Lowering ECI/MPG value by implementing sustainable foundations for houses

Master thesis

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by

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Preface

With this master thesis, I will finalise my master's study Building Engineering with a specialisation in Structural Design at the Delft University of Technology. This thesis is about lowering the MPG/ECI value by implementing a sustainable foundation for lightweight houses. My goal for this research is to show an alternative, more sustainable foundation design and process. I am curious about the development of more environmentally friendly foundation designs.

This research was conducted in collaboration with the Delft University of Technology and the Advice and Engineering department of BAM Netherlands, one of the largest contractors in the Netherlands. I want to extend my heartfelt gratitude to all BAM Advice and Engineering members for their warm welcome at their office in Bunnik and for generously sharing their invaluable experiences and knowledge on structural aspects. I wish to express a special thanks to my company supervisors, Sander Vernooij and Tom Blankendaal, for their guidance and support throughout the past nine months. I also would like to express my gratitude to the NVAF for extending an invitation to present on the topic of sustainable foundations and to get feedback on the practical part. Additionally, I am grateful to my graduate committee members, Roel Schipper, Henk Jonkers, and Mandy Korff, for their insightful feedback and engaging discussions.

In conclusion, I aspire that this thesis serves as an initial stride toward fostering a more sustainable approach in the foundation design for housing projects, ultimately contributing to a reduction in the environmental impact of the construction sector.

Sebastiaan Hogerheijde
Delft, November 2023

Abstract

Climate change is one of the most significant health problems, posing a threat to progress in development, global health, and poverty reduction. The building sector substantially contributes to this challenge, due to its elevated global warming emissions. BAM developed its timber house concept FLOW, to create more sustainable and affordable housing. However, the environmental focus lies on the engineered timber superstructure above ground, while the foundation below ground level is often forgotten. The foundation design and involved processes must also be integrated to achieve a more sustainable and comprehensive FLOW house.

This research aims to investigate and minimise the Environmental Cost Indicator (ECI) of the foundation of a lightweight housing units, such as FLOW. The following research question was formulated: How to reduce the ECI/MPG value of the timber FLOW housing units by implementing a low environmental impact foundation and installation method? To answer this main question, a detailed and extensive Life Cycle Assessment (LCA) was conducted for three foundation variants: The prestressed prefab concrete piles, the timber foundation piles with concrete caps and a shallow concrete strip foundation. Because of the different soil profiles in the Netherlands, the foundations are designed for the sand-based soil in Zwolle and the clay-based soil in Delft. The ECI values of the three foundations are calculated using the LCA method. The values are analysed and subsequently optimised to minimise the environmental impact. Finally, the foundations are compared on their ECIs and characteristics to complete a comprehensive comparison.

From the LCA study of the three foundation variants, the timber foundation design results in the lowest ECI, which is €23 in Zwolle and -€402 in Delft. These low values are caused by the negative global warming potential due to the CO_2 storage of the spruce timber. The prestressed variant results in a total ECI value of €62 in Zwolle and €390 in Delft. The shallow strip foundation is only feasible in Zwolle due to unacceptable high settlements in Delft. Due to the high required volume of concrete, the shallow strip has the largest ECI value of €89. To minimise the ECI of these foundation variants, various sustainable alternatives considering both design and processes are explored. Adopting Blast Furnace Slag cement (CEMIII) instead of Ordinary Portland Cement (CEMI), reducing concrete strength class, optimising reinforcement diameters, and using electric transport and piling rigs contribute significantly to the environmental reduction. Implementing these optimisation options results in substantial ECI reductions for each variant, with the prestressed pile experiencing more than a 30% reduction in ECI for both Zwolle and Delft. However, the most effective optimisation varies for each foundation variant and location. The results also show a high difference in environmental impact per location in the Netherlands due to the highly different soil conditions. The foundation variants comparison shows that each variant has its characteristics and that a deliberate foundation choice and optimisation must be made per project.

It is important to mention that only the foundation piles and strips are considered in detail in the LCA study and that the foundation beams and shallow strip walls are considered in a simplified manner. Because of the foundation's minimal MPG/ECI contribution, focusing on the other larger contributing elements is recommended to satisfy the 0.5 MPG requirement in 2030. A recommendation for government institutions is to subdivide installations and structural elements and establish MPG requirements for both aspects. In this way, the ECI reduction of the structural elements can also be encouraged.

Finally, timber foundation piles with concrete caps exhibit the most substantial environmental potential to realise foundations with a minimal or even positive environmental impact. Therefore, it is highly recommended to undertake further, in-depth research into the technical implications and feasibility of integrating timber foundations into the FLOW concept or similar lightweight structures.

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Nomenclature

Acronyms

AD	Abiotic Depletion
AP	Acidification
AHN	Actueel Hoogtebestand Nederland
BFS	Blast Furnace Slag (CEM III)
CC	Consequence class
CEM	Cement (class)
CLT	Cross Laminated Timber
CPT	Cone Penetration Test
ECI	Environmental Cost Indicator
EDP	Environmental Product Declarations
EP	Eutrophication
EQU	Loss of static equilibrium (Limit State)
FAETP	Freshwater Aquatic Ecotoxicity Potential
FLOW	BAM's sustainable prefab timber housing project
GEO	Internal failure of structure (incl. piles)
GFA	Gross Floor Area
GPR	Gemeentelijke Praktijk Richtlijn
GWT	Ground Water Table
GWP	Global Warming Potential
HSB	Houten Skelet Bouw (Timber Frame Construction)
HT	Human Toxicity
HVO	Hydrotreated Vegetable Oil
LCA	Life Cycle Assessment
LCI	Life Cycle Inventory
LS	Lifespan
MAETP	Marine Aquatic Ecotoxicity Potential
MPG	MilieuPrestatie Gebouwen or Building Environmental Performance
NAP	Normaal Amsterdams Peil
NVAF	Nederlandse Vereniging Aannemers Funderingswerken
NMD	Nationale Milieudata Base
NC	Normally consolidated
OCR	Over Consolidation Ratio
OC	Over Consolidated
ODP	Ozon Layer Depletion
OGB	Ontwerptool Groen Beton
OPC	Ordinary Portland Cement (CEM I)
POPC	Photochemical Oxidation Potential
SLS	Service Limit State
STR	Failure or excessive deformation ground
TETP	Terrestic Ecotoxicity Potential
UC	Unity Check
ULS	Ultimate Limit State
WBF	Water Bonding Factor
WC	Water Cement Ratio

Symbols

Symbol	Definition	Unit
$\alpha_{ct,pl}$	Factor decreasing ductility	-
α_p	Point resistance factor	-
α_s	Shaft pressure factor	-
α_t	Shaft tension factor	-
β_{ds}	Time shrinkage factor	-
γ_c	Safety factor concrete	-
γ_{dry}	Volumic weight of dry soil	kN/m^3
γ_m	Partial factors steel	-
γ_{sat}	Volumic weight of saturated soil	kN/m^3
γ_s	Volumic weight of grains	kN/m^3
γ_s	Safety factor steel	-
γ_w	Specif weight water	kN/m^3
τ_f	Maximum shear stress	kN/m^2
ϕ	Friction angle	$^\circ$
$\varnothing$	Diameter	mm
λ_y	Slenderness bending on y-axis	-
λ_z	Slenderness bending on z-axis	-
$\lambda_{rel,y}$	Relative slenderness bending on y-axis	-
$\lambda_{rel,z}$	Relative slenderness bending on z-axis	-
Φ	Creep factor	-
ϕ_d	Rotation pile	-
$\phi_r eq$	Maximum rotation pile	-
ρ	Density	kg/m^3
φ	Angle internal shear	$^\circ$
σ	Total vertical stress	MPa
σ'	Effective stress	MPa
$\sigma_{ct,eff}$	Effective tension strength concrete	MPa
σ_{cm}	Working stress concrete	MPa
σ_{gd}	Design value of soil pressure	MPa
σ_h	Horizontal stress	kPa
$\sigma_{m;y;d}$	Bending y-direction	MPa
$\sigma_{m;z;d}$	Bending z-direction	MPa
$\sigma_{p'}$	Stress due to prestressing	MPa
$\sigma_{p,i}$	Initial stress prestress steel	MPa
σ_{pw}	Working stress prestress steel	MPa
$\sigma'_{v;z,o}$	Initial vertical grain stress for low-middle depth	kN/m^2
$\Delta\sigma'_{v;z,o}$	Vertical effective stress reduction low-middle depth	kN/m^2
$\sigma'_{v,z;exc}$	Effective stress on depth z	kPa
a	Strip width beyond the wall edge	mm
A	Intersection	mm^2
A_c	Gross intersection	mm^2
A_n	Net intersection	mm^2
A_p	Surface reinforcement bar	mm^2
b_F	Width strip foundation	mm
c	Cohesion	kN/m^2
c	Concrete cover	mm
C_{sw}	Ground heave constant	-
$C_{u;d}$	Undrained shear strength	MPa
D	Equivalent pile diameter	m
D	Distance inner/outer support transport swing	m
e	Void ratio	-
e	Eccentricity	mm

E	E-modulus	MPa
$E_{0.05}$	5 percentile E-modulus	MPa
E_{ser}	Mean E-modulus parallel grain	MPa
ϵ	Strain	-
ϵ_{ca}	Autogenous shrinkage strain	-
ϵ_{cc}	Creep concrete	-
ϵ_{cd}	Basic drying shrinkage strain	-
ϵ_{cs}	Total shrinkage reduction	-
Δh	Change of height	m
F	Point load	kN
$f_{c;o;k}$	Compressive strength parallel to grain	MPa
$f_{c;90;k}$	Compressive strength perpendicular to grain	MPa
$f_{ctk;0.05}$	5 percentile value tensile strength concrete	MPa
$f_{ctd;pl}$	Design tensile strength non-reinforced concrete	MPa
F_{cd}	Vertical load on pile	kN
$f_{cm,j}$	Mean cubic strength concrete after j days	MPa
$F_{ct,eff}$	Mean tensile capacity	Mpa
$f_{m,d}$	Strength class timber	MPa
$F_{m,k}$	Bending strength timber	MPa
$F_{nsf;d}$	Negative skin friction	kN
$f_{p;o}$	Prestress force mold	kN
$F_{r;max;i}$	Maximum bearing capacity pile with CPT	kN
$F_{r;max;tip;i}$	Maximum end bearing capacity pile with CPT	kN
$F_{r;max;friction;i}$	Maximum bearing capacity skin friction with CPT	kN
f_s	Local sleeve friction	MPa
$f_{t;o;k}$	Tension strength parallel to grain	MPa
$f_{t;90;k}$	Tension strength perpendicular to grain	MPa
f_u	Tension strength steel	MPa
$f_{v;k}$	Shear strength	MPa
f_y	Yielding strength steel	MPa
G	Shear modulus	GPa
G_{ser}	Mean shear modulus	GPa
h	Pressure height	m
h_0	Fictive thickness	mm
h_F	Height strip foundation	mm
H	Capillary rise	m
H	Thickness of soil layer	m
i_y	Radius of inertia y	mm
i_z	Radius of inertia z	mm
i	Gradient	m/m
I_c	Consistency index	-
k	Binding factor aggregate	-
k	Permeability-coefficient	m/s
k	Spring constant	N/m
k	Sub-grade reaction modulus	kN/m^3
k_h	Shrinkage coefficient	-
k_h	Height factor timber	-
k_{def}	Deformation factor timber	-
k_{mod}	Modification factor strength timber	-
$K_{\gamma;a}$	Active horizontal soil pressure factor	-
k_y	Instability factors timber y-axis	-
k_z	Instability factors timber z-axis	-
M_{cr}	Cracking moment concrete	kNm
M_{ed}	Occurring moment	kNm
M_{stat}	Static moment due to transport/lifting	kNm
M_{dyn}	Dynamic moment due to transport/lifting	kNm

M_{rd}	Moment capacity beam	kNm
M_u	Fracture moment concrete	kNm
$N_{ed,max}$	Maximum centric pile load	kN
N_c	Cohesion factor (Prandtl)	-
N_j	Mean cubic strength cement after j days	MPa
N_q	Upper loading factor (Prandtl)	-
n	Porosity	-
p	Pore pressure	kPa
p	Load increment	kN/m^2
q	Volume flow	m^3/s
q	Distributed load	kN/m
$q_{b;max;i}$	maximum tip resistance with CPT i	MPa
q_c	Cone resistance	MPa
$q_{c;z;NC}$	Reduced cone resistance in OC	MPa
$q_{c;z;OC}$	Measured cone resistance in OC	MPa
$q_{c;z}$	Cone resistance before excavation	MPa
$q_{c,z;exc}$	Reduced cone resistance on depth z	MPa
R_c	Insulation value	m^2K/W
$R_{c;cal;max}$	Maximum design value of pile resistance	kN
R_{cd}	Total bearing capacity pile (excl negative skin)	kN
$R_{c;net;d}$	Total bearing capacity pile (incl negative skin)	kN
R_f	Friction ratio	%
S	Impact coefficient dynamic behaviour	-
s	Deformation of soil settlement	m
S_d	Settlement pile	m
S_r	Saturation degree	%
u	Pore Water pressure	kN/m^2
v	Poisson ratio	-
V	Mean flow rate	m/s
V_a	Volume air	m^3
V_s	Volume grains	m^3
V_w	Volume water	m^3
V_{ea}	Total volume soil	m^3
V_{pr}	Volume of the voids	m^3
W	Water ratio	-
W	Moment resistance	m^3
W_a	Weight air	kN
W_{ea}	Total weight soil	kN
W_s	Weight grains	kN
W_v	Volume water ratio	%
X_u	Height compressive zone	mm
ξ_3	Correlation factor (mean)	-
ξ_4	Correlation factor (min)	-
z	Soil depth	m
z	Potential head water	m
Z_w	Ground heave	m

Contents

1 Introduction

1.1. Research context

Climate change is one of nowadays biggest health threat problems facing humanity and threatening progress in development, global health and poverty reduction. One of the largest contributors to climate change is the increased greenhouse gas emissions in the atmosphere. Since the industrial revolution, the greenhouse gas level increased. However, in the last decades, the emission growth increased rapidly. The total amount of greenhouse gas has increased by 80% since 1970, resulting in an atmospheric CO_2 concentration of 420 ppm [42]. This increased greenhouse gases in the atmosphere resulted in too much heat in the atmosphere, resulting in an increased earth surface temperature of 0.75°C over the last century [69]. Global warming results in more extreme weather conditions, food supply disruptions, respiratory disease from smog and air pollution, increasing wildfires, floods and many more severe effects on life on Earth.

To mitigate the increasing global warming and to keep the increasing average global temperature below 2°C above the pre-industrial level, the Paris Agreement was adopted. Each country's Nationally Determined Contribution is expected to be more ambitious than the previous one. Therefore, the Netherlands aims to lower the total greenhouse gas emissions by 49% in 2030, compared to 1990, which comes down to a reduction of 116 Mton. Besides that, the Netherlands needs to be completely energy-neutral, circular and climate-resistant in 2050 because of the Paris climate change agreement. However, from the yearly climate and energy exploration, it turns out that the Netherlands has only a greenhouse gas reduction of 34%, which comes down to 15 percentage points or 34 Mton, under the stated climate goal of 2030 [43]. In the Netherlands, the building sector is responsible for 50% of the total raw material use, 40% of the energy use and 35% of the total CO_2-emission. Moreover, the production of building materials in 2018 was responsible for 11% of the total worldwide energy and process-related emissions [61].

The application of timber as a construction material increased rapidly to lower the environmental impact of materials and, thereby, buildings. Especially engineered timber is used more often because of its beneficial physical and mechanical properties. Timber is considered a sustainable building material because it can lock up carbon emissions and does not deplete the earth's natural resources. Timber can grow and be harvested over and over again, which makes it an infinite material. BAM developed its FLOW housing concept because of the increasing demand for sustainable and affordable housing. The FLOW houses have many timber elements, which are constructed off-site. With the combination of sustainability, industrialising and digitisation, homes can be built with a high amount of design freedom within a limited time. The mean project lead time can be reduced to 3 instead of 12 months. Besides, the houses are demountable and reusable because of the 'dry' connections between the timber elements. By using parameterisation and digitisation, different building plots can be used optimally. Length and width, layout and finishing, can be adapted to the client's style [6].

1.2. Problem definitions

The building sector is one of the biggest contributors to the global warming problems and, therefore plays an important role in the transition to a more sustainable and environmentally friendly world. Because of the climate change agreements between countries and parties, the building sector has a high need for more sustainable building materials and applications to lower its environmental footprint. To quantify the environmental performance of a building, the MPG (Milieuprestatie Gebouw, or Building Environmental Performance) value is developed. The MPG value can be calculated with the environmental costs indicator (ECI) divided by the lifespan and GFA of the building. The Dutch government requires an MPG calculation for every office function and new house, with a total surface larger than 100 m^2. The MPG value can be calculated by dividing the environmental impact by the total surface of the building. Since 2021, the maximum MPG value has been lowered from 1.0 to 0.8 for new houses. Eventually, the maximum MPG value will be lowered to 0.5 in 2030 for both offices and dwellings [15]. Using primarily bio-based building materials like timber in housing projects, the MPG value of those projects can really be reduced in comparison with the more regular concrete or brick/calcium-silicate houses. The production of the more regular materials like concrete and steel requires a lot of energy and raw materials, resulting in more CO_2-emission, pollution and destruction of landscapes.

Looking at the FLOW timber houses, almost the entire structure above ground level is made from engineered timber, which has a lower environmental impact than the regular concrete/steel houses. However, the whole foundation, located below ground level, is made from regular reinforced prefab concrete. The regular concrete pile and floor foundation with the corresponding highly energy-demanding pile installation, due to the high emitting diesel engines, contributes significantly to the environmental impact and, therefore MPG value of the timber housing projects. Especially in dwellings with a few building layers, the MPG value can be high because of the small surface of the total building and the relatively high amount of material used. Besides that, the share of the foundation in the total MPG value will be higher with smaller surface buildings in comparison with larger surface buildings. Looking at four regular concrete/steel reference houses and only considering the structure of the building (no installations or built-ins), the foundation contributes 14% to the total MPG value [15]. In addition to the environmental impact of using reinforced concrete as the foundation material, the transportation of prefab foundation piles to the construction site and the pile-driving process also contribute significantly to environmental degradation. These activities involve heavy machinery, which has a notable ecological footprint. For instance, a pile driving machine consumes approximately 250 litres of fuel per day, resulting in emissions of about 675 kg of CO_2 [70]. Furthermore, it should be noted that the emissions associated with foundation construction occur over a relatively short period compared to the overall duration of constructing the entire building. Unfortunately, environmental considerations are often overlooked during foundation design [51], providing room for improvement to further reduce the MPG/ECI value of FLOW timber houses. These improvements can be achieved by optimising the foundation design and involved processes.

1.3. Aim and Objectives

The goal of this research project is to minimise the MPG/ECI value of the timber FLOW housing units by implementing a low environmental impact foundation. This encompasses both the design of the foundation and the associated installation processes. The findings of this study contribute valuable insights into the environmental impact of foundations and provide measures for reducing their ecological footprint, thereby lowering the MPG/ECI value. Importantly, these outcomes can also be applied to other (lightweight) structures, broadening the potential scope of their environmental benefits. To reach the goal of this research, the following objects must be investigated:

- Determine the design requirements, boundary conditions, calculation methodology and LCA method for the foundation designs to achieve an unambiguous comparison.

- Calculate and analyse the environmental impact of each foundation design variant.

- Optimise the involved design parameters per variant to lower the environmental impact and minimise the ECI/MPG value.

- Compare the optimised design variants with each other based on MPG/ECI value, but also on other characteristics like costs, complexity and feasibility.

1.4. Research questions

This report consists of multiple chapters, all answering one or more sub-questions. Answering the individual sub-questions, mentioned in this paragraph, will result in the answer to the main research question of this thesis:

> *"How to reduce the ECI/MPG value of the timber FLOW housing units by implementing a low environmental impact foundation and installation method?"*

Chapter 2 - Literature study

The literature study will touch upon multiple different topics. The first one is to figure out the correct calculation methodology for the MPG/ECI value of a dwelling and which environmental aspects need to be included. A thorough soil mechanics analysis is required to investigate the various soil types in the Netherlands and their influence on the foundation design. Furthermore, the design requirements must be investigated, encompassing environmental considerations and other essential factors. Lastly, an examination of the existing installation and transport methods for foundation design is necessary.

- What is the right MPG/ECI calculation methodology, and what to include?
- How do soil characteristics and mechanics influence the foundation design?
- What are the design requirements for the foundation design besides the environmental impact?
- What are the possible foundation installation and transport methods?

Chapter 3 - Preconditions foundation and LCA

In this chapter, the FLOW timber housing needs to be analysed, and the preconditions for the foundation and LCA need to be defined. First, the dimensions and elements with corresponding loads of FLOW need to be investigated. Moreover, the current MPG value of the FLOW house needs to be determined to use as a benchmark for the research. Besides that, a comparison will be made between the ordinary concrete housing units and the timber FLOW dwellings concerning weights. Moreover, two typical CPTs will be used for both the west and east of the Netherlands. Based on the literature study and experiences from the past, three foundation design variants will be chosen to include in this research. Finally, the boundary conditions for the LCA will be clearly defined to get an unambiguous comparison.

- What is the design of the FLOW houses with corresponding weights?
- What is the current MPG value of the timber FLOW housing units?
- Which CPTs are typical for the Netherlands, and what are the detailed soil conditions of those CPT locations?
- What are the boundary conditions of the LCA and how to get an unambiguous comparison?

Chapter 4 - Prefab prestressed concrete pile

The first considered design variant will be the prefab prestressed concrete pile foundation. The environmental impact of the prestressed piles for both CPT locations will be calculated. Finally, the concrete pile parameters will be optimised to minimise the environmental impact and to investigate the highest measurement potential.

- How to calculate the structural capacity of the prestressed prefab foundation pile and what are the foundation dimensions for both Zwolle and Delft?

- What are the environmental impacts of the considered concrete piles for both CPT locations?
- How to optimise the concrete piles' parameters to minimise the environmental impact and MPG value?

Chapter 5 - Shallow strip foundation
The second design variant is the shallow strip foundation, which is an interesting variant because of the weight-saving of the timber FLOW unit. By applying a shallow foundation instead of a pile foundation, the total material may be lowered. Moreover, shallow foundations have a high re-use potential because of the relatively easy removal. However, due to the limited bearing capacity, it is necessary to investigate the feasibility of using shallow foundations for both CPT locations. Finally, the design parameters for the timber foundation piles will be optimised to minimise the ECI/MPG value.

- Has the shallow foundation enough bearing capacity for the FLOW unit in combination with weaker soil conditions for both locations, and what are the strip dimensions?
- What are the environmental impacts of the considered shallow foundation for both CPT locations?
- How to optimise the parameters for the shallow foundation to minimise the environmental impact and thereby MPG value?

Chapter 6 - Timber pile foundation
The third and final design variant is a timber foundation pile chosen because of the low environmental potential due to CO_2 storage. The environmental impact of the timber piles needs to be investigated. However, unlike the other variants, timber piles are more likely to have a serious degradation mechanism, especially with a low GWT. Therefore, the possible degradation mechanisms and effects need to be investigated. Finally, the design parameters for the timber foundation piles will be optimised to minimise the ECI/MPG value.

- How to deal with the possible degradation mechanism of the timber piles?
- What are the environmental impacts of the timber foundation pile design for both CPT locations?
- How to optimise the parameters for the timber foundation pile to minimise the environmental impact and, thereby MPG value?

Chapter 7 - Foundation variant comparison
This chapter will display and compare the environmental impact results and observations of all the considered design variants for both locations. Moreover, the ECI values of the variants including foundation beams and strip wall will be shown. Besides comparing the environmental impact, other design factors like costs and complexity will also be considered. This way, an extensive and comprehensive comparison between the variants can be made.

- Which design variant has the lowest environmental impact for the CPT location in Zwolle and Delft?
- What is the environmental impact of the variants including foundation beams and strip wall?
- What is the performance of the foundation variants based on other design requirements like costs and complexity?

Chapter 8 - Discussion
The chapter will discuss all boundary conditions and assumptions and their effects on the results of this research. Besides that, the environmental data and results will be critically analysed and discussed.

Chapter 9 - Conclusion and recommendations
The chapter will finalise the outcome of the previous chapters. The main research question will be answered. Besides that, recommendations for improvements and suggestions for further research will be mentioned.

1.5. Methodology

The research will follow a systematic approach to minimise the MPG/ECI value of the FLOW housing unit by implementing a low environmental impact foundation. It will involve the following steps: The first step is to conduct a comprehensive literature study on the MPG/ECI value, soil mechanics, and foundation designs. This will provide a solid foundation of knowledge for the research. Next, the design requirements and evaluation of existing foundation designs will be analysed. This will help identify areas for improvement and guide the development of the low environmental impact foundation. The FLOW housing unit will be analysed in detail, considering the weights and aspects contributing to the environmental impact. This analysis will provide insights into the specific requirements and challenges for the foundation design. Clear boundary conditions will be defined for the life cycle assessment (LCA) analysis, specifying the scope and inclusion criteria for the environmental analysis. This will ensure a consistent and comprehensive assessment of the foundation variants. Considering the high variability of soil conditions in the Netherlands, two representative Cone Penetration Test (CPT) profiles will be used. One typical for the eastern part and one for the western part of the Netherlands. These profiles will provide a representative research field for assessing the environmental impact of different foundation designs. Three foundation variants will be investigated: prefab prestressed piles, concrete shallow foundations, and timber piles with concrete caps. Each variant will be developed with proper designs and specifications. LCAs will be conducted for each foundation variant, considering both CPT locations. The resulting ECI values will be displayed and analysed. Design parameters such as strength class, reinforcement ratio, transport methods, and installation techniques will be optimised to minimise the ECI value for each variant. The (optimised) variants will be compared and evaluated for both CPT cases, analysing the effects of optimisation on their environmental impact. The different foundation variants will also be compared on important design aspects, including costs, quality, and complexity. In this way, an integrated and comprehensive comparison between the foundation variants can be created. The global methodology for this research is depicted in Figure 1.1

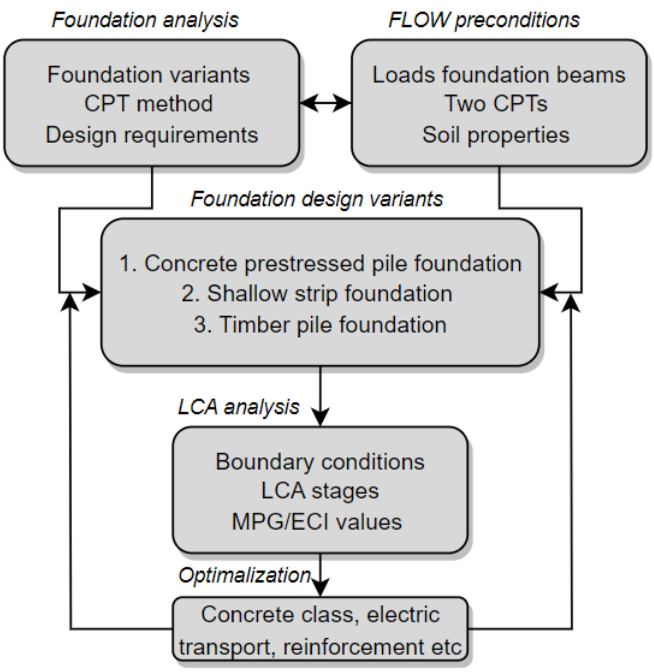


Figure 1.1: Methodology overview of the research, with an optimisation between different foundation parameters and the environmental impact

1.6. Scope

The life cycle stages that will be included in the LCA of this research will cover the entire product and construction process stage (A), except for the transport of the raw materials (A2). However, it includes the raw materials supply, manufacturing, transport, construction, and installation process (A1, A3, A4 and A5). It is assumed that the foundation is properly designed and does not require any maintenance or repairs, which results in zero environmental impact during the entire use stage (B). Furthermore, it is assumed that the pile foundations will remain in the soil after their 75-year lifespan and are therefore not considered in life cycle stages C and D. For the shallow foundation, LCA stages C and D will be included due to the relatively easy removal process without significant soil consequences and complexities.

To achieve an unambiguous calculation method for the MPG value, the Dutch National environmental database will be used as much as possible for environmental data. This commonly used environmental database will make the analysis's outcome controllable, reproducible and unambiguous. As a basis for the determination method, the European standard, NEN-EN 15804: Sustainability of construction works - Environmental product declarations, will be used [16]. Environmental data from a single database will be utilised to ensure a consistent and reliable comparison of different foundation variants. This approach enables a comprehensive and unambiguous assessment of the variants. Additionally, using data from a single source helps maintain transparency regarding the origin of the data and prevents potential environmental deviations that may arise from using multiple databases.

This research will particularly focus on the environmental impact of the foundation design and processes. However, to make this research more practical, other design requirements like costs and complexity will also be covered for the foundation variants. In this way, the variants can be compared on multiple important requirements besides the environmental impact.

This research will mainly focus on the foundation piles and the shallow strip. The foundation beams and shallow strip wall will not be analysed in depth. However, to the environmental impact of the foundation beams and strip wall will be calculated on a simplified way, to achieve an all-comphresenvive ECI value of the entire foundation design.

The technical building installations inside dwellings or offices have a significant contribution to MPG value [27]. However, this research will focus on the environmental impact of the foundation of the housing units. The improvements on the technical installations and other structural elements will be out of scope.

An essential aspect of the foundation design process is the potential for pile failure during the dynamic piling process, particularly when dealing with stronger sand layers. The likelihood of splitting or cracks occurring in the foundation pile is significant in such conditions. However, due to time constraints and the complexity of researching the dynamic forces involved in the piling processes, this research does not consider the construction calculations of the dynamic forces during the piling process.

2
Literature study

This chapter covers the literature study, which is required to analyse and reduce the MPG/ECI value of the foundation of the FLOW timber housing units. Section 2.1 covers the meaning and background of MPG/ECI value and the right LCA methodology. Section 2.2 covers the soil mechanics and the CPT method, which are both important aspects involving the design of a foundation with corresponding bearing capacity. Section 2.3 covers the foundation design requirements besides sustainability and will address both the shallow and pile foundations with possible installation and transport. Finally, the conclusion of this chapter will be given in section 2.4.

2.1. MPG/ECI value

MPG is an abbreviation for the Dutch word 'Milieuprestatie Gebouwen', which means Building Environmental Performance. The MPG value is an important criterion for the sustainability and durability of a building. The value consists of a single score, expressed in euros per square meter gross floor area (GFA) times the lifespan (LS). The MPG of a building is the sum of the shadow costs of all applied materials in the building, divided by the lifespan and total usable surface, which can be seen in equation 2.1. The lower the MPG value, the more sustainable the use of materials. The MPG is an objective instrument in the design process and can be used in the schedule of requirements to document the results of the design process [62]. The Dutch government requires an MPG calculation for new offices and houses with a total surface larger than 100 m^2. Since 2021, the maximum allowed MPG value has been lowered from 1.0 to 0.8. Eventually, the maximum allowed MPG value will be lowered to 0.5 in 2030 for both offices and dwellings [15].

$$MPG = ECI/(GFA \cdot LS) \tag{2.1}$$

2.1.1. Life Cycle Assessment (LCA)

Quantifying the degree of sustainability of a housing unit can be difficult, especially when all aspects of people, planet and profits have to be considered. A LCA is a technique which asses the environmental and health impact of a product during its entire life cycle stages. The environmental impacts particularly address the depletion of finite resources and harmful emissions. However, these are limited to aspects that can be quantified regarding environmental impact. The social aspects are excluded from the LCA studies because of the quantifying difficulties [44]. The LCA method was developed by the Institute of Environmental Sciences of Leiden University [33]. These methods were defined later in the European standards ISO 14040 and ISO 14044. A more detailed description of construction works was documented in the standard EN 15987, EN 15804 and ISO 14025 [44]. According to the ISO 14040 standard, the procedure for performing requires four specific steps, which are depicted in Figure 2.1:

1. **Defining goal and scope** of the study, including the definition of the functional unit of the targeted product or process.
2. **Life Cycle Inventory analysis (Process tree)**, i.e. listing all of the environmentally relevant inputs and outputs in the products' various life cycle stages.

3. **Impact assessment**, i.e. aggregating the various impacts into several relevant environmental impact categories and calculating scores for each one.
4. **Interpretation** of the environmental impact of the product and discussion of obtained results.

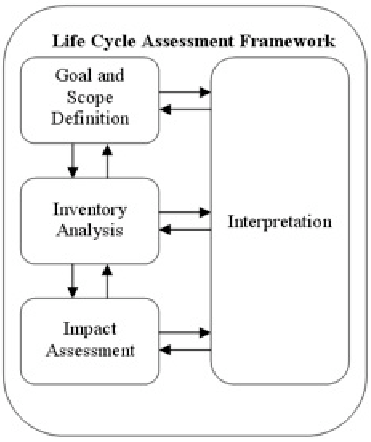


Figure 2.1: Scheme of the 4 required phases of an LCA procedure [44]

According to NEN-EN 15804 + A2:2019, the stages in Figure 2.2 must be included in the LCA analysis. Usually, in the LCA environmental impact, the 'cradle to grave' was determined, which included stages A, B and C. Alternatively, a partial LCA was conducted by considering only stages A1-A3 and A1-A5, which results in 'cradle to gate'. However, since July 2022, the entire life cycle stages must be considered in the LCA. This includes the end-of-life (C) and the benefits of end-of-life recycling (D). According to the NEN-EN 15804 + A2, the life cycle stages A1 to D must be included for a valid and complete life cycle assessment. Category D covers the substantial value at the end of the functional service life of a product, where it can be completely or partly re-used. However, re-usability is still highly unusual, especially for the regular reinforced concrete foundation. A reason for this is that, after the possible lifespan of 75 years, the re-use potential of certain materials is uncertain and that building codes may have changed in the meantime [44]. It is decided to construct a new foundation to prevent those difficulties and complications.

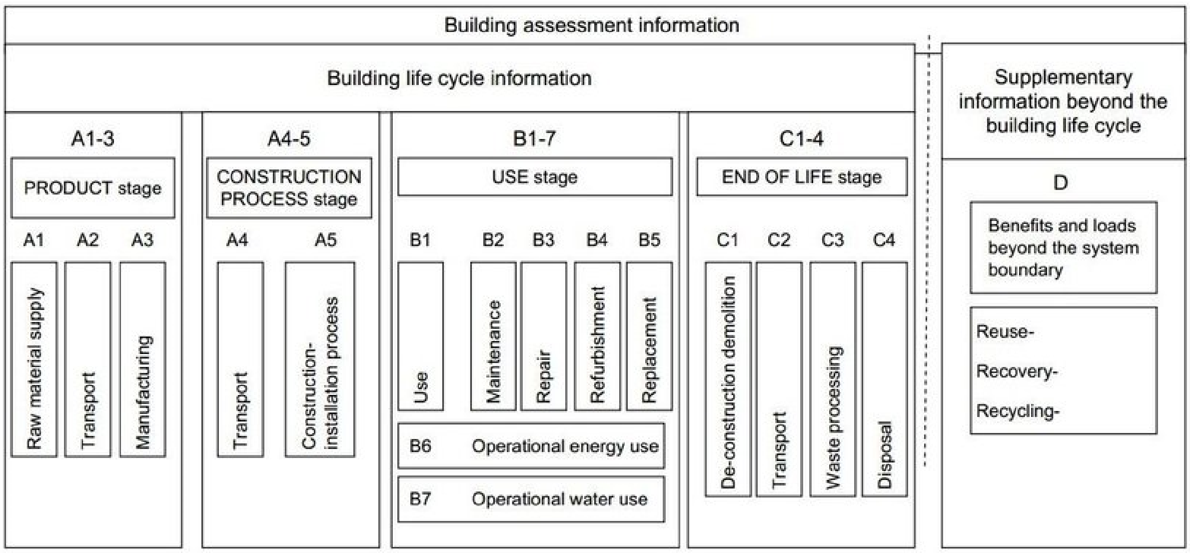


Figure 2.2: Specific life cycle stages considered in the LCA [64]

2.1.2. LCA-Calculation tools

Environmental impact studies using quantitative data are based on Life Cycle Inventory databases. These contain data about raw materials, processes, and products. However, there are many different (inter)national databases. Each database differs concerning the included environmental impact categories and local and non-local resources. Therefore, to achieve an unambiguous environmental analysis, it is not desirable to mix data from different databases. For this scientific research and to compare different projects, the underlying LCI data must be unambiguously and clear [44]. Therefore, the environmental impact data of raw materials and building materials are mainly derived from the Dutch National Environment Database (NMD). This widely nationally used database contains thousands of material and process data. Manually processing of these involved data requires time-consuming labour. Therefore, private specialised LCA and EPD programs have been developed to automatise this process. Examples of commonly used tools in the Netherlands are [47]:

- GPR Buildings
- DGBC-Breeam, Materials tool
- Dubocalc
- Ontwerptool groen beton

The MRPI, GPR and DGBC tools are suitable for building projects and Dubocalc for civil works. The ”Ontwerptool groen Beton” can be used for concrete structure elements, including processes.

2.1.3. Impact categories and ECI value

An important aspect is to quantify the environmental impact of a material or process. The Environmental Cost Indicator (ECI) merges relevant environmental impact into a single monetised score, after which the MPG value can be easily determined. The monetised value represents the costs to make the environmental impact undone. These costs are called the ’shadow costs’ of the product. The shadow costs represent the environmental damage to society. By internalising these environmental costs in the sale price, the ’polluter-pays-principle’ comes into force. In Table 2.1, the impact categories are listed according to the old EN15804 + A1 standard. By adding to each impact category from Table 2.1 the shadow costs per equivalent unit, the total ECI value can be calculated. These shadow costs are based on averaged damage and prevention costs for the individual impact categories and are assembled in the NMD [44]. However, in Table 2.2, the 13 latest core impact categories are listed, which must be considered according to the latest ECI norm NEN-EN 15804 + A2 (July 2022). This norm includes more impact categories and has a better connection with the Product Environmental Footprint (PEF) of the European Commission. However, the latest standard has some consequences for the Dutch construction sector, which is used to the old standard. Because the latest A2 standards do not contain ECI values and weighting factors (yet), both calculation standards are still required for market parties.

Impact category	Unit equivalent	Shadow costs (€)
Abiotic depletion non-fuel (ADP)	kg Antimone eq.	0.16
Abiotic depletion fuel (ADP)	kg Antimone eq.	0.16
Global warming (GWP100)	kg CO_2 eq.	0.05
Ozone layer depletion (ODP)	kg CFC-11 eq.	30.00
Photochemical oxidation (POCP)	kg Ethene eq.	2.00
Eutrophication (EP)	kg PO_4 eq.	9.00
Acidification (AP)	kg SO_2 eq.	4.00
Human toxicity (HTP)	kg 1,4-dichloro benzene	0.09
Fresh water aquatic ecotoxicity (FAETP)	kg 1,4-dichloro benzene	0.03
Marine aquatic ecotoxicity (MAETP)	kg 1,4-dichloro benzene	0.0001
Terrestrial ecotoxicity (TAETP)	kg 1,4-dichloro benzene	0.06

Table 2.1: Old environmental impact categories for ECI (NEN-EN 15804 + A1).

Impact category	Indicator	Unit equivalent
Climate change – total	Global Warming Potential total (GWP-total)	kg CO_2 eq.
Climate change fossil	Global Warming Potential fossil fuels (GWP-fossil)	kg CO_2 eq.
Climate change - biogenic	Global Warming Potential Biogenic (GWP-biogenic)	kg CO_2 eq.
Climate change - land use and change	Global Warming Potential land use and land use change (GWP-luluc)	kg CO_2 eq.
Ozone Depletion	Depletion potential of the stratospheric ozone layer (ODP)	kg CFC 11 eq.
Acidification potential	Accumulated Exceedance (AP)	mol H+ eq.
Eutrophication aquatic freshwater	Eutrophication potential fraction of nutrients reaching freshwater end compartment (EP-freshwater)	kg PO_4 eq.
Eutrophication aquatic marine	Eutrophication potential, fraction of nutrients reaching marine end compartment (EP-marine)	kg N eq.
Eutrophication terrestrial	Eutrophication potential, Accumulated Exceedance (EP-terrestrial)	mol N eq.
Photochemical ozone	Formation potential of tropospheric ozone (POCP);	kg NMVOC eq.
Depletion of abiotic resources	Abiotic depletion potential for non-fossil resources (ADP minerals and metals)	kg Sb eq.
Depletion of abiotic resources - fossil fuels	Abiotic depletion for fossil resources potential (ADP-fossil)	MJ, net calorific value
Water use	Water deprivation potential, deprivation-weighted water consumption (WDP)	m^3 world eq. deprived

Table 2.2: Latest core environmental impact indicators for ECI (NEN-EN 15804 + A2).

An important aspect of the latest NEN-EN 15804 + A2 standard is that many EPDs and databases only contain the 11 impact categories from the old standard. The environmental data about products and processes are not largely freely available, and a minimum amount of the latest environmental data is available. This is partly caused by the fact that a product declaration lasts 5 years, and therefore many products/processes are still not updated according to the latest NEN-EN 15804 + A2 standard. Besides, the shadow costs are not yet applied to the latest impact categories. Therefore, comparing different foundation variants with each other is more complex because of the different impacts per category and not one single monetised value. Therefore, it is decided to perform the LCA study based on the 11 old impact categories instead of the latest 13 categories.

2.1.4. Lifespan and GFA

Two other parts that influence the MPG value are the total lifespan of the building and the Gross Floor Area (GFA), as seen in Equation 2.1. The GFA is the total floor area inside the building envelope, including the external walls and excluding the roof. The influence of the GFA is relatively high with smaller housing units. This results from the many used materials per GFA in combination with the regular necessary installations and facilities. The final parameter on the MPG value is the lifespan of the structure. The default lifespan of housing units is 75 years. If the building has a shorter lifespan of 75 years and unmodified materials, the MPG value will increase. On the other hand, if the default building lifespan of 75 years is exceeded, the MPG value will decrease. Other important design parameters influencing the MPG value are the number of building layers, floor height, facade surface and open parts in the facade. However, because the timber FLOW units are prefabricated, these parameters will not be analysed because the focus of this research will be on the foundation part. To determine the GFA, the standard NEN 2580 must be used. In this way, an unambiguous comparison between the two buildings can be made. The specific FLOW dimensions and characteristics can be seen in subsection 3.1.1. The lifespan and GFA will be constant for this research.

2.2. Soil mechanics

One of the first steps in designing the foundation is to investigate the soil conditions underneath the structure. Technical soil investigations are performed to determine the (mechanical) properties of the soil layers. The different soil properties result in specific bearing capacities of the different soil layers. The soil type underneath a future building can be determined with a Cone Penetration Test (CPT). The soil's physical properties can influence the soil's mechanical behaviour and, therefore, bearing capacity. The characteristic values of the different soil types in the Netherlands can be found in Appendix A.1.

2.2.1. Different soil types

In the Netherlands, there is a large variety of soil types like clay, peat, sand, gravel and loam. Layers such as peat, clay and loam are considered weak layers with a minimum bearing capacity. Sand layers with enough thickness are stronger and provide enough bearing capacity for a foundation pile. Globally, it can be observed that the eastern and southern parts of the Netherlands predominantly consist of sand layers, while the western regions are characterised by compressible clay and peat layers at the top 5-10 m, as depicted in Figure 2.3. Consequently, the settlements in the western parts are much larger than in the eastern regions, as seen in Figure 2.4. Sand and gravel layers consist of relatively large grains, which have no coherence with each other. However, they can have slight hook resistance. Sand and gravel are well permeable to water and difficult to compress. The grain size determines the distinction between sand and gravel. Sand has a grain size between 0.0063 mm and 2 mm, and gravel between 2 mm and 64 mm. Stones have a grain size larger than 64 mm. If the soil grains are smaller than 0.063 mm, they are called silt clay or loam (Table. 2.3). These soil layers are compressible and have a low water permeability. Peat, however, often has a highly compressible horizontal structure with a high water content [71]. If the clay and peat layers are loaded, they will be compressed, and the structure will settle over time. These settlements can cause, for example undesired imperfections or cracks. Therefore, a deeper sand layer needs to be searched for a deeper sand layer to create enough bearing capacity. As mentioned, the sand layers in the southern and eastern parts of the Netherlands are directly beneath the surface. Therefore, placing the structure directly on the surface may be possible. When the structure is placed directly on a load-bearing top sand layer, it is categorised as a shallow foundation.

