,}15 + 0{,}51 \cdot 0{,}092 = 1{,}197 .$$

Lastly, assuming that the rail gauge tolerances are respected, we can assume φ_4 = 1,0, as provided for by ENV 1991-1-5.

Each crane, together with its guidance system and motor weigh 5 kN. Given the geometry, dimensions and volume of the various parts of the device (rails, motor, crane, etc.), each crane exerts onto the support beams a maximum vertical force of 10,5 kN and a minimum of 6,5 kN (these values are intended as characteristic ones, exclusive of the load lifted). Such values have been calculated by assuming that the weight of the rail beams is distributed uniformly onto the supports, and that the barycenter of the device (including the motor and crane) do not move further than 1,5 m from the ideal axis of support.

Now, adding the dynamic effects via coefficient φ_1, we obtain the following values for the two reactions:

$$R_{\text{max-k}} = 1{,}1 \text{ x } 10{,}5 \text{ kN} = 11{,}55 \text{ kN}$$

$$R_{\text{min-k}} = 1{,}1 \text{ x } 6{,}5 \text{ kN} = 7{,}15 \text{ kN}$$

The maximum liftable load is instead multiplied by coefficient φ_2 in order to account for dynamic effects.

$$Q_{\text{up-k}} = 100 \text{ kN x } 1{,}197 = 119{,}7 \text{ kN}$$

(this load also refers to a characteristic value).

3.4 Load combinations and structural analysis

In the following we shall examine some significant load combinations to calculate the stresses on the building's main structural elements. The structural analysis itself is conducted on the basis of the following assumptions: linear elastic material constitutive laws, first-order analysis (the effects of geometric non-linearities are certainly negligible in a one-storey building such as the one in question).

3.4.1 Analyses of loads on the roofing Y beams

The Y beams supporting the roofing panels are arranged to provide a simple abutment static scheme, spanning 15,5 m, and with an interaxis, i = 2,45 m (Figures 17 and 18).

In order to maximize the positive bending moment in the beam's midline, the following load combination must be applied:

$$1{,}35 \text{ x } (p_{\text{beam-y}} + p_{\text{roof}} \text{ x } i) + 1{,}0 \text{ x } (s \text{ x } i)$$

which yields a load equal to:

$$1{,}35 \text{ x } (2{,}552 \text{ kN/m} + 0{,}5 \text{ kN/m}^2 \text{ x } 2{,}45 \text{ m}) + 1{,}50 \text{ x } (0{,}64 \text{ kN/m}^2 \text{ x } 2{,}45 \text{ m}) =$$

$$1{,}35 \text{ x } 3{,}777 \text{ kN/m} + 1{,}50 \text{ x } 1{,}568 \text{ kN/m} = 5{,}099 \text{ kN/m} + 2{,}352 \text{ kN/m} = 7{,}451 \text{ kN/m}$$

with a corresponding positive bending moment in the middle section:

$$M = \frac{7{,}451 \cdot L^2}{8} = \frac{7{,}451 \cdot (15{,}50)^2}{8} = 223{,}763 \text{ kNm}$$

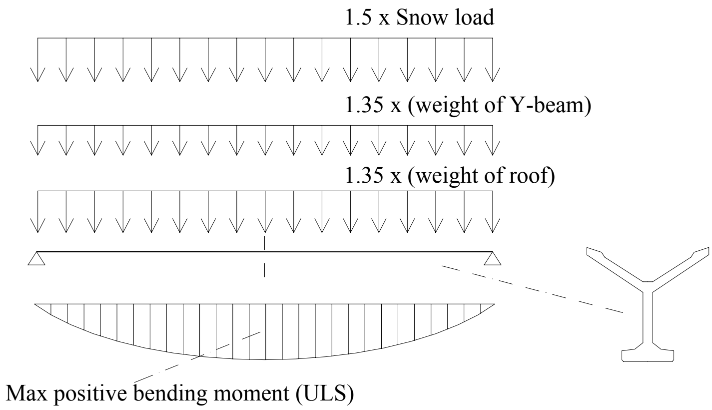


Figure 23. Load combination for maximizing the positive bending moment on the midline of the roofing Y beams.

This same load combination (that is, that which maximizes vertical loads in the downwards direction), also yields the maximum shear value on the beam, obviously in correspondence to the support of the beam itself.

Any possible inversion of sign of the bending moment can be evaluated by eliminating the snow load and maximizing the wind depression effects on the roof (to this aim, the maximum depression values are considered, that is to say, those for zone ‘G ', just as in the previous section dealing with determination of the wind load), as shown in Figure 24.

The following load combination is to be adopted for this check:

$$1{,}0 \text{ x } (p_{\text{beam-y}} + p_{\text{roof}} \text{ x } i) + 1{,}50 \text{ x } (\text{wind x } i) =$$

$$= 3{,}777 \text{ kN/m} – 1{,}50 \text{ x } (0{,}9442 \text{ kN/m}^2 \text{ x } 2{,}45 \text{ m}) =$$

$$= 3{,}777 \text{ kN/m} – 3{,}47 \text{ kN/m} = 0{,}307 \text{ kN/m}$$

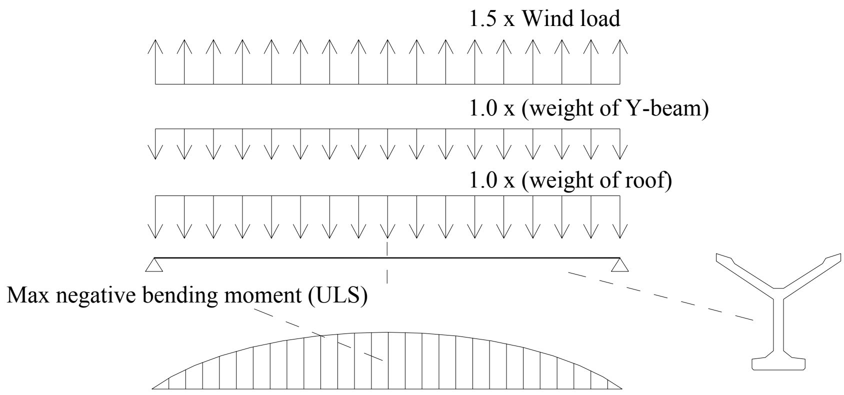


Figure 24. Load combination for evaluating possible sign inversion
of the bending moment in the midline of roofing Y beams.

Ultimately, no sign inversion occurs in the bending moment at the midline of the roofing Y beams.

Another important check of the Y beams must also be mentioned, without however going into the details of the calculations. This consists in evaluating the strength of the critical section for connection of the inclined portions under the actions of the loads exerted by the roof (as schematized in Figure 24).

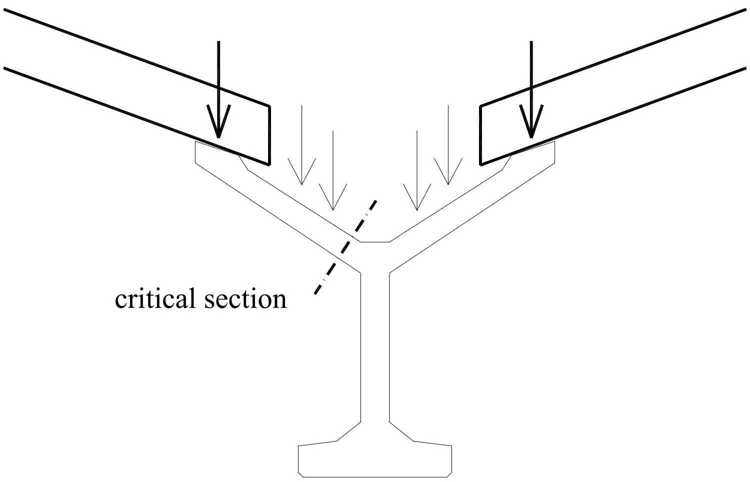


Figure 25. Y beam critical section.

3.4.2 Analyses of loads on the lateral longitudinal beams

The beams in question are simply laid over spans of 9,0857 m (the support scheme is illustrated in figure 16 and its section in Figure 17). The self-weight of these beams is 5,75 kN/m and each supports three Y beams, with the overlying roofing structures (Figure 17). The loading scheme of the longitudinal beams is shown in Figure 26.

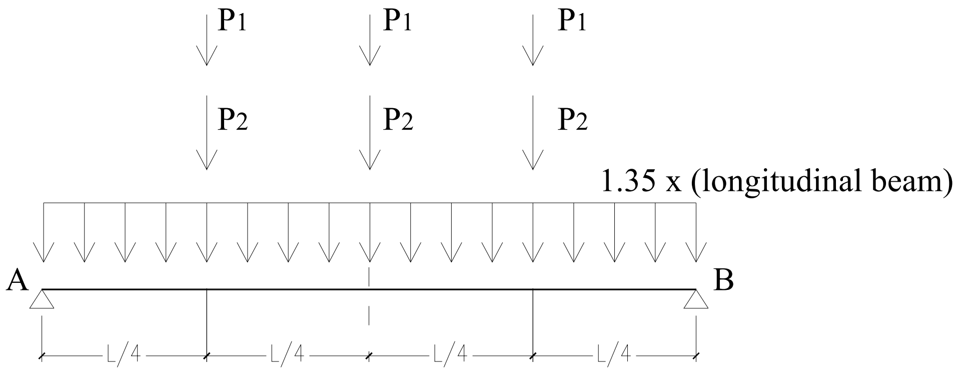


Figure 26. Static scheme of the lateral longitudinal beams with the respective loads.

P_1 indicates the actions consequent to the Y beam's self-weight and the portion of the roofing structures that each of these sustains; P_2 instead represents the actions due to the snow load. Overall, in situations of Ultimate Limit States, P_1 and P_2 correspond to the maximum reactions on the supports of the Y beams, or in other terms, the shear value that acts on the Y beams, which was calculated in the preceding section:

$$(P_1 + P_2) = \text{Max Shear (ULS)} = 57{,}745 \text{ kN}$$

It should be recalled that this value serves to account for the appropriate coefficients for combining actions at Ultimate Limit States.

Regarding the longitudinal beams' self-weight at Ultimate Limit States, we have:

$$1{,}35 \text{ x } 5{,}75 \text{ kN/M} = 7{,}762 \text{ kN/m}$$

Thus, the vertical reactions for the longitudinal beams are:

$$A = B = \frac{3}{2}\cdot(P_1 + P_2) + \frac{7{,}762\cdot 9{,}0857}{2} =$$
$$86{,}625 + 35{,}262 = 121{,}887$$

For such actions, the maximum bending moment in the midline of the longitudinal beams is therefore:

$$M = A\cdot\frac{L}{2} - (P_1 + P_2)\cdot\frac{L}{4} = 121{,}887\cdot\frac{9{,}0857}{2} - 57{,}745\cdot\frac{9{,}0857}{4} =$$
$$= 553{,}7144 - 131{,}1748 = 422{,}54 \text{ kN m}$$

3.4.3 Analyses of loads on the lateral columns

The static scheme of the columns is that of a cantilever of height h = 10 m, fixed at the base, but with its top end free (figure 27).

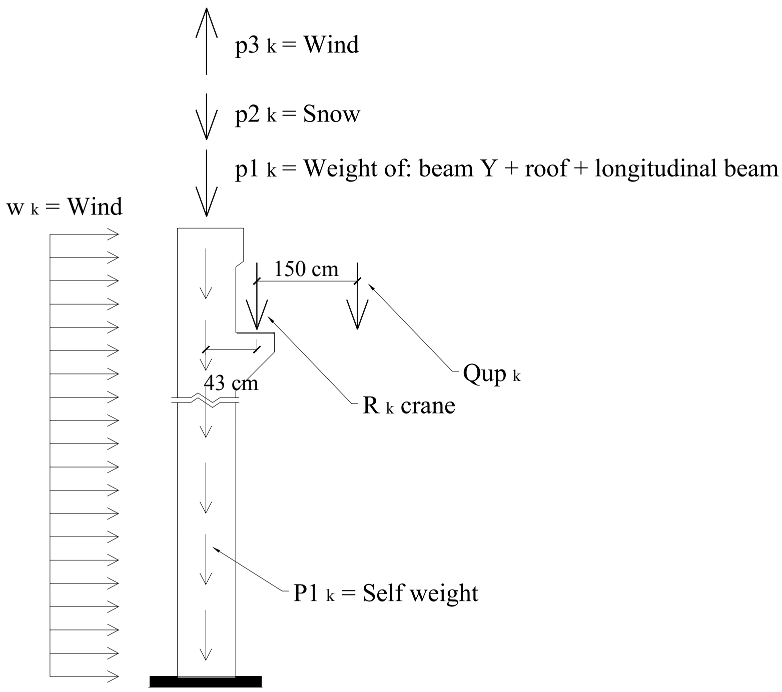


Figure 27. Constraints and loading conditions of the current lateral columns.

The characteristic values of the loads are the following:

- $p1_\mathrm{k} = 3{,}777 \cdot \dfrac{15{,}5}{2} + 5{,}75 \cdot \dfrac{9{,}0857}{2} = 55{,}393\ \text{kN}$

(self-weight and permanent loads on the column summit)

- $p2_\mathrm{k} = 0{,}64 \cdot \dfrac{15{,}5}{2} \cdot 9{,}0857 = 45{,}065\ \text{kN}$ (snow);

- $p3_\mathrm{k} = 0{,}1574 \cdot \dfrac{15{,}5}{2} \cdot 9{,}0857 = 11{,}083\ \text{kN}$

(wind in depression - zone ‘I’);

- $w_\mathrm{k} = 550{,}8 \cdot 9{,}0857 = 4{,}997\ \text{kN/m}$

(wind pressure on the directly impacted wall - zone ‘D’);

- R_k = 11500 N (= 11,50 kN), maximum vertical reaction due to the crane - including dynamic effects;
- $P_{1\mathrm{k}}$ = 76,60 kN, column self-weight (the self weight that, to be on the safe side, can be consider to be applied to the column summit);
- $Q_{\mathrm{up\ k}}$ = 119,7 kN (= 119700 N), vertical load lifted by the crane, inclusive of dynamic effects.

Indicating the various loads as ‘G’, for the self-weight and permanent loads, ‘W’ for the wind and ‘S’ for snow, the bending moments and normal stress at the column base are evaluated for the followings three load combinations (ULS):

I) $1{,}35\ G + 1{,}5\ W + 1{,}5 \times 0{,}7\ S + 1{,}5 \times 0{,}7\ Q_{up}$

II) $1{,}35\ G + 1{,}5\ S + 1{,}5 \times 0{,}7\ W + 1{,}5 \times 0{,}7\ Q_{up}$

III) $1{,}35\ G + 1{,}5\ Q_{up} + 1{,}5 \times 0{,}7\ W + 1{,}5 \times 0{,}7\ S$

Combination I

$$N_{\mathrm{I}} = 1{,}35\ (P_{1k} + p_{1k} + R_k) + 1{,}5\ (-\ p_{3k}) + 1{,}5 \times 0{,}7\ (p_{2k}) + 1{,}5 \times 0{,}7\ (Q_{up}) =$$

$= 1{,}35\ (77{,}60\ \text{kN} + 55{,}393\ \text{kN} + 11{,}50\ \text{kN}) - 1{,}5 \times 11{,}083\ \text{kN} + 1{,}5 \times 0{,}7 \times 45{,}064\ \text{kN} + 1{,}5 \times 0{,}7 \times 119{,}70\ \text{kN} = 351{,}444\ \text{kN}$

$$M_{\mathrm{I}} = 1{,}35\ (R_k \times 0{,}43\ \text{m}) + 1{,}5\ w_k\ h^2/2 + 1{,}5 \times 0{,}7\ (Q_{up} \times 1{,}93\ \text{m}) = 286{,}725\ \text{kNm}$$

Combination II

$$N_{\mathrm{II}} = 1{,}35\ (P_{1k} + p_{1k} + R_k) + 1{,}5\ (p_{2k}) + 1{,}5 \times 0{,}7\ (-\ p_{3k}) + 1{,}5 \times 0{,}7\ (Q_{up}) =$$

$= 1{,}35\ (77{,}60\ \text{kN} + 55{,}393\ \text{kN} + 11{,}5\ \text{kN}) + 1{,}5 \times 45{,}064\ \text{kN} - 1{,}5 \times 0{,}7 \times 11{,}083\ \text{kN} + 1{,}5 \times 0{,}7 \times 119{,}7\ \text{kN} = 376{,}711\ \text{kN}$

$$M_{\mathrm{II}} = 1{,}35\ (R_k \times 0{,}43\ \text{m}) + 1{,}5 \times 0{,}7\ w_k\ \mathrm{h}^2/2 + 1{,}5 \times 0{,}7\ (Q_{up} \times 1{,}93\ \text{m}) = 275{,}482\ \text{kNm}$$

Combination III

$$N_{\mathrm{III}} = 1{,}35\ (P_{1k} + p_{1k} + R_k) + 1{,}5 \times 0{,}7\ (p_{2k}) + 1{,}5 \times 0{,}7\ (-\ p_{3k}) + 1{,}5\ (Q_{up}) =$$

$= 1{,}35\ (77{,}6\ \text{kN} + 55{,}393\ \text{kN} + 11{,}5\ \text{kN}) + 1{,}5 \times 0{,}7 \times 45{,}064\ \text{kN} - 1{,}5 \times 0{,}7 \times 11{,}083\ \text{kN} + 1{,}5 \times 119{,}7\ \text{kN} = 410{,}297\ \text{kN}$

$$M_{\mathrm{III}} = 1{,}35\ (R_k \times 0{,}43\ \text{m}) + 1{,}5 \times 0{,}7\ w_k\ h^2/2 + 1{,}5 \times (Q_{up} \times 1{,}93\ \text{m}) = 379{,}441\ \text{kNm}$$

As the results clearly reveal, the last load combination is the most onerous for the column connection section.

CHAPTER VII - EXAMPLE OF A STEEL BUILDING

Igor Kovše

Institute of Metal Constructions, Ljubljana, Slovenia

Summary

The example includes the determination of loads and load combinations on the steel building, according to the European constructional standards "Eurocodes". The building is a steel industrial hall made of hot rolled steel sections. The structural system is a series of rigidly supported portal frames. The relevant loads acting on the structure are determined from the appropriate parts of the standard EN 1991-1-1 [1] and load combinations required for the ultimate limit state verification are given. The use of the steel-specific Eurocode EN 1993-1-1 is presented through an example of the partial verification of the steel column.

1 INTRODUCTION

An example of a steel industrial hall is presented in this chapter. The emphasis of this example is laid on the determination of the relevant loads and load combinations according to European standards "Eurocodes". The verification presented here is not complete and is meant to be an indication on how to proceed with the use of the material (steel) specific Eurocodes, after the effects of actions due to load combinations are obtained.

1.1 Background materials

The determination of the permanent and variable loads – imposed loads, snow loads and wind loads are treated in the relevant parts of the European standard EN 1991-1: EN 1991-1-1 [2], EN 1991-1-3 [3] and EN 1991-1-4 [4]. The verification of steel members is covered in detail in the steel-specific European standard EN 1993-1-1 [5]. Further information is available in the working material of JCSS [6] and specialized literature.

2 DEFINITION OF THE SYSTEM

2.1 The structural system

The building presented in this example is a single storey steel industrial hall. The hall is 20 m wide and 20 m long. Its structural system is a space frame consisting of 5 single bay portal frames positioned in the XZ plane, connected by 5 horizontal purlins laying in the plane of the roof and running parallel to the Y-axis (see Figure 1). The wind bracing is positioned in the roof plain between the 4th and 5th portal frame.

The span of the portal frames is 20 m and the distance between them is 5 m. The portal frames have fully rigidly supported bases. The height of the columns of the portal frame is 7 m, the height at the roof ridge is 8,5 m. The consoles supporting two crane runway girders are connected to both columns of the portal frames at the height 6 m. The connections of the purlins and the wind bracing to the portal frames is hinged and the crane runway girder is continuous, supported by consoles.

2.2 Properties of the sections

The elements of the frame are made of standard hot rolled steel sections. The columns of the portal frames are HEA 400 sections, the beams of the portal frames are IPE 550 sections, the purlins are HEA 200 sections, the crane runway girders are I 260 sections, the consoles supporting crane girders are I 280 sections and the wind bracing is made of angle sections H 60x60x6. The geometrical properties of the sections: the area A, elastic resistance moment $W_{y,el}$ and plastic resistance moment W_{ypel} are given in Table 1.

Table 1. Geometrical properties of the steel sections.

