Master's thesis

NTNU
Norwegian University of Science and Technology
Faculty of Engineering
Department of Structural Engineering

Carlos Mir Díaz

Analysis project of embodied carbon emissions in building structures with reinforced concrete components

Master's thesis in TKT4950
Supervisor: Jochen Köhler
Co-supervisor: Ramon Hingorani
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Contents

Symbols

Greek lower case letters

α_a Reducing coefficient applied to the usage overload

α_h Reduction factor for height

α_m Reduction factor for number of members

α_n Reducing coefficient applied to the usage overload on columns and walls

η Factor to define the effective strength

γ Partial safety factor

λ Factor to define the effective depth of the compression zone

λ Slenderness

μ_i Snow load shape coefficient

ϕ Inclination due to global imperfections

ψ Coefficient of simultaneity

υ_b Basic wind velocity

ξ Reduction factor for unfavourable permanent actions

Latin lower case letters

$1/r$ Curvature

c_0 Orography factor

d Effective depth

e_0 First-order eccentricity

e_2 Second-order eccentricity

e_i Eccentricity due to imperfections

f_{cd} Design value of concrete compressive strength

f_{ck} Characteristic compressive cylinder strength of concrete at 28 days

f_{ck} Characteristica yield strength of reinforcement

f_{yd} Design yield strength of reinforcement

i Minimum radius of gyration

k_r Terrain factor

l_0 Effective length

p Pressure

q_b Basic velocity pressure

s_k Characteristic value of snow load on the ground

w Uniformly distributed load

x Neutral axis depth

Latin upper case letters

A	Area
A_s	Reinforcing area
C_e	Exposure coefficient
C_pe	Wind pressure on external surfaces
C_t	Thermal coefficient
ECC	Embodied carbon coefficients
ELB	External longitudinal beam
ETB	External transverse beam
F_c	Compressive force
F_s	Tensile actions
G_k	Characteristic permanent action
GWP	Global warming potential
I	Moment of inertia
ILB	Internal longitudinal beam
ITB	Internal transverse beam
K	Stiffness
M_0	First-order moment
M_i	Moment due to imperfections
MBM	Maximum bending moment
MEd	Design bending moment at break
Mn	Bending moment at break
P	Prestressing actions
Q_k	Characteristic variable action
SMQ	Structural material quantities
$WLEC$	Whole-life embodied carbon

1 Introduction

Nowadays, buildings and the construction industry are top contributors to climate change, because it has a major determining role on the environment through consumption of land and raw materials and generation of waste.

Nearly 40% of global carbon dioxide emissions are linked to buildings and construction: the operation of buildings is responsible for 28% of global carbon dioxide emissions, whereas the construction industry, including the manufacture of building and components is responsible for the 11% of global carbon dioxide emissions.

Operational carbon dioxide emissions are due to heating, cooling, ventilation and lighting, whereas embodied carbon dioxide is associated with materials extraction, manufacturing, transportation, construction, maintenance and demolition, which are the different life cycle stages, presented in the following figure:

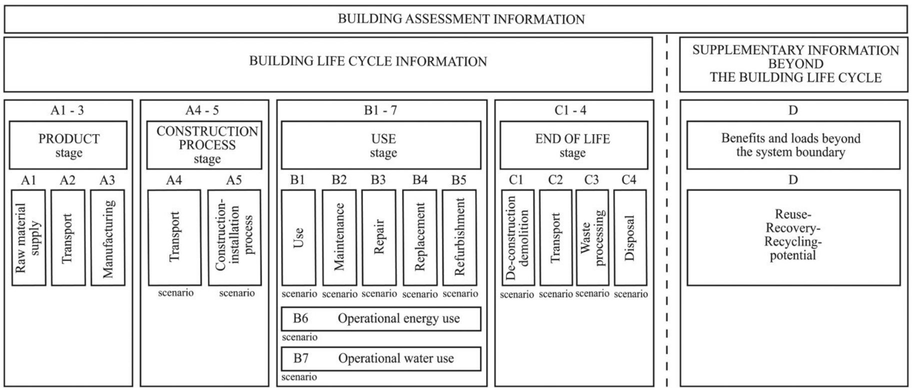


Figure 1: Life cycle stages according to CEN (2011)

Due to higher environmental awareness and innovation, the operational carbon dioxide emissions are being reduced as years go by, that is why embodied carbon dioxide will become a more significant percentage of greenhouse gas emissions caused by buildings.

1.1 Problem Statement

Although architects or engineers are dedicated to finding, among other things, which option is the most sustainable, in most cases it is the client who decides the geometry or materials that will be used for construction. The client interests are mainly governed by cost-efficiency, where environmental concerns are often disregarded in such thinking.

Although nowadays there are certificates that recognize buildings that meet certain environmental and energy efficiency criteria, a more efficient solution to this problem would be to regulate sustainability requirements by design codes and standards as has been done with decisions concerning structural safety.

Due to a lack of standardisation, manufacturers use different assumptions according to their benefit: taking into account carbon dioxide sequestration may be beneficial for timber, while taking into account recycled content may be beneficial for steel.

Moreover, systematic studies on embodied carbon assessments of structures are relatively scarce, and they are not detailed studies which allow for a case-specific parameter study: global structural geometry (e.g. beam span, number of storeys) or the partial safety factors. The only exception to that rule is the Hart et al study, presented in section 2.1.2, but it only takes into account the vertical load beating structure.

1.2 Procedure

- Define two prototypes with different shear wall configuration to analyse how sensitive results are to this
- Set up a Pyhton code where these prototypes are parametrised and designed according to Eurocodes
- Perform the design based on a specific parameter selection
- Determine the SMQ and the GWP
- Evaluate the results

1.3 Scope

This study is limited to frame reinforced concrete structures with shear walls in order to resist in-plane lateral forces, typically from wind and seismic loads.

It is also limited to *structural* material quantities. Cladding and non-structural materials are not considered for two reasons. Firstly, structure represents the largest weight in buildings and contributes to about a half of the total carbon dioxide emissions due to materials (Webster *et al.*, 2012). Secondly this helps to focus attention on well defined quantity while still having a significant impact (Wise *et al.*, 2012).

2 State of Art

2.1 Research, projects and sources consulted

2.1.1 Material quantities and embodied carbon dioxide in structures (De Wolf, Yang, Cox *et al.*)

This research analyses data from 200 existing buildings to identify the embodied environmental impact of building structures.

This paper analyses only two primary variables in order to obtain the global warming potential - structural material quantity (SMQ, expressed in kg_{material}/m^2) and embodied carbon dioxide coefficients (ECC, expressed in kg_{CO2e}/m^2)

$$GWP_{\text{building}} = \sum_{materiali=1}^{N} (SMQ_{\text{material i}} \cdot ECC_{\text{material i}}) \tag{1}$$

Material quantities were mainly extracted from building information buildings such as Revit (Autodesk, 2014). Finishes or other non-structural materials will not be taken into account. Whereas, cradle-to-gate embodied carbon coefficients were collected from existing data from the literature.

Those buildings were divided depending on their use, and it was observed that cultural buildings has the highest material usage and environmental usage, whereas office buildings have the lowest material usage usage and environmental usage.

Regarding the material used on each structure, timber is the one with a lowest impact. Although steel structures show lower material quantities than concrete they have highest embodied carbon dioxide emissions, because steel has higher embodied carbon coefficients than concrete.

2.1.2 Whole-life embodied carbon in multistory buildings: Steel, concrete and timber structures

The aim of this report is to study the greenhouse gas emissions from all life cycles stages from A to D, with exception of B6 and B7 (operational energy and water) from buildings of different number of storeys and three different materials: steel, concrete and timber.

The whole-life embodied carbon of $1m^2$ of each superstructure is:

$$WLEC = \frac{\sum_i M_i \cdot ECC_i + \sum_j M_s \cdot ECC_j}{A \cdot h} \tag{2}$$

Where:

M_i is the mass of material i (kg)

ECC_i is the embodied carbon coefficient of material i summed across all life cycle stages except construction and demolition $(kgCO_2e/kg)$

M_s is the total mass of the superstructure (kg)

ECC_j is the embodied carbon coefficient for whole structure process (construction or demolition) $(kgCO_2e/kg)$

A is the footprint of the building (m^2)

h is the number of storeys

Timber structures has less embodied carbon dioxide emissions associated compared with identical structures with reinforced concrete or steel. It is basically due to the lower energy demand for manufacturing of wood-based components. Moreover, in the carbon dioxide emission balance of timber structures, other beneficial factors must be taken into account, like the amounts of carbon dioxide sequestered from the atmosphere during tree growth and the energy recovery by the incineration of the wood.

Although there is a general consensus about the environmental benefits of timber structures, it has not yet been established whether concrete structures are more environmentally beneficial than steel structures or vice versa. It is because, for steel structures the material manufacturing processes need larger amount of energy, but at the same time, most countries have incorporated significant amounts of recycled material in steel production.

In conclusion, in any case, timber structures will be the ones that generate the least carbon dioxide emissions, and, after them, depending on the country, they will be concrete or steel structures.

3 Methodology

3.1 Layout of the structures and load transfer

To carry out this study the following two layouts were considered:

LAYOUT 1

The first layout is characterized by having perimeter shear walls and both interior and perimeter columns. The four shear walls of this layout will always have the same length as each of the building's long beams and their thickness will be determined so that second-order global effects on the building can be neglected.

BEAMS. The beams from this layout will be divided into four categories depending on the type and if they are subjected to lateral loads:

- ELB under the action of lateral loads
- ETB under the action of lateral loads
- ILB
- ITB

All the beams will have the same type of connection: the end of the beam connected to the shear wall will be hinged, whereas the end of the beam connected to the column will be fixed.

COLUMNS. On the interior columns, the shear and bending forces acting on them can be neglected, so they will designed according only to axial compression load, so they will have the minimum reinforcing steel area.

On the other hand, the perimeter columns will be under axial compression load and minor axis bending.

The connection of the columns will be rigid.

SLABS. They will be simply supported and spanning in two directions.

SHEAR WALLS. This layout has four perimeter shear walls.

FOUNDATIONS. The load on the columns and shear walls will be transferred to isolated footings.