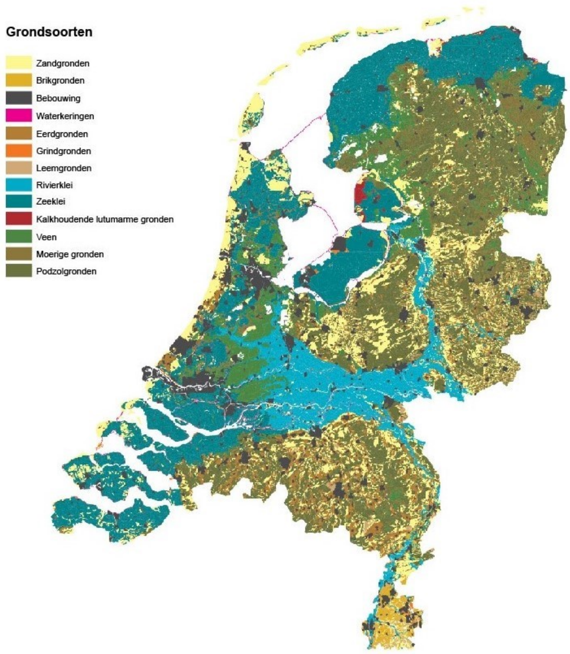


Figure 2.3: Soil Types in the Netherlands [66]

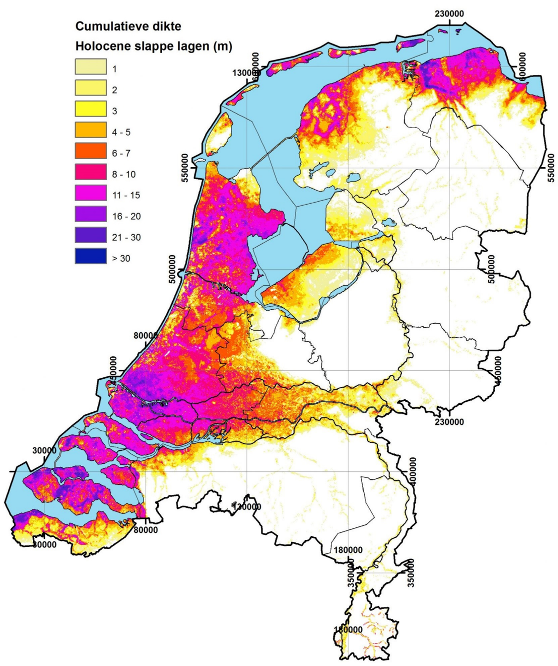


Figure 2.4: Cumulative layer settlement [19]

Each soil layer in the Netherlands has its properties and thereby bearing capacity. Therefore, the structure's location has a major influence on the bearing capacity and, therefore foundation design. The mean soil parameter properties per soil type can be found in appendix A.1. Besides the global soil type distribution around the Netherlands, the local soil type profile is also important. The soil distribution can vary locally, so a detailed soil investigation is required. The most important physical properties of the soil that influence the bearing capacity of the foundation can be found in Appendix A.1.

Soil type	Grain size [mm]	Density [kN/m3]	Permeability coefficient [m/s]
Clay	< 0.002	14 - 20	10^{-7} - 10^{-11}
Peat	-	10 - 13	10^{-7} - 10^{-9}
Loam	0.002 - 0.063	19 - 22	10^{-5} - 10^{-7}
Sand	0.063 - 2	17 - 20	10^{-2} - 10^{-5}
Gravel	2 - 63	16 - 23	10 - 10^{-2}

Table 2.3: Soil type properties (NEN 9997 table 2.b)

2.2.2. Cone Penetration Test

To investigate the local soil profile and thereby the load-bearing capacity of the soil, a Cone Penetration Test (CPT) is often carried out in the Netherlands. A CPT is a relatively simple and effective method of pushing a steel rod into the soil, measuring the force during penetration as a function of the depth. This force consists of the soil reaction at a specific point and the friction along the circumference of the rod. The method was developed in the 1930s at the TU Delft. [2]. Originally, the CPT was a mechanical test, which consisted of three movable parts with a common central axis. The CPT originally involved simple mechanical measurements of the total penetration resistance to push a tool with a conical tip into the soil. Nowadays, the CPT use an electronic cone, where both the cone resistance and the friction are measured continuously and electronically by using a system of gauges on the inside of the cone. The sensitive gauges, consisting of three parts, can measure the forces on the two lower parts of the instrument independently. To eliminate measurement errors, at least two CPTs need to be carried out [2].

With the results of the CPT, a detailed insight into the soil layers beneath the structure can be generated. The softer clay layers will have a smaller resistance than the stronger sand layers. A typical cone resistance for a sand layer is 5 or 10 MPa, while the resistance of a soft clay layer will be between 0.01 MPa and 0.1 MPa. If the friction ratio is also calculated, the difference between layer types is even more pronounced. The friction ratio is the ratio between the local sleeve friction and the cone resistance on a certain depth z, expressed in percentages (eq. 2.2). The friction ratio of a sand layer will roughly vary between 0.5% and 2%, and the friction ratio of clay will be between 3-6% as can be seen in Table 2.4 [38]. Higher friction ratios suggest a peat layer or a combination of clay and peat.

$$R_f = 100 \cdot f_s/q_c \tag{2.2}$$

Figure 2.5 shows a CPT result, where the resistance q_c and the friction angle R_f are given as a function of the depth. The allowable stress on the sand layer depends on the friction angle ϕ and its cohesion c [2]. From Figure 2.5, the first 4 meters, a clay layer can be observed because of the low resistance and high friction angle. After the 4 meters, the resistance increases and the angle decreases, indicating stronger sand layers. Conducting only one CPT will be insufficient to conclude the existence of this layer at all places. Therefore, observing the sand layers in 3 CPTs, at practically the same depth will probably be sufficient to assume the presence of the sand layer around [2]. According to NEN 9997-1 it is important to "enclose" the building surface. This can be done by placing CPTs at the corner points and on the building perimeter every 25 m. Then, the entire building surface is 'filled' with CPTs, so the distances between them, in all directions do not exceed 25 m. The depth of the CPT depends on the type of foundation. The CPT depth needs to be at least 5 meters under the pile tip for a pile foundation, while a CPT depth of 10 to 15 meters will satisfy a shallow foundation. The CPT can be performed based on four different quality classes. Class 1 and 2 have a higher precision and show besides a CPT graph numeric values about the cone resistance and time. However, for roughly 75% of the CPT in the Netherlands, classes 3 or 4 are sufficient. Only for the more specific analyses and complex situations a higher lower CPT class is needed [3].

Soil type	Friction ratio [%]	Cone resistance [MPa]
Clay	3.0 - 6.0	2.0 - 5.0
Peat	> 6.0	5.0 - 10.0
Loam	1.2 - 3.0	2.0 - 4.0
Sand	0.5 - 2.0	> 5.0
Gravel	0.2 - 0.5	15 - 30

Table 2.4: Friction ratio and cone resistance of soil types [71]

If additional information about the mechanical soil properties is needed after the CPT, drilling research will be conducted. In this way, mechanical properties like compressibility and strength can be determined. Besides that, the groundwater head levels can also be investigated with drilling research. With the given CPT results of the soil underneath the timber FLOW housing units, the bearing capacity of the layers can be determined. This is important because shallow foundation implementation can be a possible sustainable solution for the eastern parts of the Netherlands, where a thick sand layer is available directly below the ground level. However, it is important to mention that the cone resistance does not contain direct information about the bearing capacity of piles or shallow foundations. To determine the bearing capacity of a foundation, several calculations must be conducted, where different parameters follow from the cone resistance and friction ratio [71].

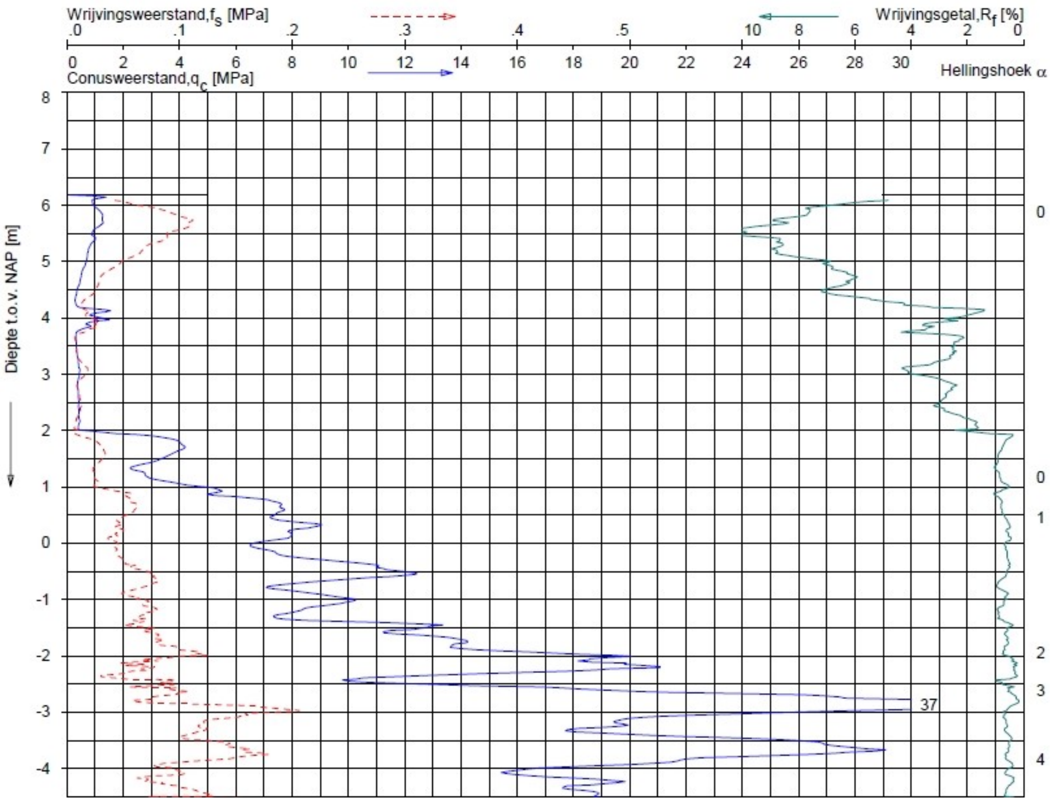


Figure 2.5: Results of a CPT, with the cone resistance and friction rate over the depth [38]

2.2.3. Method of Koppejan

The CPT result example, as seen in Figure 2.5, is a method to determine the point resistance of piles. However, this only counts for the soil displacement piles. For the other foundation types, reduction factors need to be considered. The interpretation method accepted in the Netherlands was empirically developed by Koppejan around 1950 [52]. Koppejan assumed for the bearing capacity of the pile point, the soil collapses around the pile point according to sliding planes to the Prandtl theory. He also thought that the pile transmits shear stress to the soil above the pile, contributing extra to the stresses at the pile point level and, therefore, to the pile point's bearing capacity. However, it is known that soil failure does not occur precisely according to the shear stresses. Therefore, Koppejan estimated that the influence area reached from 0.7 to 4D under the point and maximum 8D above the point. For the point resistance, there is a distinction between three influence areas, as seen in Figure 2.6. The contribution of the points' resistance under and above the point level is equal (I + II = III). The contribution of the point level exists in two equal parts (I=II).

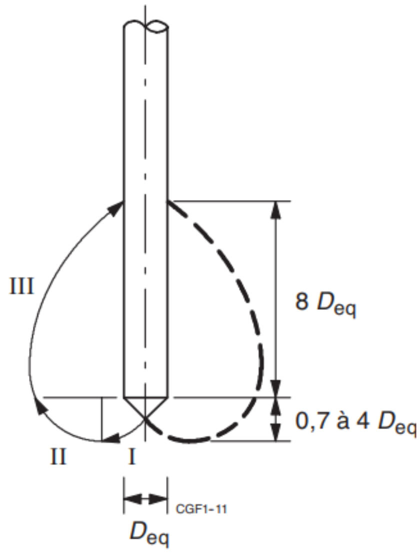


Figure 2.6: Three influence areas according Koppejan method [52]

This results in the following equation for the determination of the maximum point resistance:

$$q_{b;max} = \frac{1}{2} \cdot (\frac{q_{c;I,mean} + q_{c;II,mean}}{2} + q_{c;III,mean}) \tag{2.3}$$

The first trajectory value $q_{c;I,mean}$ runs from the pile point level to a level that is at least 0.7D and at most 4D deeper. The bottom of the path must be chosen within these limits so that $q_{b,max}$ is minimised. The $q_{c;II,mean}$ is the mean value over trajectory two, which moves from the bottom of trajectory I to pile point level. However, the resistance value can not exceed the underlying value. Finally, the $q_{c;III,mean}$ value moves from pile point to 8D higher, which can not exceed the underlying value. For auger piles, if the lowest value is greater than 2 MPa, a cone resistance equal to or smaller than 2 MPa must be considered at the bottom of path III [52].

2.2.4. Theory of Prandtl

Besides the method of Koppejan for pile foundations, there is also a theory available for soil behaviour and bearing capacity of shallow foundations. The influence of the depth of the foundation is accounted by a considered surcharge at the foundation ground level to both the left and right of the applied load. The first computations were developed by Ludwig Prandtl, who assumed a strip of infinite length and weightless soil. This is done based on the assumption that in a certain region at the soil surface, the stresses satisfy the equilibrium conditions and the Mohr-Coulomb failure criterion as seen in Figure A.4. In Figure 2.7 Prandtl shallow strip foundation theory can be seen. The foundation load of the strip foundation is denoted by p, and the surcharge next to the strip foundation is q. This can be used to represent the effect of the depth of the foundation, which is in that case q = γ * d [2].

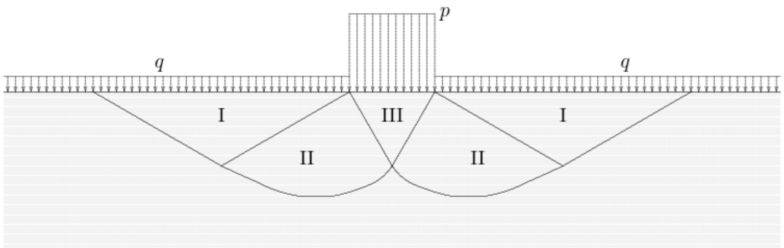


Figure 2.7: Prandtl shallow strip foundation sliding theory [2]

Prandtl's solution can be subdivided into three soil zones, as seen in Figure 2.7. Zone I represents the horizontal stress, which needs to be larger than the vertical stress, represented as the surcharge q. In zone III, the vertical normal stress is assumed to be the largest, and its value equals the unknown strip load p. The transition zone II is formed in the wedge shape (Prandtl's wedge) and is bounded by a logarithmic spiral. The analysis of the results can be written as:

$$p = C \cdot N_c + q \cdot N_q \tag{2.4}$$

Where the N_c and N_q are respectively the cohesion factor and the upper load factor, which are obtained with Prandtl's theory. The factors depend on the friction angle ϕ. If the friction angle and the cohesion factor are equal to zero, the surcharge must be similar to the bearing capacity (p=q). The factor N_y is based on theoretical analysis and experimental evidence.

$$N_q = \frac{1 + \sin(\phi)}{1 - \sin(\phi)} \cdot \exp(\pi \tan(\phi)) \tag{2.5}$$

$$N_c = (N_q - 1) \cdot \cot(\phi) \tag{2.6}$$

$$N_y = 2 \cdot (N_q - 1) \tan(\phi) \tag{2.7}$$

Prandtl's solution has been further developed by Brinch Hansen and Terzaghi, which also includes the unit soil weight. This resulted in the following formula:

$$p = c \cdot N_c + q \cdot N_q + \frac{1}{2} \cdot \gamma \cdot N_y \cdot B \tag{2.8}$$

However, the above formula 2.8 has been further extended with various correction coefficients. These factors include the shape of the loaded area, the inclination of the supposed load and the possible inclined soil surface or loading area. Brinch Hansen developed these factors into a single formula to calculate the unknown load p.

$$p = i_c s_c c N_c + i_q s_q q N_q + i_y s_y \frac{1}{2} \gamma \cdot N_y \cdot B \tag{2.9}$$

In this equation, the i_c and i_s factors are the inclination factors for the load, and the s_c and s_q are the corrections for the shape of that specific load. The other factors may be used for a sloping soil or a sloping foundation but are not considered in the shallow foundation design. The two considered checks for a shallow foundation can be derived from the NEN 9997 standard. The first verification is that the design load value needs to be smaller than the maximum bearing capacity of the soil (limit state 1A). The second check is the settlement of the shallow foundation in both the ultimate and serviceability limit state. For smaller shallow foundations on sand, the critical failure mechanism is often the maximum bearing capacity, while for larger shallow foundations on compressible layers, the settlement criteria are normative.

2.3. Foundation design

In engineering, a foundation is the element of a structure which transfers loads towards the ground. The primary task of designing a foundation is to create a technically sound, construction-feasible, and economical design to support the superstructure [55]. The superstructure is part of the building above ground level and includes elements like beams, columns, floors, and walls. The superstructure is supported by the substructure, which consists of the basement and foundation, as depicted in Figure 2.8. A distinction can be made between the shallow and pile foundations. The bearing capacity calculations of each foundation group will be researched to determine if the specific foundation type can transfer the loads of the FLOW housing.

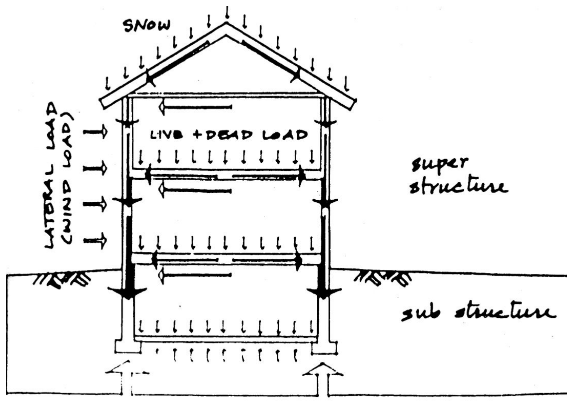


Figure 2.8: Super- and substructure elements of a building [55]

2.3.1. Design requirements

This thesis will focus on the environmental impact of the foundation design. However, besides the sustainability factor, there are multiple other requirements that the FLOW house's foundation design must meet. It is crucial to analyse those requirements because they can influence the (sustainable) design of the foundation. The following design requirements need to be analysed besides environmental impact and may affect the foundation design and installation method [3]:

- Requirements from construction
- Requirements from soil and interaction with foundation
- Requirements from building site/surroundings
- Requirements from execution
- Requirements from building physics

Requirements from construction

The main function of a foundation is to transfer the forces from the superstructure towards the soil. Therefore, it is important to investigate the line, column and slab loads of the FLOW timber houses. The pile's and shallow foundation's global dimensions can be determined with those loads. The timber FLOW housing units do not contain any basement, which makes the foundation design less complex due to the absence of buoyancy in combination with a basement.

The FLOW timber houses are single-family houses categorised as consequence class 1 structure (CC1). This implements an easy construction and foundation design without complex soil conditions or high building risks. For the design, a proper soil investigation needs to be conducted, after which the calculation can be made with the selected soil parameters.

The vertical load on the construction is not always decisive for the design. Tension forces in the pile foundation or moments caused by eccentricity can also determine the dimension of the foundation. Therefore, an elaborate load and force analysis of the FLOW houses is desirable.

Requirements from soil and interaction with foundation
As already mentioned in section 2.2, the soil conditions are important for the foundation design of the FLOW housing units. The timber FLOW houses will be constructed in multiple locations in the Netherlands. In the eastern and southern parts of the Netherlands, more sand layers are located and, therefore have a larger bearing capacity than the soft clay layers in the western part of the Netherlands. Therefore, the results of the CPT strongly influence the foundation design. It may be possible that a shallow foundation in the eastern part will satisfy, but a pile foundation is required for the western parts.

Another important aspect of the interaction between soil and foundation is the negative skin friction, as seen in Figure 2.9. In particular, the timber pile foundations resulted in numerous failures in the past. This was mainly because only timber piles were used, and the understanding of negative skin friction was not as advanced as today. Consolidating the weak layers above the sand layer resulted in an additional load on the pile foundation. Therefore, it is important to consider the additional load of the negative skin friction. [23].

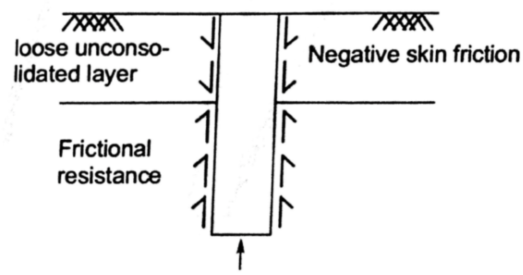


Figure 2.9: Negative and positive skin friction pile foundation [23]

Besides the interaction between soil and foundation, the groundwater level also plays a considerable role. First of all, the groundwater level table (GWT) influences the effective stress in the soil, as can be seen in equation A.17, which subsequently influences the bearing capacity of the soil. Secondly, if the groundwater decreases, settlement of buildings can occur due to desaturation of the soil. The soil can also settle simultaneously with the lowering groundwater, in which the soil develops a tension force on the foundation pile. It should also be considered that the natural groundwater level can rise rapidly during wet periods and thus strongly impede the formation of a dry building site. Finally, lowering the groundwater level can seriously affect the (older) timber foundation piles. If the timber piles are located above the water table, oxygen, fungi, and bacteria can enter the timber piles, in which degradation mechanisms can develop. If the timber piles are located under the GWT the degradation mechanism cannot occur due to a lack of oxygen. After 10-15 years of dry periods, the bearing capacity of the timber pile foundation will be lost. Therefore, during the design of a (timber) foundation, the groundwater level is of high importance [50].

Requirements from building site and surroundings
Besides the design of the foundation, the installation is also important. Especially for dense urban areas, multiple factors need to be considered. One of the most important aspects is the piling method in urban areas. Piling of foundation piles can result in vibration and noise hindrance for the surrounding areas. To limit the nuisance, there are piling alternatives that are vibration-free. Environmental noise remains a major environmental problem affecting the health and well-being of millions of people in Europe. 20% of the European population is exposed to noise harmful to health for long periods [4]. Vibrations can also be perceived as very annoying by nearby residents, and they can also cause damage to nearby buildings.

If there is a short distance between the projected new house and an existing building, there needs to be additional attention towards the condition of the existing foundation. Another crucial factor is the size and accessibility of the building site. The available space will be scarce if the newly constructed houses are located in the old city centre. This results in constraints for heavy machines for both transport and piling. However, because the FLOW houses will not be constructed in heavily dense areas and cities, and it is assumed that there will be no existing buildings around, the surrounding requirements will not be a large constraint.

Requirements from execution

Besides the design, soil, and surroundings requirements, the execution requirements are also considerable. Especially for the choice between the many different pile options, the execution requirements play an important role. Examples of execution requirements that influence the design of the foundation are production possibilities, delivery time, stability of piling machines, maximum penetration depth, limited workspace, construction time, costs or specific labour conditions. Another interesting point is the generated moments on the foundation pile during the transport and piling processes. With the support of a smooth pile on two points, the maximum moment will be 0.0214 times the distributed load and the length squared. Another important requirement is the slenderness of the foundation pile. This has to do with the transport and piling of the foundation piles. The requirements regarding the maximum length per pile width can be seen at the start of each variant chapter.

Requirements from building physics

Next to the technical requirements, the building physics aspects are also significant. Building physics requirements have a significant impact on specific aspects of foundation design, such as the installation of insulation in the crawl space and the need for exceptional detailing around various connections. For the design of a sustainable foundation, the building physics characteristics play a significant role. The insulation value R_c and the absence of thermal bridges around the foundation are important in designing a low environmental impact house.

The first building physics requirement is that the foundation needs to be watertight. Due to the water tightness of the construction parts, the quality of the indoor environment is guaranteed. According to NEN-2778:2015, the water is not allowed to permeate the construction, and the surface over a thickness of 0.01 meter cannot get humid. Especially with shallow foundations, this may be a problem because the humidity can relatively easily flow towards the indoor climate. The second requirement is the prevention of the so-called thermal bridges. In humid spots inside a building, bacteria and fungi can procreate easily, which is harmful to human health [3]. The insulation factor of the foundation itself will be sufficient. However, the connections between the floor and foundation elements are vulnerable spots that need additional attention to prevent thermal bridges. Additional insulation material is therefore desirable in vulnerable places.

The primary factor influencing the foundation design in this research is the environmental impact. The design requirements mentioned above will have a comparatively lower level of consideration in determining the foundation design. It is essential to emphasise that the main emphasis is on the environmental impact, as accommodating all of the requirements mentioned above may not be feasible within the time constraints of this research

2.3.2. Shallow foundations

Shallow foundations are placed directly on the soil of the structure and have a minimum construction depth of 0.8 meters [30]. The shallow foundation is known as "fundering op staal" in the Netherlands. Shallow foundations are an excellent option for the eastern and southern parts of the Netherlands, mainly consisting of deep and bonded sand layers, as seen in section 2.2. It may also be an interesting option for the softer clay layers in the western parts because of the relatively low self-weight of the timber FLOW building. In the Netherlands, foundation piles are often preferred over shallow foundations. The reasons are the usual practice for the contractor and thereby cost-efficient and the fear of settlements and complexities. Another reason for not using shallow foundations is the unknown soil detailed properties concerning strength and stiffness. With insufficient soil stiffness, the building can settle irregularly, resulting in cracks and foundation failure. By performing a proper soil investigation combined with a stiff shallow foundation, a lot of material may be saved compared to regular pile foundations. Moreover, shallow foundations do not need any foundation beams, probably resulting in less material and, thereby, environmental impact and cost savings. Moreover, no heavy piling rig or transport is required due to the cast-in-situ design option.

Important factors in designing shallow foundations are the groundwater level, installation level and the adjacent situations [30]. Firstly, by lowering the groundwater table in the future, the effective stress will increase, resulting in settlements (eq. A.17). Besides that, the GWT also influences the bearing capacity; the higher the GWT, the lower the effective stress and corresponding bearing capacity. Secondly, the installation level needs to be frost-free and, therefore must be located at least 800 mm below ground level. In this way, the freezing of the soil does not result in a volume increase, and the defrosting does not result in possible settlements. A deeper installation level will result in a stronger foundation because the overlying soil must be squeezed out when the soil collapses (the formation of shear planes). However, the installation level may not exceed the 1.5-meter depth because of economic and sustainable considerations [30]. Finally, the adjacent situations are important when designing a shallow foundation. Excavating the soil on behalf of the new shallow foundation can result in a collapse of the adjacent foundation because of the unilateral decay of the top load. The soil pressure under the new foundation may increase the ground pressure under the adjacent existing foundation through load spreading. Ultimately, this may cause uneven or skewed settlements [30].

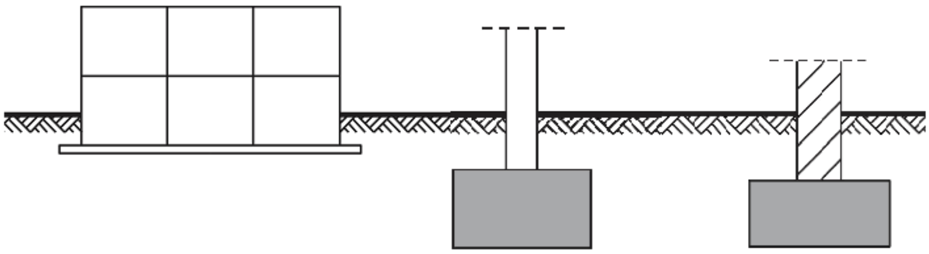


Figure 2.10: Shallow foundations; respectively Raft (point + line load), Pad (point) and Strip (line)

From the constructive load distribution and the corresponding soil mechanics, three types of shallow foundations can be distinguished: raft, pad and strip foundation (Fig. 2.10). A raft foundation is a relatively slender, highly reinforced slab with large horizontal dimensions. The normative failure mechanisms are often the moment distribution in the slab itself or the deformation of the subsoil. This is because the raft foundation cannot have a significant settlement. The second shallow foundation is a pad foundation, often a square plate with limited transverse dimensions under a point load. However, because the concentrated loads on the pads can be relatively high, the thickness must be increased (0.6 - 1.5 m). There needs to be additional attention to the eccentricity and unequal settlements because of the three-dimensional behaviour of the foundation pad. The third shallow foundation is the strip foundation. A strip foundation is often placed directly under a facade or wall and has a limited width. The load is considered a line load on the strip or a point load on the cross-section. The critical failure behaviour is often the lateral slip plane of the ground level [3].

2.3.3. Pile foundations

If the load-bearing soil layer is located a couple of meters below the construction level of a building, a deep foundation can be a proper solution. There are many different configurations and implementation options available. A deep foundation can be subdivided into three categories: the pile, cab and beam foundation [3]. However, this research will focus on the pile foundation, because the cab and beam foundation are mainly used for superstructures with larger loads than the FLOW timber building. The pile foundation design choices are determined by preconditions such as the structure above ground level, the influence of pile vibrations, the groundwater level, settlement of the weak layers above the piling level, costs, noise limitations, availability and for this research, especially sustainability. Besides the piles, the foundation beams and floors are also an important design aspect. These elements must be sufficiently stiff to transfer the loads from the structure above ground towards the foundation piles [30]. The connection between the pile, beams, and floor needs additional attention. Finally, the knowledge of soil mechanical calculations obtained in section 2.2 is required to determine the bearing capacity of the pile.

Since 1300, timber foundation piles have often been applied for housing and civil works in the Netherlands. Globally, around 1900, the first concrete and steel foundation piles were introduced because of the higher generated bearing capacity. Many more foundation piles have been introduced, mainly due to sound and vibration hindrance limitations and space limitations in and around old city centres. This has resulted in the current state of affairs, where the following types form the largest segment of the piles: timber piles (with concrete cap), driven precast concrete piles, driven cast-in-situ piles, screw-in-situ piles, and auger piles. However, many more specific pile foundations are available and may be more suitable for a particular project. In the Netherlands, the often-used materials for foundation piles are timber, steel and concrete, where concrete is the most commonly used today. The concrete piles can be both prefabricated and cast in situ. Precast concrete piles are made at the manufacturer and are delivered on-site on the day of installation, which means a less time-consuming process. However, the piling process makes the prefab piles more likely to crack. Besides that, transporting the prefab piles with heavy trucks can result in difficulties regarding the available space around the building site. Cast-in-situ piles need to be constructed in the field by excavating a hole into the soil, followed by installing the steel reinforcement and pouring concrete. This is a more time-consuming process, and there is an increased chance of mistakes because of the less controlled environment in the soil. However, the cast-in-situ piles do not require any piling process with corresponding noise and vibration hindrance [60].

There is a distinction between displacement and replacement piles for the installation of the piles in the soil. With replacement piles, the soil is removed and transferred to the surface, resulting in the soil layer loosening. Displacement piles push the soil outwards, which results in more compacted ground. Therefore, displacement piles have a positive influence on the load-bearing capacity of the soil and, therefore pile capacity. Many installation methods are possible for foundation piles, for example, driven, screwed, vibrated, pushed or jetted into the soil. Each installation method has pros and cons; per scenario, it needs to be investigated which installations fit best. As previously mentioned, depending on the soil layers, the foundation piles transfer the loads to the soil by the pile tip (end bearing) or the skin friction along the side of the pile. If the pile tip of the foundation pile is located in a sand or gravel layer, the end bearing capacity will be the highest. The friction along the side of the foundation pile can be divided into negative and positive skin friction. There is negative skin friction if the pile is placed in an unconsolidated weak layer. In this case, the additional load has to be considered, and the bearing capacity of the foundation pile will be reduced. However, if the skin friction is positive, the load-bearing capacity of the pile will increase. Therefore, the load-bearing capacity of a pile can be calculated by adding the positive skin friction with the end bearing capacity and possibly subtracting the negative skin friction, as seen in Figure 2.9. The characteristics of a pile foundation design can be seen in Figure 2.11.

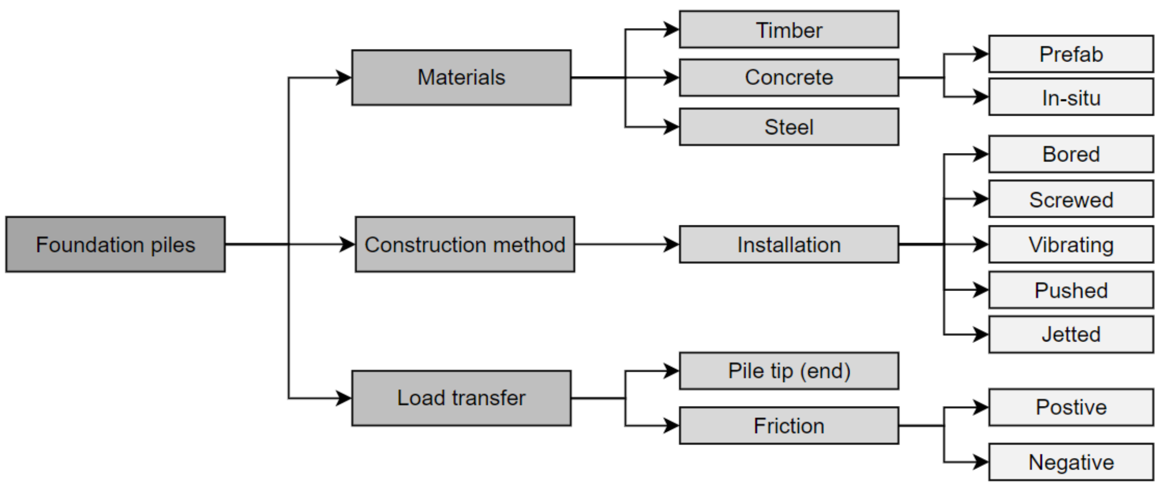


Figure 2.11: Global characteristics of pile foundations

2.3.4. Transport

The transport of the foundation piles is an important factor in the foundation design and the LCA methodology, which covers stages A2, A4 and C2. Prefab foundation piles can have large dimensions and a high self-weight, making transport a major challenge. Heavy-duty vehicles such as cranes and flatbed trucks are needed to transport foundations, which often use a significant amount of fuel to operate. Regarding logistics, a distinction can be made between prefab and cast-in-situ foundation transport. For the cast-in-situ transport, raw materials like cement and aggregates must transported. Prefab foundations are manufactured entirely off-site and often require more transport over a longer distance than cast-in-situ foundations. However, because of the limited surface area of the Netherlands and the presence of 175 concrete manufacturers, the distances between locations are limited [11].

A truck mixer or pump mixer is often used to transport the in-situ concrete foundation. If the location is difficult to access, the pump mixer is preferred over a truck mixer. The mixers have a mean load capacity of 12 m^3, where the concrete is mixed and transported. The truck's drum can rotate 360 degrees, preventing hardening and preserving the concrete's quality. A mean truck mixer with a load capacity of 12 m^3 has a net weight of roughly 14 tons. Therefore, the total combined weight of the truck is around 43 tons [9].

A flatbed truck is required to transport prefab foundation elements, which needs a length that is large enough for the foundation piles. The typical capacity of a flatbed truck lays around 32 tons and can carry a foundation pile length of 16 meters [32]. However, the maximum truck dimensions also depend on regulations. Figure 2.13 shows the transport of three piles of 27.5m. This concerns the maximum allowed divisible load of 50 tons [18]. An extended truck with supports for transporting the piles weighs approximately 25 tons, and therefore a maximum of 25 tons of piles can be transported. The maximum dimensions depend on specific surrounding regulations, like narrow passages or limited road loads. An exciting development is implementing electric, hydrogen or bio-diesel transport instead of fossil-based. Electric and hydrogen trucks are free from CO_2 emissions and particulate matter and produce less noise hindrance. Hyzon has developed a 50-ton capacity hydrogen truck with a remarkable action radius of 520 km on a single tank [36]. The hydrogen is converted into electricity within the vehicle using a fuel cell. Figure 2.12 shows the 50-ton capacity truck of Hyzon. It is interesting to investigate the environmental impact of the transport of the foundation and, thereby, the potential of implementing alternative fuel-based trucks and cranes.

Figure 2.12: Hyzon HyMax 250 hydrogen truck of BAM

Figure 2.13: Large transport prefab pile of 27.5m [18]

2.3.5. Installation

Besides transporting the foundation, the installation is also an important process. The different foundation pile systems can have different installation methods. The installation method is an important factor because it influences the soil behaviour, and therefore mechanical properties of the pile design. Due to the surrounding conditions, the installation method can have important requirements, as seen in subsection 2.3.1. There are two methods for the piling process, which are soil replacement and soil displacement piles. Displacement piles cause the soil to move radially and vertically as the pile shaft is driven or pushed into the ground. The soil underneath the pile is removed with soil replacement piles, resulting in a hole that can be filled with reinforcement and concrete or a precast concrete pile [63].

There are multiple ways to implement the soil displacement piles, which are globally pilling, vibrating, pushing, screwing and drilling. Every system has both positive and negative aspects, and the final installation choice depends on multiple factors, as seen in subsection 2.3.1. The soil replacement piles also have multiple variants for the installation for example a screw driller, a hydro milling or using coils or pulses. Other variants can be a screwed auger or one with jet grouting. There is a distinction between the soil displacement and replacement installation methods. The soil displacement can be executed with prefab elements like a timber pile with a concrete top or a prestressed concrete pile [63]. Sometimes, the piling or vibrating systems are not allowed due to restrictions like sound limitations or vulnerable surrounding objects. Therefore, a proper solution can be a drilled systems soil displacement system like, for example, a Gewi-pile. The benefit of such a system is that it is noise and vibrating-free. Besides that, the Gewi piles can both transfer tension and compression forces. However, the disadvantages are the complexity and costs in comparison with a regular prefab concrete piling system.

The other installation technique is the soil replacement design, which can globally be subdivided into two categories; drilled and screwed systems. The drilling can be executed using three methods, which are screw drilling, hydro milling or the use of pulses. Examples of each method are, respectively a drill pile, a tubular steel pile and a deep wall pile. The drilled systems are often used for heavy foundations or contaminated sites. The other system of soil replacement is the screwed system. The advantages of auger piles are that they are vibration and noise-free and, therefore suitable in areas with old (timber) buildings. However, a disadvantage is that heavy machinery is required for the screwing process. A possible solution to lower the installation's environmental impact is using an electric piling rig, as depicted in Figure 2.14 and Figure 2.15. Electric piling rigs consume less energy, produce less noise hindrance, and have power similar to regular diesel-generated piling machines [39]. However, the machine costs are high, and the battery capacity limits the maximum consecutive piling. A minimum amount of installation material is required for the cast-in-situ shallow foundation. A concrete pump is required, which divides the concrete from the truck mixer into the moulds. Besides that, a compaction needle is needed, which compacts the concrete. This compaction process helps eliminate entrapped air and voids, ultimately leading to high-quality concrete with the desired strength and durability characteristics.

Figure 2.14: Larger electric piling rig of BAM, Woltman 90DRe [7]

Figure 2.15: Smaller electric piling rig, Junttan PMx2e [39]

2.4. Conclusion

The literature study touched upon the MPG/ECI value determination process, the soil mechanics regarding the foundation, and the design aspects involved in a foundation design. The gained knowledge is important for the right MPG/ECI methodology and for designing a low environmental impact foundation design.

The MPG/ECI value is an important criterion for the sustainability and durability of a building. The value consists of a single score, expressed in euros per square meter of gross floor area and lifespan. The Dutch government requires an MPG value of 0.5 or lower for every newly constructed house in 2030. To determine the MPG value of the foundation, a complete LCA needs to be conducted, including all life cycle stages. To quantify the environmental impact of a building component, the ECI is used. This value implements the environmental shadow costs based on impact categories and is expressed with a monetised value. Several calculation tools and databases are available with corresponding defined ECI values to simplify the MPG calculation methodology.

One of the most important aspects of a foundation design is the soil property and its behaviour. To determine the soil properties underneath a house a CPT can be conducted. With a CPT the resistance of the soil is measured, and the different soil types and characteristics can be determined. Subsequently, the bearing capacity of the soil and foundation can be calculated with the CPT results. Globally, the eastern and southern parts of the Netherlands consist of stronger sand layers, and the western parts consist of compressible, weaker clay and peat layers. Therefore, a shallow foundation in the eastern part of the Netherlands is more feasible due to the shallow-located sand-bearing layer. A shallow foundation is placed directly on the soil and has a minimum foundation depth. Therefore, due to the possible material reduction, it may be an interesting low environmental impact foundation variant. Besides the different soil types, the soil properties are also important for the foundation capacity. For example, the grain size, consistency, hydraulic properties, swelling or excavation activities impact the soil stresses and physical behaviour and therefore, the bearing capacity of the soil layers.

Besides the soil properties and environmental impact, there are other requirements regarding foundation design. There are multiple requirements from construction, interaction, building site, surrounding area, execution and building physics. All these aspects must be considered in the design of the low environmental impact foundation pile. There are many different foundation pile configurations available for different prerequisites. Globally, a distinction can be made between soil replacement and soil displacement piles. Displacement piles cause the soil to move radially and vertically as the pile shaft is driven or pushed into the ground. The soil underneath the pile is removed with soil replacement piles, resulting in a hole that can be filled with reinforcement and concrete or a precast concrete pile. This has the main advantage that most piles are vibration and noise-free, which is sometimes an essential requirement for the foundation design. For both the transport and installation of the foundation, alternative power-generated trucks and piling rigs can be an exciting improvement to lower the environmental impact. A hydrogen truck or an electric piling rig can have considerable environmental potential.

3 Preconditions for foundation and LCA

This chapter describes the preconditions for the foundation design and the LCA methodology required to achieve a low environmental foundation design. The first section 3.1 will cover the structural design and loads of FLOW. The current MPG value is mentioned in section 3.2. Section 3.3 will cover the preconditions regarding soil properties and the two CPT profiles. The foundation design variants in this research are treated in section 3.4. Subsequently, section 3.5 covers the boundary conditions for the LCA method. Finally, in section 3.6, the conclusions of this chapter will be given.

Because of the increasing demand for sustainable and affordable housing, BAM developed the timber FLOW housing units. The timber construction elements are primarily manufactured off-site. With the combination of sustainability, industrialising and digitisation, houses can be built with a high amount of design freedom within a limited time. The mean project duration can be reduced to 3 instead of 12 months. Besides, the houses are demountable and reusable because of 'dry' connections between the timber elements. By using parametrisation and digitisation, different building plots can be used optimally. Moreover, the length, width, layout, and finishing can be adapted. During the construction process, the CO_2 emission of a regular concrete house will be 4.341 kg eq per house, while the FLOW house is estimated at 1.540 kg eq per house. Therefore, the CO_2 reduction is estimated at roughly 65% [6]. During the production phase of a regular concrete building, the CO_2 emission will be around 29.520 kg, while with the timber house, it is estimated at 20.102 kg CO_2. With this reduction, an emission factor of zero is assumed for the timber elements. If a negative emission factor of timber by CO_2 storage is assumed, even a lower emission can be achieved compared with the regular concrete houses [6]. Figures 3.1 and 3.2 show two FLOW variants with similar structural design and principles. This master thesis will focus on variant 1 as a reference project, depicted in Figure 3.1.

Figure 3.1: BAM timber FLOW house variant 1 [6]

Figure 3.2: BAM timber FLOW house variant 2 [6]

3.1. Structural design and loads of FLOW

For the design of the timber FLOW housing units and the design of the foundation, the following classification will be used:

- A design life span of 75 years
- A reliability class RC2
- A consequence class CC1.
- A confidence-index of (β) 3.8
- Environmental class concrete: XC2 (carbonation, alternately dry/wet)

For the calculation of the structure above ground level, the NEN-EN 1990-1993 will be used, and for the foundation design, the NEN 9997-1+C2. The primary structural materials of the building are HSB 120/140 mm panels for the walls and laminated C18 veneer lumber for the floor beams and posts. Concrete class C40/50 is used for the floors and beams, with reinforcement class B500B. All involved structural elements with corresponding weights are given in appendix B.1.

3.1.1. Dimensions

To calculate the total load on the foundation beams, floors and piles, the dimensions of the FLOW building need to be determined. The following dimensions are obtained from a FLOW building, which is preliminary but representative of future FLOW sizes. For this master thesis, the dimensions of the building will be constant. The buildings' dimensions can be seen in Figure 3.3. The total length of the building will be 9.1 meters. Both the first and second floors have a height of 3.1 meters. The top floor, measured from the middle to the top, is 4.1 meters high, which makes the total height of the FLOW housing, including the roof topping 10.7 meters. The angle of the roof will therefore be 42°. Each house has a foundation beam width of 5.1 meters and a length of 9.1 meters. Both the first and second floor has a staircase opening of 1.2 by 3.1 meters. The dimensions given in both Figure, 3.3 and with the use of Appendix B.1, the total vertical and horizontal loads of the FLOW timber building can be calculated.