Element	**Section**	A [cm^2]	$W_{y,el}$ [cm^3]	$W_{y,pl}$ [cm^3]
portal column	HEA 400	159	2310	2560
portal beam	IPE 550	134	2441	2780
purlin	HEA 200	54	389	430
crane girder	I 260	53,3	441	514
console	I 280	61	542	632
wind bracing	H 60x60x6	6,9	8,5	13,6

The material is constructional steel S 235 with the yield stress

$$f_y = 235 \text{ N/mm}^2 \tag{1}$$

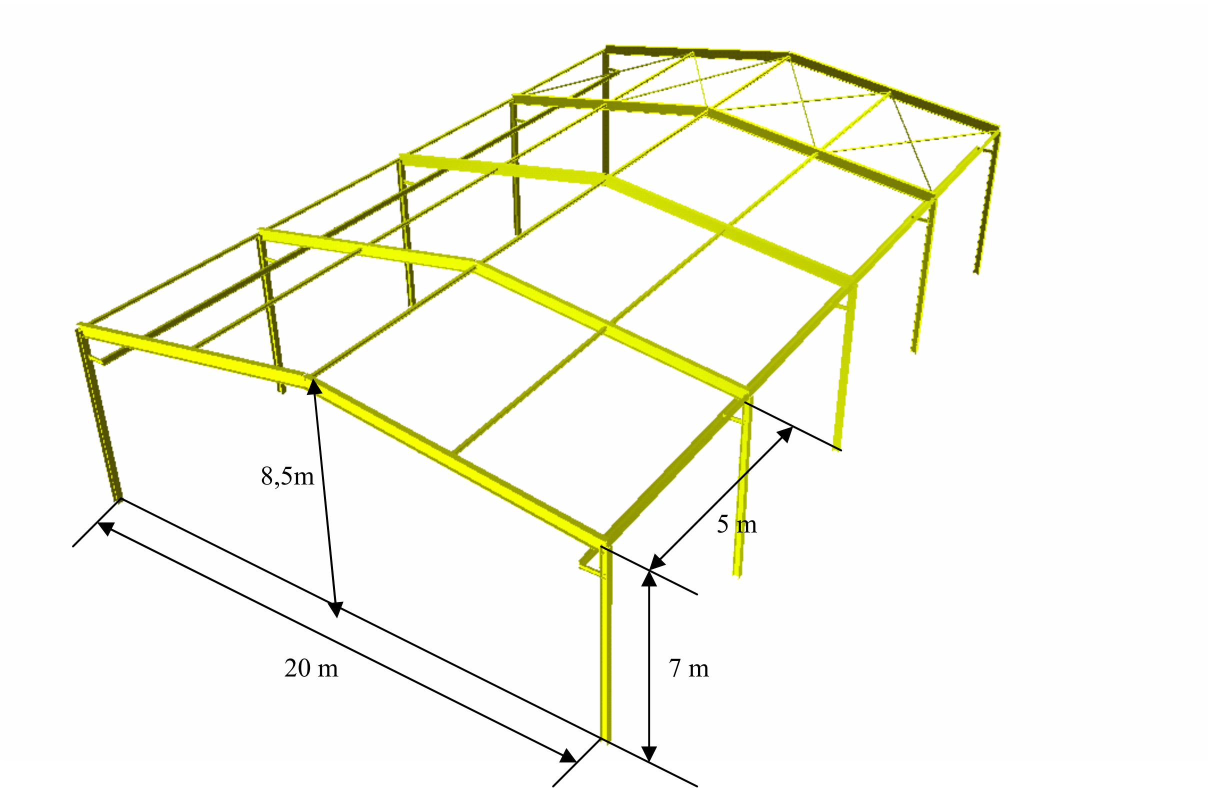


Figure 1. Isometric view of the structural system of the industrial steel hall.

3 DEFINITION OF ACTIONS

3.1 Permanent actions

According to EN 1991-1-1 [2], the self weight of the structural elements and the roof and wall cladding is classified as a permanent fixed action.

3.1.1 Self weight of structural members

Self weight of the steel members is calculated from the nominal dimensions of the members (the cross section area A) and the characteristic value for the density of steel γ. The densities of structural materials are given in the standard EN 1991-1-1 [2] with their mean value or with the range of mean values. If the range of mean values is given, the value chosen should depend on the knowledge of the source and quality of the material for the individual project. The mean value chosen is taken as a characteristic design value. For structural steel the density γ is given (EN 1991-1-1, Appendix A) in the range 77,0 kN/m^3 to 78,5 kN/m^3. In this example we choose the value

$$\gamma = 78{,}5 \text{ kN/m}^3 \tag{2}$$

Based on this characteristic value, the self weight per unit length g_k, is calculated according to formula

$$g_k = \gamma \ A \tag{3}$$

and is given for structural members in the Table 2.

Table 2. Self weight per unit length of structural members.

Element	**Section**	$\boldsymbol{g_k}$ [kN/m]
portal column	HEA 400	1,25
portal beam	IPE 550	1,05
purlin	HEA 200	0,42
crane girder	I 260	0,42
console	I 280	0,48
wind bracing	H 60x60x6	0,05

3.1.2 Roof and wall cladding

For the roof and wall cladding a specific weight per unit area of h = 0,3 kN/m^2 is taken. The weight of the roof and wall cladding is supported by portal frames. For the inner frames this distributed weight is acting on the width W = 5 m, which results in load per unit length

$$g_{roof} = 5 \cdot 0{,}3 = 1{,}5 \text{ kN/m} \ , \quad g_{wall} = 5 \cdot 0{,}3 = 1{,}5 \text{ kN/m} \tag{4}$$

and half that value for outer frames.

3.2 Imposed loads

The imposed loads are classified according to EN 1991-1-1 [2] as variable free actions.

3.2.1 Imposed loads on roofs

The imposed loads on roofs depend on the categorisation of the roof (EN 1991-1-1 [2], table 6.9). The roof of the industrial steel hall in this example corresponds to the category H: "roof not accessible except for normal maintenance and repair". For this category the recommended values for the distributed and concentrated imposed loads on roofs are: q_k = 0,4 kN/m^2 and Q_k = 1,0 kN. The concentrated load Q_k should be used for a separate verification of the local resistance of the roof structure. Since this example deals with the partial verification of the main frames, we will consider only the distributed load q_k, which results in the imposed load on the inner frame:

$$p_{\mathrm{roof}} = 5 \cdot 0{,}4 = 2{,}0 \text{ kN/m} \tag{5}$$

For outer frames only one half of the above value applies.

3.2.2 Actions imposed by cranes

The loads induced by the crane moving on a crane runway girder are determined from the pattern of wheel loads of the crane. For this example we choose the 200 kN crane of the span 20 m, running on a crane runway girder on two wheels with S = 3 m wheel span (Figure 2). The maximum and minimum loads on the runway girder are P_{max} = 180 kN, P_{min} = 60 kN, which are equally distributed on the two wheels. We assume that these values already include the dynamic effect using the suitable dynamic multiplication factor. For the verification purposes we choose the maximum load P_{max} and the position of the crane so that the centreline between the wheels is in the plane of the middle portal frame.

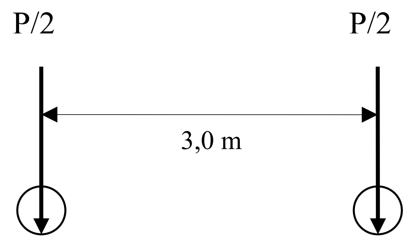


Figure 2. The wheel pattern of the crane.

For the horizontal loads induced by the movement of the crane we assume the side force H_x = 0,1 P_{max} in the plane of the frame and the braking force H_y = 0,15 P_{max} normal to the plane of the frame.

3.3 Climatic actions

3.3.1 Snow loads

The Slovenian national annex to prEN 1991-1-3 [3] gives the table of characteristic values of the ground snow load s_k for the relevant altitude and load zone (table 3).

Table 3: Characteristic values of the ground snow load s_k [kN/m^2] for Slovenia.

altitude [m]	**Snow load zone**			
	A	**B**	**C**	**D**
0	0,25	-	-	-
100	0,25	1,4	1,7	-
200	0,50	1,4	1,7	-
300	0,75	1,5	1,9	3,0
400	1,00	1,6	2,1	3,0
500	1,20	1,7	2,3	3,5
600	1,60	1,8	2,7	4,0
700	-	2,0	3,2	4,5
800	-	2,2	3,7	5,0
900	-	2,4	4,2	6,0
1000	-	2,7	5,4	7,5
1100	-	3,0	6,2	9,0
1200	-	3,3	7,0	10,5
1300	-	3,6	7,8	12,0
1400	-	3,9	8,6	13,5
1500	-	4,2	9,2	15,0

The building in this example is located in Ljubljana, Slovenia at the altitude 290 m in the snow load zone C. The interpolation from the values in table 3 gives us the characteristic value of the ground snow load at this location:

$$s_k = 1{,}88 \text{ kN/m}^2 \tag{6}$$

The exposure coefficient C_e is taken equal to 1,0 since the building is located in the industrial zone and neither the reduction due to “wind sweep” conditions nor the increment due to “sheltered” conditions apply. The normal thermal insulation conditions of the roof are assumed by choosing the thermal coefficient C_t equal to 1,0.

The snow load shape coefficients μ are dependent on the shape of the roof. In our example we have a “duopitched” type of the roof with the angles of pitch α_1 = α_2 = atan(1,5/10) = 8,5º. The snow load shape coefficients for this angle are the same: μ_1 = μ_2 = 0,8. According to prEN 1991-1-3 [3] we have to consider two snow load arrangements, as shown in Figure 3.

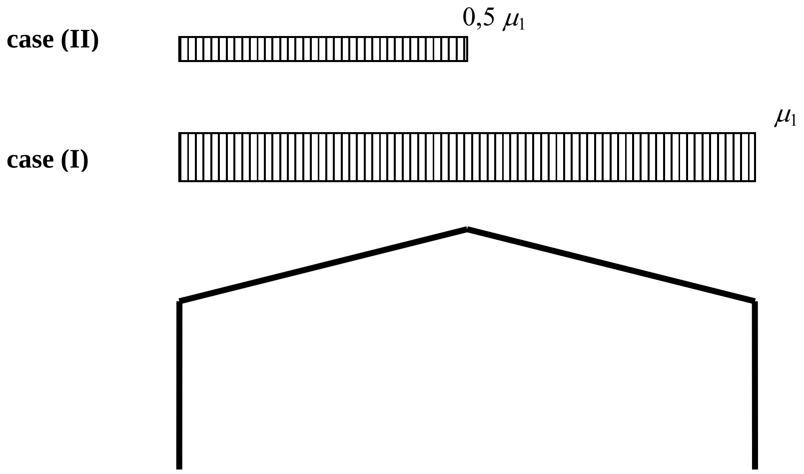


Figure 3. Different load cases for the snow load.

The snow load on the roof corresponding to the snow load shape coefficient μ_1 is obtained from the formula:

$$s_{\text{roof}} = \mu_1 \cdot C_e \cdot C_t \cdot s_k = 0{,}8 \cdot 1{,}0 \cdot 1{,}0 \cdot 1{,}88 = 1{,}5 \text{ kN/m}^2 \quad (7)$$

And the snow load per unit length acting on one inner portal frame is:

$$s = s_{\text{roof}} \cdot W = 1{,}5 \cdot 5 = 7{,}5 \text{ kN/m} \quad (8)$$

The snow load per unit length acting on outer frames is half the above value.

3.3.2 Wind loads

According to ENV 1991-2-4 [4] (Eurocode on wind actions) for buildings up to 200 m height the wind loads can be calculated using the simpler, quasi-static procedure. The first step in this procedure is to determine the basic wind velocity v_b. The most part of Slovenia, including the location of the building in this example, is located in the zone with reference wind velocity $v_{b,0}$ = 25 m/s. Using the usual value 1,0 for the directional factor c_{dir} and seasonal factor c_{season}, the basic wind velocity is

$$v_b = v_{b,0} \cdot c_{\text{dir}} \cdot c_{\text{season}} = 25 \text{ m/s} \quad (9)$$

The mean wind velocity $v_m(z)$ at the height z is calculated from the base wind velocity and two factors $c_o(z)$ and $c_r(z)$. The orography factor $c_o(z)$ takes into account the changes in terrain. Since the building is situated in the industrial zone, we choose the value 1,0. The roughness factor $c_r(z)$ accounts for height of the structure and roughness of the terrain and is calculated by

$$c_r(z) = k_r \ln(z / z_0), \qquad z_{\min} < z < 200 \text{ m} \quad (10)$$

where we have k_r = 0,22 , z_0 = 0,3 m and $z_{\min}$ = 8 m for terrain category III (suburban or industrial areas). With these factors, the mean wind velocity is

$$v_{\min}(z) = c_r(z)\, c_o(z)\, v_b = 0{,}22 \ln(8{,}5/0{,}3) \cdot 1{,}0 \cdot 25 = 18{,}4 \text{ m/s} \quad (11)$$

Next we determine the peak velocity pressure $q_p(z)$ from the equation

$$q_p(z) = \left[1+7\,\frac{k_I}{c_o(z)\ln(z/z_0)}\right] \tfrac{1}{2}\,\rho\, v_m^2(z) = 0{,}655 \text{ kN/m}^2 \qquad (12)$$

In the above equation ρ is the air density (in most regions ρ = 1,25 kg/m^3) and k_I is the turbulence factor, which is in general equal to 1,0.

In what follows we will consider only the wind direction in the plane of portal frames. The external pressure coefficients for the vertical walls of the building are c_{pe} = 0,64 and c_{pe} = -0,3 on the windward (zone D) and leeward (zone E) side of the building respectfully. These are based on the width to height ratio d/h = 20/8,5 = 2,35. The external pressure coefficients for the duopitch roof, determined for the four roof zones G, H, J and I are -1,2, -0,8, -0,3 and -0,3 respectfully. The internal pressure coefficient is taken c_{pi} = -0,3. The wind loadings per unit length w (in kN/m) are then calculated using the influence width W = 5 m for internal frame:

$$w = (c_{pe}+c_{pi})\, q_p\, W \qquad (13)$$

When performing summation of external and internal pressure coefficients in the above formula, only the unfavourable cases (equal signs) are considered. The wind loads for the internal frame are shown on the Figure 4.

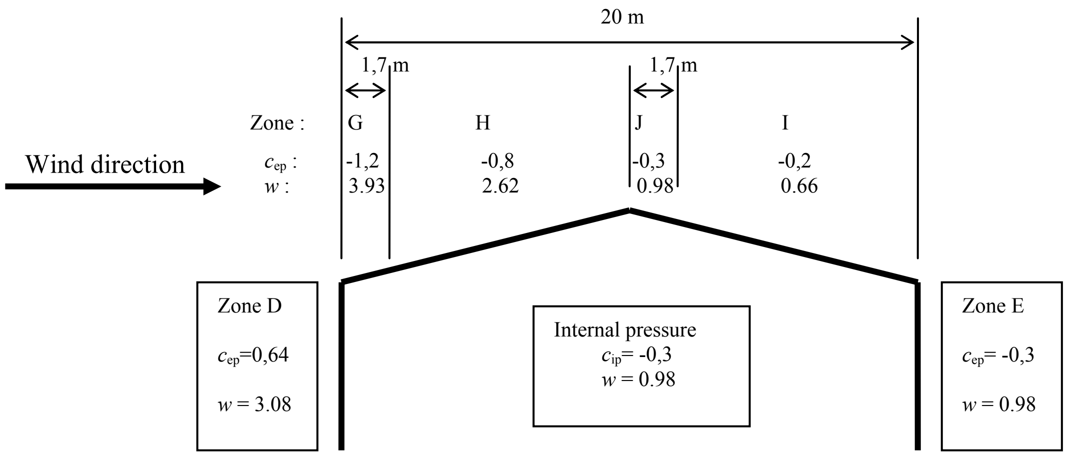


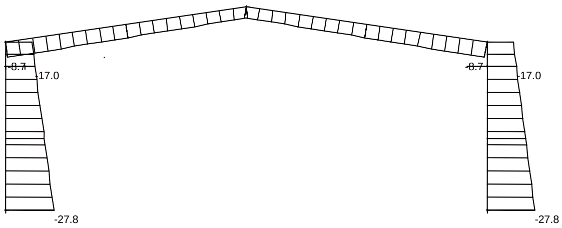
Figure 4. The pressure coefficients and wind loads w [kN/m] on the internal frame of the building.

4 CALCULATION OF INTERNAL FORCES

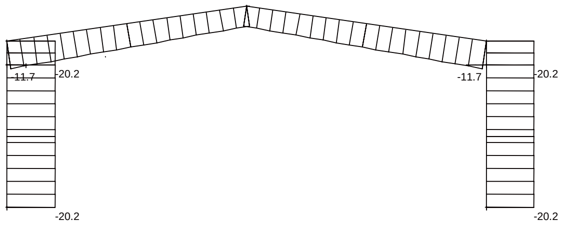
The internal forces – axial forces and bending moments - for the internal portal frame due to characteristic values of the actions for the above loading cases are calculated and presented in the following diagrams. The loads acting perpendicularly to the plane of the frame are not considered here.

4.1 Axial forces in kN

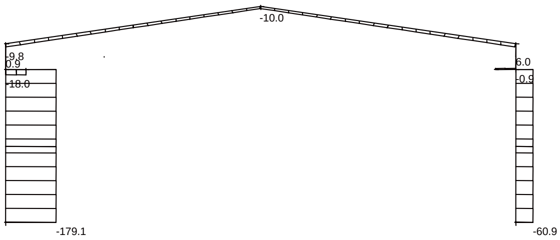
Self weight, roof and wall cladding



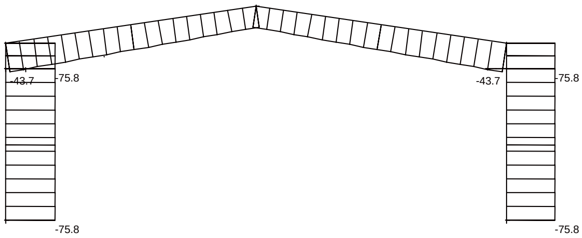
Imposed load: roof, wall



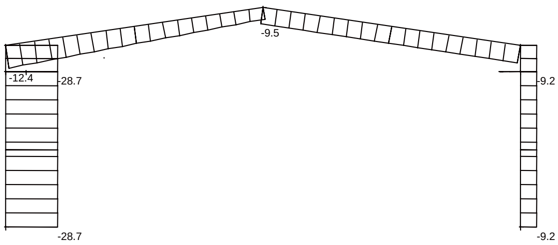
Crane: vertical load and horizontal side load



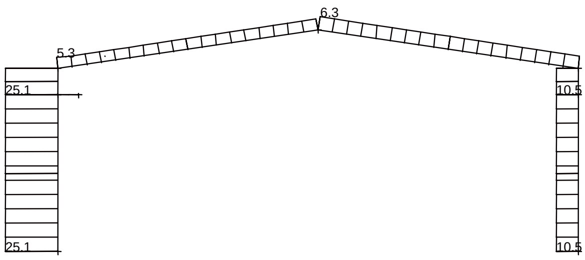
Snow load, case I



Snow load, case II

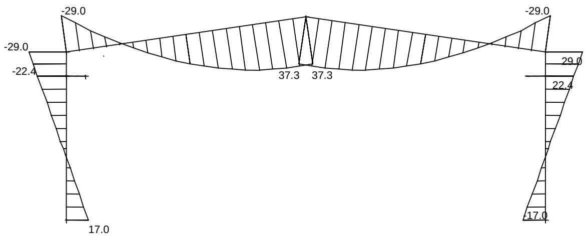


Wind load

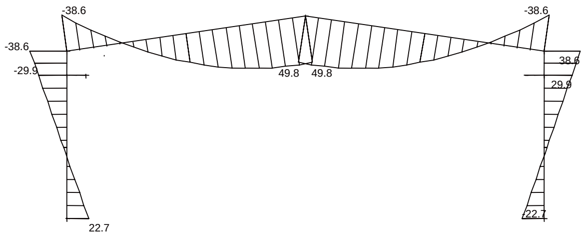


4.2 Bending moments in kNm

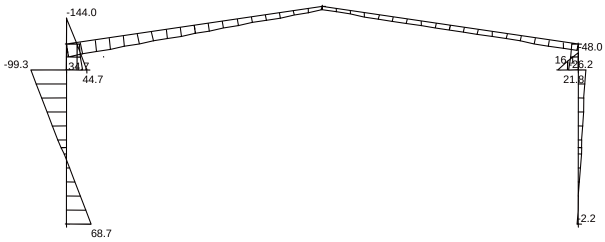
Self weight, roof and wall cladding



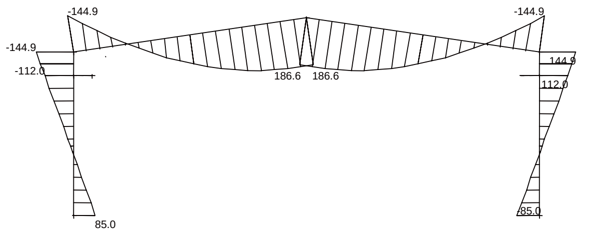
Imposed load: roof, wall



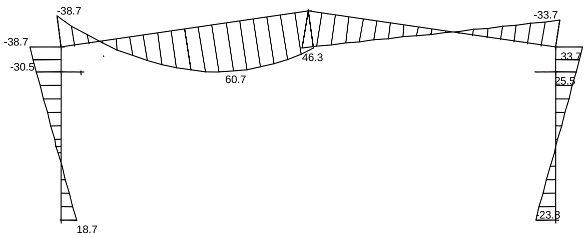
Crane: vertical load and horizontal side load



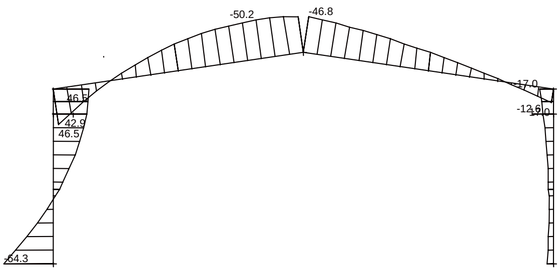
Snow load, case I



Snow load, case II



Wind load



4.3 Internal forces due to characteristic values of loads

The numerical values of the axial force *N* and the bending moment *M* for the column and the beam of the portal frame, shown in the diagrams above, are summarized in Table 4.