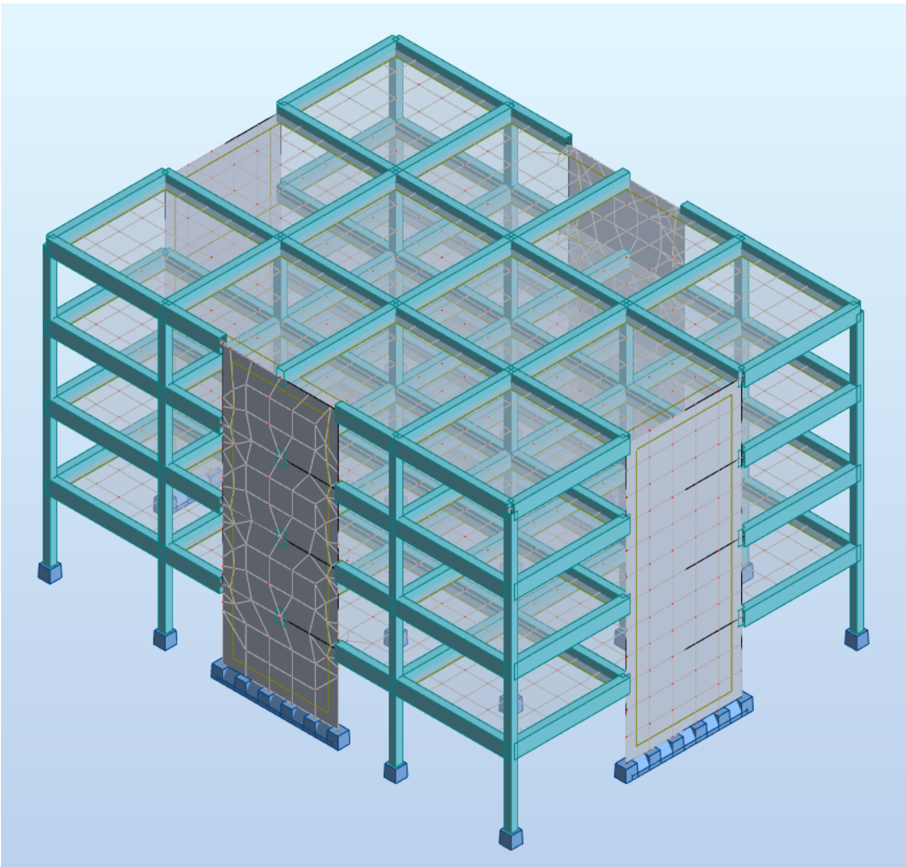


Figure 2: Isometric view of the first layout

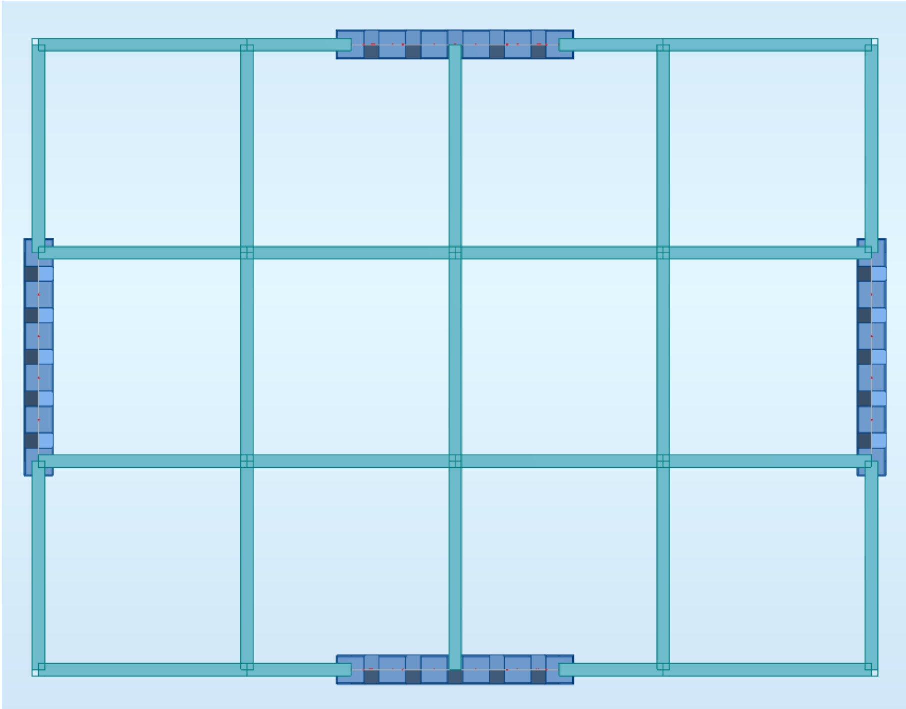


Figure 3: Floor plan of the fist layout

LAYOUT 2

Unlike the first layout, in this case, the shear walls will be interior and the columns will be perimeter. The length of these shear walls will be equal to the sum of the lengths of the interior beams of the building, forming a central core shear wall.

BEAMS. In this layout, transverse beams will have half the length of longitudinal beams. They will be divided into 5 categories depending on the type and if they are subjected to lateral loads:

- ELB
- ILB
- ILB under the action of lateral loads
- ITB
- ITB under the action of lateral loads

The external longitudinal beams will be connected at both ends to columns, and those connections will be fixed. Both the internal longitudinal and internal transverse beams will have the same type of connection: the end of the beam connected to the shear wall will be hinged, whereas the end of the beam connected to the column will be fixed.

COLUMNS. This layout only has perimeter columns. They will be designed according to axial compression load and bending moment.

The connection of the columns will be rigid.

SLABS. They will be simply supported and spanning in two directions.

SHEAR WALLS. This layout has four interior shear walls.

FOUNDATIONS. The load on the columns and shear walls will be transferred to isolated footings.

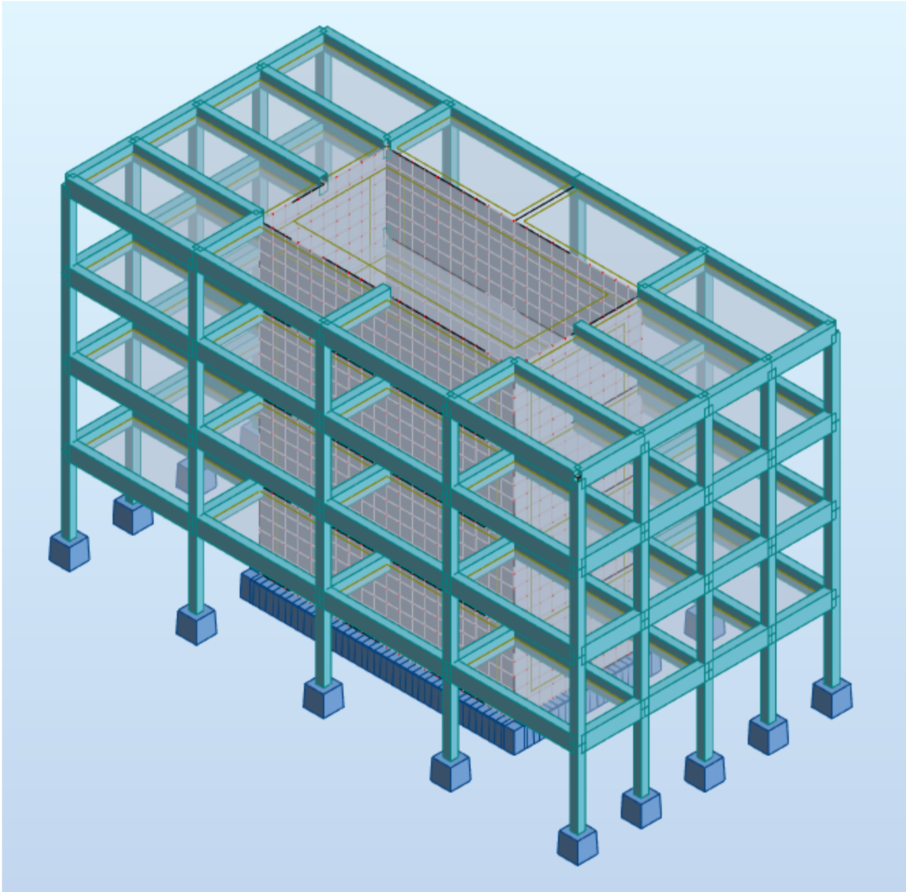


Figure 4: Isometric view of the second layout

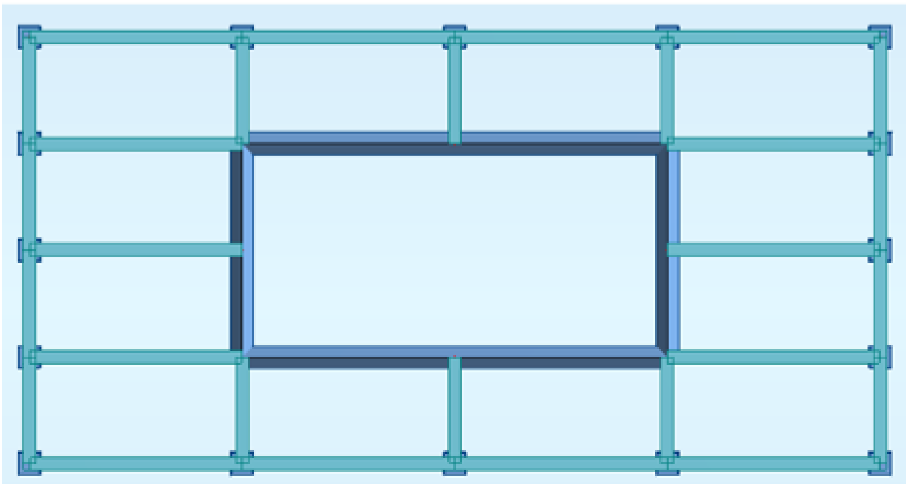


Figure 5: Floor plan of the second layout

To transfer slab loads to beams for two-way slabs, the following equations are followed:

Long span: $q = n \cdot (l_x/2) \cdot (1 - 1/3 \cdot k^2)$

Short span: $q = n \cdot (l_x/3)$

Where:

q is the load transferred from slab to the beam

n is the load at ultimate limit state

$k = l_y / l_x$

l_y is the length of long span of the slap

l_y is the length of short span of the slab

3.2 Actions and their combinations

Self-weight Reinforced concrete: $25kN/m^3$

Live load For office buildings, the characteristic usage overload value will be:

$$q_k = 3kN/m^2 \tag{3}$$

Although the live load, q_k has been modeled as a parameter, for simplicity and taking into account that this study is based on office buildings it will remain constant.

A reducing coefficient α_A can be applied to the values q_k of the usage overload for office buildings:

$$\alpha_A = \frac{5}{7}\psi_{0.\mathrm{q}} + \frac{A_0}{A} \leq 1.0 \tag{4}$$

Where:

$\psi_{0.\mathrm{q}}$ is the coefficient of simultaneity specified at the table A1.1 from the EN1990:2002(E)

$A_0 = 10.0m^2$

A is the loaded area

In addition, for office buildings, the total usage overloads on columns and walls from different floors can be multiplied by the reduction factor α_n:

$$\alpha_n = \frac{2 + (n-2) \cdot \psi_{0.\mathrm{q}}}{n} \tag{5}$$

Where:

n is the number of stories (< 2) above loaded structural elements of the same category

$\psi_{0.\mathrm{q}}$ is the coefficient of simultaneity specified at the table A1.1 from the EN1990:2002(E)

Snow overload

The snow loads on roofs for persistent and transient design situations shall be determined as follows:

$$s = \mu_i C_e C_t s_k \tag{6}$$

Where:

$\mu_i = 0.8$ (for flat floors) is the snow load shape coefficient

$C_e = 1$ (for normal conditions) is the exposure coefficient.

$C_t = 1$ is the thermal coefficient used for the reduction of snow loads on roofs with high thermal transmittance.

s_k is the characteristic value of snow load on the ground. In Trondheim it can be taken as $3,5kN/m^2$. The snow load over the roof will be taken as:

$$s_k = 0.8 \cdot 1 \cdot 1 \cdot 3.5 = 2.8kN/m^2 \tag{7}$$

Wind overload

Basic wind velocity in Trondheim, Norway: $\upsilon_b = 26m/s$

Terrain cathegory IV: $z_0 = 1$; $z_{\min} = 10$

Terrain factor: $k_r = 0.19(z_0/0.05)^{0.07} = 0.19(1/0.05)^{0.07} = 0.234m/s$

Orography factor: $c_0 = 1$

Exposure factor $c_e(z)$, taking into account turbulence:

For $z \leq 10m$:

$$c_e(z_{\min} = 10) = k_r^2 c_0 \ln(\frac{z}{z_0})(7 + c_0 \ln(\frac{z}{z_0})) = 0.23^2 \cdot 1 \cdot \ln(\frac{10}{1})(7 + 1 \cdot \ln(\frac{10}{1})) = 1.13 \tag{8}$$

For $z > 10m$:

$$c_e(z) = k_r^2 c_0 \ln(\frac{z}{z_0})(7 + c_0 \ln(\frac{z}{z_0})) = 0.053 \cdot \ln(z)(7 + \ln(z)) \tag{9}$$

Basic velocity pressure:

$$q_b = \frac{1}{2}\rho\upsilon_b^2 = \frac{1}{2} \cdot 1.25 \cdot 26^2 \cdot 10^-3 = 0.423kN/m^2 \tag{10}$$

Where the air density is assumed to be $\rho = 1.25kg/m^3$.

