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and



PRELIMINARY FOUNDATION DESIGN OF AN INDUSTRIAL BUILDING IN KAUNAS, LITHUANIA

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To my parents

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RESUMO

O contínuo desenvolvimento das metodologias adoptadas pela Engenharia Civil torna o dimensionamento de estruturas progressivamente mais seguro, fiável e ao mesmo tempo com menores implicações financeiras.

Com o surgimento do programa dos Eurocódigos Estruturais, os procedimentos de dimensionamento têm experimentado uma crescente homogeneização, a nível tecnológico e metodológico, na área definida por todos os países membros da União Europeia.

Assim, o dimensionamento da infra-estrutura de um edifício industrial na Lituânia não apresenta, nos tempos que correm, diferenças que se possam considerar significativas no que toca aos procedimentos gerais que são utilizados em Portugal.

Contudo, fenómenos como as grandes amplitudes térmicas que se fazem sentir durante o Inverno, o gelo e degelo do solo e nível freático, assim como os elevados níveis de intensidade e frequência de precipitação em estado sólido sob forma de neve, fazem com que a Lituânia possua algumas pequenas diferenças e pormenores singulares a atentar, no que respeita ao dimensionamento de edifícios e obras de Engenharia Civil, relativamente aos países do Oeste da Europa.

No projecto geral das fundações de um edifício é fundamental conhecer as características geológicas e geotécnicas do solo onde se pretende fundar a estrutura, podendo a escolha da tipologia e as dimensões das fundações a considerar ser condicionada consoante o tipo de solo onde se está a trabalhar.

Esta dissertação apresenta então uma metodologia desenvolvida com o intuito de proporcionar o correcto dimensionamento da fundação superficial ou sapata, do pilar central de um pavilhão industrial a construir na cidade de Kaunas, na Lituânia. Com esse objectivo é desenvolvida, numa primeira fase, uma abordagem à acção do gelo e degelo nos solos de fundação, focando com especial atenção os fenómenos de empoamento do solo por congelação, do enfraquecimento associado ao degelo e do "permafrost" (solo permanentemente congelado), sendo em seguida elaborados todos os cálculos com vista ao dimensionamento referido.

O cálculo do nível de possível congelação e da capacidade resistente do solo ao corte, a estimativa dos assentamentos que a estrutura poderá experimentar, bem como a avaliação da resistência da sapata à possibilidade de corte por esforço transversal e por punçoamento, respeitante às dimensões previamente pré-dimensionadas, constituem os pontos fulcrais de abordagem deste texto onde é também exposta a tipologia típica de construção de coberturas de edifícios industriais na Lituânia correspondente na maioria das vezes às orientações técnicas russas.

PALAVRAS-CHAVE: dimensionamento, fundações, época de congelação, capacidade resistente do solo, assentamento.

ABSTRACT

The continuing development of Civil Engineering methodologies makes the design of structures progressively safer, reliable and at the same time with less financial implications.

With the emergence of the Structural Eurocode Programme, the design procedures have experienced a growing technological and methodological homogenisation within the member states of the European Union.

Thus, presently, the design of the infrastructure of an industrial building in Lithuania does not present significant differences at the level of general procedures to those used in Portugal.

However, phenomena such as the large winter thermal amplitudes, the freezing of the phreatic level and of the soil, as well as the high intensity and frequency levels of snowfall, introduce some small differences and singular details to the design of buildings and Civil Engineering works in Lithuania, comparatively with Western European countries.

In the general project of a building foundation it is fundamental to determine the geological and geotechnical characteristics of the soil where the structure will be founded, since the soil type may condition the type and dimensions of the foundations to use.

This work presents a methodology developed with the aim of providing the correct design of a shallow foundation or footing, of the central column of a generic industrial pavilion to be built in the city of Kaunas, in Lithuania. With that objective in mind an approach is first developed to account for freeze and thaw actions in foundation soils, giving particular attention to the phenomenon of frost heave, thaw weakening and permafrost. All the necessary design calculations are then carried out.

The frost line and soil bearing capacity calculation, the estimate of the settlement that the structure can suffer and the evaluation of footing simple and punching shear risk constitute the main topics of this text, where is also described the typical typology of Lithuanian industrial pavilion roof construction, following most of the time the Russian technical practice.

KEYWORDS: design, foundations, frost heave, soil bearing capacity, settlement.

INDEX

ACKNOWLEDGEMENTS	i
RESUMO	iii
ABSTRACT	v
1. INTRODUCTION	1
1.1. MOTIVATION	1
1.2. AIM.....	3
1.3. TASKS.....	4
1.4. CONTRIBUTIONS OF THE THESIS.....	4
1.5. ORGANIZATION OF THE THESIS.....	4
2. BACKGROUND	7
2.1. GEOTECHNICAL AND GEOLOGICAL CONDITIONS	7
2.1.1. PORTO	7
2.1.2. KAUNAS	8
2.1.3. RELATION OF THE TYPE OF FOUNDATIONS WITH GEOTECHNICAL CONDITIONS	10
2.2. WEATHER CONDITIONS	11
2.2.1. FROST ACTION	11
2.2.1.1. Frost Heave.....	11
2.2.1.2. Thaw weakening.....	13
2.2.1.3. Permafrost	13
2.2.2. SNOW	15
2.2.2. WIND	16
2.3. FORMULATIONS	16
2.3.1. EUROCODE 7.....	17
2.3.2. EUROCODE 2.....	17
2.4. FINITE ELEMENT METHOD (FEM).....	17
3. PROJECT CHARACTERISTICS	19
3.1. BUILDING CHARACTERISTICS	19

3.2. GEOTECHNICAL CONDITIONS	21
3.2.1. FROST LINE	21
4. FOOTING DIMENSIONS	25
4.1. LOADS AT THE BASE OF COLUMN	25
4.1.1. INFLUENCE AREA	25
4.1.2. SELF WEIGHT OF COLUMN AND ROOF	27
4.1.3. WEIGHT OF SNOW	28
4.1.4. STRENGTH OF THE WIND	28
4.2. PRELIMINARY DESIGN OF THE FOOTING	28
4.2.1. VERIFICATION OF THE COMPRESSED FOOTING AREA	32
4.2.2. SHEAR AND PUNCHING VERIFICATION	32
4.2.2.1. Punching Resistance Verification	33
4.2.2.2. Shear Resistance Verification	36
5. BEARING RESISTANCE CALCULATION	39
5.1. BEARING CAPACITY IN DRAINED CONDITIONS	43
5.1.1. DIMENSIONLESS FACTORS	44
5.1.1.1. Bearing capacity factors	44
5.1.1.2. Inclination of the foundation base factors	44
5.1.1.3. Correction factors of the shape of foundation base	45
5.1.1.4. Inclination factors of the load	45
5.1.2. BEARING RESISTANCE OF THE SOIL AND THE SAFETY OF THE STRUCTURE	46
5.2. BEARING CAPACITY IN UNDRAINED CONDITIONS	47
5.2.1. DIMENSIONLESS FACTORS	48
5.2.1.1. Inclination of the foundation base factors	48
5.2.1.2. Correction factors of the shape of foundation base	48
5.2.1.3. Inclination factors of the load	48
5.2.2. BEARING RESISTANCE OF THE SOIL AND THE SAFETY OF THE STRUCTURE	48
5.3. SLIDING	50

6. SETTLEMENT ESTIMATE AND ANALYSIS	53
6.1. SETTLEMENT ESTIMATE	54
6.1.1 SETTLEMENTS FROM ELASTICITY THEORY.....	55
6.1.2. EC7 ADJUSTED ELASTICITY METHOD	58
7. DESIGN OF REINFORCING STEEL	61
8. CONCLUSION	67
REFERENCES	69
ANNEXES	73
ANNEX 1 CONSTRUCTION TECHNICAL REGULATIONS OF LITHUANIA (SNOW PRESSURE ANALYSIS)	75
ANNEX 2 CONSTRUCTION TECHNICAL REGULATIONS OF LITHUANIA (WIND PRESSURE ANALYSIS)	83
ANNEX 3 INDUSTRIAL BUILDING PROJECT DRAWS.....	87
ANNEX 4 BEARING CAPACITY CALCULATIONS IN DRAINED CONDITIONS.....	93
ANNEX 5 BEARING CAPACITY CALCULATIONS IN UNDRAINED CONDITIONS	97
ANNEX 6 NECESSARY PARAMETERS TO CALCULATE THE INCREMENT OF STRESSES UNDER A UNIFORM LOAD APPLIED IN A RECTANGULAR AREA.....	103
ANNEX 7 VALUES OF THE FACTOR I_s	109
ANNEX 8 EC2 DESIGN TABLES OF CONCRETE STRUCTURES SUBJECT TO SIMPLE BENDING	113

INDEX OF FIGURES

Fig.1.1 – Boussinesq solution (Tensão nos Solos, Bastos Cezar)	2
Fig.1.2 – Vertical strain under the centre of the footing (Schmertmann 1970)	3
Fig.2.1 – City of Porto (New York Times).....	7
Fig.2.2 – Geotechnical zoning of a soil in city of Porto (Metro of Porto Construction).....	8
Fig.2.3 – Photos of Kaunas - a) Vilnius Street (Kauno Aerouostas Website); b) Laives Avenue (Seventh European Dependable Conference); c) Aleksotas Bridge (Seventh European Dependable Conference).....	9
Fig.2.4 – Generic soil in the city of Kaunas	10
Fig.2.5 – Formation of ice lenses (WAPA Asphalt Pavement Guide)	12
Fig.2.6 – Cracks on a pavement caused by frost heave (WAPA Asphalt Pavement Guide).....	13
Fig.2.7 – Regions with permafrost (Image from Wikipedia)	14
Fig.2.8 – Damage on a building provoked by the thawing of permafrost (Journal of Geocryology)	15
Fig.2.9 – Lithuania snow load areas. Image of Statybus Techniniai Reglamentai (Construction Technical Regulations of Lithuania)	15
Fig.2.10 – Lithuania wind speed reference areas. Image of Statybus Techniniai Reglamentai (Construction Technical Regulations of Lithuania).....	16
Fig.3.1 – Plan of the industrial building	19
Fig.3.2 – Transversal cut of the building A/2-A/2'	20
Fig.3.3 – Longitudinal cut of the building 2-2'	20
Fig.3.4 – Footing location in the soil.....	23
Fig.4.1 – Influence area of column P.....	26
Fig.4.2 – Longitudinal section 2-2' with influence area of column P	26
Fig.4.3 – Transversal section A/2-A/2' with influence area of column P	27
Fig.4.4 – Snow load distribution (Statybus Techniniai Reglamentai).....	28
Fig.4.5 – Plan of the footing	31
Fig.4.6 – Example of a footing section (Fundações e Estruturas de Suporte, Arêde, António)	33
Fig.4.7 – Control perimeter for three different types of columns (EC2)	33
Fig.4.8 – Shear critical section (Fundações e Estruturas de Suporte, Arêde, António).....	36
Fig.5.1 – General shear failure (Vesic,1963)	39
Fig.5.2 – Local shear failure (Vesic,1963).....	39
Fig.5.3 – Punching shear failure (Vesic,1963)	40
Fig.5.4 – Evolution of the general shear zone of a soil under a footing loaded by a centered and continually increasing vertical load (Bearing Capacity of Foundations – University of Pretoria)	40

Fig.6.1 – Tower of Pisa, Italy (Waine, Mary in www.woodenspears.com) 53

Fig.6.2 – Generic time-settlement function of one foundation (Perloff, 1975)..... 54

Fig.6.3 – Uniform load applied on a rectangular area (Mecânica dos Solos – Conceitos e Princípios Fundamentais. Matos Fernandes, Manuel)..... 56

Fig.7.1 – Pressure in the footing-soil contact (Fundações e Estruturas de Suporte, Arêde, António).. 61

Fig.7.2 – Plan of footing..... 64

Fig 7.3 –Transversal section of footing..... 65

INDEX OF TABLES

Table 3.1 – Soil layers and estimated geotechnical indicators	21
Table 3.2 – Frost line depth.....	22
Table 4.1 – Self weight of structural elements	27
Table 4.2 – Loads at the base of the column	29
Table 4.3 – Partial safety factors.....	29
Table 4.4 – Loads to use in the design of the footing	30
Table 4.5 – Values of B_x and B_y	31
Table 5.1 – Loads at the base of the footing	42
Table 5.2 – Loads and soil parameters after applying the safety factors.....	43
Table 5.3 – Bearing capacity factors	44
Table 5.4 – Correction factors of the shape of foundation	45
Table 5.5 – Bearing resistance and global safety of the structure for drained conditions	46
Table 5.6 – $R_v, d/V_d$ for the different soil layers in drained conditions	47
Table 5.7 – Values of undrained resistance of the soil for layer 5	47
Table 5.8 – Bearing resistance and global safety of the structure for undrained conditions	49
Table 5.9 – Bearing resistance and global safety of the structure with the final dimensions of the footing.....	49
Table 5.10 – R_v/V_d for the different types of soil in undrained conditions.....	50
Table 6.1 – Increments of Stress in the soil beneath the footing	57
Table 6.2 – Immediate settlements of the footing	57
Table 6.3 – Values of I_s at the centre of the loaded area	58
Table 7.1 – Materials and geometric parameters of the foundation.....	62

SYMBOLS AND ABBREVIATIONS

SYMBOLS

Latin Letters

a - distance from the periphery of the column to the control perimeter considered (equation 4.22)

a_0 - biggest distance between the face of the column and the edge of the footing (equation 4.9)

A' - effective area of the footing

A_c - effective area of the footing (equation 5.31)

$A_{foot.}$ - area of the footing

A_s - area of steel

A_u - area of control (equation 4.21)

A_1 - area of the critical shear zone (equation 4.30)

b_w - smallest width of the cross-section in the tensile area

b - the design values of the factors for the inclination of the base, with subscripts c , q and γ

b - foundation width (equation 6.8)

B - width of the foundation

B' - effective width of the foundation

B_0 - effective dimension of the footing

B_x - dimension of the footing in direction OX

B_y - dimension of the footing in direction OY

c - cohesion intercept

c' - cohesion intercept in terms of effective stress

c_u - undrained shear strength of the soil

$C_{u;d}$ - undrained shear strength of the soil (equation 5.31)

d - mean effective depth of the footing

d_{fn} - characteristic ice depth

d_t - frost line depth

d_0 - coefficient related with the type of the soil

e_x - eccentricity of the load in direction OX

e_y - eccentricity of the load in the direction OY

E - deformation modulus

E_j - deformation modulus of the different layers of the soil (equation 6.2)

E_m - design value of the elasticity modulus

f - settlement coefficient

f_{ck} - characteristic compressive cylinder strength of concrete at 28 days (MPa)

f_{cd} - design value of concrete compressive strength

f_{syd} - design yield strength of reinforcement

h - height of the footing

h_i - thickness of the different layers of the soil (equation 5.3)

h_j - thickness of the different layers of the soil (equation 6.2)

H - horizontal force

i - the inclination factors of the load, with subscripts cohesion c , surcharge q and weight density γ

I_s - adimensional factor that depends on geometry of loaded area and of the point of the surface considered

k_h - thermal coefficient depending on the building temperature

k - coefficient dependent on the ratio between the column dimensions (equation 4.18)

k_1 - value proposed in the EC2 National Annexes (equation 4.31.a)

L - length of the foundation

L' - effective length of the foundation

L_x - dimension of the footing base in the direction OX

L_y - dimension of the footing base in the direction OY

m - exponent in formulas for the inclination factor i (equations 5.20 e 5.21)

M_{Ed} - moments applied at the base of the column

M_t - coefficient depending on the region where the building will be founded

M_x - moment in direction OX

M_y - moment in direction OY

N - bearing capacity factors, with subscripts for c , q and γ

N_{sd} - design value of the vertical load, or component of the total action acting normal to the foundation base (equations 4.4 and 4.5)

p - bearing pressure, linearly distributed on the foundation base

Q_{ult} - theoretical load of rupture

R_0 - calculated soil strength

R_d - friction of the base of the foundation

$R_{p,d}$ - design value of the resisting force caused by earth pressure on the side of the foundation

R/A' - bearing capacity of the soil

s - the shape factors of the foundation base, with subscripts for c , q and γ

s - settlement (chapter 6)

s_0 - immediate settlement

s_1 - settlement caused by consolidation

s_2 - settlement caused by creep (secondary settlement)

u - length of the control perimeter considered (equation 4.18 e 4.19)

V_{ED} - shear force applied on the footing

$V_{foot.}$ - volume of the footing

V_{pillar} - volume of the column

V_d - design value of vertical load

$V_{Rd,c}$ - design shear resistance of the member without shear reinforcement

W - parameter that considers the distribution of shear (equation 4.18)

$W_{arch.}$ - weight of both arches supported by the footing

$W_{foot.}$ - weight of the footing

W_{pillar} - self weight of the pillar

$W_{plat.}$ - weight of the concrete reinforced plates supported by the footing

W_{snow} - weight of the snow

$W_{ter.}$ - weight of the terrain above the footing

Greek Letters

γ - weight density

γ' - design effective unit weight of the soil below the foundation level

γ_c - unit weight of concrete

γ_c' - partial factor for the effective cohesion

γ_{cu} - partial factor for the undrained shear strength

γ_G - partial factor for permanent actions, G

γ_i - unit weight of the different layers of the soil (equation 5.3)

γ_q - partial factor for pressure created by the terrain on the base of the footing

γ_Q - partial factor for variable actions, Q

γ_{Rv} - partial factor for bearing resistance

γ_s - unit weight of steel

γ_φ - partial factor for the angle of shearing resistance

δ_d - friction angle between the footing and the foundation soil

μ - reduced bending moment

ν - strength reduction factor for concrete cracked in shear (equation 4.28)

ν_j - Poisson ratio of the different layers of the soil

ν_{ED} - punching shear stress (equations 4.16, 4.17, 4.18 e 4.19)

$\nu_{Rd,c}$ - design value of the maximum punching shear resistance of a footing without reinforcement (equation 4.17)

$\nu_{Rd,max}$ - design value of the maximum punching shear resistance along the control section considered (equation 4.16)

ρ - percentage of tensile reinforcement steel of the footing

ρ_l - percentage of tensile reinforcement relative to the concrete effective area (4.31.a)

ρ_{ly} - percentage of tensile reinforcement steel of the footing in direction OY

ρ_{lz} - percentage of tensile reinforcement steel of the footing in direction OZ

σ_0 - pressure in the contact between soil and footing

σ_{cp} - compressive stress in the concrete from axial load or prestressing

σ_{rd} - resistance of the soil

$\sigma_{3/4}$ - stress applied to the footing by the soil at $3/4$ of his length (equation 4.30)

ϕ - internal friction angle

ΔV_{Ed} - net upward force within the control perimeter considered i.e. upward pressure from soil minus self weight of base (equations 4.20 e 4.21)

$\Delta\sigma_{zj}$ - stress increment induced by Δq_s in direction OZ

$\Delta\sigma_{xj}$ - stress increment induced by Δq_s in direction OX

$\Delta\sigma_{yj}$ - stress increment induced by Δq_s in direction OY

ω - mechanical reinforcement ratio

ABBREVIATIONS

FEM – Finite Element Method

EC2 – Eurocode 2: Design of concrete structures

EC7 – Eurocode 7: Geotechnical design

EH-80 – Spanish instructions for design and execution of concrete structures

S.F. – Global Safety Factor of the structure against soil shear

TK6-1 – Tables of Typical Structural Buildings in Lithuania

UNESCO – United Nations Educational, Scientific and Cultural Organization

WAPA – Washington Asphalt Pavement Association

1

INTRODUCTION

1.1. MOTIVATION

The project of a building is subdivided in smaller projects of different specialties: Structural Project, Foundation Project, Acoustical Project, Electrical Project, among many others. In the last century the foundation project gained increasing importance, being nowadays known that the settlements of the buildings and bearing capacity of the soil are two aspects that must be carefully studied to avoid the risk of enormous damage or even the collapse of the structures.

The foundations of the building can be designed as shallow foundations or deep foundations according to the conditions of the soil and the dimensions of the building.

Independently of the type of foundations, the structure will transmit its weight to them and then to the soil, creating in it stresses and strains.

According to the soil bearing pressure theory, when a certain stress level is exceeded, it will form rupture surfaces in the soil that will cause huge settlements and eventually the collapse of the whole structure. However, smaller values of stresses will also result in stresses and settlements of the foundations. When settlements happen with large magnitude or not uniformly in all the foundations (differential settlements), significant damage may occur in the building, in the form of cracking and distortion of doors, windows, panels, pavements, stairs and other elements. In case of strong seismic activity, large differential settlements can provoke also the collapse of the entire building.

The value of the differential settlements depends on the distribution of the structural loads on the foundation, on the stiffness of the soil and on the uniformity of the soil and structure.

However, the study of the soils is one of the most difficult tasks for engineers because the soil conditions will always be incompletely defined, no matter how extensive prospection works are. So, engineers study the stress and strain increments caused by loadings in different types of soil, normally homogeneous ones, supporting these studies in theories and computer models and then try to make their correspondence with the reality, bearing in mind that in nature soils are not generally homogeneous.

“Unfortunately, soils are made by nature and not by man, and the products of nature are always complex...As soon as we pass from steel and concrete to earth, the omnipotence of theory ceases to exist. Natural soil is never uniform. Its properties change from point to point while our knowledge of its properties is limited to those few spots at which the samples have been collected. In soil mechanics the accuracy of computed results never exceeds that of a crude estimate, and the principal function of theory consists in teaching us what and how to observe in the field” – Karl Terzaghi.

To accommodate their doubts and uncertainties, engineers, when designing any foundation make use

of safety factors, to prevent unexpected conditions that have not been identified. They design the structures in a way that it can resist to unforeseen conditions without taking any risks, making of safety by the primordial condition of their projects.

Because foundation bearing capacity and settlement are two fundamental factors for the good health of structures it was necessary to develop theoretical expressions to calculate stress, strain and settlement in loaded soil.

Joseph Valentin Boussinesq was a pioneer in this subject, deriving expressions to determine stresses at any point of a homogeneous and isotropic half-space of infinite depth and lateral extent, when a point load is applied perpendicularly to the surface (Figure 1.1).

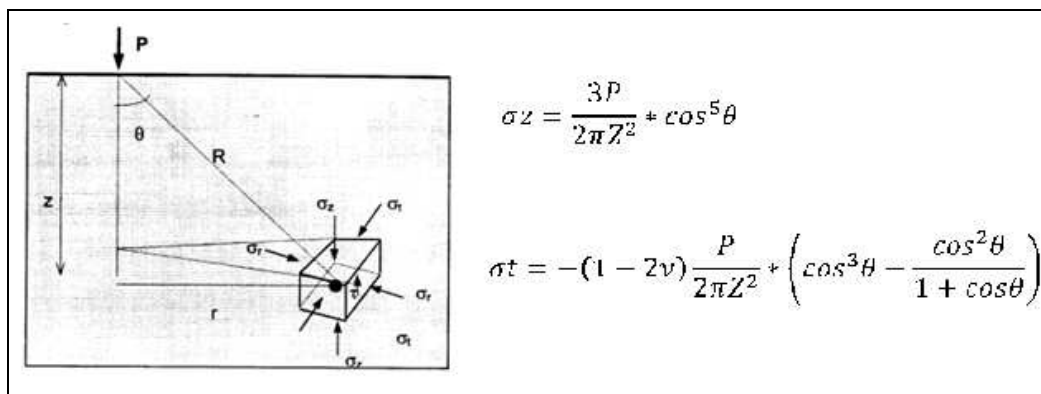


Fig.1.1- Boussinesq solution (Tensão nos Solos, Bastos Cezar)

The stress field for distributed load acting at the half space surface over any given area can be obtained by integration of the Boussinesq solution. Other solutions have been developed for layers of finite thickness, multi-layered soil masses and internally applied loadeds (Ahlvin and Ulery, 1962; Giroud, 1970; Newmark, 1942; Poulos and Davis, 1974; and others).

Before 1970 engineers often assumed that the strain under the center of a footing had its largest value immediately under the footing and decreased with depth. In May 1970 Schmertmann published a paper in the "ASCE Journal of the Soil Mechanics and Foundations Division" showing that the maximum vertical strain does not occur immediately under the loading, but at some depth function of dimensions of the loaded area (Figure 1.2). In 1978, this same author made some corrections to the first document, but the same basic ideas maintained.

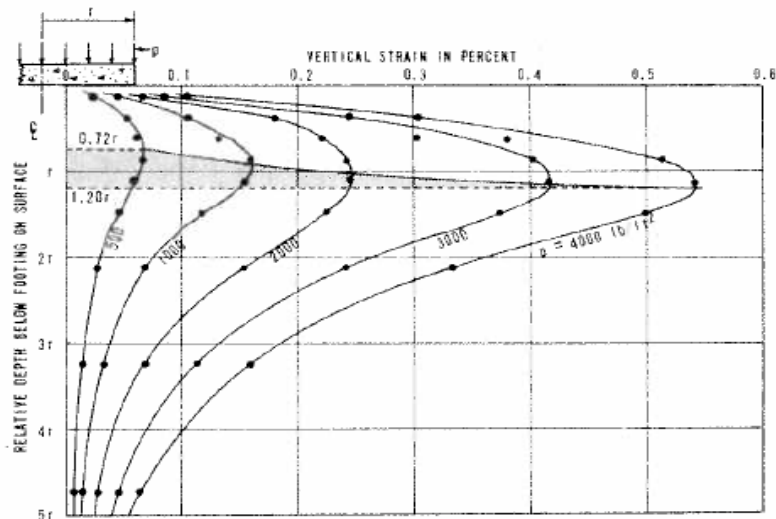


Fig.1.2 – Vertical strain under the centre of the footing (Schmertmann 1970)

Currently in the European Union foundation design is regulated by the Eurocodes, based on theories that were developed during many years. In spite of that, the study of stresses and strains in soils will never be complete, and the theories that were created can be improved or completed as new ones appear, because as it was said before, this is a very uncertain and complex area to study.

As Terzaghi said, soils are made by nature and all the phenomena that occur in nature are of very difficult representation.

In fact, before the advent of the computered technologies, the analytical representation of natural phenomena was limited to very simple cases because in spite of it being possible to formulate complex cases, it would take too much time to complete the calculations.

So, natural phenomena can be studied approximately by numerical methods like Finite Differences, Variational Methods (Ritz, Least Square, Courant), Finite Elements and many others. These methods associated to the rapid development of computer capacity and speed, became a powerful tool in the solution of daily problems of engineers.

1.2. AIM

In context with the subject of Project/Research of Geotechnical Option of the 5th year of Integrated Master in Civil Engineering of the Faculty of Engineering of University of Porto in the 2008/2009 school year, this document, developed at the Kaunas University of Technology, in the Erasmus exchange program, **aims to illustrate the influence of Lithuanian climate and soil conditions to the design of shallow foundations, by the development of the preliminary foundation design for an industrial building in Kaunas.**

Since the study was developed in the city of Kaunas in Lithuania, it is relevant to study the geotechnical conditions of the city region and, most importantly, the weather conditions, which are extremely different from those in Portugal. So the wind and the snow frequency and intensity will be studied, together with a particular phenomenon, the frost heave, caused by very low temperatures that occur in winter reaching $-30\text{ }^{\circ}\text{C}$.

In order to accomplish the proposed aim the tasks presented in the next sub-chapter will be developed.