Table 4: Axial forces and bending moments due to characteristic values of loads.

Loading	COLUMN				BEAM			
	Start point		End point		Start point		End point	
	N [kN]	*M* [kNm]	*N* [kN]	*M* [kNm]	*N* [kN]	*M* [kNm]	*N* [kN]	*M* [kNm]
self weight	-27,8	17,0	-17,0	-29,0	-8,7	-29,0	-8,0	37,3
roof	-20,2	22,7	-20,2	-38,6	-20,2	-38,6	-8,0	49,8
crane	-179,1	68,7	-9,8	-99,3	-10,0	34,7	-10,0	10,0
snow I	-75,8	85,0	-75,8	-144,9	-43,7	-144,9	-30,0	186,6
snow II	-28,7	18,7	-28,7	-38,7	-12,4	-38,7	-9,5	46,3
wind	-10,5	-12,6	-10,5	-17,0	5,3	17,0	5,3	-46,8

5 COMBINATIONS OF ACTIONS

The design value of the effects of actions E_d is calculated from the combination of actions according to the equation (6.10) of the Eurocode EN 1990 [1]:

$$\sum_{j\geq 1} \gamma_{G,j}\, G_{k,j} + \gamma_{Q,1}\, Q_{k,1} + \sum_{i>1} \gamma_{Q,i}\, \psi_{0,i}\, Q_{k,i} \tag{14}$$

where $G_{k,j}$ are characteristic values of permanent loads with corresponding partial factors $\gamma_{G,j}$ (equal to 1,35 for unfavourable effect and 1,0 for favourable effect), $Q_{k,i}$ are characteristic values of variable loads with corresponding partial factors $\gamma_{Q,j}$ (1,5 unfavourable and 0 favourable), and $\psi_{0,i}$ are combination factors (0,6 for wind, 0,5 for snow and 0,7 for accidental actions, 0,7 for moving loads and 0 for imposed roof loading – see table A.1.1 in EN 1990 [1]). $Q_{k,1}$ is the leading variable action.

From the diagrams of internal forces we see that the load case snow II has lower effects than the load case snow I and thus will not be considered further. We will use the following symbols for the rest of the load actions:

G - self weight
R - imposed load on roofs
C - crane load
S - snow load, case I
W - wind load.

According to the equation (14) the following combinations of actions have to be considered:

Combination I, leading action R: $1{,}35 \cdot G + 1{,}5 \cdot (R + 0{,}7 \cdot C + 0{,}5 \cdot S + 0{,}6 \cdot W)$
Combination II, leading action C: $1{,}35 \cdot G + 1{,}5 \cdot (C + 0{,}0 \cdot R + 0{,}5 \cdot S + 0{,}6 \cdot W)$
Combination III, leading action S $1{,}35 \cdot G + 1{,}5 \cdot (S + 0{,}0 \cdot R + 0{,}7 \cdot C + 0{,}6 \cdot W)$
Combination IV, leading action W: $1{,}35 \cdot G + 1{,}5 \cdot (W + 0{,}0 \cdot R + 0{,}7 \cdot C + 0{,}5 \cdot S)$

The values for the effect of actions due to the combinations above are presented in Table 5.

Table 5: Axial forces and bending moments due to combinations of actions.

Combination	COLUMN				BEAM			
	Start point		End point		Start point		End point	
	N [kN]	M [kNm]	N [kN]	M [kNm]	N [kN]	M [kNm]	N [kN]	M [kNm]
combination I	-322,2	181,5	-129,8	-325,3	-80,55	-154,0	-51,03	233,4
combination II	-372,5	178,4	-104,0	-312,1	-54,75	-80,48	-43,53	163,2
combination III	-348,7	211,2	-156,4	-376,1	-83,03	-204,8	-61,53	298,6
combination IV	-298,2	139,9	-105,8	-277,6	-47,07	-85,89	-35,85	130,6

6 VERIFICATION

Verification will be carried out for the column and the beam of the internal portal frame.

6.1 Resistance of the elements

The cross section of the column is HEA 400 with the web height d = 298 mm, web thickness t_w = 11 mm, flange width 2c = 300 mm and flange thickness t_f = 19 mm. The cross section of the beam is IPE 550 with the web height d = 467 mm, web thickness t_w = 11,1 mm, flange width 2c = 210 mm and flange thickness t_f = 17,2 mm. We classify these sections according to EN 1993-1-1 [5] as follows. The material coefficient ε is

$$\varepsilon = \sqrt{\frac{235}{f_y}} = 1 \qquad (15)$$

The column has the ratios d/t_w = 27,1 and c/t_f = 7,9 and according to table 5.3.1 of EN 1993-1-1 [5] it is a class 1 section. The ratios for the beam are d/t_w = 42,5 and c/t_f = 6,1 which place it also in class 1 section. The plastic resistance axial force and moment are then calculated (with the material partial factor γ_{M1} = 1,1):

Column: $N_{pl,Rd} = A\,f_y\,/\,\gamma_{M1} = 15{,}9 \cdot 235/1{,}1 = 3397$ kN
$M_{pl,Rd} = W_{pl}\,f_y\,/\,\gamma_{M1} = 2{,}56 \cdot 235/1{,}1 = 547$ kNm

Beam: $N_{pl,Rd} = A\,f_y\,/\,\gamma_{M1} = 13{,}4 \cdot 235/1{,}1 = 2871$ kN
$M_{pl,Rd} = W_{pl}\,f_y\,/\,\gamma_{M1} = 2{,}78 \cdot 235/1{,}1 = 594$ kNm

The buckling parameters of the members are (section 5.5.1 of the EN 1993-1-1 [5]):

Column: $\lambda = L/i_y = 7/0{,}168 = 41{,}7$, $\bar{\lambda} = \lambda/[\pi\,(E/f_y)^{0,5}] = 0{,}44$, $\chi = 0{,}94$
Beam: $\lambda = L/i_y = 10{,}11/0{,}223 = 45{,}3$ $\bar{\lambda} = \lambda/[\pi\,(E/f_y)^{0,5}] = 0{,}48$, $\chi = 0{,}93$

where L is buckling length of the member, i_y is radius of gyration of the member's cross section, λ is slenderness of the member and χ is buckling reduction factor, calculated from:

$$\phi = 0{,}5\,[1 + \alpha\,(\bar{\lambda} - 0{,}2) + \bar{\lambda}^2\,]\,, \quad \chi = \frac{1}{\phi + [\phi^2 - \bar{\lambda}^2]^{0,5}}$$

with the imperfection factor α = 0,21 for buckling curve a (see table 5.3.3 of EN 1993-1-1).

6.2 Verification for the ultimate limit state

The verification of the ultimate limit state will be carried out according to equation 6.8 of the Eurocode EN 1990 [1]:

$$E_d \leq R_d \qquad (16)$$

where E_d is the design value of the effects of actions, such as internal force or a moment, and R_d is the design value of the corresponding resistance. In the case of the bending with compression of the class 1 section, the above equation can be formulated as follows (equation 5.51 of the Eurocode EN 1993-1-1 [5]):

$$\frac{N_{\text{Sd}}}{\chi \; N_{\text{pl,Rd}}} + \frac{k_{\text{y}} \cdot M_{\text{y,Sd}}}{M_{\text{pl,Rd}}} \leq 1 \qquad (17)$$

The factor k_y is calculated according to section 5.5.4 of EN 1993-1-1 [5] and it depends on the internal forces of the element:

$$k_{\text{y}} = 1 - \frac{\mu_{\text{y}} N_{\text{Sd}}}{\chi \; A \; f_{\text{y}}} \,, \qquad k_{\text{y}} \leq 1{,}5 \qquad (18)$$

$$\mu_{\text{y}} = \bar{\lambda}(2\beta_{\text{My}} - 4) + \frac{W_{\text{pl,y}} - W_{\text{el,y}}}{W_{\text{el,y}}} \,, \quad \mu_{\text{y}} \leq 0{,}9 \qquad (19)$$

$$\beta_{\text{My}} = 1{,}8 - 0{,}7\,\psi \,, \qquad \psi = |M|_{\min} / |M|_{\max} \qquad (20)$$

As an example of this verification we will carry out the verification of the column for the combination III in Table 5. We have the following values

$$N_{\text{Sd}} = 348{,}7 \text{ kN}, \;\; M_{\text{y,Sd}} = 376{,}1 \text{ kNm}$$
$$\psi = 211{,}2/\text{-}376{,}1 = \text{-}0{,}56$$
$$\beta_{\text{My}} = 1{,}8 - 0{,}7 \cdot (\text{-}0{,}56) = 2{,}19$$
$$\mu_{\text{y}} = 0{,}44\ (2 \cdot 2{,}19 - 4) + (2{,}56 - 2{,}31)/2{,}31 = 0{,}28$$
$$k_{\text{y}} = 1 - (0{,}28 \cdot 328{,}7)/(0{,}94 \cdot 15{,}9 \cdot 235) = 0{,}97$$

and the verification equation (17) yields:

$$\frac{348{,}7}{0{,}91 \cdot 3397} + \frac{0{,}97 \cdot 376{,}1}{594} = 0{,}11 + 0{,}61 = 0{,}72 \leq 1$$

REFERENCES

[1] EN 1990 Eurocode - Basis of structural design. CEN 2002.

[2] EN 1991-1-1 Eurocode 1: Actions on structures. Part 1-1: General actions - Densities, self-weight, imposed loads for buildings, CEN 2002.

[3] EN 1991-1-3 Eurocode 1: Action on structures – Part 1-1: General actions – Snow loads.

[4] ENV 1991-2-4 Eurocode 1: Basis of desing and action on structures – Part 2-4:Actions on structures – Wind actions.

[5] EN 1993-1-1 Eurocode 3: Design of steel structures – Part 1-1: General rules and rules for buildings.

[6] JCSS: Probabilistic model code. JCSS working materials, http://www.jcss.ethz.ch/, 2001.

CHAPTER VIII - EXAMPLE OF A COMPOSITE BUILDING

Oliver Kraus, Christiane Butz

Institute for Steel Construction, RWTH Aachen, Germany

1 INTRODUCTION

A composite frame structure is verified according to the Eurocodes. Load assumptions are applied corresponding to EN 1991-1-1[2], EN 1991-1-3 [2], EN 1991-1-4 [3] and EN 1991-1-7 [4]. This example does not represent a complete verification of the structure but has got the purpose to clarify which loads are relevant and how the internal forces are determined in case of a composite structure.

2 DEFINITION OF THE SYSTEM

2.1 Details of the system

The building presented here is a frame structure consisting of composite slabs and of composite columns. A storage hall is located on ground level whereas the upper level is used as an office area. For determination of the wind loads it is assumed that the building is located in an industrial area and that the basic velocity is $v_b = 25\ m/s$. For determination of the snow load it is assumed that the climatic region is “Central West” with a relevant snow load on ground level of 0,4 kN/m². The height of the site is 200 m.

It shall be verified whether the resistance of the composite slabs and of the composite columns fulfill the requirements of the Ultimate Limit State of the Eurocodes and whether the deflections of the floor slab are within the limits of the Serviceability Limit State.

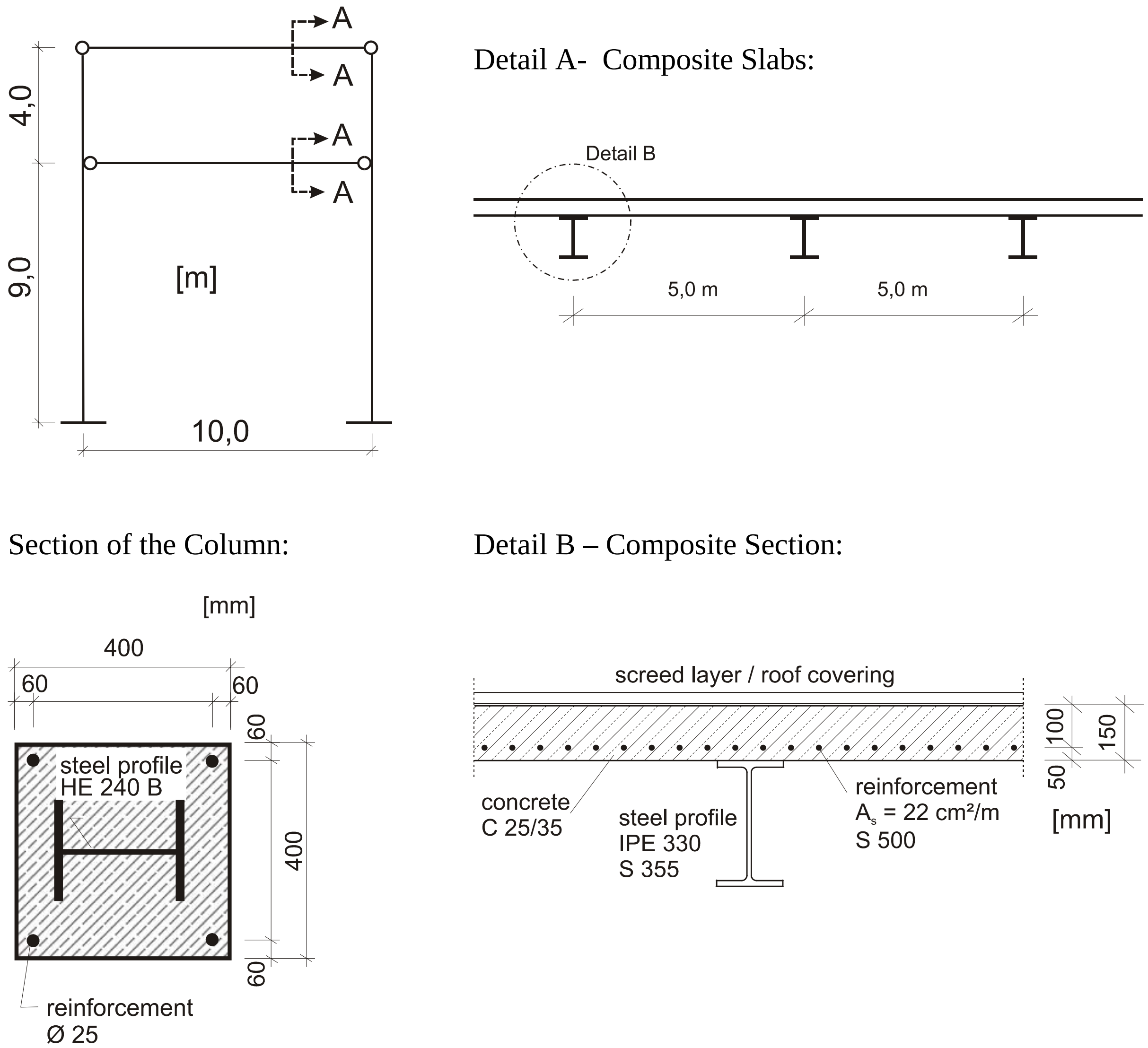


Figure 1. Composite building

2.2 Properties of the sections

2.2.1 Resistance of the slabs

An analysis of the section gives the result that the plastic neutral axis is in the concrete slab. The effective width of the concrete slab is 2,50 m. Then the design value of the plastic bending capacity of the composite sections is:

$$M_{\mathrm{pl,Rd}} = 560{,}0 \text{ kNm}$$

2.2.2 Resistance of the columns

The columns are affected by axial forces as well as by bending moments due to wind loading. Therefore the relevant *M-N*-interaction diagram has to be developed for the column. With the cross section given in Figure 1 the following resistances are obtained:

plastic axial force:	$N_{pl,Rd}$ = 6522,9 kN
reduced axial force:	$\chi\, N_{pl,Rd}$ = 4856,6 kN
plastic bending resistance:	$M_{pl,Rd}$ = 521,0 kNm
maximum bending resistance:	$M_{max,Rd}$ = 573,0 kNm (enhancement due to compression)

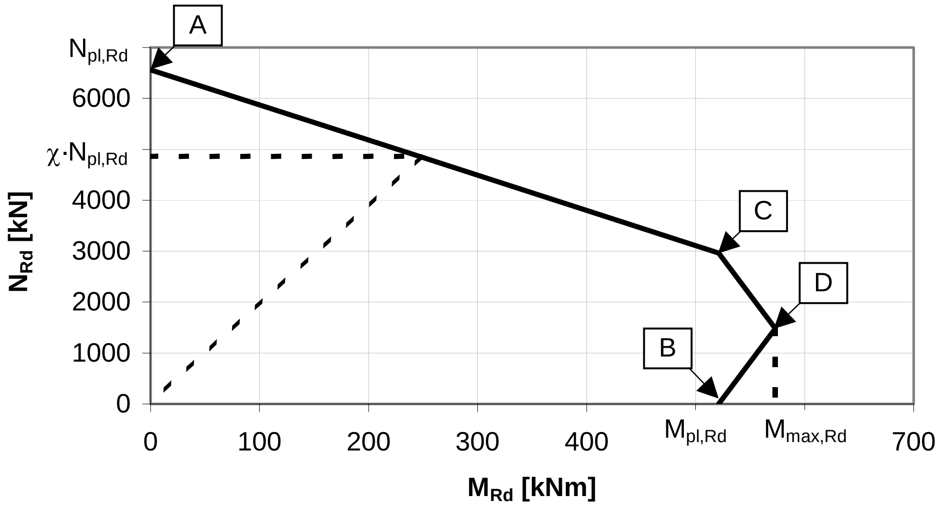


Figure 2. *M-N*-interaction diagram for the composite column

Assuming a perfectly plastic stress distribution the *M-N*-interaction diagram is sufficiently characterized by point A to D.

Point A — The axial force N_A is equal to the plastic axial resistance of the cross section $N_{pl,Rd}$, while the bending moment is zero.

Point B — The bending moment N_B is equal to the plastic bending resistance of the cross section $M_{pl,Rd}$, while the axial force N_B is zero.

Point C — The neutral axis of the stress is shifted so that the bending moment N_C is equal to the plastic bending resistance of the cross section $M_{pl,Rd}$ and in addition the axial force N_C is equal to $A_c\ \alpha_c\ f_{cd}$ and provided by the shift of the neutral axis.

Point D — The neutral axis of the stress distribution is equal to the centerline of the cross section. The bending moment M_D is equal to the maximum bending resistance $M_{max,Rd}$ and the axial force N_D is equal to ½ $A_c\ \alpha_c\ f_{cd}$ resulting from the concrete under compression

2.2.3 Moments of inertia

In order to determine the deflection in the frame of the verification for the Serviceability Limit State the moments of inertia have to be known. The differentiation between short duration and long duration actions is taken into account in case of the slabs whereas time dependent effects due to creeping and shrinkage do not have to be considered for the columns.