Peak velocity pressure:

For $z \leq 10m$:

$$q_p(z_e) = c_e(z_{\min})q_b = c_e(10) \cdot 0.563kN/m^2 \tag{11}$$

For $z > 10m$:

$$q_p(z_e) = c_e(z)q_b = c_e(z) \cdot 0.563kN/m^2 \tag{12}$$

Wind pressure on external surfaces: $C_{\text{pe}} = +0.8; C_{\text{pe}} = -0.4$

Structural factor for framed buildings with structural walls less than $100m$ high: $c_s c_d = 1.0$

Wind pressure on external surfaces:

$$w_e = q_p(z_e)c_{\text{pe}}c_s c_d = q_p(z_e)(0.8 - (-0.4)) \cdot 1 = 1.2 \cdot q_p(z_e)kN/m^2 \tag{13}$$

For $z \leq 10m$:

$$w_e(10) = 1.2 \cdot c_e(z_e) \cdot 0.56 = 0.035 \cdot \ln(10)[7 + \ln(10)] = 0.77kN/m^2 \tag{14}$$

For $z > 10m$:

$$w_e(z_e) = 1.2 \cdot c_e(z_e) \cdot 0.56 = 0.035 \cdot \ln(z_e)[7 + \ln(z_e)]kN/m^2 \tag{15}$$

3.2.1 Combination of actions

For the dimensioning of a structure, all foreseeable load conditions during the execution and use of the structure must be considered, taking into account the probability of their occurrence and the simultaneity between them. For this, combinations are established, which reflects situations that, when exceeded, may cause the construction to not comply with some of the structural requirements for which it has been designed.

$$\sum_{j \geq 1} \gamma_{\text{G,j}} G_{\text{k,j}} + \gamma_p P + \gamma_{\text{Q,1}} \psi_{0.\text{q}} Q_{\text{k,1}} + \sum_{i > 1} \gamma_{\text{Q,i}} \psi_{0.\text{q}} Q_{\text{k,i}} \tag{16}$$

$$\sum_{j\geq 1} \xi_j \gamma_{\mathrm{G,j}} G_{\mathrm{k,j}} + \gamma_p P + \gamma_{\mathrm{Q,1}} Q_{\mathrm{k,1}} + \sum_{i>1} \gamma_{\mathrm{Q,i}} \psi_{\mathrm{0.q}} Q_{\mathrm{k,i}} \tag{17}$$

Where:

G refers to permanent actions

Q refers to variable actions

P refers to the prestressing actions

γ are the applied partial safety factors

ψ are the coefficients of simultaneity

$\xi = 0.85$ is a reduction factor for unfavourable permanent actions

The representative values of the variable actions are:

- Combination value ($\psi_0 Q_\mathrm{k}$) "variable action intensity" when it acts simultaneously with another variable action with its "maximum intensity". Used in the verification of ultimate limit states and irreversible serviceability limit states.
- Frequent value ($\psi_1 Q_\mathrm{k}$) value of the variable action that is only exceeded during 1% of the reference time. Used in the verification of ultimate limit states that include accidental actions and for the verification of reversible serviceability limit states.
- Quasi-permanent value ($\psi_2 Q_\mathrm{k}$) value of the variable action that is exceeded for 50 of the reference time. Used in the verification of ultimate limit states that include accidental actions and for the verification of reversible serviceability limit states.

The applied partial safety factors (*gamma*) will be:

$\gamma_{\mathrm{G,j}} = 1.35$ for permanent actions

$\gamma_{\mathrm{G,j}} = 1.5$ for variable actions

And the coefficients of simultaneity (ψ) are specified the table A1.1 from the EN1990:2002(E).

Twelve different combinations of the actions will be considered:

From equation 6.10a:

LC1: $\gamma_g \cdot g_k + \gamma_q \cdot q_k \cdot \psi_{\mathrm{0.q}}$

LC2: $\gamma_g \cdot g_k + \gamma_q \cdot q_k \cdot \psi_{\mathrm{0.q}} + \gamma_q \cdot s_k \cdot \psi_{\mathrm{0.s}}$

LC3: $\gamma_g \cdot g_k + \gamma_q \cdot q_k \cdot \psi_{\mathrm{0.q}} + \gamma_q \cdot w_k \cdot \psi_{\mathrm{0.w}}$

LC4: $\gamma_g \cdot g_k + \gamma_q \cdot s_k \cdot \psi_{\mathrm{0.s}} + \gamma_q \cdot w_k \cdot \psi_{\mathrm{0.w}}$

LC5: $\gamma_g \cdot g_k + \gamma_q \cdot q_k \cdot \psi_{\mathrm{0.q}} + \gamma_q \cdot s_k \cdot \psi_{\mathrm{0.s}} + \gamma_q \cdot w_k \cdot \psi_{\mathrm{0.w}}$

From equation 6.10b:

LC6: $\xi \cdot \gamma_g \cdot g_k + \gamma_q \cdot q_k$

LC7: $\xi \cdot \gamma_g \cdot g_k + \gamma_q \cdot q_k + \gamma_q \cdot s_k \cdot \psi_{\mathrm{0.s}}$

LC8: $\xi \cdot \gamma_g \cdot g_k + \gamma_q \cdot q_k \cdot \psi_{\mathrm{0.q}} + \gamma_q \cdot s_k$

LC9: $\xi \cdot \gamma_g \cdot g_k + \gamma_q \cdot q_k + \gamma_q \cdot w_k \cdot \psi_{\mathrm{0.w}}$

LC10: $\xi \cdot \gamma_g \cdot g_k + \gamma_q \cdot q_k \cdot \psi_{\mathrm{0.q}} + \gamma_q \cdot w_k$

LC11: $\xi \cdot \gamma_g \cdot g_k + \gamma_q \cdot s_k + \gamma_q \cdot w_k \cdot \psi_{0.\mathrm{w}}$

LC12: $\xi \cdot \gamma_g \cdot g_k + \gamma_q \cdot s_k \cdot \psi_{0.\mathrm{s}} + \gamma_q \cdot w_k$

LC13: $\xi \cdot \gamma_g \cdot g_k + \gamma_q \cdot q_k + \gamma_q \cdot s_k \cdot \psi_{0.\mathrm{s}} + \gamma_q \cdot w_k \cdot \psi_{0.\mathrm{w}}$

LC14: $\xi \cdot \gamma_g \cdot g_k + \gamma_q \cdot q_k \cdot \psi_{0.\mathrm{q}} + \gamma_q \cdot s_k + \gamma_q \cdot w_k \cdot \psi_{0.\mathrm{w}}$

LC15: $\xi \cdot \gamma_g \cdot g_k + \gamma_q \cdot q_k \cdot \psi_{0.\mathrm{q}} + \gamma_q \cdot s_k \cdot \psi_{0.\mathrm{s}} + \gamma_q \cdot w_k$

3.2.2 Effects of global imperfections

The effect of the global imperfections generates horizontal actions on the structure:

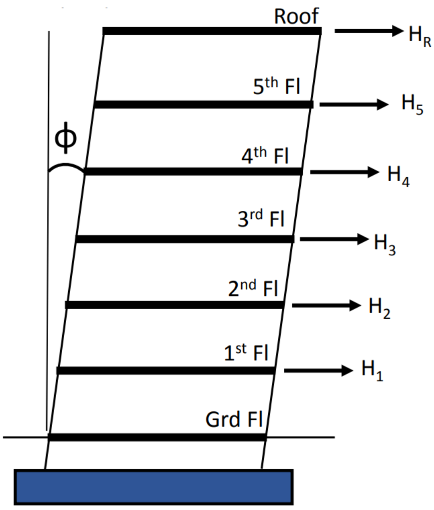


Figure 6: Horizontal actions generated on the structure by the effects of global imperfections

Where $(N_b - N_a)$ is the axial load from each level, and ϕ is the inclination, given by:

$$\phi_i = \phi_0 \cdot \alpha_h \cdot \alpha_m \tag{18}$$

ϕ_0 is the basic value. The recommended value is $\frac{1}{200}$

α_h is the reduction factor for height: $\alpha_h = \frac{2}{\sqrt{l}}; 2/3 \leq \alpha_h \leq 1$ (l is the column length)

α_m is the reduction factor for number of members: $\alpha_m = \sqrt{0.5 \cdot (1 + \frac{1}{m}}$ (m is the number of vertical members contributing to the total effect)

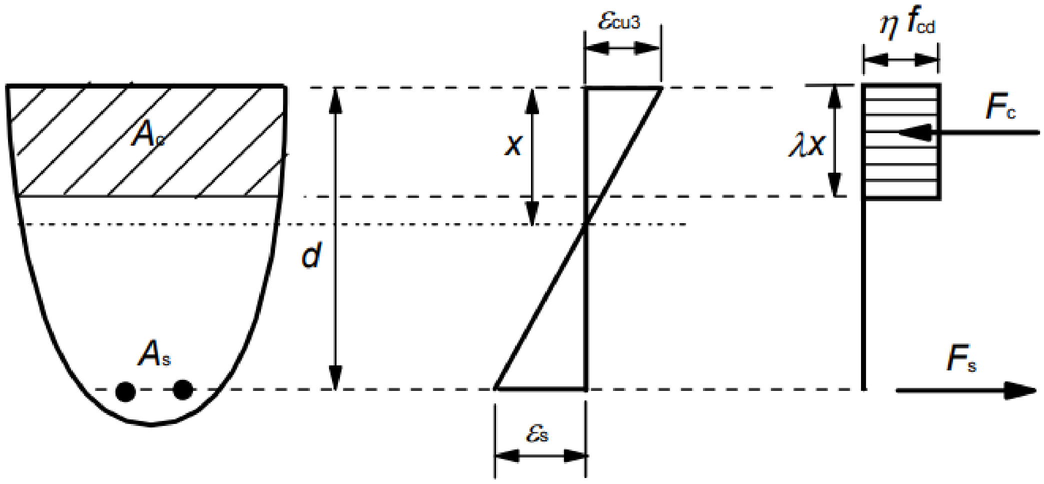
3.3 Limit States

3.3.1 Ultimate Limit States

BEAMS

In both layouts, two types of beams are found based on the stresses to which they are subjected. The first type consists of those subjected only to vertical forces with a single reinforcement, while the second type of beams are exposed not only to vertical forces but also to the horizontal forces generated by the wind, and thus requires double reinforcement.