1.3. TASKS

- Study of the Lithuanian geotechnical conditions making a brief comparison between them and those in the Porto region, to understand the effect of such differences on the type of foundations used in both countries;
- Description of Lithuanian weather conditions, accounting mainly for the snow frequency and to frost heave, presenting the definition of the latter and showing the influence of both on Lithuanian Civil Engineering;
- Development of the preliminary design of a rectangular industrial pavilion with dimensions of 42x24 meters in plant, according to the Lithuanian Typical Building Structures Tables TK6-1;
- Quantification of the loads applied by one column to the foundation according to the weight of the different elements of the building and of the snow;
- Design of the shallow foundation of the industrial building using the Spanish regulation EH-80 for a preliminary design and making all the verifications to certify its safety according to Eurocode 7 relative to the soil resistance and Eurocode 2 relative to internal stability of the footing, making all the necessary alterations to the preliminary design.

1.4. CONTRIBUTIONS OF THE THESIS

As the result of the accomplishment of all tasks, this document provides the following contributions:

- Show in what way the geotechnical differences between Portugal and Lithuania influence the foundations design in each country;
- Expose the typology of the roofs of industrial buildings motivated by the presence of snow and the definition of the minimum depth for the foundations attending to frost heave, showing the increment of the actions on the footings caused by these two factors;
- Establishment of a calculating sequence, using EH-80, Eurocodes 2 and 7, to the design of the footings of an industrial building;
- Comparison of the proximity of results between the Adjusted Elasticity Method of EC7 and Classic Elasticity Theory.

1.5. ORGANIZATION OF THE THESIS

In response to the present objectives of the thesis, this document is organized in 8 different chapters as follows:

Chapter 2 provides an overview of all concepts that will be dealt with in the text. It is divided in four different parts. In the first the geotechnical conditions of the city of Porto and the city of Kaunas are described, with a short text locating the cities demographically and geographically and explaining how the type of soil can influence the type of foundations to use. The second part of this chapter concentrates on the action of the weather in Lithuania and in what way it influences the construction, referring the frost action, the snow and the wind. The third and the fourth parts of the chapter deal with the design regulations used and with the Finite Element Method, respectively.

Chapter 3 describes the project characteristics, exposing the type of the building in study and the type of soil where it will be built. According to Lithuanian regulation it is estimated in this chapter the depth of the frost line, indispensable to know at which depth the building can be founded.

Chapter 4 consists in a preliminary design of the footing according to the soil characteristics obtained by one soil test and the loads transmitted to the base of the column due to the weight of all superstructure increased by the weight of the snow that may lie on the roof. These preliminary dimensions will be tested according to the EC2 for simple shear and punching shear.

Chapter 5 examines the question of the bearing capacity of the soil, testing if the dimensions chosen for the foundation are suitable to the ground resistance with the pressure applied on it.

Chapter 6 discusses the legitimacy of the foundation dimensions against the settlements that it will suffer during the implantation and life of the building. Two different methodologies are used to estimate settlements.

Chapter 7 covers the design of the bending reinforcement of the foundation, ensuring that this element can resist to the bending moments created in the sides of the column.

Chapter 8 concludes this text with a general appreciation of all work developed and with the final considerations about all acquired concepts, methodologies and one evaluation of all results obtained.

2

BACKGROUND

This chapter contains all the background necessary for developing the primordial objective of this thesis, the design of the footing of an industrial building.

In section 2.1 will be depicted the geological conditions of Kaunas and Porto, defining all the technical terminology and showing the principal differences in foundation design between these two countries related with their discrepancy in conditions.

In section 2.2 will be described the weather conditions of Lithuania, pointing out the more influent factors to the construction typology of the country.

Section 2.3 introduces the Eurocodes 2 and 7, giving particular attention to the sections that will be used in the footing design and finally section 2.4 provides an introduction to the Finite Element Method, describing its fundamentals, goals and its connections with computer software, describing in detail the software that will be used in this work.

2.1. GEOTECHNICAL AND GEOLOGICAL CONDITIONS

2.1.1. PORTO

The city of Porto (Oporto in English), located in the north of Portugal on the right bank of the Douro river, is the second most important city of the country and one of the oldest in Europe.

This city, with about 240,000 inhabitants, is the center of the Greater Metropolitan Area of Porto (with about 2,0 million inhabitants) and the historical area of the city (Figure 2.1), because of its architectonic and historic assets, was declared World Heritage Site by UNESCO in 1996.



Fig.2.1 – City of Porto (New York Times)

In what concerns the Geotechnical and Geological conditions, Porto is located essentially in the Centro-Iberic Zone in the limit between this zone and Ossa Morena Zone. These two zones are separated by the Porto-Coimbra-Tomar shear zone that passes near the Douro River mouth (Nevogilde). This shear zone permits the intrusion of granitic rocks in metamorphic and sedimentary ones.

So, in Porto the geological formations are of three different types of lithology:

- Sedimentary (recent mantle deposits);
- Metamorphic (mica schist, gneiss and migmatitis of the “Schist-Greywackey” – Douro Group);
- Igneous (granitic rocks named “Porto Granites”).

These last ones are the most common in this area, so normally residual granitic soils will be found when ground investigation is done before a construction.

Figure 2.2 shows an example of a soil profile in the City of Porto.

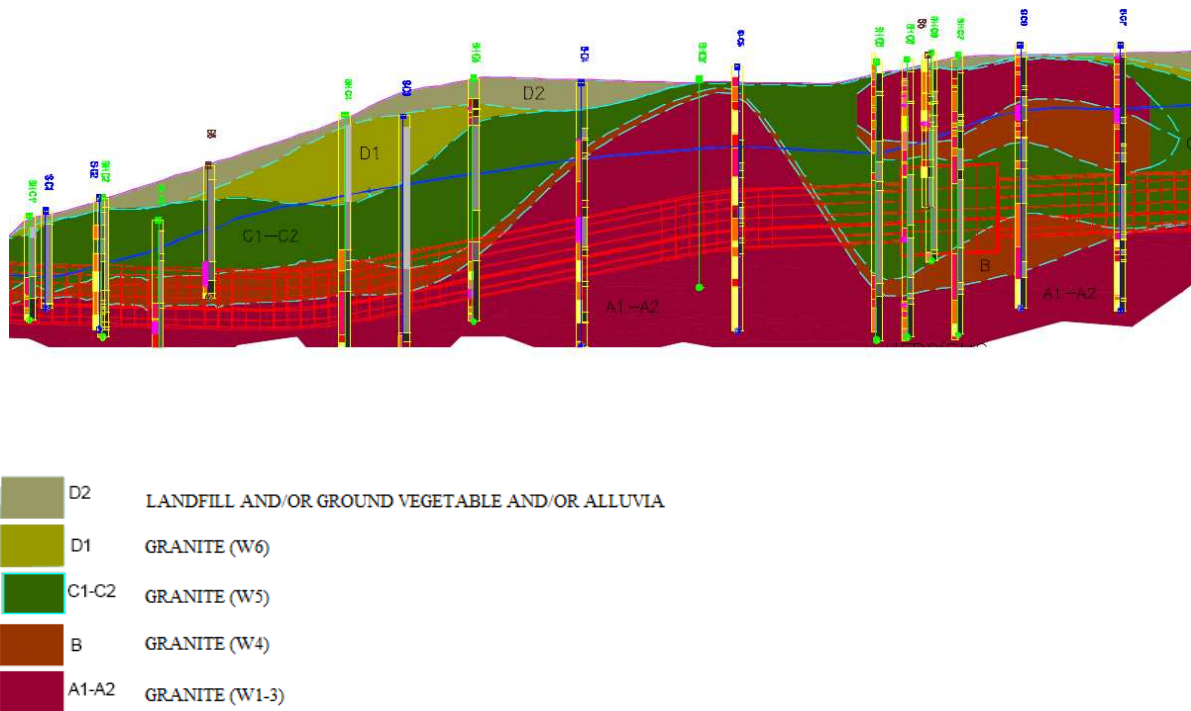


Fig.2.2 – Geotechnical zoning of a soil in city of Porto (Metro of Porto Construction)

2.1.2. KAUNAS

The city of Kaunas, situated in the confluence of the two largest Lithuanian rivers, the Nemunas and the Neris, and near the biggest lake of the country, is the second city in the country.

This city, with about 360,000 inhabitants, is the center of Kaunas country and due to its location it has been for many years the primordial cultural and business center. It was the capital of the country during 20 years following the occupation of Vilnius by Poland in 1920.

In the Second World War, Poland was divided between Germany and the Soviet Union, and so, Vilnius became Lithuanian and the capital of the country again. In spite of that Lithuania was invaded by the soviets in the same year.

Despite the fact that the city was devastated by the Second World War and many buildings and signs of Lithuanians independence were demolished during soviet occupation, the city did not lose its identity nor most of its architectonic heritage (Figure 2.3).



Fig.2.3 – Photos of Kaunas - a) Vilnius Street (Kauno Aerouostas Website); b) Laives Avenue (Seventh European Dependable Conference); c) Aleksotas Bridge (Seventh European Dependable Conference).

The city is founded in layered soils of sand and clay, rock being only at depths around a hundred meters.

With the phreatic level rounding two meters depth, because of the rigorous winters of this zone these soils have the particularity of increasing their volume in winter when the water contained in them freezes.

This phenomenon will be exposed later and its importance will be shown to the design of building foundations.

Figure 2.4 illustrates an example of the soil that can be found in this city, the same soil that will be used later in the project of the foundation of an industrial building. The line in blue represents the phreatic level in the soil.



Fig.2.4 – Generic soil in the city of Kaunas

2.1.3 RELATION OF THE TYPE OF FOUNDATIONS WITH GEOTECHNICAL CONDITIONS

There is an inevitable connection between geological conditions and foundation design and construction. If, for their considerable size, dams, bridges and transportation structures seem to induce more difficulties to Engineers, the design of a building can cause as many problems or even more, especially in what concerns foundation design.

To design the foundations of a structure it is necessary to follow three operations:

- To determine the nature of the foundation bed that will act as support;
- To calculate the loads that the structure will transmit to the foundation and this one to the soil;
- To design the foundation structure to fit the results of the two previous operations, resisting to the loads and enabling the soil to also resist to them.

In fact, the geological and geotechnical conditions have a big influence on foundation design. As to the geological conditions the primordial factor will be the depth of the bedrock. According to this depth, the foundation design characteristics may change, depending if the bedrock lies at the ground surface or so close that the building can be founded upon it. The bedrock may also occur at a depth below the ground surface that still makes possible the transfer of loads to it. Alternatively the bedrock may be so far from the surface that the building will have to be founded in weaker geological material.

In the first and third cases shallow foundations may be used but while in the first it is not necessary to have big concern with the design, in the third, foundations in soil, adequate subsurface investigations are imperative, being necessary an exploratory work until a depth at least twice the width of the structure. These foundations in soil, in which the geotechnical conditions are of extreme importance, are the most common in Engineering.

In the second case, with bedrock within reach, normally piles are used to transmit the loads of the structure to the hard rock (deep foundations). These piles can be in concrete (with the preoccupation of high sulfate content in the groundwater), in steel (but the possibility of corrosion has to be studied) or in wood (accounting for the fact that wood will deteriorate under alternating wet and dry conditions, so variations of groundwater level cannot occur).

Given those three different situations and despite the considerable differences of geotechnical conditions, both in Kaunas as in Porto the use of shallow foundations is very common.

With reference to the three cases of bedrock depth, the geotechnical conditions of the city of Porto are closer to the first situation, despite the fact that in some parts of the region the granitic formations closer to the surface are much too fragmented, while the conditions of Kaunas can be integrated in the third case and so, as it was seen before, in both cases the shallow foundations are the most recommended.

It must be said that is a simplification to think that these two cities only present this type of geological conditions. However, these ones are the most common from what can be gathered from the technical literature.

In spite of shallow foundation being the most used type of infrastructure for buildings in both cities, the soils of Porto present better resistance characteristics and so the design of this element can be done in a more favourable environment in terms of safety.

2.2. WEATHER CONDITIONS

2.2.1. FROST ACTION

In the regions of the globe where the winter is more rigorous, the variation of temperature in the soil water becomes a factor to take into account in the engineer daily work. In every site it is necessary to know the alterations in the soil created by the freeze and thaw of the water contained in it, since the variation of volume and of bearing capacity can be very prejudicial to the constructions. To explain the influence of frost action in Civil Engineering works, it is necessary to introduce three concepts, frost heave, thaw weakening and permafrost.

2.2.1.1 FROST HEAVE

Frost heave is a phenomenon that only occurs in places where the temperature in winter persists for some time too much below 0 °C. The temperature in the soil increases with depth, so when the surface is at 0 °C all the soil is above this temperature, but when the surface is at a much lower temperature a thick layer of soil can be below zero. When this happens the water in the soil freezes and the ground surface rises because of the water volume increase of about 9%. With this increase of water volume all the soil above freezing level will also increase in volume up to 5%, depending on the void ratio.

Frost heave occurs primarily in soils containing fine particles. Clean sands and gravels are non-frost susceptible. In 1932, Casagrande proposed the following widely known rule-of-thumb criterion for identifying potentially frost susceptible soils:

"Under natural freezing conditions and with sufficient water supply one should expect considerable ice segregation in non-uniform soils containing more than 3% of grains smaller than 0.02 mm, and in very uniform soils containing more than 10% smaller than 0.02 mm. No ice segregation was observed in soils containing less than 1% of grains smaller than 0.02 mm, even if the groundwater level is as high as the frost line."

With prolonged low temperature in soils with sufficient saturation much greater volume increases can occur with the formation of ice lenses. When some water in the voids freezes some moisture deficiency is created and capillary tension in areas next to ice crystals. This tension will provoke the suction of water upwards from below the frost line, provoking its freezing too and consequently the increase of the volume of frozen water, the ice lenses (Figure 2.5). Such ice lenses will increase in size with prolonged low temperatures creating in this way the frost heaves due to the vertical upwards pressure that they exert.

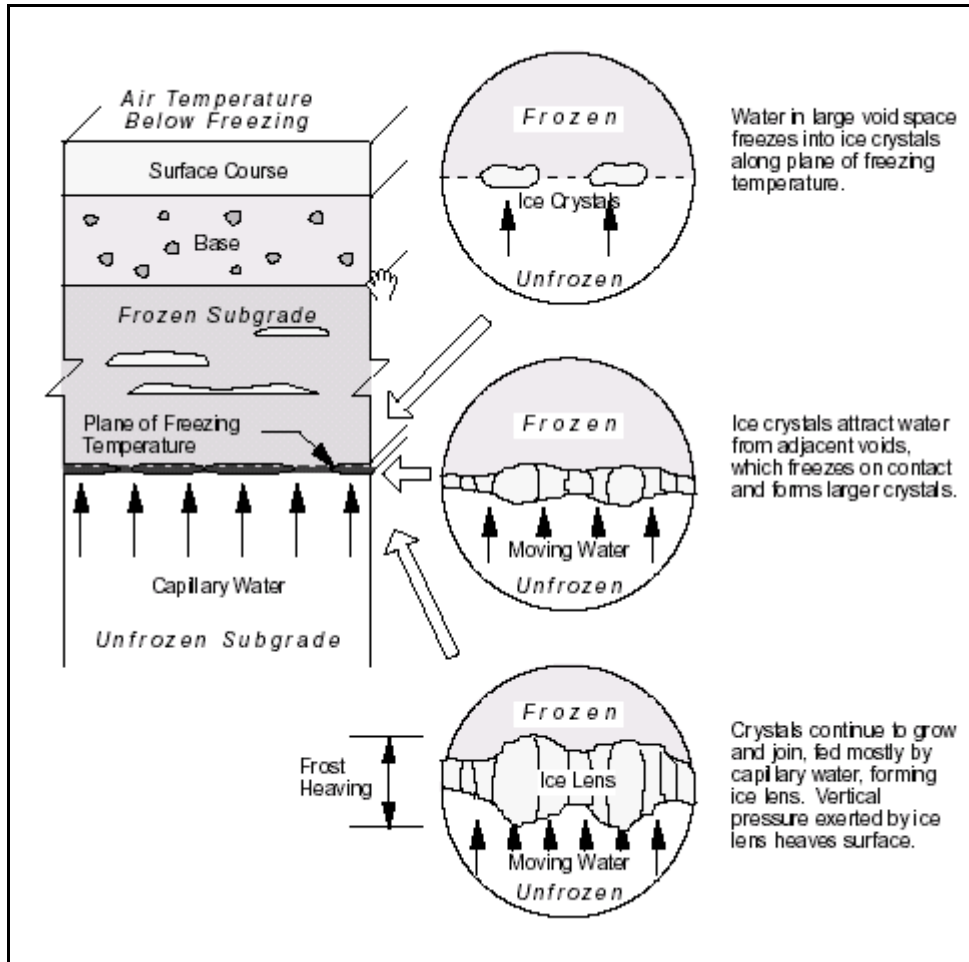


Fig.2.5 – Formation of ice lenses (WAPA Asphalt Pavement Guide)

The depth of the frost line may change with region and country but normally will be below one meter.

The increase of the soil volume is very prejudicial to constructions, so this phenomenon has to be prevented in the projects. In the case of a building it is enough to place the footings below the frost line, but for a highway pavement or a railway line other techniques have to be employed directly in the soil. Some of these techniques are listed below:

- Remove and replace the frost-susceptible soil by one free of silt or clay;
- Lower the phreatic level by means of drains or ditches;
- Use impervious blankets of plastic or asphalts cements (more costly).

In Figure 2.6 it is shown what can happen to a pavement if this phenomenon is not prevented.

But frost heave is not the only enemy of engineering works. Associated to the frost action also is thaw weakening.



Fig.2.6 – Cracks on a pavement caused by frost heave (WAPA Asphalt Pavement Guide)

2.2.1.2 THAW WEAKENING

With the beginning of spring temperature begins to rise and consequently the ice contained in the subgrade melts. This phenomenon called thaw weakening provokes the weakening of subgrade soil and the loss of its bearing capacity, because the water created by the melting of the ice cannot be drained fast enough.

Normally the thaw develops from the surface downward and thus the melted ice is trapped in the bottom by the ice that has not melted yet, turning the surface and the laterals into the only ways that water can take.

As in the case of frost heave this phenomenon can be controlled in order not to affect the constructions by placing the footings of the buildings beneath the frost line but there are some regions, the coldest in the planet, where the problems of frost action are even more serious and harder to control: the permafrost regions.

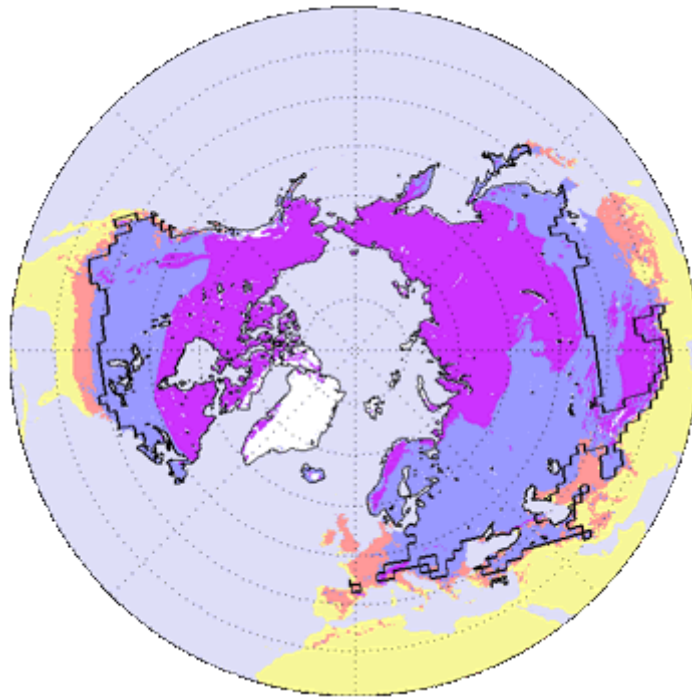
2.2.1.3 PERMAFROST

Permafrost is the abbreviation of permanent frost and defines the terrain with frozen soil or rocks for two or more years. It can be found in the Arctic Regions, covering about one-third of Russia, one-half of Canada and most of Alaska (Figure 2.7) and in mountain regions, the alpine permafrost (Andes mountain range and Central Asia region).

The slightest temperature variations can provoke thawing, thus the quantity of water contained in the soil as ice is very variable and of delicate stability.

This layer is typically 0.4 to 6 meters thick, but in the areas of continuous permafrost and harsh winters its depth can reach 1493 meters.

Today it is estimated that permafrost covers an area of 10.5 millions of km² but it is expected that with global warming this area will have a huge reduction to around 1 million km² in 2100.



LEGEND:

- Permafrost
- Soil frozen more than 15 days/year
- Soil frozen less than 15 days/year
- Limit of maximum extension of soil covered with snow

Fig.2.7- Regions with permafrost (Image from Wikipedia)

As said before, the permafrost layer is very unstable and its thickness is changeable. So it is very delicate to construct in such zones, because permafrost can form an extremely strong and stable foundation material if in the frozen state, but if it thaws, the soil will become weak and the foundations built on it will suffer huge differential or total settlements (Figure 2.8).

To construct in these regions it is necessary to recognize the importance of the frozen ground condition, to search for better-drained sands and gravels in the area and to try to minimize temperature fluctuations associated with the construction operations.

It can also be used wood piles until the base of the layer, above a gravel fill to protect the permafrost or even to introduce thermal tubes in the terrain to cool the ice and retard or prevent its thawing.



Fig.2.8 - Damage on a building provoked by the thawing of permafrost (Journal of Geocryology)

2.2.2. SNOW

One factor to be taken into account in the Eastern part of Europe is the frequency and intensity of snow. The roofs of the buildings have to be prepared to support the weight of all the snow that can be accumulated in the coldest months of winter and so their structures have to be more complex and heavy, principally in buildings with a large roof area. In simple buildings like family houses it is normally enough to increase the roof inclination, no additional reinforcement of the structure being thus needed, which saves a lot of money.

While in Portugal the low weight of the metallic roof of industrial buildings can be almost neglected in the calculation of loads carried by the columns and foundations, because this weight is favorable against the effect of lifting of the roof by the wind, in Kaunas the weight of the roof has to be calculated and aggravated by the weight of the snow that it will support.

In Lithuania there are two different zones of snow intensity, Kaunas being situated in the one of lowest intensity. In the next figure is showed the division of these zones, (figure 2.9).



Fig.2.9- Lithuania snow load areas. Image of Statybus Techniniai Reglamentai (Construction Technical Regulations of Lithuania)

According to the Statybus Techniniai Reglamentai (Construction Technical Regulations of Lithuania) in area I the buildings will be subject to a load of 1.2 kN/m^2 and in area II to 1.6 kN/m^2 .

These loads are affected by a coefficient μ which depends on the shape of the roof. The tables that show the distribution and values of this coefficient are presented in Annex1 of this work.

2.2.3. WIND

The same document (Statybus Techniniai Reglamentai) gives the reference wind velocity in Lithuania, being higher on the coast and lower inland. Figure 2.10 shows the three different wind areas, the first one having a reference wind velocity of 24 m/s , and the second and third, 28 and 32 m/s , respectively. These values and the distribution of the wind action on buildings can be consulted in the same regulation, an extract of which is presented in Annex 2.



Fig.2.10- Lithuania wind speed reference areas. Image of Statybus Techniniai Reglamentai (Construction Technical Regulations of Lithuania)

2.3. FORMULATIONS

This document comprehends the design of a shallow foundation of one industrial building constructed in Kaunas.

The development of this design is done following the formulations presented in two different Civil Engineering Eurocodes, Eurocodes 2 and 7.

The structural Eurocodes have the goal of standardizing structural projects in Europe ensuring the trustworthiness of Constructions Project design.

With the objective of eliminating technical obstacles and harmonizing technical specifications, the Commission of the European Community launched an action programme in the field of construction, which aims at standardizing in all countries represented in this Community the technical rules for the design of construction works, serving in the first stage as an alternative to the national regulations of each country and later replacing them completely.

2.3.1. EUROCODE 7

Eurocode 7 regulates the Geotechnical project and so it is used in this work in the design of the footing dimensions, certifying that the soil below it will resist the applied foundation loads and that settlements will not occur that can provoke damage in the construction.

The design of spread foundations is described in Chapter 6 of this Eurocode being the bearing capacity and the soil settlement evaluated with the help of Annexes D, E, F and G, the last one for footings founded on rock.

2.3.2. EUROCODE 2

Eurocode 2 is one of the parts of the Structural Eurocode Programme that regulates the design of concrete structures. During this work, this document will be used in the design of the concrete footing, being used formulations to guarantee that there will not be any risk of collapse by insufficient resistance of the concrete structure and its steel reinforcement.

This work will focus on chapter 6 of this Eurocode that comprehends the Ultimate Limit States for Concrete Structures, proceeding to the evaluation of shear and punching shear resistance of the footing without extra reinforcement steel.

2.4. FINITE ELEMENT METHOD (FEM)

In Engineering, as in other sciences, it is necessary to represent the phenomena that are around us. That it is done, intuitively, by dividing these complex problems in parcels, to study separately each part and then rebuild them in one again.

So the problems and situations that occur in nature can be described by models, mathematical or experimental ones, with a finite number of parameters. With the advent of high-speed electronic digital computers, numerical methods capable of solving these models were developed exponentially.

One of these numerical methods, which is applied in numerous areas, is the Finite Element Method (FEM). The expression finite element was used in the first time by Clough in 1960 and after that this method, originated in the field of structural analysis, was widely developed and applied to other objectives, namely in the areas of soil mechanics, mechanical and aeronautical engineering, but because of its adaptability and easy application, this method is today accepted in many more areas.

3

PROJECT CHARACTERISTICS

3.1. BUILDING CHARACTERISTICS

In this work it is intended to design the foundation of one column of an industrial building to be constructed in the city of Kaunas in Lithuania.

In the next figure is presented the plan of the building under study (Figure 3.1).

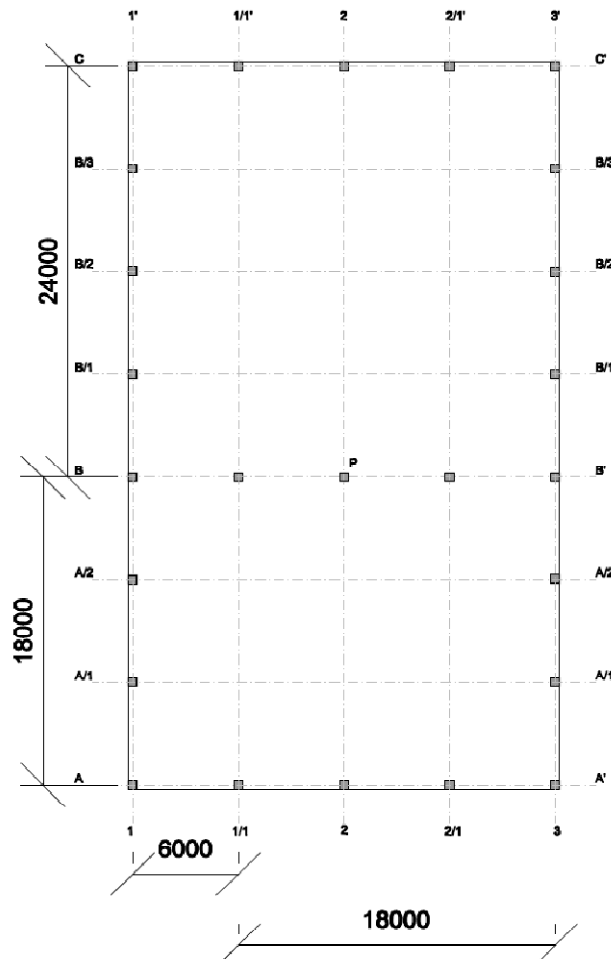


Fig.3.1- Plan of the industrial building (dimensions in mm)

As the principal objective of this work is the study of foundation design, the pavilion will be designed only with its fundamental elements: the columns, walls, pavement and roof.