Table 1. Moments of inertia for the composite section

element	moment of inertia	
	$[cm^4]$	$[mm^4]$
slab (short duration action)	55 266	$5{,}5266 \cdot 10^8$
slab (long duration action)	43 072	$4{,}3072 \cdot 10^8$
slab (shrinkage)	47 909	$4{,}7909 \cdot 10^8$
column	34 595	$3{,}4595 \cdot 10^8$

3 DEFINITION OF LOADS

3.1 Permanent load

According to EN 1991-1-1 the following densities have to be applied for the relevant materials. Furthermore in the following summary the densities applied in the example presented here are mentioned:

Table 2. Densities given in EN 1991-1-1 and applied here

	EN 1991-1-1	chosen
normal concrete + reinforcement	25,0 kN/m³	25,0 kN/m³
cement / screed	19,0 kN/m³ - 23,0 kN/m³	19,0 kN/m³
structural steel	77,0 kN/m³ - 78,5 kN/m³	78,5 kN/m³

With these densities the self weight of the composite beam can be determined:

$$g_k = 0{,}15 \cdot 5{,}00 \cdot 25{,}0 + 84{,}6 \cdot 10^{-4} \cdot 78{,}5 = 19{,}4 \text{ kN/m}$$

It is assumed that the screed layer has got a thickness of 30 mm. Then the load for the screed on the floor of the office area can be transformed into an uniformly distributed load acting on the beam:

$$g_{screed} = 0{,}03 \cdot 19 \cdot 5{,}0 = 2{,}85 \text{ kN/m}$$

For the roof cover a permanent load of 0,45 kN/m^2 resulting into a beam load of

$$g_{roof} = 2{,}25 \text{ kN/m}^2$$

is applied.

3.2 Imposed load

According to EN 1991-1-1 a room which is used as an office area corresponds to the category B for which an area load in the range of between 2,0 kN/m² and 3,0 kN/m² is recommended. Here a load of 2,0 kN/m² is chosen resulting into

$$p = 2{,}0 \cdot 5{,}0 = 10{,}0 \text{ kN/m}$$

The recommended value for an imposed load on roofs is 0,40 kN/m² yielding

$$p_{\text{roof}} = 0{,}40 \cdot 5{,}0 = 2{,}0 \text{ kN/m}$$

3.3 Snow load

In the climatic region "Central West" the ground snow load of s_0 = 0,4 kN/m² corresponds to the snow load zone 3. The ground snow load is valid for sea level and has to be transformed into a snow load for the height of the relevant location. The equations for this height adjustment depent on the climatic region. For "Central West" it is:

$$s_{\mathrm{k}} = 0{,}164 \cdot Z - 0{,}082 + \frac{A}{966} = 0{,}164 \cdot 3 - 0{,}082 + \frac{200}{966} = 0{,}62\,\mathrm{kN/m^2}$$

The roof coefficient for flat roofs is 0,8 so that for the influence width of 5,0 m the following load distribution on the composite beam is obtained:

$$s = 0{,}62 \cdot 0{,}8 \cdot 5{,}0 = 2{,}48\ \mathrm{kN/m}$$

3.4 Wind load

3.4.1 Determination of the relevant gust wind pressure

The basic wind velocity is given as

$$v_{\mathrm{b}} = 25 \text{ m/s}$$

In Chapter 1 it is mentioned that the building is located in an industrial area which generally means isolated flat buildings. With these input parameters the exposure factor for transforming the mean pressure corresponding to v_b into a gust pressure in the relevant height above ground level can be determined:

$$c_{\mathrm{e}}\,(\mathrm{z}) = c_{\mathrm{r}}^2(z)\; c_{\mathrm{o}}^2(z)\; [1+7\, I_{\mathrm{v}}(z)]$$

where: $c_{\mathrm{r}}(z) = k_{\mathrm{r}} \cdot \ln\left(\dfrac{z}{z_0}\right)$ — logarithmic velocity profile

k_{r}	terrain factor
z_0	roughness length
$c_{\mathrm{o}}^2(z)$	orography factor (takes into account isolated changes in the terrain height) – here: $c_{\mathrm{o}}^2(z)$ = 1,0
$I_{\mathrm{v}}(z) = \dfrac{1}{c_0(z) \cdot \ln(z/z_0)}$	turbulence

Since surroundings with isolated flat buildings according to prEN 1991-1-4 mean terrain category III here k_{r} = 0,22 and z_0 = 0,30 m have to be introduced in the above mentioned equations. The reference height for a building which height h is greater than its width d can be divided into several parts. Here the first height is identical to d and the second part reaches to the top of the building:

$$z_1 = 10{,}0 \text{ m}$$

$$z_2 = 13{,}0 \text{ m}$$

Consequently two different exposure factors have to be applied:

$$c_e(10{,}0\ m) = \left[0{,}22 \cdot \ln\left(\frac{10{,}0}{0{,}3}\right)\right]^2 \cdot 1{,}0 \cdot \left[1 + 7 \cdot \frac{1}{1{,}0 \cdot \ln(10/0{,}3)}\right] = 1{,}78$$

$$c_e(13{,}0\ m) = \left[0{,}22 \cdot \ln\left(\frac{13{,}0}{0{,}3}\right)\right]^2 \cdot 1{,}0 \cdot \left[1 + 7 \cdot \frac{1}{1{,}0 \cdot \ln(13/0{,}3)}\right] = 1{,}96$$

The following gust wind pressure is obtained for both reference heights:

$$q_p(10{,}0m) = c_e(z_e) \cdot \frac{\rho}{2} \cdot v_b^2 = 1{,}78 \cdot \frac{1{,}25}{2} \cdot 25^2 = 695 \text{ N/m}^2 \mathrel{\widehat{=}} 0{,}695 \text{ kN/m}^2$$

$$q_p(13{,}0m) = c_e(z_e) \cdot \frac{\rho}{2} \cdot v_b^2 = 1{,}96 \cdot \frac{1{,}25}{2} \cdot 25^2 = 766 \text{ N/m}^2 \mathrel{\widehat{=}} 0{,}766 \text{ kN/m}^2$$

3.4.2 Distribution of wind loads

For a building with cuboid shape the pressure coefficients depend on the relation between height and width. Here for the value 13/10 = 1,3 the following distribution of pressure coefficients is given:

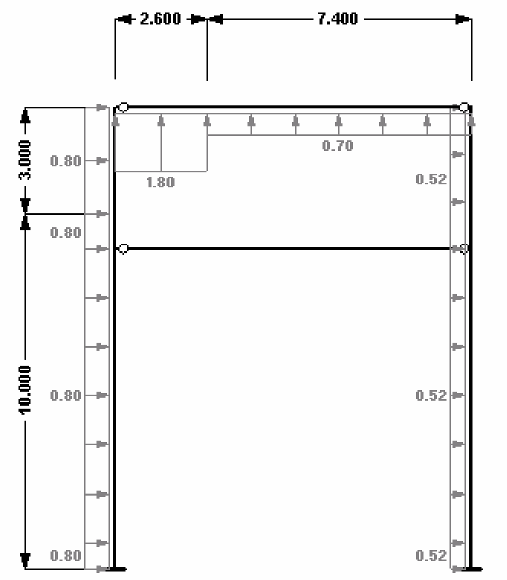


Figure 3. Distribution of pressure coefficients c_{pe}

With the influence width of 5,0 m and with the gust pressures given in 3.4.1 the following distribution of wind loading is determined:

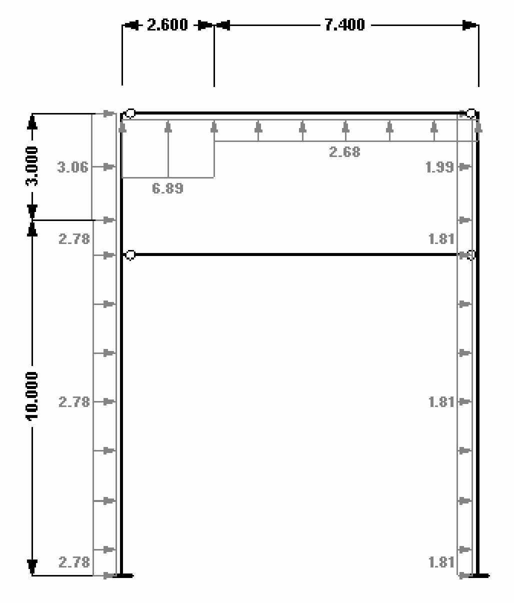


Figure 4. Wind loads [kN/m]

3.5 Impact of fork lift

Here it is assumed that a fork lift of class 4 is in use in the storage hall of the building. According to prEN 1991-1-1 the vertical force consisting of the sum of the neight weight and of the hoisting load is 100 kN.

In prEN 1991-1-7 as an accidential load case the impact of a fork lift should be taken into account. The load for this event is given as

$$F = 5 \cdot \underbrace{W}_{\substack{\textit{weight of} \\ \textit{fork lift}}} = 500 \text{ kN}$$

The height of this impact load is 0,75 m above ground level.

3.6 Effect of shrinkage

Since the slab is a simple beam shrinkage does not evoke restraint forces. Because shrinkage of concrete causes deflections the shrinkage effect has to be known for the verification of the Serviceability Limit State.

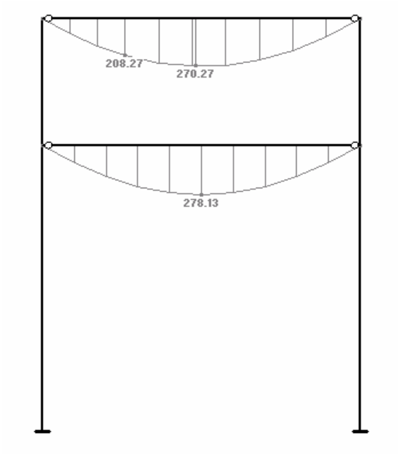
In case of the composite beam presented here the constant moment due to shrinkage is: 16 kNm

4 CALCULATION OF INTERNAL FORCES

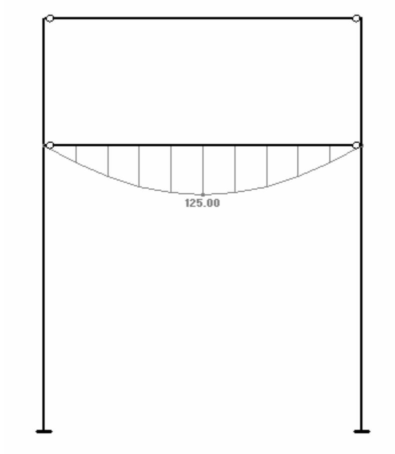
4.1 Bending moments

The following bending moments due to the characteristic values of the loads are obtained in the frame structure:

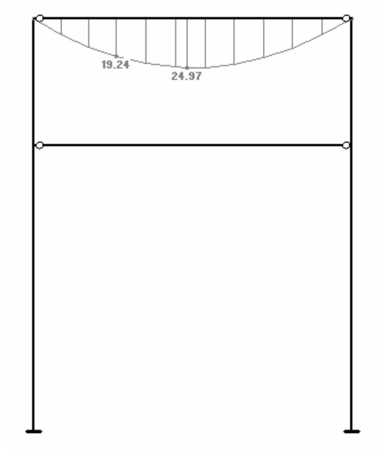
due to permanent loads:



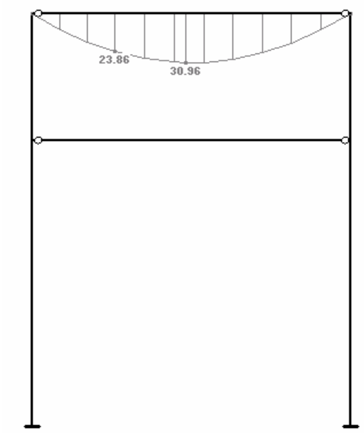
due to imposed load of the office area:



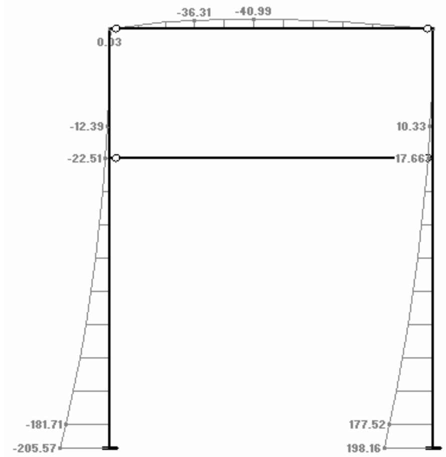
due to imposed load on the roof:



due to snow:



due to wind:



due to impact of fork lift:

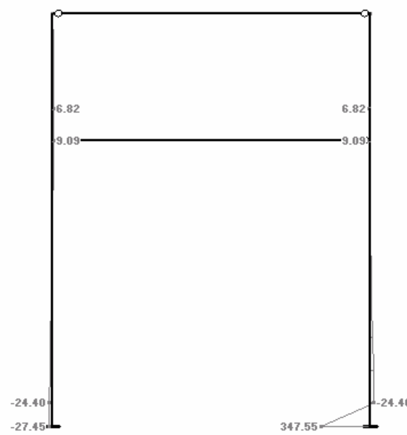
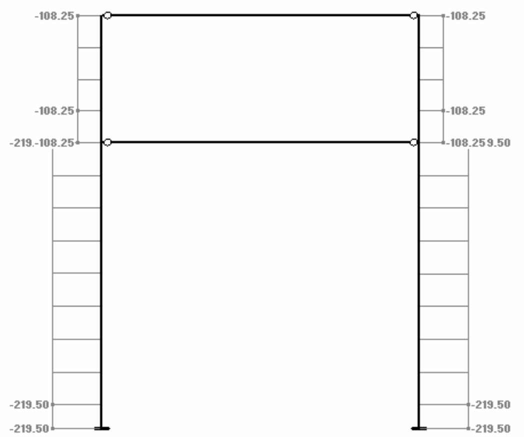


Figure 4. Bending moments [kNm] corresponding to characteristic load values

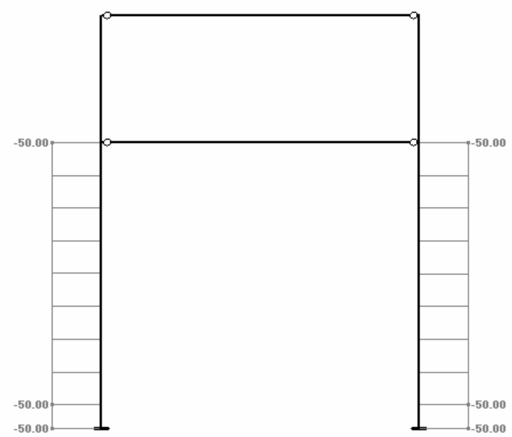
4.2 Axial forces

The following axial forces due to the characteristic values of the loads are obtained:

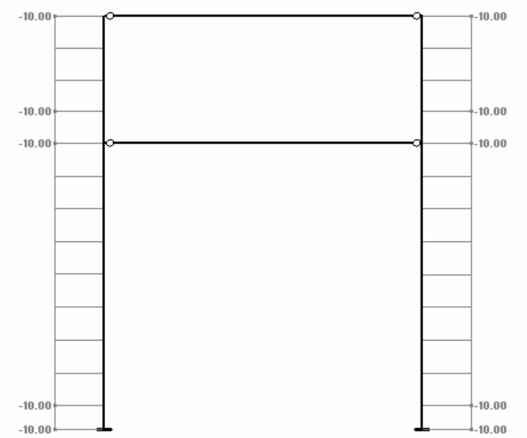
due to permanent load:



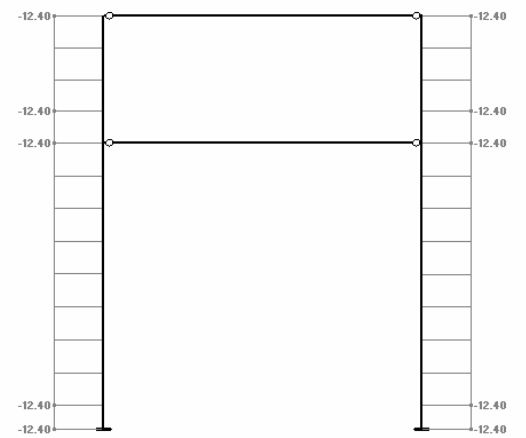
due to imposed load of office area:



due to imposed load on the roof:



due to snow load:



due to wind load:

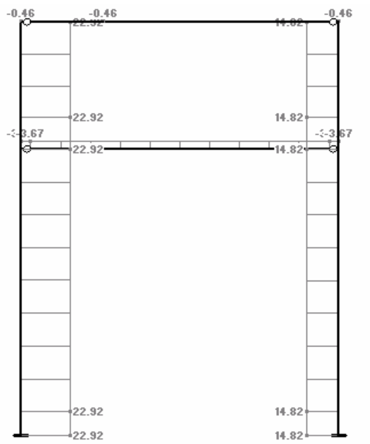


Figure 5. Axial forces [kN] corresponding to characteristic load values

5 VERIFICATIONS

5.1 Verification for the Ultimate Limit State

5.1.1 General

Here the verification for the Ultimate Limit State shall be carried out by using equation 6.10 of EN 1990:

$$\sum_{j\geq 1}\gamma_{G,j}G_{k,j}\text{"+"}\gamma_P P\text{"+"}\gamma_{Q,1}Q_{k,1}\text{"+"}\sum_{i>1}\gamma_{Q,i}\psi_{0,i}Q_{k,i}$$

where: G permanent loads

P prestressing (here not relevant)

Q_k characteristic value of variable action

γ_G partial factor for permanent load; 1,35 (unfavourable) or 1,0 (favourable)

γ_P partial factor for prestressing; here: not relevant

γ_Q partial factor for variable actions; 1,5 (unfavourable) or 0,0 (favourable)

ψ_0 combination factor: for imposed load: 0,7; for snow: 0,5; for wind: 0,6

The partial factors and combination factors can be applied on the action effects (e.g. bending moments).

5.1.2 Verification of the composite beam of the office area

It has to be checked whether the resistance of the composite beam is sufficient to carry the design values of the loads. Introducing the bending moments given in figure 4 together with the relevant partial factors gives the following design values of bending moments:

$$M_{\text{Ed}} = \underbrace{1{,}35\cdot 278}_{\text{permanent load}} + \underbrace{1{,}50\cdot 125}_{\text{imposed load}} = 563\ \text{kNm} \approx M_{\text{pl,Rd}} = 560\ \text{kNm}$$

5.1.3 Verification of the composite beam of the roof

The dominating variable action is snow load whereas the imposed load is accompanying and therefore is reduced using the combination factor ψ_0. Wind loads act favourable, hence they are not considered.

$$M_{\text{Ed}} = \underbrace{1{,}35\cdot 270}_{\text{permanent loads}} + \underbrace{1{,}50\cdot 31}_{\text{snow load}} + \underbrace{0{,}7\cdot 1{,}50\cdot 25}_{\text{imposed load}} = 437\ \text{kNm} < M_{\text{pl,Rd}} = 560\ \text{kNm}$$

5.1.4 Verification of the columns

In case the imposed load is introduced as dominating no bending moment has to be taken into account since wind evokes favourable axial forces. Then the design value of the resulting axial force is:

$$N_{\text{Ed}} = \underbrace{-1{,}35\cdot 219{,}5}_{\text{permanent loads}} \underbrace{-1{,}5\cdot 50}_{\text{imposed load office}} \underbrace{-0{,}5\cdot 1{,}50\cdot 12{,}4}_{\text{snow load}} \underbrace{-0{,}7\cdot 1{,}50\cdot 10{,}0}_{\text{imposed load roof}}$$

$$= -391{,}13\ \text{kN} < N_{\text{Rd}} = -4857\ \text{kN}$$

The M-N-interaction has to be carried out introducing the wind effects. The total normal compression force is less than 1500 kN which means that the bending moment capacity increases as a function of the compression force (see Figure 2). It means that all actions leading to compressive normal forces are favourable actions and hence should be neglected in the combination of actions.

For wind from the right hand side:

$$N_{Ed} = \underbrace{-1{,}00 \cdot 219{,}5}_{\text{permanent load}} + \underbrace{1{,}50 \cdot 14{,}82}_{\text{wind load from rhs}} = -197{,}27\text{kN}$$

$$M_{Ed} = \underbrace{1{,}50 \cdot 199}_{\text{wind load from rhs}} = 298{,}5 \text{ kNm}$$

For wind from the left hand side:

$$N_{Ed} = \underbrace{-1{,}00 \cdot 219{,}5}_{\text{permanent load}} + \underbrace{1{,}50 \cdot 22{,}92}_{\text{wind load from lhs}} = -185{,}12\text{kN}$$

$$M_{Ed} = \underbrace{1{,}50 \cdot -206}_{\text{wind load from lhs}} = -309\text{kNm}$$

The interaction diagram, see Figure 2, shows that the verification is fulfilled for both load combinations.