To obtain the design ultimate limit state function of the beams under simple bending, the following method is used:



Where the factor λ is multiplied by the neutral axis depth and defines the effective height of the compression zone:

$$\lambda = 0.8 \qquad \text{for } f_{\text{ck}} \leq 50MPa \tag{19}$$

$$\lambda = 0.8 - \frac{f_{\text{ck}-50}}{200} \qquad \text{for } 50MPa < f_{\text{ck}} \leq 90MPa \tag{20}$$

And the factor η defines the effective strength:

$$\eta = 1 \qquad \text{for } f_{\text{ck}} \leq 50MPa \tag{21}$$

$$\eta = 1 - \frac{f_{\text{ck}-50}}{200} \qquad \text{for } 50MPa < f_{\text{ck}} \leq 90MPa \tag{22}$$

The neutral axis depth is obtained by equating the tensile and compressive stresses:

$$F_c = F_s \tag{23}$$

$$\eta \cdot f_{\text{cd}} \cdot \lambda \cdot x \cdot b = f_{\text{yd}} \cdot A_s \tag{24}$$

$$x = \frac{f_{\text{yd}} \cdot A_s}{\eta \cdot f_{\text{cd}} \cdot \lambda \cdot b} \tag{25}$$

And the bending moment at break of the beam taking moments about the centroid of the compressive force:

$$M_n = F_s \cdot (d - \frac{\lambda \cdot x}{2}) = f_{\text{yd}} \cdot A_s \cdot (d - \frac{\lambda \cdot x}{2}) \tag{26}$$

For beams with pinned supports on both sides, subjected only be vertical loads, the maximum bending moment at mid-span will be:

$$M_{\text{Ed,max}} = w \cdot \frac{L^2}{8} \tag{27}$$

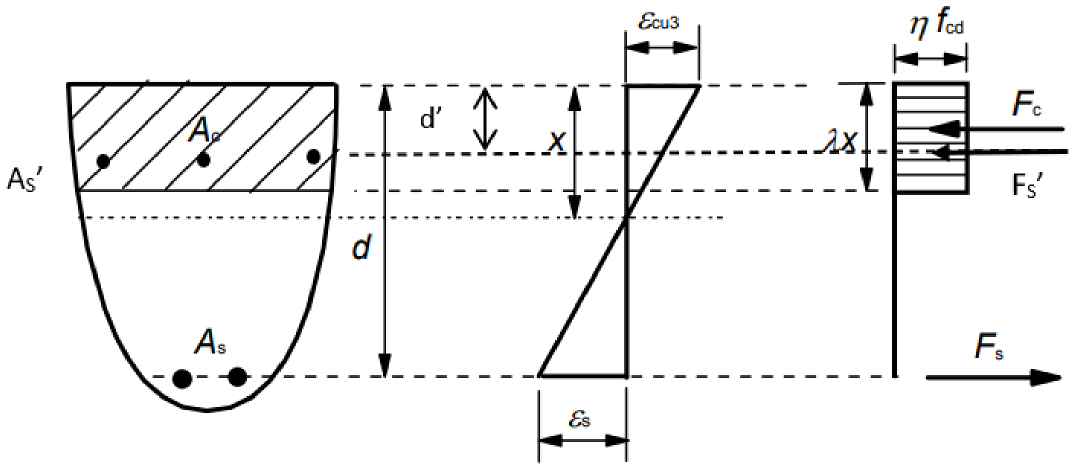
And for beams with fixed supports, the maximum bending moment at the edge will be:

$$M_{\text{Ed,max}} = w \cdot \frac{L^2}{12} \tag{28}$$

The design ultimate limit state function will be:

$$f_{\text{yd}} \cdot A_s \cdot (d - \frac{\lambda \cdot x}{2}) \geq M_{\text{Ed,max}} \tag{29}$$

The design ultimate limit state function of the beams under compound bending will be:



The neutral axis depth is obtained by equating the tensile and compressive stresses:

$$F_c + F_s' = F_s \tag{30}$$

$$\eta \cdot f_{\text{cd}} \cdot \lambda \cdot x \cdot b = f_{\text{yd}} \cdot A_s - f_{\text{yd}} \cdot A_s' \tag{31}$$

$$x = \frac{f_{\text{yd}} \cdot A_s - f_{\text{yd}} \cdot A_s'}{\eta \cdot f_{\text{cd}} \cdot \lambda \cdot b} \tag{32}$$

The bending moment at break of the beam taking moments about the neutral axis depth:

$$M_n = f_{\text{yd}} \cdot A_s \cdot (d - x) + f_{\text{yd}} \cdot A_s' \cdot (x - d') + \eta f_{\text{cd}} \cdot \lambda x \cdot b \cdot (x - \frac{\lambda x}{2}) \tag{33}$$

For compound bending, the maximum bending moment at mid-span of the beams with pinned supports will be:

$$M_{\text{Ed,max}} = w \cdot \frac{L^2}{8} + P \cdot (x - d') \tag{34}$$

And for beams with fixed supports:

$$M_{\text{Ed,max}} = w \cdot \frac{L^2}{12} + P \cdot (x - d') \tag{35}$$

And the ultimate limit state function:

$$f_{\text{yd}} \cdot A_s \cdot (d - x) + f_{\text{yd}} \cdot A_s' \cdot (x - d') + \eta f_{\text{cd}} \cdot \lambda x \cdot b \cdot (x - \frac{\lambda x}{2}) \geq M_{\text{Ed,max}} \tag{36}$$

COLUMNS

For compression members in regular frames with braced members, the effective length l_0 is determined in the following way:

$$l_0 = 0.5 \cdot l \cdot \sqrt{(1 + \frac{k_1}{0.45 + k_1}) + (1 + \frac{k_2}{0.45 + k_2})} \tag{37}$$

Where k_1 and k_2 are the relative flexibilities of rotational restraits at ends 1 and 2 respectively.

The slenderness around the y-axis:

$$\lambda_{\mathrm{y}} = \frac{l_{0,\mathrm{y}}}{i_y} \tag{38}$$

The slenderness around the z-axis:

$$\lambda_{\mathrm{z}} = \frac{l_{0,\mathrm{z}}}{i_y} \tag{39}$$

Where $i = \sqrt{I/A}$ is the minimum radius of gyration.

Second-order effects may be ignored is the slenderness λ is below a certain value λ_{lim}.

$$\lambda_{\mathrm{lim}} = 20 \cdot A \cdot B \cdot C/\sqrt{n} \tag{40}$$

Where:

$$A = 0.7 \tag{41}$$
$$B = 1.1 \tag{42}$$
$$C = 0.7 \tag{43}$$
$$n = N_{\mathrm{Ed}}/Ac \cdot f_{\mathrm{cd}} \tag{44}$$

If $\lambda_{\mathrm{y}}, \lambda_{\mathrm{z}} \leq \lambda_{\mathrm{lim}}$ second-order effects should not be taken into account.

The total design moment will be calculated as follows:

$$M_{\mathrm{tot}} = M_{0\mathrm{Ed}} = N_{\mathrm{Ed}}(e_0 + e_i) \quad \text{(not taking into account second-order effects)} \tag{45}$$
$$M_{\mathrm{tot}} = M_{0\mathrm{Ed}} + M_2 = N_{\mathrm{Ed}}(e_0 + e_i + e_2) \quad \text{(taking into account second-order effects)} \tag{46}$$

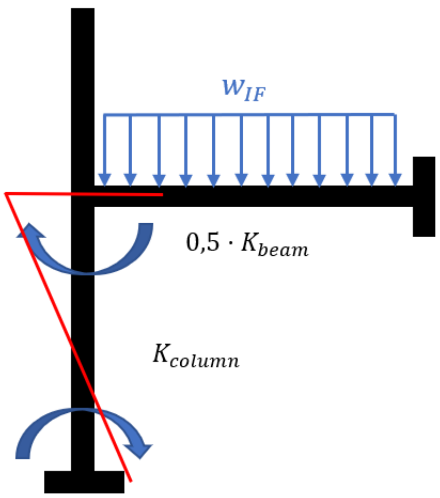
Where: e_0 is the first-order eccentricity

e_i is the eccentricity due to imperfections

e_2 is the second-order eccentricity

First-order eccentricity (e_0)

First floor columns



The first-order end moments will be different in the top and bottom of the column, $M_{01} \neq M_{02}$.

$$M_{02} = M_{\text{top}} = \frac{K_{\text{column}}}{2 \cdot K_{\text{column}} + 0.5 \cdot K_{\text{beam}}} \cdot MBM_{\text{IF}} \tag{47}$$

$$M_{01} = M_{\text{bottom}} = 0.25 \cdot M_{\text{top}} \tag{48}$$

Where:

The stiffness of each beam and column will be respectively:

$$K_{\text{beam}} = \frac{E \cdot I}{L} = \frac{E \cdot \frac{1}{12} \cdot b_{\text{beam}} \cdot h^3_{\text{beam}}}{beamspan} \tag{49}$$

$$K_{\text{column}} = \frac{E \cdot I}{L} = \frac{E \cdot \frac{1}{12} \cdot b^3_{\text{column}} \cdot h_{\text{column}}}{storeyheight} \tag{50}$$

The maximum bending moment generated on the columns by uniformly distributed load on the beams is:

$$MBM = w \cdot \frac{(beamspan)^2}{12} \tag{51}$$

Where w may be different from the top floor (w_{TF}), and the interior ones (w_{IF})

When first-order end moments at the top and bottom of the column are different, an equivalent first-order end moment may be used:

$$M_0 = 0.6 \cdot M_{02} + 0.4 \cdot M_{02} \geq 0.4 \cdot M_{02} \tag{52}$$

For cross sections with symmetrical reinforcement load by a compression force, it is necessary to assume the minimum eccentricity, $e_0 = h/30$ but not less than $20mm$, where h is the depth of the section.

The first-order moment be:

$$M_0 = max\{M_0, N_{\text{Ed}}\frac{h}{30}, N_{\text{Ed}} \cdot 20\}(kN \cdot m) \tag{53}$$

Eccentricity due to imperfections (e_i) According to chapter 5.2.9 (EN1992-1-1:2004(E)), as a simplified alternative for walls and isolated columns in braced systems, the moment due to imperfections may be taken as:

$$M_i = N_{\text{Ed}} \cdot \frac{1}{200} \cdot \frac{l_0}{2}(kN \cdot m) \tag{54}$$

Second-order eccentricity (e_2) The second-order eccentricity according to the nominal curvature method will be:

$$e_2 = (\frac{1}{r}) \cdot \frac{l_0^2}{c} \tag{55}$$

Where:

$1/r$ is the curvature

l_0 is the effective length

c is a factor depending on the curvature distribution (for constant cross section, $c = \pi^2$)

For members with constant symmetrical cross sections, the curvature is:

$$(\frac{1}{r}) = K_r \cdot K_\varphi \cdot (\frac{1}{r_0}) \tag{56}$$

Where:

K_r is a correction factor depending on axial load

K_φ is a factor that takes into account the creep

$1/r_0 = \epsilon_{\text{yd}}/0.45d$

$\epsilon_{\text{yd}} = f_{\text{yd}}/E_s$

d is the effective depth

The correction factor depending on axial load should be taken as:

$$K_r = \frac{n_u - n}{n_u - n_{\text{bal}}} \leq 1 \tag{57}$$

Where:

$n_u = 1 + \frac{\rho \cdot f_{\text{yd}}}{f_{\text{cd}}}$, where ρ is the reinforcing ratio estimated value

$n = N_{\text{Ed}}/a_c \cdot f_{\text{cd}}$ is the relative normla force

n_{bal} is the value of n at maximum moment resistance (0.4 may be used)

The effect of creep should be taken into account by the following factor:

$$K_\varphi = 1 + \beta\varphi_{\text{ef}} \geq 1 \tag{58}$$

Where:

$\varphi_{\text{ef}} = 0.8 \cdot \epsilon_{\text{cu}}/(1 + 0.2 \cdot \epsilon_{\text{cu}})$ is an approximation of the effective creep ratio for lightweight concrete with a compressive strength not greater than $90MPa$, and ϵ_{cu} is the ultimate deformation of concrete in compression

$\beta = 0.35 + f_{\text{ck}}/200 - \lambda/150$

After the total design moment is obtained, the mechanical reinforcement ratio (w_{tot}) can be obtained using the interaction diagram for a symmetric reinforced rectangular cross section. The two expressions that need to be used in order to read the charts are:

$$\mu_{\text{Ed}} = \frac{M_{\text{tot}}}{b \cdot h^2 \cdot f_{\text{cd}}} \tag{59}$$

$$\upsilon_{\text{Ed}} = \frac{M_{\text{tot}}}{b \cdot h \cdot f_{\text{cd}}} \tag{60}$$

The total reinforcement area of the columns will be:

$$A_{\text{s,tot}} = A_{\text{s1}} + A_{\text{s2}} = w_{\text{tot}} \frac{b \cdot h}{\frac{f_{\text{yd}}}{f_{\text{yd}}}} \tag{61}$$

SHEAR WALLS

The thickness of the shear walls will be chosen so that global second order effects on the building can be neglected.