With square columns of 0.5 meters side, value proposed in the project, the roof will be designed based on Lithuanian and Russian tables and regulations for industrial building design. (TK6-1 – Tables of Typical Structural Buildings in Lithuania).

So, according to the plan and that document the roof will be composed of arches with 24 and 18 meters span. These arches will have to support the roof of the building when loaded with snow.

So following the TK6-1 it was possible to draw the sections of the building in the transversal direction (figure 3.2), (only one half being represented due to symmetry) and in the longitudinal one (figure 3.3). The same sections are presented in Annex 3 with all the dimensions included.

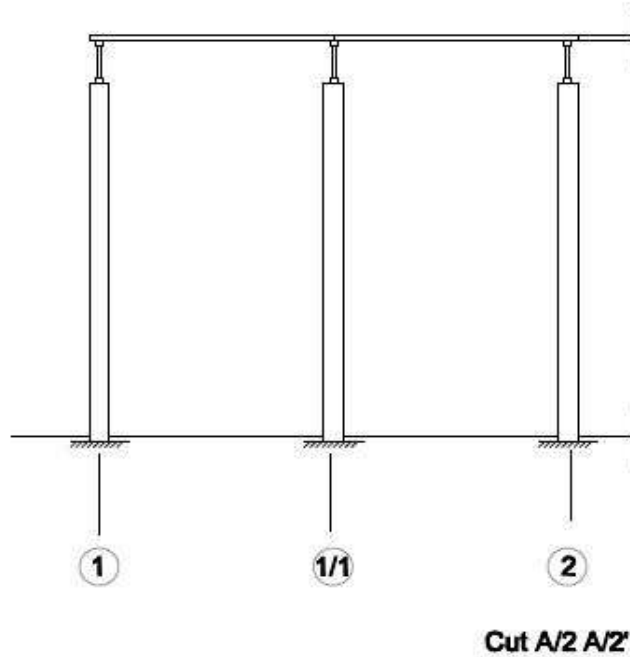


Fig.3.2- Transversal cut of the building A/2-A/2'

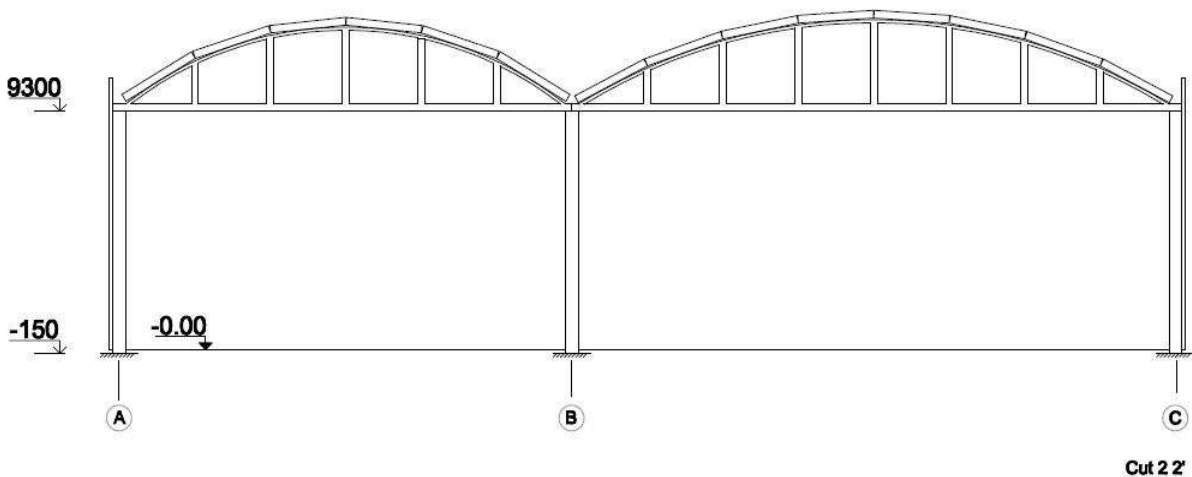


Fig.3.3- Longitudinal cut of the building 2-2' (dimensions in mm)

The columns of the building are 9.3 meters long, partly under the soil surface. The column length buried in the soil will be calculated in 3.2.2 according to the depth of the frost line.

3.2. GEOTECHNICAL CONDITIONS

Table 3.1 was constructed with the results of ground investigation. It contains the depth of the different soil layers where the building will be founded (already presented in figure 2.4), as well as their geotechnical characteristics.

Table.3.1- Soil layers and estimated geotechnical indicators

Eng. Geological Layer No.	Soil Description	Consistency or Density	Thickness (m)	Depth (m)	Weight Density, γ kN/m ³	Cohesion, c (kPa)	Internal Friction Angle, ϕ (°)	Deformation Modulus, E (MPa)	Calc. Soil strength, R_0 (kPa)
1	Concrete	-	0.2	-0.2	23.0	-	-	-	-
10	Soil	-	0.1	-0.3	13.0	-	-	-	-
2	Grey Soil	Medium Density	0.3	-0.6	16.9	0	32	12.0	150
3	Poured Soil	Hard Plastic	0.1	-0.7	18.0	-	-	11.9	170
4	Stripped Clay	Hard Plastic	0.2	-0.9	20.5	23	21	18.0	180
4a	Layered Loam	Soft Plastic	0.3	-1.2	21.0	12	12	6.3	90
5	Loam, Layered	Hard Plastic	0.6	-1.8	21.9	18	25	15.0	210
6	Moraine Loam	Semi-Hard	0.8	-2.6	23.2	84	27	27.0	270
7	Moraine Loam	Hard	0.4	-3.0	23.3	84	27	64.0	770
8	Fine Sand	Dense	1.0	-4.0	20.7	4	37	68.0	1100

3.2.1. – FROST LINE

According to Lithuanian regulations and formulations the frost line can be calculated by the following equation:

$$d_t = k_h \cdot d_{fn} \quad (3.1)$$

where d_t = frost line depth (meters), k_h = thermal coefficient depending on the building temperature
 d_{fn} = characteristic ice depth.

The value of coefficient k_h fluctuates between 0.5 and 0.8 according to the temperature of the buildings. In this project it will be arbitrated a temperature of 10 °C that corresponds to $k_h = 0,7$.

The characteristic ice depth (d_{fn}) can be calculated by the following equation:

$$d_{fn} = d_0 \cdot \sqrt{M_t} \quad (3.2)$$

where d_0 = coefficient related with the type of the soil, M_t = coefficient depending on the region where the building will be founded.

For clay d_0 takes the value of 0.23 and for sand 0.28 and M_t can vary between 8.4 and 19.2. M_t is the absolute value of the atmospheric temperature for the coldest month of the place in question. Considering the city of Kaunas a value of 15 corresponds to a normal year.

The soil in this study is mainly composed by clay and loam, but there are some layers of sand. As the frost line normally has a depth below but close to one meter, it is considered the layer 4 composed by clay for this calculation and so the value of 0.28 is used for d_0 and thus, introducing these values in the equations it is obtained a value of 0.759 m for the frost line depth.

The following table contains all the values introduced in the formulas as well as the values that were obtained from them.

Table 3.2 – Frost line depth

K_h	d_0	M_t	d_{fn}	d_t (m)
0.7	0.28	15	0.891	0.759

Given the value obtained for the frost line depth, the footing can only be founded beneath 0.759 meters of depth, but as can be seen in table 3.1 at the depth of 0,9 meters there is a soft plastic layer that does not have good characteristic of resistance to support the building. In these conditions, the foundation will be based in the next layer of better characteristics (Layer 5 - Loam, layered, brown – Hard Plastic) at a depth of 1.2 meters (figure 3.4).

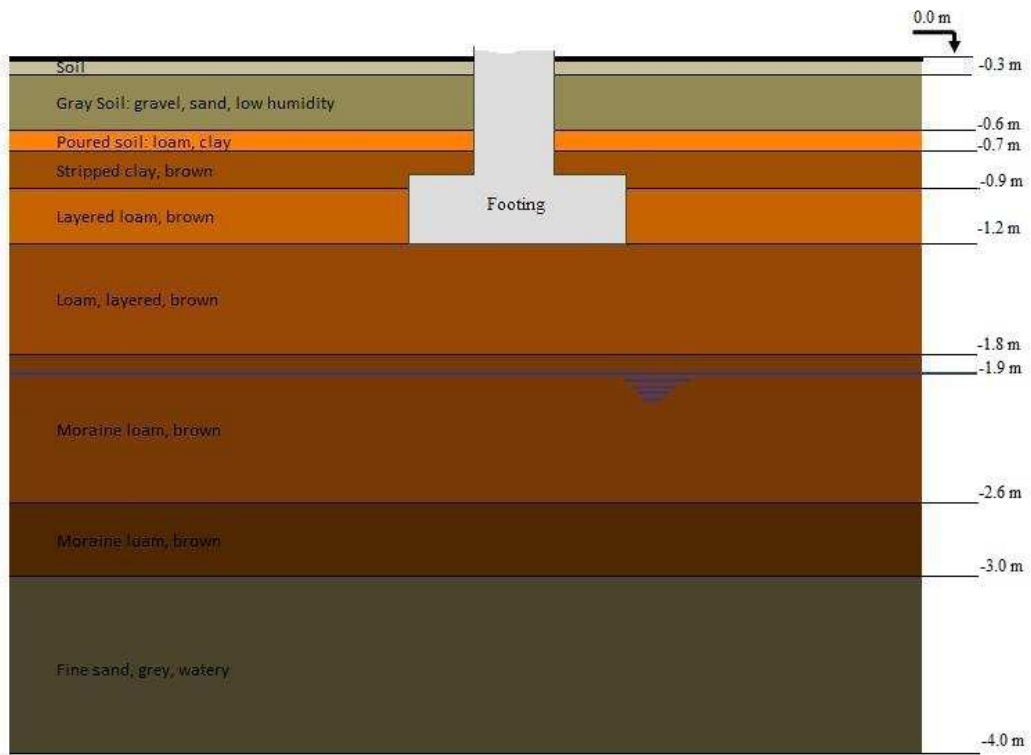


Fig. 3.4 – Footing location in the soil

4

FOOTING DIMENSIONS

4.1. LOADS AT THE BASE OF COLUMN

To give dimensions to the footing it is necessary to know to what loads it will be subject. So, in this sub-chapter will be calculated all the weights of the elements that will be supported by the foundation under study.

To determine the loads at the base of the column it is necessary to calculate:

- Self weight of the Column;
- Self weight of the Roof;
- Snow load.

To know the self weight of the roof and of the snow that will be directed to the central column it is necessary to determine the area of the roof that will be supported by the footing in question.

4.1.1. – INFLUENCE AREA

The area of influence is the area of the roof that will be supported by the column P and so, the part of the totality of roof weight that will load the foundation to be designed.

As it is possible to see on the plan and sections presented in chapter 3, the arches distribute all the roof weight to columns that support the extremity of the arches along the lines A-A', B-B' and C-C'.

In the other direction the weight will be distributed equally by all the columns and so the column in line B-B' supports twice the weight of the columns positioned in A-A' and C-C'. In the other directions the columns in lines 1/1-1/1', 2-2' and 2/2-2/2' support twice the weight relative to the lines of the walls.

With this in mind, in the next three figures is presented the plan and sections of the building with the influence area of column P in red, with dimensions in plan of 6 by 21 meters. In figure 4.3 is represented only half of the structure and of the influence area due to symmetry.

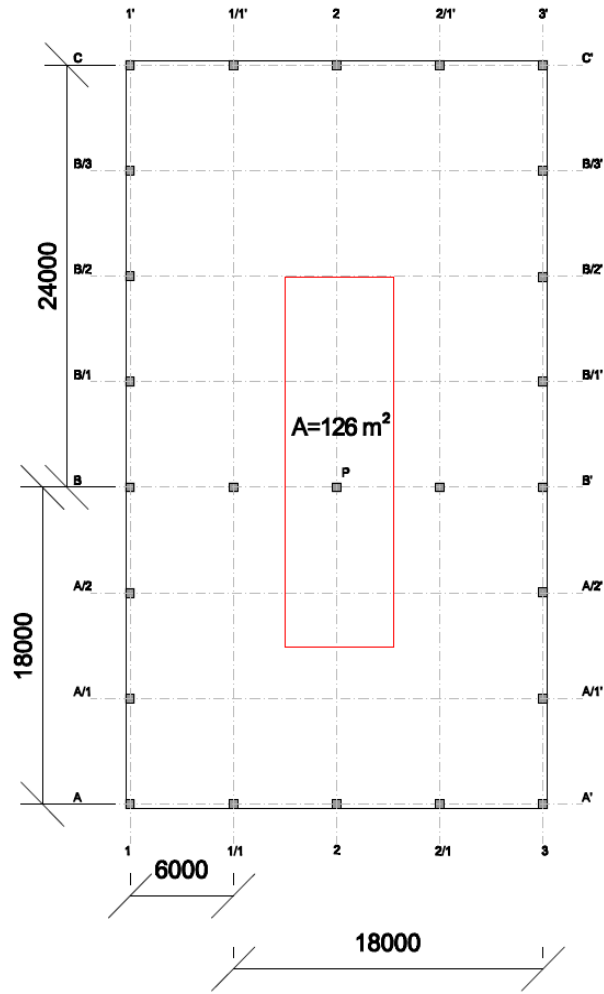


Fig.4.1- Influence area of column P (dimensions in mm)

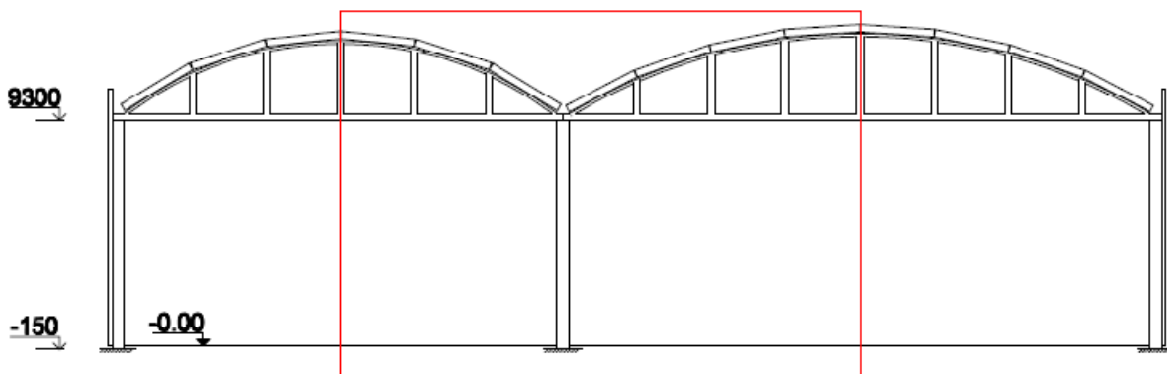


Fig.4.2- Longitudinal section 2-2' with influence area of column P (dimensions in mm)

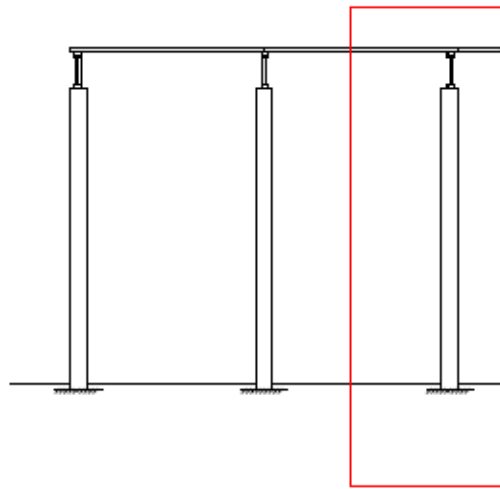


Fig.4.3- Transversal section A/2-A/2' with influence area of column P

4.1.2- SELF WEIGHT OF COLUMN AND ROOF

As it was said before, the construction of industrial buildings in Lithuania is regulated by the Russian tables. Those tables give us the weights of every element that composes the structure and so consulting them the values of table 4.1 were obtained, where only the column was calculated by the following formula:

$$W_{pillar} = V_{pillar} \cdot \gamma_c \quad (4.1)$$

where W_{pillar} =self weight of the column, V_{pillar} =volume of the column and γ_c =unit weight of concrete with $\gamma_c = 25 \text{ kN/m}^3$.

Table 4.1 – Self weight of structural elements

Element	Self weight (kN)
Column	58.125
Reinforced Concrete	
Arch (18m span)	77
Reinforced Concrete	
Arch (24m span)	117
Reinforced Concrete	
Plates (1 unit)	26.5

Consulting figures 4.1, 4.2 and 4.3 it is possible to see which part of these elements is supported by the column in question. As the plates have 3x6x0.3 meters, to the base of the column will be transmitted the weight of 7 ($2 \times \frac{7}{2}$) plates and half of the weight of both arches.

So, the column, the concrete arches, with 24 meters and 18 meters span, and the reinforced plate loads are 58.125, 58.5, 38.5 and 185.5 (kN), respectively.

4.1.3- WEIGHT OF THE SNOW

In chapter 2 were referred the snow load areas, their limits and values and it was possible to note that, in Kaunas, the snow accounts for a uniform load on roofs of 1.2 kN/m^2 .

In this work was taken the option of using an arched roof and so consulting Annex 1 and considering the influence area defined before it will be possible to calculate the load due to snow on column P.

As in both arches $f/l > 0.1$ (figure 4.4) the snow load on the roof will follow the second variant presented in Annex 1 for this type of roof, but because the influence area comprehends half of both arches there will be no differences from the first variant.

So, with an area of 6×21 meters we will have a force of 151.2 kN in the column due to snow load.

The following figure shows the two variants for distribution of the snow load according to the roof shape in use, where the second one has been used. This picture was taken from Annex 1.

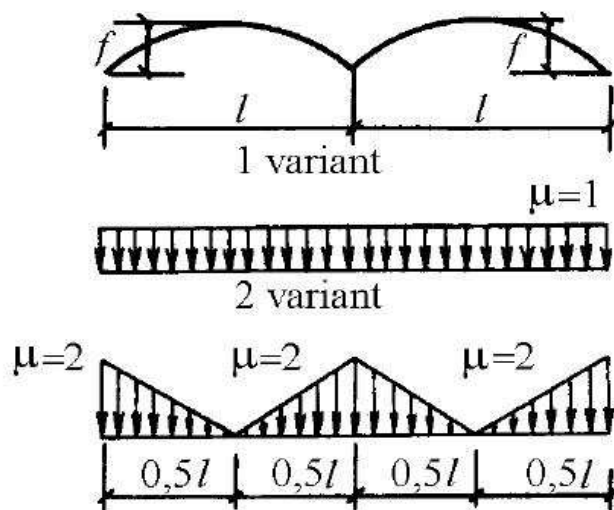


Fig.4.4- Snow load distribution (Statybus Techniniai Reklamantai).

4.1.4- STRENGTH OF THE WIND

The reference wind speed in the area of the project is 24m/s. From this velocity it is necessary to calculate the dynamic pressure that will be applied to the walls and roof. However, as only the central column of the building is being studied, the effect of wind will be despised considering that it will be totally supported by the frame of the building.

4.2. PRELIMINARY DESIGN OF THE FOOTING

The dimensions of the footing will be determined considering the loads at the base of the column. Since there are no horizontal forces or moments a square footing will be designed. So, in the next table are presented the loads that will be used in the dimension of the area of the foundation.

Table 4.2 – Loads at the base of the column

Element	Vertical load (kN)
Column	58.1
Reinforced Concrete Arch (18m span)	38.5
Reinforced Concrete Arch (24m span)	58.5
Reinforced Concrete Plates	185.5
Snow	151.2
Wind	-
Total	471.7

However, taking into account the partial safety factors of EC7, listed in table 4.3, the value of the total vertical load and moments will be increased.

Table 4.3 – Partial safety factors

Combination	γ_G	γ_Q	γ_q	γ_ϕ	$\gamma_{c'}$	γ_{cu}	γ_{Rv}
DA-1 a)	1.35	1.55	1.5	1.0	1.0	1.4	1.0
DA-1 b)	1.0	1.3	1.3	1.25	1.25	1.4	1.0
DA-2	1.35	1.5	1.5	1.0	1.0	1.0	1.4
DA-3	1.35	1.5	1.3	1.25	1.25	1.4	1.0

where, γ_G is applied to the permanent structural loads, called in Lithuania as *Dead Loads*, γ_Q is applied in the variable loads, called in Lithuania as *Live Loads*, γ_q is applied on pressure created by the terrain on the base of the footing, γ_ϕ is applied on the internal friction angle of the soil, $\gamma_{c'}$ is applied on the soil cohesion value, γ_{cu} is applied on the undrained cohesion of the soil and γ_{Rv} is applied on the soil resistance.

So using the equations 4.2 and 4.3 we will obtain the loads at the base of the column for every combination (Table 4.4). As horizontal loads do not exist, moments do not exist either.

$$V_d = \sum \gamma_{G,j} \cdot H_{G_{K,j}} + \sum \gamma_{Q,i} \cdot V_{Q_{K,i}} \quad (4.2)$$

$$H_d = \sum \gamma_{G,j} \cdot H_{G_{K,j}} + \sum \gamma_{Q,i} \cdot H_{Q_{K,i}} \quad (4.3)$$

Table 4.4 – Loads to use in the design of the footing

Combination	V _d (kN)
DA-1 a)	686.64
DA-1 b)	537.19
DA-2	686.64
DA-3	686.64

As the results of the ground investigation provide estimates of the soil strength, those values will be used to pre-design the footing. In the next chapters those values can be adjusted if necessary to comply with the Eurocodes regulation where the resistance of the soil can take values different from the adopted estimate.

Using EC2 and the Spanish Regulation EH-80 will be presented next the equations required for the preliminary design of footings with centered square columns.

So, to determine the area of the foundation:

$$\sigma_{sd} \leq \sigma_{rd} \quad (4.4)$$

$$\frac{N_{sd}}{B_0^2} \leq \sigma_{rd} \Leftrightarrow B_0 = \sqrt{\frac{N_{sd}}{\sigma_{rd}}} \quad (4.5)$$

$$B_x = B_0 + 2 \cdot e_x \quad (4.6)$$

$$B_y = B_0 + 2 \cdot e_y \quad (4.7)$$

$$e_{x,y} = \frac{M_{y,x}}{V} \quad (4.8)$$

where B_x and B_y are the dimensions of the footing, N_{sd} is the vertical load applied to the base of the column plus the weight of the footing, σ_{rd} is the resistance of the soil and $e_{x,y}$ the eccentricity of the load in the directions OX and OY.

Since there are no moments eccentricity does not exist and so the dimensions of the footing, B_x and B_y will be equal ($B_x = B_y = B_0$).

In chapter 3 was presented the soil where the building will be founded and the depth of foundation was defined at 1.2 meters in a layer of loam, layered, brown (Layer 5). This layer, as can be seen in table

3.1, has a strength or resistance (σ_{rd}) of 210 kPa. Affecting the soil resistance with the safety factors of each combination, only in the second one (DA-2) will this resistance be reduced to 150 kPa ($R_{v,d} = R_v/\gamma_{R_v}$). As the footing dimensions are still unknown its weight will be considered as 10% of the vertical load.

So, according with the last equations:

Table 4.5 – Values of B_x and B_y

Combination	B_x (m)	B_y (m)
DA-1 a)	1.90	1.90
DA-1 b)	1.68	1.68
DA-2	2.24	2.24
DA-3	1.90	1.90

To guarantee the internal stability of the footing, it will be designed with the largest dimensions required by the different combinations. So, the foundation will be 2.3x2.3 meters.

Now it is necessary to dimension the height of the footing. This height has to be sufficiently large to resist shear but not exceedingly large that might implicate a waste of money. Thus, some equations will be used to define this dimension and then, with all the dimensions already set, the structure stability against shear and punching will be tested. With this we can guarantee that this element will not need extra steel to prevent those two phenomena.

So based on Spanish regulation EH-80:

$$h/2 \leq a_0 \leq 2h \quad (4.9)$$

where h =height of the footing and a_0 =biggest distance between the face of the column and the edge of the footing (figure 4.5).

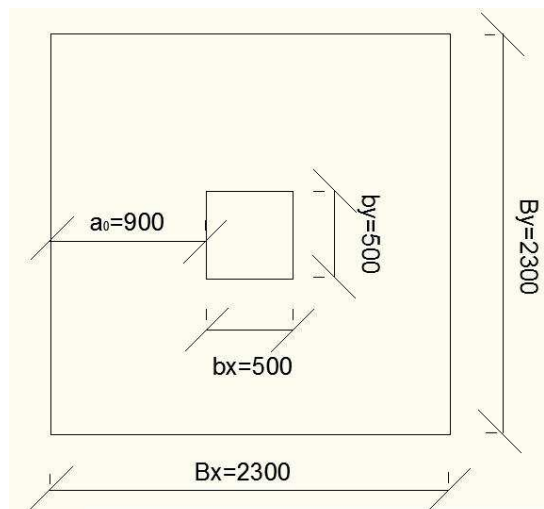


Fig.4.5 – Plan of the footing (dimensions in mm)

As can be seen in the last figure a_0 is the distance between the column and the edge of the foundation and so, with 0.5 meters for the side of the column and the dimensions presented above, $a_0 = 0.90 \text{ m}$.

We now use equation 4.9 considering, as first iteration, the smallest value of h , $h = a_0/2$, a value that will be checked below.

So, taking $h = a_0/2 = 0.45 \text{ m} \Rightarrow 0.5 \text{ m}$, $B_x = B_y = 2.3 \text{ m}$ the verification of the stability of the foundation will now be performed.

4.2.1. – VERIFICATION OF THE COMPRESSED FOOTING AREA

The footing cannot apply tensile stresses to the soil. If the eccentricity of the load is excessive part of the footing base will lift off the ground creating instability in the building and the probability of differential settlements. To prevent this, the dimensions of footing area have to be enough to reduce eccentricity, as can be seen in the following equations taken from the book Hormigón Armado:

General Case,

$$\sigma = \frac{N}{A_{foot}} \pm \frac{M_x}{I_x} \cdot y \pm \frac{M_y}{I_y} \cdot x \quad (4.10)$$

For the corner of the footing,

$$\sigma = \frac{N}{A_{foot.}} \pm \frac{6 \cdot M_x}{B_x \cdot B_y^2} \pm \frac{6 \cdot M_y}{B_y \cdot B_x^2} \quad (4.11)$$

Simplifying,

$$\frac{e_x}{B_x} + \frac{e_y}{B_y} \leq 1/6 \quad \Rightarrow \text{THE BASE IS FULLY COMPRESSED} \quad (4.12)$$

$$\frac{e_x}{B_x} + \frac{e_y}{B_y} > 1/6 \quad \Rightarrow \text{THE BASE IS NOT FULLY COMPRESSED} \quad (4.13)$$

Since in the case study there are no moments and consequently no eccentricities $\frac{e_x}{B_x} + \frac{e_y}{B_y} = 0 \leq 1/6$ and the dimensions were chosen to verify $\sigma_{max} \leq \sigma_{rd}$, the dimensions of the footing in plan are suitable.

4.2.2. – SHEAR AND PUNCHING VERIFICATION

Punching is one of the most serious problems in footings and this phenomenon must be carefully studied but when these elements have an elongated shape the punching is no longer the most problematic situation, being all the concerns redirected to the effects of the shear.