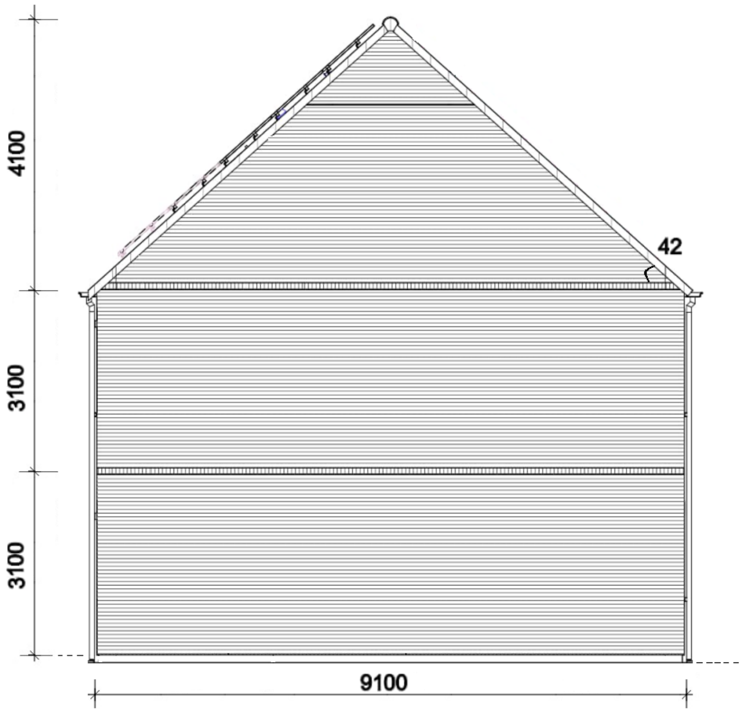


Figure 3.3: Global dimensions of the FLOW timber house

3.1.2. Structural design

The load-bearing superstructure of the FLOW housing unit mainly consists of engineered timber elements. The roof exists out of roof tiles and is supported by OSB plates and laminated veneer lumber beams. The load-bearing elements of the floors exist of laminated lumber beams and rafts, which support the multiplex and OSB plates. The load-bearing facade exists of HSB panels. To protect the panels from heavy weather influences, the panels are covered with an insulation layer, a fermacell Powerpanel and mineral stone strips. The Powerpanel is a water-resistant construction board. In this way, degradation mechanisms of the timber elements can be prevented, which elongates the lifespan of the housing unit. The ground floor has a 280 mm prefab ribbed floor and a finishing cement screed. The ribbed floor system has high insulation properties and fast construction time. The two staircase gaps for the stairs are strengthened with a small steel profile beam. These are the only steel load-bearing elements in the FLOW building. The prefab foundation beams consist of L beams, which carry the foundation floor and transfer all the loads towards the foundation piles. The stiff staircase provides the stability system of the FLOW house. The load-bearing structure elements of FLOW can be seen in Figure 3.4 and 3.5.

Figure 3.4: Structural elements FLOW house side view

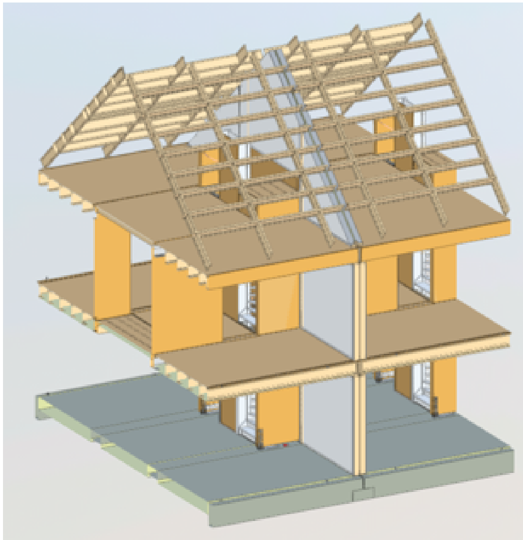


Figure 3.5: Structural elements FLOW house front view

3.1.3. Loads

With the given dimensions of the FLOW housing unit in subsection 3.1.1, the permanent and variable loads, and the combination factors in Appendix B.1, the normative loads on the foundation beams can be calculated. The governing loads on the foundation beams also include the self-weights of the beams. It is assumed that the FLOW house is detached with no connected housing units. Therefore, only the loads of the single housing units need to be carried by the foundation beams. The governing loads on the foundation beam with a total length of 9.1 meters can be seen in Figure 3.6 and Table 3.1. What can be seen is that the foundation beams at the front (3m) and back of the house (3m) have the largest distributed loads and that at the end of the stairs (3.1m), there are two point loads. The 2nd-floor facade loads result in a triangular distributed load over the entire length of the beam with a maximum value in the middle of 4.3 kN/m and zero at the edges. The concentrated load F1 results from the self-weight of the front facade structure on both sides of the longitudinal beam (Fig. 3.7). F2 and F3 are the results from the CLT walls, the second floor, the overflow and the bathroom unit, which are transferred, at ground floor level, by a concreted point load towards the foundation beam. The force F1 is the load of the front foundation beam placed on top of the longitudinal foundation beam at both ends.

Loads	Values
F1	24.2 kN
F2	42.2 kN
F3	30.9 kN
q1	54.5 kN/m
q2	33.7 kN/m
q3	4.3 kN/m

Table 3.1: Design loads on longitudinal foundation beam

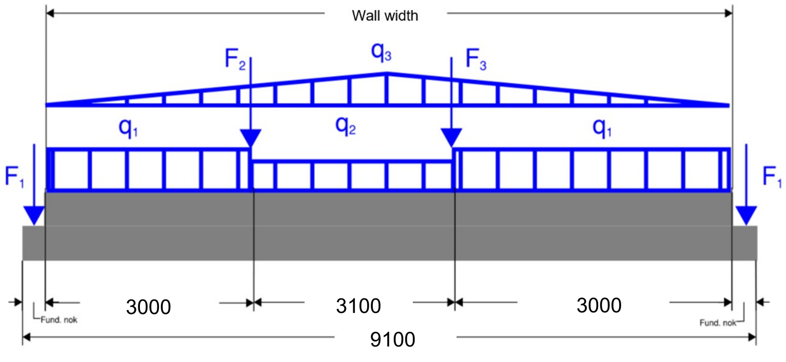


Figure 3.6: Forces on the longitudinal foundation beams of 9.1 meters

The governing loads on the front foundation beams, including self-weight, can be seen in Figure 3.7 and Table 3.2. The loads on the two front foundation beams of the building originate from the weights of the front facade of both the first and second floors. Again, the self-weight of the foundation beam is included. Therefore, it can be concluded that the total loads on the front foundation beams are significantly smaller than the loads on the longitudinal beams. This is because the front beam only carries the self-weight of the facade elements. This is done to give the front facade a high degree of design configuration freedom, resulting in easy adaption of doors and windows. The relatively low loads on the front foundation beam may result in a different foundation pile configuration because placing a foundation pile underneath such a low-weighting front foundation beam may be unsustainable. The front facade loads are transferred to the longitudinal beam by force F1 as depicted in Figure 3.6).

Loads	Values
q1	9.5 kN/m

Table 3.2: Design load on front foundation beam

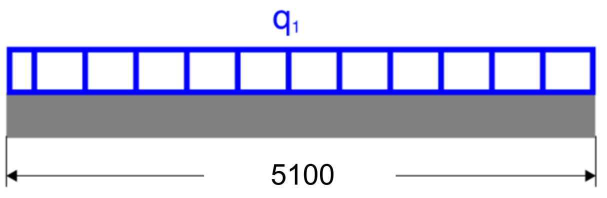


Figure 3.7: Design load on the facade beam with a length of 5.1 meters

The total load of the timber FLOW housing units can be calculated with Tables 3.1 and 3.2, and their corresponding lengths of respectively 9.1 and 5.1 meters. The total load of the FLOW building, including the foundation floor and beams, is 1145 kN. This is also the load that the foundation system must transfer because it also includes the variable loads. The total load of the timber FLOW building is significantly lower than that of a similar BAM concrete and steel house, which has a total load of roughly 3800 kN. This results in a load saving of 70%. This load saving may impact the foundation design concerning the environmental impact because less load-bearing foundation is needed.

3.2. Current MPG value FLOW

The structure's environmental impact in the Netherlands is often expressed with the ECI value as discussed in subsection 2.1. The MPG value also contains the lifespan and usable floor area of a house or office building. The focus of this research is on the environmental impact of the foundation design. However, besides the ECI values of the different foundations, it is desirable to investigate the MPG reduction because of the MPG requirement of 0.5 in 2030. Therefore, the current MPG value of the timber FLOW building must be found. Conducting a complete and detailed LCA of the entire FLOW building is an intensive job requiring much data. Therefore, as seen in subsection 2.1.2 there are calculation tools available that calculate the ECI value for various materials. BAM uses the GPR material calculation tool to determine the MPG value of the FLOW building. Considering the lifespan and GFA of 75 years and 161 m^2, an MPG value of €0.58 / $m^2 * y$ is calculated as shown in Figure 3.8. It must be noted that the MPG calculation is preliminary because of the probably (minor) changes in the design of the FLOW building, and therefore the MPG value. Besides, the GPR material tools do not contain detailed elements like CLT panels, the CLT panels are calculated with laminated European coniferous wood. This will result in deviations in the environmental impact of the FLOW house. Using Equation 2.1 and considering the lifespan and GFA, the ECI of the FLOW housing unit is estimated at €7063.

As already mentioned in subsection 2.1 and expected, the (electrical) installations have the largest contribution to the MPG value, as shown in Figure 3.8. The electrical installation is responsible for 47% of the total MPG/ECI value of the FLOW building. This is caused by the installations' limited lifespan of 25 years, and therefore three required replacements over 75 years. The electric central heating systems and the PV panels significantly contribute to the environmental impact. From Figure 3.8, it can also be concluded that the facade is responsible for 23% of the total environmental impact. This is again partly caused by replacing and repairing those elements over the entire lifespan. This is due to the weather influences and, thereby deterioration of the facade elements. However, the facade element masonry strips also have a relatively large environmental impact due to the high energy-demanding manufacturing process.

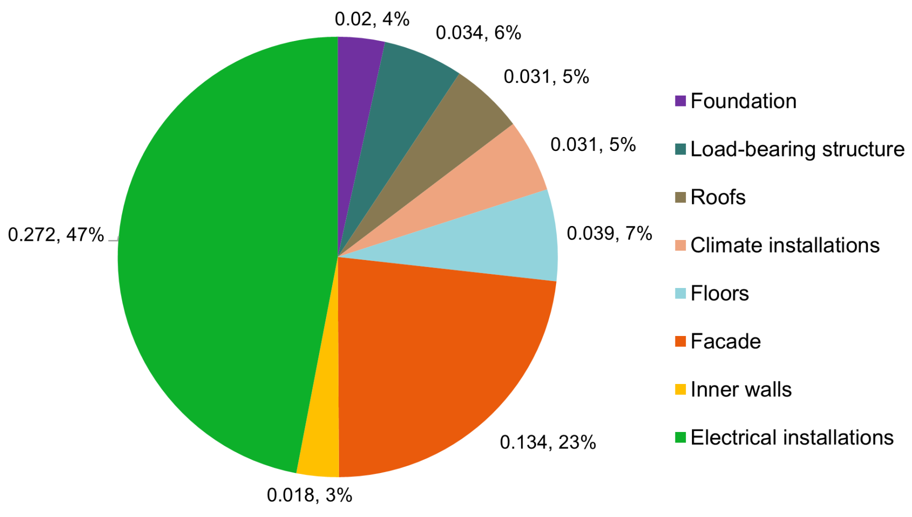


Figure 3.8: MPG value of the FLOW house per element, a total of 0.58

What also strikes is the low environmental contribution of the load-bearing structure and roof. This is due to the application of engineered timber, which has a relatively low environmental impact. In this MPG calculation, an emission factor of zero is assumed for the timber elements. Therefore, in the future, the contribution of the timber parts can be lowered if a negative emission factor can be considered due to CO_2 storage. However, a condition is that the timber elements should not be burned at the end of the service life since this would release the CO_2 emissions again. The high environmental contribution of repairing and replacing the installations and facade elements can also be seen in Figure

3.9. The use stage (B1-7) is accountable for 47% of the total MPG value, almost equal to the product stage. The product stage is responsible for 52% of the total MPG value, the largest environmental impact contribution. The construction stage is only accountable for 2% of the total MPG value. This is caused because of the absence of any heavy machines for both transport and construction in the GPR calculation tool. Both the end-of-life stage (C1-4) and the beyond-life stage (D) have a negative environmental impact on the MPG value. This is caused by for example the use of granulate, derived from concrete, in asphalt for new road projects. Figure 3.9 displays the MPG value of the regular concrete houses of BAM. The total MPG value of the concrete house is calculated at 0.74. Especially in the product stage (A1-A3), the MPG for the timber FLOW building is significantly lower than the regular concrete variant. The above calculated 0.58 MPG value for FLOW must be considered a rough estimation. For this master thesis, the benchmark MPG value of 0.58 will be employed to investigate the influence and impact of adopting a more sustainable foundation design.

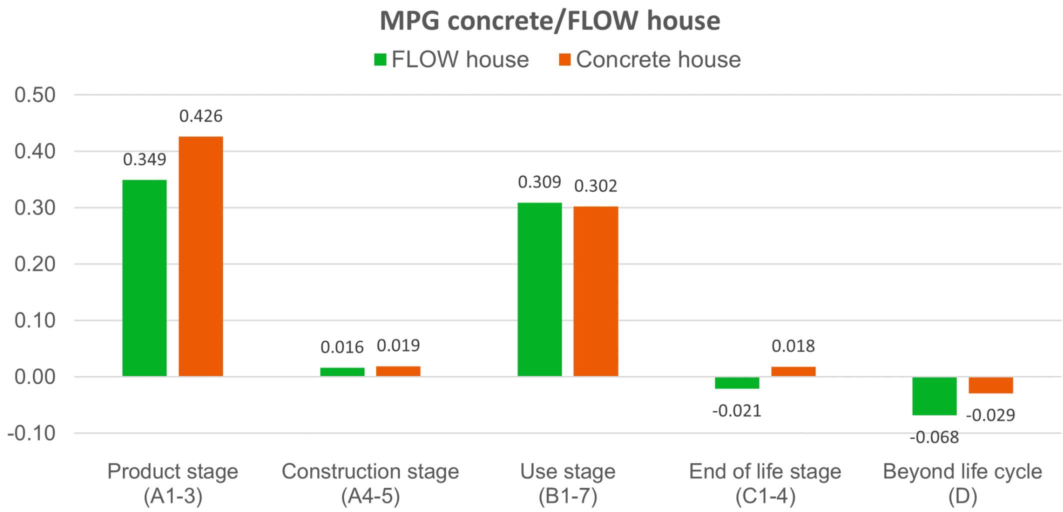


Figure 3.9: MPG value of the FLOW house per LCA stage

3.3. Considered soil conditions (CPT)

As discussed in section 2.2, the soil types can vary heavily in the Netherlands. Globally, it was noticed that the eastern and southern parts of the Netherlands consist of stronger sand layers, and the western parts consist of the more compressible, weaker clay and peat layers (Fig. 2.3). The soil conditions, and therefore CPT, greatly influence the foundation design and environmental impact. Thus, an almost unique suitable foundation needs to be designed for each location. However, to simulate a realistic soil condition, a typical CPT profile will be used for both a location in the western and eastern parts. In this way, a global comparison between foundation variants can be made for two typical soil properties in the Netherlands. At DINOloket from TNO, soil profile data can be found for multiple locations in the Netherlands, including CPT results and groundwater levels. A CPT around the city of Zwolle will be used for the eastern part of the Netherlands. A typical CPT around Delft will be used for the western part of the Netherlands.

3.3.1. CPT Zwolle and Delft

Around the city of Zwolle, there are many sand layers located. Figure 3.10 shows the results of a CPT near Zwolle. The ground level is located at +45cm NAP. What can be seen is that the cone resistance directly beneath the surface (0-5 m) is significant and lies around 5-10 MPa. Between the 5 and 10

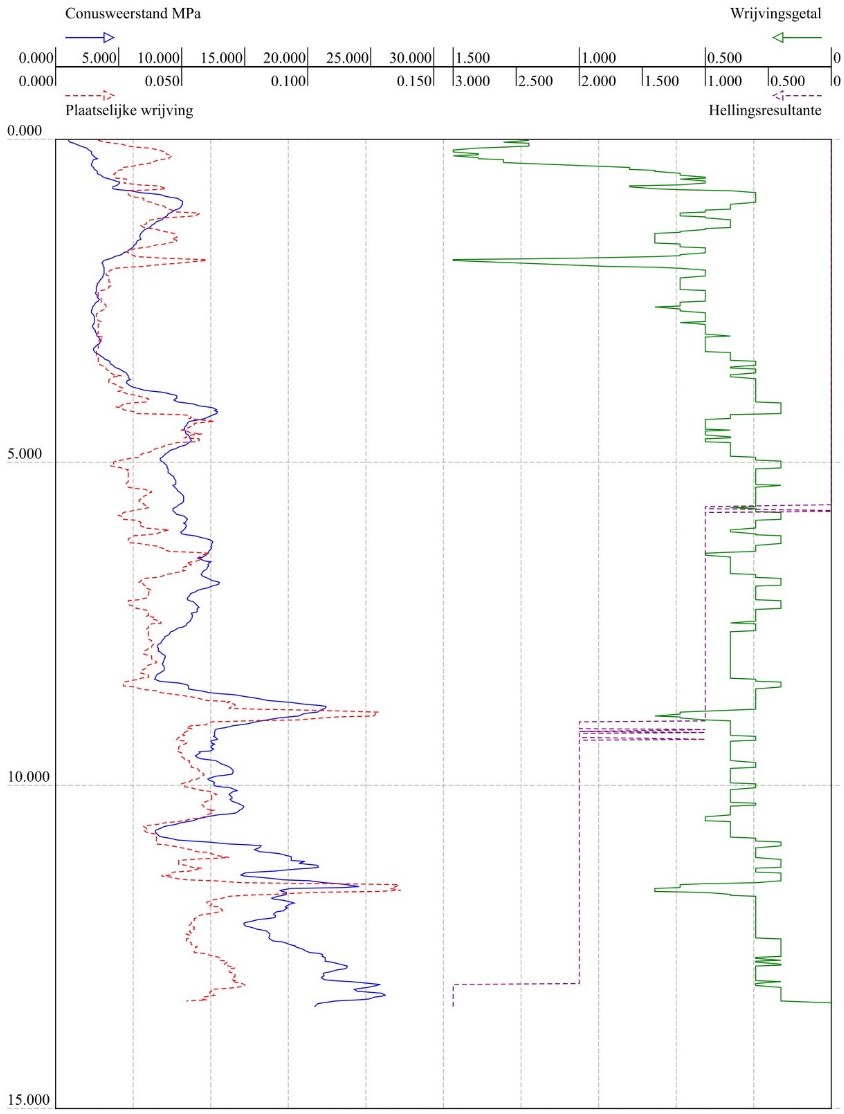


Figure 3.10: CPT of Zwolle, depth relative to ground level (RD: 208594.500, 505477.300) [1]

meters, the cone resistance varies between 10 and 15 MPa with an outlier at 8 meters depth with a resistance of 20 MPa. Below the 10-meter depth, the cone resistance varies roughly between 10 and 20 MPa. The CPT also shows the friction rate, which is the local sleeve friction divided by the cone resistance, as seen in Equation 2.2. The friction rate along the entire depth of the CPT does not exceed 1.5%. The maximum friction rate can be found directly beneath the surface and at a depth of 2 meters. The higher friction rates of 1.5% emphasise a siltier sand layer, and the friction rates of roughly 0.5% emphasise a more regular sand layer. Using Table 2.3 it can be concluded that the entire soil profile of the CPT is sand from a depth of 3 meters, with some deviations in the specific type of sand. This can also be confirmed by using the more detailed lithology of Zwolle as shown in Figure 3.15 and 3.16.

Besides the stronger eastern sand layers in the Netherlands, the most densely populated areas are located in the softer (clay) western part. A typical CPT of Delft will be used to investigate the sustainable foundation design in those parts of the Netherlands. The following CPT in Figure 3.11 is located at the TU Delft campus between the faculty of Civil Engineering and X Sports Delft. The ground level is determined at -55cm NAP. What can be concluded from the CPT in Delft is that the

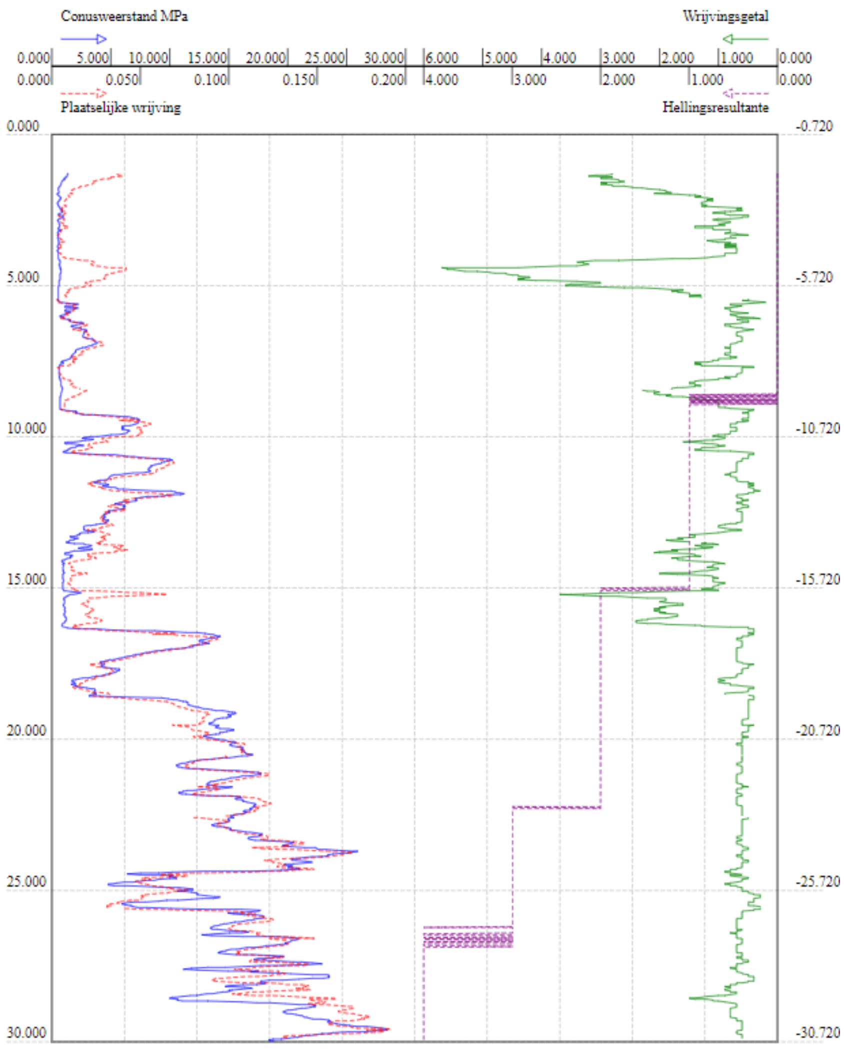


Figure 3.11: CPT of Delft campus, depth relative to ground level (RD: 85579.200, 445815.200) [1]

cone resistance, the first 9 meters, does not exceed 5 MPa. At a depth of 10 to 12 meters, the cone resistance is measured at roughly 5 to 10 MPa. From the 12 to 16 meters, there is a low resistance of 2-3 MPa. From a depth of 17 meters, an increasing cone resistance is measured, starting from 15 MPa at 17 meters depth to roughly 30 MPa at 24 meters depth. The friction rate around ground level is about 3% and varies to 1.0% at 3 meters depth. At 5 meters depth, there is a large friction rate outlier of 6.0%. From a depth of 5 meters to 30 meters, the friction rate fluctuates around 1-2%, with an outlier of 4% at 15 meters. Using the above values and using table 2.3 and the lithology in appendix A.2, it can be concluded that the soil profile, the first 10 meters mostly contains clay layers. Between a depth of 10 to 13 meters, there is a small loam layer located. The loam layer has a high friction rate, which corresponds with figure 3.11 and table 2.3. The stiff/moderate category clay layers are located between 13 and 17 meters deep. As seen in Figure 3.11, sand is situated after a depth of 19 meters because of the typical cone resistance of >10 MPa and a friction rate of 0.5-1.0%. Therefore, it can be concluded the bearing capacity sand layers are located after 19 meters depth. The detailed lithology graph of Delft, with specification, can be seen in figures 3.16 and 3.15.

3.3.2. Ground Water Table Zwolle and Delft

Besides the above CPT graphs, with the corresponding cone resistance and friction rates, more soil conditions need to be considered. As discussed in section 2.2, the GWT can largely influence the foundation design. It can influence the bearing capacity of the soil, affect the possible degradation mechanism of a timber foundation pile, and cause possible settlements of the shallow foundations. The GWT fluctuates per day and year and is influenced by multiple factors. However, a proper approximation can be given by looking at the maximum and minimum GWT values over a couple of years as seen in Appendix A.2.

For the CPT position around Zwolle, the GWT is relatively constant for ten years, with two outliers in 1999 and 2000. The groundwater level varies roughly from -60 cm to +40 cm relative to NAP [1]. However, for the foundation design, it is important to know the GWT relative to the ground level. The position of the CPT is located at +15 cm NAP. The GWT varies between -105 cm and -5 cm relative to ground level. Consequently, it can be assumed that the GWT will not be lower than -105cm concerning the ground level. The mean GWT relative to ground level is -31 cm. The GWT figure of Zwolle over the years can be found in Appendix A.2

For the CPT position around Delft, the year 2018-2019 will be used as shown in Appendix A.2. Using the data from DINOloket, the mean GWT relative to NAP in the year 2018/19 is -238 cm, with a maximum level of -164 cm around 1 May and a minimum level of -256.3 cm around the middle of August [1]. However, again the GWT is relative to NAP and not relative to the surface. The ground level concerning NAP can be determined by using the AHN viewer of the exact location as the GWT measurement. The ground level is determined at -55 cm relative to NAP [48]. Therefore, the GWT, relative to ground level, varies between -201 cm and -109 cm and has a mean level of 183 cm, as shown in Figure 3.12.

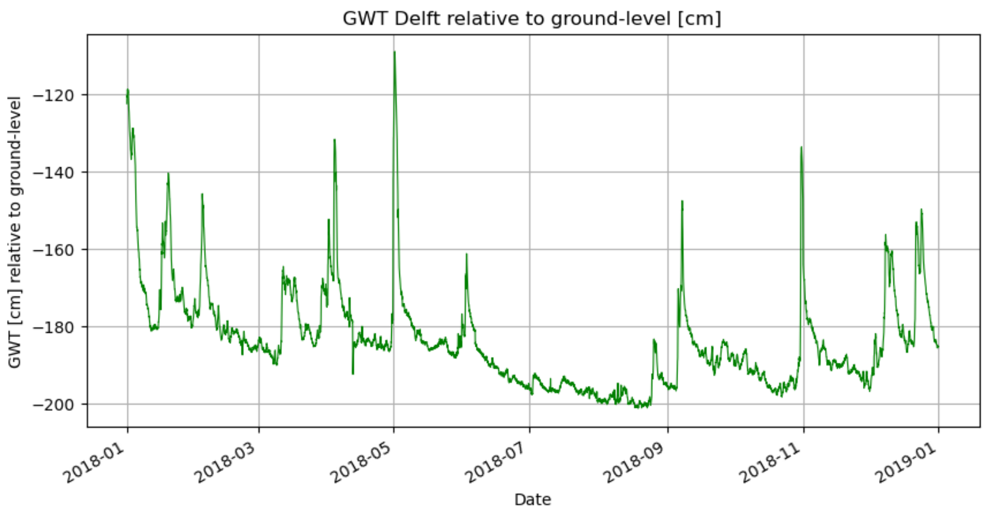


Figure 3.12: GWT of CPT Delft 2018/19, relative to ground-level (RD: 85516.900, 445880.700)

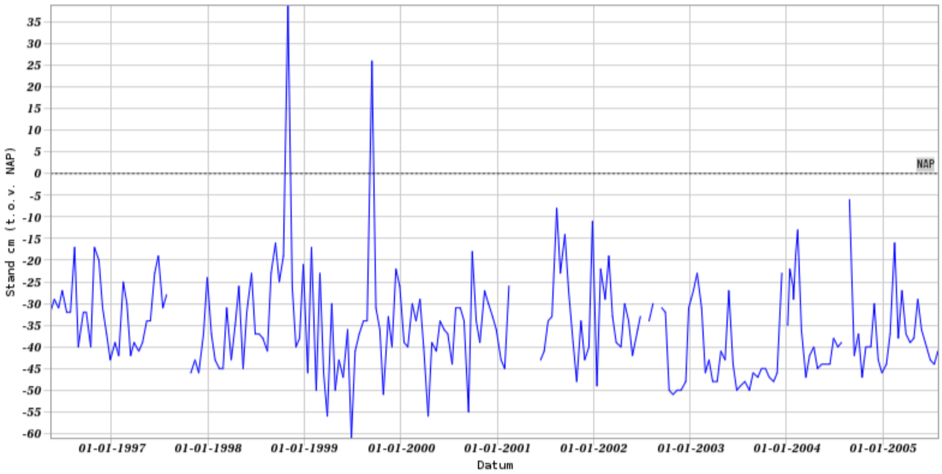


Figure 3.13: GWT of CPT Zwolle 2018/19, relative to NAP (RD 208594.500, 505477.300) [1]

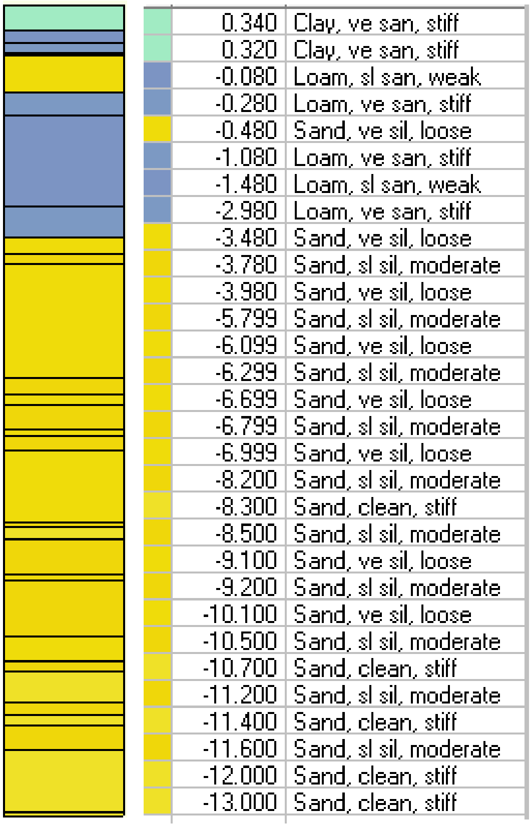
3.3.3. Litholgy of Zwolle and Delft

Another important condition is the lithology profile of both soil locations, which describes the soil characteristics. To classify the different soil materials in both CPT profiles, the following standard materials according to the NEN 9997-1:2016 will be used as shown in Figure 3.14 below. The saturated values, unsaturated values and friction angles of the different soil types are listed according to the standard and Table A.1.

Soil name	Soil type [-]	Gamma-unsat [kN/m³]	Gamma-sat [kN/m³]	Friction angle (phi) [degree]
Clay, clean, moderate	Clay	19.00	19.00	17.50
Clay, clean, stiff	Clay	20.00	20.00	25.00
Clay, clean, weak	Clay	17.00	17.00	17.50
Clay, organ, moderate	Clay	16.00	16.00	15.00
Clay, organ, weak	Clay	15.00	15.00	15.00
Clay, sl san, moderate	Clay	20.00	20.00	22.50
Clay, sl san, stiff	Clay	21.00	21.00	27.50
Clay, sl san, weak	Clay	18.00	18.00	22.50
Clay, ve san, stiff	Clay	20.00	20.00	32.50
Gravel, sl sil, loose	Gravel	18.00	20.00	35.00
Gravel, sl sil, moderate	Gravel	19.00	21.00	37.50
Gravel, sl sil, stiff	Gravel	20.00	22.00	40.00
Gravel, ve sil, loose	Gravel	19.00	21.00	32.50
Gravel, ve sil, moderate	Gravel	20.00	22.00	35.00
Gravel, ve sil, stiff	Gravel	21.00	22.50	40.00
Loam, sl san, moderate	Loam	21.00	21.00	32.50
Loam, sl san, stiff	Loam	22.00	22.00	35.00
Loam, sl san, weak	Loam	20.00	20.00	30.00
Loam, ve san, stiff	Loam	20.00	20.00	35.00
Peat, mod pl, moderate	Peat	13.00	13.00	15.00
Peat, not pl, weak	Peat	12.00	12.00	15.00
Sand, clean, loose	Sand	18.00	20.00	32.50
Sand, clean, moderate	Sand	19.00	21.00	35.00
Sand, clean, stiff	Sand	20.00	22.00	40.00
Sand, sl sil, moderate	Sand	19.00	21.00	32.50
Sand, ve sil, loose	Sand	19.00	21.00	30.00

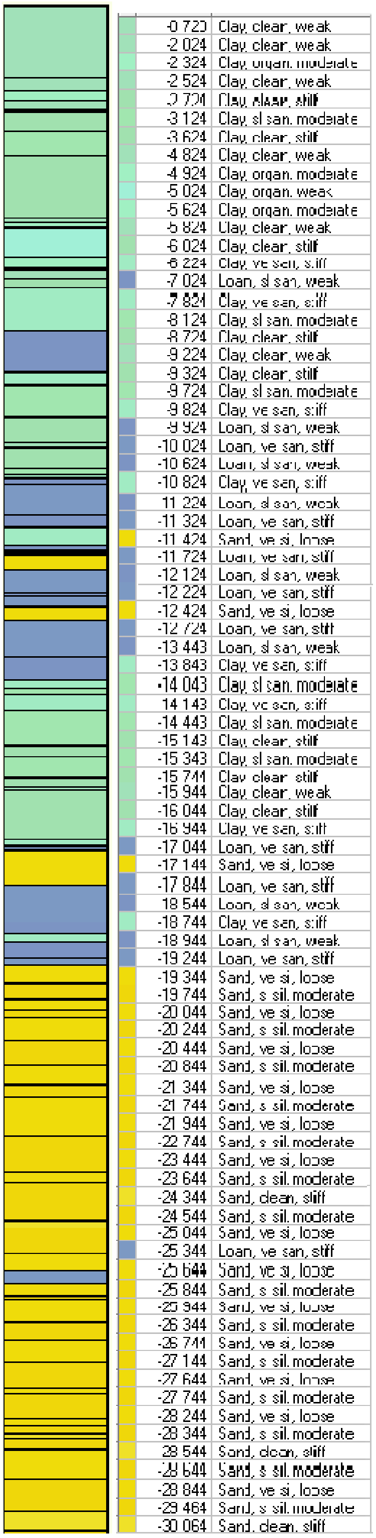
Figure 3.14: Considered different soil types in calculation

In Figure 3.15 and 3.16, the lithology of Zwolle and Delft are displayed (relative to ground level). What can be seen is that the mechanically cored borehole of Zwolle is conducted to a depth of 13 meters and in Delft to a depth of 30 meters. In Zwolle, the entire soil consists of moderate/loose sand layers from a depth of 3 meters. The first 3 meters exist of mainly loam. It can be concluded that the entire soil profile of Zwolle to a depth of 13 meters exists of various types of (stronger) medium category sand layers from a depth of 3 meters. Delft, in contrast, has an entirely different borehole log profile. To a depth of 17 meters, the soil layers primarily exist out of clay and loam. A loam layer is located between a depth of 10 to 14 meters. However, as seen in the CPT Figure 3.11, after 20 meters, there will be sand layers because of the typical cone resistance of >10 MPa and friction rate of 0.5-1.0%. It can be concluded that the sand type in Zwolle has a much larger bearing capacity than the softer clay layers in Delft.



0.340	Clay, ve san, stiff
0.320	Clay, ve san, stiff
-0.080	Loam, sl san, weak
-0.280	Loam, ve san, stiff
-0.480	Sand, ve sil, loose
-1.080	Loam, ve san, stiff
-1.480	Loam, sl san, weak
-2.980	Loam, ve san, stiff
-3.480	Sand, ve sil, loose
-3.780	Sand, sl sil, moderate
-3.980	Sand, ve sil, loose
-5.799	Sand, sl sil, moderate
-6.099	Sand, ve sil, loose
-6.299	Sand, sl sil, moderate
-6.699	Sand, ve sil, loose
-6.799	Sand, sl sil, moderate
-6.999	Sand, ve sil, loose
-8.200	Sand, sl sil, moderate
-8.300	Sand, clean, stiff
-8.500	Sand, sl sil, moderate
-9.100	Sand, ve sil, loose
-9.200	Sand, sl sil, moderate
-10.100	Sand, ve sil, loose
-10.500	Sand, sl sil, moderate
-10.700	Sand, clean, stiff
-11.200	Sand, sl sil, moderate
-11.400	Sand, clean, stiff
-11.600	Sand, sl sil, moderate
-12.000	Sand, clean, stiff
-13.000	Sand, clean, stiff

Figure 3.15: Lithology of Zwolle [1]



-0 720	Clay, clear, weak
-2 024	Clay, clear, weak
-2 324	Clay, organ, moderate
-2 524	Clay, clear, weak
-2 724	Clay, clear, stiff
-3 124	Clay, sl san, moderate
-3 624	Clay, clear, stiff
-4 824	Clay, clear, weak
-4 924	Clay, organ, moderate
-5 024	Clay, organ, weak
-5 624	Clay, organ, moderate
-5 824	Clay, clear, weak
-6 024	Clay, clear, stiff
-6 224	Clay, ve san, stiff
-7 024	Loan, sl san, weak
-7 824	Clay, ve san, stiff
-8 124	Clay, sl san, moderate
-8 724	Clay, clear, stiff
-9 224	Clay, clear, weak
-9 324	Clay, clear, stiff
-9 724	Clay, sl san, moderate
-9 824	Clay, ve san, stiff
-9 924	Loan, sl san, weak
-10 024	Loan, ve san, stiff
-10 624	Loan, sl san, weak
-10 824	Clay, ve san, stiff
11 224	Loan, sl san, weak
-11 324	Loan, ve san, stiff
-11 424	Sand, ve si, loose
-11 724	Loan, ve san, stiff
-12 124	Loan, sl san, weak
-12 224	Loan, ve san, stiff
-12 424	Sand, ve si, loose
-12 724	Loan, ve san, stiff
-13 443	Loan, sl san, weak
-13 843	Clay, ve san, stiff
-14 043	Clay, sl san, moderate
14 143	Clay, ve san, stiff
-14 443	Clay, sl san, moderate
-15 143	Clay, clear, stiff
-15 343	Clay, sl san, moderate
-15 744	Clay, clear, stiff
-15 944	Clay, clear, weak
-16 044	Clay, clear, stiff
-16 944	Clay, ve san, stiff
-17 044	Loan, ve san, stiff
-17 144	Sand, ve si, loose
-17 844	Loan, ve san, stiff
18 544	Loan, sl san, weak
-18 744	Clay, ve san, stiff
-18 944	Loan, sl san, weak
-19 244	Loan, ve san, stiff
-19 344	Sand, ve si, loose
-19 744	Sand, s sil, moderate
-20 044	Sand, ve si, loose
-20 244	Sand, s sil, moderate
-20 444	Sand, ve si, loose
-20 844	Sand, s sil, moderate
-21 344	Sand, ve si, loose
-21 744	Sand, s sil, moderate
-21 944	Sand, ve si, loose
-22 744	Sand, s sil, moderate
-23 444	Sand, ve si, loose
-23 644	Sand, s sil, moderate
-24 344	Sand, clean, stiff
-24 544	Sand, s sil, moderate
-25 044	Sand, ve si, loose
-25 344	Loan, ve san, stiff
-25 644	Sand, ve si, loose
-25 844	Sand, s sil, moderate
-25 944	Sand, ve si, loose
-26 344	Sand, s sil, moderate
-26 744	Sand, ve si, loose
-27 144	Sand, s sil, moderate
-27 644	Sand, ve si, loose
-27 744	Sand, s sil, moderate
-28 244	Sand, ve si, loose
-28 344	Sand, s sil, moderate
28 544	Sand, clean, stiff
-28 644	Sand, s sil, moderate
-28 844	Sand, ve si, loose
-29 464	Sand, s sil, moderate
-30 064	Sand, clean, stiff

Figure 3.16: Lithology of Delft [1]

3.4. Considered foundation design variants

As discussed in subsection 2.3.1 there are many different foundation designs, existing out different materials and installations. Not all different foundation possibilities can be analysed in depth for this research. Therefore, the considered foundation designs that will be analysed will be limited. In collaboration with BAM, the performed literature study in chapter 2 and by looking at the sustainability potential, the following foundation designs for FLOW in this research will be included:

1. **Prefab prestressed pile foundation**
2. **Shallow strip foundation**
3. **Timber pile foundation**

For both the CPT in Delft and Zwolle, a suitable design variant will be created with corresponding calculations. However, it may be possible that a design variant will not be feasible for one CPT location due to requirements or unallowed settlements. In that case, the specific design variant will not be considered for the CPT location.

3.4.1. Prefab prestressed pile foundation

The first foundation design to be analysed is the currently FLOW housing unit concrete prestressed pile foundation, as depicted in Figure 3.17. BAM has chosen prefab prestressed concrete foundation piles because of the high standardisation factor. This is because of the high bearing capacity and, therefore high flexibility regarding locations in the Netherlands. Besides that, prefab prestressed foundation piles are often applied and have less complexity, making the piles more cost-effective. Therefore, it is interesting to analyse and optimise the currently used foundation design on environmental impact and compare it to the other foundation designs.

Figure 3.17: Regular prefab prestressed house pile plan

3.4.2. Shallow strip foundation

The second foundation design variant is the shallow strip foundation system, described in subsection 2.3.2 and depicted in Figure 3.18. A shallow strip is chosen because of the distributed loads on the foundation beams. Moreover, the shallow strip foundation is the most applied shallow foundation type compared to shallow raft and pad foundations. Therefore, for this research, the focus will be on the shallow concrete strip foundation. A shallow foundation may be a proper solution because of the 70% FLOW weight saving compared to the regular concrete housing unit. Especially for the stronger sand layer location near Zwolle, the shallow strip foundation may result in material savings in comparison with pile foundations. A shallow foundation may not have enough bearing capacity for the CPT location in Delft because of the weaker clay layers and high settlements. The application of shallow foundations is expected to result in (concrete) material savings, compared to pile foundations. This is because shallow foundations do not require foundation beams to transfer the loads towards the piles. The shallow foundation itself can be seen as one load-bearing capacity foundation. Additional attention is needed against the possible settlements of the shallow foundation, especially in combination with a lowering GWT. Besides that, the transport and installation of shallow foundations may also result in environmental reduction because of the absence of piling and the less needed capacity during transport. Finally, the shallow strip foundation has a high recycling potential as it can be easily removed without disturbing the soil conditions heavily. Therefore, this research's second foundation type variant will be a shallow concrete strip foundation.

Figure 3.18: Shallow strip foundation variant [28]

3.4.3. Timber pile foundation

This research's third and final foundation variant is the timber foundation pile variant with a concrete cap, as depicted in Figures 3.19 and 3.20. It is chosen for a timber foundation pile variant because the entire building above ground level is already made from engineered timber and because of timber's low expected environmental impact compared with concrete, especially if it can be assumed that CO_2 will be stored in the timber parts during and after service life. However, as mentioned in subsection 2.3.1, the degradation mechanism of timber needs to be adequately researched. Especially in combination with a low GWT, the degradation of the timber piles can result in settlements and decreasing bearing capacity. A proper solution to this problem is the implementation of a concrete cap as depicted in figure 3.19. This ensures that the timber part remains below the GWT and possible degradation mechanisms are prevented. Timber piles have a limited bearing and structural capacity, and therefore additional piles may be installed compared to concrete piles.

Figure 3.19: Concrete cap for a timber pile

Figure 3.20: Timber pile foundation variant [46]

3.5. Boundary conditions LCA method

As concluded in subsection 2.1.1, a clear and unambiguous methodology for conducting an LCA is crucial. Especially for comparing the different foundation variants, it is important to have the same environmental impact approach per variant. The LCA for each variant will be conducted using Figure 3.21. As discussed in section 2.1.1, an LCA must include all life cycle stages (A-D) according to the NEN-EN 15804 + A2:2019. However, performing a complete LCA according to the standard has some difficulties regarding a foundation design.