Furthermore the accidential load case of an impact of a fork lift has to be verified.

The combination rule is:

$$\sum_{j \geq 1} G_{k,j} \text{ "+" } P \text{ "+" } A_d \text{ "+" } (\underbrace{\psi_{1,1} \textit{ or } \psi_{2,1}}_{\substack{\textit{decision in dependence} \\ \textit{on design situation}}}) \, Q_{k,1} \text{ "+" } \sum_{i>1} \psi_{2,i} \, Q_{k,i}$$

where: ψ_1 combination factor for frequent design situations;
here: 0,5 for imposed loads; 0,2 for wind loads and snow loads

ψ_2 combination factor for quasi-permanent design situations
here: 0,3 for imposed loads; 0,0 for wind loads and snow loads

Then the following design values are determined using $\psi_{1,1}$ for the (accompanying) wind and snow actions:

$$N_{Ed} = \underbrace{-219{,}5}_{\substack{\text{permanent} \\ \text{load}}} \underbrace{-0{,}5 \cdot 10{,}0}_{\substack{\text{imposed} \\ \text{load}}} + \underbrace{0{,}2 \cdot 14{,}82}_{\text{wind load from rhs}} = -221{,}54 \text{ kN}$$

$$M_{Ed} = \underbrace{348}_{\substack{\text{accidental} \\ \text{action}}} + \underbrace{0{,}2 \cdot 199}_{\substack{\text{wind} \\ \text{load from rhs}}} = 388 \text{ kNm}$$

5.2 Verification of the floor-slab for the Serviceability Limit State

It shall be checked whether the maximum deflection *w*, i.e. the deflection in the middle of the span, corresponds to a value greater than $L/w = 250$.

The verification shall be carried out using equation 6.15b of EN 1990 for frequent design situations, which is recommendend in particular in case of reversible effects:

$$\sum_{j\geq 1} G_{k,j} \text{"+"} P \text{"+"} \psi_{1,1} Q_{k,1} \text{"+"} \sum_{i>1} \psi_{2,i} Q_{k,i}$$

where: ψ_1 combination factor for frequent design situations;
here: 0,5 for imposed loads
ψ_2 combination factor for quasi-permanent design situations
here: 0,3 for imposed loads

The deflection is calculated for the middle of the simple beam affected by uniformly distributed loads:

$$w_{\max} = \frac{1}{9{,}6} \cdot \frac{M_{\max} \cdot L^2}{EI}$$

The moment of inertia has to be introduced in dependence on the kind of action, i.e. whether it is a long duration or a short duration load.

- Deflection due to permanent loads:

$$w_{\max} = \frac{1}{9{,}6} \cdot \frac{278000 \cdot 10000^2}{210 \cdot 4{,}3072 \cdot 10^8} = 32 \text{ mm}$$

- Deflection due to imposed loads:

$$w_{\max} = \frac{1}{9{,}6} \cdot \frac{125000 \cdot 10000^2}{210 \cdot 5{,}5266 \cdot 10^8} = 11{,}2 \text{ mm}$$

- Deflection due to shrinkage:

In case of a simple beam with constant moment M the following deflection is obtained:

$$w_{\max} = \frac{M \cdot L^2}{8 \cdot EI} = \frac{16000 \cdot 10000^2}{8 \cdot 210 \cdot 4{,}7909 \cdot 10^8} = 2 \text{ mm}$$

With these input parameters the resulting deflection is:

$$w = 32 + 2 + 0{,}5 \cdot 11{,}2 = 39{,}6$$

$$\frac{L}{w} = \frac{10000}{39{,}6} = 253 > 250$$

REFERENCES

[1] EN 1990, Basis of structural design. European Committee for standardisation, 2002

[2] EN 1991-1-1, Actions on structures – General actions – Densitites, self-weight, imposed loads for buildings.

[3] EN 1991-1-3, Actions on structures – General actions – Snow loads.

[4] EN 1991-1-4, Actions on structures – General actions – Wind actions.

[5] EN 1991-1-7, Actions on structures – General actions – Accidential actions.

ANNEX - PROPERTIES OF SELECTED MATERIALS

Igor Kovše[1], Angel Arteaga[2]

[1]Metal Construction Institute, Ljubljana, Slovenia
[2]Eduardo Torroja Institute for Construction Research - CSIC, Madrid, Spain

Summary

Material properties are relevant to the structural design process because of their effect on the strength, serviceability and durability of structures. A number of models for materials and their respective properties are addressed in this chapter. The specific properties of structural steel and concrete and the methods for determining their characteristic and design values are discussed in greater detail.

1 INTRODUCTION

1.1 Background

Among the basic variables considered in the structural design process are material characteristics. A general description of the methods for determining property and design values is given in section 4.2 of European standard EN 1990 [1]. More specific information on steel and concrete is set out in Eurocodes 2 and 3 (1992 and 1999) [2, 3] and European technical specifications. Further details are available in JCSS working materials [4] and specialized literature on the mechanical properties of materials (e.g. [5]).

2 GENERAL MODELS AND PROPERTIES

2.1 Introduction

Structural design must take account of material properties, inasmuch as they impact the strength, serviceability and durability of structures. In keeping with the reliability requirements laid down in the Eurocodes, the focus in this paper is on the mechanical properties of materials. Other characteristics - acoustic, electrical and most thermal properties - are not addressed in the Eurocodes.

Most material modelling assumes that a given material is homogeneous, i.e. its properties are identical throughout. This is never actually true, since non-uniformities can be found in any material if observed at a small enough scale. Nevertheless, this assumption can be adopted as a reasonable approximation of real material behaviour at or above a certain minimum scale, sometimes referred to as the *characteristic volume of the material*.

In some materials the properties are direction-dependent, i.e. they are anisotropic. Wood stiffness varies, for instance, depending on whether it is loaded in a direction perpendicular to or parallel with its fibres. Some anisotropic materials can be taken to be isotropic when the anisotropy has a negligible effect on calculations. One example is reinforced concrete, but even steel, which is usually assumed to be typically isotropic, is anisotropic if rendered sufficiently plastic.

The mechanical properties of materials determine the relationships between the actions on a structure (usually expressed as mechanical or thermal stress) and its deformation (expressed as strain or displacement). In any discussion of material properties, a distinction needs to be drawn between the model and actual material properties (or parameters), strictly speaking. Once a model is chosen for a material, the respective parameters are ascertained by experiment, which means that the model corresponds to the experimental behaviour exhibited by the material.

For example, the model for a linear elastic material (Figure 1 (a)) defines a linear relationship between one-dimensional stress σ and strain ε, in which the sole material parameter is the elastic modulus or E.

2.2 One-dimensional material models

The simplest and most widely used models are described in terms of one-dimensional stresses and strains. They can be used for structures consisting of linear elements (trusses, beams, columns, …) and the only stresses are parallel to member axes. For other types of structures (shells, solid bodies) three-dimensional stresses and strains are used to model the material.

2.2.1 Elastic material model

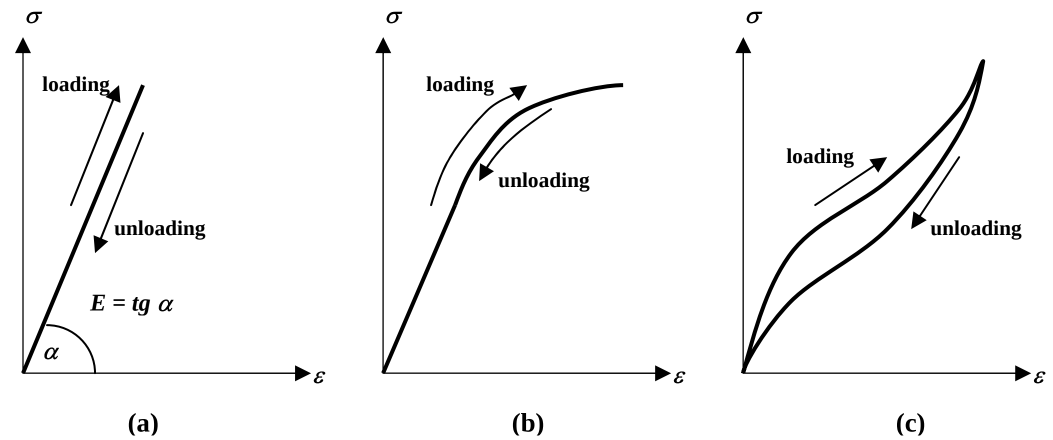


Figure 1. Elastic material models. (a) Linear elastic model; (b) Non-linear elastic model; (c) Elastic hysteresis.

Historically, the elastic model was the first model ever proposed for materials (Hooke, 1678) and it is still the one most widely used. The elastic material model defines a unique relationship between stress σ and strain ε, which is the same when the material is loaded as when it is unloaded.

$$\sigma = \sigma(\varepsilon) \tag{1}$$

Where this relationship is linear (Figure 1 (a)), it describes a *linear elastic material*. The only material parameter considered in this model is the *elastic modulus* or E (also known as Young's modulus), which is defined as the stress-strain ratio:

$$E = \frac{\sigma}{\varepsilon} \tag{2}$$

The elastic modulus is a measure of stiffness: the higher the value of modulus E, the greater the force needed to produce a given deformation. Its dimension is the same as for stress (force/length^2). Modulus E is usually determined from specimens loaded with normal, one-dimensional tension or compression stress. When shear stress, τ, rather then normal stress, σ, is applied to the test specimen, the resulting shear strain, γ, can be used to find the *shear modulus* or G

$$G = \frac{\tau}{\gamma} \tag{3}$$

When relationship (1) is non-linear, it describes a *non-linear elastic material* (Figure 1(b)). The elastic modulus, which is not constant for this material but rather a function of the stress (or strain), is called the *tangent elastic modulus*:

$$E_t = \frac{d\sigma}{d\varepsilon} \tag{4}$$

This kind of model is usually applied to materials such as rubber. In some rubbers, however, the stress-strain curve is not the same during loading as during unloading (Figure 1 (c)). Although this property is sometimes called *elastic hysteresis* and the material returns to its initial shape upon the removal of applied forces, the behaviour involved is not truly elastic, since not all the energy deriving from loading is recovered after unloading. These types of materials are used as vibration dampers.

One-dimensional elastic models may also describe the response of the material to thermal changes ΔT. When an unconstrained body is subjected to a change in temperature ΔT, it expands or contracts with the strain

$$\varepsilon_T = \alpha_T \, \Delta T \tag{5}$$

The material parameter α_T is called the *linear thermal expansion coefficient* and its dimension is (1/temperature). The linear thermal expansion model defined by equation (5), in which parameter α_T is a constant, is accurate enough for the changes in atmospheric temperature encountered in civil engineering. This is not the case for very high temperatures, however, such as in the event of a fire.

2.2.2 Plastic material model

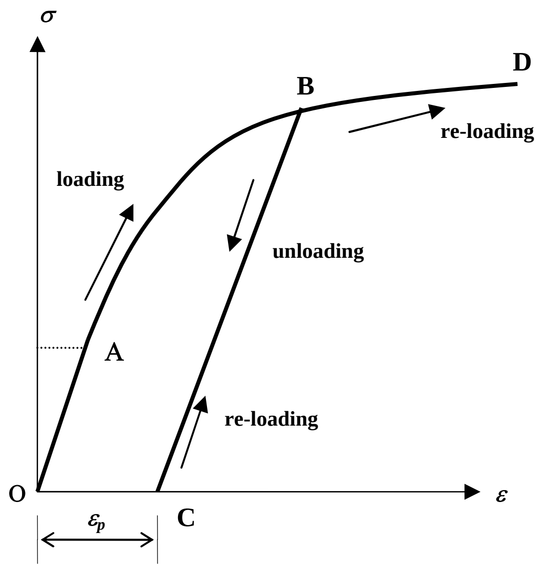


Figure 2. Elastoplastic material model.

In the plastic material model, when a material is loaded from an initial (unloaded) state to a certain level of stress and then unloaded to the initial level, it undergoes a permanent change in shape called *plastic deformation*. The behaviour of most materials is approximately linearly elastic (or simply elastic) up to a certain level of stress. This level is called the *proportional limit* (or *elastic limit*) (σ_y at point A in Figure 2). The material displays linear elastic behaviour in accordance with elastic modulus E within the range defined by this limit. When the limit is exceeded, the stress-strain diagram becomes non-linear and plastic deformation takes place. For the loading sequence only, the curve is the same as for a non-linear elastic material. The material unloads, however (from point B on the figure), along a path parallel to the elastic portion of the stress-strain curve (line B-C), rather than along the loading path. After the force applied is fully removed, the material retains permanent *plastic strain,* ε_p, otherwise known as *permanent set*. When the material is re-loaded, its stress-strain diagram is again described by the straight line C-B. This essentially means that the elastic limit of the material is increased, a phenomenon known as *strain hardening*. Under further loading after point B, the model conforms to the non-linear curve B-D.

When engineering stresses and strains,

$$\sigma = \frac{F}{A_0},\ \varepsilon = \frac{\Delta L}{L_0}, \tag{6}$$

the initial cross section A_0 and length L_0, are used to plot the experimental σ-ε curve (Figure 3); the maximum stress, $\sigma_{u,}$ found at point U is known as the *ultimate strength* (or *tensile* or *compressive strength*, depending on the type of test).

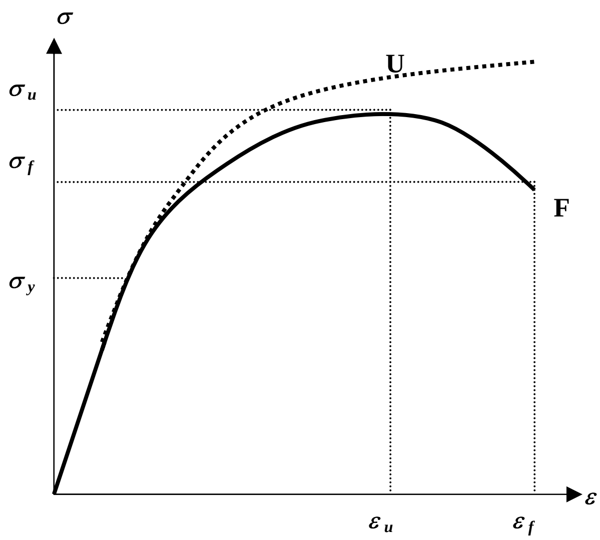


Figure 3. Engineering stress-strain (solid line) and true stress-strain (dashed line) diagrams.

The strain at this point is denoted as ε_u. Up to this point the strain in the specimen is uniform along its entire length. At point U localised deformation occurs in the form of necking, and from there on the engineering stress decreases up to material failure at point F. When stresses and strains are based on instantaneous cross section A and length L,

$$\sigma = \frac{F}{A},\ \varepsilon = \frac{\Delta L}{L}, \tag{7}$$

the stress in the diagram continues to increase until the material fails (dashed line in Figure 3).

2.2.3 Viscoelastic material model

It is assumed, in the foregoing discussion, that stress and strain are independent of time, i.e., the conditions affecting the material are static. However, certain materials exhibit time-dependent deformation when exposed to constant stress. This type of material can be modelled with the viscoelastic material model. To visualize this model, imagine an elastic spring connected in parallel to a viscous dashpot. The elastic spring behaves like a linear elastic material:

$$\sigma_e = E\ \varepsilon_e \tag{8}$$

while the behaviour of the linear viscous dashpot can be represented as a linear relationship between the *rate* of the strain $d\varepsilon/dt$ and the stress σ

$$\frac{d\varepsilon_V}{dt} = \frac{\sigma_V}{\eta} \tag{9}$$

where η is the viscosity coefficient. In a parallel connection between the elastic and viscous components, each is subjected to identical strain and the sum of the stresses in the components is equal to the stress applied or σ:

$$\eta \frac{d\varepsilon_V}{dt} + E\ \varepsilon_e = \sigma \tag{10}$$

Where the stress, σ applied is constant and the initial strain at time t_0=0 is zero ($\varepsilon(t_0)$=0), integrating the above equation yields time-dependent strain

$$\varepsilon(t,t_0) = \frac{\sigma_0}{E}\ [1\text{-}\exp(-(E/\eta)t)] \tag{11}$$

This means that the total strain in the viscoelastic model asymptotically approaches elastic strain. The notation $\varepsilon(t,t_0)$ represents the strain at time t caused by the stress applied at time t_0. The Boltzmannn superposition principle is applicable to linear viscoelastic materials: the total strain $\varepsilon(t)$ at time t resulting from the stresses σ_i applied at times t_i is the sum of the strains $\varepsilon_i(t)$ caused by individual stresses σ_i:

$$\varepsilon(t) = \Sigma_i\ [\varepsilon_i(t-t_i)] \tag{12}$$

Introducing the *creep function* $\Phi(t,t_0)$ with the expression

$$\varepsilon(t,t_0) = \sigma(t_0)\ \Phi(t,t_0) \tag{13}$$

equation (12) can be re-written as follows:

$$\varepsilon(t) = \Sigma_i\ \Phi(t,t_i)\ \Delta\sigma_i \tag{14}$$

2.3 Three-dimensional material models

Material models that take account of all the stress and strain tensors are used when dealing with structures such as shells and solid bodies, for which the one-dimensional material model is inadequate.

2.3.1 Three-dimensional elastic model

The extension of Hooke's linear elastic material model to three-dimensional stress and strain fields is represented by the equations:

$$\begin{aligned}
\varepsilon_{xx} &= \frac{1+\nu}{E}\sigma_{xx} - \frac{\nu}{E} I_1 + \varepsilon_T \\
\varepsilon_{yy} &= \frac{1+\nu}{E}\sigma_{yy} - \frac{\nu}{E} I_1 + \varepsilon_T \\
\varepsilon_{zz} &= \frac{1+\nu}{E}\sigma_{zz} - \frac{\nu}{E} I_1 + \varepsilon_T \\
\varepsilon_{xy} &= \frac{1+\nu}{E}\sigma_{xy} \\
\varepsilon_{yz} &= \frac{1+\nu}{E}\sigma_{yz} \\
\varepsilon_{xz} &= \frac{1+\nu}{E}\sigma_{xz}
\end{aligned} \tag{15}$$

where I_1 represents the first invariant of the stress tensor:

$$I_1 = \sigma_{xx} + \sigma_{yy} + \sigma_{zz} \tag{16}$$

The Poisson ratio ν is obtained from one-dimensional tension or compression testing and is defined as the ratio

$$\nu = -\frac{\varepsilon_{yy}}{\varepsilon_{xx}} \tag{17}$$

between the normal strain ε_{xx} in the direction of loading and the strain ε_{yy} in the direction perpendicular to the force applied. Since $2\varepsilon_{xx} = \gamma$, a comparison of the last three equations in expression (15) to equation (3) shows that the shear modulus G can be expressed in terms of the elastic modulus E and the Poisson ratio ν

$$G = \frac{E}{2(1+\nu)} \tag{18}$$

3 PROPERTIES OF STRUCTURAL STEEL

3.1 Introduction

Steel is an iron-based alloy in which carbon is the essential alloying constituent. The carbon content by volume in structural steel is typically 0.1% - 0.2%. Higher carbon contents increase yield stress and tensile strength and decrease ductility and weldability. Steel conforms to the elastic-plastic material model.

3.2 Steel properties deduced from the stress-strain diagram

The engineering stress-strain diagram for structural steel deriving from the one-dimensional tensile test is given in Figure 4. The basic properties if this material are:

f_y *yield strength*. This is the elastic limit. If, after peaking at the end of the linear portion (f_H), the curve is observed to drop (f_L), a lower value (f_L) is used for f_y. If the curve transitions smoothly from the linear to the non-linear region with no distinct discontinuity between the two (which is typically the case in high-strength steels and aluminium), the '0.2% proof stress' or $f_{0,2}$, is defined to be the point where a line drawn parallel to the linear part of the curve from point ($\sigma = 0$, $\varepsilon = 0{,}002$) intersects the stress-strain curve.