Following the Spanish norm EH80 conjugated with the indications contained in EC2 for the study of these two effects (Shear and Punching) in laminar elements, the suitability of the height of the footing determined in sub-chapter 4.2 will be verified.

To distinction between an elongated and a non elongated footing is based on the relation between the shortest side of the footing and the largest distance between the column face and the edge of the footing in directions OX or OY (figure 4.5). That relation is expressed below:

$$a_0 \geq 1,5 \cdot B \quad \Rightarrow \text{ENLONGATED FOOTING} \quad (4.14)$$

$$a_0 < 1,5 \cdot B \quad \Rightarrow \text{NON ENLONGATED FOOTING} \quad (4.15)$$

In the present case a_0 will be, like in all square footings, smaller than $1,5 \cdot B$ ($0,9 \leq 1,5 \times 2,3$) and therefore it will be only necessary to verify the structure resistance against punching shear.

4.2.2.1. Punching resistance verification

The verification of the punching shear resistance will be performed according to EC2. In agreement with this regulation, this type of shear has to be tested at the face of the column and at the basic control perimeter, u_1 , defined at the distance of $2 \cdot d$ from the column, being d the distance between the bending reinforcement steel and the top of the footing, or in other words, the effective depth of the footing (figures 4.6 and 4.7).

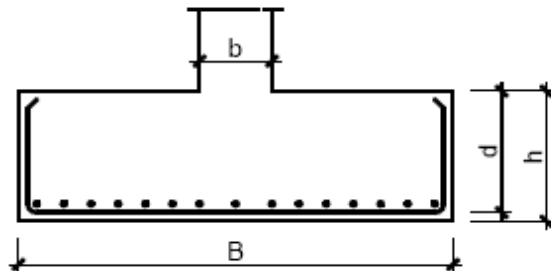


Fig.4.6 – Example of a footing section (Fundações e Estruturas de Suporte, Arêde, António)

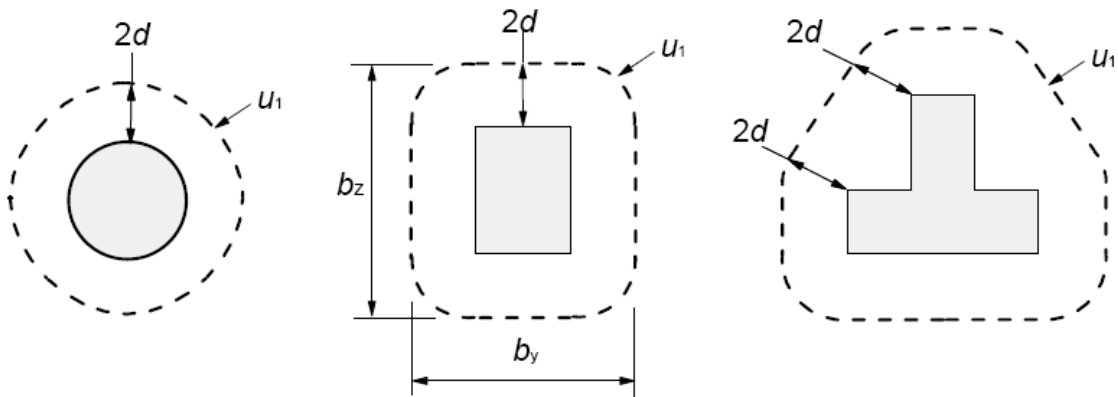


Fig.4.7 – Control perimeter for three different types of columns (EC2)

So, based on EC2, the following conditions have to be satisfied:

in the perimeter of the column,

$$v_{Ed} < v_{Rd,max} \quad (4.16)$$

in the control perimeter,

$$v_{Ed} < v_{Rd,c} \quad (4.17)$$

where, v_{ED} = punching shear stress, $V_{Rd,c}$ = design value of the maximum punching shear resistance of a footing without reinforcement and $V_{Rd,max}$ = design value of the maximum punching shear resistance along the control section considered.

The equations that determine the parameters V_{ED} = are now presented.

for eccentric loads,

$$v_{Ed} = \frac{V_{Ed,red}}{u \cdot d} \cdot \left[1 + k \frac{M_{Ed} \cdot u}{V_{Ed,red} \cdot W} \right] \quad (4.18)$$

And for concentric loads,

$$v_{Ed} = \frac{V_{Ed,red}}{u \cdot d} \quad (4.19)$$

where,

$$V_{Ed,red} = V_{Ed} - \Delta V_{Ed} \quad (4.20)$$

and, u = length of the control perimeter considered, d = mean effective depth of the footing, V_{ED} = shear force applied on the footing, ΔV_{Ed} = net upward force within the control perimeter considered i.e. upward pressure from soil minus self weight of base, k = coefficient dependent on the ratio between the column dimensions, W = parameter that considers the distribution of shear and M_{Ed} = moments applied at the base of the column.

$$\Delta V_{Ed} = \sigma_0 \cdot A_u \quad (4.21)$$

where, σ_0 = pressure in the contact between soil and footing and A_u = area of control.

Looking to figure 4.7 we can define the control perimeter of punching shear and the respective area. Since $h = 0.5$ we have $d = 0.9 \times h = 0.45 \text{ m}$ and so $2 \cdot d = 0.9 \text{ m}$.

$$u = 4 \times 0.5 + 2 \times \pi \times 0.9 = 7.655 \text{ m}$$

$$A_u = 4 \times 0.5 \times 0.9 + \pi \times 0.9^2 = 4.35 \text{ m}^2$$

Applying equation 4.21 to the control perimeter, considering the largest vertical load obtained in the various combinations, v_{Ed} it will be,

$$\Delta V_{Ed} = \frac{686.64}{2.3 \times 2.3} \times 4.35 = 563.94 \text{ kN}$$

$$V_{Ed,red} = 686.64 - 563.94 = 122.70 \text{ kN}$$

$$v_{Ed} = \frac{122.70}{7.66 \times 0.45} = 35.62 \text{ kPa}$$

If the same equation is applied to the column perimeter for the same vertical load,

$$\Delta V_{Ed} = \frac{686.64}{2.3 \times 2.3} \times 0.5^2 = 32.45 \text{ kN}$$

$$V_{Ed,red} = 686.64 - 32.45 = 654.19 \text{ kN}$$

$$v_{Ed} = \frac{654.19}{0.5 \times 4 \times 0.45} = 726.88 \text{ kPa}$$

To verify equations 4.16 and 4.17, we must be calculate $v_{Rd,c}$ and $v_{Rd,max}$, via the application of the following formula:

$$v_{Rd,c} = C_{Rd,c} \cdot k \cdot (100 \cdot \rho \cdot f_{ck})^{1/3} \times 2 d/a \geq v_{min} \times 2d/a \quad (4.22)$$

where,

$$k = 1 + \sqrt{\frac{200}{d}} \leq 2,0 \quad d \text{ in mm} \quad (4.23)$$

$$\rho = \sqrt{\rho_{ly} \cdot \rho_{lz}} \leq 0,02 \quad (4.24)$$

where ρ =percentage of tensile reinforcement steel of the footing being ρ_{ly} and ρ_{lz} the percentages in the directions OY and OZ, a =distance from the periphery of the column to the control perimeter considered, f_{ck} =characteristic compressive cylinder strength of concrete at 28 days (MPa), d was defined before in figure 4.6 and $C_{Rd,c}$ and v_{min} are values that can be found on the National Annex being recommended equations 4.25 and 4.26.

$$C_{Rd,c} = 0,18/\gamma_c \quad (4.25)$$

$$v_{min} = 0,035 \cdot k^{3/2} \cdot f_{ck}^{1/2} \quad (4.26)$$

Next are presented all the steps of the $v_{Rd,c}$ calculation where for γ_c , the partial safety factor for concrete, was utilized a value of 1.5 and for f_{ck} 25 MPa for C25/30 concrete. It must be referred that, in Lithuania, the most used concrete is C20/25.

$$C_{Rd,c} = \frac{0.18}{1.5} = 1.2$$

$$k = 1 + \sqrt{\frac{200}{450}} = 1.6(6) \leq 2.0 \Rightarrow O.K.$$

$$v_{min} = 0.035 \cdot 1.67^{3/2} \cdot 25^{1/2} = 0.38$$

$$v_{min} \times 2d/a = 0.38 \times 2 \times \frac{450}{0.9} = 377 \text{ kPa}$$

As the reinforcement has not yet been designed a value of 0,5% will be considered for ρ . So,
 $v_{Rd,c} = 0,12 \cdot 1,67 \cdot (100 \cdot 0,005 \cdot 25)^{1/3} \times 2 \times 450 / (2 \times 0,45) = 465 \text{ kPa} \geq 377 \text{ kPa} \Rightarrow O. K.$

The value of $v_{Rd,max}$ can be found in the National Annex for each country but it is recommended the value of:

$$v_{Rd,max} = 0.5 \cdot v \cdot f_{cd} \quad (4.27)$$

$$v = 0.6 \left[1 - \frac{f_{ck}}{250} \right] \quad (f_{ck} \text{ in MPa}) \quad (4.28)$$

where, v =strength reduction factor for concrete cracked in shear, f_{cd} =design value of concrete compressive strength= f_{ck}/γ_c .

$$v = 0.6 \left[1 - \frac{25}{250} \right] = 0.54$$

$$v_{Rd,max} = 0.5 \cdot 0.54 \cdot \frac{25000}{1.5} = 4500 \text{ kPa}$$

So, in the column perimeter,

$$v_{Ed} < v_{Rd,max} \Leftrightarrow 726.88 \text{ kPa} < 4500 \text{ kPa} \Rightarrow O. K.$$

And in the control perimeter,

$$v_{Ed} < v_{Rd,c} \Leftrightarrow 35.62 \text{ kPa} < 465 \text{ kPa} \Rightarrow O. K.$$

4.2.2.2. Shear resistance verification

In 4.2.2 was proved that this foundation, given its dimensions, does not require the verification of shear resistance. However in this sub-chapter it is demonstrated how to do this verification when necessary.

In the case of the design of an elongated footing, $a_0 \geq 1.5 \cdot B$, it is necessary to verify shearing on the line defined by the face of the column, figure 4.8.

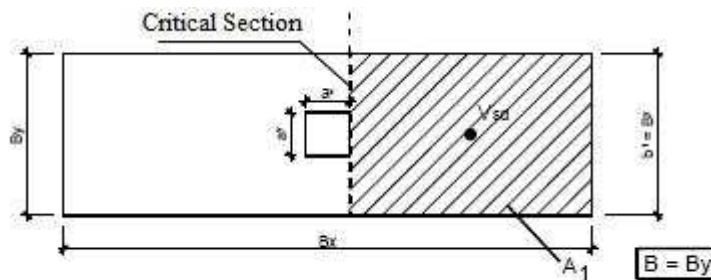


Fig.4.8- Shear critical section (Fundações e Estruturas de Suporte, Arêde, António)

When loaded by the column the footing will transmit that load to the soil and receive the soil reaction. Thus, there will be a downward vertical load at the center of the footing and upward forces on each side of the column, generating the possibility of shear of the footing at the column edges. For this shear not to happen it is fundamental that the following condition be satisfied.

$$V_{Ed} \leq V_{Rd,c} \quad (4.29)$$

$$V_{Ed} = \sigma_{3/4} \cdot A_1 \quad (4.30)$$

where $\sigma_{3/4}$ = stress applied to the footing by the soil at $3/4$ of his length, A_1 = area of the critical shear zone, V_{Ed} = design value of the shear force and $V_{Rd,c}$ = design shear resistance of the member without shear reinforcement.

As there are no moments at the base of the column $\sigma_{3/4} = \sigma_0 = \frac{N_{sd}}{A_{foot.}}$ and so,

$$A_1 = 2.3 \times 0.90 = 2.07 \text{ m}^2$$

$$V_{Ed} = \frac{686.64}{2.3 \times 2.3} \times 2.07 = 268.69 \text{ kN}$$

$$V_{Rd,c} = [C_{Rd,c} \cdot k \cdot (100 \cdot \rho_l \cdot f_{ck})^{1/3} + k_1 \cdot \sigma_{cp}] \cdot b_w \cdot d \quad (4.31.a)$$

with a minimum of,

$$V_{Rd,c} = (v_{min} + k_1 \cdot \sigma_{cp}) \cdot b_w \cdot d \quad (4.31.b)$$

where $C_{Rd,c}$, f_{ck} , k , d and v_{min} are defined in equation 4.22, ρ_l = percentage of tensile reinforcement relative to the concrete effective area, k_1 = value proposed in the EC2 National Annexes, the recommended value being 0.15, σ_{cp} = compressive stress in the concrete from axial load or prestressing and b_w = smallest width of the cross-section in the tensile area (mm).

Thus, being $b_w = 2300 \text{ mm}$, considering $\rho_l = 0.5\%$ as the reinforcement has not yet been defined and given that there are no forces in the direction parallel to the soil surface $\sigma_{cp} = 0$,

$$V_{Rd,c} = [0.12 \cdot 1.67 \cdot (100 \cdot 0.005 \cdot 25)^{1/3} + 0.15 \cdot 0] \cdot 2300 \cdot 0.45 = 481.36 \text{ kN}$$

So

$$V_{Ed} \leq V_{Rd,c} \Leftrightarrow 268.69 \text{ kN} \leq 481.37 \text{ kN} \Rightarrow O.K.$$

In this chapter the footing dimensions have been defined and its internal structural stability checked. In the next chapter will be tested the soil resistance and determined the settlement that the foundation will be suffer.

5

BEARING RESISTANCE CALCULATION

Bearing resistance is the capacity of soil to support loads applied on it and can be calculated using bearing capacity theory. In the case of a shallow footing the bearing capacity is the maximum contact pressure between the foundation and the soil which will not cause shear failure in the soil.

Besides the ultimate bearing capacity, the foundations are commonly designed according to the allowable bearing capacity, i.e., the ultimate bearing capacity divided by a factor of safety.

There are three types of soil failure that limit the bearing capacity: the general, the local and punching shear failure (figures 5.1, 5.2 and 5.3).

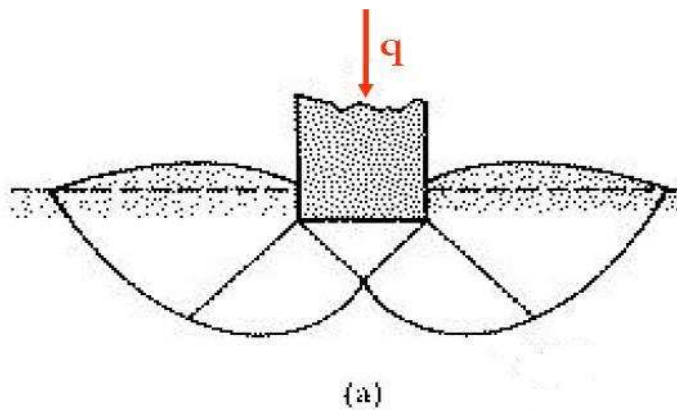


Fig.5.1 – General shear failure (Vesic,1963)

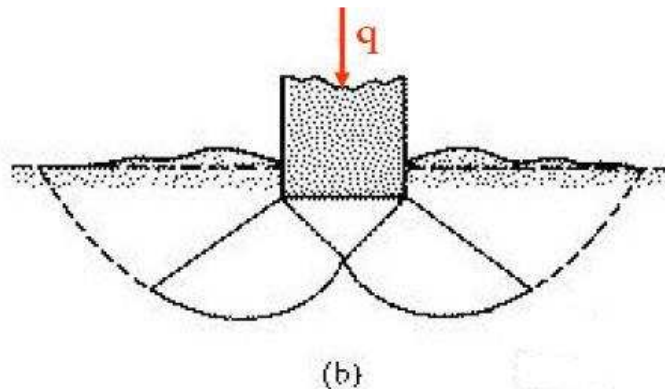


Fig. 5.2 - Local shear failure (Vesic,1963)

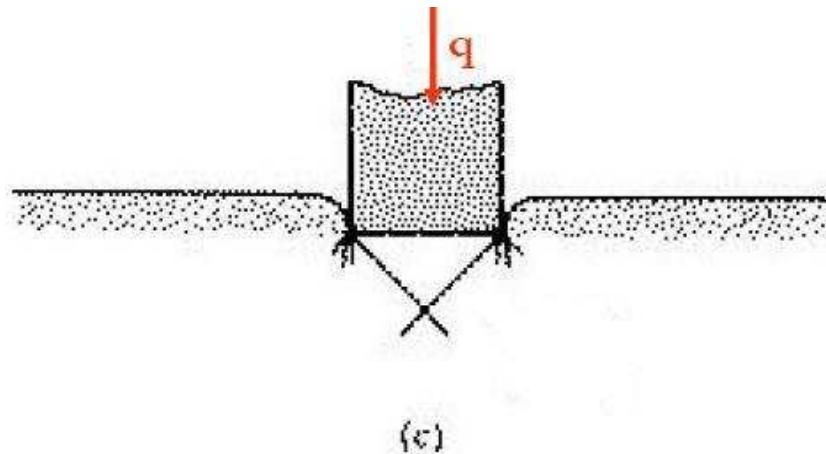


Fig. 5.3 – Punching shear failure (Vesic,1963)

The failure is related to the soil reaching its ultimate limit state, thereby exhausting its capacity to withstand the applied load, experiencing very large vertical settlements and possibly leading all the supported structure to collapse.

When one footing is loaded by one vertical centered load continually increasing in magnitude, there will be one value of the load which causes the rupture of a limited zone of the soil. If the load continues to increase that zone will spread until a failure mechanism fully developed, with failure surfaces reaching the ground surface. This phenomenon can be called general shear of the soil and can be seen in figure 5.4 where $Q_3 > Q_2 > Q_1$.

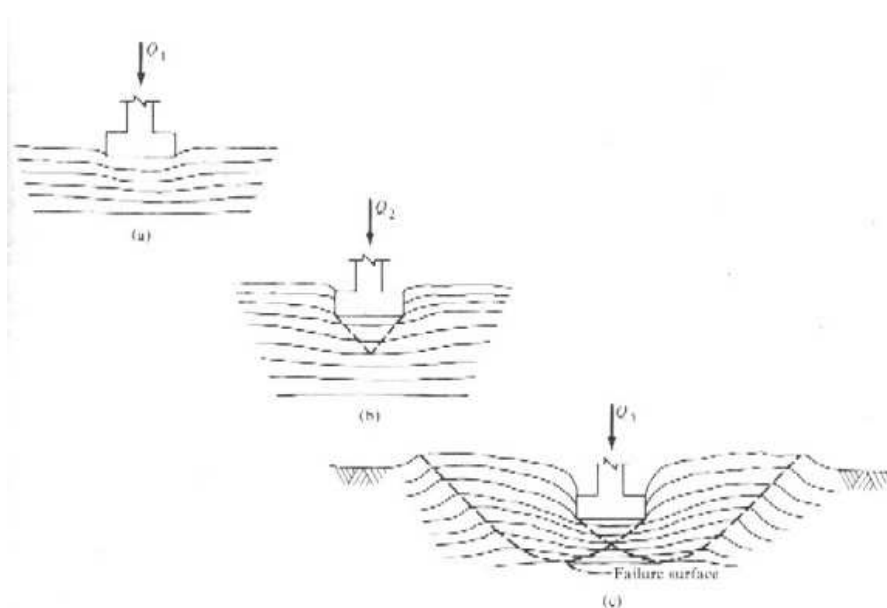


Fig.5.4 – Evolution of the general shear zone of a soil under a footing loaded by a centered and continually increasing vertical load (Bearing Capacity of Foundations – University of Pretoria).

In this chapter will be determined the bearing capacity of the footing under study and compared with the vertical load applied to the footing by the weight of the building and snow. The foundation bearing capacity will be calculated, according to EC7, in two different conditions, drained and undrained, being in both cases equal to the theoretical load of rupture (Q_{ult}), in each condition, divided by the area of the footing base.

$$q_{ult} = \frac{Q_{ult}}{B \cdot L} \quad (5.1)$$

where, B and L are the smaller and larger dimensions of the rectangular footing area, respectively.

So, in the next two sub-chapters all the necessary calculations will be performed with the goal of determining the safety of the structure against the soil shear.

In this document a layered soil is being considered with layers of different characteristics and so different bearing resistance.

In spite of the fact that all parameters used in this study have been obtained with only one geotechnical test, it is assumed that the soil characteristics remain constant in the horizontal direction.

To achieve an approximation of the real bearing capacity of the soil, several cases will be considered, admitting, in each one, the existence of an homogeneous soil with the characteristics of just one of the layers. The real bearing capacity will be a pondered value of those obtained for the different layers having in consideration the importance of each one to the stability of the structure.

Since the footing will be founded at a depth of 1.2 meters, there are four different layers below it of increasing resistance in depth. We shall present here the calculations for the first and less resistant layer, giving for the other layers only the final results. The full calculations are presented in Annexes 4 and 5, for drained and undrained conditions, respectively.

But before the bearing capacity calculations it is necessary to define the loads that will be applied and the effective area of the footing that will apply pressure on the ground.

In the preliminary design of the footing have been already calculated the loads at the base of the column (table 4.2). Now it will be only necessary to add the weight of the length of the column beneath the surface, the weight of the footing and the weight of the soil above it.

In the last chapter the footing was defined with $2.3 \times 2.3 \times 0.5$ meters and in chapter 3 it was concluded that it would be founded at 1.2 meters depth, with the first 0.3 meters to be removed during construction (concrete and soil).

With

$$W_{foot.} = V_{foot.} \cdot \gamma_c \quad (5.2)$$

where $W_{foot.}$ =weight of the footing, $V_{foot.}$ =volume of the footing and γ_c =unit weight of concrete.

$$W_{ter.} = \Sigma(\gamma_i \cdot h_i) \quad (5.3)$$

where $W_{ter.}$ =weight of the terrain above the footing, γ_i =unit weight of the different layers and h_i =thickness of the different layers.

And for the weight of the column can be used equation 4.1 considering the total length of the column between the roof and the footing.

So, according to figure 5.5 representing the position of the footing on the soil and the respective unit weight and thickness of the layers and considering 25 kN/m² the unit weight of concrete we have:

$$W_{pillar} = 58,125 + [(1.2 - 0.5) \times 0.5^2] \times 25 = 62.5 \text{ kN}$$

$$W_{foot.} = 25 \times (2.3 \times 2.3 \times 0.5) = 66.125 \text{ kN}$$

$$W_{ter.} = 16.9 \times 0.3 + 18 \times 0.1 = 6.87 \text{ kN}$$

In the next table are listed all the loads that will be considered in the verification of the soil resistance.

Table 5.1 – Loads at the base of the footing

Element	Vertical Load (kN)
Column	62.50
Reinforced Concrete Arch (18m span)	38.50
Reinforced Concrete Arch (24m span)	58.50
Reinforced Concrete Plates	185.50
Snow	151.20
Footing	66.13
Terrain	6.87
Total	569.20

In the case of a footing subject to an eccentric load it is necessary to determinate the effective dimensions of the footing that will be used in the bearing capacity calculation.

In this case study is being designed a square foundation with just a centered vertical load applied on it and so the effective area is coincident with the total foundation area.

Nevertheless the equations for determination of the effective dimensions of the footing are:

$$B' = B - 2 \cdot e_x \quad (5.4)$$

$$L' = L - 2 \cdot e_y \quad (5.5)$$

$$A' = B' \cdot L' \quad (5.6)$$

In this case we have:

$$B' = 2.3 \text{ m}$$

$$L' = 2.3 \text{ m}$$

$$A' = 2.3 \times 2.3 = 5.29 \text{ m}^2$$

5.1. BEARING CAPACITY IN DRAINED CONDITIONS

In drained conditions the bearing capacity will be calculated in terms of effective stress and can be expressed by the following equation presented in EC7:

$$R/A' = c' \cdot N_c \cdot b_c \cdot s_c \cdot i_c + q' \cdot N_q \cdot b_q \cdot s_q \cdot i_q + 0,5 \cdot \gamma' \cdot B' \cdot N_\gamma \cdot b_\gamma \cdot s_\gamma \cdot i_\gamma \quad (5.7)$$

where R/A' = bearing capacity of the soil (q_{ult}), c' = cohesion intercept in terms of effective stress, q' = design effective overburden pressure at the level of the foundation base, γ' = design effective unit weight of the soil below the foundation level, B' = the effective foundation width and all the other parameters will be defined later.

In drained conditions, it is considered that the loads will be slowly applied to the soil, giving time for dissipation of the excess pore water pressure created by loading. In fact this methodology is close to the reality, because the industrial building will be constructed in stages giving time for the soil to consolidate and in this way dissipate excess pore water pressure created by the increase of weight supported by it.

So, with this method the soil bearing capacity will be higher than in undrained conditions and partial safety factors will be applied to the loads and to the soil parameters, increasing the former and reducing the latter.

This new methodology introduced by the Eurocodes provides more safety and the possibility to give more weight to some parameters in detriment of others. Before the Eurocodes the bearing capacity was determined and only to the final result was applied a safety factor imposing that the force applied to the soil had to be smaller than the bearing capacity divided by that factor of safety.

So, according to EC7 in drained conditions it will be necessary to perform the calculations for the different combinations using the safety factors from the Lithuanian National Annex already referred in chapter 4 (table 4.3).

For the first layer below the footing (layer5, figure 3.4) we have:

Table 5.2- Loads and soil parameters after applying the safety factors

Combination	V_d (kN)	φ (°)	c' (kPa)
DA-1 a)	792.12	25.0	18.0
DA-1 b)	616.62	19.2	14.4
DA-2	792.12	25.0	18.0
DA-3	790.75	19.2	14.4

In table 5.2 the following expression was employed to calculate the load at the base of the footing:

$$V = (\Sigma V_{G_i}) \cdot \gamma_G + (\Sigma V_{Q_i}) \cdot \gamma_Q + (\Sigma V_{q_i}) \cdot \gamma_q \quad (5.8)$$

And so,

$$V = (W_{pillar} + W_{arch.} + W_{plat.} + W_{foot.}) \cdot \gamma_G + (W_{snow}) \cdot \gamma_Q + (W_{ter.}) \cdot \gamma_q$$

where $W_{pillar.}$ =weight of pillar, $W_{arch.}$ =weight of both arches supported by the footing, $W_{plat.}$ =weight of the concrete reinforced plates supported by the footing, $W_{foot.}$ =weight of the footing, W_{snow} =weight of the snow and $W_{ter.}$ =weight of the terrain above the footing.

To calculate the bearing capacity of the soil in drained conditions it is necessary to define and determine the remaining parameters of equation 5.7.

5.1.1. – DIMENSIONLESS FACTORS

5.1.1.1. Bearing capacity factors

Consulting EC7 we have the following equations:

$$N_q = e^{\pi \cdot \tan \varphi'} \tan^2(45 + \varphi'/2) \quad (5.9)$$

$$N_c = (N_q - 1) \cdot \cot \varphi' \quad (5.10)$$

$$N_\gamma = 2 \cdot (N_q - 1) \tan \varphi' \quad \text{where } \delta \geq \varphi'/2 \text{ (rough base)} \quad (5.11)$$

In the next table are presented the respective values of each bearing factor for the different combinations:

Table 5.3- Bearing capacity factors

Combination	N_c	N_q	N_γ
DA-1 a)	20.721	10.662	9.011
DA-1 b)	14.145	5.938	3.447
DA-2	20.721	10.662	9.011
DA-3	14.145	5.938	3.447

5.1.1.2. Inclination of the foundation base factors

The inclination of the foundation base factors can be determined by the following equations:

$$b_c = b_q - (1 - b_q)/N_c \times \tan \varphi' \quad (5.12)$$

$$b_q = b_\gamma = (1 - \alpha \cdot \tan \varphi)^2 \quad (5.13)$$

However the foundation that is being designed does not have any angle with the horizontal and so b_c , b_q and b_γ are equal to 1.