Global second-order effects in buildings may be ignored if:

$$F_{\upsilon,Ed} \leq k_1 \frac{n_s}{n_s + 1.6} \cdot \frac{\sum E_{\text{cm}} I_c}{L^2} \tag{62}$$

Where:

$F_{\upsilon,\text{Ed}}$ is the total vertical load

$k_1 = 0.61$ (recommended value)

n_s is the number of storeys

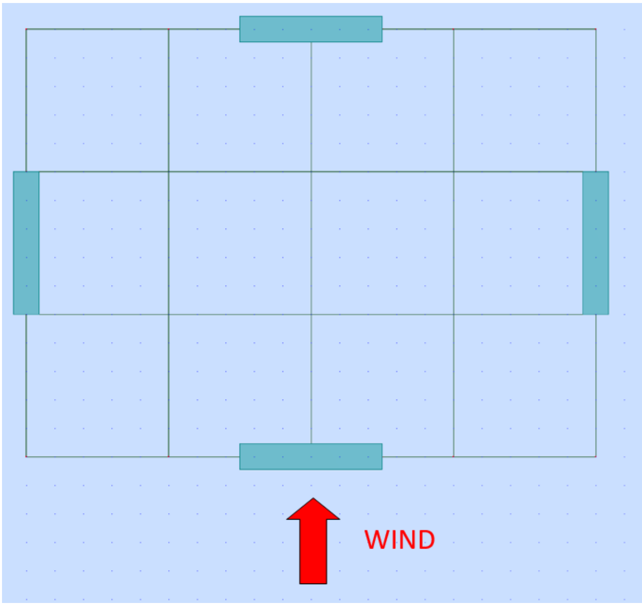
E_{cm} is the design value of the modulus of elasticity of concrete

I_c is the second moment of area of bracing members

L is the total height of building above level of moment restraint

WIND N-S//S-N

Layout 1

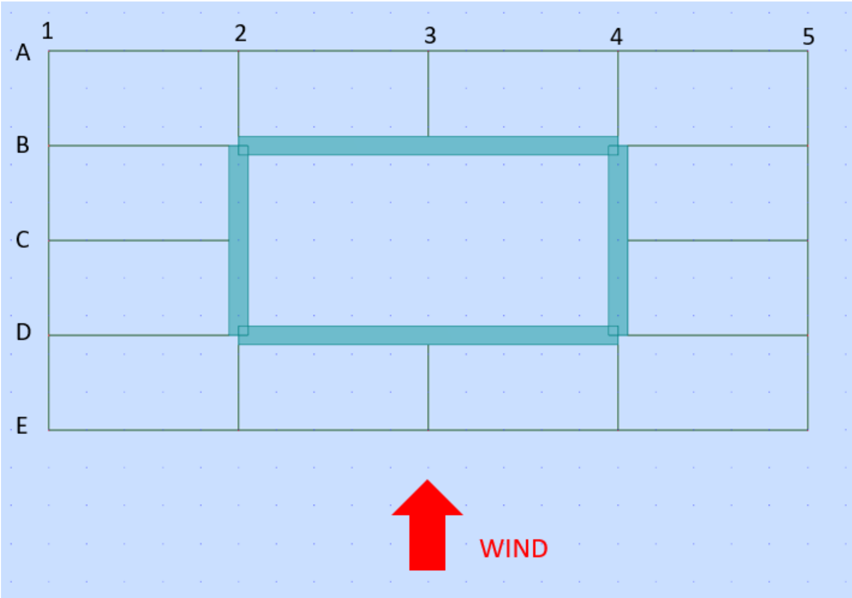


The moment of inertia around a centroid axis parallel to the X-global axis are:

$$I = b \cdot (3L)^3/12 \qquad \text{for shear walls 1 and 5} \qquad (63)$$

$$I = b^3 \cdot (2L)/12 \qquad \text{for shear walls A and D} \qquad (64)$$

Layout 2



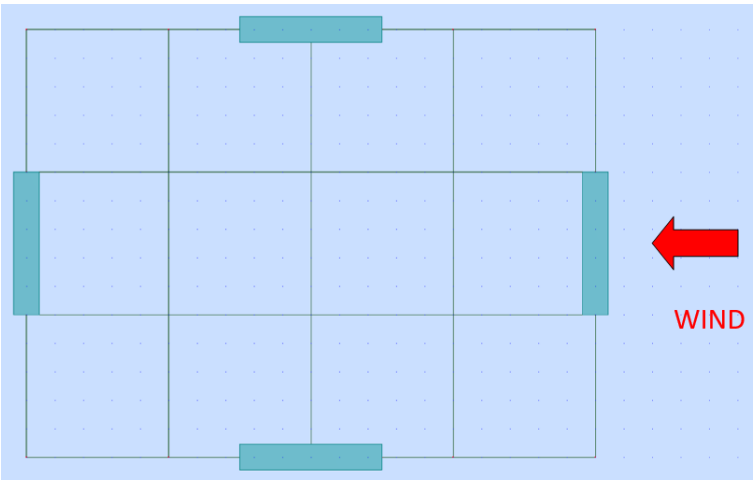
The moment of inertia around a centroid axis parallel to the X-global axis are:

$$I = b \cdot (2l_x)^3/12 \qquad \text{for shear walls 1 and 5} \qquad (65)$$

$$I = b^3 \cdot (2ly)/12 \qquad \text{for shear walls A and D} \qquad (66)$$

WIND E-W//W-E

Layout 1

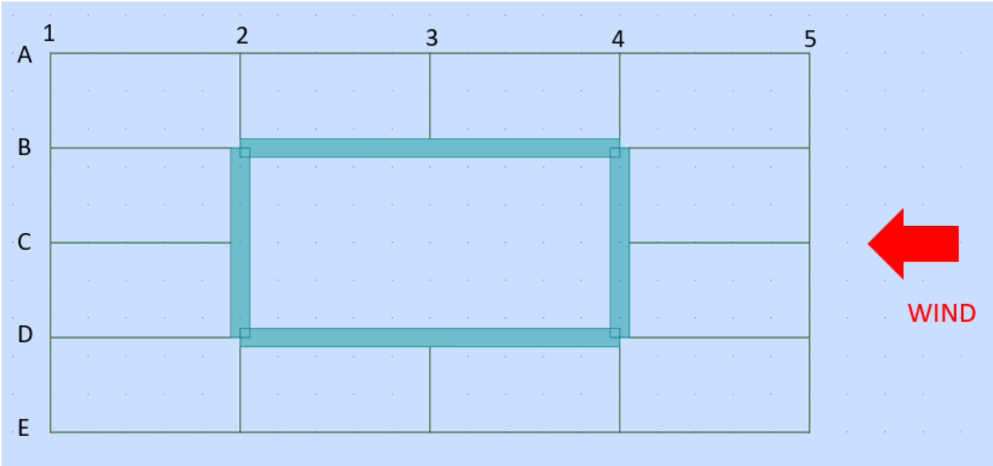


The moment of inertia around a centroid axis parallel to the Y-global axis are:

$$I = b^3 \cdot (3L)/12 \qquad \text{for shear walls 1 and 5} \qquad (67)$$

$$I = b \cdot (2L)^3/12 \qquad \text{for shear walls A and D} \qquad (68)$$

Layout 2



The moment of inertia around a centroid axis parallel to the Y-global axis are:

$$I = b^3 \cdot (2l_x)/12 \qquad \text{for shear walls 1 and 5} \tag{69}$$

$$I = b \cdot (2l_y)^3/12 \qquad \text{for shear walls A and D} \tag{70}$$

In the plane of the wall, the magnitude of the applied action is determined by splitting the wall into some series of $1meter$ strips. These can each be considered subject to axial compression and minor axis bending only, with the design of reinforcement based on the extreme fibre stresses in the wall, f_t. All applicable axial forces and bending moments are applied to each segment, and each unit length is treated as a structural column.

The resulting stress is then multiplied by the wall thickness to create a stress per meter. The reinforcement can then be determined using a column design methodology where the bending of the minor axis as well as the axial stresses due to both the major axis bending and the axial forces are applied.

SLABS

For simply supported slabs spanning in two directions, the maximum bending moment in the two directions are given by:

$$M_{\text{sx}} = n \cdot \frac{l_x^2}{8} \qquad \text{in direction of span lx} \tag{71}$$

$$M_{\text{sy}} = n \cdot \frac{l_y^2}{8} \qquad \text{in direction of span ly} \tag{72}$$

Where:

n is the total ultimate load per unit area

ly is the length of the longer side

lx is the length of the shorter side

The area of reinforcement in directions l_x and l_y respectively is:

$$A_{\text{sx}} = \frac{M_{\text{sx}}}{0.87 \cdot f_{\text{yk}} \cdot z} \tag{73}$$

$$A_{\text{sy}} = \frac{M_{\text{sy}}}{0.87 \cdot f_{\text{yk}} \cdot z} \tag{74}$$

Where z is the lever arm between the resultant forces acting on the cross-section of the slab, and is given by:

$$z = d[0.5 + \sqrt{(0.25 - K/1.134}] \tag{75}$$

Where $K = M/bd^2 f_{\text{ck}}$

FOUNDATIONS

The foundations of all the columns of the two layouts will be isolated square footings. In this case, there is no moment and the pressure is uniform:

$$p = \frac{N}{BD} \tag{76}$$

The following steps are followed in order to design the footings of the buildings:

1. Calculate the plan size of the footing using the permissible bearing pressure and the critical loading arrangement for the serviceability limit state. In this case, a permissible bearing pressure of $300kN/m^2$ will be considered.

$$\text{Bearing pressure} = \frac{\text{Total desing axial load}}{\text{Required base area}} \tag{77}$$

$$\text{Required base area} = \frac{1.0G_k + 1.0Q_k(kN)}{300(kN/m^2)} \tag{78}$$

2. Calculate the bearing pressures associated with the critical loading arrangement at the ultimate limit state.

3. Assume a suitable value for the thickness (h) and the effective depth (d).

4. Carry out a preliminary check for punching shear to ensure that the footing thickness gives a punching shear stress which is within the likely range of acceptable performance.

   The basic control perimeter for checking punching shear is at distance $2d$.

$$\text{Basic control perimeter} = \text{column perimeter} + 4 \cdot \pi \cdot d \tag{79}$$

   The punching shear force will be:

$$V_{\text{Ed}} = \text{factored soil pressure} \cdot (\text{footing base area} - \text{basic control area}) \tag{80}$$

   And the punching shear stress:

$$\nu_{\text{Ed}} = \frac{V_{\text{Ed}}}{\text{perimeter} \cdot d} \tag{81}$$

5. Determine the reinforcement required to resist bending.

6. Make a final check for the punching shear

3.4 Material strength

The value of the design compressive strength of concrete is defined as

$f_{\text{cd}} = \alpha_{\text{cc}} \cdot f_{\text{ck}}/\gamma_{\text{c}}$

Where:

$\alpha_{\text{cc}} = 0.85$ is the coefficient taking account long term effects on the compressive strength and of unfavorable effects resulting from the way the load is applied

$gamma_{\text{c}}$ is the partial safety factor for concrete

And the value of the design yield strength of reinforcement is

$f_{\text{yd}} = f_{\text{yk}}/\gamma_{\text{s}}$

Where γ_{c} is the partial safety factor for steel

3.5 Maximum and minimum reinforcing steel areas

BEAMS AND SLABS

The area of longitudinal tension reinforcement should not be taken less than $A_{\text{s,min}}$.

$$A_{\text{s,min}} = 0.26 \cdot \frac{f_{\text{ctm}}}{f_{\text{yk}}} \cdot b \cdot d \geq 0.0013 \cdot b \cdot d \quad (82)$$

Where f_{ctm} is the mean value of axial tensile strength of concrete:

$$f_{\text{ctm}} = 0.3 \cdot f_{\text{ctm}}^{(\frac{2}{3})} \leq C50/60 \quad (83)$$

$$f_{\text{ctm}} = 2.12 \cdot \ln(1 + (f_{\text{ck}} + 8)/10)) > C50/60 \quad (84)$$

And it should not exceed $A_{\text{s,max}}$

$$A_{\text{s,max}} = 0.04 \cdot A_c \quad (85)$$

COLUMNS

The area of longitudinal tension reinforcement should not be taken less than $A_{\text{s,min}}$.

$$A_{\text{s,min}} = max\{\frac{0.10N_{\text{Ed}}}{f_{\text{yd}}}; 0.002 \cdot A_c\} \quad (86)$$

And it should not exceed $A_{\text{s,max}}$

$$A_{\text{s,max}} = 0.04 \cdot A_c \quad (87)$$

3.6 Emission intensities

Table 1 provides ECC values after ICE and EcoInvent to propose average default values.