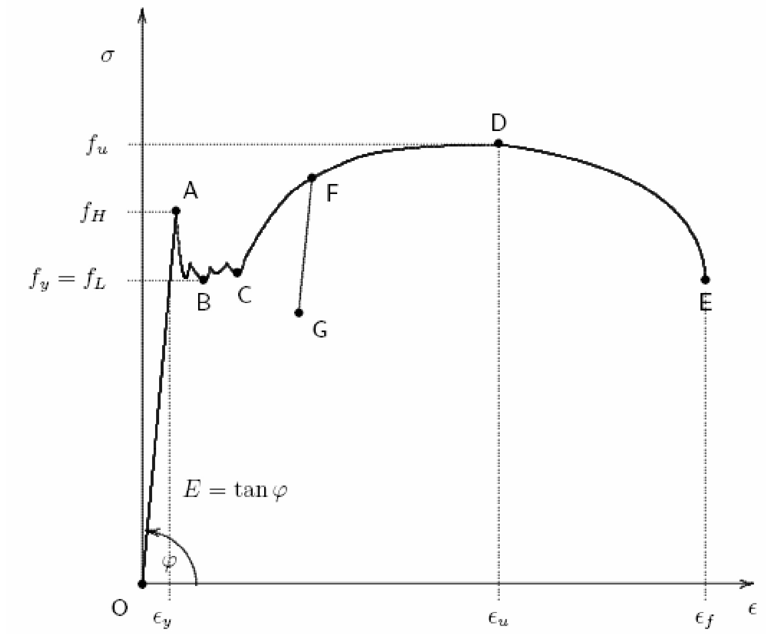


Figure 4. Engineering stress-strain diagram for a structural steel specimen generated by tensile testing.

f_u *ultimate (tensile) strength*. This is the maximum stress on the engineering stress-strain curve, where strain localisation ('necking') begins.

E *modulus of elasticity*. This is stress/strain ratio in the linear part of the curve. In ENV 1993-1-1 its value is fixed at 210000 N/mm^2.

ε_y *yield strain*. This is the strain corresponding to yield strength f_y.

ε_u *ultimate strain*. This is the strain corresponding to the ultimate tensile strength f_u. In ENV 1993-1- (section 3.2.2.2) it is used to define one of the requirements to be met for plastic analysis to be applicable.

ε_f *strain at fracture*. This is strain at which the material fails.

In Eurocode ENV 1993-1-1 (Design of steel structures) the characteristic values of yield strength f_y and ultimate tensile strength f_u to be used in design calculations are taken as nominal values. The nominal values for various grades of steel laid down in standards EN 10025 and EN 10113 are given in Table 1.

Table 1. Nominal values of yield strength f_y and ultimate tensile strength f_u for structural steel.

Steel grade	Thickness t [mm]			
	t <= 40 mm		40 mm <= t <= 100 mm	
	f_y [N/mm^2]	f_u [N/mm^2]	f_y [N/mm^2]	f_u [N/mm^2]
EN10025				
Fe 360	235	360	215	340
Fe 430	275	430	255	410
Fe 510	355	510	335	490
EN 10113				
Fe E 275	275	390	255	370
Fe E 355	355	490	335	470

3.3 Fatigue

Material fatigue can severely affect the safety of a structure. It is a particularly important process in steel structures because, unlike concrete, steel can withstand high tensile stresses, which are the chief cause of cracks and fatigue. Fatigue limit states are dealt with separately in ENV 1990 and in the Eurocodes on specific materials. Since fatigue is caused by repeated or alternating loads that are not common in buildings, it is mentioned only briefly in this section of Handbook 3, but will be addressed in detail in Handbook 4 on bridges.

3.4 Other properties of structural steel

Other properties of steel that are used to calculate ultimate and serviceability limit states are:

ν_y *Poisson ratio*. Defined in section 2.3.1 (see equation (17)).

G *shear modulus*. This is the shear loading equivalent of the elastic modulus in normal loading experiments. It can be calculated from elastic modulus E and the Poisson ratio ν (see equation (18)).

α_T *coefficient of linear thermal expansion*. Defined in section 2.2.1 (see equation (5)).

ρ *unit mass*. This is the mass per unit volume. It is normally used to calculate the self-weight of an element.

The values fixed for the above properties of steel in Eurocode ENV 1993-1-1 are shown in Table 2 below.

Table 2. Values of steel parameters to be used in design calculations.

Property	Symbol	Value
modulus of elasticity	E	210 000 N/mm^2
Poisson ratio	ν	0,3
shear modulus	G	$E/2(1+\nu)$
coefficient of linear thermal expansion	α_T	12 x 10^{-6} 1/^{o}C
unit mass	ρ	7850 kg/m^3

3.5 Characteristic and design values for steel properties

The characteristic values X_k for steel parameters are generally taken to be nominal values. Therefore, the values for f_y and f_u are both nominal and characteristic values, which is why the subscript k is omitted in Table 1. In some cases the characteristic value of a given property can be taken to be a fractile in its assumed statistical distribution. As a general rule, the lower fractile (5% fractile) is the one used. Values may be expressed in terms of the higher fractile (e.g., 95% fractile), however, when over-strength has adverse effects on structural safety.

The design value X_d of a property used in calculations, is defined as

$$X_d = \frac{X_k}{\gamma_M} \tag{18}$$

where γ_M is the partial safety factor for the property.

4 PROPERTIES OF CONCRETE

4.1 Introduction

Concrete is a non-homogenous material composed of stone aggregate, cement and water. Plane concrete can withstand considerable compressive stresses, but only small tensile stresses. When used for construction works, it is reinforced with steel reinforcement or cables which take up the tensile stresses. Many mechanical material properties of concrete are time dependent. Besides that concrete has a number of material effects that are not commonly addressed in other materials (e.g. creep, shrinkage, development of cracks).

4.2 Concrete properties deduced from the stress-strain diagram

The engineering stress-strain diagram for a concrete cylindrical specimen generated by the uniaxial compression test (upper part of the diagram) and the tension test (lower part of the diagram) is shown schematically in Figure 5. Since the mechanical properties of concrete vary with time (concrete hardens), standardised properties are obtained from the tests conducted on 28-day concrete specimens. Intrinsically, concrete tensile strength is lower than its compressive strength. The following properties of concrete are shown in the diagram:

f_c *compressive strength*. This is the minimum stress attained during the compressive testing of a concrete specimen (compressive stresses are considered negative here). It is also known as *peak stress*.

ε_{c1} *compressive strain at peak stress, f_c.*

ε_{cu} *ultimate compressive strain*. This is the strain at specimen fracture.

σ_{cu} *ultimate compressive strength*. This is the stress value at the ultimate compressive strain.

E_c *tangent modulus of elasticity*. This is the value of the modulus of elasticity (tangent to the stress-strain curve) at stress $\sigma_c = 0$.

E_{cm} *secant modulus of elasticity*. This value is obtained as the slope of the secant between two points on the stress strain curve, (ε_1, σ_1) and (ε_2, σ_2), where ε_1 = 0,00005 and $\sigma_2 = 0{,}4\, f_c$.

f_{ct} *tensile strength*. This is the maximum stress attained during tensile testing of a concrete specimen.

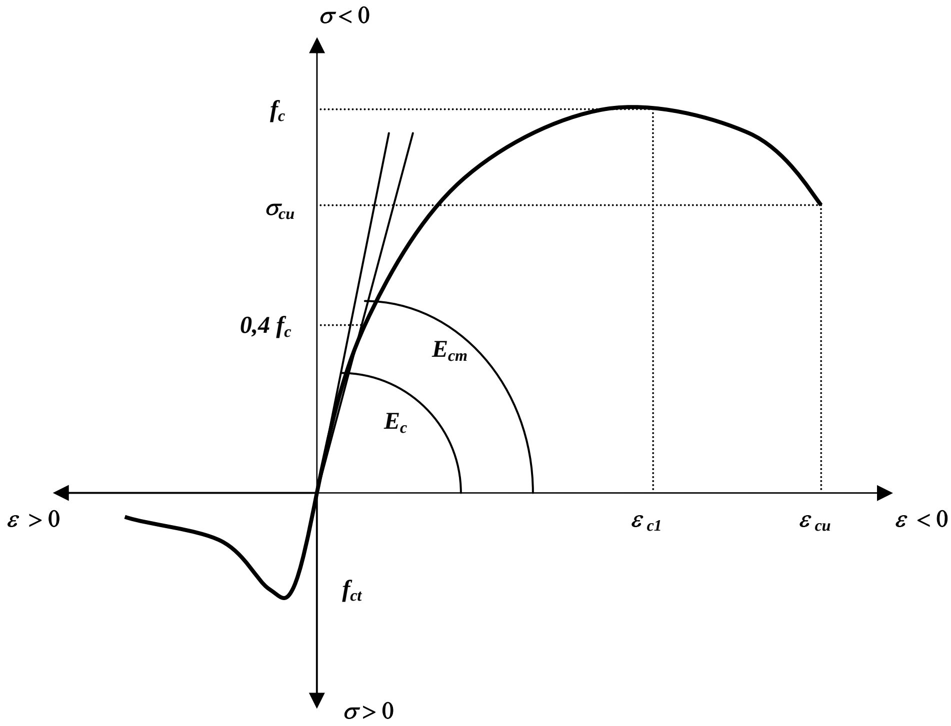


Figure 5. Engineering stress-strain diagram for a concrete cylinder generated by compressive (above ε axis) and tensile (below ε axis) testing.

The above are nominal values of the properties obtained from testing a single concrete specimen. The characteristic values of the mechanical properties of concrete are taken to be a fractile of the assumed statistical distribution, if known, of this property. Generally speaking, the 5% fractile is used for concrete compressive strength, f_{ck}, and tensile strength, f_{ctk}, both. However, the 95% fractile of these distributions may be used for this purpose if higher compressive ($f_{ck\ 0,95}$) or tensile strength ($f_{ctk\ 0,95}$) values jeopardize the safety of the structure (e.g. in a capacity design or when calculating the effect of indirect actions). In the case of the modulus of elasticity the mean value, E_{cm}, is usually the one of greatest interest. Mean values are likewise normally considered in connection with other properties such as creep or shrinkage.

As in other materials, the values for concrete are obtained from standardised tests performed under the conditions specified in EN 206.1 In particular, since properties are heavily dependent on specimen age, unless otherwise indicated, all tests are conducted on 28-day concrete.

In European standard EN 1992-1-1 (Design of concrete structures: General rules and rules for buildings) [2]) the concrete strength classes are based on characteristic compressive strength, f_{ck}. The recommended maximum value of this strength is 90 kN/m^2.

If no accurate testing data are available, the characteristic values of these properties can be obtained from Table 3 below (extracted from [2]) for several classes of concrete (defined as characteristic compressive strength for cylindrical/cubic specimens).

Table 3. Characteristic compressive strength, f_{ck} , mean compressive strength, f_{cm}, characteristic tensile strength, f_{ctk}, (in MPa) and secant modulus of elasticity, E_{cm}, (in MN/m^2) of concrete, by strength class.

Strength class of concrete	C20/25	C30/37	C40/50	C50/60	C60/75	C70/85	C80/95	C90/105
f_{ck}	20	30	40	50	60	70	80	90
f_{cm}	28	38	48	58	68	78	95	98
$f_{ctk0.05}$	1,5	2,0	2,5	2,9	3,1	3,2	3,4	3,5
$f_{ctk0.95}$	2,9	3,8	4,6	5,3	5,7	6,0	6,3	6,6
E_{cm}	30	33	35	37	39	41	42	44

The design values of the compressive and tensile strength of concrete are calculated from the following expressions

$$f_{cd}=\alpha_{cc} f_{ck} / \gamma_c , \qquad \text{and} \qquad f_{ctd}=\alpha_{ct} f_{ctk0,05} / \gamma_c$$

where:

γ_c is the partial safety factor for concrete. The recommended value is 1,5 for persistent and transient situations and 1,2 for accidental situations.

α_{cc}, α_{ct} are coefficients taking account of long-term effects and the way the load is applied. They should lie between 0,80 and 1,00.

4.3 Time-dependence of concrete mechanical parameters

Most concrete properties are heavily dependent on time. Concrete strength increases with time, more or less quickly depending of the kind of cement and curing conditions. Although values continue to rise for much longer, after the age of 28 days the rate of increase slows considerably. The same pattern is observed for the elastic modulus. The effects of other properties, such as creep and shrinkage, also lengthen the time needed to reach what might be regarded to be a steady state.

4.3.1 Compressive strength

When concrete compressive strength is needed for ages other than 28 days (e.g.: demoulding, prestress) and no tests are to be conducted, the characteristic value at time t, $f_{ck}(t)$, may be approximated from the expression:

$$f_{ck}(t) = f_{cm}(t) - 8 \text{ [kN/m}^2\text{]}, \qquad \text{for } 3 < t < 28 \text{ days,}$$

$$f_{ck}(t) = f_{ck}, \qquad \text{for } t \geq 28 \text{ days.}$$

Compressive strength depends on the kind of cement and the curing temperature and conditions. When the concrete is cured under normal conditions, the mean value may be estimated as

$$f_{cm}(t) = \beta_{cc}(t)\, f_{cm}$$

Taking:

$$\beta_{cc}(t) = \exp\{s[1\text{-}(28/t)^{1/2}]\};$$

where:

$f_{cm}(t)$	is the mean concrete compressive strength at age t (in days),
f_{cm}	is the mean concrete compressive strength at 28 days,
$\beta_{cc}(t)$	is a coefficient depending of the age and type of cement,
t	is the age in days,
s	is a coefficient depending on the cement class s = 0,2 for cement classes: CEM 42.5 R, CEM 53.5N and CEM 53.5R (Class R) s = 0,35 for classes CEM 32.5 R. CEM 42.5 N (Class N) s = 0,38 for classes CEM 32.5 N (Class S)

These expressions are valid for $t > 3$ days. The strength of concrete less than 3 days after casting must be determined by testing. For concretes over 28 days the characteristic value is assumed to be equal to f_{ck}.

The table below shows the values of the characteristic compressive strength for different concrete and cement classes at 3, 7 and 14 days as computed from the foregoing expressions.

Table 4. Characteristic concrete compressive strength at different ages (in kN/m^2).

	f_{ck} [kN/m^2]	**Age** [days]		
		3	**7**	**14**
Class R. $s = 0{,}20$	30	17,2	23,1	27,0
	50	30,5	39,5	45,4
	70	43,7	55,9	63,8
Class N. $s = 0{,}35$	30	10,5	18,8	24,9
	50	20,3	32,9	42,2
	70	30,0	47,0	59,5
Class S. $s = 0{,}38$	30	9,4	18,0	24,5
	50	18,6	31,7	41,6
	70	27,7	45,3	58,6

The characteristic concrete compressive strength for two concrete and two cement classes is plotted in Figure 6.

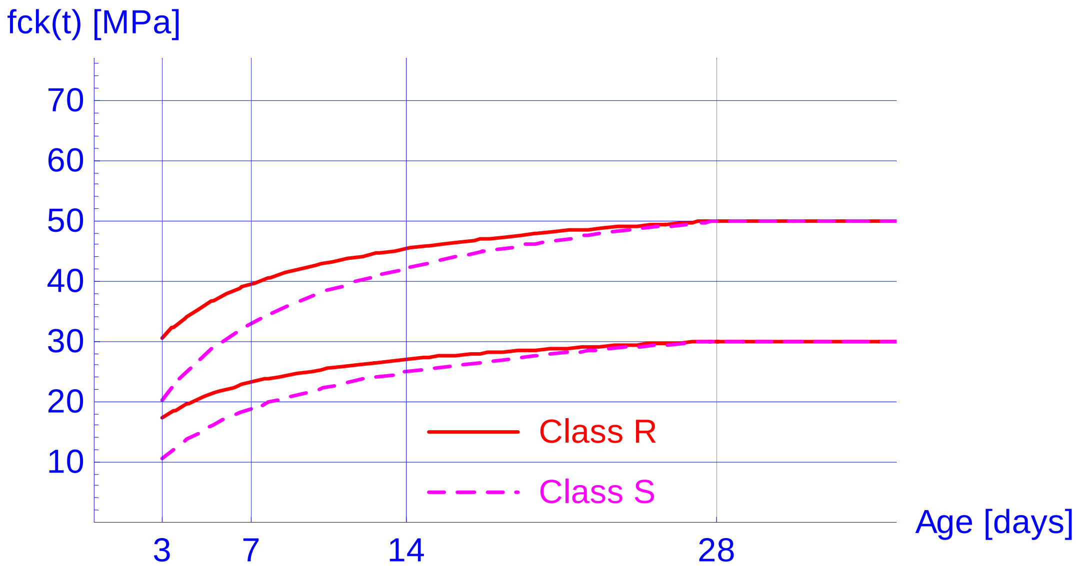


Figure 6. Time variation of characteristic concrete strength

4.3.2 Elastic modulus

The value of the mean elastic modulus of concrete at 28 days, E_{cm}, can be estimated from mean compressive strength with the following expression:

$$E_{cm}\ [\mathrm{MN/m^2}] = 22\ [(f_{cm}\ /10)]^{0,3}\ ; \qquad f_{cm} \text{ in } [\mathrm{kN/m^2}].$$

The following equation can be used to compute the variation of the elastic modulus with time:

$$E_{cm}(t) = (f_{cm}(t)/f_{cm})^{0,3}\ E_{cm}\ .$$

The table below shows the values of the mean elastic modulus of concrete for different concrete and cement classes at 3, 7, 14 and 28 days as computed from the foregoing expressions:

Table 5 Mean elastic modulus of concrete at different ages (in MN/m^2).

	f_{ck} [MPa]	**Age** [days]			
		3	**7**	**14**	**28**
Class R.	30	29,0	30,9	32,0	32,8
s = 0,2	50	33,0	35,1	36,4	37,3
	70	36,0	38,4	39,7	40,7
Class N.	30	26,5	29,6	31,4	32,8
s = 0,35	50	30,0	33,6	35,7	373
	70	32,8	36,7	39,0	40,7
Class S.	30	26,0	29,3	31,3	32,8
s = 0,38	50	29,5	33,3	35,6	37,3
	70	32,2	36,4	38,9	40,7

These values are also plotted in Figure 7 for two concrete and two cement classes.

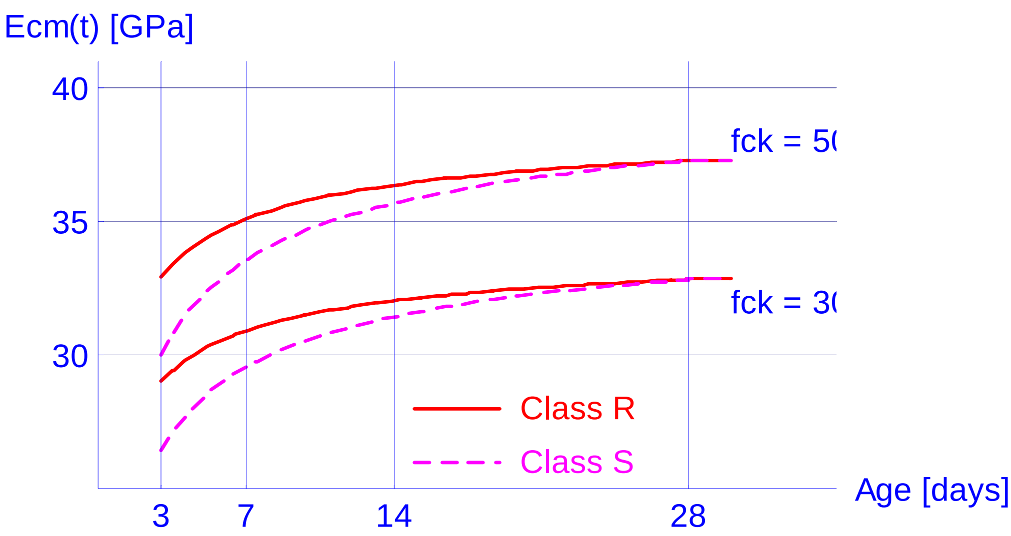


Figure 7. Time variation of mean elastic modulus of concrete

4.4 Creep

Due to the chemical and mechanical processes taking place in concrete, concrete members creep, i.e. the strain on a member increases over time at constant loading. Many factors affect creep rate and magnitude, including relative humidity of the surrounding air, member dimensions and concrete composition. Where creep is significant in design, the viscoelastic linear model (section 2.2.3) should be used.