5.1.1.3. Correction factors of the shape of foundation base

The correction factors of the shape of foundation base are determinate by the following equations:

$$s_c = (s_q \cdot N_q - 1) / (N_q - 1) \text{ for rectangular, square or circular shape} \quad (5.14)$$

$$s_q = 1 + (B'/L') \cdot \sin\varphi' \text{ for a rectangular shape} \quad (5.15 a)$$

$$s_q = 1 + \sin\varphi' \text{ for a square or circular shape} \quad (5.15 b)$$

$$s_\gamma = 1 - 0.3 (B'/L') \text{ for a rectangular shape} \quad (5.16 a)$$

$$s_\gamma = 0.7 \text{ for a square or circular shape} \quad (5.16 b)$$

As a square footing is being designed,

Table 5.4 – Correction factors of the shape of foundation

Combination	s_c	s_q	s_γ
DA-1 a)	1.466	1.423	0.7
DA-1 b)	1.396	1.330	0.7
DA-2	1.466	1.423	0.7
DA-3	1.396	1.330	0.7

5.1.1.4. Inclination factors of the load

To determinate these factors is necessary to apply, on the following equations, the values of cohesion in effective stress shown in table 5.2. However as there are no horizontal loads all the factors will be equal to one.

$$i_c = i_q - (1 - i_q) / N_c \cdot \tan\varphi' \quad (5.17)$$

$$i_q = [1 - H / (V + A' \cdot c' \cdot \cot\varphi')]^m \quad (5.18)$$

$$i_\gamma = [1 - H / (V + A' \cdot c' \cdot \cot\varphi')]^{m+1} \quad (5.19)$$

where,

$$m = m_B = [2 + (B'/L')] / [1 + (B'/L')] \text{ when } H \text{ acts in the direction of } B' \quad (5.20)$$

$$m = m_L = [2 + (L'/B')] / [1 + (L'/B')] \text{ when } H \text{ acts in the direction of } L' \quad (5.21)$$

In cases where the horizontal load component acts in a direction forming an angle θ with the direction of L' , m may be calculated by:

$$m = m_{\theta} = m_L \cdot \cos^2\theta + m_B \cdot \sin^2\theta \quad (5.22)$$

5.1.2. – BEARING RESISTANCE OF THE SOIL AND THE SAFETY OF THE STRUCTURE

To proceed to the calculation of safety of the structure against soil shear we have yet to calculate the value of q' , the design effective overburden pressure at the level of the foundation base and introduce in equation 5.7 this value and the values of all determined parameters to then compare by equation 5.23 the soil resistance with the load applied to the soil.

$$R_{v,d} \leq V_d \quad (5.23)$$

q' will be the thickness of the layers above the base of the footing multiplied by their respective effective unit weight. Thus:

$$q' = 16,9 \times 0,3 + 18,0 \times 0,1 + 20,5 \times 0,2 + 21 \times 0,3 = 17,27 \text{ kN}$$

However this value has to be multiplied by the partial safety factors corresponding to each combination.

Table 5.5 presents the final values of the soil bearing capacity, the load applied by the foundation to the soil and the difference between them for each combination.

Table 5.5 – Bearing resistance and global safety of the structure for drained conditions

Combination	V_d (kN)	R/A' (kPa)	R_v (kN)	$R_{v,d}$ (kN)	$R_{v,d}/V_d$
DA-1 a)	792.12	1089.99	5766.07	5766.07	7.28
DA-1 b)	616.62	519.09	2746.00	2746.00	4.45
DA-2	792.12	1089.99	5766.06	4118.62	5.20
DA-3	790.75	519.09	2746.00	2746.00	3.47

where R_v =bearing capacity (R/A') multiplied by the footing effective area and $R_{v,d} = R_v$ divided by the partial safety factor γ_{R_v} .

In drained conditions normally the minimum recommended values for the difference between the load and the soil resistance ($R_{v,d}/V_d$) in spread foundations are between 3.0 and 4.0 and so there is no risk of shearing of the foundation soil in this case.

The values of the same difference when considered the characteristic of the other layers are presented next but, as should be expected, they are larger than the previous ones because those soil layers have better resistance characteristics than the soil of layer 5.

Table 5.6 – $R_v, d/V_d$ for the different soil layers in drained conditions

Combination	Layer 5	Layer 6	Layer 7	Layer 8
DA-1 a)	7.28	24.90	24.91	21.33
DA-1 b)	4.45	15.09	15.09	6.72
DA-2	5.20	17.79	17.79	15.23
DA-3	3.47	11.77	11.77	5.24

5.2. BEARING CAPACITY IN UNDRAINED CONDITIONS

As it was said in the last sub-chapter, in these conditions the soil will present less resistance being the value of the bearing capacity, in general, lower than the real value, because during construction the soil always dissipates part of the excess pore water pressure and the undrained hypothesis considers a situation equivalent to the installation of the whole building on the soil in just one instant.

Following EC7 will be presented next the methodology to compare, for these conditions, the soil resistance with the load applied to it, considering the same dimensions of the footing presented in the beginning of this chapter and the loads listed in table 5.1.

So, the bearing resistance of the soil in undrained conditions is given by the following equation:

$$R/A' = (\pi + 2) \cdot c_u \cdot b_c \cdot s_c \cdot i_c + q \quad (5.24)$$

When verifying the soil resistance in undrained conditions only the safety factor relative to the undrained resistance of the soil (C_u) is used and so it will be considered the total value presented in table 5.1 for the footing load and for the cohesion (c') will be used the values obtained in the soil geotechnical test.

Again only the intermediate calculations for the case of layer 5 will be presented. The calculations for the other layers are included in Annex 3, being only presented in this chapter the final results for the global safety factors.

In table 5.7 it is possible to see the values of C_u for the different combinations that will be used in the next calculations.

Table 5.7 – Values of undrained resistance of the soil for layer 5

Combination	c' (kPa)	γ_{c_u}	c_u (kPa)
DA-1 a)	18	1.4	12.86
DA-1 b)	18	1.4	12.86
DA-2	18	1	18
DA-3	18	1.4	12.86

5.2.1. – DIMENSIONLESS FACTORS

5.2.1.1. Inclination of the foundation base factors

In undrained conditions, as can be seen in equation 5.24, it is only necessary to calculate the factors related with cohesion. So, the dimensionless factor for the inclination of the foundation base is given by:

$$b_c = 1 - 2 \cdot \alpha / (\pi + 2) \quad (5.25)$$

As the foundation that is being dimensioned is horizontal $b_c = 1$.

5.2.1.2. Correction factors for the shape of the foundation base

The dimensionless factor for the shape of the footing base in undrained conditions can be calculated by the next equations, depending of the shape:

$$s_c = 1 + 0.2 \times (B'/L') \text{ for a rectangular shape} \quad (5.26 \text{ a})$$

$$s_c = 1.2 \text{ for a square or circular shape} \quad (5.26 \text{ b})$$

As the footing in study is square $s_c = 1.2$.

5.2.1.3. Inclination factors of the load

The inclination factor of the load related to the cohesion is given by:

$$i_c = \frac{1}{2} \left(1 + \sqrt{1 - \frac{H}{A' \cdot C_u}} \right) \quad (5.27)$$

with, $H \leq A' \cdot C_u$

As there are no horizontal loads affecting the footing i_c will be equal to 1.

After determining all the necessary parameters for equation 5.24 next will be calculated the bearing resistance of the soil in undrained conditions to then compare it with the value of the load applied in the soil.

5.2.2. – BEARING RESISTANCE OF THE SOIL AND SAFETY THE OF THE STRUCTURE

Being q =overburden or surcharge pressure at the level of the foundation base it will be utilized again the value for drained conditions. So, with $q = 17.27 \text{ kN}$ we can test, finally, the global safety of the structure in undrained conditions given by equation 5.28.

$$R_v \leq V_d \quad (5.28)$$

This equation only differs from equation 5.23 in that the partial safety factor related to the soil resistance (γ_{Rv}) is not applied to the bearing capacity of the soil.

Table 5.8 presents the global safety factors as well the soil bearing resistance in this particular condition.

Table 5.8 – Bearing resistance and global safety of the structure in undrained conditions

Combination	V_d (kN)	R/A' (kPa)	R_v (kN)	R_v/V_d
DA-1 a)	569.20	96.60	511.00	0.90
DA-1 b)	569.20	96.60	511.00	0.90
DA-2	569.20	128.33	678.86	1.19
DA-3	569.20	96.60	511.00	0.90

As can be seen in the table, the ratio (R_v/V_d) in three of the four combinations is smaller than one, proving in this way the instability of all the system.

To increase the safety of the structure it is just need to increase the footing contact area with the soil and so after four more iterations, described in Annex it was concluded that the this ratio in all combinations to undrained conditions is higher than one for a square footing with 2.5 meters side. In order to guarantee and certify the safety of all infrastructure and in that way of the superstructure too, the footing it will be designed with 3.2 meters side providing in this way a ratio between the soil resistance and the applied load of 1.5.

Table 5.9 – Bearing resistance and global safety of the structure with the final dimensions of the footing

Combination	V_d (kN)	R/A' (kPa)	R_v (kN)	R_v/V_d
DA-1 a)	631.07	96.60	989,16	1.57
DA-1 b)	631.07	96.60	989.16	1.57
DA-2	631.07	128.33	1314.08	2.08
DA-3	631.07	96.60	989.16	1.57

With these dimensions using equation 4.8 the height of the footing will change as we can see next:

$$h = a_0/2 = \left(\frac{3.2}{2} - \frac{0.5}{2}\right)/2 = 0.675 \text{ m} \Rightarrow 0.7 \text{ m}$$

The footing weight will change and the weight of the terrain and so the value of V_d . However it is unnecessary to calculate again the bearing resistance of the soil because with the increase of the footing the bearing capacity necessarily increased too.

The calculated ratio is much closer to unit in undrained that in drained conditions because in this conditions we are being too conservative and so it can be considered that, as these conditions never happen entirely, the soil resistance is higher than the values calculated.

In the last chapter was calculated the risk of punching shear of the footing and because of this variation of dimensions the punching risk should be calculated again. However, as the structure was comfortably safe against this phenomenon it is considered a waste of time to repeat the same calculation because in spite of the variation of the footing area, the height of the footing increased too and so these variations surely will not interfere with the internal stability of the element.

To finish this chapter table 5.10 presents the values of the ratio (R_v/V_d) in the cases of the foundation soil being homogeneous with the characteristics of layers 6, 7 or 8 and already with the final dimensions of the footing rearranged.

Table 5.10 – R_v/V_d for the different types of soil in undrained conditions

Combination	Layer 5	Layer 6	Layer 7	Layer 8
DA-1 a)	1.57	5.85	5.85	0.53
DA-1 b)	1.57	5.85	5.85	0.53
DA-2	2.08	8.08	8.08	0.63
DA-3	1.57	5.85	5.85	0.53

In this table it can be seen that if the foundation was founded in a layer with the characteristics of layer 8, the soil would suffer a general shear failure in undrained conditions. However, considering that it is sand this conclusion does not apply this time because sand layer are always drained. And this soil layer had presented a good response in drained conditions.

To check the external stability it is also required to verify the possibility of sliding of the foundation base.

Base sliding has to be resisted by the soil on the side opposite to the application of the horizontal load and by the friction between the base and the soil.

However, in this case study the foundation is not subject to any horizontal load. Nevertheless the equations necessary for the verification of footing base sliding will now be given.

The next chapter will be devoted to the question of footing settlement.

5.3. SLIDING

The verification of the safety is given in this case by the following relation of EC7:

$$H_d \leq R_d + R_{p;d} \quad (5.29)$$

where H_d =acting horizontal load, R_d =friction of the base and $R_{p;d}$ =design value of the resisting force caused by earth pressure on the side of the foundation.

The value of base friction depends on the type of the soil where the foundation is founded and so in drained conditions (in general for sandy soils):

$$R_d = V_d' \cdot \tan(\delta_d) \quad (5.30)$$

where V_d' =vertical load applied to the soil, δ_d =friction angle between the footing and the foundation soil.

For cast-in-situ concrete footings $\delta_d = \phi_d'$ and for smooth precast footings $\delta_d = 2/3 \cdot \phi_d'$.

In undrained conditions (in clayey soils):

$$R_d = A_c \times C_{u;d} \quad (5.31)$$

where A_c =effective area of the footing and $C_{u;d}$ =undrained shear strength of the soil.

6

SETTLEMENT ESTIMATE AND ANALYSIS

Many structures can admit some foundation settlement without suffering any damage but all have a stipulated limit for that.

Beyond that limit the structures will crack and will become weakened and thus, with less global resistance. The structure will stay functional, but if the stipulated settlement limit is too much exceeded the structure also has the risk of total collapse.

So in most buildings the vertical settlements are limited to 50 mm (EC 7), with some modifications according to the type and purpose of the structure.

Besides vertical settlement there is another problem that the structures have to face. The contact of any structure with the ground is normally along a surface. If the structural load distribution and the soil deformability are not uniform, some parts of the soil beneath the structure may suffer more strain than others and consequently some parts of the structure settle more than others, giving rise to differential settlements.

The most famous example of a structure severely affected by the differential settlements is the Tower of Pisa in Pisa, Italy (Figure 6.1).



Fig.6.1 – Tower of Pisa, Italy (Waine, Mary in www.woodenspears.com)

These differential settlements, besides the problems of cracking and distortion of elements of the structure, as windows, doors, panels, pavements and stairs, can induce problems with neighbouring structures and interfere with machinery and with people. In general the differential settlement should not exceed 20 mm (EC 7).

In this text it is being considered the central footing of an industrial building founded on a layered soil considered transversely homogeneous. Since the influence of the wind is considered to be supported by the frame of the building and there are no more horizontal forces acting on the structure, the differential settlements are not a concern in this study.

However it is necessary to verify the limit of 50 mm to the vertical settlements.

Since the determination of the settlements that structures will experience after construction is in general very difficult, it is more appropriate to use the term estimate instead of calculation, having in mind that all values obtained can be totally unrealistic.

6.1. SETTLEMENT ESTIMATE

To evaluate if the dimensions chosen for one particular foundation are suitable given the soil characteristics it is necessary to estimate the vertical settlement associated to the vertical loads applied by the foundation to the ground beneath it.

The total settlement (s) can be divided into three different components: the immediate settlement (s_0), the settlement caused by consolidation (s_1), and the settlement caused by creep or secondary settlement (s_2).

$$S = S_0 + S_1 + S_2 \quad (6.1)$$

It is possible to represent the total settlement by one generic time-settlement function, illustrated in figure 6.2, where it is understood the influence of each type of settlement for the final value and when and during how much time each one occurs.

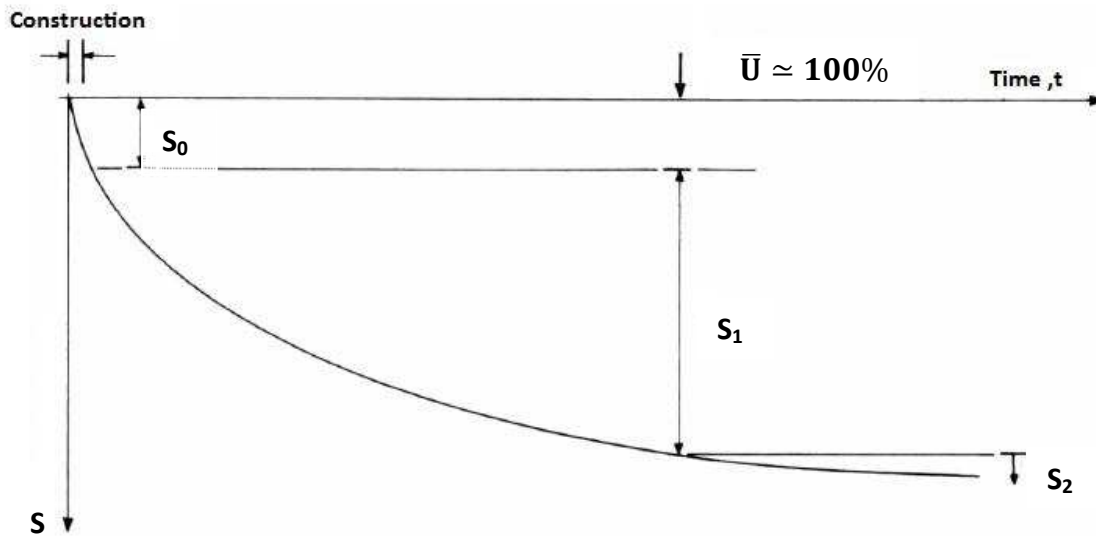


Fig. 6.2 – Generic time-settlement function of one foundation (Perloff, 1975).

This figure treats a generic case, where most of the settlement occurs by consolidation. In fact the development of the curve depends essentially on the type of the soil beneath the foundation. For sandy soils the immediate settlement constitutes the most part of the total settlement, the consolidation of the soil being practically null and the creep only relevant in the case of large variations of the applied load (Mecânica dos solos – Conceitos e princípios fundamentais, Matos Fernandes, Manuel), but in the case of clayey soils, the immediate settlement is lower and the consolidation of the soil can take huge proportions.

So, buildings founded in sandy soils do not have any problems with the distortion of facades and secondary construction elements because when they are installed all the settlements already occurred.

This chapter does treat consolidation nor creep, being only concerned with the estimate of immediate settlements. This estimation it will be done by means of two different methodologies, the elasticity theory and the adjusted elasticity method presented in EC7.

6.1.1. –SETTLEMENTS FROM ELASTICITY THEORY

The vertical settlement under an area carrying a uniform pressure Δq_s on the surface of a soil composed by layers with a linear stress-strain relationship can be expressed by the application of Hooke's law as:

$$s_i = \sum_{j=1}^n \frac{1}{E_j} \cdot [\Delta\sigma_{zj} - \nu_j \cdot (\Delta\sigma_{xj} + \Delta\sigma_{yj})] \cdot h_j \quad (6.2)$$

where $\Delta\sigma_{zj}$, $\Delta\sigma_{xj}$ and $\Delta\sigma_{yj}$ are the stress increments induced by Δq_s and h_j , E_j and ν_j are the thickness, the deformation modulus and the Poisson ratio of the generic layer.

The stress increments along a vertical line passing at the vertex of a rectangle with sides b and l (Giroud, 1970) can be calculated by the following three expressions based on the elasticity theory. So to know the stress variation with depth at the center of the footing b and l will be half size of the footing and the results given by the expressions must be multiplied by four.

$$\Delta\sigma_{zj} = \Delta q_s \cdot K_1 \quad (6.3)$$

$$\Delta\sigma_{xj} = \Delta q_s \cdot [K_2 - (1 - 2\nu) \cdot K_2^1] \quad (6.4)$$

$$\Delta\sigma_{yj} = \Delta q_s \cdot [L_2 - (1 - 2\nu) \cdot L_2^1] \quad (6.5)$$

The equations can be better understood consulting the next figure and the values of K_1 , K_2 , K_2^1 , L_2 and L_2^1 are listed in Annex 6.

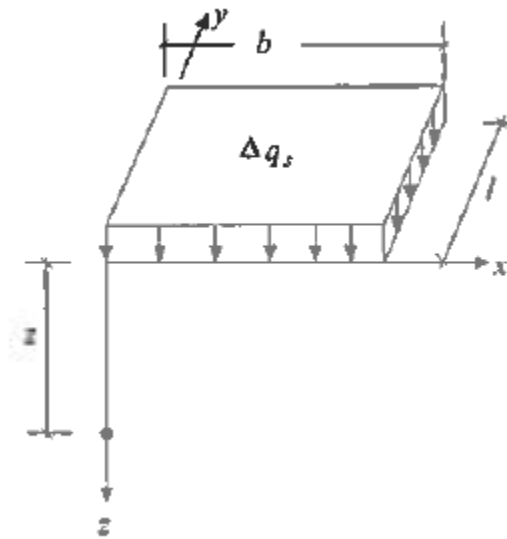


Fig.6.3- Uniform load applied on a rectangular area (Mecânica dos Solos – Conceitos e Princípios Fundamentais. Matos Fernandes, Manuel)

Firstly it is necessary to define the uniform load that will be applied to the ground. As it was seen in chapter 5 the vertical load at the base of the footing was 569.13 kN, however with the increase of the footing dimensions this value changed for 678.78 kN. Once more with the purpose of guaranteeing the total safety of the design the safety factors recommended by the Lithuanian National Annex of Eurocode 7 will be considered and so we get $Q = 939,54 \text{ KN}$ for the worst case, i.e. the largest value obtained in all combinations.

Δq_s corresponds to the stresses applied at the base of the footing by the calculated vertical load and can be expressed by:

$$\Delta q_s = Q/A \quad (6.6)$$

As in the last chapter the sides of the footing have 3,2 meters each,

$$\Delta q_s = \frac{939,54}{3,2 \times 3,2} = 91,75 \text{ kN/m}^2$$

The Poisson ratio is not known and so it will be necessary to choose consciously the value to use for each layer.

The Poisson ratio expresses the ratio between the lateral and longitudinal strain of a material when compressed or stretched in one direction. For example, when a sample cube of material is compressed in one direction, it tends to expand in the other two directions perpendicular to the first and if the first is stretched the other two tend to contract. The Poisson ratio is the value of deformation in the direction perpendicular to the applied force divided by the value of deformation in the direction of the applied load (equation 6.7).

$$\nu = -\frac{\varepsilon_{trans.}}{\varepsilon_{axial}} \quad (6.7)$$

Most materials have Poisson ratios between 0.0 and 0.5, being the most normal situation that materials present one value near 0.3. In the soils this ratio does not vary too much also, presenting in sands values from 0.20 to 0.45 and in clays between 0.30 and 0.45 and from 0.4 to 0.5 when undrained.

Beneath the foundation base 4 different layers have been identified (figure 3.4): one of loam, two of moraine loam and one of fine sand.

First at all, it is necessary explain the concepts of loam and moraine. Loam is a fertile soil constituted by sand, clay and decomposed vegetable matter. Moraine, consulting the Oxford English Dictionary, can be defined as a mass of rocks and sediment carried along and deposited by a glacier.

The soil influencing to evaluation of settlements is mostly composed by sand, clay, vegetable matter and some rock masses and so, given to the limits listed above, we shall adopt a Poisson ratio of 0.35 for every layer, thus standardizing all calculations.

Now, according to the equations 6.3, 6.4 and 6.5, table 6.1 presents the values of stress increment in the middle of each layer.

Table 6.1 – Increments of Stress in the soil beneath the footing

Layer no.	Depth (m)	Thickness (m)	Poisson ratio	$\Delta\sigma_z$ (kPa)	$\Delta\sigma_y$ (kPa)	$\Delta\sigma_x$ (kPa)
5	1.6	0.6	0.35	64.22	8.95	8.95
6	2.2	0.8	0.35	54.82	5.16	5.16
7	2.8	0.4	0.35	39.73	1.61	1.61
8	3.5	1.0	0.35	25.33	0.10	0.10

The stress increments and the deformation modulus of each layer identified in table 3.1 enabled the use of equation 6.2 to estimate the immediate settlements that the footing will suffer during construction, as shown in table 6.2.

Table 6.2 – Immediate settlements of the footing

Layer no.	Depth (m)	Thickness (m)	Poisson ratio	E (MPa)	s (mm)
5	1.5	0.6	0.35	15	0,58
6	2.2	0.8	0.35	27	0,38
7	2.8	0.4	0.35	64	0.06
8	3.5	1.0	0.35	68	0.09
Total	-	-	-	-	1.11

The soil beneath the footing presents characteristics that would lead to anticipate small immediate settlements, because the layer where the footing lies has a hard plastic consistency and the two next ones are semi-hard and hard. So, the only layer that probably could experience larger settlements is situated at a depth where the load applied by the foundation no longer has a significant influence. The layer where the footing is founded is prone to consolidation settlement because of its plastic characteristics. However, according to the aim of this text it only the immediate settlements will be estimated.

Thus,

$$1.11 \text{ mm} \leq 50 \text{ mm} \Rightarrow O.K.$$

The limit of 50 mm is respected and so the stability and the function of the structure is totally guaranteed.

6.1.2. – EC7 ADJUSTED ELASTICITY METHOD

According to EC7, the total settlement of a foundation on cohesive or non-cohesive soil may be evaluated by the equation (6.8) developed from elasticity theory. We then have:

$$s = p \times b \times f / E_m \quad (6.8)$$

where, p = bearing pressure, linearly distributed on the foundation base, b = foundation width, f = settlement coefficient and E_m = design value of the elasticity modulus.

The value of the settlement coefficient (f) depends on the shape and dimensions of the foundation area, of the variation of stiffness with depth and of the thickness of the compressible formation in the following way:

$$f = (1 - \nu^2) \cdot I_s \quad (6.9)$$

where, I_s = adimensional factor that depends on geometry of loaded area and of the point of the surface considered.

The variation of the adimensional factor (I_s) can be better understood in the next table where are included the values for different geometries of uniform loads for rigid and infinitely flexible footings. Only the values for the soil beneath the centre of the load will be compared, since it is the place where larger settlement values are expected. In Annex 6 is included the complete table.

Table 6.3 – Values of I_s at the centre of the loaded area

Shape of the footing	Rigid footing	Infinitely flexible footing
Circular	0.79	1.00
Square	0.92	1.12
Rectangular (L/B=1.5)	1.13	1.36
Rectangular (L/B=2)	1.27	1.53
Rectangular (L/B=2,5)	?	1.67
Rectangular (L/B=3)	1.51	1.78

As real foundations are much closer to rigid foundations, the value of the table corresponding to a square and rigid footing, $I_s = 0.92$, will be used.

A constant value of the Poisson ratio will be taken for all foundation soil layers, as was discussed before. Since equation 6.8 has been derived for soils with constant characteristics in depth, it will be necessary to adopt a uniform deformability modulus for all soil layers beneath the foundation. To obtain the required value the deformability modulus of the four layers will be averaged taking into account the thickness of each of them.

$$\bar{E}_m = \frac{\Sigma(E_{m_i} \cdot h_i)}{\Sigma h_i} = \frac{(15 \times 0.6 + 27 \times 0.8 + 64 \times 0.4 + 68 \times 1.0)}{(0.6 + 0.8 + 0.4 + 1.0)} = 44.36 \text{ MPa}$$

Being the value of the uniform load the same as in 6.1.1, $p = 124.04 \text{ kN/m}^2$:

$$f = (1 - 0.35^2) \cdot 0.92 = 0.81$$

$$s = 91.75 \times 3.2 \times 0.81 / 44.36 = 5.34 \text{ mm}$$

$$5.34 \text{ mm} \leq 50 \text{ mm} \Rightarrow O.K.$$

However in this estimate it is considered that all soils have the same importance to the total value of the settlements, which it is not true. The first layers under the foundation will suffer more settlement because the stress increments on them are larger.

So, the same equation will be using again but considering now the worst situation, i.e., that all soil has the characteristics of layer 5.

Thus, $E = 15 \text{ MPa}$ and $\nu = 0,35$ and then:

$$s = 91.75 \times 3.2 \times 0.81 / 15 = 15.8 \text{ mm}$$

$$15.8 \text{ mm} \leq 50 \text{ mm} \Rightarrow O.K.$$

One more time it was verified that the safety against excessive settlement is totally ensured, as in this second method the obtained value is, once again, very far from the imposed limit of 50 mm. It becomes clear too that the adjusted elasticity method does not give such an approximate estimate as the first one where all the layers are independently considered.