As mentioned in chapter 2, the reuse of foundation piles is highly unusual. In almost all cases, foundation piles remain in the soil after reaching the end of the building's service life. Removing foundation piles poses several serious difficulties. Firstly, the piles may break during the extraction process. Additionally, the procedure is quite expensive and demands the use of heavy machinery. Moreover, the resulting gap often needs to be filled with bentonite; otherwise, undesired changes in the groundwater table may occur. Hardly ever the foundation piles are re-used for a new construction on top. As a result, the environmental impact quantification of LCA stages C and D is very complex and inaccurate. Besides that, the service life of a foundation is, in many situations, longer than the required 75 years. In the product stage, transporting the raw materials to the manufacturer (A2) will be out of the scope of this research. This is because determining the transport of all raw materials of the concrete mixture will be a labour-intensive task and really manufacturer-dependent and highly variable. This results in a high inaccuracy of data. Therefore, for this research, the variants will be compared for the raw material supply and manufacturing stage (A1 & A3), and the construction process stage (A4 & A5). For the use stage (B) of the LCA method, it is assumed that the foundation is designed correctly and does not require any maintenance or reparations. This results in zero environmental impact for the entire LCA use stage. For the end-of-life stage (C) and the benefits beyond the life cycle (D), it is assumed that all the pile foundation remains in the soil after the use stage, and therefore has zero environmental impact. For the shallow foundation, it is assumed that the concrete will be deconstructed and re-used as granulation. Unlike the foundation piles, the shallow foundation can easily be removed without large soil consequences. Therefore, LCA phases C1 to C4 and D will be included for the shallow foundation as shown in Figure 3.21. The environmental benefits from LCA stages C and D are minor compared to the impact in phases A to B as concluded from Figure 3.9. For the environmental impact calculation of the product stage (A1 & A3) and the construction stage (A4-5), category 1 and 2 data must be used as much as possible. This type of data is reviewed by an independent, qualified third party and is proprietary data (cat.1) or non-proprietary data (cat.2). In this way, a more specific and accurate environmental analysis can be created. However, sometimes, the more inaccurate category 3 data must be used. However, for category 3 data, a surcharge of 30% must be considered, according to the NMD [27].

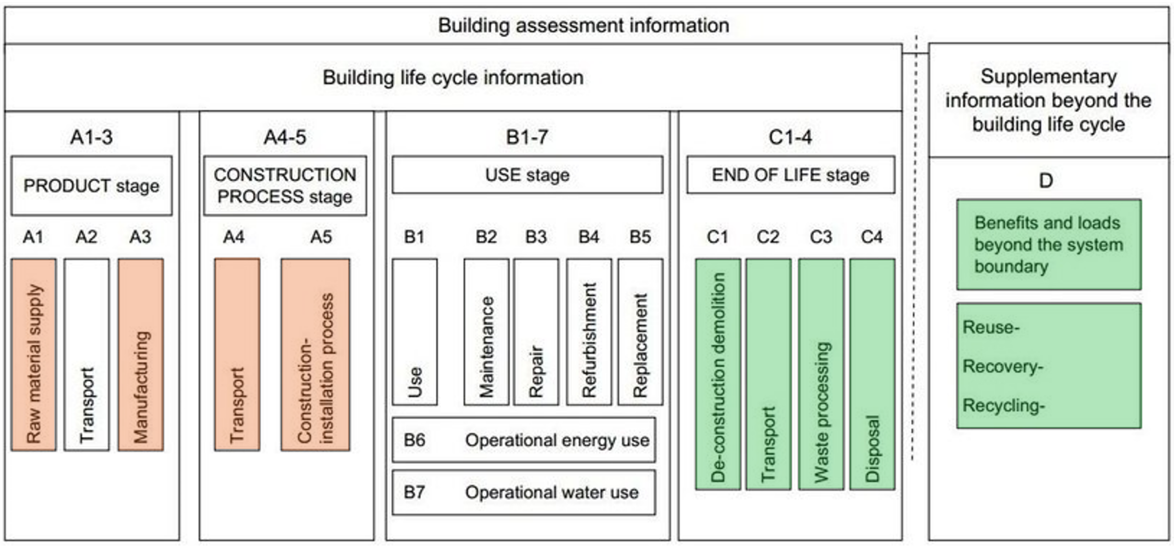


Figure 3.21: LCA stages that are analysed: for pile foundations (only orange) and shallow foundations (orange + green) [64]

3.6. Conclusion

Chapter 3 touched upon the preconditions of the foundation design and the LCA methodology. Setting clear and elaborate boundary conditions about the design conditions and LCA is of great importance because it considerably influences the research outcome.

The first important boundary condition is the load analysis of the FLOW building itself because it determines the necessary bearing capacity of the foundation design. The first step was the determination of the global dimension of the regular FLOW building. Moreover, all the materials and the structural design are analysed. By considering the variable and permanent loads and the corresponding load combination factors, the governing loads on the foundation beams have been calculated (Fig. 3.6 and 3.7). It was concluded that the two longitudinal foundation beams have to transfer the largest loads of FLOW and that the two front foundation beams only have to transfer the self-weight of the front facade. The total weight of the FLOW building is calculated at 1145 kN and therefore has a reduction of roughly 70% compared with the regular concrete housing units of BAM. It can be concluded that the foundation of the timber FLOW housing requires significantly less bearing capacity. The MPG value of FLOW, excluding foundation piles, is estimated at 0.58. The primary contributor is the electrical installations, responsible for approximately 47%. This is caused by the required replacements because of their limited service life. It also needs to be mentioned that the MPG value is estimated with the GPR calculation tool and does not contain detailed information. However, the 0.58 MPG value will be used as a benchmark for the environmental impact reduction of the various foundation variants.

The final critical boundary condition regarding the foundation design is the soil condition underneath the FLOW building. The Netherlands has a large variety of soil profiles and situations. For each location, an almost unique suitable foundation needs to be designed. However, to simulate a realistic soil condition approach, an average CPT will be used for both a location in the western and eastern parts of the Netherlands. For the stronger eastern region, a CPT of Zwolle is used, as shown in Figure 3.10. The soil profile of Zwolle exists entirely of sand after 3.5 meters depth, with some deviations in the type of sand. The GWT, relative to ground level, varies between -105 cm and -0.05 cm. Therefore, it can be assumed that the GWT will not be lower than -105cm. For the (weaker) western part, a CPT profile of Delft Campus is used, as shown in Figure 3.11. The first 17 meters mostly consist of soft clay. The stronger bearing sand layers are located after a depth of 17.5 meters. The GWT, relative to ground level, varies between -183 cm and -109 cm, as shown in Figure 3.12. By using chapter 2 and in collaboration with BAM the following three design variants will be analysed and optimised for the FLOW building on their environmental impact:

1. Prefab prestressed concrete piles (often applied)
2. Shallow strip foundation (material saving and high re-use potential)
3. Timber piles (low environmental impact due to CO_2 storage)

The regular prestressed prefab piles are being investigated because they represent the most frequently employed foundation system for BAMs housing units. Another design variant is the shallow strip foundation, which may save material and require less transport and installation. Moreover, it has a high reuse potential. Finally, a timber foundation pile will be considered for analysis because of its low environmental impact potential due to CO_2 storage and alignment with the super-structure being made from (engineered) timber.

In this research, the pile variants will be compared for the product stage (A1-A3) and the construction process stage (A4-A5) using the LCA method. For the use stage (B) of the LCA, it is assumed that the foundation is designed correctly and does not require any maintenance. LCA stages C and D will only be included in the shallow foundation variant since the foundation pile variants are assumed to remain in the soil, resulting in zero environmental value.

4 Prefab prestressed concrete pile

This chapter covers the prefab prestressed concrete pile variant and its environmental impact. Section 4.1 will cover the foundation plan and assumptions. The bearing- and structural capacity calculations will be covered in section 4.2. The resulting design and dimensions of the foundation follow in section 4.3. Section 4.4 discusses the environmental impact (ECI) of the designed foundation piles for both Zwolle and Delft. Moreover, section 4.5 covers the pile ECI optimisation. Finally, the conclusion of this chapter will be given in section 4.6.

4.1. Assumptions prefab prestressed pile

The first design variant to be investigated is the prefab concrete pile foundation with prestressed reinforcement. This particular foundation design is one of the most commonly applied methods in the Netherlands. In almost all cases, BAM uses prefab prestressed piles for their timber housing units. Various design parameters are critical in influencing the environmental impact when considering the foundation design plan. These parameters include the concrete class, pile quantities, positions, and reinforcement. For the prefab prestressed concrete design of the FLOW building, the foundation plan will be utilised as discussed in section 4.2.1. This plan comprises standardised dimensions and characteristics, incorporating six prefab concrete piles along the longitudinal foundation beams. Regarding the prefab prestressed foundation piles, specific requirements regarding moment capacity must be considered. The occurring moment is primarily caused by the transport and piling of the piles. While the exact deliverable pile lengths may vary slightly depending on the manufacturer, they generally fall within the same order of magnitude. Consequently, we will consider the maximum deliverable prestressed prefab foundation pile lengths as presented in Table 4.1.

Transverse dimensions [mm]	Max deliverable length [m]
220 x 220	19.0 - 21.0
250 x 250	22.0 - 23.0
290 x 290	25.0 - 28.0
320 x 320	27.0 - 29.0
350 x 350	28.0 - 31.0
380 x 380	29.0 - 34.0
400 x 400	30.0 - 36.0
420 x 420	30.0 - 36.0
450 x 450	32.0 - 37.0

Table 4.1: Maximum deliverable pile length prefab prestressed concrete piles [29]

Positioning the foundation piles at the same longitudinal length would lead to them being close to the foundation piles of a possible adjacent house, potentially negatively impacting the adjacent pile's bearing capacity. To mitigate this influence, the positions of the piles along the longitudinal beam are intentionally varied for the two foundation beams. This approach enables the placement of a potential adjacent house next to the FLOW house, ensuring that the foundation piles of each structure do not interfere with one another.

For both Zwolle and Delft, the foundation piles consist of a squared prefab concrete base with standardised dimensions (Table. 4.1) and manufactured with concrete strength class C45/55, reinforcement steel B500B and prestressed tensile strength Y1860 as depicted in Figure 4.1. The environmental class will be XC2 because the foundation will be placed in a wet environment, which is rarely dry, as shown in Appendix A.2. In this research, the CPT results of both Zwolle and Delft will be used for the entire foundation pile plan. However, especially with adjacent FLOW houses, multiple CPTs need to be conducted because the soil conditions can change locally and influence the bearing capacity. The considered soil type classifications can be seen in Figure 3.14 and are specified according to the NEN-9997-1 standard. The construction sequence for all foundation variants is assumed to be as follows: Initially, a CPT will be conducted, followed by potential excavations, and ultimately, the installation of foundation piles will occur. This often applied sequence is chosen due to the significant influence of the sequence on the soil behaviour and, consequently, the bearing capacity of the foundation piles. Regarding the phreatic level, the mean GWT from both Zwolle and Delft will be considered (relative to ground level). Accordingly, for Delft, the phreatic level, relative to ground level, will be considered at -183 cm, while for Zwolle, it will be considered at -31 cm, as indicated in Appendix A.2.

As elaborated in Appendix A.1.2, the consolidation of the soil plays an important role in reducing the cone resistance. According to Equation A.26, the cone resistance needs to be reduced with an OCR greater than 1.0. However, for all foundation design variants, it will be assumed that the soil is normally consolidated, which means that no previous stress is applied to the soil area (i.e., old structures). Another important aspect of the bearing capacity of the pile is the positive and negative skin friction, as mentioned in subsection 2.3.1. To enforce the negative skin friction in the D-foundation software, the expected ground settlement of 0.11 meters will be considered. Consolidating the weak layers above the sand layer resulted in additional load on the foundation pile. This way, the negative influence due to negative skin friction can be calculated. Besides that, it is important to determine the bottom of the negative skin friction zone and the top of the positive friction zone. In Zwolle, the first 3.5 meters mainly consist of loose and weak loam. From the depth of 3.5 meters, there will especially be moderate sand layers located, as shown in Figure 3.15. Therefore, both the top of the positive skin friction and the bottom of the negative skin friction are assumed to be located at -3.5 meters. Since the first 3.5 meters comprise a 0.5-meter thick sand layer, negative skin friction is considered a conservative and sufficiently realistic approach. For the CPT in Delft, the ratio of negative skin friction is more significant. This is because the first 19.3 meters of the soil mainly consists of very unconsolidated moderate clay and weak loam. Therefore, the dividing line between the negative and positive skin friction in Delft will be located at 19.3 meters depth.

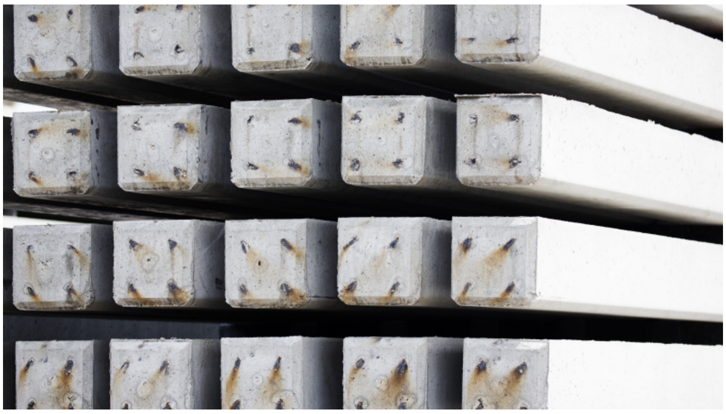


Figure 4.1: Regular prefab prestressed concrete piles [72]

4.2. Capacity calculations

The first step in calculating the environmental impact of the concrete prefabricated prestressed pile foundation variant is calculating the pile foundation dimension and configuration. To calculate the design dimensions of the concrete prefab prestressed piles, the reaction forces on the foundation piles need to be considered. Therefore, a detailed foundation pile plan is required. Subsequently, the design of a foundation pile consists of two checks: the bearing capacity of the pile (shaft + tip) in the soil and the structural capacity of the pile itself. The first step is the bearing capacity of the pile in the CPT profile of both Zwolle and Delft. This will influence the pile length and width. Subsequently, with the calculated length of each pile, the structural analysis of the pile itself can be conducted. This will influence the reinforcement quantity of the pile, and thereby the configuration and environmental impact.

4.2.1. Foundation pile plan

For the foundation design plan, many design parameters will influence the environmental impact, like the strength class, pile quantities, positions, and reinforcement. For the position of the prestressed concrete pile, the foundation pile plan of FLOW will be used, which contains standardised dimensions and characteristics. In this way, the FLOW units' high standardisation and workability can be achieved. The foundation plan consists of 6 piles, which are all located along the longitudinal foundation beams. This is because the front foundation beams have a much smaller load to transfer, as discussed in section 3.1. Therefore, positioning a foundation pile underneath the front facade beam will result in over-dimensioning and unnecessary environmental impact. The foundation pile plan of the prestressed variants is depicted in Figure 4.2.

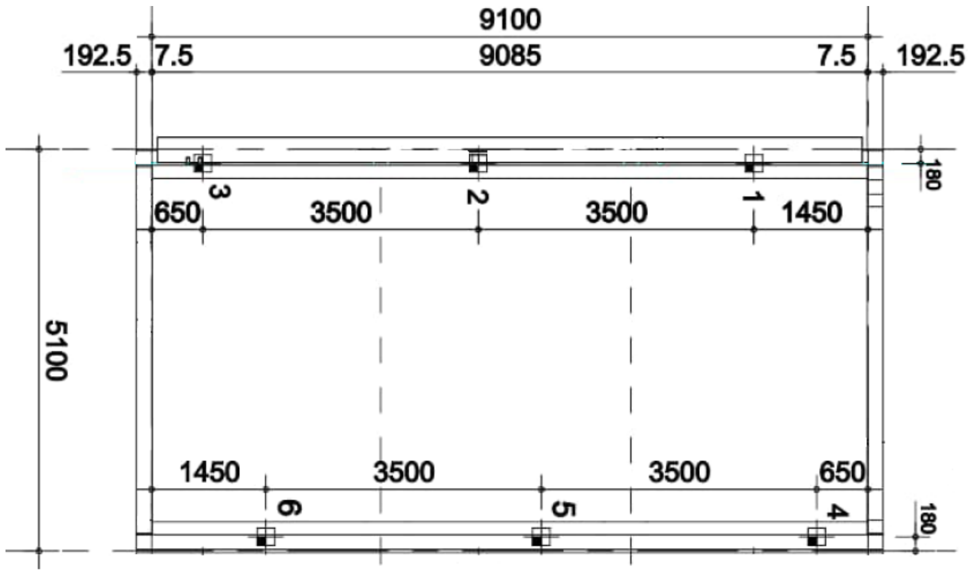


Figure 4.2: Foundation plan of six prestressed concrete foundation piles

The position of the foundation piles is not symmetric because of the adjacent FLOW houses. Placing the foundation piles in the same position will be placed next to each other, negatively influencing the adjacent pile's bearing capacity. Therefore, the positions of the piles along the longitudinal beam change per beam. Foundation piles 1 and 6 are positioned 1.45 meters from the front beam, and piles 3 and 4 are placed 0.65 meters from the other front beam as shown in Figure 4.2. Foundation piles 2 and 5 are placed 3.5 meters next to the other piles, which is done to minimise the support moments resulting in less reinforcement. The optimisation of the position of the foundation piles will be out of scope. This is because the pile position optimisation will influence the design of the foundation beams and floors and therefore decrease the standardisation of FLOW. It should be noted that the aforementioned foundation pile plan is tailored for the concrete pile foundation variant. Consequently, adjustments might be necessary if the foundation plan is intended for other variants, such as the timber pile foundation.

The first step in calculating the bearing capacity of the pile variants is determining the foundation pile reaction forces. In this way, the pile configurations can be determined to investigate the environmental impact of the pile foundation variants subsequently. By considering the total loads of FLOW as discussed in section 3.1 and using Technosoft software packages, the reaction force can be determined. The assumptions about stiffness and calculations can be seen in Appendix C.1. This is because the pile stiffness of the variants determines the force distribution among the six pile positions. Appendix C.1 shows the shear force distribution along the foundation beams. It is assumed that the supporting moments, given in Appendix C.2 are taken by the concrete foundation beams and soil and are not transferred to the piles. Table 4.2 shows all governing forces on the six prefab concrete foundation piles. What can be concluded is that the outside foundation piles 1 and 6 have the largest vertical loads and the inner piles 3 and 4 have the smallest vertical loads. It is assumed that the total horizontal wind load acting on the FLOW house will be distributed among the stiff concrete floor and beams and corresponding adjacent soil. Therefore, the horizontal load will not be considered in the structural capacity of the foundation piles. Besides that, it is assumed that the soil profile is equal left and right of the foundation pile, and therefore no resulting horizontal force is considered. The eccentricity of the piles is caused by a small imperfection of the pile itself and installation deviations. By multiplying the total pile eccentricity with the vertical normal force, the moments due to eccentricity in both ULS and SLS can be calculated. For all the foundation piles, the standard eccentricity of 50 mm will be used. In this way, the moments due to imperfections will be considered.

Pile number	Vertical ULS [kN]	Vertical SLS [kN]	Eccentricity pile [mm]	Moment ULS [kNm]	Moment SLS [kNm]
Pile 1 (S1)	219	162	50	10.95	8.10
Pile 2 (S2)	195	144	50	9.75	7.20
Pile 3 (S3)	159	118	50	7.95	5.90
Pile 4 (S4)	155	115	50	7.75	5.75
Pile 5 (S5)	193	143	50	9.65	7.15
Pile 6 (S6)	224	166	50	11.20	8.30

Table 4.2: Support reactions prefab prestressed pile foundations

4.2.2. Bearing capacity of the piles

The first calculation check is the bearing capacity of the six piles in combination with both CPT profiles. For the bearing capacity of the piles, the software program D-foundation of Deltares will be used. For both the locations in Zwolle and Delft, the lithography is obtained from DINOloket as shown in Appendix A.2. Both CPT results can be imported into the D-foundation software. The foundation plan given in Figure 4.2 will be used for both Delft and Zwolle. Because the pile dimensions are constant and prefabricated, and the assumed installation method is driven, the following pile classification factors must be considered. For the point resistance α_p a value of 0.7 must be considered because the pile is driven (post-2017 standard NEN-9997). The shaft pressure α_s and tension α_t pile classification factors are respectively 0.010 and 0.007. For the determination of the settlements of the piles, load-settlement curve 1 of the NEN-9997 needs to be considered. No additional slip layer will be applied to the foundation piles, and therefore the representative adhesion will be zero. For the pile tip shape factor β, a value of 1.0 must be considered because of the prefab rectangular shape. The pile tip cross-section factor s is not applicable because it only counts for open-ended steel pipes. Besides that, it is assumed that there will not be any additional surcharge before or during the foundation process.

For the bearing capacity calculation, both the vertical loads in SLS and ULS, given in Table 4.2 need to be connected to each pile in D-foundation. The FLOW building is considered non-rigid because it can not fully transfer moments in its connections. This influences the correlation factor ξ_4 for the minimum value of the calculated pile resistance. The other overrule factors remain default according to the EC7-NL standard. Because some piles are positioned within 5 meters of each other, the pile group effects must be considered, influencing the negative skin friction. However, because of the lack of multiple CPTs, it will be assumed that the two CPTs are representative and consistent for both foundation plans. However, because there is only one available CPT, the ξ_3 and ξ_4 factor in the bearing capacity calculation of the pile will be 1.39 (NEN-9997 Table A.10). An excavation level of 0.34 will be considered, which is the top of the CPT level and therefore no reduced q_c needs to be considered. For the location of Delft, no reduced q_c will be considered.

For the maximum allowed settlement in Limit State and Serviceability State, the default values of respectively 0.15 m and 0.05 m are used (NEN 9997). For the maximum allowed relative rotation, the default settings will also be used for ULS and SLS, respectively 0.01 and 0.003. The following bearing capacity checks must be conducted: Verification of Limit State EQU, verification of Limit State STR/GEO, and verification of Serviceability Limit State, which can also be seen in Appendix C.2

4.2.3. Structural capacity of the piles

Besides the check for the bearing capacity of the piles in the soil, the structural capacity of the piles also needs to be performed. The structural capacity deals with the forces and moments on and in the pile design. Besides that, it determines the amount of reinforcement, which has a relatively high environmental impact, as mentioned in chapter 2. Therefore, it is important to minimise the amount of reinforcement to reduce the environmental impact of the prefab foundation piles. In Table 4.2, the forces on the pile can be seen. The supporting moments originating from the loads of the foundation beams will be transferred by the spiral reinforcement of the foundation beam itself. The focus will be on the foundation pile design and its reinforcement calculations. The structural analysis of the foundation beams will be out of scope. However, because the foundation pile can not be implemented exactly and has some slight deviations, a standard eccentricity of 50 mm needs to be considered. This eccentricity, combined with the normal force on the pile, will result in a moment as shown in Table 4.2.

Of the applied prefab foundation piles in the Netherlands, 95% are prestressed [12]. By applying prestressed reinforcement, permanent pressure in the concrete is created. This has the main advantage that the concrete can take the tension forces developed during transporting and lifting. When a tensile force acts on the foundation pile, a vertical normal force or a bending moment can lead to a decrease in compressive stress in the concrete due to the prestressing. When the tensile force provides greater tensile stress than the compressive stress of the prestress, there will be tension in the concrete, and eventually, cracks will occur, after which the concrete will fail. The prestress strands normally exist of 3 or 7 wires per strand. The individual wires are twisted around each other, generating a high tension capacity. For both Zwolle and Delft, prestressed steel Y1860 MPa will often be used for prestressing strands. The strands will be prestressed with a total stress of 3 or 5 N/mm^2 before the concrete is poured into the mould. After prestressing the strands and corresponding elongation, the strands want to shrink again. However, this is prevented by the surrounding concrete, and as a result, the prestressing strands remain under tension, and the concrete comes under pressure.

To determine the piles' structural capacity and the reinforcement amount, a couple of structural checks need to be performed according to NEN-EN 1992-1-1 + C2;2016. The first checks are the transport, lifting, and piling process of a prestressed concrete foundation pile, which are often the decisive factors in the structural capacity of a prefab foundation pile. This is because, during the transport and lifting, the foundation pile is tilted like a beam on two supports, which generates moments under its self-weight, which are not present if the foundation pile is placed in the soil. It is assumed that the transport of the piles is conducted with two swings according to Figure 4.3.

Figure 4.3: Scheme for transport prefab piles on two swings

The distance D between the outer and inner support of the swing can vary between each transport. It is assumed that for longer foundation piles, this distance is 1.5m and for shorter piles 0.5m. The length of D influences the occurring moment during the transport. The static moment, resulting from the self-weight of the concrete beam, can be calculated using the support reactions A = B times (0.207*L-1.10) for long foundation piles and times (0.207*L-0.6) for short foundation piles. Besides that, a dynamic moment must be considered because of the dynamic response of prefab concrete foundation piles during the transport and lifting process [24]. TNO researched the dynamic reaction of foundation piles during transport, and it was concluded that besides a fixed part dynamic stretch, the dynamic stretch also increased linearly with the static stretch. This resulted in equation 4.1 for the dynamic moment during transport:

$$M_{dyn} = 1.45 \cdot M_{stat} + 1.5 \cdot W \cdot 10^{-6} \tag{4.1}$$

Besides the structural capacity during transport, the lifting process must also be checked. The foundation piles will be lifted with two hoists, resulting in the following situation as shown in Figure 4.4. The static moment, resulting from the self-weight of the concrete beam, can be calculated using the distance of 0.207L times the support reactions A = B. For the dynamic moment during lifting, an impact coefficient S of 1.25 must be considered (TNO - report 93-CON-RO488).

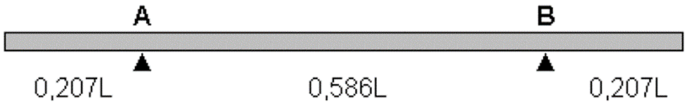


Figure 4.4: Scheme for lifting prefab piles on two hoists

The dynamic moment of both the transport and the lifting process needs to be lower than the cracking moment capacity of the foundation pile in SLS. This is because developing cracks in the concrete during transport or lifting are not allowed. The cracking moment capacity can be calculated according to article 8.7.1 of the VBC and can be seen in equation 4.2.

$$M_{cr} = W \cdot (1.3 \cdot f_{ct,eff} - \sigma_{cm}) \tag{4.2}$$

$$U.C = M_{dyn}/M_{cr} \leq 1 \tag{4.3}$$

The final check that needs to be conducted is the ultimate bearing moment capacity of the prestressed foundation pile due to both the normal force and moment, as shown in equation 4.4. This is because the normal force on the foundation pile results in a higher moment capacity due to the ”prestress” of the cross-section. The reactive moment originates from the total vertical force times the eccentricity of the pile as calculated in Table 4.2. The calculation regarding the structural capacity of the prestressed prefab piles can be found in Appendix C.3.

$$U.C = Ned \cdot e/M_{rd} \leq 1 \tag{4.4}$$

Besides the prestressed reinforcement, there also needs to be additional regular head and base spiral reinforcement because of the additional stresses during the driving process. The amount of spiral reinforcement depends on the prefab manufacturer and sometimes per project. The calculations regarding the amount of spiral reinforcement are out of scope due to time limitations. However, according to BRL 2352 concrete foundation piles standard, the pile head must have spiral or bracket reinforcement over a minimum length of 500 mm. Besides that, it needs at least nine windings, and in many cases, the spiral reinforcement is double-equipped. The base reinforcement has fewer restrictions because of the less concentrated forces. Therefore, the pile base needs at least five windings of spiral or bracket reinforcement over 200 mm. The pile part between the head and base can be executed without spiral or bracket reinforcement because no transverse force reinforcement is required [5].

4.3. Design and dimensions

Using the above results in section 4.2 for both the bearing and structural capacity of the piles, and by considering the loads and soil conditions, the pile designs in both Zwolle and Delft can be determined. The soil conditions are given in Figure 3.15 and 3.16, the foundation plan in Figure 4.2, the loads on the piles in Table 4.2 and the shear and moment distribution in Appendix C.1.

4.3.1. Location Zwolle

The prefab concrete pile lengths of Zwolle can be seen in Table 4.3 and are derived from the bearing capacity of the foundation piles in Zwolle and the verification checks in Table 4.4. The checks that must be performed are mentioned in Appendix C.2. The first step is the verification in Limit State EQU, where the results can be seen in Table 4.3. For the EQU verification, the $R_{c;d}$ needs to be considered, which does not have to include the negative skin friction, according to the NEN 9997. For the SLS and STR/GEO checks, the negative skin friction must be included, and therefore, the value $R_{c;net;d}$ needs to be considered. In all cases, except for pile 3, the STR/GEO verification was normative for the pile length. The lengths for piles 1/6, 2/5, and 2/3 are respectively 5.00, 4.70, and 4.20 meters long. However, a uniform pile length is selected for all six foundation piles due to considerations of workability and to minimise the likelihood of installation errors. This results in a small over-dimension of pile lengths 2, 3, 4, and 5. However, different pile lengths for one location would have been very unusual for a housing project. The negative skin friction of 22 kN for all piles is equal because of the transition towards positive skin friction at -3.50 meters.

Pile number	Length [m]	$F_{c;d}$ [kN]	$R_{c;cal;max}$ [kN]	$R_{c;d}$ [kN]	$F_{nsf;d}$ [kN]	$R_{c;net;d}$ [kN]
Pile 1 (220x220)	-5.00	219	420	252	22	230
Pile 2 (220x220)	-5.00	195	420	252	22	230
Pile 3 (220x220)	-5.00	159	420	252	22	230
Pile 4 (220x220)	-5.00	155	420	252	22	230
Pile 5 (220x220)	-5.00	193	420	252	22	230
Pile 6 (220x220)	-5.00	224	420	252	22	230

Table 4.3: Bearing capacity prefab prestressed piles foundation Zwolle

Besides the EQU verification, the STR/GEO and SLS verification must also be conducted. These verifications included the negative skin friction and were normative for five pile lengths. The settlement was often 99 meters for those five piles because of the weak compressible loam layers at a depth of -3.78 meters and, thereby, too much negative skin friction. This resulted in the failure of the pile-bearing capacity and therefore unlimited settlement. The maximum values for the settlement and rotation can be seen in Table 4.4. The maximum rotations between the neighbouring pile elements can be seen in Table 4.4, where the brackets implement the normative rotation between the two pile numbers.

Pile number	Length [m]	S_d STR/GEO [m]	ϕ_d STR/GEO	S_d SLS [m]	ϕ_d SLS
Max value:	-20.00	0.150	0.01	0.05	0.003
Pile 1 (220x220)	-5.00	0.021	0.0025 (1-4)	0.005	0.0004 (1-4)
Pile 2 (220x220)	-5.00	0.023	0.0037 (2-3)	0.005	0.0005 (2-3)
Pile 3 (220x220)	-5.00	0.022	0.002 (2-3)	0.006	0.0005 (2-3)
Pile 4 (220x220)	-5.00	0.022	0.0018 (1-4)	0.005	0.0005 (1-4)
Pile 5 (220x220)	-5.00	0.022	0.0032 (5-6)	0.005	0.0005 (5-6)
Pile 6 (220x220)	-5.00	0.023	0.0032 (5-6)	0.005	0.0005 (5-6)

Table 4.4: Verification checks bearing capacity Zwolle

With the derived pile length and corresponding cross-section dimensions, as seen in Table 4.1, the structural capacity and the reinforcement design can be calculated. Because all six piles have the same length, it is desired to have the same amount of reinforcement because of the simplicity during manufacturing. From the structural capacity calculation in Appendix C.3 the following pile design is developed. The width and height of the pile remain 220x220 mm, which originates from the bearing capacity of the pile in Zwolle. As mentioned in section 4.1, the concrete class will be C45/55, the reinforcement steel B500B, and the prestressed steel will be Y1860. The longitudinal reinforcement will consist of 4 prestressed strands, with each 3-wire. The total diameter of the strand is 7.5 mm, which makes the surface of each strand 29 mm^2, and the total surface of prestressed reinforcement 116 mm^2. The 7.5 mm diameter strand is the only available diameter in the Netherlands for a 3-wire strand, as listed in NEN 3868:2001. The concrete cover is 30 mm, which satisfies the requirements regarding the minimal cover. Therefore, the distance from the edge to the middle reinforcement will be 33.75 mm on each side. The head and base reinforcement of the pile can also be seen in Figure 4.5. The spiral reinforcement has nine double windings with a diameter of 6 mm over a total length of 550 mm, as prescribed in BRL 2352. The 6 mm single base reinforcement is equipped with four windings over a length of 200 mm. What also can be seen in Figure 4.5 are the cut edges on the downside of the cross-section. These edges are cut because straight corners have a low stiffness and would break down during the casting process.

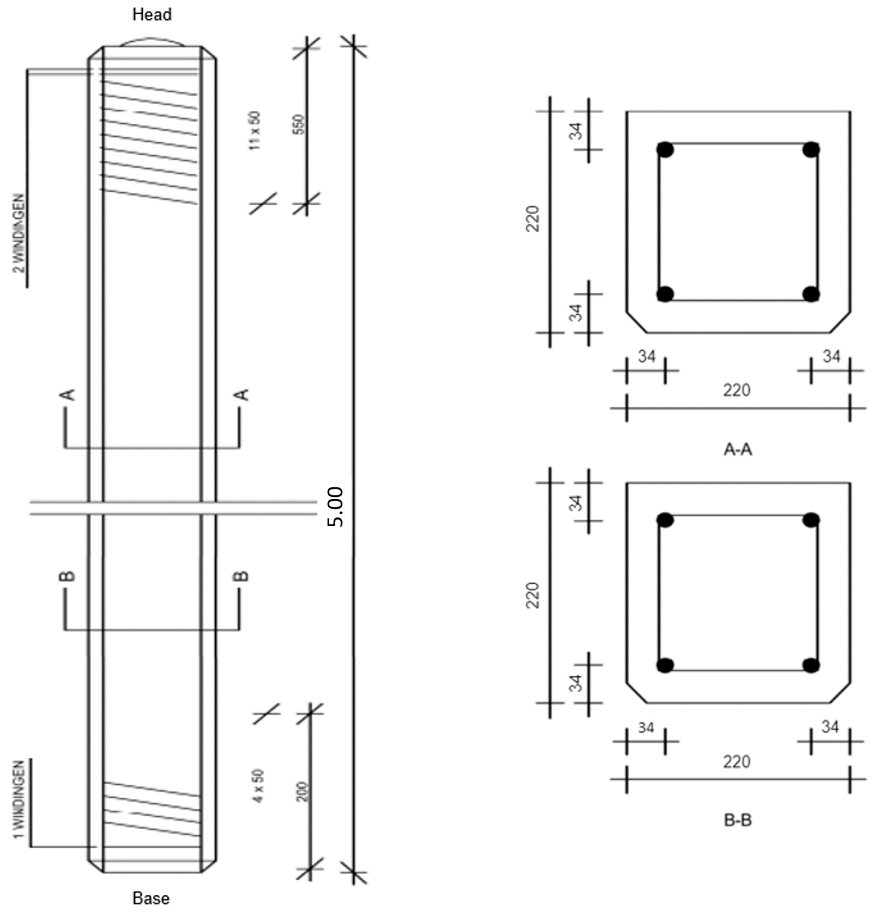


Figure 4.5: Design and dimensions prefab prestressed pile foundation Zwolle, with length 5.0 meters

The strands will be prestressed with 3.0 N/mm^2, which makes the total prestress force 145 kN and 36.3 kN per strand. However, the prestress losses must be considered as elaborated in Appendix C.3. These losses are the initial prestress loss, creep, shrinkage, and relaxation. The total stress reduction due to creep, shrinkage, and relaxation is calculated at 82.28 N/mm^2. Therefore, the total working stress of the prestressed steel σ_{pw} is 1220 N/mm^2. The calculated centric pile load capacity of the prestressed pile, therefore results in 1352 kN. In Table 4.5, the most important structural capacity values of the foundation piles in Zwolle can be seen. The values per pile change because of the small different normal forces acting on the piles as seen in Table 4.3. From Table 4.5, it can be concluded that the minimum reinforcement of 3-wires, 4∅7.5 mm fulfils all required capacity checks. Because of the limited length of the piles (max. 5m), the dynamic moments during transport and lifting are limited. The maximum moment before the six concrete, prestressed foundation piles start to crack is 14 kNm. What also can be seen is that, as already mentioned, the moment capacity of the foundation pile increases with an increasing normal force due to the additional ”prestressing” in the concrete. The combination of normal force plus the moment due to the eccentricity and the moment resistance results in a unity check of around 0.6 for all six piles, as concluded in Appendix calculation C.3. This is also the critical unity check, because of the limited pile length and thereby limited dynamic moment due to transport or lifting. It can be concluded the foundation design in Figure 4.5 satisfies all design requirements. The calculation and the weights of all required elements can be found in Appendix C.3.

Pile number	Reinfor-cement	$M_{ed;dyn;T}$ [kNm]	$M_{ed;dyn;L}$ [kNm]	M_{cr} [kNm]	M_u [kNm]	$N_{rd,max}$ [kN]	N_{ed} [kN]	M_{ed} [kNm]	M_{rd} M+N
1 (220x220)	4 ∅ 7.5	2.85	0.94	14.0	19.1	1352	370	18.5	29.9
2 (220x220)	4 ∅ 7.5	2.85	0.94	14.0	19.1	1352	346	17.3	28.7
3 (220x220)	4 ∅ 7.5	2.85	0.94	14.0	19.1	1352	310	15.5	26.6
4 (220x220)	4 ∅ 7.5	2.85	0.94	14.0	19.1	1352	306	15.3	26.3
5 (220x220)	4 ∅ 7.5	2.85	0.94	14.0	19.1	1352	344	17.2	28.6
6 (220x220)	4 ∅ 7.5	2.85	0.94	14.0	19.1	1352	375	18.8	30.2

Table 4.5: Structural capacity of prestressed foundation piles Zwolle

4.3.2. Location Delft

For the determination of the pile length in Delft, the constraints given in subsection 4.2.2, the CPT results in Figure 3.16, and the loads given in section 4.2 will be used. The prefab concrete pile lengths can be seen in Table 4.6. However, the transverse dimensions are increased to 290 x 290 mm because of the maximum pile length requirement as shown in Table 4.1. What can be seen is that every pile has the same pile length of -23.00 meters. This pile length is significantly larger than the 5-meter pile in Zwolle. This is caused due to the 19.3 meters-thick compressible weak clay and loam layers in Delft. At a depth of 23 meters in Delft, an ultimate bearing capacity of 1261 kN and a design bearing capacity of 756 kN is generated. For the Service Limit state, which includes the negative skin friction of 518 kN at a depth of 23 meters, a total bearing capacity of 238 kN is generated. The pile bearing capacity for the first 20 meters is very low due to the low base resistance and zero positive skin friction, as shown in Appendix C.2.

Pile number	Length [m]	$F_{c;d}$ [kN]	$R_{c;cal;max}$ [kN]	$R_{c;d}$ [kN]	$F_{nsf;d}$ [kN]	$R_{c;net;d}$ [kN]
Pile 1 (290x290)	-23.00	219	1261	756	518	238
Pile 2 (290x290)	-23.00	195	1261	756	518	238
Pile 3 (290x290)	-23.00	159	1261	756	518	238
Pile 4 (290x290)	-23.00	155	1261	756	518	238
Pile 5 (290x290)	-23.00	193	1261	756	518	238
Pile 6 (290x290)	-23.00	224	1261	756	518	238

Table 4.6: Bearing capacity prefab piles foundation Delft

However, for all piles, the normative factor for the length determination was the STR/GEO and SLS settlements. This is because of the high negative skin friction the first 20 meters and thereby low net bearing capacity $R_{c;net}$. At a pile level of 23 meters, the balance between the base and shaft resistance is maximum, compared with the attached depths (Appendix. C.2). This is because, at a depth of 22 meters, the total generated bearing capacity was not satisfied, and at a depth of 24 meters, the base resistance was reduced from 753 kN to 568 kN. This reduction is caused by a small loam layer at a pile level of 25.3 meters, thereby decreasing the base resistance. This subsequently resulted in the fact that the prefab concrete pile levels are all placed at a level of -23.00 meters in Delft. Compared with Zwolle, the prefab concrete pile length increased by 396% With the derived pile length and corresponding cross-section dimensions, as seen in Table 4.1, the structural capacity and the reinforcement design can be calculated. Because all six piles have the same length of 23 meters, it is desired to have the same amount of reinforcement because of the simplicity during manufacturing. From the structural capacity calculation in Appendix C.3, the following pile design is developed as depicted in Figure 4.6. The width and height of the pile remain 290x290 mm, which originates from the bearing capacity of the pile in Delft. As mentioned in section 4.1, the concrete class will be C45/55, the reinforcement

Pile number	Length [m]	S_d STR/GEO [m]	ϕ_d STR/GEO	S_d SLS [m]	ϕ_d SLS
Max value:	-28.00 [m]	0.150 [m]	0.01 [-]	0.05 [m]	0.003 [-]
Pile 1 (290x290)	-23.00	0.035	0.0027 (1-4)	0.019	0.0017 (1-4)
Pile 2 (290x290)	-23.00	0.029	0.0025 (2-3)	0.018	0.0016 (2-3)
Pile 3 (290x290)	-23.00	0.023	0.0021 (2-3)	0.016	0.0015 (2-3)
Pile 4 (290x290)	-23.00	0.023	0.0021 (4-5)	0.016	0.0015 (4-5)
Pile 5 (290x290)	-23.00	0.028	0.0025 (4-5)	0.018	0.0016 (4-5)
Pile 6 (290x290)	-23.00	0.036	0.0027 (3-6)	0.019	0.0017 (3-6)

Table 4.7: Verification checks bearing capacity Delft

steel B500B, and the prestressed steel will be Y1860 (S7). The longitudinal reinforcement consists of 6 prestressed strands, three on the top and three at the bottom. The total diameter of the stand is enlarged to 9.3 mm, a commonly used diameter for prestressed steel (NEN 3868:2001). However, because of the enlarged diameter, the strand consists of 7 wires instead of 3, twisted around a single wire. The surface of each strand is 52 mm^2, and therefore the total surface of prestressed reinforcement is 312 mm^2. The concrete cover also needs to be increased to 45 mm, which makes the distance edge to the c.o.g. strand 49.65 mm. The head and base reinforcement is the same as the foundation pile in Zwolle and corresponds with the BRL 2352 standard. Again, the edges are cut because of the casting restrictions.

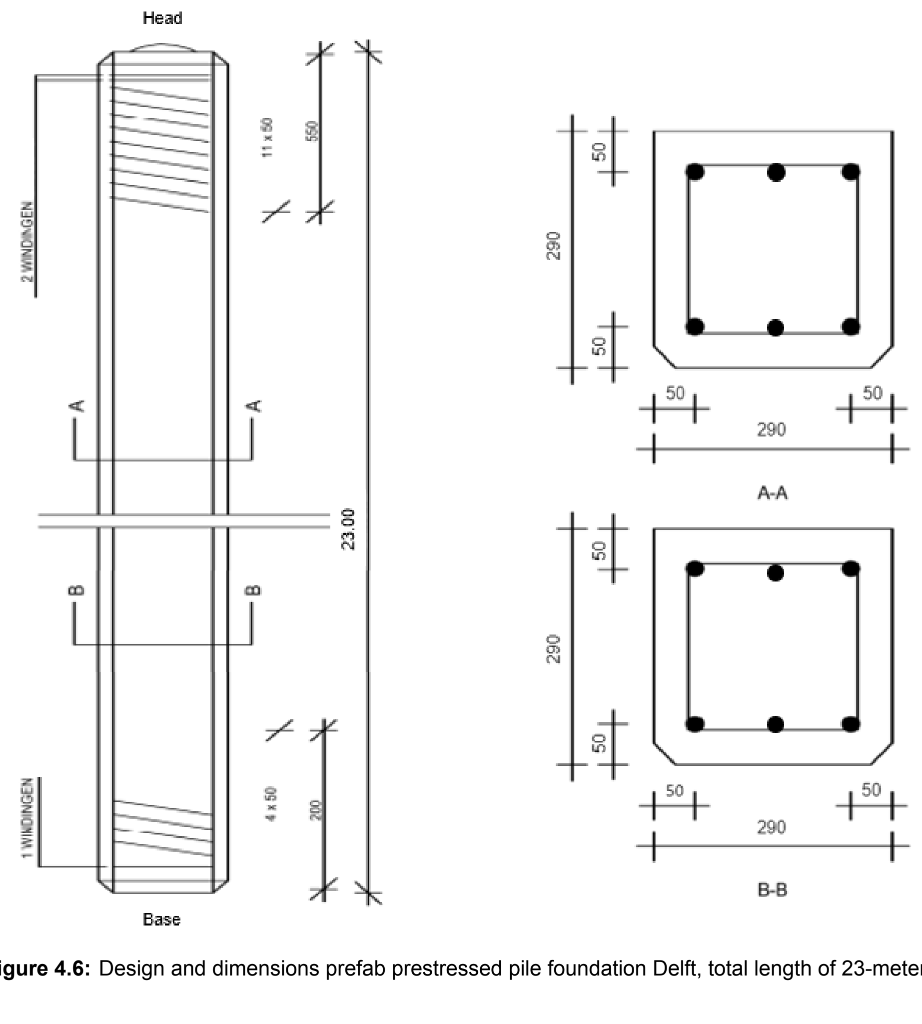


Figure 4.6: Design and dimensions prefab prestressed pile foundation Delft, total length of 23-meters

The strands will be prestressed with 5.0 N/mm^2, which makes the total prestress force 421 kN and 70.1 kN per strand. However, the prestress losses must be considered as elaborated in Appendix C.3. The absolute value stress reduction due to creep, shrinkage, and relaxation is calculated at 53.53 N/mm^2. Therefore, the total working stress of the prestressed steel σ_{pw} is 1248 N/mm^2. Therefore, the prestressed pile's calculated centric pile load capacity results in 2246 kN. In Table 4.8, the most important structural capacity values of the foundation piles in Delft can be seen. The values per pile change because of the small different pile normal forces on the pile as seen in Table 4.6. From Table 4.8, it can be concluded that the reinforcement of 6∅9.3 mm fulfils all required capacity checks. Because of the relatively large pile length of 23 meters, the (dynamic) moments during transport and lifting increase significantly. The unity check for dynamic transport was normative for the foundation design. As shown in Table 4.8, the dynamic moment for lifting is 34.6 kNm. The maximum moment before the six concrete, prestressed foundation piles start to crack is 35.4 kNm (M_r), which results in a UC of 0.98 due to lifting. The combination of normal force plus the moment due to the eccentricity and the moment resistance results in a unity check of around 0.5 for all six piles. It can be concluded the foundation design in Figure 4.5 satisfies all design requirements. The calculations and the weights of all materials can be found in Appendix C.3.