Table 1: Recommended default values for the ECC of structural materials

	Material	**ECC ($kg_{\mathbf{CO2e}}/kg$)**
Concrete	Standard	0.11
	High Strength	0.13
Steel	Sections	1.14
	Sheeting	2.56
	Studs	1.24
	Plates	2.46
Rebar	65% recycled content	1.24

4 Results

4.1 Number of storeys

To study the influence that the number of storeys has on the embodied carbon dioxide emissions of the building, the following building of the first layout was analyzed: the floor area of the building is 300 square meters and the span of the beams will be 5 meters, except of the longitudinal beams connected to the shear walls, which span will be 2.5 meters. It will have three longitudinal divisions and four transverse divisions.

Table 2: First layout data to analyze the influence of the number of storeys

Characteristics	**Values**
Floor area	$300m^2$
Long beams span	$5m$
Short beams span	$2.5m$
Longitudinal divisions	3
Transverse divisions	4

And the following building from the second layout: the floor area of the building is 200 square meters and the span of the beams will be 5 meters for the long ones and 2.5 for the short ones. It will have four longitudinal divisions and four transverse divisions.

Table 3: Second layout data to analyze the influence of the number of storeys

Characteristics	**Values**
Floor area	$200m^2$
Long beams span	$5m$
Short beams span	$2.5m$
Longitudinal divisions	4
Transverse divisions	4

The material used is reinforced concrete - $25MPa$ with design yield strength of reinforcement of $400MPa$.

The type of soil will be a non-cohesive soil, like sand or gravel, and its allowable pressure will be $300kN/m^2$

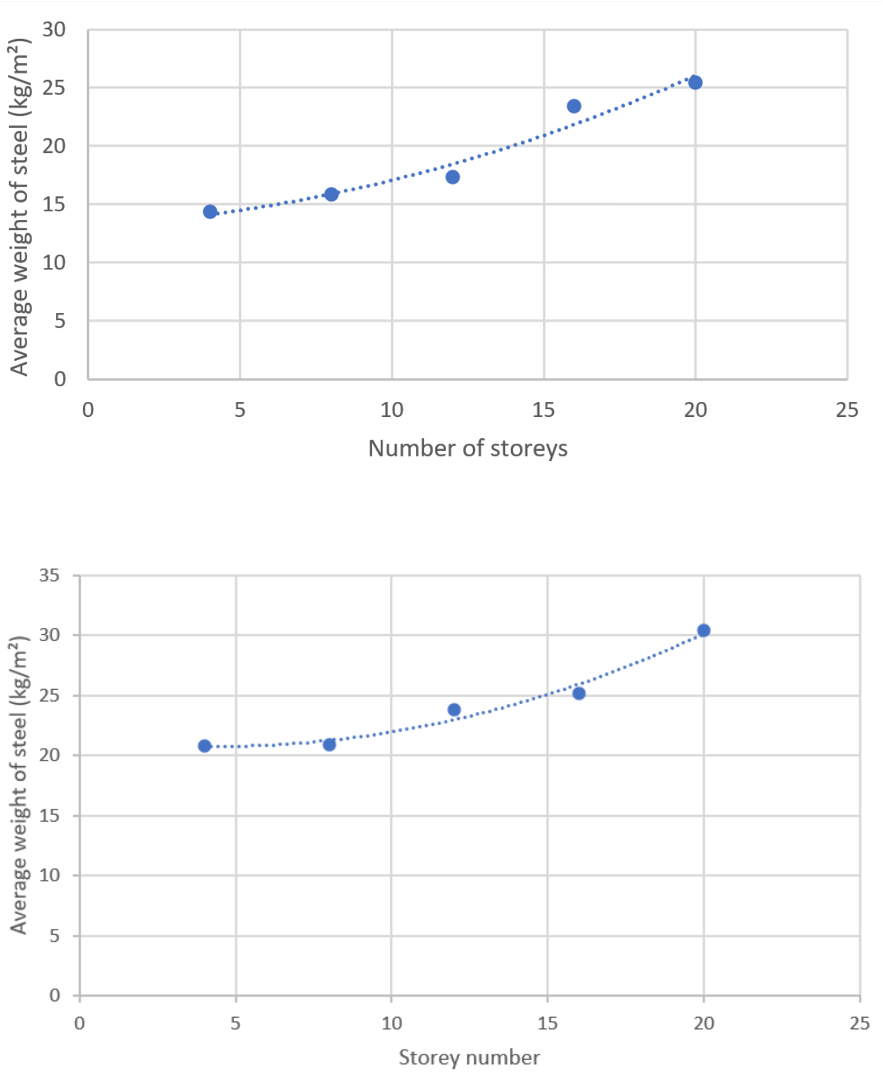


Figure 7: Weights of steel per number of storeys of the layout 1 and 2 respectively

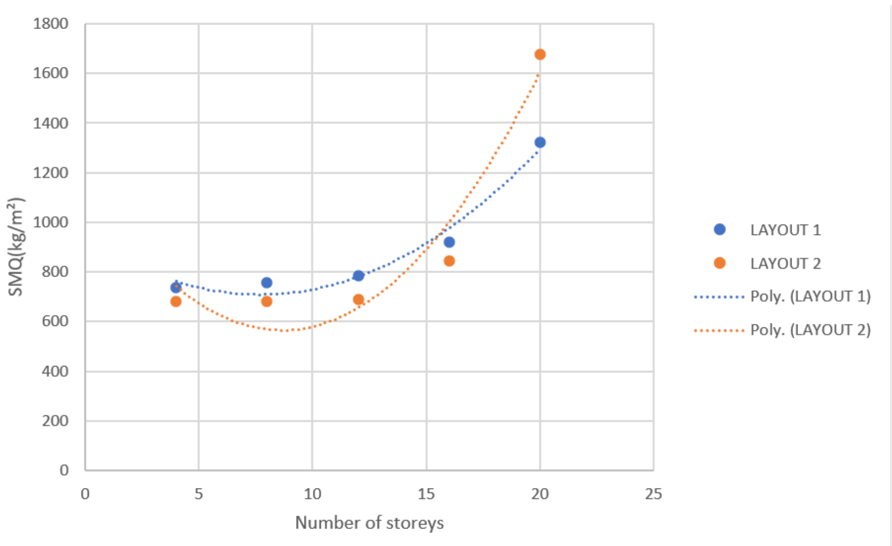


Figure 8: Structural material quantities per number of storeys for the layout 1 and 2

Taking the values of $1.24 kg_{\mathrm{CO2e}}/kg$ for steel and $0.11 kg_{\mathrm{CO2e}}/kg$ for concrete as the embodied carbon coefficients, the global warming potential for each one of the five types of building is obtained for each one of the layouts:

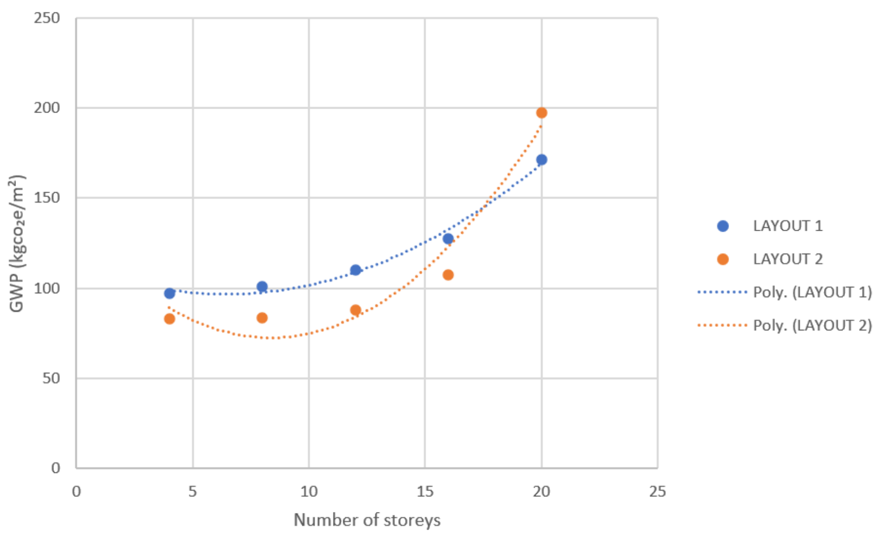
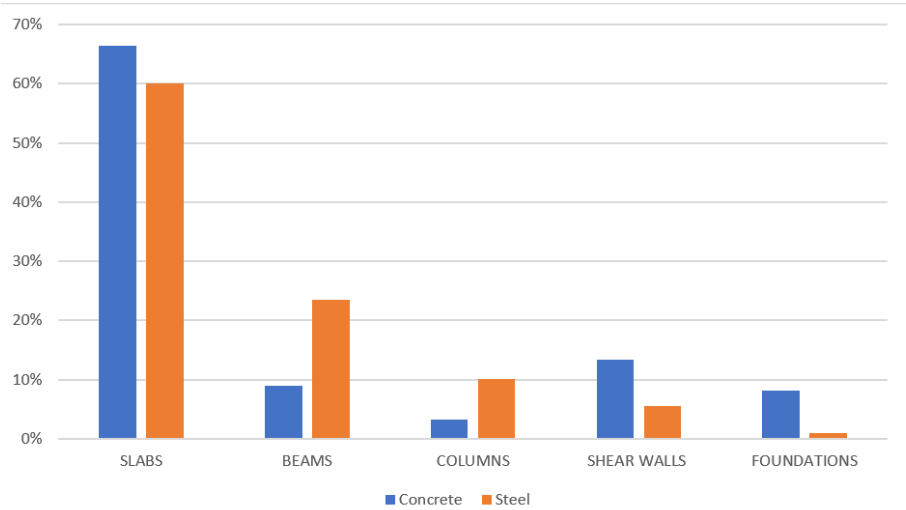


Figure 9: Global warming potential per number of storeys for the layout 1 and 2

On the following figures, it can be seen the percentage of material used on each structural member for the two layouts for a building of 4 storeys and of 20. It can be seen that increasing the number of storeys will mean an increase on the percentage of material for the shear walls and a decrease for the slabs.