7

DESIGN OF REINFORCING STEEL

In this chapter will be designed the steel reinforcement of the footing in directions OX and OY. For that it is necessary to define the moments that act on the footing created by the soil upwards pressure and then, determine the necessary steel for those moments.

So:

$$M_{x,y} = \frac{\sigma_3 \times L_{y,x} \times l_{x,y}}{2} + \frac{(\sigma_1 - \sigma_3) \times L_{y,x} \times l_{x,y}^2}{3} \quad (7.1)$$

where $L_{y,x}$ = dimensions of the footing base, l_x is defined in the next equation and σ_1 and σ_3 are the stresses in the contact between the soil and the footing, σ_1 being the largest pressure value and σ_3 the contact pressure at the distance of $l_{x,y}$ from the edge of the footing. In figure 7.1 is represented the pressure diagram for a footing loaded by a vertical load and moment in one direction.

$$l_{x,y} = \frac{L_{x,y}}{2} - \frac{b_{x,y}}{2} + 0,15 \cdot b_{x,y} \quad (7.2)$$

where $b_{x,y}$ = dimensions of the column cross section.

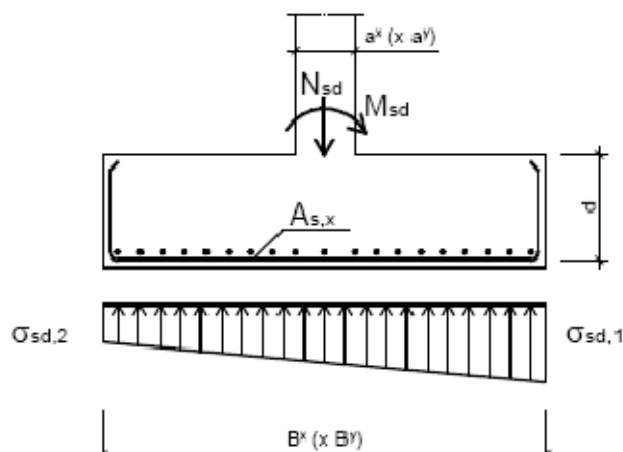


Fig.7.1 – Pressure in the footing-soil contact (Fundações e Estruturas de Suporte, Arêde, António)

In figure 7.1 is represented one generic footing that supports a column subjected to a vertical load and a moment in one direction at the base. As it is possible to see the moment at the base of the column causes the ground pressure not to be uniform, presenting larger values in one edge that may induce

some rotation of the footing. However in this footing case study no moments are present and so the pressure diagram will be uniform and so $\sigma_1 = \sigma_2 = \sigma_3$. This value has already been calculated in chapter 6 as Δq_s .

It should be noted that since the footing is square and there are no applied moments M_x will be equal to M_y .

So,

$$l_x = l_y = \frac{3.2}{2} - \frac{0.5}{2} + 0.15 \times 0.5 = 1.425 \text{ m}$$

$$M_x = M_y = \frac{91.752 \times 3.2 \times 1.425}{2} + \frac{(91.752 - 91.752) \times 2.5 \times 1.075^2}{3} = 209.194 \text{ kN.m}$$

To design the steel it is necessary to define some specifications related to the construction, such as the type and resistance of the concrete and of the steel that will be used and the thickness of the reinforcing steel cover.

Table 7.1 features the materials that will be used in the footing and the dimensions that will be considered for steel calculation.

Table 7.1 – Materials and geometric parameters of the foundation

Concrete	Steel	h (mm)	c (mm)	d (mm)
C25/30	S400	700	70	620

Using the simple bending tables of the Portuguese Regulation for Reinforced Concrete we have:

$$\mu = \frac{Msd}{b \cdot d^2 \cdot fcd} \quad (7.3)$$

$$\omega = \frac{A_s \cdot fsyd}{b \cdot d \cdot fcd} \quad (7.4)$$

where, μ =reduced bending moment, fcd =design value of the concrete compressive strength, ω =mechanical reinforcement ratio, $fsyd$ =design yield strength of reinforcement and A_s =area of steel.

With:

$$fcd = fck / \gamma_c \quad (7.5)$$

$$fsyd = fsyk / \gamma_s \quad (7.6)$$

To the lower reinforcing steel:

$$Msd^+ = 209.194 \text{ kN.m}$$

$$\mu = \frac{209.194}{3.2 \times 0.62^2 \times 25000/1.5} = 0.01$$

With this result of μ will be used the table 1_S400 of the design tables according to EC2 developed by Prof. Helena Barros and Prof. Joaquim Figueiras presented in Annex 7, appropriate for this design, with the objective of determining the mechanical reinforcement ratio.

So, according with this table, for a S400 steel, C25/30 concrete and $\mu = 0.01$:

$$\omega = 0.01$$

Based on equation 7.4 the area of steel required can be now determined.

$$0.01 = \frac{A_s \times 400000/1.15}{3.2 \times 0.62 \times 25000/1.5} \Rightarrow A_s = 9.50 \text{ cm}^2$$

However, Eurocode 2 imposes to the longitudinal tensile reinforcement a minimum of:

$$A_{s,min} = 0.26 \cdot \frac{f_{ctm}}{f_{yk}} \cdot b_t \cdot d \quad (7.7 \text{ a})$$

but not less than,

$$A_{s,min} \geq 0.0013 \cdot b_t \cdot d \quad (7.7 \text{ b})$$

According to table 3.1 of EC2 for C25/30 $f_{ctm} = 2.6 \text{ MPa}$ and so:

$$A_{s,min} = 0.26 \times \frac{2600}{400000} \times 3.2 \times 0.62 = 33.53 \text{ cm}^2 \geq 0.0013 \times 3.2 \times 0.62 = 23.79 \text{ cm}^2$$

$$A_{s,min} = 33.53 \text{ cm}^2$$

So, as the value of the required steel is smaller than the minimum prescribed by EC2 we will have:

$$A_{s,inf} = 33.53 \text{ cm}^2 \Rightarrow 12\phi 20 (37.7 \text{ cm}^2)$$

In this design it was considered that the footing will be founded on an infinitely rigid soil but it has to be understood that if the foundation soil is considered with its deformation modulus of 15 MPa then the moments would be much higher if the pressure diagram in the contact face would not be totally uniform.

It is not necessary to calculate the distribution reinforcement taking into account that the steel in one direction will be the distribution reinforcement in the other.

Besides negative moments were not considered and so, if the footing was designed in Portugal, for the top reinforcement would be adopted a constructive steel of #10//20, but as the design is being developed in Lithuania this will be neglected.

In the next figures the footing is presented with the final dimensions and with the reinforcing steel:

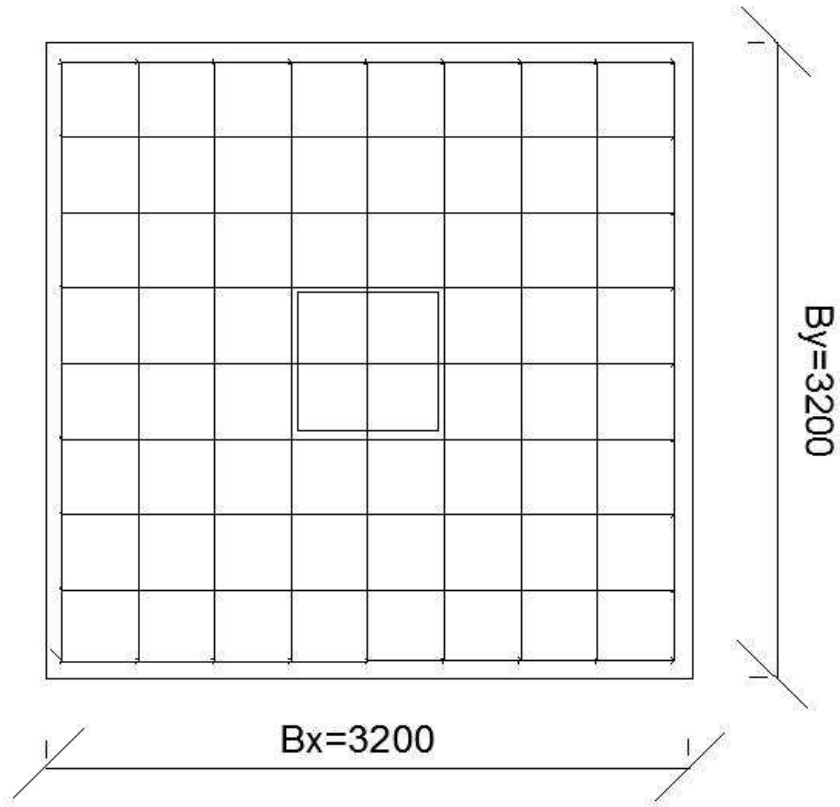


Fig.7.2 – Plan of footing (mm)

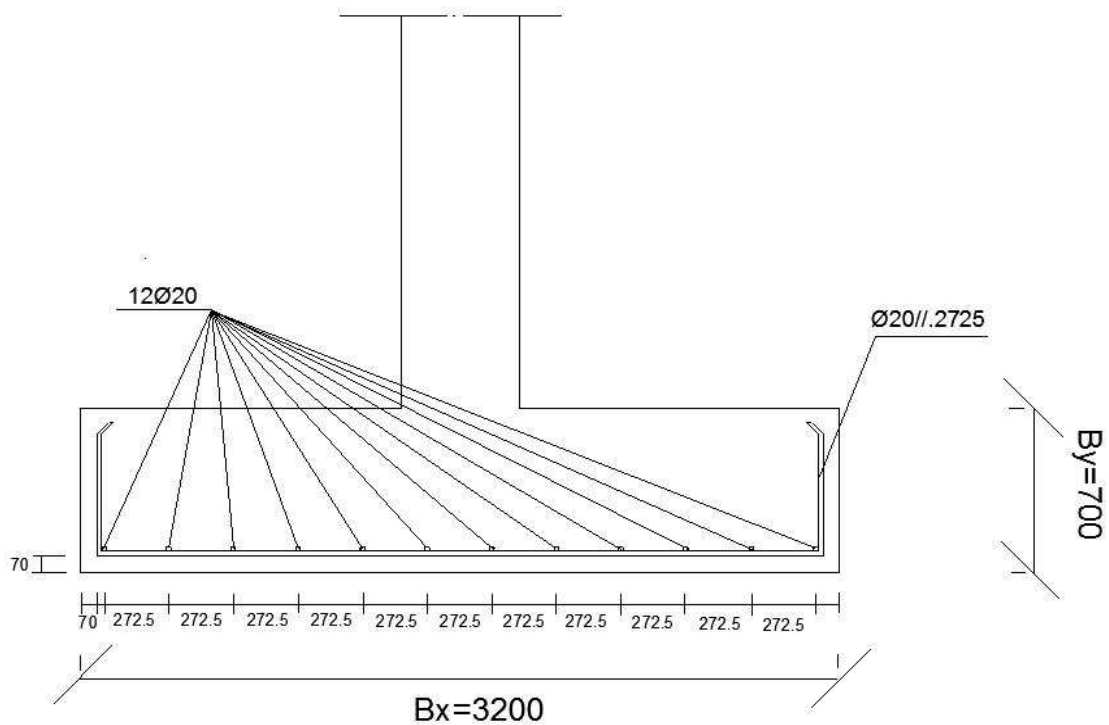


Fig 7.3 –Transversal section of footing (mm)

It must be said to that the rods in both directions are tied up because electrical solder, besides unnecessary, would be expensive.

In chapter 4 the footing dimensions were defined to support shear and punching shear without any specific reinforcement requirement. In spite of the base dimensions having been slightly increased, the punching shear stress was sufficient low to guarantee that these alterations do not bring the risk of collapse of the structure by this phenomenon and so the foundation will not need any extra shear reinforcement.

8

CONCLUSIONS

The work presented in this thesis has the primary objective of identifying of the principal differences between Lithuanian and Portuguese methodology of shallow foundations design. With the creation of the Structural Eurocode Programme, the design of structures became standardize almost all over Europe, existing only some variations according to the National Annexes of each country.

So, after the completion of this work were achieved the following conclusions:

1. Besides the formulation presented on the Eurocodes it was realized that the weather and geotechnical conditions could bring modifications to the procedures that are the norm in Portugal;
2. All the footing design follows the Eurocodes procedures and so, the test of soil resistance and its tendency to settle when charged was verified by applying Eurocode 7 that contemplates safety coefficients specific for Lithuania that divide the calculations into 4 different combinations;
3. The cities of Porto and Kaunas are wide apart in Europe and this geographic distance can be denoted in their geological and geotechnical characteristics. The layered soils of Kaunas where predominate sands, clays and loams face the residual soils of Porto granite but, as it was referred in chapter 2 of this work, such differences are not translated in the type of foundations commonly used, being in both cases spread or shallow foundations a current choice for construction on both countries;
4. Snow and cold characterize this country of Eastern Europe and influence the culture, the traditions, the personality of the people and of course the architecture and civil engineering works. The cold besides being felt, can also be seen in the buildings where the huge thermal variations cause cracks in building walls and undulation of roads and sidewalks. This undulation has its cause in two phenomena very particular in countries that experience very cold winters, frost heave and thaw weakening;
5. The tables of TK6-1 contain very useful information that makes the design of structures easier because all elements are already established and their weights calculated. However with the goal of constructing cheaper buildings engineers have the possibility of not using the standard elements contained in that tables;
6. The action of frost implies that the foundation of the buildings must be founded under the soil layer subject to variations of volume that otherwise would cause the lifting of the footings if this depth was not respected;
7. The entire procedure of footing design was developed during this work, making possible its consultation to further projects as it presented the methodologies for different types of foundations that can be adapted to different soil conditions;
8. It is more recommendable to perform all the internal structural stability verifications after the foundation soil resistance and settlement studies. During the calculations it was realized that the latter are more influenced by foundation area dimensions, while for internal stability verifications

the footing height is the biggest concern, although this dimension does influence very little the soil verifications;

9. The total absence of moments allied to a relative small vertical load applied to the footing is due to the light superstructure used in the Industrial Buildings of Lithuania. It originates the adoption of a square footing design of small dimensions without the need for almost any reinforcement steel, most of it being merely constructive.

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Annexes

Annex 1

Construction Technical Regulations of
Lithuania (Snow Pressure Analysis)

LIETUVOS SNIEGO APKROVOS RAJONAI

1. Sniego antžeminės apkrovos charakteristinė reikšmė s_k yra apibrėžta Reglamento 153 p.
2. Sniego antžeminės apkrovos charakteristinės reikšmės s_k konkreitiems Lietuvos rajonams yra pateiktos šio priedo 1 lentelėje, o rajonai parodyti 1 paveiksle.

1 lentelė.

Sniego antžeminės apkrovos s_k charakteristinės reikšmės

Sniego apkrovos rajonas	s_k , kN/m ²
I	1,2
II	1,6

pav.



Lietuvos sniego apkrovos rajonai

Žymenys:

I, II – sniego apkrovos rajonas

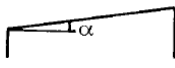
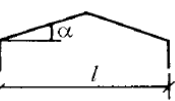
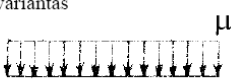
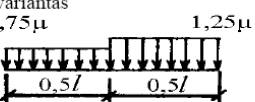
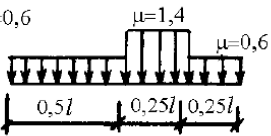
Pastaba:

sniego apkrovos rajonų ribos nustatomos pagal administracinio rajono ribas.

SNIEGO APKROVOS SCHEMOS IR KOEFICIENTAI M

1 lentelė.

Sniego apkrovos schemos ir koeficientai M

Schemos Nr.	Stogų profiliai ir sniego apkrovų schemos	Koeficientai μ ir schemų taikymo sritis
1.	<p>Statiniai su vienslaičiais ir dvislaičiais stogais</p> <p>a) </p> <p>b) </p> <p>1 variantas </p> <p>2 variantas </p> <p>3 variantas </p>	<p>$\mu = 1$, kai $\alpha \leq 25^\circ$; $\mu = 0$, kai $\alpha \geq 60^\circ$. 2 ir 3 variantus reikia įvertinti statiniams su dvislaičiais stogais (profilis b), be to, 2 variantas – kai $20^\circ \leq \alpha \leq 30^\circ$; 3 variantas – kai $10^\circ \leq \alpha \leq 30^\circ$, tik esant apžiūros tilteliams arba aeracinei įrangai ant stogo kraigo.</p>
2.	<p>Statiniai su skliautiniiais ir panašaus į juos kontūro stogais</p>	<p>$\mu_1 = \frac{1}{8f}$, bet ne daugiau kaip 1,0 ir ne mažiau kaip 0,4. 2 variantą reikia įvertinti, kai $\frac{f}{l} \geq \frac{1}{8}$</p> <p>$\frac{f}{l}$ $\frac{1}{8}$ $\frac{1}{6}$</p>

		$\geq \frac{1}{5}$ μ_2 1.6 2.0 2.2 Gelžbetoninėms stogo plokštėms koeficientą μ reikia imti ne didesnę kaip 1.4.
2'		Kai $\beta \geq 15^\circ$, būtina taikyti 1b schemą, imant $l=l'$, kai $\beta < 15^\circ$ – 2 schema.
3.	<p>Statiniai su išilginiais stoglangiais</p> <p>Vėją sulaikantis skydas</p> <p>Planas</p> <p>C zona A zona B zona</p>	$\mu_1 = 0.8; \mu_2 = 1 + 0.1 \frac{a}{b};$ $\mu_3 = 1 + 0.5 \frac{a}{b_1}$, bet ne daugiau: 4.0 – santvaroms ir sijoms, kai norminis stogo svoris 1,5 kPa ir mažiau; 2,5 – santvaroms ir sijoms, kai norminis stogo svoris daugiau kaip 1,5 kPa; 2.0 – gelžbetoninėms stogo plokštėms 6 m bei trumpesnėms ir plieniniam profiliuotam paklotui; 2.5 – gelžbetoninėms stogo plokštėms ilgesnėms kaip 6 m, taip pat ir sijoms neatsižvelgiant į aną; $b_1 = h_1$, bet ne daugiau b . Nustatant apkrovą ties stoglangio galu, B zonai μ koeficiento reikšmė abiem atvejais imamas lygus 1,0.

	<p>1 variantas $\mu=1$ A zona C zona</p> <p>2 variantas A zona C zona</p> <p>+</p>	<p>Pastabos:</p> <p>1, 2 variantų schemas reikia taip pat taikyti dvišlaičiams ir skliautiniams dviejų-trijų tarpinių statinių stogams su stoglangiais statinių viduryje.</p> <p>Vėją sulaikančių skydų įtaka sniego apkrovos pasiskirstymui greta stoglangių neįvertinama.</p> <p>Plokštiesiems šlaitams, kai $b > 48$ m, reikia įvertinti vietinę padidintą apkrovą ties stoglangiu, kaip ir ties peraukštėjimu (žr. 8 schema).</p>
4.	<p>Šediniai stogai</p> <p>1 variantas $\mu=1$</p> <p>2 variantas $\mu=1,4$ $\mu=0,6$ $\mu=1,4$ $\mu=0,6$</p> <p>$0,5l$ $0,5l$ $0,5l$ $0,5l$</p>	<p>Schemas reikia taikyti šediniams stogams, taip pat su pasvirusiu išstiklinimu ir skliautinio kontūro stogu</p>
5.	<p>Dviejų tarpinių ir daugiatarpsniai statiniai su dvišlaičiais stogais</p> <p>1 variantas $\mu=1$</p> <p>2 variantas $\mu=1,4$ $\mu=0,6$ $\mu=0,6$</p> <p>$0,5l$ l $0,5l$</p>	<p>2 variantas taikomas, kai $\alpha \geq 15^\circ$.</p>
6.	<p>Dviejų tarpinių ir daugiatarpsniai statiniai su skliautiniais ir artimo jiems kontūro stogais</p>	

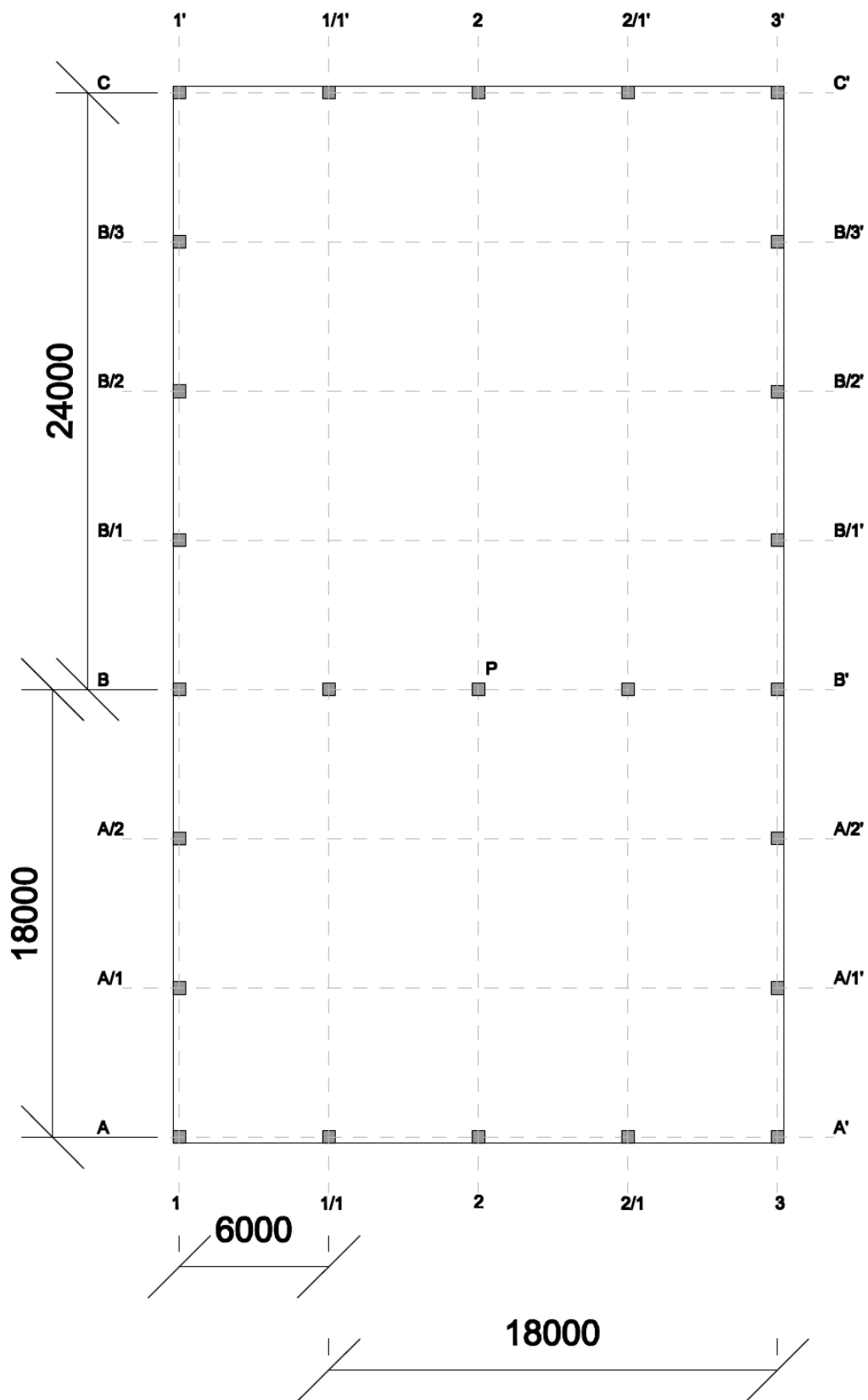
	<p>1 variantas $\mu=1$</p> <p>2 variantas $\mu=2$</p> <p>$\mu=2$ $\mu=2$ $\mu=2$</p> <p>$0,5l$ $0,5l$ $0,5l$ $0,5l$</p>	<p>2 variantą reikia įvertinti, kai $f/l > 0,1$. Gelžbetoninėms stogo plokštėms koeficientus μ reikia imti ne didesnius kaip 1,4.</p>
7.	<p>Dviejų tarpinių ir daugiatarpsniai statiniai su dvišlaičiais ir skliautiniiais stogais su išilginiais stoglangiais</p>	<p>Koeficientą μ reikia imti tarpiniams su stoglangiais pagal 3 schemas 1 ir 2 variantus, tarpiniams be stoglangių – pagal 5 ir 6 schemas 1 ir 2 variantus.</p> <p>Plokštiesiems dvišlaičiams ($\alpha < 15^\circ$) ir skliautiniams ($f/l < 0,1$) stogams, kai $l' > 48$ m, reikia įvertinti vietinę padidintą apkrovą, kaip ir ties perkryčiais (žr. 8 schema).</p>
8.	<p>Statiniai su aukščių perkryčiu a)</p>	<p>Viršutinio stogo sniego apkrovą reikia imti pagal 1–7 schemas, o apatiniam – kaip nepalankiausią pagal 1–7 arba 8 schemas.</p> <p>Koeficientas μ imamas lygus:</p> $\mu_0 = 1 + \frac{1}{h} (m_1 l'_1 + m_2 l'_2),$ <p>bet neturi viršyti:</p> $\frac{2h}{s_k}$ <p>(čia h – m, s_k – kPa);</p> <p>4 – statiniams (a profilis); 6 – stoginėms (b profilis).</p> <p>Viršutinio (apatinio) stogo reikšmės m_1 (m_2) atsižvelgiant į jo profilį reikia imti lygias:</p> <p>0,5 – plokštiesiems stogams su $\alpha \leq 20^\circ$ ir skliautiniams su $f/l \leq 1/8$; 0,3 – plokštiesiems stogams su $\alpha > 20^\circ$, skliautiniams – su $f/l > 1/8$ ir stogams su skersiniais stoglangiais.</p> <p>Apatiniams stogams, kai plotis $a < 21$ m (b</p>