Pile number	Reinfor-cement	$M_{ed;dyn;T}$ [kNm]	$M_{ed;dyn;L}$ [kNm]	M_{cr} [kNm]	M_u [kNm]	$N_{rd,max}$ [kN]	N_{ed} [kN]	M_{ed} [kNm]	M_{rd} M+N
1 (290x290)	6 ∅ 9.3	31.7	34.6	35.4	60.6	2246	688	34.4	71.6
2 (290x290)	6 ∅ 9.3	31.7	34.6	35.4	60.6	2246	664	33.2	70.0
3 (290x290)	6 ∅ 9.3	31.7	34.6	35.4	60.6	2246	628	31.4	67.6
4 (290x290)	6 ∅ 9.3	31.7	34.6	35.4	60.6	2246	624	31.2	67.3
5 (290x290)	6 ∅ 9.3	31.7	34.6	35.4	60.6	2246	662	33.1	69.9
6 (290x290)	6 ∅ 9.3	31.7	34.6	35.4	60.6	2246	693	34.6	71.9

Table 4.8: Structural capacity of prestressed foundation piles Delft

4.4. Environmental impact

In the previous sections, the design of the prestressed prefab foundation pile is determined. With that, all foundation elements and their quantities are known, which is required to investigate the environmental impact in detail. In Appendix C.3, the total volume and weights of the concrete, prestressed reinforcement, and regular reinforcement are calculated. The environmental impact of the concrete elements can be calculated using the ”Ontwerptool Groen Beton V6”. This tool contains the environmental data of concrete elements from LCA-stage A to D. The calculation tool can be divided into four steps. The first is implementing the detailed concrete mixture for a specific concrete strength class. The second step is the prefab stage, where the reinforcement, casting materials, energy, and fuel can be entered. The third stage is the process to the building site, in which the transport and installation of the prefab elements can be completed. The final stage is the building element, a combination of one or more above-mentioned elements. This stage also contains the representation of the environmental impact. The environmental data per element can be found in Appendix F.

4.4.1. Concrete mixture (A1)

The first step is determining the concrete mixture for the given concrete strength class C45/55. The locations Zwolle and Delft have the same concrete class and requirements and, therefore same concrete mixture. The concrete mixture can be divided into the following elements: cement, sand, gravel, water, plasticiser, and powdered fly ash. In Table 4.9, the ratios between the elements can be seen for a C45/55 concrete mixture for both ordinary Portland cement (OPC) mixture and Blast Furnace cement (BFS). The CEM III has the A classification, which means the mixture needs to contain at least 40% Blast Furnace Slag. The value of 52.5 in both mixtures represents the minimum 28-day strength in MPa. The letter R represents a rapid initial strength, and the letter N a neutral initial strength of the cement. It must be mentioned that the data is rounded because of confidential reasons from concrete suppliers. The C45/55 OPC concrete has a density of 2398 $\mathrm{kg/m^3}$ and a water bonding factor of 0.43, whereas the Blast Furnace C45/55 concrete has a density of 2402 $\mathrm{kg/m^3}$ and a water bonding factor of 0.39. The downside of BFS cement concrete is the slower hardening time, displayed as neutral (N) instead of rapid (R), as elaborated in Appendix C.3.4. This is because CEM III is a latent hydraulic substance, meaning the water reaction only happens properly with an activator [13]. However, it can be concluded that the CEM III mixture reaches a mean concrete strength of 26 MPa after one day instead of 29 MPa after one day with the OPC mixture. The elaborate strength development of both mixtures can be found in Appendix C.3.4. Therefore, the difference in hardening time is limited between OPC and BFS. However, this is the primary reason prefab manufacturers do not frequently use CEM III in their concrete mixtures because of the standardised and cost-efficient 1-day pouring and demolding process. The manufacturers of prefab prestressed components aim to accelerate the demolding process to optimise production and increase output quantities. This requires a concrete mixture with a rapid hardening time and high strength due to the relatively high forces experienced during lifting. CEM III also has a benefit over CEM I, which is the high permeability resistance against chloride intrusion. Therefore, it is highly applicable in marine (sea) environments [13]. The two concrete compositions in table 4.9 can be filled in the ”Ontwertool Groen Beton V6”. It is assumed that fresh tap water will be used for the water element. For the sand type, regular concrete coarse sand is used. For workability, specific requirements are placed on the grain structure and shape of the sand.

Elements concrete mixture	Portland cement concrete [kg/m3]	Blast Furnace concrete [kg/m3]
CEM I 52,5R*	310	0
CEM III/A 52,5N*	0	360
Powdered fly ash	110	0
Sand (concrete)	800	750
Gravel	1025	1150
Water	150	140
Plasticizer	2.8	1.8

*ENCI / HeidelbergCement, NMDv3.5 (C1)

Table 4.9: Concrete mixture C45/55 prefab prestressed pile both Zwolle and Delft

4.4.2. Prefab pile foundation manufacturing (A3)

With the calculated concrete mixtures, the composition of the prefab elements can be created, which contain prestressed longitudinal reinforcement and regular head and base spiral reinforcement. The quantities of the pile elements are calculated in Appendix C.3. It is assumed that the concrete mixture plant is directly placed next to the prefab foundation manufacturer, and therefore no additional transport is needed. The formwork of the prestressed prefab elements is done with a steel mould. Given that this steel mould can be reused thousands of times and is readily available from manufacturers, the environmental impact of the moulds will not be taken into consideration. For the required energy for the entire pouring and prestress process, it is assumed that the average heavy machine has a power of 20 kW [49]. Besides that, it is assumed that the 5-meter Zwolle pile has an electric process time of 90 minutes and that the 23-meter Delft pile has a pouring time of 150 minutes. Therefore, the total amount of electricity per pile in Zwolle and Delft for the prefabrication process is estimated on respectively 30 kWh and 50 kWh. On top of this required energy, an average concrete plan usage of 3.63 kWh electricity, 0.12 l diesel, and 0.13 gas m^3 [10] per cubic meter of concrete mixture will be considered during the A3 manufacturing phase.

4.4.3. Transport and installation (A4+A5)

Next to the environmental impact of the prefab elements in subsection 4.4.2, the impact of the transport and installation processes also needs to be determined. Firstly, the transport of the six piles towards the location in both Zwolle and Delft needs to be performed. The distance between the prefab manufacturer and the building site will be the same to achieve a fair comparison between all the foundation variants. This way, the results will not depend on the distances between the building site and the manufacturer. Therefore, it is assumed that the prefab factory is placed exactly between the locations in Delft and Zwolle. This results in a distance of 82 km between the building site in Delft/Zwolle to the manufacturer. The transport of the piles to Zwolle and Delft will be performed with EURO 6 (>32 ton) diesel trucks, which comply with the latest Euro emissions standard [53]. Because of the latest emission restrictions, these trucks have the lowest possible diesel emissions. As observed in subsection 2.3.4, the transportation of 27.5-meter foundation piles was carried out in sets of three, which is dictated by the maximum permissible divisible load of 50 tons, as discussed in [18]. An extended truck with supports for transporting the piles is approximately 25 tons; therefore, a maximum of 25 tons can be transported. The six, 23-meter prefab prestressed piles weigh 28200 Kg. Therefore, the maximum number of piles per truck is 5, to remain below the 50 tons Dutch transport weight restriction. As a consequence, the 23-meter piles of Delft will be transported per two trucks, with each three transported piles. For the transport of the shorter piles in Zwolle, a single truck will satisfy. This is because the total weight of the six piles is 3600 kg, which satisfies the maximum of 50 tons easily [18]. For Zwolle, a single empty return journey of the truck is considered, and for Delft, two empty return journeys are considered.

As seen in subsection 2.3.4, the installation of the prefab concrete foundation piles can be done in multiple ways. The most used method for installing prestressed prefab foundation piles is piling. Piling machines are usually equipped with a large diesel engine. A similar diesel piling machine will be used for both Zwolle and Delft. The rig provides both the lifting and piling of the piles. It is assumed that the piling rig will be used for 2.0 hours in total for the six relatively long piles in Delft. This is because the first 18 meters, the pile only had to be piled through weak clay and loam. The piling rig will be assumed to be used for 15 houses per location. Therefore, the total transport of the rig will be 82*2 km divided by 15 houses. This results in a total transport distance of 11 km per single house (double trip). The transport of the rig will be done using the same EURO6 truck. The piling rig will be used for 1.0 hours for Zwolle because of the shorter pile lengths and piling time. Again, no dismantling or recycling will be considered because it is presumed that the piles remain in the soil after a lifespan of 75 years. It is also assumed that no additional lifting machine is needed.

4.4.4. Representations of ECI value by category

The environmental impacts of the prestressed foundation piles in Zwolle and Delft can be displayed by utilising the prefab prestressed design calculations outlined in section 4.2, along with the aforementioned transport and installation processes. Due to the two concrete mixtures, given in Table 4.9, a distinction will be made between OPC (CEM I) and BFC (CEM III) prestressed concrete pile designs in the environmental analysis.

ECI representation Zwolle

For the shorter, 5.0-meter-long prestressed prefab piles in Zwolle, the total ECI value is calculated at €61.74 and €53.87 €/m^2$*$y, for respectively CEM I and CEM III, as depicted in Figure 4.7. This comes down to an MPG value of 0.0051 and 0.0045, as can be calculated using equation 2.1. It can be inferred from Figure 4.7 that the global warming potential (GWP) contributes the most to the ECI value. Besides that, Human Toxicity (HT), Acidification (AP), and Eutrophication (EP) have a significant contribution to the environmental impact of the foundation design. The production of cement accounts for 64% of the total GWP for the CEM I foundation piles. The cement production especially causes the AP and EP contributions. The Y1860 prestressed steel contributes approximately 13% to the GWP. The Human Toxicity environmental impact is primarily caused by the production of cement and steel, which roughly have the same contribution. The transportation of the piles also contributes 30% to the HT impact. However, what especially strikes is the reduction in GWP by applying BFS instead of OPC to the foundation piles. Implementing CEM III results in a total ECI reduction of 16% of the six Zwolle foundation piles as shown in Figure 4.7.

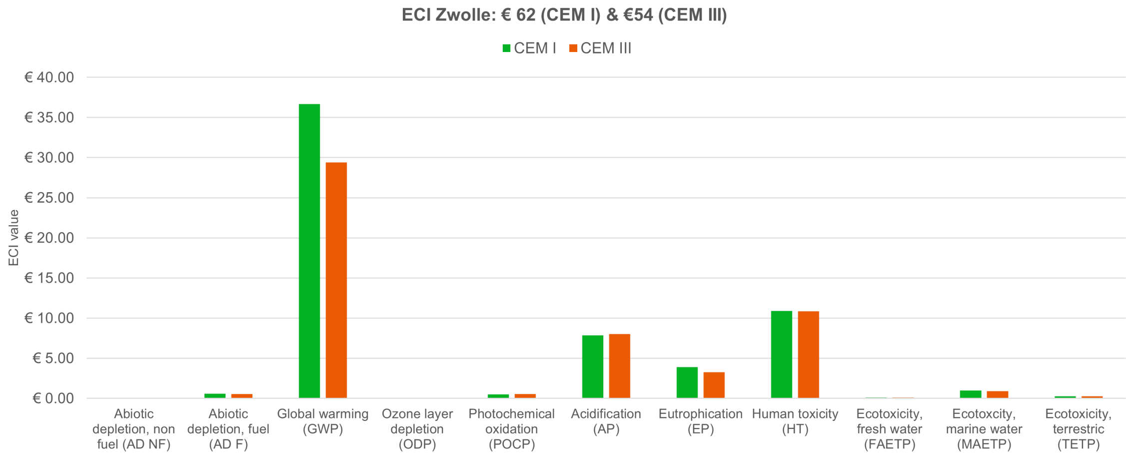


Figure 4.7: ECI value of Zwolle per impact category CEM I/III

Figure 4.11 shows Zwolle's environmental impact per LCA stage. The raw materials supply stage (A1) contributes the most to the ECI value for CEM I and III. However, for CEM III this share is almost equal to the piling process. The manufacturing process (A3) has a larger relative contribution to the ECI compared to Delft. This discrepancy in ECI values is primarily attributed to the minimum energy requirement during the prefabrication of the concrete piles, which is depicted in Figure 4.9. Compared to Delft, the transport phase to the building site (A4) has a relatively smaller contribution to the ECI value in Zwolle. This is due to a single truck trip from the manufacturer to the building site and the low self-weight of the piles in Zwolle. The construction and installation phase (A5) has a larger share in the total ECI value for Zwolle. This is because the smaller piles necessitate a minimum piling time, resulting in a larger proportion of the total ECI. This can also be confirmed by Figure 4.9, where the diesel piling process is the second largest contributor to the ECI.

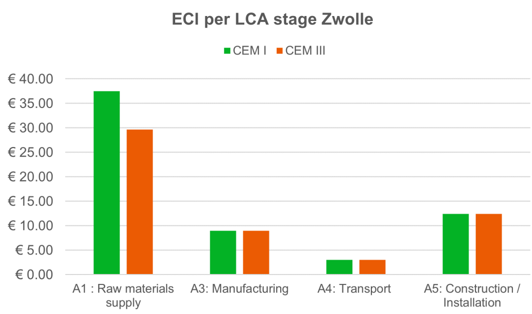


Figure 4.8: ECI value per LCA stage Zwolle

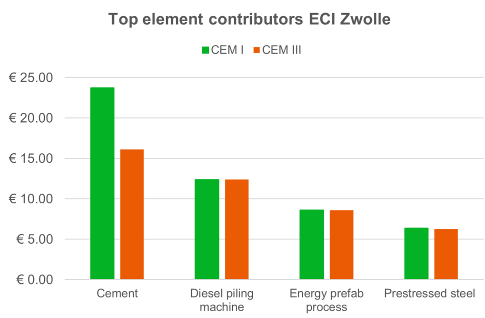


Figure 4.9: Top 4 elements contribution ECI Zwolle

ECI representation Delft

The total Environmental Cost Indicator values for CEM I and CEM III in Delft are €389 and €326, respectively, as shown in Figure 4.10. This comes down to an MPG of 0.032 and 0.027 €/m^2*y for, respectively CEM I and CEM III. It can be concluded from Figure 4.10 that the GWP is again the largest contributing impact category on the total ECI value, which is caused by the high environmental impact of the cement.

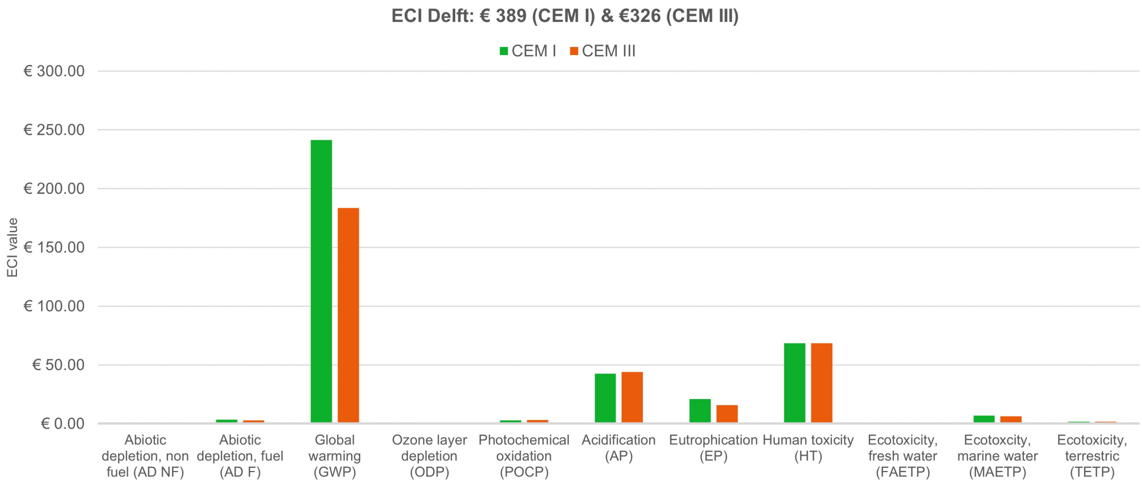


Figure 4.10: ECI value of Delft per impact category CEM I/III

The large ECI contribution of cement can also be confirmed by Figure 4.12. The cement has the largest contribution to the total ECI value for the CEM I and III designs. The second largest element contribution is the Y1860 prestressed steel. This can also be seen in Figure 4.11, where the raw materials supply (A1) is the greatest environmental impact stage. Furthermore, the transportation of the six 23-meter-long foundation piles significantly influences the environmental impact. This is particularly attributed to the requirement of two truck trips due to the 50-ton weight restriction (A4). The hydraulic diesel piling and, thereby construction and installation process (A5) have a relatively small ECI contribution in comparison with the raw materials supply because of the short 2-hour piling time. Consequently, it can be concluded that electric transport would likely have the greatest impact on reducing the ECI value of the foundation in Delft, while electric piling would be more effective in Zwolle. The prefab manufacturing process (A3) is the smallest LCA contribution stage because of the relatively low required energy for the prefab production process. This is mainly caused by the high standardisation and effectiveness of the prefab prestressed pile process. Besides the raw materials supply (A1), the environmental impact stages and elements are similar for the CEM I and CEM III mixtures. It can be concluded that the location of the foundation pile in the Netherlands plays a crucial role in the overall environmental impact of the foundation. The ECI value of the foundation in Zwolle has an 84% reduction compared to the ECI value of the foundation in Delft. This reduction is due to the weak soil conditions in Delft, which require larger foundation element dimensions of these processes.

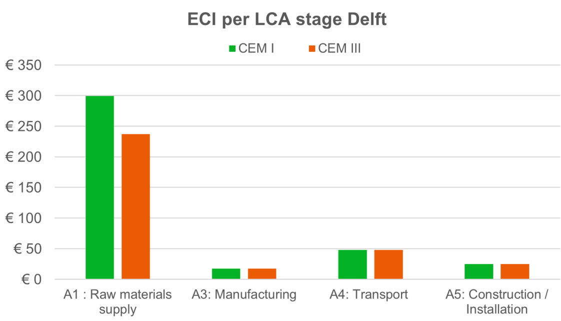


Figure 4.11: ECI value per LCA stage Delft

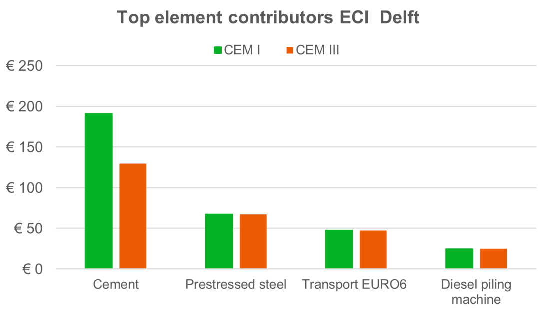


Figure 4.12: Top 4 elements contribution ECI Delft

4.5. ECI Optimisation

From the calculated and analysed environmental impact in section 4.4, the foundation design in both Zwolle and Delft can be optimised to lower the ECI value. The first optimisation was the application of BFS (CEM III) instead of OPC (CEM I), as seen in section 4.4. This resulted in a 16.2% ECI reduction of the foundation piles in Delft and a 12.9% reduction for Zwolle. However, in this section, other optimisations will be investigated. These optimisations are lowering the concrete strength class with optimum reinforcement and optimising the transport and piling processes.

4.5.1. Lowering concrete strength class and optimum reinforcement

The first optimisation is to lower the concrete strength class. This will result in a higher w/c ratio, which subsequently reduces the cement ratio in the concrete mixture. However, reducing the concrete strength class will also result in a lower bearing capacity of the foundation pile and subsequently increase the required (prestressed) reinforcement steel. Therefore, it is interesting to investigate if there is a more optimum ratio between strength class and reinforcement than the regular C45/55 for both the locations Zwolle and Delft. First, there are specific requirements regarding the minimum concrete class for foundation piles. The EN 12794:2005 + A1:2007 standard mentions that the reinforced or prestressed precast foundation piles shall be at least C35/45. It must be mentioned that with a lower class, the chance of failure during piling will increase. However, it is assumed that the C35/45, combined with spiral reinforcement, provides enough capacity. Only the concrete strength classes C45/55, C40/50, and C35/45 will be considered in the optimisation. A higher strength class than C45/55 will result in a higher w/c ratio and environmental impact. Therefore, higher concrete classes will not be considered. Lowering the concrete strength class will change the following input parameters in the calculation sheet in Appendix C: The compressive strength (initial + 28 days), the E-modulus (initial + 28 days), and the mean tensile strength. These values can be obtained using the concrete design table according to EN1992-1-1. Reducing these parameters will result in a lower bearing capacity of the piles. However, the prestressed reinforcement steel has standardised dimensions in the Netherlands and is made from Y1860 steel. The available diameters and dimensions in the Netherlands can be found in Appendix C.3.3. Furthermore, it is assumed that the tensile strength Y1860 can not be deviated from. The moments due to transport, lifting, and eccentricity remain constant.

The unity checks for the C45/55, 5-meter foundation piles in Zwolle are pretty conservative. This is primarily due to the standardisation of the foundation piles combined with their relatively short length. The 4 ∅ 7.5 mm. 3-wire prestressed reinforcement strands are commonly used in 220x220 mm prefab piles. However, in many instances, the 220x220 mm foundation piles, in general, are longer than the calculated 5-meters. As a result, the critical UC is caused by the eccentricity of the pile load and is calculated at 0.58, as can be found in Appendix C. The unity checks for both transport and piling are 0.23 and 0.09 and are therefore not the dimensional checks. If the concrete mixture of the prefab prestressed foundation is changed from C45/55 to C35/45, the resulting unity check for the occurring moment due to the eccentricity of the pile is 0.63 as can be seen in Table 4.10. This relatively small reduction in resistance is due to the remaining capacity provided by the prestressing forces. The prestressing reinforcement steel significantly contributes to the ultimate moment resistance of the beam. Additionally, the cracking moment can be calculated by considering both the tensile strength of the concrete class and the compressive strength resulting from prestressing. Therefore, it can be concluded that lowering the concrete strength class of the 5-meter Zwolle piles to C35/40 still fulfils all the requirements, and no additional (prestressed) reinforcement is needed. The 7.5 mm reinforcement steel is the minimum and only available 3-wire strand diameter in the Netherlands, therefore the diameter can not be reduced to a lower dimension. However, the optimum reinforcement depends on requirements about the minimum amount of reinforcement. In Table 4.10 the optimum reinforcement can be seen, which depends on the minimum required reinforcement. However, these steel prestressed reinforcement configurations are unavailable in the Netherlands and will therefore not be considered in the optimisation.

The unity checks for the longer 23-meter piles in Delft are more critical compared to those in Zwolle. This is primarily due to the significantly higher moments that occur during the (dynamic) transportation and lifting of the piles. Specifically, for the 290x290 mm, 6 ∅ 9.3 mm, C45/55 prestressed prefab foundation piles, the critical UC was calculated at 0.98, considering the dynamic lifting of the piles

(Appendix C). Reducing the concrete strength class to C40/50, results in the normative dynamic lifting unity check of 1.02, which is not allowed. Due to the reduced tensile strength, the foundation pile will crack during the lifting process. To increase the cracking moment capacity, the prestressed reinforcement needs to be increased. The most simple and low-environment effective solution is to increase the diameter of the prestressed strands. However, as can be found in Appendix C.3.3, there is not a wide variety of strand diameters available in the Netherlands. This limitation is also why many prefab prestressed foundation piles are equipped with larger diameter strands than necessary, which results in over-dimensioning of the pile. Therefore, the diameter of the reinforcement bars will be increased to 6 ∅ 12.5 mm, resulting in a total prestressed reinforcement area of 558 mm^2. The larger diameter leads to a higher working prestress in the prestressing steel σ_{pw} and a higher A_p/A_n ratio. As a result, the working stress in the concrete σ_{cw} is increased and thereby the cracking moment capacity. This results in a cracking moment capacity M_r of 48.25 kNm and a critical UC of 0.72 due to the dynamic lifting process. The optimum diameter for the C45/55 six prestressed reinforcement would be 9.6 mm, which results in a UC of 0.98. However, this reinforcement design is not a usual quality and diameter, as mentioned in the NEN 3868:2001. For the concrete strength class C35/45, the cracking moment capacity for the 6 ∅ 12.5 mm prestressed reinforcement steel in Delft is calculated at 46.9 kNm. This results in a UC due to the dynamic lifting of 0.74. This decrease in cracking capacity arises from the reduced tensile capacity of the lower concrete strength category. Furthermore, the optimal diameter of prestressed reinforcement for the C35/45 Delft pile is 10 mm. However, this diameter does not conform to the standard prestressed steel sizes in the Netherlands, necessitating 12.5 mm Y1860(S7) reinforcement steel. The design and verification of the piles with optimum and available reinforcement can be seen in Table 4.10.

	Zwolle			**Delft**		
Pile characteristic	**C45/55**	**C40/50**	**C35/45**	**C45/55**	**C40/50**	**C35/45**
Pile length [m]	5.0	5.0	5.0	23.0	23.0	23.0
Cross section [mm]	220x220	220x220	220x220	290x290	290x290	290x290
Optimum reinforcement [mm]	4 ∅ 6.5	4 ∅ 6.8	4 ∅ 7.0	6 ∅ 9.2	6 ∅ 9.6	6 ∅ 10
Available reinforcement [mm]	4 ∅ 7.5	4 ∅ 7.5	4 ∅ 7.5	6 ∅ 9.3	6 ∅ 12.5	6 ∅ 12.5
Cracking capacity M_r [kNm]	12.7	12.1	11.5	35.4	48.3	46.9
Normative U.C [-]	0.58	0.60	0.63	0.98	0.72	0.74

Table 4.10: Concrete mixtures with available and optimum reinforcement

By determining the ratio between the concrete strength classes and the quantity of prestressed reinforcements, the quantities of the elements can be recalculated. As mentioned earlier, reducing the concrete strength class will lead to an increase in the w/c ratio of the concrete mixture, resulting in a lower environmental impact. Table 4.9 presents the composition of the C45/55 mixture, including CEM I and CEM III. This mixture will be a benchmark for determining the C40/50 and C35/45 concrete mixtures. The environmental class assigned to the foundation will be XC2, as it will be installed in a consistently wet environment, as evidenced in Appendix A.2. The maximum w/c factor in an XC2 environmental class is 0.60. Therefore, the concrete mixtures for the foundation piles may not exceed a w/c ratio of 0.6. However, the CEM I 52.5R mixture also contains powdered fly ash as aggregate. This is because it increases the workability of the mixture and requires less cement. The powdered fly ash aggregate also influences the w/c ratio, leading to a water bonding factor (wbf). However, only a portion of the aggregate can be considered a binder, which can be evaluated using a k-factor. The wbf can be calculated with equation 4.5, where the factor k depends on the type of aggregate. For powdered fly ash in CEM I 52.5R, this k-factor is 0.4, where it is not allowed to consider the fly ash more than 1/3 of the cement mass [14]. The wbf factor for the C45/55 CEM I concrete mixture is calculated as 0.43. To determine the C40/50 and C35/45 CEM I concrete mixtures, the wbf ratio will be increased to respectively 0.48 and 0.53 as calculated with equation 4.6 (NEN-EN 206-1). This adjustment allows for the composition of the desired concrete strength class.

$$wbf = w/(c + k \cdot m) \tag{4.5}$$

$$w/c = 25/(f_{cm,j} + 45 - 0.8 * N_j) \tag{4.6}$$

For the CEM III/A 52.5N concrete mixture, the w/c is calculated at 0.39, using equation 4.5. Using equation 4.6, the w/c ratio of C40/50 and C35/45 CEM III is calculated at respectively 0.46 and 0.53. From Table 4.11, it can be concluded that reducing the concrete strength class from C45/55 to C35/45 leads to a reduction in cement usage of 19.4% for CEM I and 26.4% for CEM III. Additionally, both the CEM I and CEM III mixtures maintain a constant amount of water, while the amounts of sand and gravel are increased to achieve a similar density in the concrete mixture. The powdered fly ash ratio will reduce at a similar rate as the cement reduction. It should be noted that the calculated concrete mixtures for the lower strength class may exhibit slight deviations from the mixtures used by different manufacturers. This is due to each manufacturer's unique preferences and practices, which can result in minor variations.

	CEM I 52.5R			**CEM III/A 52.5N**		
Elements	**C45/55**	**C40/50**	**C35/45**	**C45/55**	**C40/50**	**C35/45**
CEM I / CEM III	310	275	250	360	305	265
Powdered fly ash	110	100	90	0	0	0
Sand (concrete)	800	820	836	750	771	786
Gravel	1025	1051	1072	1150	1182	1206
Water	150	150	150	140	140	140
Plasticizer	2.8	2.8	2.8	1.8	1.8	1.8
Wbf & w/c [-]	0.43	0.48	0.53	0.39	0.46	0.53
Density [kg/m^3]	2398	2398	2401	2402	2400	2399

Table 4.11: Concrete mixtures with different strength classes

With the determined configurations of lower concrete strength class and prestressed reinforcement steel, the corresponding environmental impact can be determined. It is assumed that only the raw materials phase (A1) is influenced by the reduction of concrete strength class. The ECI values of the other LCA phases will remain similar to the calculated rates in subsection 4.4.4. As mentioned, the available prestressed reinforcement diameters in the Netherlands (Table 4.10) will be used and not the optimum calculated diameter. Again, there will be a distinction between the location of Zwolle and Delft.

Optimisation concrete class Zwolle

For the concrete strength class optimisation in Zwolle, no additional (prestressed) reinforcement is required for the C45/55, C40/50, and C35/45 classes as can be seen in Table 4.10. However, because of the available diameters in the Netherlands, the minimum diameter reinforcement is 7.5 mm. It is assumed that the reduction of strength class will not require additional (energy) processes during the manufacturing stage (A3). Considering the concrete mixtures of the lower strength classes in Table 4.11, the ECI values can be calculated for both CEM I and CEM III variants. Figure 4.13 shows the ECI value of Zwolle per impact category for the three concrete strength classes CEM III.

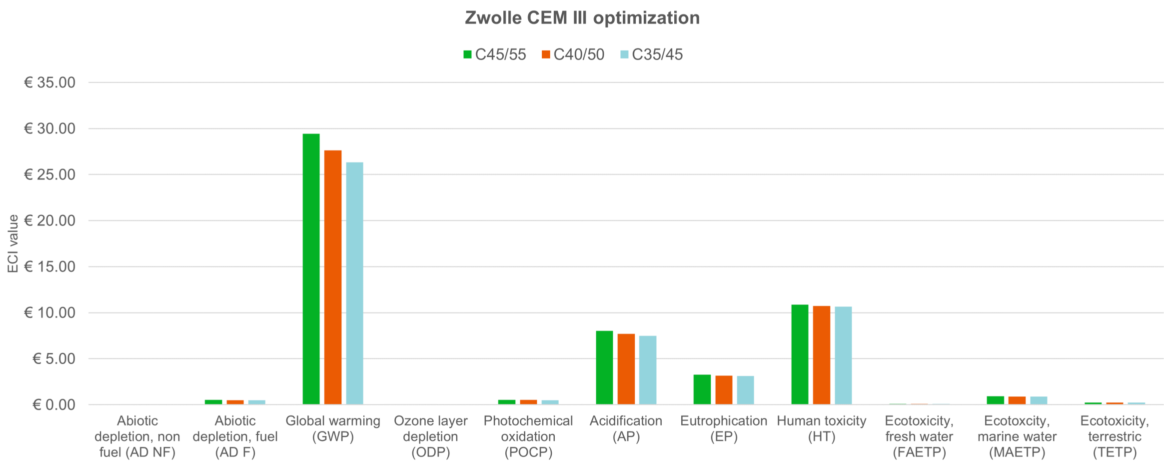


Figure 4.13: ECI value Zwolle per impact category of each strength class CEM III

It can be concluded, as expected, that the ECI values decrease with a decreasing strength class. Specifically, the GWP potential exhibits the most significant reduction. This is attributable to the reduced amount of cement required in the concrete mixtures of lower concrete classes. The other impact categories have a smaller relative environmental reduction because of similar processes. The CEM I concrete mixture variant has similar relative reductions in impact categories as the CEM III variant. The total Environmental Cost Indicator (ECI) value of CEM I Zwolle foundation piles decreases from €61.74 (C45/55) to €57.20 (C35/45) when the concrete strength class is reduced. This reduction corresponds to a decrease of 7.35%. Similarly, for CEM III Zwolle foundation piles, the ECI value decreases from €53.87 (C45/55) to €49.81 (C35/45), resulting in a reduction of 7.53%. It is important to note that this reduction only affects the raw materials supply stage (A1) of the life cycle assessment. Based on Figure 4.14, it can be concluded that the use of CEM III instead of CEM I has a greater impact on the ECI value than reducing the concrete strength class. In conclusion, combining using CEM III and reducing the concrete strength class to C35/45 results in the most favourable environmental outcome regarding the ECI value for the Zwolle foundation piles, with a value of €48.81. Compared with the CEM I, C45/55 option, this has a total decrease of 19.3%

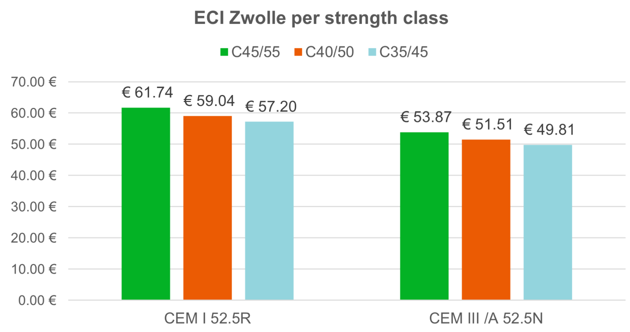


Figure 4.14: Total ECI value Zwolle per strength class for both CEMI/III

In addition to analysing the overall ECI reduction and its impact on the categories, it is also valuable to explore the (new) major contributors within the lower concrete strength class options. In Figure 4.15 the top 4 elements ECI contributions can be seen for the CEM I Zwolle foundation. What can be depicted is that the cement has still a relatively high contribution for all three strength classes in comparison with the other elements/processes. In Figure 4.16 the ECI contribution of the CEM III Zwolle variant can be seen. Because of the lower ECI value of Blast Furnace Slag, it has a lower relative contribution to the total value. Moreover, for the C35/45, the cement contribution is lower than the

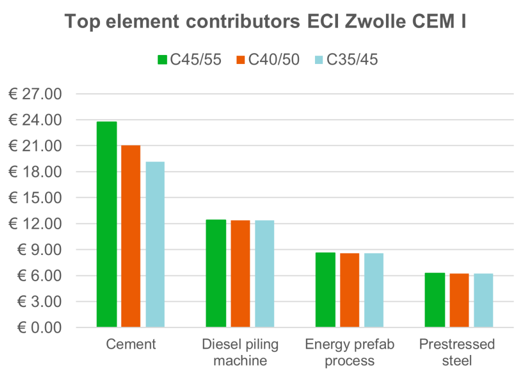


Figure 4.15: Top 4 elements contribution ECI Zwolle optimised CEM I 52.5R

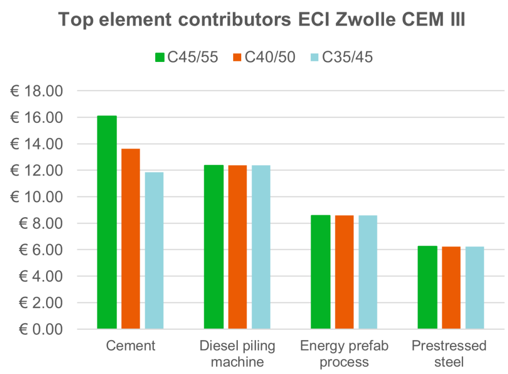


Figure 4.16: Top 4 elements contribution ECI Zwolle optimised CEM III/A 52.5N

diesel piling installation machine. The diesel piling rig has a total environmental cost of €13.63, while the CEM III cement has a total ECI of €12.37. Therefore, it can be concluded that particularly for this particular variant, the use of an electric piling rig can significantly reduce the environmental impact. It should be noted that in the analysis, no additional energy requirements were assumed for the lower concrete strength classes. As a result, the ECI value for the energy prefab process remains constant at €8.58, as indicated in both Figure 4.15 and Figure 4.16. Similarly, the ECI value for prestressed steel remains unchanged due to the limited availability of prestressed reinforcement diameters. As a result, the amount of prestressed reinforcement remains €6.24 across all concrete strength variants.

Optimisation concrete class Delft
For the concrete strength class optimisation in Delft, additional prestressed reinforcement is required for the C40/50 and C35/45 concrete strength classes as can be seen in Table 4.10. It is assumed that reducing the strength class and increasing reinforcement will not require additional (energy) processes during the manufacturing stage (A3). Considering the concrete mixtures of the lower strength classes in Table 4.11, the ECI values can be calculated for both CEM I and CEM III variants. Figure 4.17 shows the ECI value in Delft per impact category for the three concrete strength classes CEM III. What is particularly striking is that the highest concrete strength class C45/55 has the lowest ECI value for each impact category. This is primarily due to the lesser amount of required prestressed reinforcement steel compared to the other strength classes. Both the C40/50 and C35/45 variants necessitate a total of 105.1 kg of prestressed reinforcement steel, specifically 6 ∅ 12.5 strands. On the other hand, the C45/55 class requires only 58.8 kg of prestressed steel, which corresponds to the 6 ∅ 9.3 strands. Especially for the HT, this difference can be seen because that category is primarily influenced by the amount of (prestressed) steel.

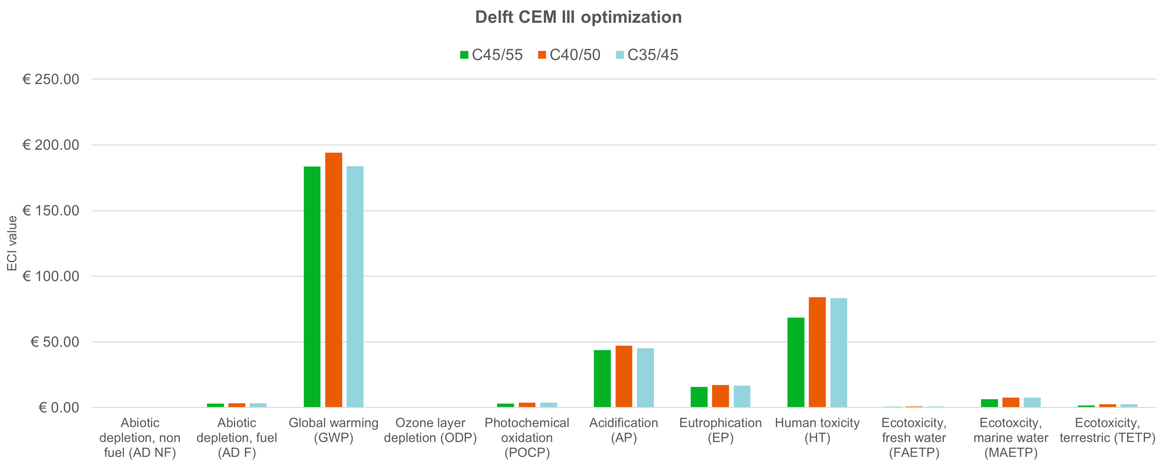


Figure 4.17: ECI value Delft per impact category of each strength class CEM III

This can also be seen in Figure 4.18, where the total ECI value per strength class for both CEM I/III of location Delft can be seen. The C45/55 CEM III variant has the lowest environmental impact for the Delft foundation piles, with a total cost of €389, while the C40/50 CEM I variant has a total ECI value of €421.

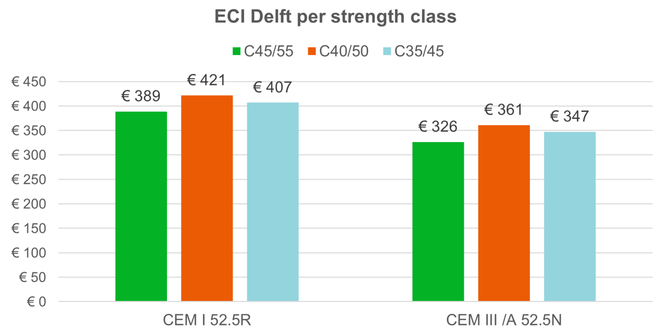


Figure 4.18: Total ECI value Delft per strength class for both CEMI/III

The implementation of CEM III cement instead of CEM I cement in the C45/55 variant, results in a reduction of €63 in the ECI value. This equates to a significant environmental impact saving of 16.2% as can be seen in Figure 4.18. However, if the concrete strength class is reduced to C40/50, this leads to a significantly increasing amount of reinforcement and thereby the ECI value rises to €421. The primary factor contributing to this increase is the limited availability of additional reinforcement and a restricted range of (prestressed) reinforcement steel diameters in the Netherlands. The reinforcement for both the C40/50 and C35/45 variants is 6 ∅ 12.5, resulting in a higher ECI value for the C35/45 variant, regardless of whether CEM I or CEM III cement is used. The cement contribution can also be seen in Figures 4.19 and 4.16. It can be noticed that the C45/55 variant has a significantly smaller ECI contribution due to the prestressed Y1860(S7) steel, despite the most considerable cement ECI contribution. The piles' dimensions remain constant; therefore, the transport and diesel piling processes have a similar ECI contribution for the Delft, and are respectively €47.48 and €24.74.

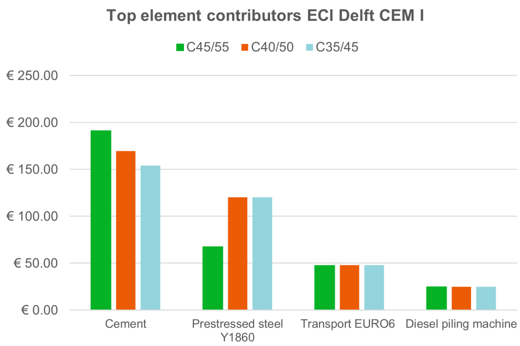


Figure 4.19: Top 4 elements contribution ECI Delft optimised CEM I 52.5R

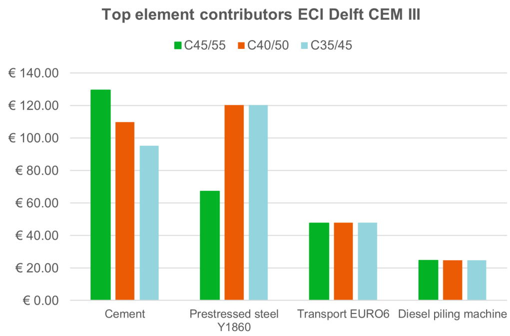


Figure 4.20: Top 4 elements contribution ECI Deft optimised CEM III/A 52.5N

It can be concluded that for Delft, the C45/55 CEM III variant has the lowest environmental impact. This is caused by the UC of 0.98 and therefore, the optimum balance between the concrete strength class and the required amount of reinforcement. By decreasing the concrete strength class, the amount of reinforcement increases substantially due to the limited available prestressed diameter in the Netherlands. In Table 4.10, the optimum reinforcement diameters of the different strength classes can be noticed. For the C45/55 variant, the optimal diameter is 6 ∅ 9.2. For the C40/50 variant, the optimal diameter is 6 ∅ 9.6, while for the C35/45 variant, it is 6 ∅ 10. Figure 4.21 displays the ECI values of the different concrete strength classes with the optimum reinforcement dimensions. It can be concluded that the available prestressed reinforcement dimensions can have a large influence on the environmental impact. In the case of Delft, reducing the concrete class to C40/50 and C35/45 would result in an increase of 46.4 kg of prestressed steel due to limited available dimensions. Since prestressed steel has a higher environmental impact compared to cement, lowering the concrete class is not effective for Delft, as depicted in Figure 4.18. However, if all dimensions of prestressed steel were available, reducing the concrete class would have been effective for Delft, as indicated in Figure 4.21.