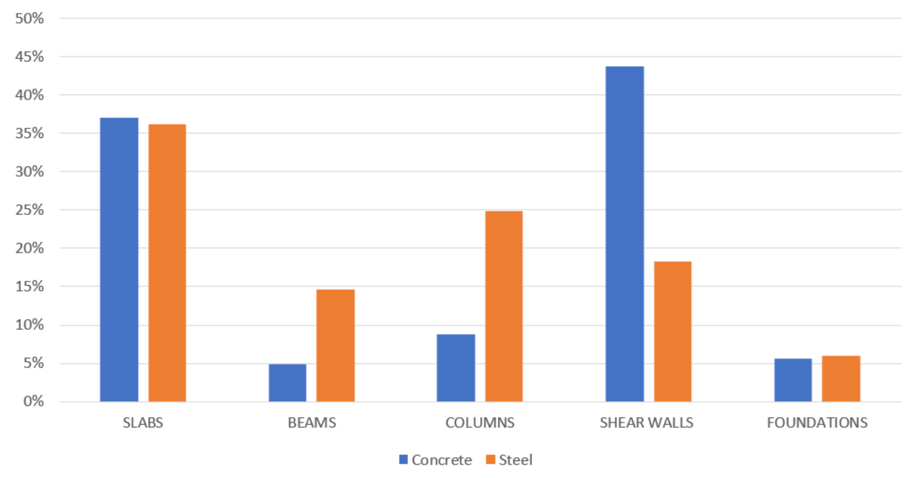
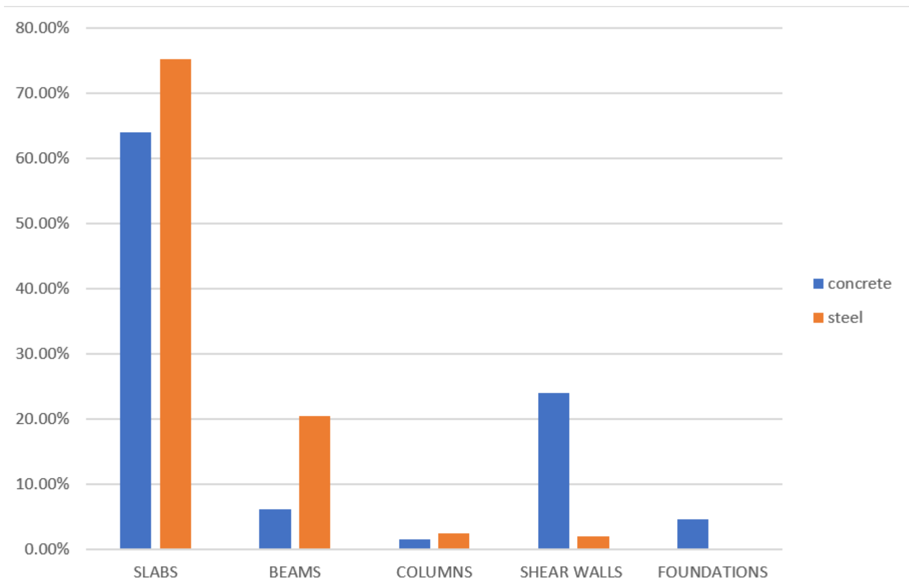


Figure 10: % of concrete and steel for each structural element of the building from the fist layout with four and twenty storeys respectively



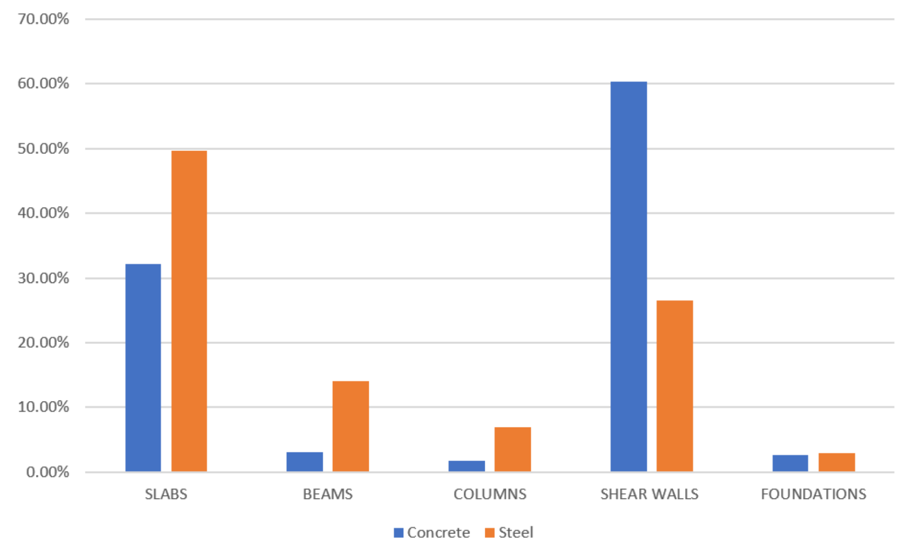


Figure 11: % of concrete and steel for each structural element of the building from the second layout with four and twenty storeys respectively

4.1.1 Floor area

To study the influence that the floor area of each layout has on the embodied carbon dioxide emissions of the building, the following building of the first layout was analyzed: the building has 6 storeys and the span of the beams will increasing by 1 meter from 4 to 7 meters, except of those which are connected to shear walls of type 2, which span will be from 2 to 3.5 meters. It will have three longitudinal divisions and four transverse divisions.

Table 4: First layout data to analyze the influence of the beams span

Characteristics	Values
Number of storeys	6
Longitudinal divisions	3
Transverse divisions	4

And the following building from the second layout: the building has 6 storeys and the span of the beams will increasing by 1 meter from 4 to 7 meters, except of those which are connected to shear walls of type 2, which span will be from 2 to 3.5 meters. It will have four longitudinal divisions and four transverse divisions.

Table 5: Second layout data to analyze the influence of the beams span

Characteristics	Values
Number of storeys	6
Longitudinal divisions	4
Transverse divisions	4

The material used is reinforced concrete - $25MPa$ with design yield strength of reinforcement of $400MPa$.

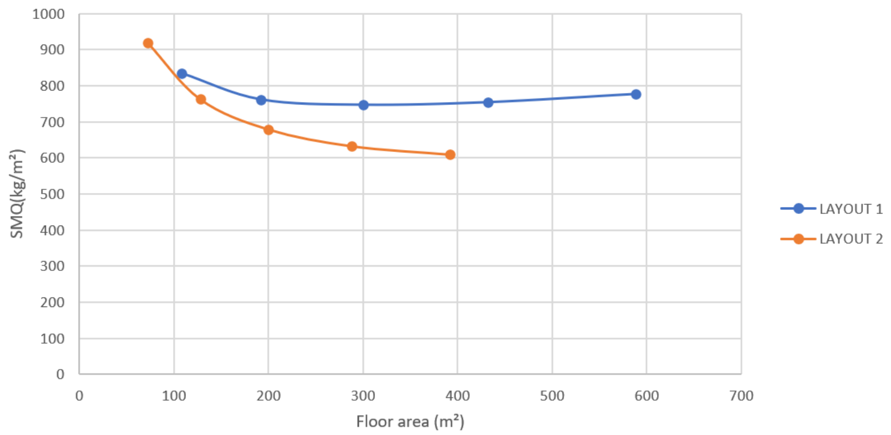


Figure 12: Structural material quantities per floor area for the layout 1 and 2

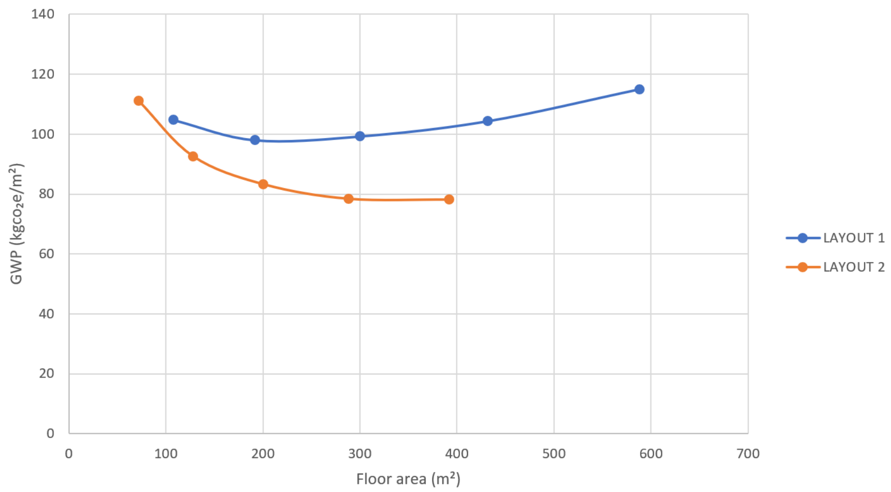


Figure 13: Global warming potential per floor area for the layout 1 and 2

4.1.2 Partial safety factors

To analyze the influence of the partial safety factors on the embodied carbon dioxide emissions of the structure, the following building from the first layout was analyzed:

Table 6: First layout data to analyze the influence of the partial safety factors

Characteristics	Values
Floor area	$300m^2$
Long beams span	$5m$
Short beams span	$2.5m$
Longitudinal divisions	3
Transverse divisions	4
Number of storeys	6

The material used is reinforced concrete - $25MPa$ with design yield strength of reinforcement of $400MPa$.

The type of soil will be a non-cohesive soil, like sand or gravel, and its allowable pressure will be $300kN/m^2$

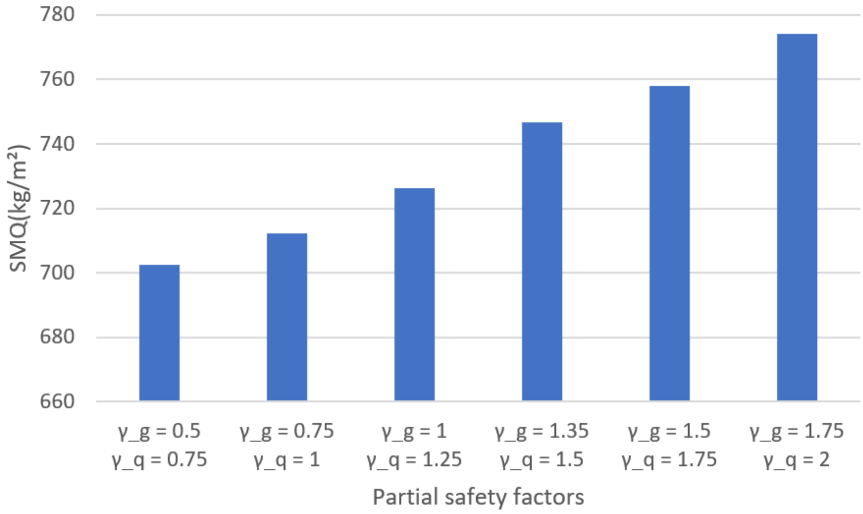


Figure 14: Structural material quantities depending on the partial safety factors of the layout 1

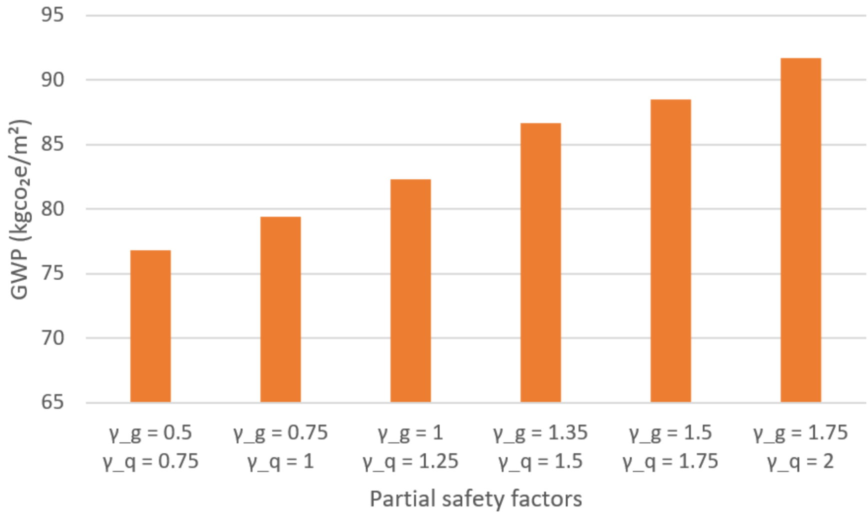


Figure 15: Global warming potential depending on the partial safety factors of the layout 1

And the same was done for the second layout with a building with the following characteristics:

Table 7: Second layout data to analyze the influence of the partial safety factors