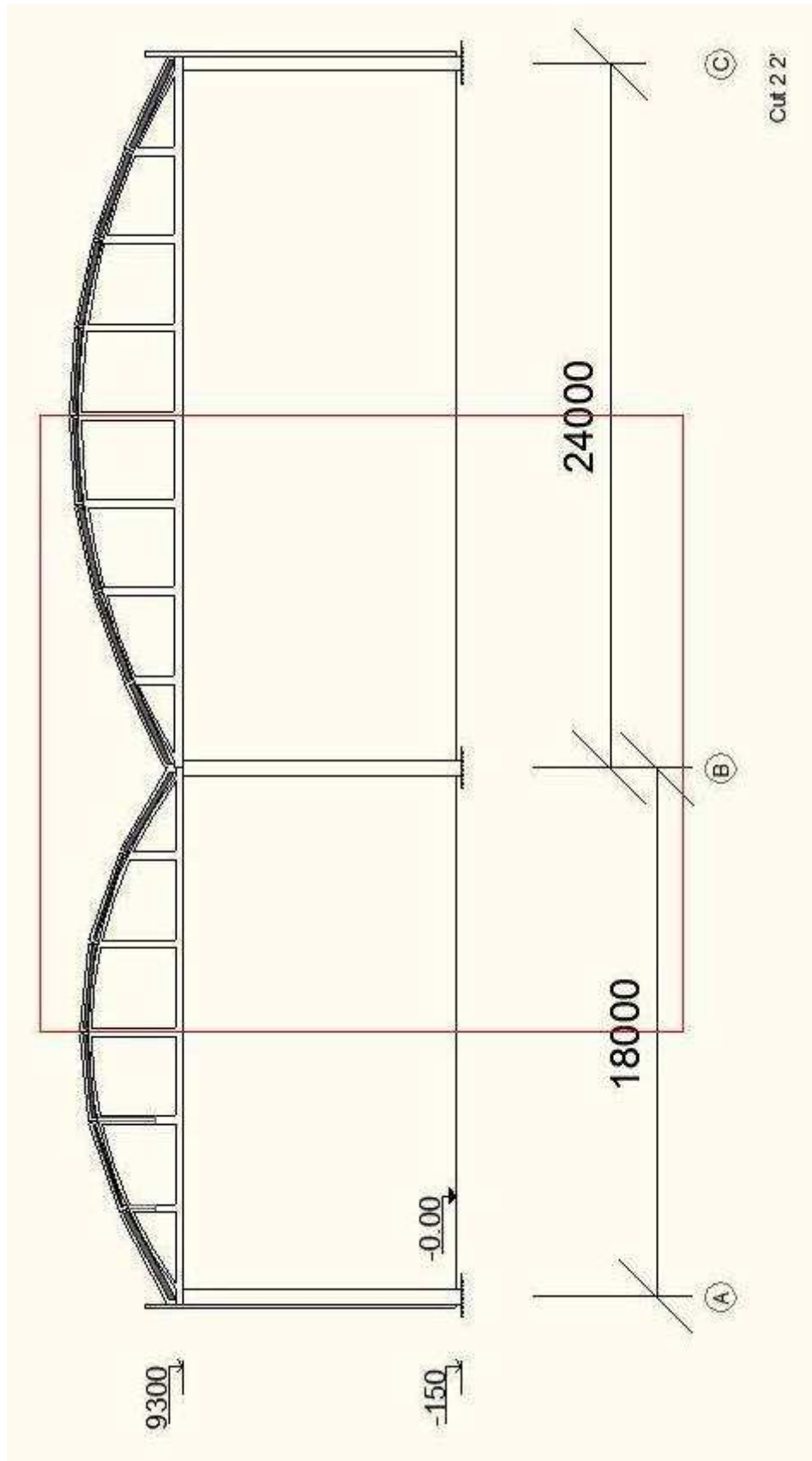
Annex 2

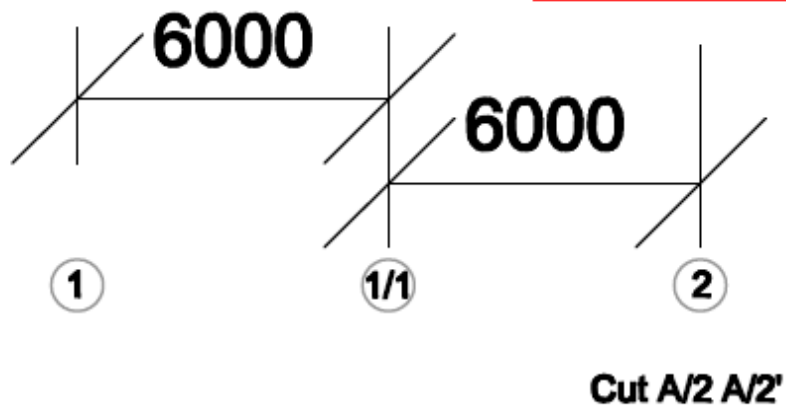
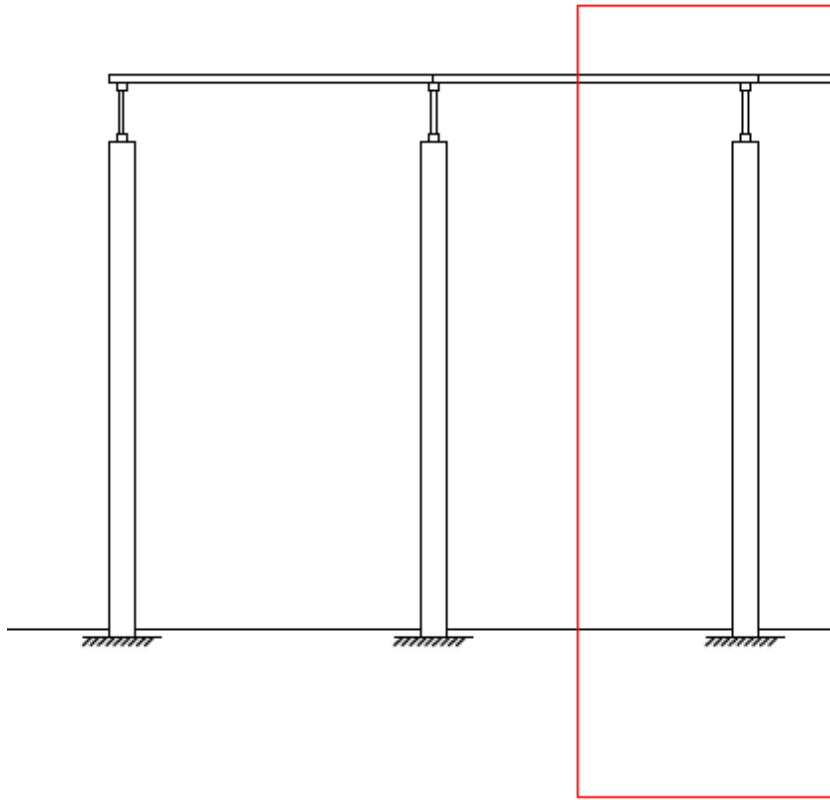
Construction Technical Regulations of Lithuania (Wind Pressure Analysis)

Annex 3

Industrial Building Project Draws








Annex 4

Bearing Capacity Calculations in Drained Conditions

Layer 6

Combinations	Vd (kN)	φ' (°)	c' (kPa)	γ (kN/m ³)
1 a)	792.124	27.0	84	23.2
1 b)	616.616	20.7	67.2	23.2
2	792.124	27.0	84	23.2
3	790.750	20.7	67.2	23.2


Combinations	Nc	Nq	N γ	Sc	Sq	S γ	bc	bq	b γ	lc	lq	l γ
1 a)	23.942	13.199	12.432	1.491	1.454	0.700	1	1	1	1	1	1
1 b)	15.468	6.831	4.397	1.413	1.353	0.700	1	1	1	1	1	1
2	23.942	13.199	12.432	1.491	1.454	0.700	1	1	1	1	1	1
3	15.468	6.831	4.397	1.413	1.353	0.700	1	1	1	1	1	1

Combinations	Rv (kN)	Rv,d (kN)	R/A' (kPa)		Rv,d/ V _d
1 a)	19722.984	19722.984	3728.352		24.899
1 b)	9303.070	9303.070	1758.614		15.087
2	19722.984	14087.846	3728.352		17.785
3	9303.070	9303.070	1758.614		11.765

Layer 7

Combinations	Vd (kN)	φ' (°)	c' (kPa)	γ (kN/m ³)
1 a)	792.124	27.0	84	23.3
1 b)	616.616	20.7	67.2	23.3
2	792.124	27.0	84	23.3
3	790.750	20.7	67.2	23.3


Combinations	Nc	Nq	N γ	Sc	Sq	S γ	bc	bq	b γ	lc	lq	l γ
1 a)	23.942	13.199	12.432	1.491	1.454	0.700	1	1	1	1	1	1
1 b)	15.468	6.831	4.397	1.413	1.353	0.700	1	1	1	1	1	1
2	23.942	13.199	12.432	1.491	1.454	0.700	1	1	1	1	1	1
3	15.468	6.831	4.397	1.413	1.353	0.700	1	1	1	1	1	1

Combinations	Rv (kN)	Rv,d (kN)	R/A' (kPa)		Rv,d/ V _d
1 a)	19728.278	19728.278	3729.353		24.906
1 b)	9304.943	9304.943	1758.968		15.090
2	19728.278	14091.627	3729.353		17.790
3	9304.943	9304.943	1758.968		11.767

Layer 8

Combinations	Vd (kN)	φ' (°)	c' (kPa)	γ (kN/m ³)
1 a)	792.124	37.0	4	20.7
1 b)	616.616	27.3	3.2	20.7
2	792.124	37.0	4	20.7
3	790.750	27.3	3.2	20.7

Combinations	Nc	Nq	N γ	Sc	Sq	S γ	bc	bq	b γ	lc	iq	i γ
1 a)	55.630	42.920	63.178	1.616	1.602	0.700	1	1	1	1	1	1
1 b)	24.521	13.668	13.089	1.495	1.459	0.700	1	1	1	1	1	1
2	55.630	42.920	63.178	1.616	1.602	0.700	1	1	1	1	1	1
3	24.521	13.668	13.089	1.495	1.459	0.700	1	1	1	1	1	1


Combinations	Rv (kN)	Rv,d (kN)	R/A' (kPa)		Rv,d/ V _d
1 a)	16892.844	16892.844	3193.354		21.326
1 b)	4142.785	4142.785	783.135		6.719
2	16892.844	12066.317	3193.354		15.233
3	4142.785	4142.785	783.135		5.239

Annex 5

Bearing Capacity Calculations In Undrained Conditions


Layer 5 - Area Dimensions (2.5x2.5 m)

Combinations	Vd (kN)	c' (kPa)	Cu (kPa)	bc	Sc	ic
1 a)	581.195	18	12.85714	1	1.2	1
1 b)	581.195	18	12.85714	1	1.2	1
2	581.195	18	18	1	1.2	1
3	581.195	18	12.85714	1	1.2	1

Combinations	Rv	R/A'		Rv / V _d
1 a)	603.7339345	96.59743		1.03878
1 b)	603.7339345	96.59743		1.03878
2	802.0525082	128.3284		1.380006
3	603.7339345	96.59743		1.03878


Layer 5 - Area Dimensions (3.0x3.0 m)

Combinations	Vd (kN)	c' (kPa)	Cu (kPa)	bc	Sc	ic
1 a)	615.570	18	12.85714	1	1.2	1
1 b)	615.57	18	12.85714	1	1.2	1
2	615.57	18	18	1	1.2	1
3	615.57	18	12.85714	1	1.2	1

Combinations	Rv	R/A'		Rv / V _d
1 a)	869.3768656	96.59743		1.412312
1 b)	869.3768656	96.59743		1.412312
2	1154.955612	128.3284		1.876238
3	869.3768656	96.59743		1.412312


Layer 5 - Area Dimensions (3.1x3.1 m)

Combinations	Vd (kN)	c' (kPa)	Cu (kPa)	bc	Sc	ic
1 a)	623.195	18	12.85714	1	1.2	1
1 b)	623.195	18	12.85714	1	1.2	1
2	623.195	18	18	1	1.2	1
3	623.195	18	12.85714	1	1.2	1

Combinations	Rv	R/A'		Rv /V _d
1 a)	928.3012976	96.59743		1.489584
1 b)	928.3012976	96.59743		1.489584
2	1233.235937	128.3284		1.978893
3	928.3012976	96.59743		1.489584

Layer 5 - Area Dimensions (3.2x3.2 m)

Combinations	Vd (kN)	c' (kPa)	Cu (kPa)	bc	Sc	ic
1 a)	631.070	18	12.85714	1	1.2	1
1 b)	631.07	18	12.85714	1	1.2	1
2	631.07	18	18	1	1.2	1
3	631.07	18	12.85714	1	1.2	1

Combinations	Rv	R/A'		Rv /V _d
1 a)	989.1576782	96.59743		1.567429
1 b)	989.1576782	96.59743		1.567429
2	1314.082829	128.3284		2.082309
3	989.1576782	96.59743		1.567429

Layer 6 - Area Dimensions (3.2x3.2x0.7 m)

Combinations	Vd (kN)	c' (kPa)	Cu (kPa)	bc	Sc	ic
1 a)	678.780	84	60	1	1.2	1
1 b)	678.78	84	60	1	1.2	1
2	678.78	84	84	1	1.2	1
3	678.78	84	60	1	1.2	1

Combinations	Rv	R/A'
1 a)	3967.638232	387.4647
1 b)	3967.638232	387.4647
2	5483.955604	535.5425
3	3967.638232	387.4647



Rv/V _d
5.845249
5.845249
8.079136
5.845249

Layer 7 - Area Dimensions (3.2x3.2x0.7 m)

Combinations	Vd (kN)	c' (kPa)	Cu (kPa)	bc	Sc	ic
1 a)	678.780	84	60	1	1.2	1
1 b)	678.78	84	60	1	1.2	1
2	678.78	84	84	1	1.2	1
3	678.78	84	60	1	1.2	1

Combinations	Rv	R/A'
1 a)	3967.638232	387.4647
1 b)	3967.638232	387.4647
2	5483.955604	535.5425
3	3967.638232	387.4647



Rv/V _d
5.845249
5.845249
8.079136
5.845249

Layer 8 - Area Dimensions (3.2x3.2x0.7 m)

Combinations	Vd (kN)	c' (kPa)	Cu (kPa)	bc	Sc	ic
1 a)	678.780	4	2.857143	1	1.2	1
1 b)	678.78	4	2.857143	1	1.2	1
2	678.78	4	4	1	1.2	1
3	678.78	4	2.857143	1	1.2	1

Combinations	Rv	R/A'
1 a)	357.3587729	34.89832
1 b)	357.3587729	34.89832
2	429.5643621	41.94964
3	357.3587729	34.89832



Rv / V _d
0.526472
0.526472
0.632848
0.526472

Annex 6

Parameters for the calculation of stress increments due to a uniform load applied on a rectangular area

Values of K_1

z/b	l/b															
	0	0,1	0,2	1/3	0,4	0,5	2/3	1	1,5	2	2,5	3	5	10	∞	
0	0,000	0,250	0,250	0,250	0,250	0,250	0,250	0,250	0,250	0,250	0,250	0,250	0,250	0,250	0,250	
0,2	0,000	0,137	0,204	0,234	0,240	0,244	0,247	0,249	0,249	0,249	0,249	0,249	0,249	0,249	0,249	
0,4	0,000	0,076	0,136	0,187	0,202	0,218	0,231	0,240	0,243	0,244	0,244	0,244	0,244	0,244	0,244	
0,5	0,000	0,061	0,113	0,164	0,181	0,200	0,218	0,232	0,238	0,239	0,240	0,240	0,240	0,240	0,240	
0,6	0,000	0,051	0,096	0,143	0,161	0,182	0,204	0,223	0,231	0,233	0,234	0,234	0,234	0,234	0,234	
0,8	0,000	0,037	0,071	0,111	0,127	0,148	0,173	0,200	0,214	0,218	0,219	0,220	0,220	0,220	0,220	
1	0,000	0,028	0,055	0,087	0,101	0,120	0,145	0,175	0,194	0,200	0,202	0,203	0,204	0,205	0,205	
1,2	0,000	0,022	0,043	0,069	0,081	0,098	0,121	0,152	0,173	0,182	0,185	0,187	0,189	0,189	0,189	
1,4	0,000	0,018	0,035	0,056	0,066	0,080	0,101	0,131	0,154	0,164	0,169	0,171	0,174	0,174	0,174	
1,5	0,000	0,016	0,031	0,051	0,060	0,073	0,092	0,121	0,145	0,156	0,161	0,164	0,166	0,167	0,167	
1,6	0,000	0,014	0,028	0,046	0,055	0,067	0,085	0,112	0,136	0,148	0,154	0,157	0,160	0,160	0,160	
1,8	0,000	0,012	0,024	0,039	0,046	0,056	0,072	0,097	0,121	0,133	0,140	0,143	0,147	0,148	0,148	
2	0,000	0,010	0,020	0,033	0,039	0,048	0,061	0,084	0,107	0,120	0,127	0,131	0,136	0,137	0,137	
2,5	0,000	0,007	0,013	0,022	0,027	0,033	0,043	0,060	0,080	0,093	0,101	0,106	0,113	0,115	0,115	
3	0,000	0,005	0,010	0,016	0,019	0,024	0,031	0,045	0,061	0,073	0,081	0,087	0,096	0,099	0,099	
4	0,000	0,003	0,006	0,009	0,011	0,014	0,019	0,027	0,038	0,048	0,055	0,060	0,071	0,076	0,076	
5	0,000	0,002	0,004	0,006	0,007	0,009	0,012	0,018	0,026	0,033	0,039	0,043	0,055	0,061	0,062	
10	0,000	0,000	0,001	0,002	0,002	0,002	0,003	0,005	0,007	0,009	0,011	0,013	0,020	0,028	0,032	
15	0,000	0,000	0,000	0,001	0,001	0,001	0,001	0,002	0,003	0,004	0,005	0,006	0,010	0,016	0,021	
20	0,000	0,000	0,000	0,000	0,000	0,001	0,001	0,001	0,002	0,002	0,003	0,004	0,006	0,010	0,016	
50	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,001	0,001	0,002	0,006	

Values of K_2

z/b	l/b															
	0	0,1	0,2	1/3	0,4	0,5	2/3	1	1,5	2	2,5	3	5	10	∞	
0	0,000	0,250	0,250	0,250	0,250	0,250	0,250	0,250	0,250	0,250	0,250	0,250	0,250	0,250	0,250	
0,2	0,000	0,069	0,116	0,149	0,159	0,169	0,177	0,184	0,187	0,188	0,188	0,188	0,188	0,188	0,188	
0,4	0,000	0,031	0,058	0,085	0,095	0,106	0,118	0,128	0,133	0,134	0,134	0,134	0,135	0,135	0,135	
0,5	0,000	0,022	0,043	0,064	0,073	0,083	0,094	0,105	0,110	0,112	0,112	0,112	0,113	0,113	0,113	
0,6	0,000	0,017	0,032	0,049	0,056	0,065	0,075	0,086	0,091	0,093	0,093	0,094	0,094	0,094	0,094	
0,8	0,000	0,009	0,018	0,029	0,034	0,040	0,047	0,057	0,062	0,064	0,064	0,065	0,065	0,065	0,065	
1	0,000	0,006	0,011	0,018	0,021	0,025	0,030	0,037	0,042	0,044	0,045	0,045	0,045	0,045	0,045	
1,2	0,000	0,003	0,007	0,011	0,013	0,016	0,020	0,025	0,029	0,031	0,032	0,032	0,032	0,032	0,032	
1,4	0,000	0,002	0,004	0,007	0,008	0,010	0,013	0,017	0,020	0,022	0,023	0,023	0,023	0,023	0,023	
1,5	0,000	0,002	0,004	0,006	0,007	0,008	0,011	0,014	0,017	0,019	0,019	0,020	0,020	0,020	0,020	
1,6	0,000	0,001	0,003	0,005	0,006	0,007	0,009	0,012	0,015	0,016	0,017	0,017	0,017	0,017	0,017	
1,8	0,000	0,001	0,002	0,003	0,004	0,005	0,006	0,008	0,011	0,012	0,012	0,013	0,013	0,013	0,013	
2	0,000	0,001	0,001	0,002	0,003	0,003	0,004	0,006	0,008	0,009	0,009	0,010	0,010	0,010	0,010	
2,5	0,000	0,000	0,001	0,001	0,001	0,002	0,002	0,003	0,004	0,005	0,005	0,005	0,006	0,006	0,006	
3	0,000	0,000	0,000	0,001	0,001	0,001	0,001	0,002	0,002	0,003	0,003	0,003	0,003	0,003	0,003	
4	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,001	0,001	0,001	0,001	0,001	0,001	0,002	0,002	
5	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,001	0,001	0,001	0,001	0,001	
10	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	
15	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	
20	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	
50	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	

Values of K_2^1

i/b z/b	0	0,1	0,2	1/3	0,4	0,5	2/3	1	1,5	2	2,5	3	5	10	∞
0	0,000	0,234	0,219	0,199	0,189	0,176	0,156	0,125	0,094	0,074	0,061	0,051	0,031	0,016	0,000
0,2	0,000	0,059	0,097	0,118	0,121	0,122	0,118	0,103	0,082	0,067	0,056	0,048	0,030	0,016	0,000
0,4	0,000	0,026	0,048	0,069	0,075	0,082	0,086	0,083	0,071	0,060	0,051	0,045	0,029	0,015	0,000
0,5	0,000	0,019	0,036	0,054	0,060	0,067	0,073	0,074	0,066	0,056	0,049	0,043	0,028	0,015	0,000
0,6	0,000	0,015	0,028	0,043	0,049	0,056	0,062	0,066	0,061	0,053	0,047	0,041	0,028	0,015	0,000
0,8	0,000	0,009	0,018	0,029	0,033	0,039	0,046	0,052	0,052	0,047	0,043	0,038	0,026	0,015	0,000
1	0,000	0,007	0,013	0,021	0,024	0,029	0,035	0,042	0,044	0,042	0,039	0,035	0,025	0,014	0,000
1,2	0,000	0,005	0,009	0,015	0,018	0,022	0,027	0,034	0,037	0,037	0,035	0,032	0,024	0,014	0,000
1,4	0,000	0,004	0,007	0,012	0,014	0,017	0,021	0,027	0,032	0,033	0,032	0,030	0,023	0,014	0,000
1,5	0,000	0,003	0,006	0,010	0,012	0,015	0,019	0,025	0,029	0,031	0,030	0,029	0,022	0,014	0,000
1,6	0,000	0,003	0,006	0,009	0,011	0,013	0,017	0,023	0,027	0,029	0,028	0,027	0,022	0,013	0,000
1,8	0,000	0,002	0,005	0,007	0,009	0,011	0,014	0,019	0,024	0,025	0,026	0,025	0,021	0,013	0,000
2	0,000	0,002	0,004	0,006	0,007	0,009	0,012	0,016	0,020	0,023	0,023	0,023	0,020	0,013	0,000
2,5	0,000	0,001	0,002	0,004	0,005	0,006	0,008	0,011	0,015	0,017	0,018	0,019	0,017	0,012	0,000
3	0,000	0,001	0,002	0,003	0,003	0,004	0,006	0,008	0,011	0,013	0,015	0,015	0,015	0,011	0,000
4	0,000	0,000	0,001	0,002	0,002	0,002	0,003	0,005	0,007	0,008	0,010	0,011	0,012	0,010	0,000
5	0,000	0,000	0,001	0,001	0,001	0,002	0,002	0,003	0,004	0,006	0,007	0,008	0,009	0,009	0,000
10	0,000	0,000	0,000	0,000	0,000	0,000	0,001	0,001	0,001	0,002	0,002	0,002	0,003	0,005	0,000
15	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,001	0,001	0,001	0,001	0,002	0,003	0,000
20	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,001	0,001	0,002	0,000
50	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000

Values of L_2

i/b z/b	0	0,1	0,2	1/3	0,4	0,5	2/3	1	1,5	2	2,5	3	5	10	∞
0	0,000	0,250	0,250	0,250	0,250	0,250	0,250	0,250	0,250	0,250	0,250	0,250	0,250	0,250	0,250
0,2	0,000	0,010	0,045	0,094	0,112	0,134	0,158	0,184	0,201	0,208	0,211	0,214	0,217	0,218	0,219
0,4	0,000	0,002	0,010	0,032	0,045	0,064	0,091	0,128	0,156	0,169	0,176	0,179	0,186	0,188	0,189
0,5	0,000	0,001	0,006	0,020	0,029	0,044	0,068	0,105	0,136	0,151	0,159	0,164	0,172	0,175	0,176
0,6	0,000	0,000	0,003	0,013	0,019	0,031	0,051	0,086	0,118	0,134	0,144	0,149	0,158	0,163	0,164
0,8	0,000	0,000	0,001	0,006	0,009	0,016	0,029	0,057	0,087	0,106	0,117	0,124	0,135	0,141	0,143
1	0,000	0,000	0,001	0,003	0,005	0,009	0,017	0,037	0,064	0,083	0,095	0,103	0,116	0,123	0,125
1,2	0,000	0,000	0,000	0,002	0,003	0,005	0,011	0,025	0,047	0,065	0,077	0,085	0,100	0,108	0,111
1,4	0,000	0,000	0,000	0,001	0,002	0,003	0,007	0,017	0,035	0,051	0,062	0,071	0,087	0,095	0,099
1,5	0,000	0,000	0,000	0,001	0,001	0,003	0,005	0,014	0,030	0,045	0,056	0,064	0,081	0,090	0,094
1,6	0,000	0,000	0,000	0,001	0,001	0,002	0,004	0,012	0,026	0,040	0,051	0,059	0,076	0,085	0,089
1,8	0,000	0,000	0,000	0,000	0,001	0,001	0,003	0,008	0,020	0,031	0,041	0,049	0,066	0,077	0,081
2	0,000	0,000	0,000	0,000	0,001	0,001	0,002	0,006	0,015	0,025	0,034	0,041	0,058	0,069	0,074
2,5	0,000	0,000	0,000	0,000	0,000	0,000	0,001	0,003	0,008	0,014	0,021	0,027	0,043	0,055	0,061
3	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,002	0,004	0,008	0,013	0,018	0,032	0,045	0,051
4	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,001	0,002	0,003	0,006	0,008	0,018	0,031	0,039
5	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,001	0,002	0,003	0,004	0,011	0,022	0,031
10	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,001	0,006	0,016
15	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,002	0,011
20	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,001	0,008
50	0,000	0,000	0,000	0,0>0	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,003

Values of L_2^1

l/b z/b	0	0,1	0,2	1/3	0,4	0,5	2/3	1	1,5	2	2,5	3	5	10	∞
0	0,000	0,016	0,031	0,051	0,061	0,074	0,094	0,125	0,156	0,176	0,189	0,199	0,219	0,234	0,250
0,2	0,000	0,013	0,025	0,041	0,049	0,060	0,076	0,103	0,130	0,148	0,160	0,169	0,188	0,203	0,219
0,4	0,000	0,010	0,020	0,032	0,039	0,047	0,061	0,083	0,106	0,122	0,133	0,141	0,159	0,174	0,189
0,5	0,000	0,009	0,017	0,029	0,034	0,042	0,054	0,074	0,096	0,111	0,121	0,129	0,146	0,161	0,176
0,6	0,000	0,008	0,015	0,025	0,030	0,037	0,048	0,066	0,086	0,100	0,110	0,118	0,134	0,149	0,164
0,8	0,000	0,006	0,012	0,020	0,023	0,029	0,037	0,052	0,069	0,082	0,091	0,098	0,114	0,127	0,143
1	0,000	0,005	0,009	0,015	0,018	0,023	0,029	0,042	0,056	0,067	0,075	0,082	0,097	0,110	0,125
1,2	0,000	0,004	0,007	0,012	0,015	0,018	0,024	0,034	0,046	0,056	0,063	0,069	0,083	0,096	0,111
1,4	0,000	0,003	0,006	0,010	0,012	0,015	0,019	0,027	0,038	0,046	0,053	0,058	0,072	0,084	0,099
1,5	0,000	0,003	0,005	0,009	0,011	0,013	0,017	0,025	0,035	0,043	0,049	0,054	0,067	0,079	0,094
1,6	0,000	0,002	0,005	0,008	0,010	0,012	0,016	0,023	0,032	0,039	0,045	0,050	0,062	0,074	0,089
1,8	0,000	0,002	0,004	0,007	0,008	0,010	0,013	0,019	0,027	0,033	0,039	0,043	0,055	0,066	0,081
2	0,000	0,002	0,003	0,006	0,007	0,008	0,011	0,016	0,023	0,029	0,033	0,038	0,048	0,059	0,074
2,5	0,000	0,001	0,002	0,004	0,005	0,006	0,007	0,011	0,016	0,020	0,024	0,027	0,036	0,047	0,061
3	0,000	0,001	0,002	0,003	0,003	0,004	0,005	0,008	0,012	0,015	0,018	0,021	0,028	0,038	0,051
4	0,000	0,000	0,001	0,002	0,002	0,002	0,003	0,005	0,007	0,009	0,011	0,013	0,018	0,026	0,039
5	0,000	0,000	0,001	0,001	0,001	0,002	0,002	0,003	0,005	0,006	0,007	0,009	0,013	0,019	0,031
10	0,000	0,000	0,000	0,000	0,000	0,000	0,001	0,001	0,001	0,002	0,002	0,002	0,004	0,007	0,016
15	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,001	0,001	0,001	0,001	0,002	0,003	0,011
20	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,001	0,001	0,002	0,008
50	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,003