Total creep strain $\varepsilon_{cc}(t, t_0)$ at any given time t may be calculated from equations (14), after estimating the creep function (or coefficient) $\varphi(t, t_0)$, in other words, by multiplying instantaneous deformation by the function $\varphi(t, t_0)$.

$$\varepsilon_{cc}(t, t_0) = \varphi(t, t_0)\,(\sigma_c/E_c)$$

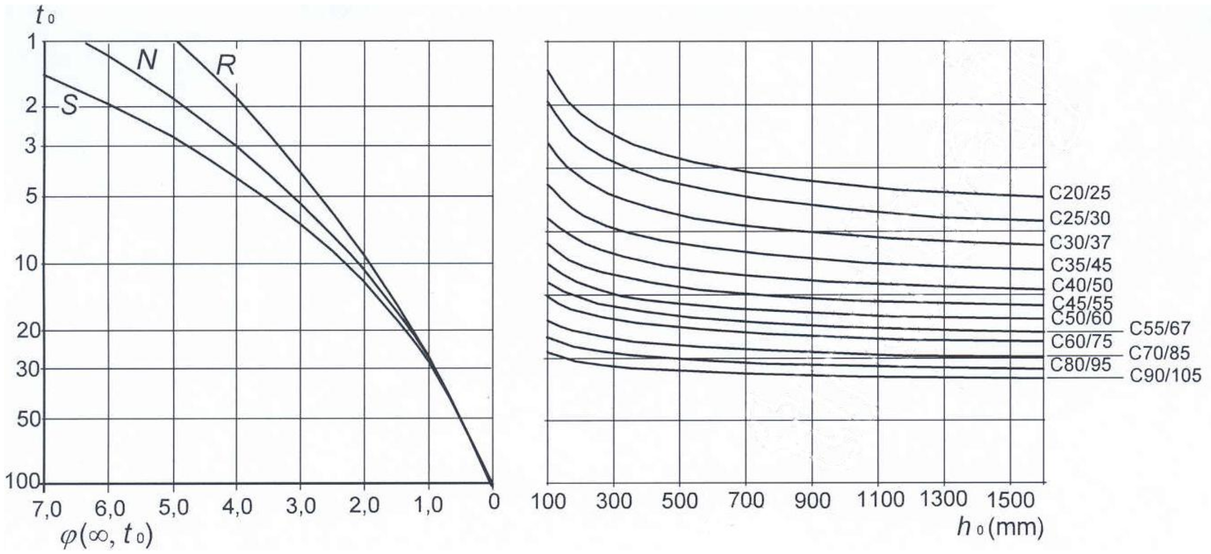
When the stress applied to the concrete at time t_0 is higher than 0,45 $f_{ck}(t_0)$, whereby the characteristic strength at that time is $f_{ck}(t_0)$, account must be taken of non linear effects, and the notional creep coefficient, $\varphi_{.k}(t, t_0)$, should be used:

$$\varphi_{.k}(t, t_0) = \varphi(t, t_0)\exp[1.5\,(k_\sigma - 0{,}45)];$$

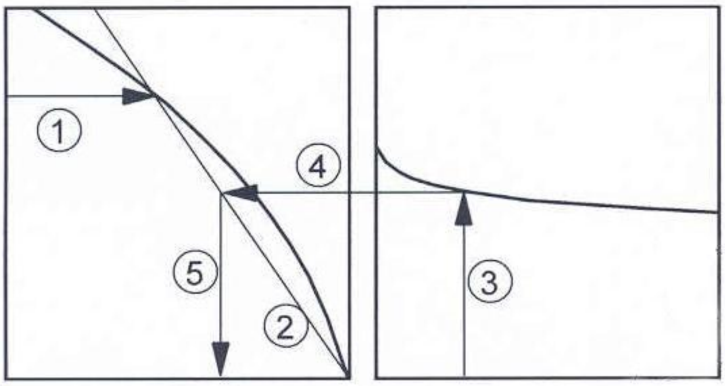
where k_σ is the stress-strength ratio, $\sigma_c/f_{cm}(t_0)$, under loading at time t_0.

The function $\varphi\,(t, t_0)$ depends primarily on: concrete compressive strength, stress and the ages when it is applied, relative humidity, the cross section of the member relative to its external perimeter, and the class of cement. A detailed description of a method for calculating this parameter is given in Annex B of [2].

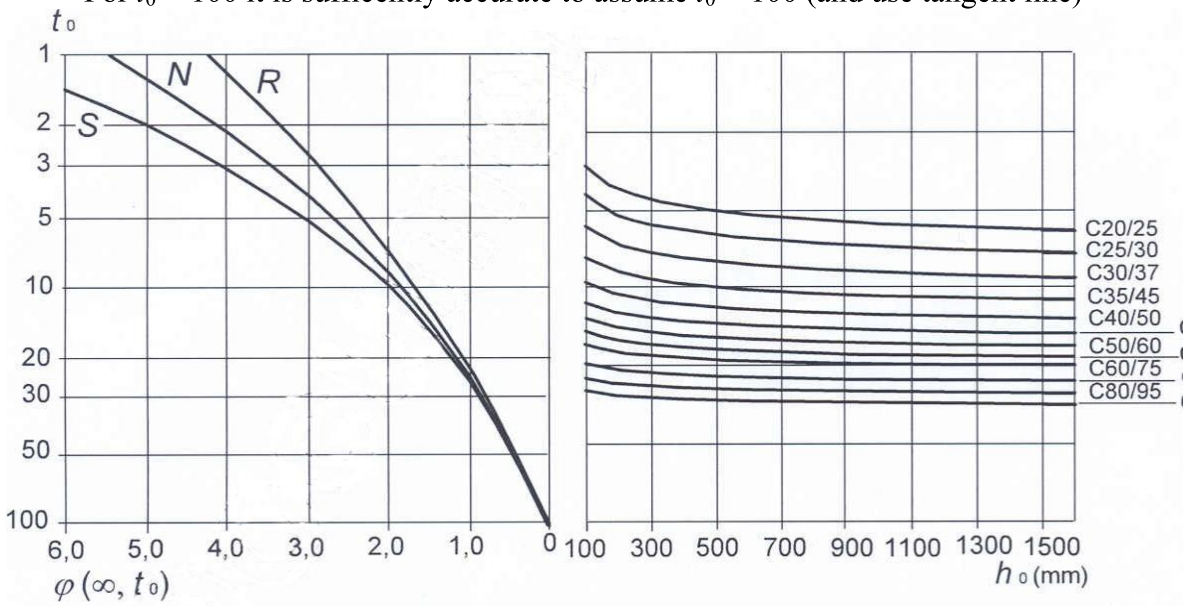
As a rule, the value of greatest interest is the final creep deformation computed from the final creep coefficient $\varphi(\infty, t_0)$. Values of this coefficient are shown graphically in [2], from which Figure 8 was taken.



a) Inside conditions RH =50%



Note: Intersection point between line 4 and 5 can also be above point 1
For t_0 > 100 it is sufficently accurate to assume t_0 = 100 (and use tangent line)



b) outside conditions RH =80%

Figure 8. Graphic computation of $\varphi(\infty, t_0)$ under normal conditions

Some sample values of the final creep coefficient, $\varphi(\infty, t_0)$ are given in Table 6, while Figure 9 contains a plot of the variation of the creep function, $\varphi(t, t_0)$, with time. The following parameters were chosen for these calculations: cement class N, age at first loading, t_0, equal to 28 days, and the notional size, h_0, equal to 500 mm Both the table and the figure were obtained by running the formulas in annex B [2] on the *Mathematica*® package *creepbuilding.nb* given in the annex in this chapter.

Table 6. Final creep coefficients $\varphi(\infty, t_0)$ for C30/37 concrete.

Age at loading t_0 (days)	Curing conditions											
	RH = 30 %				RH = 50 %				RH = 80 %			
	Notional size = 2 A_c/u (in mm)											
	100	200	500	1000	100	200	500	1000	100	200	500	1000
1	5,9	5,2	4,5	4,0	4,9	4,4	3,9	3,6	3,4	3,2	3,0	2,9
7	4,1	3,6	3,1	2,8	3,4	3,1	2,7	2,5	2,39	2,25	2,1	2,0
28	3,2	2,8	2,4	2,2	2,6	2,4	2,1	1,9	1,8	1,7	1,6	1,6
90	2,5	2,2	1,9	1,7	2,1	1,9	1,7	1,5	1,5	1,4	1,3	1,2
360	1,9	1,7	1,5	1,3	1,6	1,4	1,3	1,2	1,1	1,1	1,0	1,0

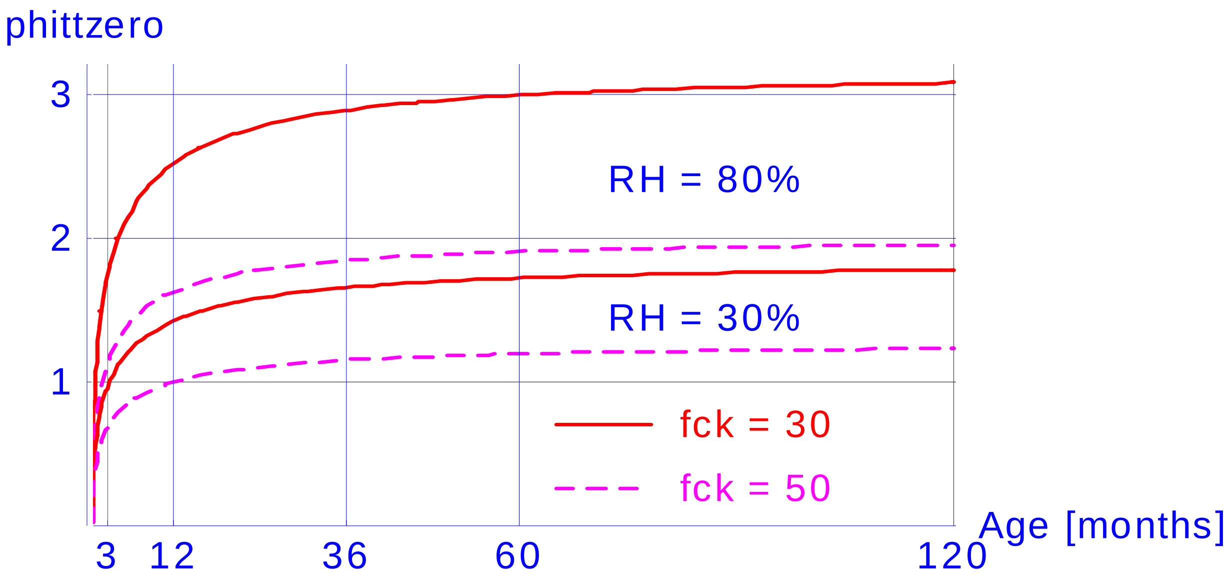


Figure 9. The time evolution of the creep coefficient $\varphi(t, t_0)$

4.5 Shrinkage

Concrete shrinkage is caused by the evaporation, drying and chemical bonding of water and cement. It is a common cause of cracking in concrete when deformation takes place too quickly or is overly restrained, and the resulting chloride penetration may corrode the reinforcing steel. It is related not to concrete stress, but to curing conditions. Like creep, shrinkage is a time-dependent process and is impacted by a number of parameters, such as concrete composition (water-cement ratio in particular) and relative humidity. Where relative humidity is high, concrete may actually swell. Shrinkage is modelled by means of the viscoelastic material model and accounted for in much the same way as creep.

Two main types of shrinkage must be addressed: drying shrinkage and autogenous shrinkage. The strain due to drying shrinkage develops slowly and is caused by the migration of the internal water through the hardened concrete. Since autogenous shrinkage is due to the chemical reactions taking place during concrete hardening, strain develops more quickly. The total shrinkage strain ε_{cs}, is the sum of the two strains.

$$\varepsilon_{cs} = \varepsilon_{cd} + \varepsilon_{ca}$$

Drying shrinkage

The final drying shrinkage, $\varepsilon_{cd,\infty}$, can be obtained by multiplying the basic drying shrinkage, $\varepsilon_{cd,0}$ by coefficient k_h, a function of the notional size h_0.

$\varepsilon_{cd,\infty} = k_h \cdot \varepsilon_{cd,0}$

The values of $\varepsilon_{cd,0}$ are given in Table 7 for different types of cement and relative humidities. These values were computed with the formulas in Annex B of [2]. The h_0 and k_h values are shown in Table 8

Table 7 Values of coefficient $\varepsilon_{cd,0}$ for different classes of cements and concretes

Cement class	f_{ck} kN/m^2	Relative Humidity [%]					
		20	40	60	80	90	100
S	30	0,46	0,43	0,36	0,22	0,12	0
	50	0,36	0,34	0,28	0,18	0,10	0
	70	0,28	0,27	0,22	0,14	0,08	0
N	30	0,55	0,52	0,43	0,27	0,15	0
	50	0,43	0,41	0,34	0,21	0,12	0
	70	0,34	0,32	0,27	0,17	0,09	0
R	30	0,73	0,69	0,58	0,36	0,20	0
	50	0,57	0,54	0,45	0,28	0,16	0
	70	0,45	0,43	0,36	0,22	0,12	0

Table 8. Values of k_h in function of the notional size

$\boldsymbol{h_0}$	100	200	300	≥ 500
$\boldsymbol{k_h}$	1,00	0,85	0,75	0,70

The development of drying shrinkage with time can be expressed as follows:

$$\varepsilon_{cd}(t) = \beta_{ds}(t, t_s) \cdot k_h \cdot \varepsilon_{cd,0}.$$

The coefficient $\beta_{ds}(t, t_s)$ is calculated as:

$$\beta_{ds}(t, t_s) = \frac{t - t_s}{t - t_s + 0{,}04\sqrt{h_0^3}};$$

where:

t is the age of the concrete at the time considered in days

t_s is the age of the concrete at the end of the curing period in days

The drying shrinkage pattern depicted in Figure 10 was plotted for h_0 = 100 mm and t_0 = 7 days.

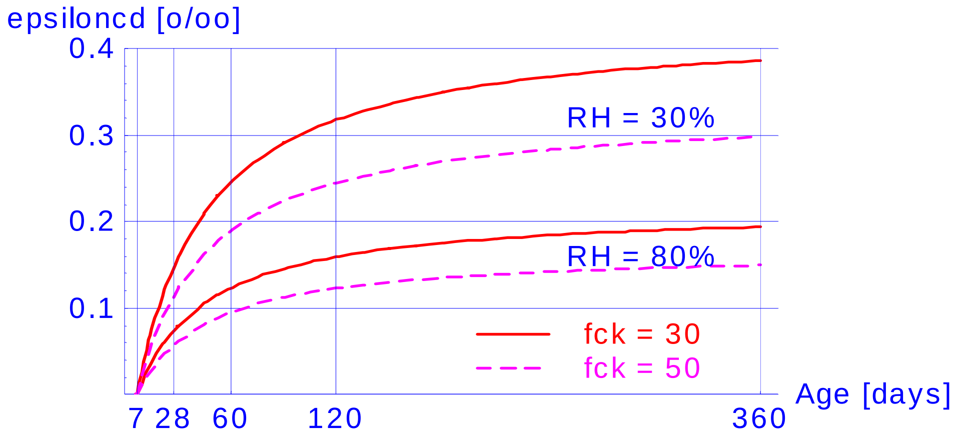


Figure 10 Evolution with time of the coefficient of drying shrinkage

Autogenous shrinkage

The strain due to autogenous shrinkage can be found with the expression:

$$\varepsilon_{ca}(t) = \beta_{as}(t)\, \varepsilon_{ca}(\infty);$$

where:

$\varepsilon_{ca}(\infty) = 2{,}5\,(f_{ck} - 10)\, 10^{-6}$,
$\beta_{as}(t) = 1 - \exp(-0{,}2\, t^{0{,}5})$,
t is the time in days.

Therefore, autogenous shrinkage depends on concrete class only, i.e., the characteristic strength of the concrete and time.

The development of autogenous shrinkage with time for two concrete classes is shown in Figure 11

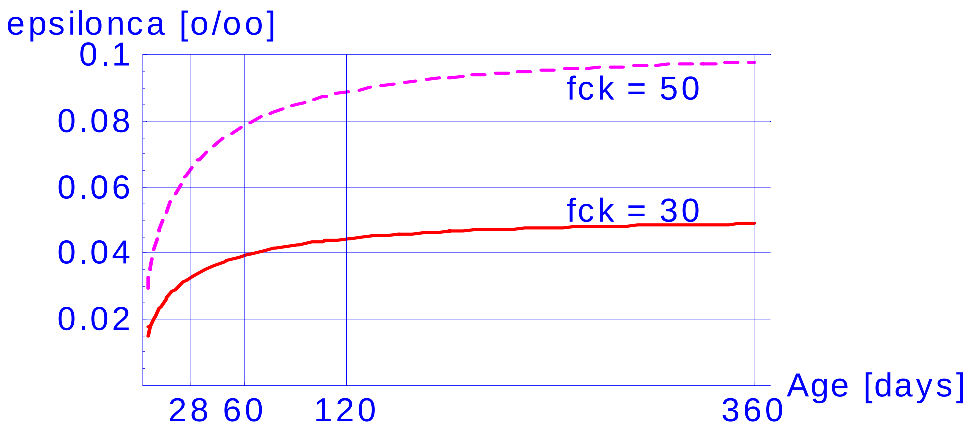


Figure 11. Evolution with time of the autogenous shrinkage

4.6 Durability

All recommendations related with durability are included in the general part of the EN 1992 [2] for all kind of structures.

Some attempts have been made in order to deal with this issue in the same way we deal with other actions; that is: defining probabilistically the environmental actions, its effects, the resistance of the structure and the Limit States, as indicated in EN 1990 [1], see, for instance, [7].

The most common way of solving this problem is by the use of deemed to satisfy rules, forcing the use of minimum concrete quality and cover thickness.

In order to state these rules [2] defines, in first place, the different environments that can influence the structures from the point of view of durability:

1 no risk of attack, X;
2 corrosion induced by carbonation, XC;
3 corrosion induced by chloride, XD;
4 corrosion induced by chlorides from the sea water, XS;
5 freeze/thaw attack, XF; and
6 chemical attack, XA.

For each one of these environments, but for the first one, in Table 4.1, are established three or four degrees of attack, Exposure Class, 1 to 3 or 4, from less to more severe attack. Then, for each degree and kind of environment, the Annex E recommends the minimum concrete class, from C20/25 to C35/45.

The following step is to define the Structural Class, from S1 to S6. Starting from the class S4, the class is increased or reduced depending of: the design working life, the strength class, the member geometry (slab or not) or ensured Quality Control, Table 4.3N.

Finally, for each Structural Class and each Exposure Class a minimum cover thickness for durability reasons, $c_{\min,\mathrm{dur}}$, is required in Tables 4.4N and 4.5N, for reinforcement steel or prestressing steel respectively.

This minimum cover for durability can be corrected following the indications of the National Annex, if they stated it, taking account of an additional safety margin or the use of stainless steel or additional protection.

The final minimum cover thickness is not fixed only by the durability reasons, but it also takes account of the need of cover to assure the bond between the steel and the concrete. The minimum cover for bond, $c_{\min,\mathrm{b}}$, is given the diameter of the reinforcement, in the case of separated bars, or the equivalent diameter, $\O_{\mathrm{n}}$, in the case of bundle bars.

The equivalent diameter is defined by the formula $\O_{\mathrm{n}} = \O\sqrt{n_{\mathrm{b}}}$, where: $\O$ is the individual bar diameter, and n_{b} the number of bars in the bundle, limited to 4 in the case of vertical bars in compression or bars in a lapped joint, and to 3 in other cases.

The final minimum cover is defined as:

$$c_{\min} = \max\{c_{\min,\mathrm{b}};\ c_{\min,\mathrm{dur}};\ 10\ \mathrm{mm}\}.$$

From this minimum cover, a nominal cover, c_{nom}, the value included in the drawings, is obtained adding to this minimum cover an allowance in design for deviation, Δc_{dev}:

$$c_{\mathrm{nom}} = c_{\min} + \Delta c_{\mathrm{dev}};$$

The value of Δc_{dev} can be defined in the National Annex.

As an illustration of these rules the following a simple example is given:

Example

A reinforced concrete bridge near the sea shore and submitted to de-icing salts on the deck.

Design characteristics:

Working life	100 years
Column and beams concrete	C 40/50
Deck concrete	C30/37
Maximum bar diameter	25 mm

Minimum cover:

Conditions	**Columns and beams**	**Deck**
Exposure class (Table 4.1)	XS 1	XD 3
Minimum strength class (Annex E)	C30/37	C35/45[1]
Adopted strength class	C40/50	C35/45
Initial structural class	S4	S4
Design working life 100 yr (Table 4.3N)	+2	+2
Strength class (Table 4.3N)	-1	=
Member slab geometry (Table 4.3N)	=	-1
Special Quality Control	=	=
Final Strength Class	S5	S5
Minimum cover for durability (Table 4.4N)	40 mm	50 mm
Minimum cover for bond (Table 4.2)	25 mm	25 mm
Minimum cover, c_{min}	40 mm	50 mm

[1]This strength class is higher than the first chosen in the design; we adopt this new strength class for the slab.