Characteristics	**Values**
Floor area	$200m^2$
Long beams span	$5m$
Short beams span	$2.5m$
Longitudinal divisions	4
Transverse divisions	4
Number of storeys	6

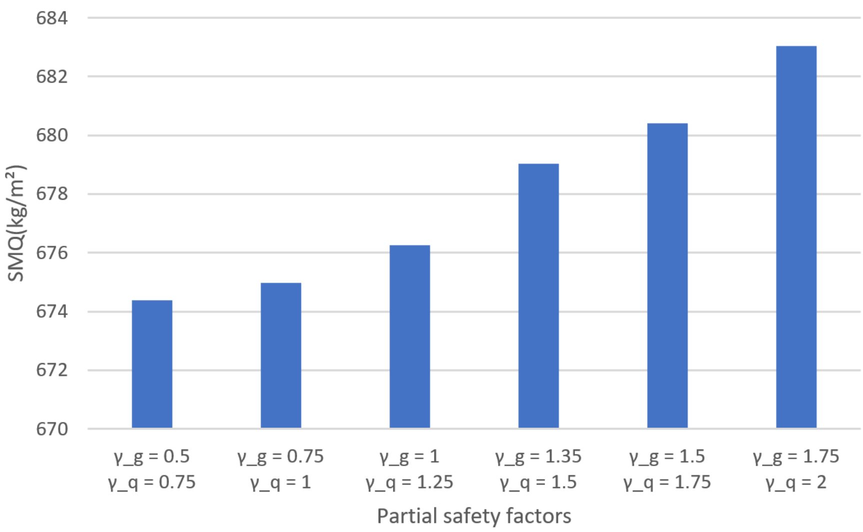


Figure 16: Structural material quantities depending on the partial safety factors of the layout 2

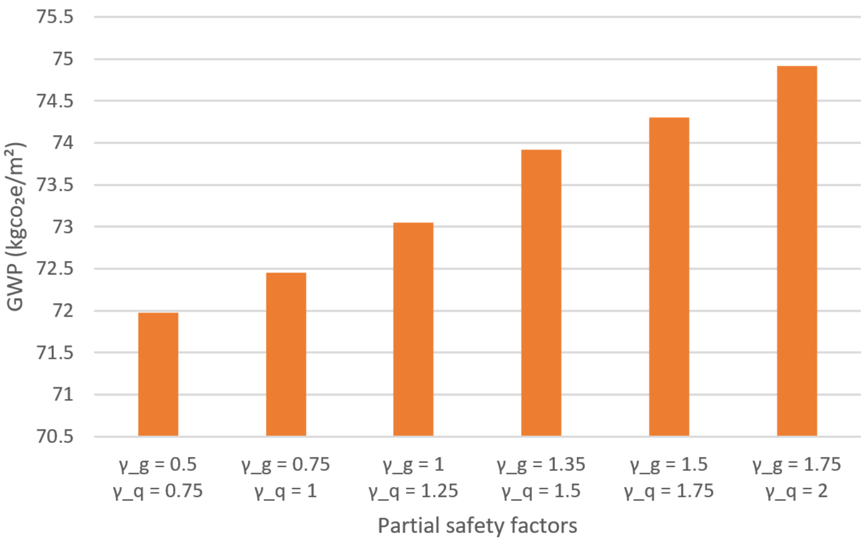


Figure 17: Global warming potential depending on the partial safety factors of the layout 2

5 Discussion

From the figure 7, it can be seen that with an increasing number of storeys, the average weight of steel per storey floor area not only increases due to the columns and shear walls required for gravity loads, but also due to the increasing amounts of structure required to resist lateral wind load. These data are plausible by comparing them, for example, with the study made by Fazlur Khan (Khan and Rankie, 1981), which measures the weights of steel per number of storeys:

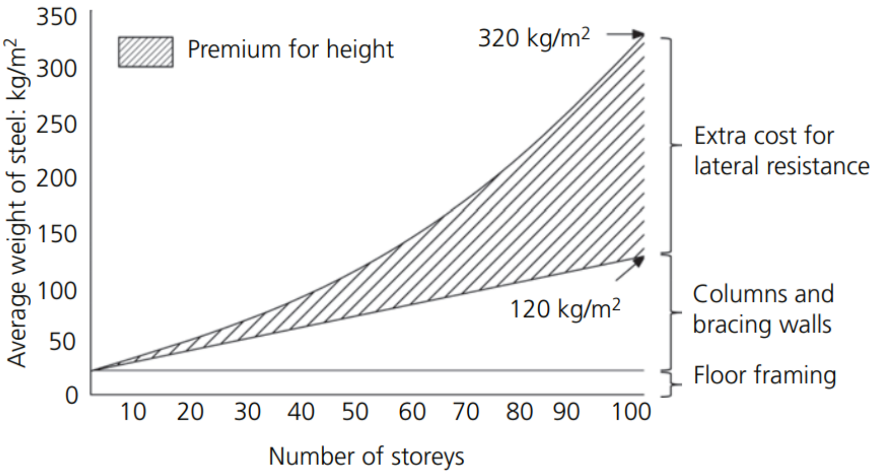


Figure 18: Weights of steel per number of storeys (adapted from Khan and Rankie (1981) and Ali (2001))

From the figure 8 it can be observed that, for this type of building, the increase of the structural material quantities becomes more pronounced as the number of floors increases. For the first layout, while increasing the building from five to ten storeys will mean an increase of the structural material quantities of around $50kg/m^2$, increasing it from 16 to 20 storeys will make the structural material quantities increase around $400kg/m^2$. Whereas for the second one, increasing the storeys of the building from five to ten will not mean an increase of the structural material quantities, and increasing it from 16 to 20 storeys will make the structural material quantities increase around $850kg/m^2$.

One of the reasons of this is the thickness of the shear walls on both layouts in order to neglect global second order effects on the building: the thickness of the shear wall from the first layout goes from $435mm$ for 16 storeys to $1180mm$ for 20 storeys, and for the second layout it goes from $325mm$ to $1000mm$.

Comparing the results of the structural material quantities per number of storeys with the study done by De Wolf, Yang; Cox et al., shown in the figure 19, it can be seen that the results in both cases are very similar: the structural material quantities begins to grow from the range of 10 to 100 storeys .

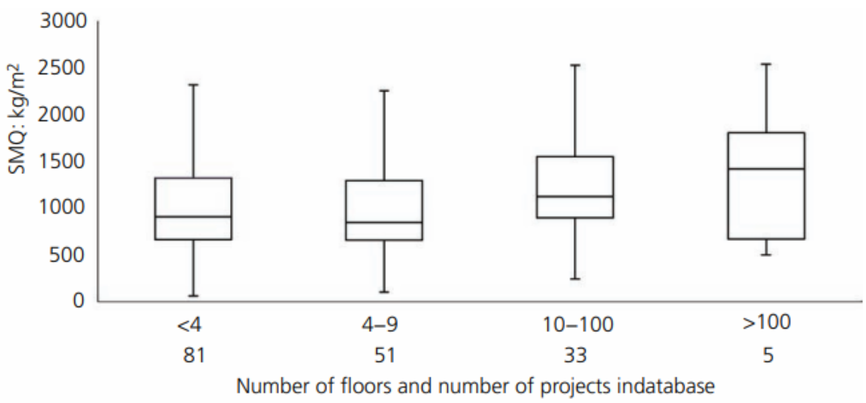


Figure 19: Ranges of material quantities and embodied carbon dioxide equivalent for 200 real projects per size in height

Comparing the results of the structural material quantities per floor area with the same study, it can be seen that the results in both cases are very similar: in both cases the structural material quantities for a floor area less than $1000m^2$ is between 910 and 250 kg/m^2 as it can be seen in the figure 20.

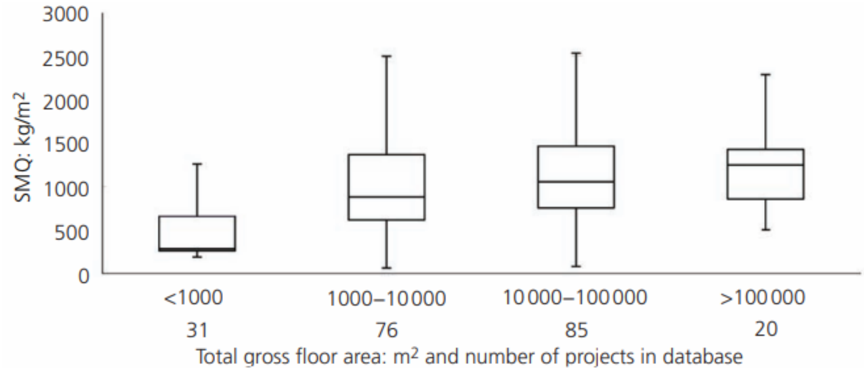


Figure 20: Ranges of material quantities and embodied carbon dioxide equivalent for 200 real projects per size in total floor area

Structural material quantities in my study are greater those of said study. This difference may be due to the fact that the study focuses on analyzing 200 existing buildings of various types and built with different materials, while my study focuses on reinforced concrete office buildings.

Comparing my results with the systematic study done by Hart et al.(which is shown on the figure 21 and has its results represented by the dashed lines), it can be seen that they are pausible: whereas the embodied carbon dioxide emission per square meter of reinforced concrete buildings according to that study is between 40 and 290 kg_{CO2e}/m^2, on my study it is between 80 and 200 kg_{CO2e}/m^2.

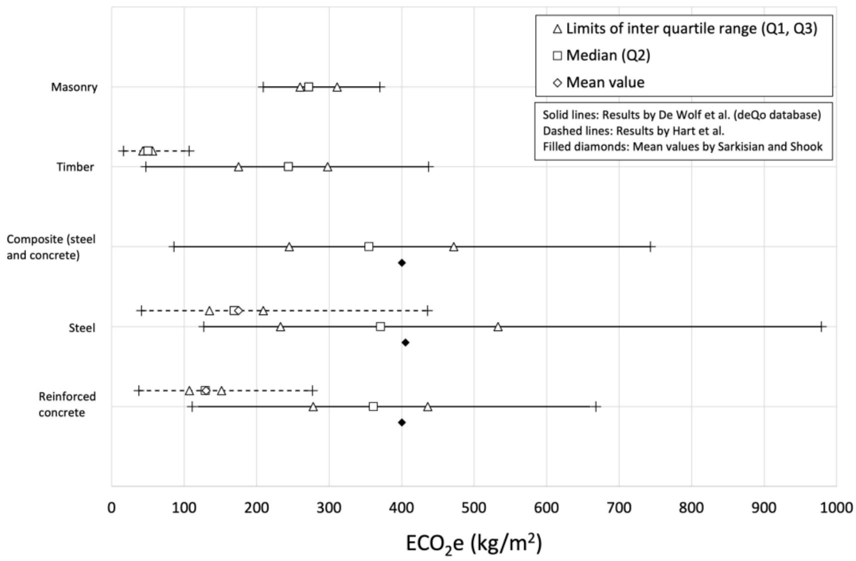


Figure 21: Statistical description of different data sets for ECO2e of structures of different constructive materials

6 Conclusion

In conclusion, the study of embodied carbon dioxide (CO2) emissions in buildings has provided insights into the significant environmental impact they generate throughout their lifecycle. Identifying effective strategies to reduce these emissions has become a critical priority for addressing climate change and promoting sustainability in the construction sector.

The major contribution of this study is to achieved a more unified method to determine the embodied carbon dioxide on structures.

In order to develop a uniform method to determine embodied carbon dioxide, the aim of this research is to parametrize two different reinforced concrete building layouts, to analyze and compare the embodied carbon emissions generated by each building design with the specified dimensions throughout their production stage. Being a case-specific parameter study allows to determine the optimal frame geometry that can be used to increase material efficiency in different typologies of building structures.

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