Annex 7

Values of the factor I_s

Shape of Footing	I_y . Footing infinitely flexible (uniform load)					I_s Rigid footing
	centre	vertex	middle of smaller size	middle of bigger size	average	
circular	1.00	---	0.64	0.64	0.85	0.79
square	1.12	0.56	0.77	0.77	0.95	0.92
rectangular $L/B = 1.5$	1.36	0.68	0.89	0.97	1.15	1.13
= 2.0	1.53	0.77	0.98	1.12	1.30	1.27
= 2.5	1.67	0.83	1.05	1.25	?	?
= 3.0	1.78	0.89	1.11	1.36	1.52	1.51
= 4.0	1.97	0.98	1.20	1.53	?	?
= 5.0	2.10	1.05	1.27	1.67	1.83	1.81
= 7.0	?	?	?	?	?	2.01
= 10.0	2.54	1.27	1.49	2.10	2.25	2.25

Annex 8

Design Tables of Concrete Structures
subject to Simple Bending

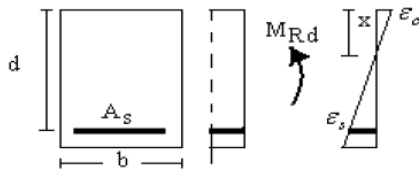
Tabelas de Dimensionamento de Estruturas de Betão

TABELA 1_S500

FLEXÃO SIMPLES

Secções rectangulares simplesmente armadas

S400; S500; S600



$$\alpha = \frac{x}{d}; \mu = \frac{M_{R,d}}{bd^2 f_{cd}}; \varpi = \frac{A_s f_{yd}}{bd f_{cd}}$$

$$f_{yd} = f_{yk} / 1.15; f_{cd} = f_{ck} / 1.5$$

μ	C12-C50		C55		C60		C70		C80		C90	
	α	ϖ	α	ϖ	α	ϖ	α	ϖ	α	ϖ	α	ϖ
0,005	0,021	0,005	0,023	0,005	0,024	0,005	0,026	0,005	0,027	0,005	0,027	0,005
0,010	0,030	0,010	0,033	0,010	0,035	0,010	0,037	0,010	0,038	0,010	0,039	0,010
0,015	0,037	0,015	0,041	0,015	0,043	0,015	0,046	0,015	0,047	0,015	0,048	0,015
0,020	0,044	0,020	0,048	0,020	0,050	0,020	0,053	0,020	0,055	0,020	0,055	0,020
0,025	0,050	0,026	0,054	0,025	0,057	0,026	0,060	0,026	0,062	0,026	0,062	0,025
0,030	0,055	0,031	0,060	0,031	0,063	0,031	0,066	0,031	0,068	0,031	0,069	0,030
0,035	0,061	0,036	0,065	0,036	0,068	0,036	0,072	0,036	0,074	0,036	0,075	0,036
0,040	0,066	0,041	0,071	0,041	0,074	0,041	0,077	0,041	0,079	0,041	0,080	0,041
0,045	0,071	0,046	0,076	0,046	0,079	0,046	0,082	0,046	0,084	0,046	0,086	0,046
0,050	0,076	0,052	0,081	0,052	0,084	0,052	0,087	0,052	0,090	0,052	0,091	0,051
0,055	0,081	0,057	0,086	0,057	0,089	0,057	0,092	0,057	0,095	0,057	0,097	0,057
0,060	0,087	0,062	0,091	0,062	0,094	0,062	0,098	0,062	0,104	0,062	0,106	0,062
0,065	0,092	0,067	0,096	0,067	0,099	0,068	0,106	0,068	0,113	0,068	0,115	0,067
0,070	0,097	0,073	0,102	0,073	0,105	0,073	0,115	0,073	0,122	0,073	0,125	0,073
0,075	0,102	0,078	0,107	0,078	0,113	0,078	0,123	0,078	0,131	0,079	0,134	0,078
0,080	0,107	0,084	0,113	0,084	0,121	0,084	0,132	0,084	0,140	0,084	0,144	0,084
0,085	0,113	0,089	0,120	0,089	0,128	0,089	0,140	0,090	0,150	0,090	0,153	0,090
0,090	0,118	0,095	0,128	0,095	0,137	0,095	0,149	0,095	0,159	0,095	0,163	0,095
0,095	0,124	0,100	0,135	0,100	0,144	0,100	0,158	0,101	0,168	0,101	0,173	0,101
0,100	0,131	0,106	0,143	0,106	0,153	0,106	0,167	0,106	0,178	0,107	0,183	0,107
0,105	0,137	0,111	0,150	0,111	0,161	0,112	0,176	0,112	0,188	0,112	0,192	0,112
0,110	0,144	0,117	0,158	0,117	0,169	0,117	0,185	0,118	0,197	0,118	0,202	0,118
0,115	0,151	0,123	0,166	0,123	0,177	0,123	0,194	0,124	0,207	0,124	0,212	0,124
0,120	0,159	0,128	0,173	0,129	0,186	0,129	0,203	0,129	0,217	0,130	0,223	0,130
0,125	0,166	0,134	0,181	0,135	0,194	0,135	0,212	0,135	0,227	0,136	0,233	0,136
0,130	0,173	0,140	0,189	0,140	0,202	0,141	0,222	0,141	0,237	0,142	0,243	0,142
0,135	0,180	0,146	0,197	0,146	0,211	0,147	0,231	0,147	0,247	0,148	0,254	0,148
0,140	0,188	0,152	0,205	0,152	0,220	0,153	0,241	0,153	0,257	0,154	0,264	0,154
0,145	0,195	0,158	0,213	0,158	0,228	0,159	0,250	0,159	0,267	0,160	0,275	0,160
0,150	0,202	0,164	0,221	0,164	0,237	0,165	0,260	0,165	0,277	0,166	0,285	0,166
0,155	0,210	0,170	0,229	0,170	0,246	0,171	0,269	0,172	0,288	0,173	0,296	0,173
0,160	0,217	0,176	0,238	0,176	0,255	0,177	0,279	0,178	0,298	0,179	0,307	0,179
0,165	0,225	0,182	0,246	0,183	0,264	0,183	0,289	0,184	0,309	0,185	0,318	0,185
0,170	0,232	0,188	0,254	0,189	0,272	0,189	0,299	0,191	0,320	0,192	0,329	0,192
0,175	0,240	0,194	0,263	0,195	0,282	0,196	0,309	0,197	0,331	0,198	0,340	0,199
0,180	0,248	0,201	0,271	0,201	0,291	0,202	0,319	0,203	0,342	0,205	0,352	0,205
0,185	0,256	0,207	0,280	0,208	0,300	0,209	0,330	0,210	0,353	0,211	0,363	0,212
0,190	0,264	0,213	0,289	0,214	0,309	0,215	0,340	0,217	0,364	0,218	0,375	0,219
0,195	0,271	0,220	0,297	0,221	0,319	0,222	0,350	0,223	0,375	0,225	0,386	0,225
0,200	0,280	0,226	0,306	0,227	0,328	0,228	0,361	0,230	0,387	0,232	0,398	0,232
0,205	0,288	0,233	0,315	0,234	0,338	0,235	0,372	0,237	0,398	0,239	0,410	0,239
0,210	0,296	0,239	0,324	0,241	0,348	0,242	0,382	0,244	0,410	0,246	0,422	0,246
0,215	0,304	0,246	0,333	0,247	0,357	0,248	0,393	0,251	0,422	0,253	0,435	0,254
0,220	0,312	0,253	0,342	0,254	0,367	0,255	0,404	0,258	0,434	0,260	0,447	0,261
0,225	0,321	0,260	0,352	0,261	0,377	0,262	0,415	0,265	0,446	0,267	0,460	0,268
0,230	0,329	0,266	0,361	0,268	0,387	0,269	0,427	0,272	0,458	0,275	0,472	0,276
0,235	0,338	0,273	0,370	0,275	0,398	0,276	0,438	0,279	0,470	0,282	0,485	0,283
0,240	0,346	0,280	0,380	0,282	0,408	0,284	0,450	0,287	0,483	0,290	0,499	0,291
0,245	0,355	0,287	0,390	0,289	0,419	0,291	0,461	0,294	0,496	0,297	0,512	0,299
0,250	0,364	0,295	0,399	0,296	0,429	0,298	0,473	0,302	0,509	0,305	0,525	0,306

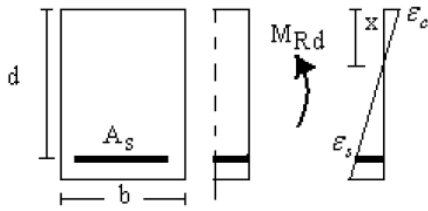
Nota: a taxa mecânica de armadura mínima em vigas é dada pelo maior de:

$$\varpi_{\min} = 0.226 \frac{f_{ctm}}{f_{cd}} \quad \text{e} \quad \varpi_{\min} = 0.0013 \frac{f_{yd}}{f_{cd}} \quad (\text{cláusula 9.2.1.1 do EC2})$$

FLEXÃO SIMPLES

Secções rectangulares simplesmente armadas

S400; S500; S600



$$\alpha = \frac{x}{d}; \quad \mu = \frac{M_{Rd}}{bd^2 f_{cd}}; \quad \varpi = \frac{A_s f_{yd}}{bd f_{cd}};$$

$$f_{yd} = f_{yk} / 1.15; \quad f_{cd} = f_{ck} / 1.5$$

μ	C12-C50		C55		C60		C70		C80		C90	
	α	ϖ	α	ϖ	α	ϖ	α	ϖ	α	ϖ	α	ϖ
0,255	0,373	0,302	0,409	0,304	0,440	0,306	0,485	0,309	0,522	0,313	0,539	0,315
0,260	0,382	0,309	0,419	0,311	0,451	0,313	0,498	0,317	0,535	0,321	0,553	0,334
0,265	0,391	0,316	0,429	0,319	0,462	0,321	0,510	0,325	0,549	0,335	0,567	0,363
0,270	0,400	0,324	0,440	0,326	0,473	0,329	0,522	0,333	0,563	0,363	0,582	0,394
0,275	0,409	0,331	0,450	0,334	0,484	0,336	0,535	0,341	0,577	0,394	0,596	0,430
0,280	0,419	0,339	0,460	0,342	0,495	0,344	0,548	0,349	0,591	0,428	0,611	0,469
0,285	0,428	0,347	0,471	0,349	0,507	0,352	0,561	0,368	0,605	0,465	0,626	0,512
0,290	0,438	0,355	0,482	0,357	0,519	0,360	0,574	0,398	0,620	0,508	0,642	0,561
0,295	0,448	0,362	0,493	0,366	0,531	0,369	0,588	0,431	0,635	0,554	0,658	0,617
0,300	0,458	0,370	0,504	0,374	0,543	0,377	0,602	0,467	0,651	0,607	0,674	0,680
0,305	0,468	0,379	0,515	0,382	0,555	0,386	0,616	0,507	0,666	0,667	0,690	0,751
0,310	0,478	0,387	0,526	0,391	0,567	0,394	0,630	0,551	0,682	0,734	0,707	0,834
0,315	0,488	0,395	0,538	0,399	0,580	0,418	0,645	0,600	0,699	0,812	0,725	0,931
0,320	0,499	0,404	0,550	0,408	0,593	0,450	0,660	0,656	0,715	0,901	0,742	1,044
0,325	0,509	0,412	0,562	0,417	0,606	0,486	0,675	0,718	0,732	1,005	0,761	1,180
0,330	0,520	0,421	0,574	0,426	0,620	0,526	0,690	0,789	0,750	1,128	0,779	1,344
0,335	0,531	0,430	0,586	0,435	0,633	0,569	0,706	0,871	0,768	1,276	0,799	1,547
0,340	0,542	0,439	0,599	0,465	0,647	0,618	0,722	0,964	0,787	1,455	0,819	1,804
0,345	0,554	0,448	0,611	0,501	0,661	0,673	0,739	1,074	0,806	1,678	0,840	2,143
0,350	0,565	0,458	0,624	0,540	0,676	0,734	0,756	1,202	0,826	1,962	0,861	2,601
0,355	0,577	0,467	0,638	0,585	0,691	0,803	0,774	1,357	0,846	2,335	0,883	3,260
0,360	0,589	0,477	0,651	0,634	0,706	0,883	0,792	1,545	0,868	2,849	0,907	4,287
0,365	0,601	0,487	0,665	0,688	0,721	0,973	0,811	1,779	0,890	3,598	0,931	6,111
0,370	0,613	0,497	0,680	0,750	0,737	1,079	0,830	2,075	0,913	4,791		
0,375	0,626	0,528	0,694	0,820	0,754	1,203	0,850	2,468	0,937	7,006		
0,380	0,639	0,570	0,709	0,900	0,771	1,350	0,871	3,006				
0,385	0,653	0,617	0,725	0,992	0,788	1,529	0,892	3,795				
0,390	0,666	0,670	0,740	1,099	0,806	1,748	0,915	5,056				
0,395	0,680	0,729	0,757	1,224	0,825	2,026	0,939	7,391				
0,400	0,695	0,796	0,774	1,375	0,844	2,389						
0,405	0,710	0,873	0,791	1,556	0,865	2,879						
0,410	0,725	0,961	0,809	1,783	0,886	3,583						
0,415	0,741	1,065	0,828	2,069	0,908	4,678						
0,420	0,757	1,188	0,847	2,448	0,932	6,618						
0,425	0,774	1,335	0,868	2,970								
0,430	0,792	1,516	0,890	3,730								
0,435	0,811	1,743	0,913	4,955								
0,440	0,830	2,036	0,937	7,255								
0,445	0,850	2,431										
0,450	0,872	2,995										
0,455	0,896	3,865										
0,460	0,921	5,384										
0,465	0,949	8,800										

Nota: A tabela é válida para os aços S400, S500 e S600 até à linha a cheio (parte superior da tabela). A partir dessa linha deve-se proceder da seguinte forma:

S400 usar as taxas mecânicas de armadura indicadas na tabela entre a linha a cheio e a linha a tracejado. A partir da linha tracejada para obter a taxa mecânica de armadura multiplicar o valor dado na tabela por $f_{yk} / 500 = 400 / 500 = 0,8$;

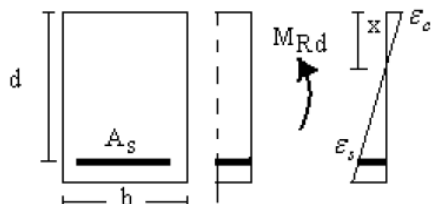
S500 a tabela é válida para toda a gama de valores;

S600 a partir da linha a cheio para obter a taxa mecânica de armadura multiplicar o valor dado na tabela por $f_{yk} / 500 = 600 / 500 = 1,2$.

TABELA 1_S400
FLEXÃO SIMPLES

Seções rectangulares simplesmente armadas

S400



$$\alpha = \frac{x}{d}; \mu = \frac{M_{Rd}}{bd^2 f_{cd}}; \bar{\omega} = \frac{A_s f_{yd}}{bd f_{cd}}$$

$$f_{yd} = f_{yk} / 1.15; f_{cd} = f_{ck} / 1.5$$

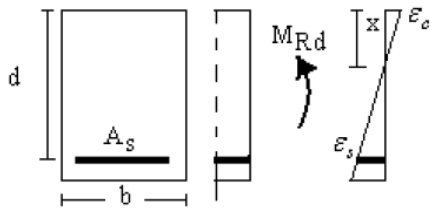
μ	C12-C50		C55		C60		C70		C80		C90	
	α	$\bar{\omega}$	α	$\bar{\omega}$	α	$\bar{\omega}$	α	$\bar{\omega}$	α	$\bar{\omega}$	α	$\bar{\omega}$
0,005	0,021	0,005	0,023	0,005	0,024	0,005	0,026	0,005	0,027	0,005	0,027	0,005
0,010	0,030	0,010	0,033	0,010	0,035	0,010	0,037	0,010	0,038	0,010	0,039	0,010
0,015	0,037	0,015	0,041	0,015	0,043	0,015	0,046	0,015	0,047	0,015	0,048	0,015
0,020	0,044	0,020	0,048	0,020	0,050	0,020	0,053	0,020	0,055	0,020	0,055	0,020
0,025	0,050	0,026	0,054	0,025	0,057	0,026	0,060	0,026	0,062	0,026	0,062	0,025
0,030	0,055	0,031	0,060	0,031	0,063	0,031	0,066	0,031	0,068	0,031	0,069	0,030
0,035	0,061	0,036	0,065	0,036	0,068	0,036	0,072	0,036	0,074	0,036	0,075	0,036
0,040	0,066	0,041	0,071	0,041	0,074	0,041	0,077	0,041	0,079	0,041	0,080	0,041
0,045	0,071	0,046	0,076	0,046	0,079	0,046	0,082	0,046	0,084	0,046	0,086	0,046
0,050	0,076	0,052	0,081	0,052	0,084	0,052	0,087	0,052	0,090	0,052	0,091	0,051
0,055	0,081	0,057	0,086	0,057	0,089	0,057	0,092	0,057	0,095	0,057	0,097	0,057
0,060	0,087	0,062	0,091	0,062	0,094	0,062	0,098	0,062	0,104	0,062	0,106	0,062
0,065	0,092	0,067	0,096	0,067	0,099	0,068	0,106	0,068	0,113	0,068	0,115	0,067
0,070	0,097	0,073	0,102	0,073	0,105	0,073	0,115	0,073	0,122	0,073	0,125	0,073
0,075	0,102	0,078	0,107	0,078	0,113	0,078	0,123	0,078	0,131	0,079	0,134	0,078
0,080	0,107	0,084	0,113	0,084	0,121	0,084	0,132	0,084	0,140	0,084	0,144	0,084
0,085	0,113	0,089	0,120	0,089	0,128	0,089	0,140	0,090	0,150	0,090	0,153	0,090
0,090	0,118	0,095	0,128	0,095	0,137	0,095	0,149	0,095	0,159	0,095	0,163	0,095
0,095	0,124	0,100	0,135	0,100	0,144	0,100	0,158	0,101	0,168	0,101	0,173	0,101
0,100	0,131	0,106	0,143	0,106	0,153	0,106	0,167	0,106	0,178	0,107	0,183	0,107
0,105	0,137	0,111	0,150	0,111	0,161	0,112	0,176	0,112	0,188	0,112	0,192	0,112
0,110	0,144	0,117	0,158	0,117	0,169	0,117	0,185	0,118	0,197	0,118	0,202	0,118
0,115	0,151	0,123	0,166	0,123	0,177	0,123	0,194	0,124	0,207	0,124	0,212	0,124
0,120	0,159	0,128	0,173	0,129	0,186	0,129	0,203	0,129	0,217	0,130	0,223	0,130
0,125	0,166	0,134	0,181	0,135	0,194	0,135	0,212	0,135	0,227	0,136	0,233	0,136
0,130	0,173	0,140	0,189	0,140	0,202	0,141	0,222	0,141	0,237	0,142	0,243	0,142
0,135	0,180	0,146	0,197	0,146	0,211	0,147	0,231	0,147	0,247	0,148	0,254	0,148
0,140	0,188	0,152	0,205	0,152	0,220	0,153	0,241	0,153	0,257	0,154	0,264	0,154
0,145	0,195	0,158	0,213	0,158	0,228	0,159	0,250	0,159	0,267	0,160	0,275	0,160
0,150	0,202	0,164	0,221	0,164	0,237	0,165	0,260	0,165	0,277	0,166	0,285	0,166
0,155	0,210	0,170	0,229	0,170	0,246	0,171	0,269	0,172	0,288	0,173	0,296	0,173
0,160	0,217	0,176	0,238	0,176	0,255	0,177	0,279	0,178	0,298	0,179	0,307	0,179
0,165	0,225	0,182	0,246	0,183	0,264	0,183	0,289	0,184	0,309	0,185	0,318	0,185
0,170	0,232	0,188	0,254	0,189	0,272	0,189	0,299	0,191	0,320	0,192	0,329	0,192
0,175	0,240	0,194	0,263	0,195	0,282	0,196	0,309	0,197	0,331	0,198	0,340	0,199
0,180	0,248	0,201	0,271	0,201	0,291	0,202	0,319	0,203	0,342	0,205	0,352	0,205
0,185	0,256	0,207	0,280	0,208	0,300	0,209	0,330	0,210	0,353	0,211	0,363	0,212
0,190	0,264	0,213	0,289	0,214	0,309	0,215	0,340	0,217	0,364	0,218	0,375	0,219
0,195	0,271	0,220	0,297	0,221	0,319	0,222	0,350	0,223	0,375	0,225	0,386	0,225
0,200	0,280	0,226	0,306	0,227	0,328	0,228	0,361	0,230	0,387	0,232	0,398	0,232
0,205	0,288	0,233	0,315	0,234	0,338	0,235	0,372	0,237	0,398	0,239	0,410	0,239
0,210	0,296	0,239	0,324	0,241	0,348	0,242	0,382	0,244	0,410	0,246	0,422	0,246
0,215	0,304	0,246	0,333	0,247	0,357	0,248	0,393	0,251	0,422	0,253	0,435	0,254
0,220	0,312	0,253	0,342	0,254	0,367	0,255	0,404	0,258	0,434	0,260	0,447	0,261
0,225	0,321	0,260	0,352	0,261	0,377	0,262	0,415	0,265	0,446	0,267	0,460	0,268
0,230	0,329	0,266	0,361	0,268	0,387	0,269	0,427	0,272	0,458	0,275	0,472	0,276
0,235	0,338	0,273	0,370	0,275	0,398	0,276	0,438	0,279	0,470	0,282	0,485	0,283
0,240	0,346	0,280	0,380	0,282	0,408	0,284	0,450	0,287	0,483	0,290	0,499	0,291
0,245	0,355	0,287	0,390	0,289	0,419	0,291	0,461	0,294	0,496	0,297	0,512	0,299
0,250	0,364	0,295	0,399	0,296	0,429	0,298	0,473	0,302	0,509	0,305	0,525	0,306

TABELA 1_S400 (cont.)

FLEXÃO SIMPLES

Secções rectangulares simplesmente armadas

S400



$$\alpha = \frac{x}{d}; \mu = \frac{M_{Rd}}{bd^2 f_{cd}}; \bar{\omega} = \frac{A_s f_{yd}}{bd f_{cd}}$$

$$f_{yd} = f_{yk} / 1.15; f_{cd} = f_{ck} / 1.5$$

μ	C12-C50		C55		C60		C70		C80		C90	
	α	$\bar{\omega}$	α	$\bar{\omega}$	α	$\bar{\omega}$	α	$\bar{\omega}$	α	$\bar{\omega}$	α	$\bar{\omega}$
0,255	0,373	0,302	0,409	0,304	0,440	0,306	0,485	0,309	0,522	0,313	0,539	0,315
0,260	0,382	0,309	0,419	0,311	0,451	0,313	0,498	0,317	0,535	0,321	0,553	0,323
0,265	0,391	0,316	0,429	0,319	0,462	0,321	0,510	0,325	0,549	0,329	0,567	0,331
0,270	0,400	0,324	0,440	0,326	0,473	0,329	0,522	0,333	0,563	0,337	0,582	0,339
0,275	0,409	0,331	0,450	0,334	0,484	0,336	0,535	0,341	0,577	0,346	0,596	0,348
0,280	0,419	0,339	0,460	0,342	0,495	0,344	0,548	0,349	0,591	0,354	0,611	0,375
0,285	0,428	0,347	0,471	0,349	0,507	0,352	0,561	0,358	0,605	0,372	0,626	0,410
0,290	0,438	0,355	0,482	0,357	0,519	0,360	0,574	0,366	0,620	0,406	0,642	0,449
0,295	0,448	0,362	0,493	0,366	0,531	0,369	0,588	0,375	0,635	0,444	0,658	0,493
0,300	0,458	0,370	0,504	0,374	0,543	0,377	0,602	0,384	0,651	0,486	0,674	0,544
0,305	0,468	0,379	0,515	0,382	0,555	0,386	0,616	0,405	0,666	0,533	0,690	0,601
0,310	0,478	0,387	0,526	0,391	0,567	0,394	0,630	0,441	0,682	0,587	0,707	0,667
0,315	0,488	0,395	0,538	0,399	0,580	0,403	0,645	0,480	0,699	0,649	0,725	0,745
0,320	0,499	0,404	0,550	0,408	0,593	0,412	0,660	0,525	0,715	0,721	0,742	0,835
0,325	0,509	0,412	0,562	0,417	0,606	0,421	0,675	0,574	0,732	0,804	0,761	0,944
0,330	0,520	0,421	0,574	0,426	0,620	0,431	0,690	0,631	0,750	0,902	0,779	1,075
0,335	0,531	0,430	0,586	0,435	0,633	0,456	0,706	0,697	0,768	1,021	0,799	1,238
0,340	0,542	0,439	0,599	0,444	0,647	0,494	0,722	0,772	0,787	1,164	0,819	1,444
0,345	0,554	0,448	0,611	0,454	0,661	0,538	0,739	0,859	0,806	1,342	0,840	1,714
0,350	0,565	0,458	0,624	0,463	0,676	0,587	0,756	0,962	0,826	1,569	0,861	2,081
0,355	0,577	0,467	0,638	0,473	0,691	0,643	0,774	1,086	0,846	1,868	0,883	2,608
0,360	0,589	0,477	0,651	0,507	0,706	0,706	0,792	1,236	0,868	2,279	0,907	3,430
0,365	0,601	0,487	0,665	0,551	0,721	0,779	0,811	1,423	0,890	2,878	0,931	4,889
0,370	0,613	0,497	0,680	0,600	0,737	0,863	0,830	1,660	0,913	3,833		
0,375	0,626	0,507	0,694	0,656	0,754	0,962	0,850	1,974	0,937	5,604		
0,380	0,639	0,518	0,709	0,720	0,771	1,080	0,871	2,405				
0,385	0,653	0,528	0,725	0,794	0,788	1,223	0,892	3,036				
0,390	0,666	0,540	0,740	0,879	0,806	1,399	0,915	4,045				
0,395	0,680	0,583	0,757	0,979	0,825	1,621	0,939	5,913				
0,400	0,695	0,637	0,774	1,100	0,844	1,911						
0,405	0,710	0,698	0,791	1,245	0,865	2,303						
0,410	0,725	0,769	0,809	1,427	0,886	2,866						
0,415	0,741	0,852	0,828	1,655	0,908	3,743						
0,420	0,757	0,951	0,847	1,958	0,932	5,295						
0,425	0,774	1,068	0,868	2,376								
0,430	0,792	1,213	0,890	2,984								
0,435	0,811	1,395	0,913	3,964								
0,440	0,830	1,629	0,937	5,804								
0,445	0,850	1,945										
0,450	0,872	2,396										
0,455	0,896	3,092										
0,460	0,921	4,307										
0,465	0,949	7,040										
0,470												
0,475												
0,480												
0,485												
0,490												
0,495												
0,500												