4.7 Other concrete properties

Other concrete properties used for calculating the ultimate and serviceability limit states are given in the table below [2]. The definitions are the same as for steel.

Table 9. Values of concrete parameters to be used in design calculations.

Property	**Symbol**	**Value**
secant modulus of elasticity	E_{cm}	See table 4
Poisson ratio	ν	0,2
shear modulus	G	$E/2(1+\nu)$
coefficient of linear thermal expansion	α_T	10×10^{-6} 1/^{o}C
unit mass, plain concrete	ρ	2400 kg/m^3
unit mass, reinforced concrete	ρ	2500 kg/m^3

Annexes

Mathematica® notebooks:

concreteprop_HB3.nb	to calculate the values of concrete characteristic strength and elastic moduluis at different ages.
creep_HB3.nb	for calculate the creep coefficient
shrinkage_HB3.nb	for calculate the shrinkage coefficients

References

[1] EN 1990 Eurocode - Basis of structural design. CEN 2002.
[2] EN 1992-1-1 Eurocode 2: Design of concrete structures – Part 1-1: General rules and rules for buildings. Brussels, 2005
[3] EN 1993-1-1 Eurocode 3: Design of steel structures – Part 1-1: General rules and rules for buildings.
[4] JCSS: *Probabilistic model code.* JCSS working materials, http://www.jcss.ethz.ch/, 2001.
[5] Stanek, M., Turk, G.: *The fundamentals of the mechanics of solid bodies*, University of Ljubljana, FGG, Ljubljana 1998.
[6] EN 1991-1-1 Eurocode 1 Actions on structures. Part 1-1 General actions. Densities, self-weight, imposed loads for buildings, CEN 2002.
[7] A.J.M. Siemes, A.C.W.M. Vrouwenvelder, A. van den Beuken: Durability of buildings: a reliability analysis, *Heron*, vol 30, 1985.

ANNEXES
Concreteprop_HB3.nb
creep_HB3.nb
shrinkage_HB3.nb

Shrinkage of concrete for buildings

```
 $TextStyle={FontSizeØ14,FontFamily->"Arial"}                              ;
 months=30;
```

Autogenous Shrinkage

Parameter values lists

```
 fcklist={30,50}; (*List of strength values*)
(*RHlist={20,40,60,80,90,100};(*List of Relative Humidities
values*)
timelist={3,7,14,28}; (* time in days in which evaluate the
propertie*)*)
```

Functions

```
 eca[t_,fck_]:= betaas[t] epsiloncainfty

 epsiloncainfty:=2.5(fck-10) 10^-3
betaas[t_]:=1-Exp[-0.2t^0.5]
fcm28:=fck+8
```

Drawing

Drawing parameters

```
drawingecaut=Range[Length[fcklist]];
maxordaut=0.1;
malla=Graphics[{Line[{{28,0},{28,maxordaut}}],

                        Line[{{60,0},{60,maxordaut}}],
                        Line[{{120,0},{120,maxordaut}}],
                        Line[{{360,0},{360,maxordaut}}],
                        Line[{{720,0},{720,maxordaut}}]}];

leyendecaut=Graphics[{Text["fck = 50",{250,0.090},{-1,0}],
                        Text["fck = 30",{250,0.053},{-1,0}]}];


 Do[fck=fcklist[[i]];

    drawingecaut[[i]]=
            Plot[eca[t,fck],{t,3,12months},
            PlotStyleØ{RGBColor[1,0.,i-1],

                  Thickness[0.007],Dashing[{.02,(i-
     1)/60}]},
     DisplayFunction->Identity ],
     {i,1,Length[fcklist]}];
     shrinkageaut=Show[drawingecaut,malla,leyendecaut,

TicksØ{{28,60,120,360},Automatic}, GridLinesØ
{None,{Automatic,RGBColor[0,0,1]}},
PlotRangeØ{{0,12months+10},{0,maxordaut}}, DefaultColor->RGBColor[0.,0.,1],
AxesLabelØ{"Age [days]","epsilonca [o/oo]"}, DisplayFunction-
>$DisplayFunction, FormatTypeØOutputForm];
```

Properties of concrete for buildings

Parameter values

```
 slist={0.2,0.35,0.38}; (*List of S values of interest*)
fcklist={30,50,70}; (*List of strength values*)
RHlist={30,80};(*List of Relative Humidities values*)
timelist={3,7,14,28}; (* time in days in which evaluate the
propertie*)
```

Internal parameters

```
drawing=drawingElas=drawingeca=drawingecd=drawingcreep=Range[4
];
 maxord=1.1Max[fcklist];
months=30;
 malla=Graphics[{Line[{{3,0},{3,20maxord}}],
                   Line[{{7,0},{7,20maxord}}],
                   Line[{{14,0},{14,20maxord}}],
                   Line[{{28,0},{28,20maxord}}],
                   Line[{{60,0},{60,20maxord}}],
                   Line[{{120,0},{120,20maxord}}],
                   Line[{{360,0},{360,20maxord}}],
                   Line[{{720,0},{720,20maxord}}]}];
leyendaresis=Graphics[{{RGBColor[1,0,0],Thickness[0.007],Line[
{{15,15},{19,15}}]},

     {Dashing[{.02,1/60}],Thickness[0.007],RGBColor[1,0,1],Line
[{{15,5},{19,5}} ]},
                             {RGBColor[1,0,0],Text["Class
R",{20,15},{-1,0}]},
                             {RGBColor[1,0,1],Text["Class
S",{20,5},{-1,0}]}}];
leyendaelast=Graphics[{{RGBColor[1,0,0],Thickness[0.007],Line[
{{15,28},{19,28}}]},

     {Dashing[{.02,1/60}],Thickness[0.007],RGBColor[1,0,1],Line
[{{15,26},{19,26}} ]},
                             {RGBColor[1,0,0],Text["Class
R",{20,28},{-1,0}]},
                             {RGBColor[1,0,1],Text["Class
S",{20,26},{-1,0}]},
                             Text["fck = 50",{30,38},{-1,0}],
                             Text["fck = 30",{30,32},{-
1,0}]}];
leyendaeca=Graphics[{Text["fck = 50",{250,90},{-1,0}],
                             Text["fck = 30",{250,53},{-
1,0}]}];
leyendaepscd=Graphics[{Text["RH = 80%",{250,160},{-1,0}],
                             Text["RH = 30%",{250,320},{-
1,0}],
```

```
     {RGBColor[1,0,0],Line[{{200,70},{240,70}}]},

     {Dashing[{.02,1/60}],Thickness[0.007],RGBColor[1,0,1],Line
[{{200,30},{240,30}} ]},
                              {RGBColor[1,0,0],Text["fck =
30",{260,70},{-1,0}]},
                              {RGBColor[1,0,1],Text["fck =
50",{260,30},{-1,0}]}}]  ;
```

Compressive Strength

Functions

```
fck[t_, s_]:= If[t > 28, fcm[t, s]- 8, fck28];
fcm[t_, s_]:= betacc[t, s] fcm28
betacc[t_, s_] := If[t < 28, Exp[s (1- Sqrt[28/t])],1]
fcm28:= fck28 + 8
Ecm[t_, s_] := 22 (betacc[t, s] fcm28/ 10)^0.3

 Print["fck     ",timelist[[1]],"      ",    timelist[[2]],"
",timelist[[3]],"      ",timelist[[4]],"    "];
Do[s= slist[[j]];Print[" s  = ",s];
     Do[fck28=fcklist[[k]];
      Print[fck28,"
",Table[fck[timelist[[i]],s],{i,1,Length[timelist]}]],

          {k,1,Length[fcklist]}],
    {j,1,Length[slist]}];
```

Drawing

```
 $TextStyle={FontSize→14,FontFamily->"Arial"} ;
 Do[fck28=fcklist[[i]];
     Do[s= slist[[j]];index=If[i==1,j,2+j];

     drawing[[index]]=Plot[fck[t,s],{t,3,2 months},

     PlotStyle→{RGBColor[1,0.,j-
1],Thickness[0.007],Dashing[{.02,(j-1)/60}]},
                                        DisplayFunction-
>Identity],
                              {j,1,2}],
     {i,1,2}];

strength=Show[drawing[[1]],drawing[[2]],drawing[[3]],drawing[[
4]],malla,leyendaresis,

Ticks→{{3,7,14,28},Automatic},
                                   GridLines→
{None,{Automatic,RGBColor[0,0,1]}},

     PlotRange→{{0,1.2months},{0,maxord}},
```

```
                                  DefaultColor-
>RGBColor[0.,0.,1],
                                  AxesLabel→{"Age
[days]","fck(t) [MPa]"},
                                  DisplayFunction-
>$DisplayFunction,
                                  FormatType→OutputForm];
```

Elastic Modulus

Functions

```
Ecm[t_, s_]:= 22 (betacc[t, s] fcm28/ 10)^0.3

 Print["Ecm     ",timelist[[1]],"       ",    timelist[[2]],"
",
     timelist[[3]],"       ",timelist[[4]],"    "];
Do[s= slist[[j]];Print[" s  = ",s];
     Do[fck28=fcklist[[k]];
      Print[fck28,"
",Table[Ecm[timelist[[i]],s],{i,1,Length[timelist]}]],

          {k,1,Length[fcklist]}],
    {j,1,Length[slist]}];
```

Drawing

```
 $TextStyle={FontSize→14,FontFamily->"Arial"} ;
 Do[fck28=fcklist[[i]];
     Do[s= slist[[j]];index=If[i==1,j,2+j];
               drawingElas[[index]]=Plot[Ecm[t,s],{t,3,months},

     PlotStyle→{RGBColor[1,0.,j-1],

     Thickness[0.007],Dashing[{.02,(j-1)/60}]},
                                                    DisplayFunction-
>Identity],
                              {j,1,2}],
     {i,1,2}];

Elastic=Show[drawingElas[[1]],drawingElas[[2]],drawingElas[[3]
],
               drawingElas[[4]],malla,leyendaelast,
                    Ticks→{{3,7,14,28},{25,30,35,40}},
                   GridLines→ {None,{30,35,40}},
                   PlotRange→{{0,35},{25,41}},
                   DefaultColor->RGBColor[0.,0.,1],
                   AxesLabel→{"Age [days]","Ecm(t) [GPa]"},
                   DisplayFunction->$DisplayFunction,
                   FormatType→Outp
```

Creep of concrete for buildings

Creep for infinite time

Parameter values

```
 fcklist={30,70}; (*List of strength values*)
RHlist={30,80};(*List of Relative Humidities values*)
timelist={1,7,28,360}; (* time in days in which evaluate the
propertie*)
h0={100, 200, 500 ,1000};(* notional size: h0 = 2 Ac/u *)
tzerolist={7,28,90,360}; (* age at initial loading*)
```

Variables

```
 fcm28:=fck+8
phi0[h0_,tzero_] :=phiRH[tzero]* betafcm*betat0[tzero]
phiRH[tzero_]:=If[fcm28 >35,(1+(1-RH/100)/(0.1
h0^(1/3))alpha1)alpha2, (1+(1-RH/100)/(0.1 h0^(1/3)))]
betafcm:=16.8/Sqrt[fcm28]
betat0[tzero_]:=1/(0.1+tzero^0.2)
betah:=If[ fcm28>35 ,1.5(1+(0.012RH)^18)h0 +250 alpha3,
1.5(1+(0.012RH)^18)h0 +250]
alpha1:=(35/fcm28)^0.7
alpha2:=(35/fcm28)^0.2
alpha3:=(35/fcm28)^0.5
```

Calculus

```
 Do[fck=fcklist[[i]];
  Do[RH=RHlist[[j]];Print["fck = ",fck," ; RH = ",RH];
    Do[tzero=tzerolist[[k]];

     Print[{tzero,Table[phi0[h0,tzero][[m]],{m,1,Length[h0]}]}]
,
               {k,1,Length[tzerolist]}],
     {j,1,Length[RHlist]}],
  {i,1,Length[fcklist]}]
```

Creep evolution

Variables

```
 phittzero[t_,h0_,tzero_]:=phi0[h0,tzero]  betac[t]
betac[t_]:=((t-tzero)/(betah+t-tzero))^0.3
```

Drawing

```
 $TextStyle={FontSize→14,FontFamily->"Arial"} ;
 maxord=3.2;months= 30;maxtime=120 months;
 mallacreep=Graphics[{Line[{{3 months,0},{3 months,maxord}}],
                   Line[{{12 months,0},{12 months,maxord}}],
                   Line[{{36 months,0},{36 months,maxord}}],
                   Line[{{60 months,0},{60 months,maxord}}],
                   Line[{{120 months,0},{120
months,maxord}}]}];
leyendacreep=
     Graphics[{Text["RH = 30%",{72months,0.45maxord},{-1,0}],
          Text["RH = 80%",{72months,0.75maxord},{-1,0}],
```

```
     {RGBColor[1,0,0],Thickness[0.007],Line[{{65months,.22maxor
d},{78months,.22maxord}}]},

     {Dashing[{.02,1/60}],RGBColor[1,0,1],Thickness[0.007],

     Line[{{65months,0.08maxord},{78months,.08maxord}} ]},
               {RGBColor[1,0,0],Text["fck =
30",{82months,.22maxord},{-1,0}]},
               {RGBColor[1,0,1],Text["fck =
50",{82months,.08maxord},{-1,0}]}}]  ;
```

Parameters

```
 h0 = 100;
tzero = 28;
fcklist = {30,50};
RHlist = {30,80};
drawingcreep = Range[Length[fcklist]*Length[RHlist]];
 Do[RH=RHlist[[j]];
  Do[fck=fcklist[[i]];index=If[j==1,i,2+i];

     drawingcreep[[index]]=Plot[phittzero[t,h0,tzero],{t,28,max
time},
               PlotStyle→{RGBColor[1,0.,i-
1],Thickness[.007],Dashing[{.02,(i-1)/60}]},
                   DisplayFunction->Identity ],
          {i,1,Length[fcklist]}],
  {j,1,Length[RHlist]}]
 creep=Show[drawingcreep,mallacreep,leyendacreep,
      Ticks→{{{3 months,3},{12 months,12},{36 months,36},{60
months,60},
                        {120 months,120}},{1,2,3}},
     GridLines→ {None,{1,2,3}},
     PlotRange→{{28,maxtime+10},{0,maxord}},
     DefaultColor->RGBColor[0.,0.,1],
     AxesLabel→{"Age [months]","phittzero"},
     AxesOrigin→{0,0},
     DisplayFunction->$DisplayFunction,
     FormatType→OutputForm];
```

Shrinkage of concrete for buildings

```
 $TextStyle={FontSize→14,FontFamily->"Arial"} ;
 months=30;
```

Autogenous Shrinkage

Parameter values lists

```
 fcklist={30,50}; (*List of strength values*)
(*RHlist={20,40,60,80,90,100};(*List of Relative
Humidities values*)
  timelist={3,7,14,28}; (* time in days in which
evaluate the propertie*)*)
```

Functions

```
 eca[t_,fck_]:= betaas[t] epsiloncainfty
 epsiloncainfty:=2.5(fck-10) 10^-3
betaas[t_]:=1-Exp[-0.2t^0.5]
fcm28:=fck+8
```

Calculus

Drawing

Drawing parameters

```
 drawingecaut=Range[Length[fcklist]];

 maxordaut=0.1;
malla=Graphics[{Line[{{28,0},{28,maxordaut}}],
                Line[{{60,0},{60,maxordaut}}],
                Line[{{120,0},{120,maxordaut}}],
                Line[{{360,0},{360,maxordaut}}],
                Line[{{720,0},{720,maxordaut}}]}];
leyendecaut=Graphics[{Text["fck = 50",{250,0.090},{-
1,0}],
                         Text["fck =
30",{250,0.053},{-1,0}]}];

 Do[fck=fcklist[[i]];
    drawingecaut[[i]]=
         Plot[eca[t,fck],{t,3,12months},
             PlotStyle→{RGBColor[1,0.,i-
1],Thickness[0.007],Dashing[{.02,(i-1)/60}]},
             DisplayFunction->Identity ],
    {i,1,Length[fcklist]}];
 shrinkageaut=Show[drawingecaut,malla,leyendecaut,
              Ticks→{{28,60,120,360},Automatic},
             GridLines→
{None,{Automatic,RGBColor[0,0,1]}},
```

```
    PlotRange→{{0,12months+10},{0,maxordaut}},
            DefaultColor->RGBColor[0.,0.,1],
            AxesLabel→{"Age [days]","epsilonca
[o/oo]"},
            DisplayFunction->$DisplayFunction,
            FormatType→OutputForm];
```

Drying Shrinkage

Parameter values lists

```
 fcklist={30,50,70}; (*List of strength values*)
RHlist={20,40,60,80,90,100};(*List of Relative
Humidities values*)
```

Functions

```
epsiloncdcero[RH_] := (0.85*((220 + 110*alphads1)*
       Exp[(-alphadstwo)*(fcm28/fcm0)])*betarh[RH])/10^3
betarh[RH_] := 1.55*(1 -(RH/RHcero)^3)
epsiloncd[t_] := betads[t]*kh*epsiloncdcero[RH]
betads[t_] := (t -ts)/((t -ts) + 0.04*Sqrt[h0^3])
fcm28 := fck + 8
```

Parameters

```
 fcm0=10;
RHcero=100;
h0=100;kh=1;
ts=7;
```

Parameters depending on the kind of cement

```
 alphads1list={3,4,6};(* cement class {S,N,R} *)
alphadstwolist={0.13,0.12,0.11}; (* cement class
{S,N,R} *)
cementlist={S,N,R};
```

Calculus

```
 RH=.
 Do[alphads1=
alphads1list[[k]];alphadstwo=alphadstwolist[[k]];
    Print["Cement class ",cementlist[[k]]," RH% "
,Table[ RHlist[[i]],{i,1,Length[RHlist]}]];
    Do[fck=fcklist[[i]];
        Print[{fck,Table[epsiloncdcero[RH]/.RH-
>RHlist[[j]],{j,1,Length[RHlist]}]}],
    {i,1,Length[fcklist]}],
  {k,1,Length[cementlist]}]
```

Drawing

```
 drawingecd=Range[Length[fcklist]*Length[RHlist]];
```

```
RHlist={30,80};(*List of Relative Humidities values
for drawing*)
fcklist={30,50}; (*List of strength values for
drawing*)
alphads1=
alphads1list[[1]];alphadstwo=alphadstwolist[[1]]; (*
cement class S *)
```

Drawing characteristics

```
 maxordry=0.4; (* max. ordenate in the drawing*)
malladry=Graphics[{Line[{{7,0},{7,maxordry}}],
                 Line[{{28,0},{28,maxordry}}],
                 Line[{{60,0},{60,maxordry}}],
                 Line[{{120,0},{120,maxordry}}],
                 Line[{{360,0},{360,maxordry}}],

    Line[{{720,0},{720,maxordry}}]}];leyendepscd=Grap
hics[{Text["RH = 80%",{250,.16},{-1,0}],
        Text["RH = 30%",{250,.320},{-1,0}],

    {RGBColor[1,0,0],Thickness[0.007],Line[{{200,.070
},{240,.070}}]},

    {Dashing[{.02,1/60}],Thickness[0.007],RGBColor[1,
0,1],Line[{{200,.030},{240,.030}} ]},
        {RGBColor[1,0,0],Text["fck =
30",{260,.070},{-1,0}]},
        {RGBColor[1,0,1],Text["fck =
50",{260,.030},{-1,0}]}}]  ;
 Do[RH=RHlist[[j]];
  Do[fck=fcklist[[i]];index=If[i==1,j,2+j];
    drawingecd[[index]]=
            Plot[epsiloncd[t],{t,0,12months},
                PlotStyle→{RGBColor[1,0.,i-
1],Thickness[0.007],Dashing[{.02,(i-1)/60}]},
                DisplayFunction->Identity ],
    {i,1,Length[fcklist]}],
  {j,1,Length[RHlist]}]
 shrinkagedry=Show[drawingecd,malladry,leyendepscd,
         Ticks→{{7,28,60,120,360},Automatic},
        GridLines→
{None,{Automatic,RGBColor[0,0,1]}},
        PlotRange→{{0,12months+10},{0,maxordry}},
```

```
DefaultColor->RGBColor[0.,0.,1],
AxesLabel→{"Age [days]","epsiloncd [o/oo]"},
DisplayFunction->$DisplayFunction,
FormatType→OutputForm];
```