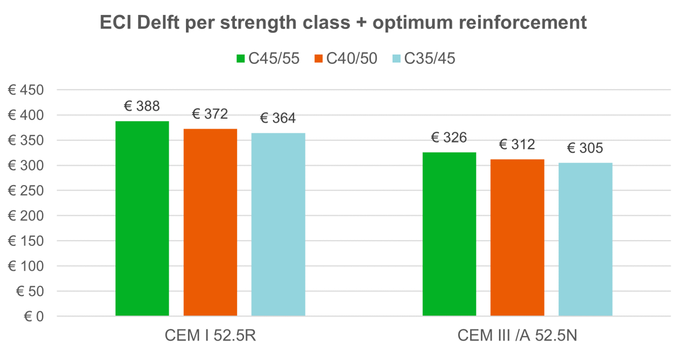


Figure 4.21: Total ECI value Delft per strength class for both CEMI/III with optimal reinforcement

4.5.2. Optimised transport and installation

Besides optimising the foundation pile design, it can also be effective to investigate the involved processes. For Zwolle, the diesel piling installation process is the second largest element contribution to the total ECI value as seen in Figure 4.9. This is caused by the relatively long piling process of the six prefab prestressed foundation piles. Meanwhile, for Delft, the EURO6 transport makes a significant contribution, primarily due to the two necessary truck trips resulting from weight restrictions as depicted in Figure 4.12. As seen in subsections 2.3.5 and 2.3.4, electric transport and installation rigs are used more often nowadays. This subsection will investigate the environmental reduction due to alternative transport of the prefab piles, and it will research the electric piling potential.

Transport foundation piles

Replacing the EURO6 diesel trucks with different power-generated trucks can reduce the emissions and thereby environmental impact of the transport stage (A4). However, the 50 tons maximum load restrictions in the Netherlands are also required for the more sustainable transport options. If the self-weight and freight load are below 50 tons, no additional permits are required for the transport. This requirement is often an important factor in terms of costs and efficiency.

The first transport alternative is the electric truck transport. It is assumed that the electric transport will be done with the Volvo FMX electric truck. This electric truck has a maximum combined capacity of 50 tons and a battery capacity of 540 kWh, which results in a maximum action radius of 300 km [68]. The electric truck has a weight of 27 tons, which makes the available weight for the transport 23 tons. The six Zwolle foundation piles weigh 3600 kg, which is well within the Dutch truck weight restrictions. The six Delft foundation piles weigh in total 28200 kg. Therefore, two electric truck trips are required towards the building site for the Delft foundation piles. The second transport alternative is using bio-generated fuel instead of regular diesel. There is a distinction between first and second-generation bio-diesel. The first-generation bio-diesel is derived from agricultural crops such as rapeseed and palm oil. The second-generation bio-diesel is derived from waste and is also known as Hydrotreated Vegetable Oil (HVO). A significant advantage of using bio-diesel fuel is its affordability and compatibility with existing infrastructure and trucks, requiring minimal modifications. However, particularly in the case of first-generation bio-diesel, there can be negative consequences such as large-scale deforestation and reduced food production. Therefore, nowadays policy is to use 2nd generation bio-diesel instead of 1st generation. However, the first generation will also be included, to investigate the ECI difference between the first and second generation. Another option to consider is a hydrogen-generated truck. Hyzon has developed a 50-ton capacity hydrogen truck with a remarkable action radius of 520 km on a single tank [36]. The hydrogen is converted into electricity within the vehicle using a fuel cell. Since the hydrogen truck also has a total capacity of 50 tons, only one trip is necessary for Zwolle, while two trips are required for Delft. One advantage of hydrogen-generated trucks is their extended action radius of 520 km compared to 300 km for electric trucks. However, a disadvantage of hydrogen trucks is their high cost and the limited availability of infrastructure. Figure 4.22 and 4.23 illustrates the ECI value for different transport variants options for respectively Zwolle and Delft.

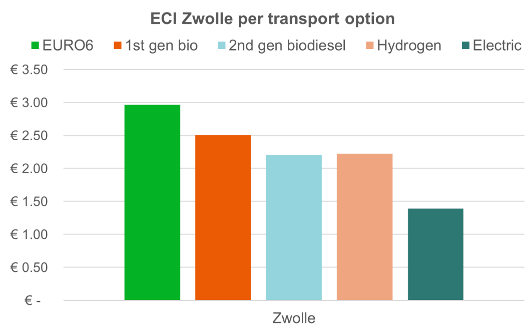


Figure 4.22: ECI value Zwolle per transport option variant

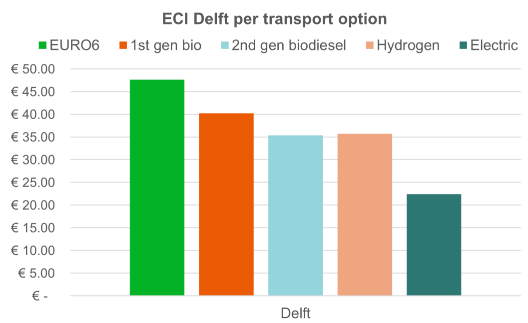


Figure 4.23: ECI value Delft per transport option variant

It can be concluded that the environmental impact reduction for Zwolle is very limited by implementing different transport variants. This is because the environmental impact part of the transport phase (A4) is minimal, due to the short single trip distance and the low freight weight of the six short Zwolle foundation piles. For the Delft, on the other hand, the absolute reduction has a higher effectiveness. Using green-generated electric transport, the ECI value of the foundation piles can be reduced by €25.23, equivalent to a 6.5% reduction. Moreover, it can be noticed that the first-generation bio-diesel has the smallest reduction in comparison with the EURO6 diesel truck. The hydrogen truck and second-generation bio-diesel have a similar reduction of roughly 3.65%. The green electric-generated transport has the lowest environmental impact and results in a total ECI reduction of 6.5%.

Figure 4.24 illustrates the ECI contribution per impact category per transport variant for Delft when considering only the transport life cycle stage (A4). The electric truck exhibits a significantly smaller ECI value compared to the EURO 6 diesel truck. By only considering the A4 stage, this results in a reduction of 53%, as depicted in the rightmost bar chart in Figure 4.24. The ratios between the ECI values remain consistent for Zwolle, but the quantities vary. The environmental impact per impact category varies slightly per transport variant. What strikes is that the EURO 6 diesel truck results in a relatively high ECI value for GWP and HT. Besides that, the hydrogen truck has a relatively high HT value in comparison with the bio-diesel fuels and the electric transport. Overall, it can be concluded that using alternative transport like electric, hydrogen, or bio-diesel generated fuel has a large relative saving on environmental impact in Delft. However, the ECI contribution for the transport phase (A4) is low compared to the ECI value of the design of concrete prestressed foundation piles (A1). Therefore, for Delft, the maximum ECI reduction by implementing electric trucks instead of EURO6 transport is €25.23, equivalent to a 6.5% reduction. For Zwolle, using electric transport results in €1.57, which means a 2.55% reduction. It needs to be noticed that these reduction values are based on the non-optimised C45/55 CEM I/III variants. By using the optimised concrete prestressed variants, the transport optimisation in Delft can have a larger influence in lowering the total ECI value.

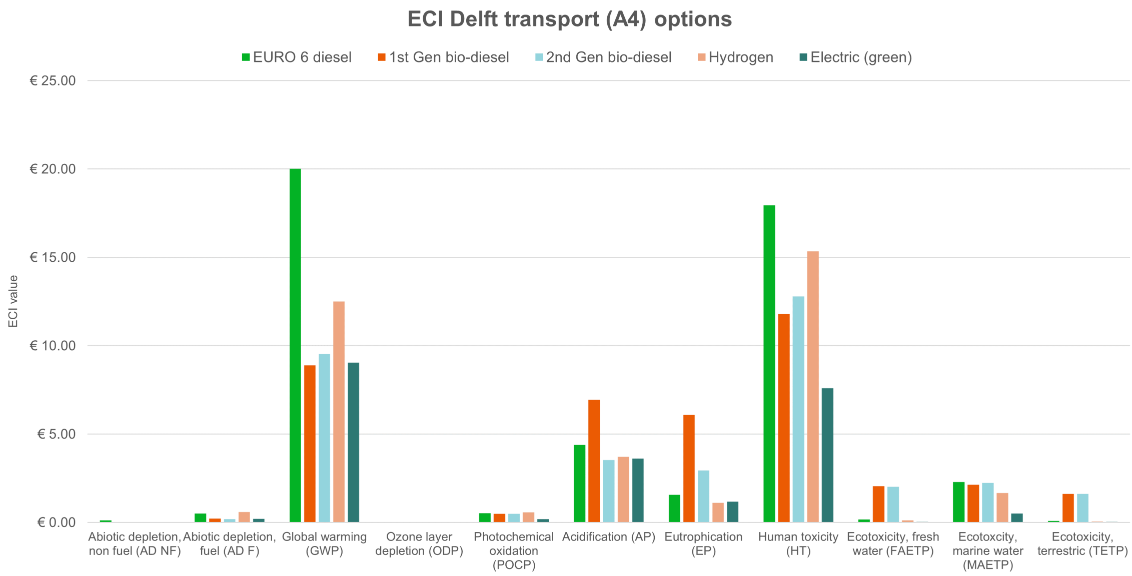


Figure 4.24: ECI value of transport variants Delft

Installation foundation piles

Besides the optimisation of the pile transport, the installation also has a significant contribution to the ECI value. Especially for Zwolle, the diesel piling machine has, besides cement, the largest environmental contribution. The regular heavy piling machines run on diesel, which has a high environmental impact. Therefore, it is interesting to investigate the effectiveness of an electric piling machine on the ECI value as mentioned in subsection 2.3.5. For the location in Zwolle, the piles are 5 meters long, which means a smaller and lighter machine is required compared to Delft. In Zwolle, it is assumed that the electric piling process will be carried out using the Junttan PMx2e piling rig. This rig has a total operational weight of 68,000 kg and can handle piles with a maximum length of 20 meters. Therefore, it is not suitable for the 23-meter piles in Delft. The Junttan PMx2e has two detachable 396 kWh battery packs, which provide 8-13 hours of continuous pile driving [39]. Based on this, it is assumed that the piling rig will consume an average of 75 kWh during the piling process, resulting in a total consumption of 75 kWh in Zwolle, considering one hour of piling. Due to the high self-weight of the piling rig, it is currently not feasible to transport it electrically, as electric trucks have limited freight weight capacity. Therefore, it is assumed that the rig will be transported using a larger EURO6 truck. Considering the limited resources available near the construction site, the rig will be charged with grey electricity. Using an electric piling rig in Zwolle leads to an ECI reduction of €8.80 in the installation stage (A5) as depicted in Figure 4.25. This results in an ECI reduction of 71.1% during the A5 stage. On the total ECI value, electric installation results in an ECI reduction of 14.2%

For Delft, a heavier piling rig is required because of the 23-meter-long piles. It is assumed that the piling process will be performed with the Woltman 90DRe electric drilling rig, as seen in Figure 2.14. The Woltman 90DRe has a total weight of 76 tons for transportation and can handle piles with a maximum length of 36 meters. It has a 613 kW electric power system and two batteries with a combined capacity of 1200 kWh. The available battery capacity is sufficient to support a full day of piling work, and the rig's peak power is not always necessary during regular piling operations. Therefore, it is estimated that the piling rig will have a continuous piling capacity of 10 hours, resulting in a total energy consumption of 120 kWh. Considering that the rig will be required for 2 hours, the overall energy demand for the piling process amounts to 240 kWh. Regarding the transportation of the piling rig, it is still necessary to employ an EURO6 diesel truck due to weight restrictions. Implementing the electric Woltman 90DRe in Delft results in an ECI reduction of €13.34, which comes down to 54% less environmental impact in the installation stage (A5), as depicted in Figure 4.25. This comes down to a reduction of 3.43% on the total ECI value.

Figure 4.25 illustrates the ECI values per impact category for both the diesel and electric piling machine installations in Zwolle and Delft. It can be concluded that replacing the diesel rig with an electric one results in fewer harmful emissions, particularly in terms of AP, EP, and HT impact categories. What strikes is the relatively high GWP with the electric-generated rigs, which is caused by the generation of grey electricity. Besides the lower ECI value with the electric piling process, the implementation will also result in less local noise and emission hindrance, which is often an essential requirement regarding the piling process. The disadvantages, however, are the additional self-weight during transport and the high investment costs.

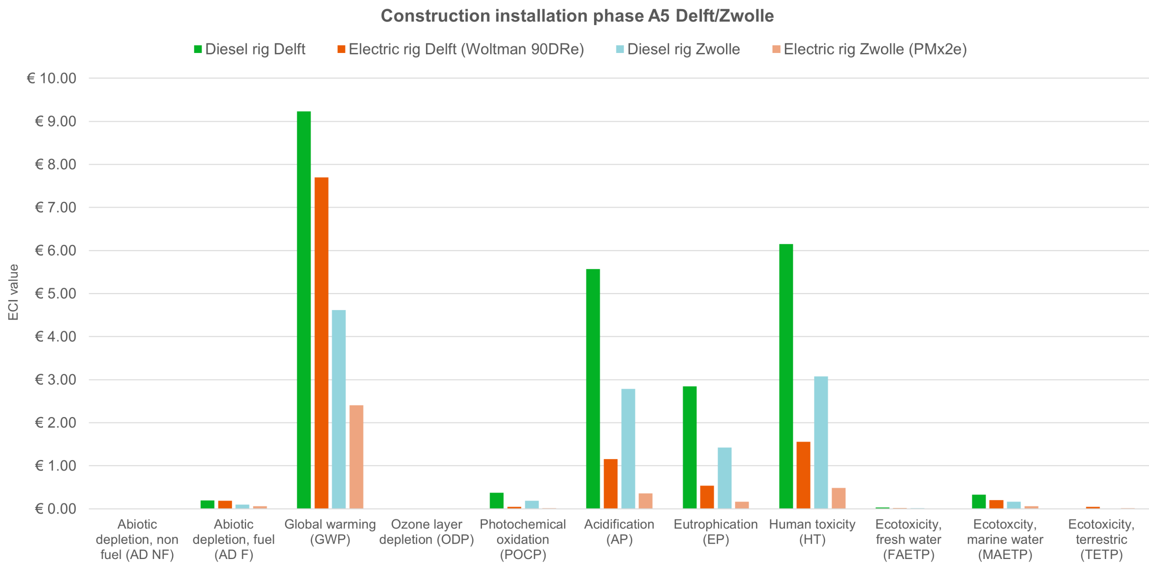


Figure 4.25: ECI value of installation with diesel and electric rig in Zwolle and Delft

4.5.3. Total optimised ECI value representation

With the above-optimised variants design and processes for the foundation in Zwolle and Delft, the total optimised ECI can be displayed. The involved optimisations of the prestressed foundation pile are: Lowering concrete strength class and thereby higher w/c ratio, optimum reinforcement design for the specific length, alternative transport to the building site, and the installation with an electric piling rig.

Location Zwolle

Figure 4.26 shows the ECI reduction per optimisation step in Zwolle. The initial design benchmark consists of C45/55, CEM I with 4 ∅ 7.5 mm prestressed reinforcement, as calculated in subsection 4.4.4. The initial design is transported with EURO6 trucks, and the installation will be performed with diesel piling machines. For Zwolle, it is assumed that the optimum calculated reinforcement cannot be applied due to the minimum available diameter requirement of 7.5 mm. From Figure 4.26 the different optimisations and their effects on the total ECI value can be seen. It can be concluded that the electric piling machine PMx2e has the largest reduction, which is 14.25%. This is caused because of the relatively long piling process of the six foundation piles and its corresponding high environmental impact. The second largest ECI reduction can be achieved by the implementation of CEM III/A 52.5 instead of regular CEM I 52.5R, which results in a 12.75% reduction. The lowering of the concrete class to C35/45 instead of C45/55 results in a 6.6% reduction and the electric truck transport in a 2.54% reduction. The reinforcement and the required energy during the prefab process are not optimised and therefore no lowered value is illustrated in Figure 4.26. Combining all the optimisation options results in a total ECI value of €39.43 instead of €61.74, which comes down to a 36.14% reduction.

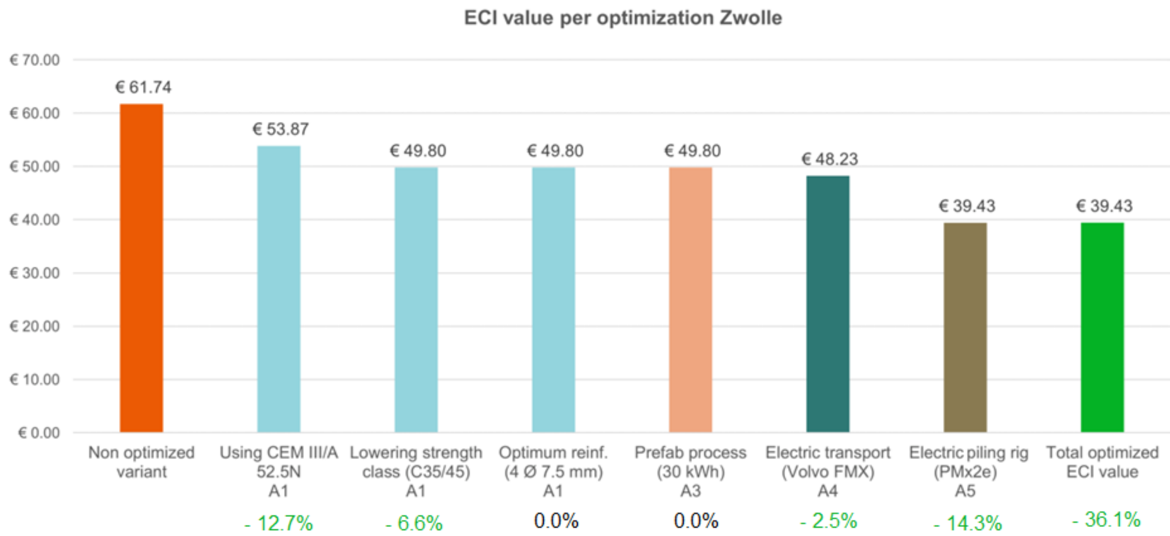


Figure 4.26: ECI reduction Zwolle prestressed pile, per each optimisation

Location Delft

For the Delft location, the same benchmark design as in Zwolle will be employed, as outlined in subsection 4.4.4. However, it is important to note that the application of the lowered concrete strength class C35/45, cannot be implemented independently without increasing the amount of reinforcement, as indicated in Table 4.10. This is caused by the significantly higher critical Unity Check associated with the 23-meter-long piles in Delft compared to the 5-meter-long piles in Zwolle. Therefore, the reduction in the concrete strength class and the optimised reinforcement will be combined. Furthermore, it should be mentioned that if deviations from standard dimensions are not permissible, the configuration of C45/55 with a diameter of 6 ∅ 9.3 exhibits the lowest ECI value, as illustrated in Figure 4.18. Figure 4.27 shows the ECI value reduction of each optimisation option in Delft. Based on the analysis, it can be concluded that the utilisation of Blast Furnace Slag Cement (CEM III 52.5N) has the most significant impact on reducing the ECI, resulting in a total reduction of 16.03%. The electric transport has the second largest impact, which results in a 6.5% reduction. Lowering the concrete strength class with optimum reinforcement in Delft results in a relatively low reduction of 5.62%. The original C45/55 variant, with 6 ∅ 9.3 reinforcement combination, already had a high UC and thereby optimum environmental impact. The implementation of the electric Woltman 90DRe piling machine, instead of a regular diesel-generated piling machine results in a 3.43% reduction.

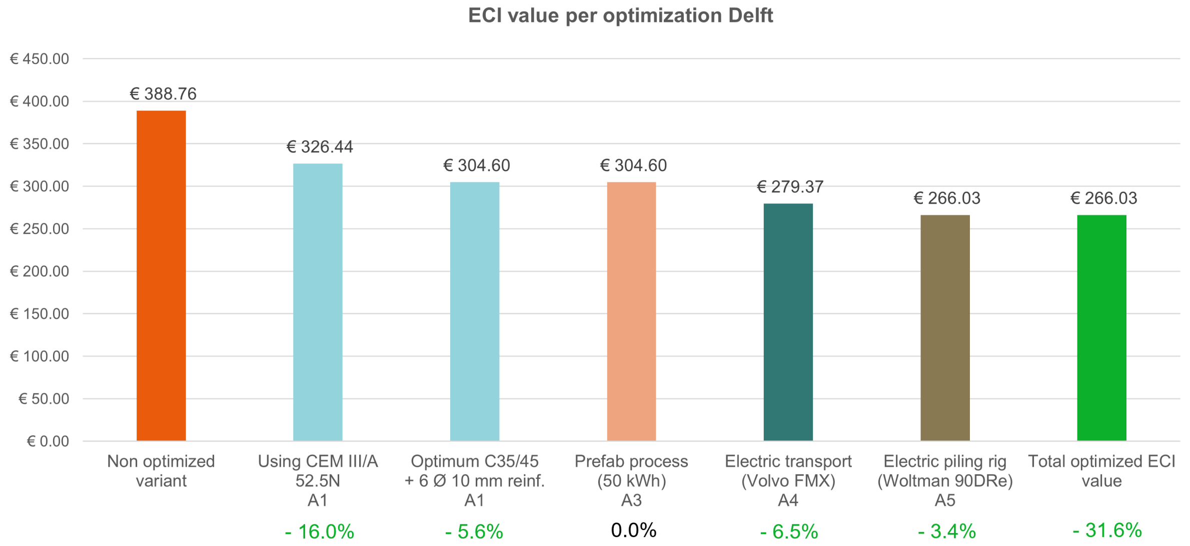


Figure 4.27: ECI reduction Delft prestressed pile, per each optimisation

From Figures 4.26 and 4.27, it can be concluded that, on average, applying CEM III instead of CEM I has the largest ECI reduction for the prestressed prefab foundation pile. Besides that, lowering the concrete strength class and applying an electric piling rig also have some significant impact. However, it also needs to be mentioned that per specific project for the prestressed prefab pile, the maximum ECI reduction variant needs to be investigated. This was also confirmed by the relatively significant difference in optimisation options between Delft and Zwolle. Therefore, there is no one-size-fits-all solution besides implementing CEM III instead of CEM I. However, increasing the formwork time to 2 days instead of 1 day is highly undesirable due to its impact on the process efficiency of prefab manufacturers.

4.6. Conclusion

Chapter 4 addresses the first foundation variant, the prestressed prefab concrete foundation pile, which is widely utilised in the Netherlands and exhibits a high standardisation factor. As a result, the analysis and optimisation of this variant are applicable in numerous cases.

Considering the bearing capacity of both Zwolle and Delft, the required pile lengths are derived. In most cases, the maximum allowed settlements and rotations (STR/GEO) are critical for determining the pile length rather than the net bearing capacity $R_{c;net;d}$ with the given normal force F_{cd}. As a result, for Zwolle, six piles measuring 5 meters in length and 220x220 mm in dimensions are necessary, while for Delft, six 23-meter piles and 290x290 mm in dimensions are required. Subsequently, the structural capacity of these piles is analysed to ascertain the (prestressed) reinforcement design. The critical UC often revolves around the cracking moment capacity during the dynamic lifting and transport processes. The standardised concrete class C45/55 with Y1860 prestressed steel is used for both locations. The structural capacity analysis for standardised foundation designs results in a 4 ∅ 7.5 mm, 3-wire prestressed reinforcement design for Zwolle, which will be prestressed with 3.0 N/mm^2. For Delft, 6 ∅ 9.3 mm, seven-wire prestressed reinforcement is determined, which will be prestressed with 5.0 N/mm^2. These reinforcement designs are derived from commonly applied standardised dimensions and values. Detailed bearing and structural capacity calculations can be found in Appendix C.

From the determined foundation design elements, the environmental impact of both locations is calculated per LCA stage. For the standardised designs, both CEM I and CEM III concrete mixtures are used (Fig.4.11). For the manufacturing process (A3), the standardised 3.63 kWh electricity, 0.12 l diesel, and 0.13 m^3 gas per cubic meter concrete with additional 30 kWh and 50 kWh for respectively Zwolle and Delft are considered. The transport (A4) will be performed with EURO6 trucks over 82 km per single trip, where two trips are required for the Delft piles. The installation (A5) will be performed with diesel piling rigs for one hour straight in Zwolle and two hours in Delft. Considering the above conditions, the total ECI value of the six prestressed prefab foundation piles results in Zwolle of €61.74 and €53.87 for CEM I and CEM III respectively. The cement content is the largest contributor, followed by the diesel piling machine, as seen in Figure 4.9. For Delft, the total ECI is calculated at €388.76 (CEM I) and €326.44 (CEM III). Similarly, in Delft, cement constitutes the largest contributor to ECI, followed by prestressed Y1860 steel and the EURO6 transportation of the piles, as illustrated in Figure 4.9.

With the calculated and analysed ECI values of the standardised dimensions, the pile designs of both Zwolle and Delft are optimised to lower the environmental impact. The first and most effective optimisation is using CEM III instead of CEM I. This results in an ECI reduction of 12.75% for Zwolle and 16.03% for Delft. Using the electric piling rig PMx2e in Zwolle will lower the total ECI value by 14.25% and using the Woltman 90DRe in Delft by 3.4%. Reducing the concrete strength class and implementing an optimum reinforcement configuration will result in a reduction of respectively 6.59% and 5.62% for Zwolle and Delft. Using electric transport for the prestressed piles reduces the environmental impact by 2.54% in Zwolle and 8.28% in Delft. It can be concluded that combining all optimisation options results in a total ECI reduction of 36.14% in Zwolle and 31.57% in Delft, as depicted in Figures 4.26 and 4.27.

5
Shallow strip foundation

This chapter discusses the concrete shallow strip foundation variant on its environmental impact. Section 5.1 will cover the foundation plan and assumptions. The calculations on both the bearing- and structural capacity will be covered in section 5.2. Following this, section 5.3 will present the resulting design and dimensions of the shallow foundation. Section 5.4 discusses the ECI of the designed shallow strip in Zwolle. Moreover, section 5.5 covers the pile optimisation to lower the ECI value. Finally, the conclusion of this chapter will be given in section 5.6.

5.1. Assumptions shallow strip foundation

The second design variant is the shallow concrete strip foundation. A shallow strip foundation is a long strip with limited width, often placed beneath a wall or facade. The strip foundation is analysed due to its potential for material quantity reduction and the high re-use potential (C+D). Due to the risk of erosion by water, it is recommended that the foundation element always be embedded in the ground to the thickness of the concrete slab. Additionally, a deeper construction is necessary for buildings to prevent the risk of foundation freezing. A frost-free installation depth of 0.6 to 0.8 meters is generally accepted in the Netherlands [3]. NEN 9997-1+C2 also stipulates that a shallow foundation may not be laid deeper than five times its smallest transverse dimension. Ground coverage is important if the load on the ground adjacent to the foundation element has a load-bearing effect. It is assumed that before the shallow foundation is (fully) loaded, the excavated soil around will be returned, and the ground coverage will be ensured as given in the CPT. However, to model some ground coverage losses, the first 0.4 m clay layer in Zwolle and Delft will not be considered. This is because the ground coverage has a positive influence on the bearing capacity. The soil behaviour and bearing capacity principles of the shallow foundation are discussed in section 2.2.4 (Prandtl). For the calculation of the bearing capacity and the settlements, the effective width (e_b) and length (e_l) need to be assumed. This computational assumed strip width is used to calculate with a normal centric load instead of the occurring eccentric load (NEN-9997-1). The settlement calculation in D-foundation follows the model described by Boussinsesq.

The loads on the front facade foundation beams and the longitudinal beams differ significantly, as seen in Appendix B. It is assumed that the connection between the front and longitudinal strip elements needs to be rigid to transfer the loads and prevent unequal settlements properly. To achieve a fixed connection between the two beams, a cast-in-situ strip foundation will be considered. Moreover, a cast-in-place strip foundation delivers a higher horizontal capacity than a prefab, as the contact surface with the soil is assumed to be rougher. Another important aspect is that the total weight of FLOW is assumed to be distributed over the four shallow strip foundation elements as mentioned in subsection 3.1.3. This assumption follows the standardised FLOW design, in which the front and longitudinal foundation beams carry significantly different loads. However, the only difference is that in the shallow strip design, the front strip will not be placed on top of the longitudinal strip. The load distribution on the strips is given in Appendix D.2. This results in a total load of 590 kN on the longitudinal and 90 kN on the front foundation strip (ULS). This also includes the self-weight of the soil on top of the strip. These point loads need to be used as the load on the strip foundation in D-foundation. An additional eccentricity of 50 mm is considered for the vertical forces acting on the strip foundation. This results in an effective width of 0.7 meters. Because the shallow foundation needs to be placed at least 0.6 meters below ground level, it is assumed that the first layers of clay will be excavated. The mean GWT from both Delft and Zwolle will be considered for the phreatic level. In Zwolle, this will be -31 cm, and in Delft, it will be -183 cm, as indicated in Appendix A.2. The characteristic value of 1.06 kN/m^2 will be used for the horizontal wind load. Considering the dimensions of FLOW this results in a total horizontal load of 79.6 kN (ULS), divided over the two 9.1 meter longitudinal strip foundations and 41.2 (ULS) kN over the two front foundation strips. It needs to be noticed that the wind forces are simplified, and an extensive wind analysis needs to be performed for a complete verification. Besides that, it is assumed that the wind forces are equally distributed over the two shallow strip foundations. The environmental class of the shallow foundation will be XC2, given that it will be placed in a rarely dry environment. Therefore, the maximum allowed w/c factor will be 0.6, which is important in determining the allowable concrete class and mixture. It is assumed that the minimum concrete strength class for the shallow strip foundation is C20/25. The strip foundation will not be exposed to frost because it will be placed beneath 0.6m with respect to ground level. For the concrete mixtures the same mixtures as with the prestressed prefab variant in Table 4.11 will be used. However, additional mixtures must be determined because of the lower required concrete strength class with the shallow strip foundation. Again, the standard B500B reinforcement steel will be used. Again, the superstructure of FLOW is considered a non-rigid superstructure.

For the design of the shallow strip foundation, it is assumed that the standardised FLOW concrete foundation beams will be placed on top of the strip foundation to enable the transfer of the wall loads towards the strip foundation. However, because the environmental impact of the foundation beams in both the prestressed and timber pile are not considered, only the ECI value of the strip needs to be calculated to achieve a fair comparison between the variants. It may also be argued that the engineered timber wall can continue to the concrete strip. However, it is assumed that the timber can not be placed in the wet soil because of the degradation mechanism and reduction in lifespan. Concluding, only the ECI value of the concrete shallow strip needs to be researched and analysed.

Another important note is that an additional small sand layer in the CPT profile in Zwolle will be implemented. This is because, below the 0.6 m thick sand layer at level -0.48m, there is a 2-meter thick loam layer, as shown in Figure 3.15. Because of the limited bearing capacity and high consolidation, these loam layers result in a high settlement and rotation of the shallow foundation. Without a CPT change, a shallow foundation in Zwolle, without CPT adjustment, is not feasible because of the large settlement. Therefore, to investigate the ECI of a shallow foundation, a small 0.5m sand layer below the loam layer will be implemented. The added sand layer would not or barely influence the design of the pile variant. Therefore, the shallow foundation in Zwolle can still be compared unambiguously with the pile foundations. The adjustment in CPT in Zwolle can be found in Appendix D.1.

Figure 5.1: Cast-in-situ shallow strip foundation process [31]

Figure 5.2: Shallow concrete strip foundation [31]

5.2. Capacity calculations

The first step in calculating the environmental impact of the concrete shallow strip foundation is to investigate the feasibility of a shallow foundation in both Zwolle and Delft and to calculate the dimensions and configurations subsequently. Again, the capacity calculations of the strip foundation can be divided into the bearing and structural capacity. However, the determination of the bearing capacity of a shallow strip foundation differs from the determination of both pile foundation variants. The Prandtl principles of the bearing capacity of the shallow strip foundation can be found in subsection 2.2.4. From the determined foundation width in the bearing capacity analysis, the concrete height and reinforcement of the shallow strip can be determined in the structural analyses.

5.2.1. Foundation plan

The foundation design plan of the shallow foundation design uses the standardised FLOW dimensions. The position of the concrete strip foundation is placed beneath the front- and longitudinal walls. In this way, the shallow foundation can be applied without major adaption of the superstructure of FLOW. The foundation plan has a cast-in-situ foundation strip along both the longitudinal and front sides. However, the front facade strip has a smaller load than the longitudinal strip. However, the width of the entire strip foundation will be similar. The dimensions of the shallow strip foundation are depicted in Figure 5.3. The four shallow strip sides are numbered, where 1 and 2 are the longitudinal strip foundation, and 3 and 4 are the front foundation strips.

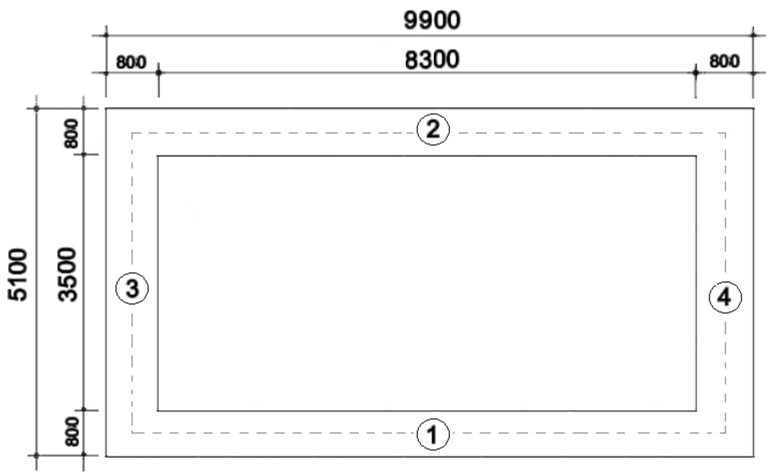


Figure 5.3: Foundation plan of the shallow concrete strip foundation

5.2.2. Bearing capacity shallow strip

The first check is the bearing capacity analysis of the shallow strip foundation. For the bearing capacity, the software D-foundation of Deltares will be utilised. As already mentioned, an additional 0.5m sand layer in Zwolle is assumed, therefore the CPT in Appendix D.2 will be used. The construction sequence will be similar to the prestressed pile variant, which is first conducting the CPT, subsequently, the excavations take place, and finally, the shallow strip will be installed. For the soil characteristics of the shallow foundation design, the values given in Appendix D.1 will be used, which differ from those that need to be considered for the pile foundations. For the bearing capacity of the shallow strip foundation, both the drained and undrained situation needs to be considered. A fully drained condition exists when no water pressure is present. The undrained situation occurs in cohesive layers, where water pressure occurs during and immediately after loading. For this calculation, a calculation must have been made in which the strength of the soil is derived from the undrained shear strength $C_{u;d}$. The angle of internal friction φ should be equal to zero. For a foundation in non-cohesive soil, the drained situation is critical. For a foundation in a cohesive soil, both states should be calculated [41]. The FLOW building is assumed to be placed in a continuous soil area without any slopes or adjacent soil structures. Multiple verifications need to be performed for the bearing capacity of a shallow strip foundation. The first verification is the vertical bearing capacity in the drained and undrained situations. Because the soil profiles in both Zwolle and Delft contain clay and loam, both cases need to be checked. For the principle of the drained situation, the principle of Prandtl can be used. The second check is the punch through of the shallow foundation. If there is a presence of a layer below a stiffer upper layer, whose characteristic value of the angle of internal friction φ'_k deviates more than 6° from that of the layer above it, the failure of punching through needs to be checked. Moreover, if a strip foundation has a l'/b' ratio of >10, and is placed on a cohesive layer, squeezing of the undrained situation must also be considered. Squeezing is a more favourable approach for the undrained bearing capacity (punching), and therefore can only be applied if the cohesive layer is thin enough with respect to the width of the load surface [41]. The fourth verification is the horizontal bearing capacity check. This must be performed to prevent the gliding of the shallow strip foundation due to the horizontal forces. Moreover, a check on the stability of the shallow strip needs to be carried out. The soil below and in front of the shallow strip foundation must stabilise the horizontal soil pressure and loads. The stability can be checked by considering all forces and levers around a critical point. The final checks are the maximum settlement and rotations of the strip foundation. For the maximum allowed settlement in Limit State and Serviceability State, the default values of respectively 0.15 m and 0.05 m are used (NEN 9997). For the maximum allowed relative rotation, the default settings will also be used for ULS and SLS, respectively 0.01 and 0.003.

5.2.3. Structural capacity shallow strip

With the derived 0.8m width of the strip foundation in the bearing capacity analysis, the height, concrete class, and corresponding reinforcement of the strip can be determined. The first step in the structural analysis is determining the occurring forces and moments on the strip foundations. It is assumed that the foundation beams that connect the strip with the FLOW walls are placed centric on top of the strip. The load distribution and the reaction forces on the four shallow strip foundations can be found in Appendix D.2. It can be concluded that the maximum sagging moment is 12.2 kNm, and the maximum hogging moment is 8.1 kNm. Both maximum moments occur on the longitudinal foundation strip. On the front foundation strip, the moment is only 5.0 kNm. The maximum shear force is 30.5 kN. Because the reaction forces on the strip foundation are very limited, a low concrete strength class can be used. It is assumed that the minimum strength class needs to be C20/25. As concluded in chapter 4, a lower concrete class results in a lower ECI value, provided that no additional reinforcement needs to be implemented. The height of the strip needs to be minimised to lower the amount of concrete and thereby cement. Therefore, a total height of 0.2 meters will be considered for the strip. This commonly used height for a strip foundation provides enough capacity for the reaction forces. Because of the limited reaction forces and moments, designing the strip foundation without reinforcement may be possible. Following NEN 1992-1-1 Article 12.3.1, the design tension capacity can be calculated using equation 5.1. The recommended value for $\alpha_{\text{ct,pl}}$ is 0.8, as a result of the decreasing ductility characteristics of the non-reinforced concrete. Another criterion, according to NEN 1992-1-1 Article 12.9.3, for non-reinforced strip foundations, is the conditions in equation 5.2 are met. h_F and a are given in Figure 5.4 and σ_{gd} is the soil pressure, expressed in the same unit as the design tensile capacity of the concrete.

$$f_{ctd,pl} = \alpha_{ct,pl} \cdot f_{ctk;0,05}/y_c \tag{5.1}$$

$$\frac{0.85 \cdot h_F}{a} \geq \sqrt{(3 \cdot \sigma_{gd}/f_{ctd,pl})} \tag{5.2}$$

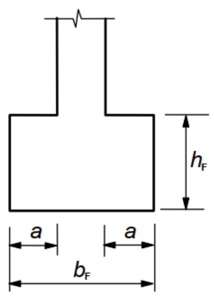


Figure 5.4: Design rule non-reinforced concrete strip foundation

5.3. Design and dimensions

By applying the methodology outlined in Section 5.2 and considering the soil conditions, the design and dimensions for the locations in both Zwolle and Delft can be determined. The soil conditions for Zwolle are provided in Figure 3.15, and for Delft, they are illustrated in Figure 3.16. The foundation plan and the associated reaction forces can be found in Appendix D.2.

5.3.1. Location Zwolle

A foundation strip width of 0.8 meters is determined for the location Zwolle. With the foundation plan depicted in Figure 5.3, the bearing capacity checks can be performed. Table 5.1 shows both the vertical and horizontal bearing capacity of the shallow strip foundation, placed at 0.8 m below ground level. Because the layers in Zwolle also contain loam, both the undrained and drained situation needs to be performed. From Table 5.1, it can be derived that the 0.8 m width satisfies both bearing capacity checks. The squeezing and punch-through do not have to be considered because of the soil conditions and shallow foundation design in Zwolle. Besides that, the stability verification is also performed. With a total internal friction angle of the critical soil layer ($f_{mean,d}$) of 23.7 degrees, both the tip-over stability and total stability are guaranteed.

Strip	Vertical cap undrained		Vertical cap drained		Horizontal cap		
element	V_d [kN]	R_d [kN]	V_d [kN]	R_d [kN]	H_d [kN]	$R_{d,u}$ [kN]	$R_{d,d}$ [kN]
1	611	5243	590	605	40	975	159
2	611	5243	590	605	40	975	159
3	103	2878	90	233	20	502	24
4	103	2878	90	233	20	502	24

Table 5.1: (Un)drained horizontal and vertical bearing capacity verification Zwolle

Besides the bearing capacity and stability check, the settlements and rotations requirements must also be verified. Both the STR/GEO and SLS verification need to be performed. The Dutch standard NEN 9997-1:2016 uses a 20% limit to determine which layers should be considered in the determination of the settlement. Only layers, of which the increase in the effective vertical stress due to the placement of the foundation is larger than 20% of the original effective vertical stress, are considered to have any effect on the settlement. However, Deltares' opinion is that with a 5% limit, a better, more conservative approach can be achieved [22]. Therefore, the 5% limit will be used in the settlement determination. The verification results in the 0.8m width shallow strip foundation in Zwolle can be found in Table 5.2, and all bearing capacity checks satisfy the requirements. It can be concluded that the settlement and rotational checks in SLS are critical verifications.

	ULS STR/GEO				SLS			
Strip element	S_{max} [m]	S_{tot} [m]	ϕ_{max} [-]	ϕ_d [-]	S_{max} [m]	S_{tot} [m]	ϕ_{max} [-]	ϕ_d [-]
1	0.15	0.061	0.01	0.005	0.05	0.038	0.003	0.003
2	0.15	0.061	0.01	0.005	0.05	0.038	0.003	0.003
3	0.15	0.034	0.01	0.005	0.05	0.018	0.003	0.0025
4	0.15	0.034	0.01	0.005	0.05	0.018	0.003	0.0025

Table 5.2: Settlement and rotation checks ULS and SLS Zwolle

With the calculated strip width of 0.8m in Zwolle, the corresponding structural design can be determined. A total foundation strip height of 0.2m is assumed to limit the concrete material. Because all strips have the same width and height, it is desired to have the same amount of reinforcement because of the simplicity during casting. The following shallow strip design is developed from the structural capacity calculation in Appendix D.3. The width and height of the pile remain 800x200 mm, which originates from the bearing capacity of the pile in Zwolle and standardised sizes. As mentioned in section 5.3.1, the concrete class will be C20/25 and the reinforcement steel B500B. The longitudinal reinforcement will consist of four bars at the top and bottom, each with an 8 mm diameter. This makes the total surface of the eight longitudinal bars 402 mm^2. Because the environmental class is XC2 and the concrete will be poured on the equalised soil, the concrete cover will be 40 mm. The maximum shear force acting on the strip foundation is 30.5 kN. The C20/25 concrete with an 800x200 mm cross-section has a total shear force capacity of 55.3 kN, as indicated in Appendix Table D.5. Therefore, no additional stirrups are required for the strip foundation to resist shear forces. However, it is essential to position the longitudinal reinforcement bars correctly. To achieve this, 8 mm stirrups need to be installed at intervals of 500 mm. It is important to note that these stirrups are intended solely to maintain the proper positioning of the longitudinal reinforcement and are not designed to bear the shear forces. The stirrups are anchored according to 9.2.3 of NEN 1992. The design of the 800x200 mm shallow strip foundation in Zwolle is depicted in Figure 5.5. Again, it is important to mention that only the strip is calculated and will be considered in the environmental impact. The wall on top of the strip will not be considered in the ECI calculations because, in the pile variant, the foundation beams are also not considered. However, a wall thickness of 300 mm is considered, which results in two 250 mm wide strips on which the soil will be placed. As listed in Appendix Table D.6, the total amount of reinforcement steel is 120 kg, and the total amount of concrete is 10254 kg.

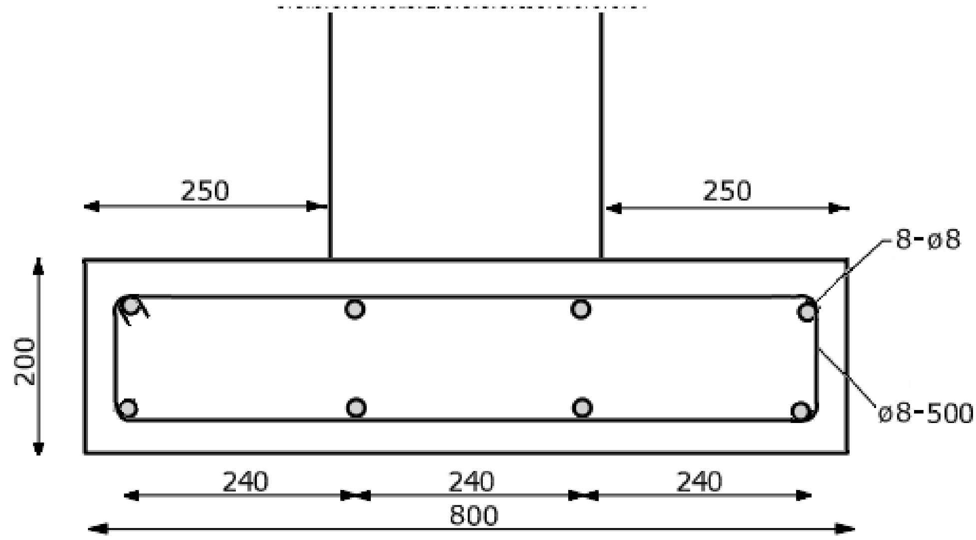


Figure 5.5: Design and dimensions shallow strip foundation in Zwolle

5.3.2. Location Delft

The soil profile in Delft contains a compressible weak clay layer extending to a depth of 7 meters, as shown in Figure 3.16. Attempting to establish a shallow strip foundation on this soil is not feasible. This is primarily due to the significant presence of compressible clay layers, resulting in unacceptably high settlement levels even with potential soil improvement measures. Table 5.3 provides settlement and rotation results for a 1.5-meter-wide strip foundation at 0.8m below ground level. The results indicate that settlement and rotation values largely fail to meet the specified requirements. This is attributed to the extensive weak clay layers beneath the foundation, which are susceptible to compression and consolidation under the applied load. In conclusion, the feasibility of a shallow (strip) foundation in Delft is unattainable due to unfavourable soil conditions characterised by extensive weak clay layers. Therefore, further analysis of shallow foundations will not be pursued in Delft, with the environmental impact assessment focused solely on Zwolle.

Foundation	ULS STR/GEO				SLS			
Strip element	S_{max} [m]	S_{tot} [m]	ϕ_{max} [-]	ϕ_d [-]	S_{max} [m]	S_{tot} [m]	ϕ_{max} [-]	ϕ_d [-]
1	0.15	0.85	0.01	0.076	0.05	0.55	0.003	0.030
2	0.15	0.85	0.01	0.076	0.05	0.55	0.003	0.06
3	0.15	0.46	0.01	0.060	0.05	0.25	0.003	0.06
4	0.15	0.46	0.01	0.060	0.05	0.25	0.003	0.037

Table 5.3: Settlement and rotation checks ULS and SLS Delft

5.4. Environmental impact

In the previous sections, the design of the shallow strip foundation in Zwolle is determined. With that, all foundation elements and their quantities are known, which is required to investigate the environmental impact in detail. In Appendix D.3, the total volume and weights of the concrete with reinforcement are mentioned. The environmental impact of the concrete elements can be calculated with the ”Ontwerptool Groen Beton V6” and the environmental data given in Appendix F. The difference between the prestressed concrete variant is that the shallow strip foundation includes the end-of-life stage (C) and the benefits beyond the building life cycle (D). This is because it is assumed that the shallow strip foundation can be easily removed after service life, without large soil consequences or difficulties, as discussed in section 3.5.

5.4.1. Concrete mixture (A1)

The first step is determining the right concrete mixture for the given lower concrete strength classes C20/25, C25/30, and C30/37. The mixtures C25/30 and C30/37 are also listed because of the non-reinforced concrete optimisation in section 5.5 and thereby required higher concrete class. The concrete mixture can be divided into the following elements: cement, sand, gravel, water, plasticiser, and powdered fly ash. In Table 4.9, the ratios between the elements can be seen for the concrete mixtures. The difference between the mixtures given in Table 4.11 and Table 5.4 is that instead of CEM I 52.5R and CEM III/A 52.5N, the lower cement strength CEM I 42.5N and CEM III/B 42.5N will be used. This is because the w/c requirement of 0.6 for XC2 needs to be followed. The value of 42.5 in both mixtures represents the minimum 28-day strength in MPa. The letter N represents a neutral initial strength of the cement. Because the shallow strip foundation will be cast in situ, the hardening time is less important because the strip will not be fully loaded, and no large moments during transport have to be considered. Again, it must be mentioned that the data is rounded because of confidential reasons from concrete suppliers. It is assumed that fresh tap water will be used for the water element in the mixture. For the sand type, regular concrete coarse sand is used. For the OPC cement, the CEM I 42.5 N (G1), ENCI / HeidelbergCement (Cat.1) data will be used, and for the BFS, the CEM III/B 42.5 N (G8), ENCI / HeidelbergCement (Cat.1) will be utilised. The detailed environmental data of the mixture components are listed in Appendix F.

	CEM I 42.5N			CEM III/B 42.5N		
Elements	**C30/37**	**C25/30**	**C20/25**	**C30/37**	**C25/30**	**C20/25**
CEM I 42.5 N	260	240	220	0	0	0
CEM III/B 42.5 N	0	0	0	280	260	235
Powdered fly ash	90	80	75	0	0	0
Sand (concrete)	700	714	727	680	694	707
Gravel	1100	1122	1142	1200	1222	1244
Water	150	150	150	140	140	140
Plasticizer	2.8	2.8	2.8	1.8	1.8	1.8
Wbf & w/c [-]	0.51	0.55	0.60	0.50	0.54	0.60
Density [kg/m^3]	2303	2309	2317	2302	2317	2328

Table 5.4: Concrete mixtures with different strength classes

5.4.2. Concrete mixture manufacturing (A3)

With the calculated mixtures, the composition of concrete can be created. For the required energy for the composition of the concrete, again the average concrete plan usage of 3.63 kWh electricity, 0.12 l diesel, and 0.13 gas m^3 [10] per cubic meter of concrete mixture will be considered during the A3 manufacturing phase. Because the concrete pouring will occur on-site, it is assumed that no additional electricity is needed for the manufacturing process.

5.4.3. Transport and installation (A4+A5)

Next to the environmental impact of the concrete mixture composition, the impact of the transport and installation processes also needs to be determined. The distance between the concrete plant and the building site will be the same to achieve a fair comparison between all the foundation variants. This results in a distance of 82 km from the concrete plant towards Zwolle. The transport of the concrete plant towards Zwolle will be performed with a mean truck mixer, as discussed in section 4.4.3. The drum in the truck can rotate 360 degrees, which mixes the composition and prevents the hardening of the concrete. Besides the 82 km trip towards Zwolle, an empty return trip will also be considered. The total concrete volume for the shallow foundation in Zwolle is 4.27 m^3.

A significant advantage of using a cast-in-situ strip foundation is the absence of the need for heavy piling rigs during installation. In this approach, concrete is poured directly onto the soil surface, and the casing is typically constructed using recycled timber. As a result, the environmental analysis does not consider the casing material's impact. Furthermore, the casing material can be reused multiple times, similar to the prestressed variant, which spreads the environmental impact across its numerous uses. The concrete pouring process employs a medium-sized pump that evenly distributes the concrete within the shallow strip casing. Subsequently, a compaction needle is utilised to compact the concrete. This compaction process helps eliminate entrapped air and voids, ultimately leading to high-quality concrete with the desired strength and durability characteristics. It is worth noting that the compaction needle requires 0.33 kWh per cubic meter of concrete, and the electricity used for both the needle and the pump is assumed to be sourced from grey electricity.

5.4.4. End of Life stage (C)

The end-of-life stage (C1-C4) includes the required processes after the end of the life of the shallow foundation. The first stage is the de-construction and demolition of the shallow foundation. The default values for breaking relatively small concrete elements with reinforcement will be used to demolish the cast-in-situ strip reinforcement. In this way, the strips will be broken into smaller, and therefore more convenient for the transport (C1). Besides that, small soil work, including the excavation, will be considered. For the transport of the concrete elements, a standardised EURO 6 diesel truck will be used (C2). The NMD default value of 50 km will be considered for the transport distance towards the waste processor. Besides that, an empty trip towards Zwolle of 50 km is assumed. For the waste processing and disposal, it is assumed that 5% of the reinforcement will be disposed, and 1% of the concrete will be disposed (C3+C4). The default environmental values of Betonhuis for both the concrete and reinforcement will be utilised in the LCA calculation.

5.4.5. Beyond building life cycle (D)

The beyond building life cycle phase includes the benefits of the supplementary beyond life cycle and includes the reuse, recovery, and recycling of all the used materials. The LCA stage D is an offset of the so-called "raw material equivalent" in the Determination Method 3.0 of the NMD. For concrete, this means that the environmental impact of the primary material is replaced by concrete granulation because of the technical similarity. In the Netherlands, the granulate is often used for road foundations. In the road foundation, the granulate will function as a sand-cement stabilisation. Based on available research reports, Betonhuis conservatively assumes that 3% of the binder used in module A is allocated to the cement component in cement concrete production [10]. Moreover, it is assumed that 95% of the granulate will be used in road foundations, 4% will be used as gravel in new concrete, and 1% of the granulate will be disposed of. The default values of the "Ontwerptool Groenbeton" will be used for the reinforcement steel. These default values assumed that 5% of the reinforcement steel will be disposed and that 95% of the ferrous metals will be reclaimed. Also, the default environmental values of Betonhuis will be used for the reinforcement steel.

5.4.6. Representations of ECI value by category

By utilising the shallow strip foundation design calculations outlined in section 5.2.3, along with the aforementioned transport, installation, end-of-life, and beyond-life processes, the environmental impact of the strip in Zwolle can be displayed. Due to the two concrete mixtures, given in Table 5.4, a distinction can be made between OPC (CEM I) and BFC (CEM III) concrete. As mentioned in section 5.2.3 only the environmental impact of Zwolle will be analysed and optimised because a shallow strip will not be feasible in Delft. The environmental data of the different elements and processes are noticed in Appendix F.

For the 200x800 mm C20/25 strip foundation in Zwolle, the total ECI values are determined to be €88.65 and €70.83 for CEM I 42.5N and CEM III/B 42.5N, respectively. The ECI contribution per impact category of both concrete mixtures is also illustrated in Figure 5.6. The ECI value translates to an MPG value of 0.0073 for OPC and 0.0059 $€/m^2{*}y$ for BFS, as calculated using equation 2.1. It can be inferred from Figure 5.6 that the GWP has the highest ECI contribution for both CEM I and CEM III concrete mixtures. Besides that, the HT, AP, and EP significantly contribute to the foundation design's environmental impact. The AP and EP contributions are especially caused by cement production. The HT is primarily caused by the production of reinforcement steel. What also strikes is the relatively large reduction in GWP by applying CEM III/B 42.5N instead of CEM I 42.5N in the strip foundation. This is caused by the relatively large required concrete and the high blast furnace slag ratio in the CEM III/B mixture. Implementing CEM III instead of CEM I in the strip foundation results in a total ECI reduction of 20.1%, which comes down to €17.82.

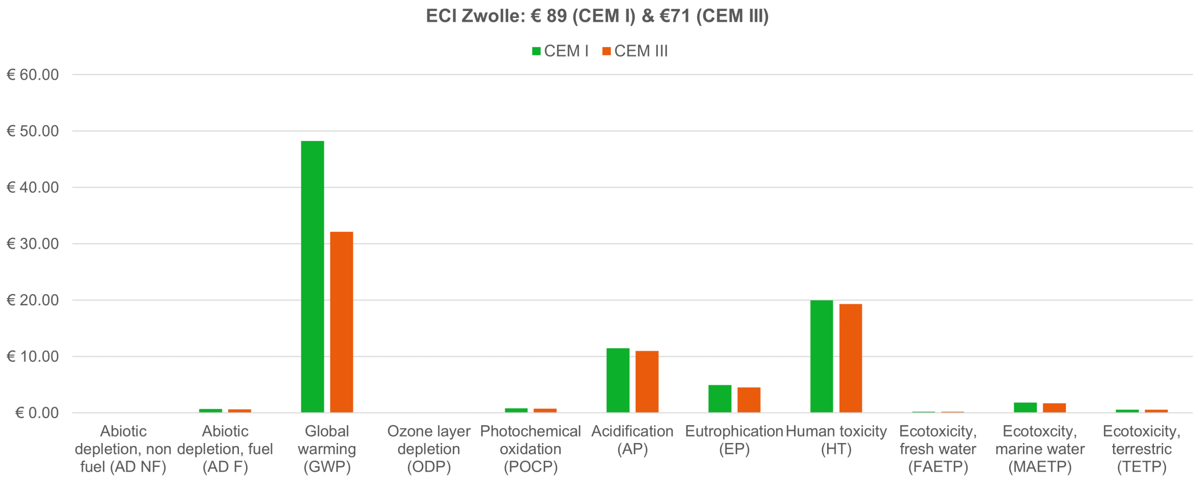


Figure 5.6: ECI value strip foundation Zwolle per impact category CEM I/III

Figure 5.7 shows the environmental impact per LCA stage in Zwolle. The raw materials supply stage (A1) has by far the greatest ECI contribution for both the CEM I and CEM III mixtures. This is again caused by the large environmental impact of the production of cement and reinforcement steel. This can also be confirmed by Figure 5.8, where the cement and reinforcement steel are the two largest environmental contributing elements. The 82 km truck mixer transport of the concrete mixture towards the building site in Zwolle also has a significant contribution and is responsible for €10.99.

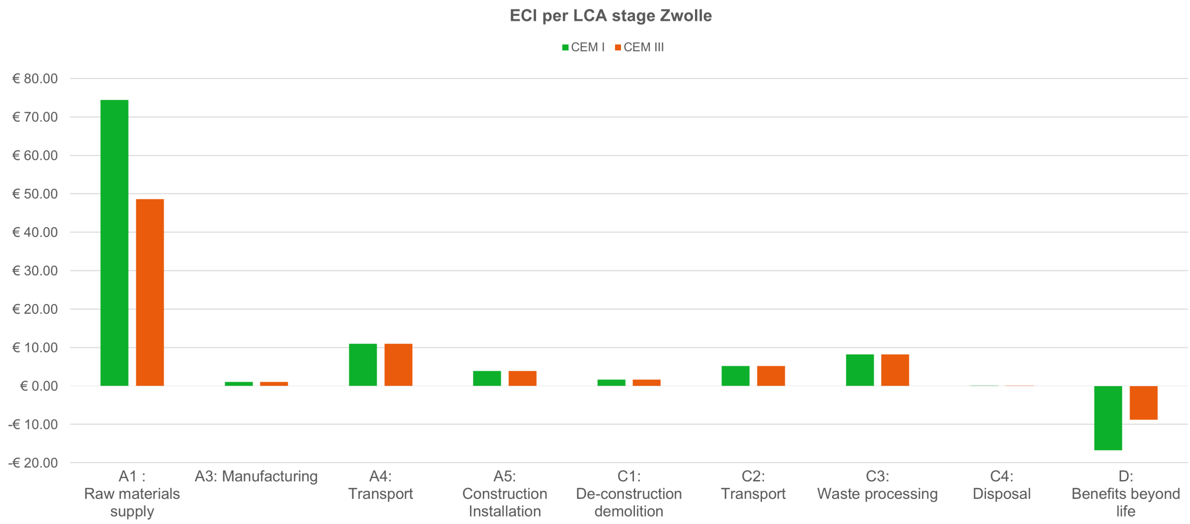


Figure 5.7: ECI value per LCA stage strip Zwolle

From Figure 5.7 it can also be concluded that because of the cast-in-situ strip foundation, the construction and installation phase (A5) has a minor contribution to the ECI value. This is because only a small concrete pump and compaction needle are needed during the construction of the shallow foundation. The concrete's deconstruction and demolition process (C1) also exhibits a low ECI value because it involves breaking the concrete into easily transportable pieces without the need for heavy machinery, owing to the low strength of the concrete used. However, the subsequent transport (C2) of the broken concrete pieces with reinforcement contributes significantly to the ECI value due to the high self-weight of the broken concrete and emissions from EURO 6 diesel trucks. After transportation to the waste processing facility, the concrete undergoes crushing, sieving, and cleaning to produce granulate suitable for reuse (C3). This stage entails using heavy machinery and energy, resulting in a significant ECI value, as indicated in Figure 5.7. Notably, only 1% of the concrete and 5% of the reinforcement steel end up as waste, resulting in a minimal ECI value for the disposal stage (C4), as evident in Figure 5.7.

Furthermore, recycling beyond the life cycle (D) is considered, which results in a negative ECI value of -€16.74 for CEM I and -€8.79 for CEM III. The primary environmental benefit arises from using granulate as a replacement for sand in road foundations. When considering the after-service life stages (C) and the beyond-life cycle stage (D), the net environmental impact is -€1.66 for the CEM I mixture and €6.29 for the CEM III mixture. This difference is due to the larger negative value of the raw material equivalent for the CEM I mixture compared to the CEM III equivalent. Considering the after-service life stages (C) and the beyond life cycle stage (D), the net environmental impact for the CEM I mixture is -€1.66 and €6.29 for the CEM III mixture. This is because the raw material equivalent for the CEM I mixture has a larger negative value than the CEM III equivalent. Therefore, it can be concluded that considering the life cycle stage C and D results in a slightly lower ECI value with the CEM I option and a higher value with the CEM III variant. However, optimising the transport, which has a relatively large share in the after-service life stage, the C and D phases may have a negative ECI value for both the CEM I and CEM III mixtures. In figure 5.8, the element contributions to the ECI value can be seen for both the CEM I 42.5N and CEM III/B 42.5N compositions. It can be concluded that in both the CEM I and CEM III mixtures, the cement has the highest contribution with a value of respectively €42.65 and €23.26. This high value results from the relatively large amount of required concrete because of the 0.8 x 0.2m strip dimension over the entire circumference. Additionally, the 122 kg of required steel reinforcement in both mixtures accounts for €23.12. Therefore, it would be worthwhile to explore the possibility of performing the strip foundation without reinforcement and finding an optimal balance between concrete and reinforcement to minimise the ECI value.

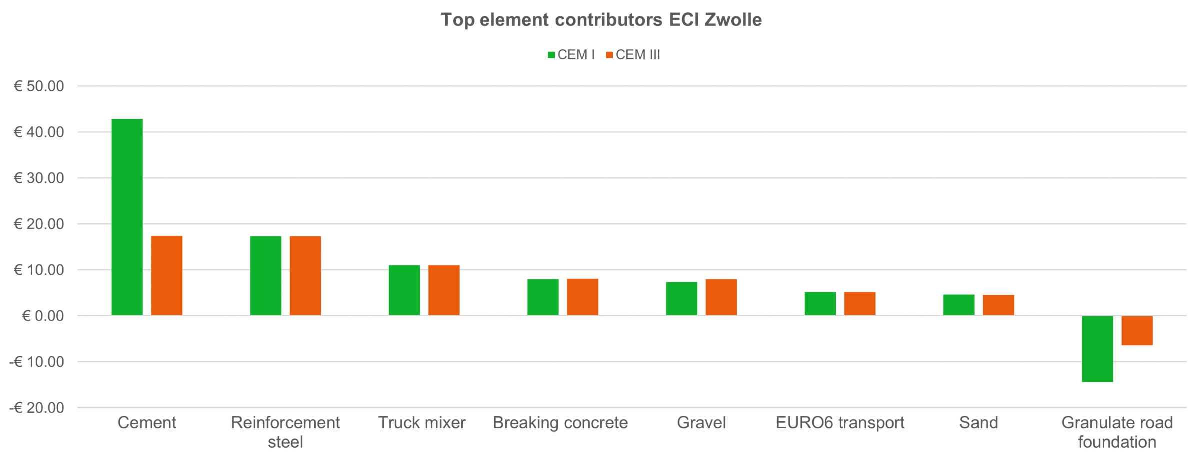


Figure 5.8: Top elements contribution ECI strip Zwolle

5.5. ECI optimisation

From the calculated and analysed environmental impact in section 5.4, the foundation design in Zwolle can be optimised to lower the ECI value. The first optimisation is using CEM III/B 42.5N instead of CEM I 42.5N and CEM I 42.5N, as seen in subsection 5.4.6. This optimisation results in a 20.1% ECI reduction because of the more environmentally friendly BFS instead of OPC. As concluded from section 5.4, the cement and reinforcement steel have the largest ECI. Therefore, a non-reinforced strip foundation will be designed to investigate the environmental savings subsequently. Besides that, it will be researched if an increasing strip height or increasing concrete class will be the most ECI-effective solution for the non-reinforced variant. Again, the alternative transport of the truck mixer and the EURO 6 diesel truck will be considered. Because of the limited data and time constraints of this thesis, optimisation on the beyond of life cycle (D) of the concrete and steel will not be performed. Because the hardening time of the concrete in the cast-in-situ variants is less important, the optimisation options will only be performed on the CEM III design variants and not on the CEM I.

5.5.1. Non reinforced strip foundation

The first optimisation of the CEM III, C20/25 strip foundation is to investigate the environmental impact of a non-reinforced design. As concluded in section 5.4.6, the CEM III cement and B500B reinforcement steel have a similar ECI. However, it must be mentioned that the C20/25, 4.27 m^3 concrete mixture, including sand and gravel, has a larger ECI value than the 121 kg reinforcement steel. Consequently, it is important to minimise the amount of concrete when removing reinforcement steel because of the higher ECI value of cement. For the design of the non-reinforced, the 800 mm strip width can not be adjusted because of the required bearing capacity of the strip. Therefore, only the height and concrete class of the strip can be adapted to achieve a non-reinforced variant. According to NEN 1992-1-1 Article 12.3.1, the tension capacity of the concrete needs to be reduced because of the decreasing ductility properties of the non-reinforced concrete. The design tension capacity can be calculated by using equation 5.1. This calculation incorporates various factors, including the 5th percentile tension capacity, a reduction factor of 0.8, and a material factor of 1.5 for concrete. The maximum designed sagging moment on the strip foundation is computed as 12.2 kNm, and the maximum hogging moment is calculated as 8.1 kNm, as detailed in Appendix D.2. Therefore, the maximum design moment of 12.2 kNm needs to be considered. The check that needs to be performed is to ensure that the tension capacity exceeds the occurring tension stress to prevent concrete cracking. Besides that, the shear force and capacity of the non-reinforced cross-section must be calculated. Moreover, according to the NEN 1992-1-1 Article 12.9.3, the following requirements, given in equation 5.3 must to fulfilled.

$$\frac{0.85 \cdot h_F}{a} \geq \sqrt{(3 \cdot \sigma_{gd}/f_{ctd,pl})} \tag{5.3}$$

With the given maximum moment of 12.2 kNm and the 800 mm strip width, the required height of the strip can be calculated for the given strength classes to full fill the capacity and dimension requirements. Table 5.5 shows the required height of the given concrete class to satisfy the cracking moment and the dimension checks. The calculations can be found in Appendix D.7. Because the calculated minimum required heights $H_{F,min}$ are not common sizes, it is rounded up to the more standard dimension H_F. It can be concluded increasing the strength class results in a roughly 25 mm decreasing height of the strip. However, the increasing strength classes C25/30 and C30/37 also have a higher w/c factor and therefore a high ECI value per m^3 as seen in Table 5.4. The also counts for the CEM III/A 52.5N C35/45, C40/50 and C45/55 mixtures. The design dimension requirement given in equation 5.3 are

	f_{td} [MPa]	a [mm]	h_{min} [mm]	h_F [mm]	σ_{ed} [MPa]	UC	Eq. 5.3	V[m^3]
C20/25	0.83	250	333	350	0.75	0.91	1.13 ≥ 0.26	7.50
C25/30	0.96	250	309	325	0.89	0.93	1.05 ≥ 0.24	6.97
C30/37	1.08	250	291	300	1.02	0.94	0.99 ≥ 0.22	6.43
C35/45	1.20	250	276	280	1.17	0.97	0.94 ≥ 0.21	6.00
C40/50	1.31	250	264	270	1.30	0.99	0.90 ≥ 0.20	5.79
C45/55	1.42	250	254	260	1.41	0.99	0.86 ≥ 0.19	5.57

Table 5.5: Non-reinforced strip foundation checks, various concrete classes

also verified with the calculated height H_F and are given in Table 5.5. With the previously calculated non-reinforced strip foundations, the ECI values for the three different variants can be determined. The total required volume of concrete for the different strength classes is calculated and can be noticed in Table 5.5. Utilising the specified mixture proportions provided in Table 5.4 and these volume quantities, the environmental impact of the non-reinforced variants can be assessed. It is important to note that the assumptions outlined in Section 5.4 remain unchanged. The critical difference lies in the absence of reinforcement steel and the increased quantity of used concrete. It also needs to be highlighted that the C20/25 to C30/37 has CEM III/B 42.5N as cement and that the higher C35/45 to C45/55 has CEM III/A 52.5N as cement. This is important because the ECI value of the CEM III/B is much lower than CEM III/A because of the larger BFS share in the mixture, as highlighted in Appendix F. Figure 5.9 presents the ECI values for the six concrete strength class non-reinforced strip foundation. Surprisingly, Figure 5.9 shows that the non-reinforced C30/37 variant has the lowest ECI value, of €86.78. This is due to the optimum balance between the relatively low CEM III/B 42.5N cement impact and the low 300 mm required concrete strip height. The C20/25 and C25/30 variants have a height of respectively 325 mm and 350 mm, which results in 0.5 m^3 additional concrete. This additional required concrete has a larger increasing ECI value than the saving of the decreasing CEM III/NB 42.5N quantity. The additional required concrete results in a higher environmental impact during transport and breaking-down processes. Therefore, having a C30/37 CEM III/B mixture is more environmentally friendly than a C20/25.

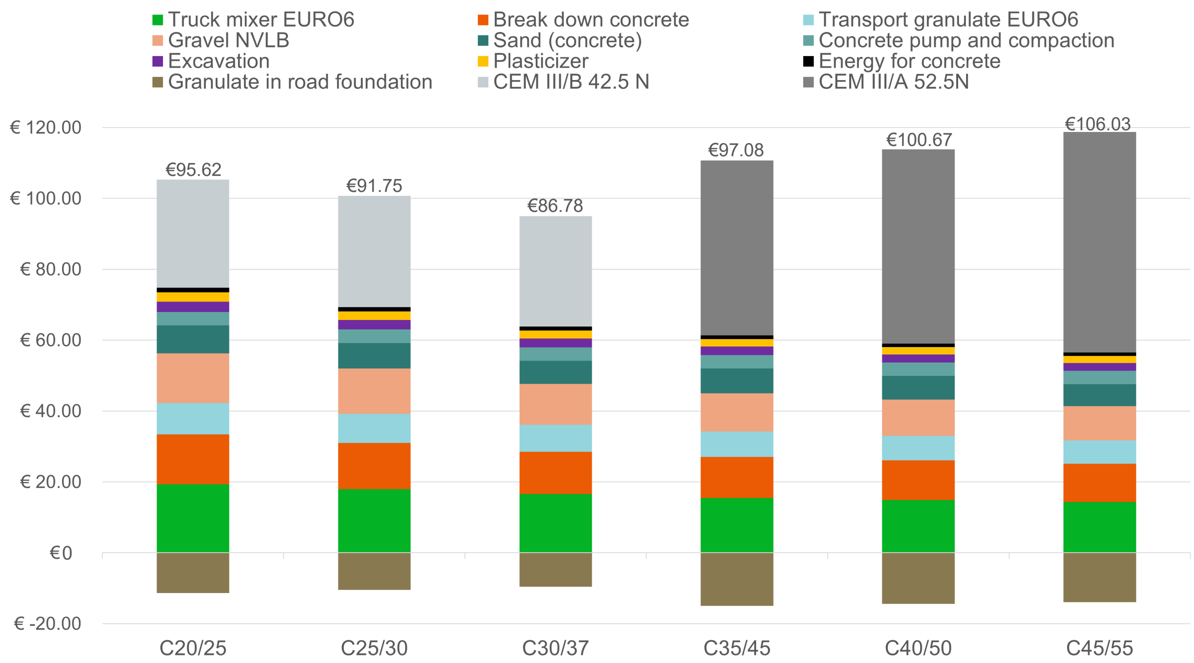


Figure 5.9: ECI values non-reinforced strip foundation

However, increasing the non-reinforced strip C30/37 to a higher concrete class is ineffective because of the significantly higher ECI value of the required CEM III/A 52.5 N in the concrete mixture. This can also be noticed in Figure 5.10, where all the LCA stages have a decreasing ECI value, with an increasing concrete class, except starting from the C35/45, where the ECI value of the Raw materials supply (A1) have a significant higher ECI value. The ECI value increases with the C35/45, C40/50, and C45/55 because the height reduction is limited (280, 270, and 260 mm), and the amount of CEM III/A 52.5 N in the mixture increases significantly (265, 305 and 360 kg/m^3) as can be seen in Table 4.11. Therefore, only considering the CEM III/A 52.5N concrete mixtures, the lowest concrete strength class C35/45 results in the lowest environmental impact. It can be concluded that the non-reinforced strip variants have a higher ECI value compared to the (minimum) C20/25 reinforced variants. The C20/25 CEM III/B, 800x200 mm, 8 $\varnothing$ 8mm strip foundation has a total ECI of €70.83, while the most effective non-reinforced C30/37 CEM III/B, 800 x 300 mm has a total ECI value of €86.78. Besides the increasing environmental impact, the structural capacity of the non-reinforced strip is also significantly lower, which increases the construction risk. However, if the maximum moment on the strip foundation were below 4.4 kNm, the 800x200 mm C20/25 would also be performed without reinforcement, which would have resulted in a lower ECI value. However, the sagging and hogging moments are too high to install a non-reinforced strip foundation without increasing the height. Because the non-reinforced strip foundation design did not decrease the ECI value, the reinforced C20/25 CEM III/B 42.5 N design will be used further in the optimisation.

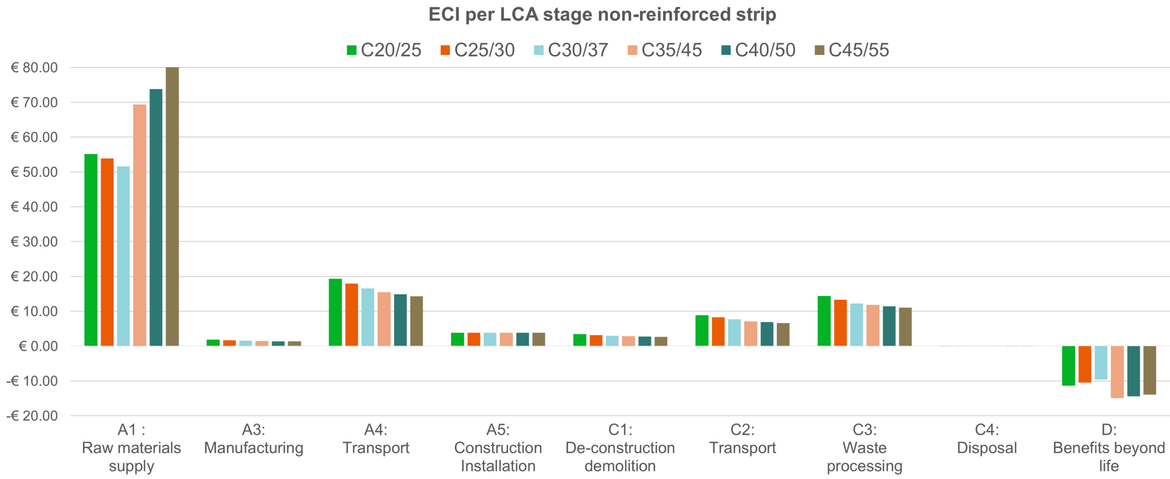


Figure 5.10: ECI per LCA stage non-reinforced strip foundation

5.5.2. Optimised transport

The 82 km diesel truck mixer transport of the C20/25 concrete mixture towards Zwolle has an ECI value of €10.90, while the 50 km EURO6 diesel truck transportation of the concrete pieces towards the waste processor has ECI value of €5.25 (Fig. 5.7 and Fig. 5.8). Both transports are accountable for €16.15, which comes down to 22.80% on the total ECI value. Therefore, it is interesting to investigate the effectiveness of the alternative transport of both the truck mixer towards Zwolle and the transport truck towards the waste processor. For the 50 km transport of the broken concrete towards the waste processor, the same four transport alternatives will be considered as with the prestressed variant. These are the first and second-generation biodiesel, a hydrogen truck, and an electric truck as discussed in subsection 4.5. The maximum combined capacity of the alternative trucks is 50 tons. Considering a truck weight of 27 tons (electric), the total maximum transportable weight is 23 tons. The C20/25 reinforced strip foundation weighs in total 10375 kg. Therefore, a single trip will satisfy, given the 50-ton weight restrictions. Because of alternative truck mixers EPDs, it is assumed that the ratios between

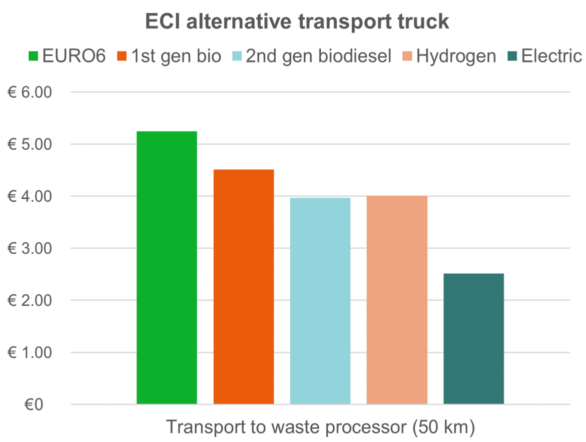


Figure 5.11: ECI value Zwolle alternative truck mixer

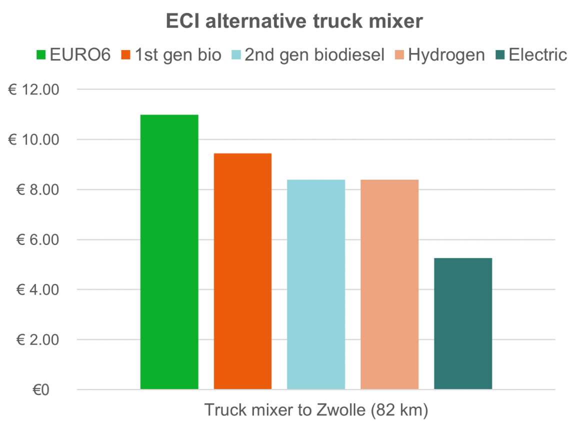


Figure 5.12: ECI value Zwolle alternative truck transport

the diesel truck mixer and the alternative options are similar to the EURO6 diesel truck with the alternatives. Figure 5.11 and 5.12 shows the alternative transport of both the truck mixer towards Zwolle and the truck towards the waste processor. Implementing an electric truck mixer and truck transport can reduce the ECI value by €8.47.

5.5.3. Total optimised ECI value representation

Considering the electric transport towards the waste processor, the entire End of Life (C) and Beyond Life (D) stages have a net ECI value of -€4.33 for the CEM I variant and +€3.62 for the CEM III variant. This is caused by the higher raw material equivalent (D) of the OPC than for the BFS. Therefore, the beyond life cycle of the CEM I mixture is higher, which results in a net positive environmental impact. For the CEM III/B mixture, the End-of-Life and Beyond-life stages have a negative environmental impact, because of the limited raw material equivalent and the relatively high transport and waste processing ECI values. Because the C and D LCA stages are not considered in the prestressed and timber pile variants, the €3.62 environmental impact of the C and D phase will not be considered in the variant comparison. In this way, recycling the shallow foundation will not result in a negative environmental value. Therefore, it can be concluded that recycling the shallow strip foundation results in a higher environmental impact than leaving the entire strip foundation in the soil.

For optimising the shallow strip foundation, only the implementation of CEM III/B 42.5 N instead of CEM I/A 52.5R and the electric transports result in a lower environmental impact. It is assumed that the minimum required C20/25 concrete class and the 8 ∅ 8 mm reinforcement can not be lowered. The optimisation process has revealed that a 200x800 mm strip foundation with the minimum required reinforcement already attains the lowest achievable ECI value of €70.83. The examination of the non-reinforced strip foundations, as detailed in subsection 5.5.1, demonstrates that they result in higher ECIs due to their increased concrete volume. When applying the higher concrete class, C30/37 (CEM III/B), with an optimal non-reinforced strip height, the ECI increases to €86.78. This non-reinforced option exhibits a 22.5% ECI rise and therefore is not a viable consideration for optimisation. Figure 5.13 illustrates the total ECI value of the optimised shallow strip foundation. As mentioned, including the C and D phases resulted in an ECI increase and therefore will not be further included in the ECI results because of the unambiguous comparison between the pile variants, which will stay in the soil. Figure 5.13 also highlights that all optimisation strategies yield a notable 29.66% reduction in the total ECI, bringing it down to €62.36. The €58.74 ECI with the non-considered phases C and D will be used for subsequent comparisons and analysis.

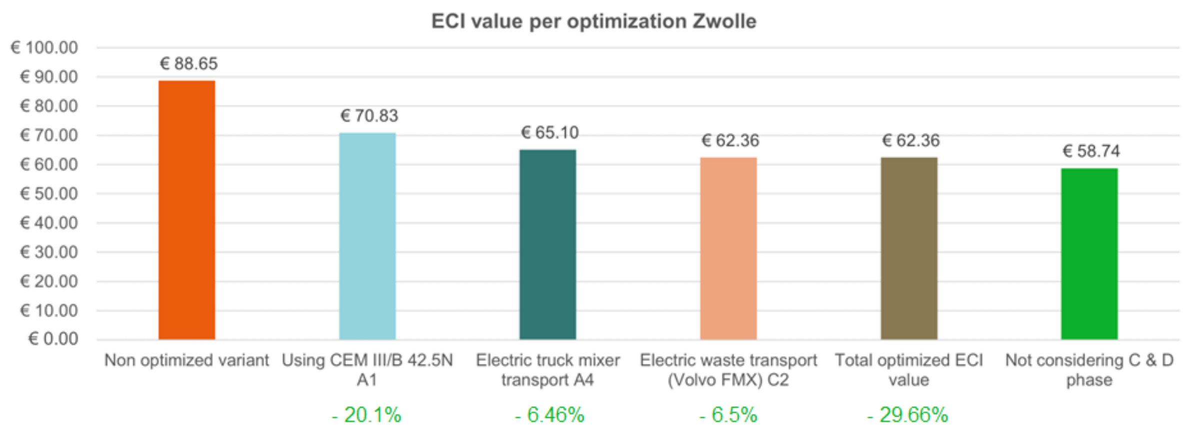


Figure 5.13: ECI reduction Zwolle strip foundation, per each optimisation

5.6. Conclusion

Chapter 5 addresses the second foundation variant, the cast-in-situ shallow strip foundation, chosen because of the possible material saving and re-use potential. The shallow foundation is often used for lightweight structures in the stronger soil parts of the Netherlands.

Considering the bearing capacity of the shallow foundation, the strip's width is determined, resulting in a strip width of 800 mm in Zwolle. For Delft, even considering a 1500 mm strip, a shallow strip foundation will not be feasible due to the unfavourable soil conditions characterised by the weak clay layers and, thereby, unacceptable settlements. Therefore, only in Zwolle shallow strip foundation is analysed. Subsequently, the height and reinforcement are determined by the structural capacity of the strip. Considering the imposed constraints of a maximum shear force of 30.5 kN and limited positive (+12.2 kNm) and negative (-8.1 kNm) moments applied to the strip, it is evident that a low concrete class, combined with a minimal amount of longitudinal reinforcement and stirrups is sufficient. This resulted in a C20/25, 800 x 200 mm strip with B500B 8 $\varnothing$ 8 mm longitudinal and $\varnothing$ 8 mm stirrup reinforcement design in Zwolle, as depicted in Figure 5.5. It is assumed that the strip must have a minimum height of 200 mm due to workability constraints. Additionally, the longitudinal and stirrup reinforcements have a minimum diameter of 8 mm. Detailed force analysis and calculations for the strip can be found in Appendix D.2.

From the determined strip foundation design, the environmental impact of the shallow strip foundation in Zwolle is calculated and analysed. Because of the limited forces, a C20/25 with CEM I 42.5N and CEM III/B 42.5N cement are used for the concrete mixture of the strip. For the manufacturing process (A3), the standardised 3.63 kWh electricity, 0.12 l diesel, and 0.13 m^3 gas per cubic meter concrete are used. The 82km transport (A4) will be performed with a mean-sized diesel truck mixer. For the installation and construction phase (A5), only a concrete pump with a compaction needle is utilised instead of a heavy piling rig. Because the shallow foundation can be relatively easily removed after service life, the End of Life (C) and Beyond Life (D) are included. The deconstruction of the concrete strip will be broken into smaller and transportable sizes (C1), after which the concrete debris is transported with an EURO6 diesel truck over 50 km towards the waste processor (C2). For both the waste processing and disposal, it is assumed that 5% of the reinforcement will be disposed and 1% of the concrete needs to be disposed (C3+C4). Finally, the concrete will be re-used as sand-cement stabilisation in road foundations (D). Considering these conditions, the total ECI value of the shallow strip foundation in Zwolle, including C and D, results in €89 (CEM I 42.5N) and €71 (CEM III/B 42.5N). The cement and reinforcement steel have the highest environmental impact as illustrated in Figure 5.8.

Due to limited reaction forces, a non-reinforced strip foundation is investigated. However, the limited design tension capacity of the concrete resulted in a significant height increase of the non-reinforced cross-section, leading to a higher ECI value than the reinforced design. Even with the optimal non-reinforced C30/37 variant, the ECI value (€87, CEM III/B) remained higher than that of the reinforced C20/25 strip variant (Figure 5.9). The utilisation of an electric truck mixer for transportation to Zwolle and electric truck transport to the waste processor resulted in an ECI reduction of €8.50, representing a 12.49% reduction. Additionally, the end-of-life and beyond-life stages yielded a net ECI value of €3.62, indicating a negative environmental impact. However, since these stages are not included in the pile variants, the €3.62 will not be considered in the comparison to ensure a fair and unambiguous assessment. Therefore, the optimised C20/25 concrete strip foundation in Zwolle results in an ECI value of €59, as shown in Figure 5.13.

6 Timber pile foundation

This chapter analyses the timber pile foundation variant and its environmental impact. The first section 6.1 will cover the foundation plan and the assumptions about the timber foundation piles, including degradation. Capacity calculations will be covered in section 6.2. The resulting design and dimensions of the foundation follow in section 6.3. Section 6.4 analyses the ECI of the designed foundation piles for both Zwolle and Delft. Moreover, section 6.5 cover the pile optimisation to lower the ECI value. Finally, the conclusion of this chapter will be given in section 6.6.

6.1. Assumptions timber foundation pile

until 1925, all Dutch foundations were made of wood, and the number of houses standing on wooden piles at that time was about 425,000. Between 1925 and 1950, most foundations consisted of wooden piles with a concrete upper part. After 1950, concrete piles became common and gradually substituted the wooden piles [57]. This is because concrete foundation piles have a higher capacity than timber piles. Timber foundation piles are often applied for temporary and lightweight structures nowadays. Especially because of the low environmental impact of timber in comparison with concrete and steel. Not all timber types are suitable for pile foundations. The following types apply to foundation piles: Spruce, Larch, Douglas and Azobe hardwood. Pine is not used (anymore) because of the large proportion of sapwood [63]. Azobe hardwood will not be used in this research because it originates from Africa, so overseas transport is required. Because the transport phase of the product (A2) will not be included in this research, Azobe will have a significantly larger environmental impact than calculated without the A2 phase. For this research, C18 spruce will be used because it is often applied as a foundation and because of the availability of environmental data of the spruce timber. Table 6.1 shows the pile characteristics of commonly used timber piles derived from different manufacturers [3].

Diameter point (tip) [mm]	Circumference point [mm]	Color mark	Maximum length [m]
110	340 - 400	Red	23.0
130	400 - 430	Green	22.0
140	430 - 460	Blue	20.0
150	460 - 490	Yellow	20.0
200	> 490	White	22.0

Table 6.1: Characteristic values of timber pile [3]

Another critical aspect of the foundation process is the potential for pile failure during the dynamic piling process, mainly when dealing with solid sand layers. The likelihood of splitting or cracks occurring in the foundation pile is significant in such conditions. Therefore, it may be necessary to conduct additional research on the dynamic forces and soil resistance acting on the timber pile. One possible solution is to reduce the drop height of the diesel piling ram, which would subsequently decrease the dynamic forces applied to the timber foundation piles. Additionally, the concrete cap incorporates spiral reinforcement to transfer the concentrated forces during the piling process effectively. However, due to time constraints and the complexity of researching the dynamic forces involved in the piling processes, an investigation into timber pile failure is beyond the scope of this research.

6.1.1. Degradation and pile cap

An important note in the design of a timber foundation pile is the lowering of the groundwater level. This can severely affect timber foundation piles, as can be seen in Figure 6.1. If the timber piles are located above the water table, oxygen and fungi can enter the timber, and fungi-related deterioration can develop. If the timber piles are situated under the GWT, the degradation mechanism cannot occur due to a lack of oxygen. After 10-15 years of dry periods, the bearing capacity of the timber pile foundation will be lost. Therefore, during the design of a timber foundation pile, the groundwater level is of high importance [50]. To design a timber foundation pile, a concrete pile cap must be implemented as depicted in Figure 6.2. The concrete cap will be placed around the top of the timber pile, ensuring that the timber part stays below the GWT, thereby preventing degradation. Moreover, the concrete cap will create a more straightforward connection between the pile and the concrete foundation beam. However, because of this connection, possible horizontal loads, moments, tension forces and eccentricities must be minimised.

Figure 6.1: Degradation of timber pile [56]

Figure 6.2: Concrete pile cap with steel tube [21]

The determination of the minimum length of the concrete cap can be based on the GWT in Zwolle and Delft. In Zwolle, the GWT has shown relative stability over ten years, ranging from a minimum level of -105 cm to a maximum level of -5 cm, as observed in subsection 3.3.2. To ensure the timber part remains submerged below the GWT, it must be positioned below -150 cm. This accounts for various safety factors if the GWT drops below the 150 cm threshold. On the other hand, in Delft, the GWT has been monitored over one year, revealing fluctuations between a minimum level of -201 cm and a maximum level of -109 cm relative to ground level, as mentioned in subsection 3.3.2. To ensure the timber pile variant against potential degradation due to dryness and a lower GWT than the minimum of -201 cm, the timber part must be positioned at a level lower than -350 cm. This ensures additional safety in such circumstances. In conclusion, a concrete cap with a minimum length of 150 cm is required in Zwolle, while a length of 350 cm is necessary in Delft to prevent any degradation of the timber piles. The concrete caps are generally delivered in concrete classes C35/45, C40/50 and C45/55, depending on the manufacturer. These concrete strength classes are similar to the prestressed prefab foundation pile. The deliverable standardised lengths of the caps can be seen in Table 6.2 and are also partly dependent on manufacturer [63].

Intersection [mm]	Maximum length [m]
∅ 230	2.5
∅ 280	3.0
∅ 310	3.5 - 4.0
∅ 350	4.0

Table 6.2: Characteristics dimensions concrete cap [21]

6.2. Capacity calculations

The first step in calculating the environmental impact of the timber pile foundation variant is the calculation of the pile foundation dimensions and configuration. To calculate the design dimensions of the timber foundation pile, the reaction forces on the foundation beams, as calculated in subsection 3.1.3, need to be considered. The capacity calculations can be divided again into the piles' bearing and structural capacity. However, the first step is redistributing the timber pile foundation plan because of the lower structural capacity of timber in comparison with concrete.

6.2.1. Foundation pile plan

The maximum capacity of wooden piles varies between 75 and 150 kN [63]. Therefore, additional piles need to be implemented on top of the standardised 6-pile plan of FLOW, to lower the vertical ULS load per pile. The number of piles will be doubled to 12 piles to subsequently lower the maximum normal force (ULS) per foundation pile. Figure 6.3 shows the foundation plan of the twelve timber foundation piles. The foundation piles will be equally divided over the longitudinal foundation beam with a distance of 1.55 meters between the two piles. There will be 12 timber foundation piles, which are symmetric and equally divided at a length of 0.65m, 2.20m, 3.75m, 5.30m, 6.85m, and 8.40m.

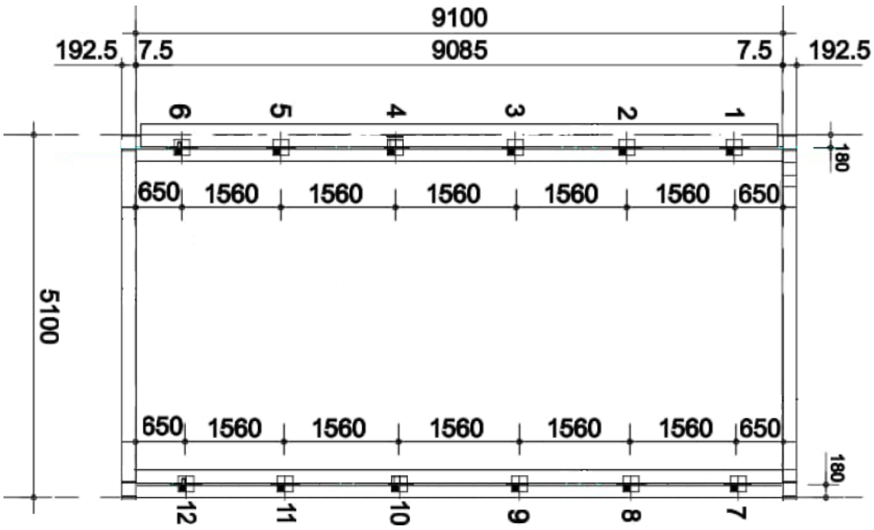


Figure 6.3: Foundation plan for 12 timber foundation piles [3]

The design of a foundation pile involves two essential checks: the bearing capacity of the pile (shaft + tip) in the soil and the structural capacity of the pile itself. The initial step is to assess the bearing capacity of the pile in the CPT profiles of both Zwolle and Delft, as this significantly influences the required pile length. Subsequently, the calculated length of each pile is used to conduct the structural analysis, which helps determine the diameter. The assumptions made for the bearing capacity of the timber piles can be found in Section 6.1. The reaction forces for the prefab concrete piles are presented in Table 4.2. Due to the symmetry in the foundation plan, the normal forces on the timber foundation piles are almost identical, with slight deviations resulting from asymmetric loads. The eccentricity of the piles is caused by small imperfections in the pile itself and installation deviations. By multiplying the total pile eccentricity by the vertical normal force, the moments in both ULS and SLS can be calculated. The standard eccentricity of 50 mm will be used for all the foundation piles. For the simplicity of the calculation, it is assumed that all piles have a normal force of 96 kN, similar to piles 5, 6, 11 and 12.

Pile numbers	Vertical ULS [kN]	Vertical SLS [kN]	Eccentricity pile [mm]	Moment ULS [kNm]	Moment SLS [kNm]
Pile 1, 2, 3, 4, 7, 8, 9, 10	95	70	50	4.75	3.52
Pile 5, 6, 11, 12	96	71	50	4.80	3.56

Table 6.3: Support reactions timber pile foundations

6.2.2. Bearing capacity of the piles

The first calculation check is the bearing capacity of the twelve timber piles in combination with the CPT profiles in both Zwolle and Delft. Again, the bearing capacity of the piles will be calculated using the software program D-foundation of Deltares. The lithography of Zwolle and Delft is obtained from DINOloket and shown in Appendix A.2. For Zwolle, the foundation piles consist of straight timber foundation piles with standardised dimensions according to Table 6.1. This is because the 3.5-meter timber pile element can easily be sawn, and standardised dimensions can be used. For the longer Delft piles, tapered timber piles need to be used because only tapered piles are available for piles with long lengths. Moreover, tapered piles significantly improve shaft, end bearing and lateral capacity compared to uniform cross-section piles of similar size. Due to the usage of taper piles, there is a significant saving in material and costs. Besides that, taper piles will mitigate the failure due to earthquakes or natural disasters [54]. The timber foundation piles have a taper of 7.5 mm per meter length. The foundation plan given in Figure 6.3 will be used for both Delft and Zwolle. The following pile classification factors must be considered for the timber foundation piles. For the point resistance α_p, a value of 0.7 must be considered because the pile is driven (post-2017 standard NEN-9997). The shaft pressure α_s for the tapered and straight timber piles are respectively 0.012 and 0.010. The tension α_t pile classification is 0.007. For the determination of the settlements of the piles, the load-settlement curve 1 of the NEN-9997 must be considered. No additional slip layer will be applied to the foundation piles; therefore, the representative adhesion will be zero. For the pile tip shape factor β, a value of 1.0 needs to be considered. The pile tip cross-section factor s is not applicable because it only counts for open-ended steel pipes. Besides that, it is assumed that there will not be any additional surcharge before or during the foundation process.

For the bearing capacity calculation, it is essential to connect the vertical loads in both SLS and ULS, as given in Table 6.3, to each pile in the D-foundation. The FLOW building is considered non-rigid, as it cannot transfer moments in its connections, which impacts the correlation factor ξ_4 for the minimum value of the calculated pile resistance. The other override factors remain default according to the EC7-NL standard. Due to some piles being positioned within 5 meters of each other, the pile group effects must be considered, influencing the negative skin friction. However, because of the lack of multiple CPTs, it will be assumed that the two CPTs are representative and consistent for both foundation plans. As there is only one available CPT, the ξ_3 and ξ_4 factors in the bearing capacity calculation of the pile will be 1.39 (NEN-9997 Tab. A.10). The excavation level will be 0.34, which corresponds to the top of the CPT level. Consequently, there is no need to account for any reduced q_c in this case. For the specific location of Delft, the excavation level is at zero meters, making it again unnecessary to consider any q_c reduction.

For the maximum allowed settlement in Limit State and Serviceability State, the default values of respectively 0.15 m and 0.05 m are used (NEN 9997). The default settings will also be used for the maximum allowed relative rotation, which is 0.01 and 0.003 for ULS and SLS, respectively. The following bearing capacity checks need to be conducted: Verification of Limit State EQU, verification of Limit State STR/GEO and verification of Serviceability Limit State, which can also be seen in Appendix C.2

6.2.3. Structural capacity of the piles

Besides the check for the bearing capacity of the piles in the soil, the structural capacity of the piles also needs to be conducted. The structural capacity deals with the forces and moments on and in the pile design. The methodology of the structural capacity of the timber piles is different from the concrete structural capacity because the transport and lifting will not be the governing checks because of the low self-weight, no required hardening time and low self-weight. The transport verification can be found in Appendix E.3.2. Besides that, the maximum applicable length is determined by the maximum growth of the tree. However, because timber is a natural material, additional (material) factors need to be applied.

For the timber foundation piles, different timber materials can be used. For this research, spruce (Picea abies) C18 will be used. This is because spruce wood is available in the Netherlands (not native) [35]. The calculation will be done with strength class C18, which corresponds with quality class C according to the NEN 5468. As a result, the design verification counts for all C18 or higher timber piles. The strength classes with corresponding material characteristics for sawn softwood timber can be found in Appendix E.3. The capacity calculations assume that the soil is stiff enough to prevent any buckling of the timber pile. Therefore, the timber part of the foundation pile needs to be checked on the combination of normal force and bending moment due to eccentricity according to NEN-EN 1995-1-1+C1+A1 article 6.3.2. This is because it is assumed that the caused bending moment due to eccentricity will be transferred from the concrete cap towards the timber foundation pile. For timber foundation piles exposed to pressure and bending, formulas 6.19 and 6.20 of the NEN-EN 1995-1-1+C1+A1 can be used. Because both $\lambda_{rel,y}$ and $\lambda_{rel,z}$ are below 0.3, equations 6.23 and 6.24 do not have to be considered. The structural calculation verification of the timber element piles in both Zwolle and Delft can be found in Appendix E.3

$$(\frac{\sigma_{c,o,d}}{f_{c,o,d}})^2 + k_m \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \tag{6.1}$$

$$(\frac{\sigma_{c,o,d}}{f_{c,o,d}})^2 + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \tag{6.2}$$

In which the following variables need to be determined:

$$k_{c,y} = \frac{1}{k_y + \sqrt{k_y^2 - \lambda_{rel,y}^2}} \tag{6.3}$$

$$k_{c,z} = \frac{1}{k_z + \sqrt{k_z^2 - \lambda_{rel,z}^2}} \tag{6.4}$$

$$k_y = 0.5(1 + \beta_c(\lambda_{rel,y} - 0.3) + \lambda_{rel,y}^2) \tag{6.5}$$

$$k_z = 0.5(1 + \beta_c(\lambda_{rel,z} - 0.3) + \lambda_{rel,z}^2) \tag{6.6}$$

$$\lambda_{rel,y} = \frac{\lambda_y}{\pi} \cdot \sqrt{\frac{f_{c,o,k}}{E_{0,05}}} \tag{6.7}$$

$$\lambda_{rel,z} = \frac{\lambda_z}{\pi} \cdot \sqrt{\frac{f_{c,o,k}}{E_{0,05}}} \tag{6.8}$$

For the design of the concrete cap and its connection with the timber pile, the BRL 1721 will be used. This certification specifies specific requirements regarding the concrete cap and its connection. The concrete class must be at least C35/45 with minimum S310 steel reinforcement. A reference design of Adelaar-beton will be used to design the concrete caps. These calculated concrete caps for timber foundation piles include the capacity calculations. The concrete caps have environmental class XC3 and concrete strength class C45/45. The cap must have a minimum 6 ∅ 6 mm longitudinal reinforcement. Moreover, it needs at least 8 ∅ 4.5 mm spiral reinforcement at the head and 5 ∅ 4.5 mm spiral reinforcement at the foot, according to BRL 1721. Besides the longitudinal and spiral reinforcement, a steel tube with a 4 mm wall is needed to connect the timber pile. The steel reinforcement quality will again be B500B, and the steel tube will be made from S235. The structural calculations of the steel tubes can be seen in Appendix E.3. The calculations of the concrete cap are validated by Adelaar-beton. Because the length of the concrete caps remains below 5.0 meters and the normal force below 200 kN, it is assumed that the concrete caps design more than meets the structural requirements. Besides that, the design of the concrete caps is satisfied with the BRL 1721. However, detailed calculations on the concrete cap can be requested at their website. The steel tube connection between the concrete part and the timber pile must transfer at least 0.08 * F_d due to the horizontal forces, eccentricity and initial misalignment (BRL 1721). The steel tube calculations of both Zwolle and Delft can be found in Appendix E.3.

The (dynamic) transport and lifting verification of the timber pile can also be seen in Appendix E.3. These verifications were the critical checks for the concrete prestressed variant. However, these checks are not critical for the timber foundation piles because of the low weight, limited length and no hardening time.

6.3. Design and dimensions

Using the methodology mentioned above in section 6.2 for both the bearing and structural capacity of the timber piles and considering the loads and soil conditions, the design and dimensions in both Zwolle and Delft can be determined. The soil conditions are given in Figure 3.15 and 3.16, the foundation plan in Figure 4.2, the loads on the piles in Table 6.3 and the shear and moment distribution in Appendix E.1.

6.3.1. Location Zwolle

The results of the bearing capacity of the timber foundation piles can be seen in Table 6.4 and are derived from the bearing capacity of the foundation piles in Zwolle and the verification checks in Table 6.5. This results in twelve 200 mm pile tip diameter piles, with a total length of 5.00 meters. The first step is the verification in Limit State EQU, where the results can be seen in Table 6.4. For the EQU verification, the $R_{c;d}$ needs to be considered, which does not have to include the negative skin friction, according to the NEN 9997. For the SLS and STR/GEO checks, the negative skin friction needs to be considered; therefore, the value $R_{c;net;d}$ needs to be considered. In all cases, the STR/GEO verification was normative for the pile length. The piles lengths for piles 1 to 12 are all 5.00 meters because of the similar normal forces. It is assumed that the concrete cap will be placed at -1.50m because of the GWT restrictions mentioned in subsection 6.1.1. This leads to a combined pile design consisting of a 1.50-meter concrete cap and a 3.50-meter timber pile section. As the forces acting on all twelve piles are similar, the design for each pile will be uniform. Hence, the calculations will be performed on one single timber pile because the piles are identical.

Pile number	Length [m]	$F_{c;d}$ [kN]	$R_{c;cal;max}$ [kN]	$R_{c;d}$ [kN]	$F_{nsf;d}$ [kN]	$R_{c;net;d}$ [kN]
Pile 1-12 (∅ 200)	-5.00	96	289	173	14	159

Table 6.4: Bearing capacity timber foundation pile Zwolle

Besides the EQU verification, the STR/GEO and SLS verification must also be conducted. These verifications include negative skin friction. The maximum values for the settlement and rotation can be seen in Table 6.5. Table 6.5 shows the maximum occurring rotations between the neighbouring pile elements.

Pile number	Length [m]	S_d STR/GEO [m]	ϕ_d STR/GEO	S_d SLS [m]	ϕ_d SLS
Max value:	-	0.150	0.01	0.05	0.003
Pile 1-12 (∅ 200mm)	-5.0	0.011	0.0023	0.007	0.0016

Table 6.5: Verification checks bearing capacity timber pile Zwolle

With the derived pile length and corresponding cross-section dimensions, as seen in Table 6.1, the structural capacity of the timber pile and the reinforcement in the concrete cap can be calculated. The structural calculations can be found in Appendix E.3. Besides that, the design is based on the BRL 1721, which sets the requirements for a concrete cap. The concrete class is C35/45 with reinforcement steel B500B. The total width of the timber pile is determined in subsection 6.2.2 and comes down to 200 mm in diameter. Therefore, the surrounding S235 steel tube will be ∅ 210 mm, including a steel thickness of 4 mm, which makes the free clearance for the timber pile 2 mm around the entire circumference. The concrete cap on top has a diameter of 230 mm and a concrete cover of 30 mm. The six longitudinal reinforcement bars will have a diameter of ∅ 6 mm, according to the minimum required diameter. The required spiral reinforcement will be ∅ 4.5 mm and have eight windings at the head over a total distance of 200 mm, which makes the space between spirals 25 mm. This satisfies the required minimum distance of 20 mm according to BRL 1721 standard. Also, a total of 8 ∅ 4.5 mm over a length of 300 mm will be implemented for the foot spiral reinforcement. This additional winding is implemented because of the connection of the steel tube and, thereby, additional capacity around this critical part. The total length of the concrete cap is 1.5 meters, ensuring that the timber pile remains below the GWT. Therefore, the C18 timber spruce pile, placed in the steel tube, has a total length of 3.5 meters and a diameter of 200 mm, as derived from the bearing capacity calculations. The 5.0-meter long spruce foundation pile design with concrete cap for the location Zwolle is depicted in Figure 6.4. It must be mentioned that the timber part will be straight and not tapered in Zwolle.

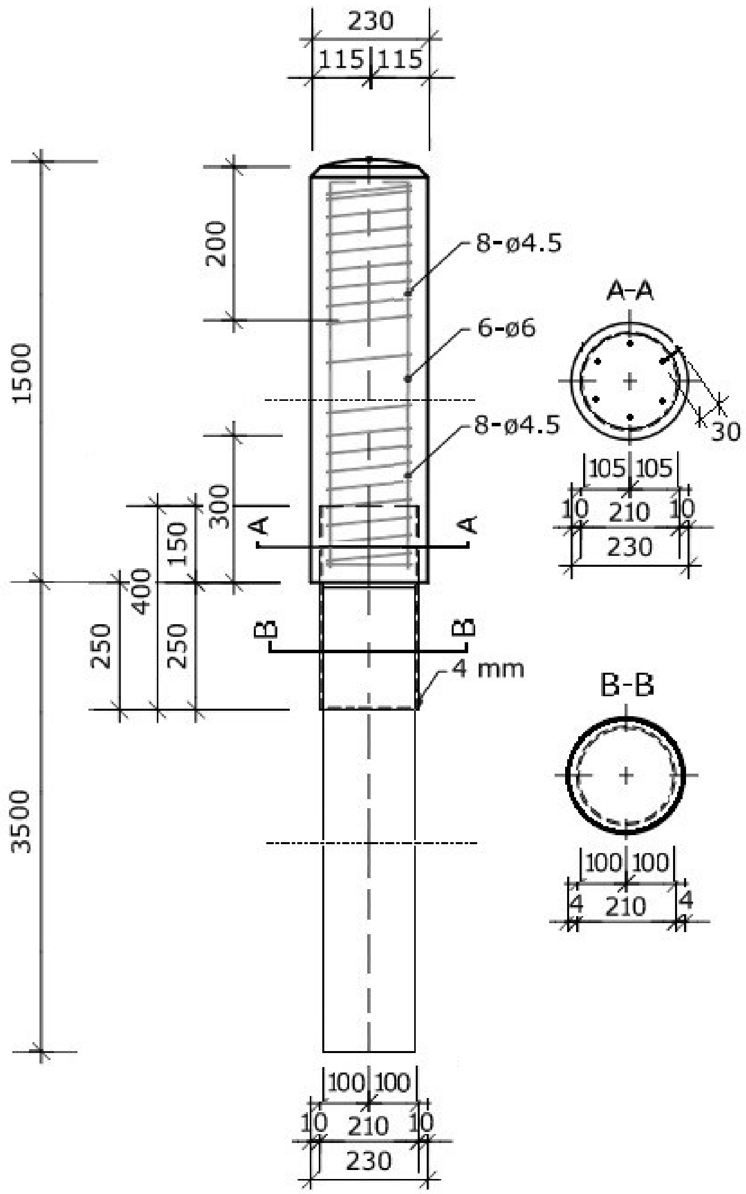


Figure 6.4: Design and dimensions of spruce timber pile with concrete cap in Zwolle

6.3.2. Location Delft

The results of the bearing capacity of the tapered timber foundation piles in Delft can be seen in Table 6.6 and are derived from the bearing capacity of the foundation piles in Delft and the verification checks in Table 6.7. The lengths for piles 1 to 12 are 23.0 meters because of the similar normal forces. It is assumed that the concrete cap needs to be placed at -3.50m because of the GWT restrictions mentioned in subsection 6.1.1. To satisfy the normative verification in SLS, each pile's minimum diameter must be 450 mm at the top and 320 mm at the tip, as seen in Table 6.7. This is because the SLS and STR/GEO checks include the negative skin friction $R_{c;net;d}$, which is significant in Delft. This leads to a combined pile design consisting of a 3.50-meter concrete cap and a 19.50-meter timber pile section, which satisfies the maximum timber pile length of 20 meters. As the forces acting on all twelve piles are similar, the design for each pile will be uniform.

Pile number	Length [m]	$F_{c;d}$ [kN]	$R_{c;cal;max}$ [kN]	$R_{c;d}$ [kN]	$F_{nsf;d}$ [kN]	$R_{c;net;d}$ [kN]
1-12 (∅ 450-320)	23.00	96	1982	1188	651	537

Table 6.6: Bearing capacity timber foundation pile Delft

Pile number	Pile length [m]	S_d STR/GEO [m]	ϕ_d STR/GEO	S_d SLS [m]	ϕ_d SLS
Max value:	-	0.150	0.01	0.05	0.003
1-12 (∅ 450-320)	-23.0	0.046	0.01	0.042	0.008

Table 6.7: Verification checks bearing capacity Zwolle

However, due to the non-rigid categorisation of the FLOW house, it becomes unfeasible to fulfil the criteria for relative rotation, illustrated in Table 6.7. The limitation stems from the permitted maximum rotation of 0.003, whereas the computed rotation using equation C.5 amounts to 0.009. This discrepancy is attributed to the notably elevated settlement S_d, a result of the structure's diminished stiffness (EA), and the small heart-to-heart spacing of the timber piles. Consequently, the rotation analysis within the SLS framework fails to align with the stipulated requisites outlined in NEN 9997-1:2016, article 2.4.9. However, if the FLOW building is regarded as rigid, the rotation assessments could be waived, and the foundation design could be valid.

The specified tip diameter for the tapered pile in the Delft is 320 mm, coupled with a corresponding timber length of 19.5 meters. However, achieving this requirement is implausible due to the substantial dimensions involved. While it might theoretically be achievable to produce a timber foundation pile with a diameter of 450 mm at the top, 320 mm at the tip, and a length of 19.5 meters, such occurrences are exceedingly uncommon. Notably, it should be acknowledged that timber piles with these larger dimensions are predominantly found in non-European countries.

Given that the timber foundation pile in the Delft fails to meet the required rotation checks in the SLS, and is burdened by excessive dimensions, the viability of a timber foundation pile in Delft is highly improbable. Nonetheless, to explore the environmental implications of extended timber piles and compare them with other alternatives, it is assumed that the design of the timber foundation can be realised. This approach enables the presentation of the environmental impact of longer timber foundation piles and investigates the environmental potential. It is crucial to remember that the actual implementation of a timber foundation pile in Delft remains highly unlikely.

With the hypothetically derived pile length and corresponding cross-section dimensions, as seen in Table 6.1, the structural capacity of the timber pile and the reinforcement in the concrete cap can be calculated. The capacity calculations can be found in Appendix E.3. Besides that, the design is based on the BRL 1721, which sets the requirements for a concrete cap. Because of the large diameter timber pile, the standard concrete cap dimension given in Table 6.2 can not be used. However, the standardised dimension will be used as a reference design for the concrete cap in Delft. Therefore, the design of the concrete cap is as follows: The concrete class is C35/45 with reinforcement steel B500B. The total width of the timber pile is determined in subsection 6.2.2 and comes down to 450 mm in diameter. To ensure that the timber pile part remains below GWT the length of the cap will be 3.5 meters. Therefore, the surrounding S235 steel tube will be ∅ 460 mm, including a steel thickness of 4 mm, which makes the free clearance for the timber pile 2 mm, around the entire circumference. The concrete cap on top has a diameter of 480 mm and a concrete cover of 30 mm. The six longitudinal reinforcement bars will have a diameter of ∅ 8 mm, according to the minimum required diameter. The required spiral reinforcement will be ∅ 6 mm and have eight windings at the head over a total distance of 200 mm, which makes the space between spirals 25 mm. This satisfies the required minimum distance of 20 mm according to BRL 1721 standard. Also, the foot spiral reinforcement of 8 ∅ 6 mm over a length of 300 mm will be implemented. This additional winding is implemented because of the steel tube's connection and additional capacity around this critical part. The total length of the concrete cap is 1.5 meters, ensuring that the timber pile remains below the GWT. The C18 timber spruce pile in the steel tube, therefore has a total length of 19.5 meters and a diameter of 450 mm, as derived from the bearing capacity calculations. The 23-meter-long tapered spruce foundation pile design with concrete cap for the location Delft is depicted in Figure 6.5.

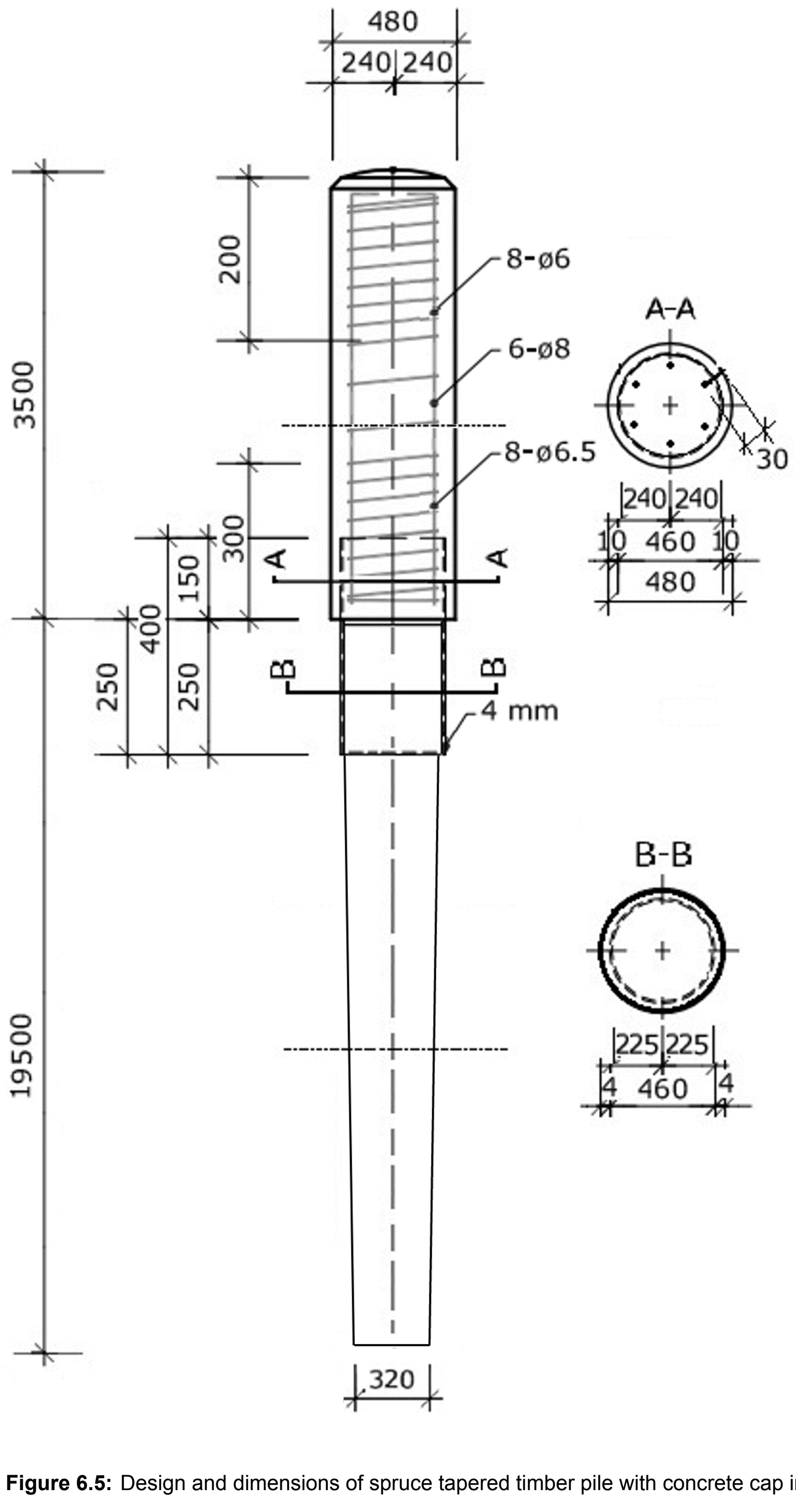


Figure 6.5: Design and dimensions of spruce tapered timber pile with concrete cap in Delft

6.4. Environmental impact

In the previous sections, the design of the timber foundation pile with concrete cap is determined. With that, all the elements and their quantities are known, which are needed to investigate the environmental impact in detail. In Appendix E.3, the total volume and weights of the timber piles and concrete cap with reinforcement are calculated. The environmental impact of the concrete cap with reinforcement can be calculated with the "Ontwerptool Groen Beton V6" and has the same methodology as the concrete prestressed variant. This tool contains the environmental data of concrete elements from the LCA-stage A to D of the concrete cap element. No standard spruce timber foundation pile EPD is available as category 1 data. The 2023 environmental data for spruce C18 timber in the Netherlands originates from Centrum Hout [34]. Therefore, this data will be used for the timber part of the foundation pile. However, the specific environmental data is confidential and can not be shared in detail in this research. A request can be made at the Centrum Hout website for the specific data. The ECI calculation can be divided into the timber pile and concrete cap with reinforcement. The environmental data per element can be found in Appendix F.

6.4.1. Sawn spruce timber and concrete mixture (A1)

The first step is the raw materials supply phase (A1) of the sawn spruce and the concrete mixture with reinforcement. For the timber part, C18 spruce will be used. European spruce originates from the Picea abies and is available in the Netherlands (not native). For the environmental impact of that part, the from-origin Swedish sawn spruce reference will be used. The latest 2023 spruce softwood data is collected from 44 sawmills and covers the production of 10, 190.000 m^3 sawn dried timber, and results in an average environmental impact. The sawn dried timber is used as raw material in wood production and for Swedish logs, which have the same basis as timber foundation piles. The dried sawn spruce timber has a moisture content of 16% and an average density of 469 kg/m^3. The environmental impact in the raw material supply A1 phase includes the extraction of the timber, including harvesting, thinning, planting, etc. Besides that, it also includes the production of electricity and heat for the extraction. This includes a diesel consumption for forwarders including thinning of 0.8 l/m^3, and for harvesters, including thinning of 1.06 l/m^3 [65]. For the diesel consumption of forest management, which includes planting, soil preparation, clearing and fertilisation, 0.29 l/m^3 diesel is considered. Moreover, it is assumed that all wood is harvested sustainably and that the wood in the system fulfils the criteria of biogenic carbon neutrality over its life cycle. This is because the timber foundation piles will remain in the soil after the service life of the building. This results in biogenic carbon storage of the timber of 715 kg CO_2/m^3 during the life cycle of 100 years. This CO_2/m^3 storage results in a total GWP value of –577 $kgCO_2/m^3$ during the A1-A3 LCA stage [65]. The specific data for the spruce softwood cannot be given because of confidentially restrictions. However, the ECI values of the timber foundation piles will also be displayed if it is assumed that the timber part will not absorb any CO_2. This way, the differences between absorption and releasing CO_2 of the timber elements can be investigated.

The concrete cap on top of the timber part will consist of the minimum required C35/45 concrete with B500B reinforcement steel. The environmental classes and corresponding requirements are similar for the concrete cap as the prestressed foundation pile discussed in section 4.1. Therefore, for the mixture of the concrete cap, the same composition as the prestressed prefab concrete mixture, as seen in Table 4.11, will be used. The CEM I 52.5R and the CEM III/a 52.5N mixtures have a w/c ratio of 0.53. The processes, including the raw materials supply for the concrete are similar. The higher concrete strength classes will not be considered for the concrete cap because it will result in a higher environmental impact due to a larger cement ratio.

6.4.2. Sawing, drying and prefab pile manufacturing (A3)

The manufacturing stage encompasses the timber pile and concrete cap production processes. The fabrication of the sawed timber pile involves several steps: debarking, sawing, drying, and sorting. While the EPD for the Swedish spruce timber (Centrum Hout) lacks intricate specifications regarding the environmental impacts for each process within the A3 phase, it is assumed that the environmental data from the A3 phase reasonably aligns with the processes related to a timber foundation pile. The same methods are valid for the concrete production as with the prestressed pile. This includes the average concrete plan use of 3.63 kWh electricity, 0.12 l diesel and 0.13 gas m^3 [10] per cubic meter of concrete mixture in the A3 manufacturing phase. On top of that, an additional electricity use of 20 kWh in Zwolle and 30 kWh in Delft is considered because of the electric processes involved in the pouring process (lifting, pouring, etc). Again, the concrete mixture plant is assumed to be directly placed next to the prefab foundation manufacturer, and no additional transport is required.

6.4.3. Transport and installation (A4+A5)

Next to the environmental impact of the timber/concrete elements in subsection 6.4.3, the impact of the transport and installation processes needs to be determined. Firstly, transporting the twelve timber piles towards the location in Zwolle and Delft needs to be performed. The distance between the manufacturer and the building site will be the same to achieve a fair comparison between all the foundation variants. In this way, the results will not depend on the distance between the building site and the manufacturer. It is assumed that the concrete and timber manufacturers are located separately, and both the timber piles and the concrete caps need to be transported over a distance of 82 km. This results in a distance of 82 km between the manufacturer and the building site in both Zwolle and Delft. The transport of both locations will be performed with EURO 6 (>32 ton) diesel trucks, which comply with the latest Euro norm emissions [53]. These trucks have the lowest possible emissions because of the latest emission restrictions. The total weight of the twelve timber pile parts is 2537 kg for Zwolle and 31461 kg for Delft.

As seen in subsection 2.3.4 the foundation piles can be installed in multiple ways. The regular installation method for timber piles is piling. The piling is performed with a relatively light-weighted pile hammer of roughly 500-800 kg. The rig provides both the lifting and piling of the piles and will also be used for installing and lifting the concrete caps. It is assumed that for the 12 relatively short piles in Zwolle, the piling rig will be used for 1 hour in total, including installing the concrete cap. For the longer Delft piles, it is assumed that the piling will be used for 2.0 hours. This slight increase is caused because most of the time will be spent on the lifting and positions instead of the piling itself. Besides that, it is assumed that the piling rig will be used for 15 houses per location. Therefore, the total transport of the rig will be 11 km per single house for both Zwolle and Delft. The piling rig will be transported using the same EURO6 truck. Again, no dismantling or recycling will be considered because it is presumed that the piles remain in the soil after a lifespan of 75 years. It is also assumed that no additional lifting machine is needed because the lifting mechanism of the piling rig is considered sufficient.

6.4.4. Representations of ECI value by category

The environmental impacts of the twelve piles in Zwolle and Delft can be displayed by utilising the spruce timber design calculations outlined in section 6.3, along with the aforementioned transport and installation processes. Due to the two concrete mixtures, given in Table 4.9, a distinction can be made between CEM I and CEM III concrete cap mixtures.

ECI representation Zwolle

For the 5.0-meter long timber piles with a diameter of 200 mm, along with the 1.5-meter diameter 230 mm concrete cap in Zwolle, the total ECI values are calculated as €23.22 and €19.52 for CEM I and CEM III, respectively. The ECI contribution per impact category of both concrete mixtures is also illustrated in Figure 6.6. The ECI value translates to an MPG value of 0.0019 for OPC and 0.0016 $€/m^2{*}y$ for BFS, as calculated using equation 2.1. It can be inferred from Figure 6.6 that the GWP has a negative contribution to the ECI value. This negative value is caused by the absorption of 577 $kgCO_2/m^3$ by the twelve timber spruce elements, which results in a value of -€27.78. As mentioned, it is assumed that the timber piles remain in the soil after the service life, and the biogenic carbon storage can be assumed [65]. Besides that, the HT, AP and EP have a significant contribution to the environmental impact of the foundation design. The AP and EP contributions are mainly caused by cement production. The HT is primarily caused by chemical treatment and occupational exposure during the timber harvesting and processing. What also strikes is the relatively small reduction in GWP by applying BFS instead of OPC to the foundation piles. This is caused by the relatively small amount of concrete in the timber foundation variant. The implementation of CEM III instead of CEM I results in a total ECI reduction of 15.9%, which comes down to €3.70 of the twelve foundation piles as shown in Figure 6.6.

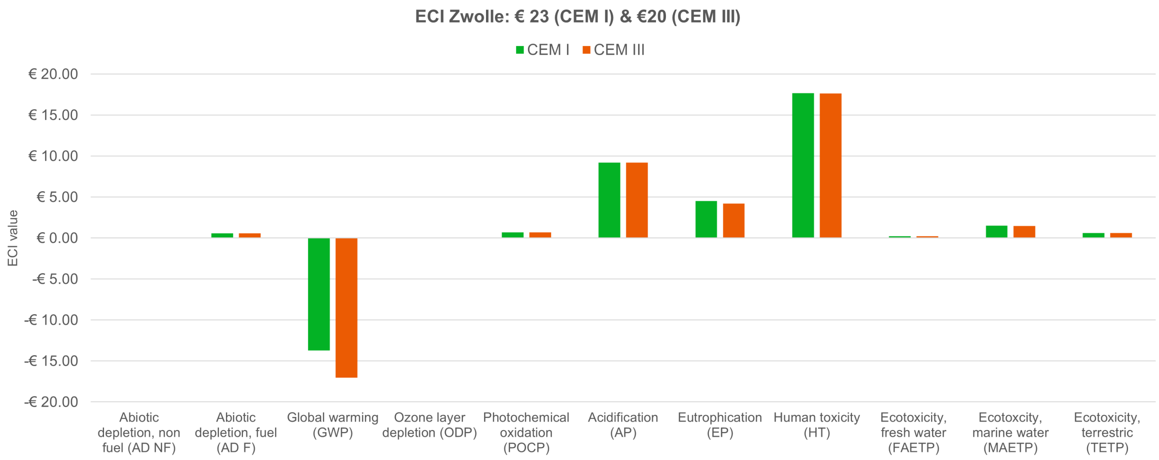


Figure 6.6: ECI value timber pile Zwolle per impact category CEM I/III

Figure 6.7 shows the environmental impact per LCA stage in Zwolle. The raw materials supply stage (A1) has the lowest contribution to the ECI value for CEM I and III. This is again caused by the timber part of the foundation that absorbs CO_2 from the air. The manufacturing process (A3) is the second largest contributing stage to the ECI. This is caused by the required energy for the concrete pouring process and the sawing of the spruce timber. The transport phase (A4) has a relatively small contribution to the total ECI value because of the low self-weight of the concrete and timber elements. Because the environmental impact is measured in tons per km, the emission of both trucks is low. The construction and installation stage (A5) of the timber foundation piles with concrete caps has the largest contribution to the total ECI value in Zwolle. This is because the twelve timber piles with concrete caps necessitate a minimum piling time of 1 hour, resulting in a larger proportion on the total ECI value. This can also be confirmed by Figure 6.8, where the diesel piling process is the largest contributor to the ECI.

Figure 6.8 displays the most contributing elements with their ECI contribution for the timber foundation pile in Zwolle. It can be concluded that the 3.5-meter-long timber part results in a total negative ECI value of -€27.09. The diesel piling process is the largest contributing element. The second largest ECI contributor is the steel ∅ 210 mm tube that connects the concrete cap with the timber pile. Cement has a relatively low contribution because of the low concrete quantities. It can be concluded that applying an electric piling rig may have the largest effect on the ECI value. The optimisation of the concrete strength class with optimum reinforcement is less effective because of the already relatively low ECI contribution.

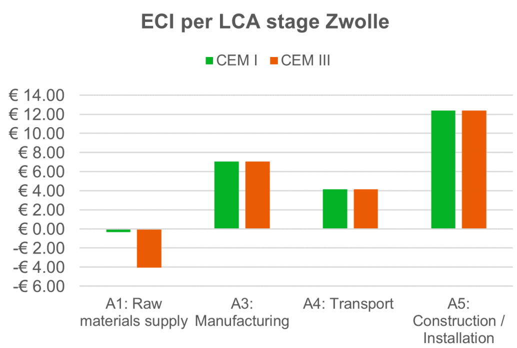


Figure 6.7: ECI value per LCA stage Zwolle

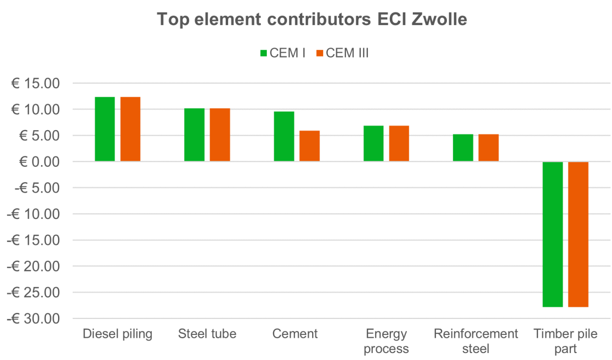


Figure 6.8: Top elements contribution ECI Zwolle

However, if it is assumed that 577 $kgCO_2/m^3$ storage in the timber does not occur due to disposal or burning at the end of life, the ECI value dramatically changes. Assuming no CO_2 storage in the timber part results in a total ECI value of €67.29 (CEM I) and €63.59 (CEM III) in Zwolle, as depicted in Figure 6.9. The other ECI values per impact categories remain similar with or without CO_2 storage as concluded from Figure 6.6.

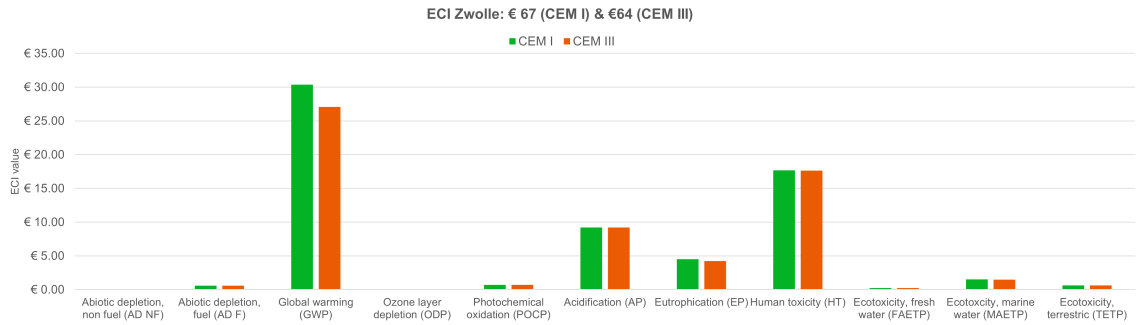


Figure 6.9: ECI value of Zwolle per impact category considering no CO_2 storage

ECI representation Delft

The total ECI values for the timber pile design in Delft are respectively -€319 and -€357, for CEM I and CEM III, as depicted in Figure 6.10. This comes down to an MPG of -0.0264 and -0.0296 €$/m^2*$y for respectively CEM I and CEM III. It can be concluded from Figure 6.10 that the GWP has a high negative ECI value, which means that the CO_2 storage contributes to a positive environmental impact. Because of the total 27.5 m^3 of timber, the total CO_2 storage is 15867 kg, which results in a total ECI value of -€787. Again, the HT has a relatively high ECI contribution due to the sawing and chemical treatment of the timber. However, the negative GWP compensates for the other impact category contributions because of the very high CO_2 storage.

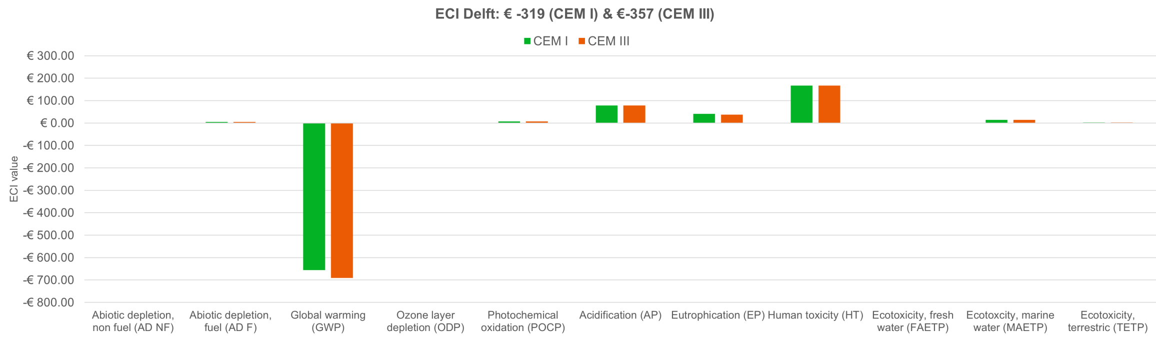


Figure 6.10: ECI value of Delft per impact category CEM I/III

Figure 6.11 and Figure 6.12 shows respectively the ECI values per LCA stage and the high contributing elements for the tapered timber pile with concrete cap design in Delft. From both Figures, it can be concluded that the timber part has a major influence on the environmental impact. Because of the high timber quantity in the design, the A1 stage has the largest impact. The second largest impact stage is the transport stage (A4) because of the relatively high 18193 kg self-weight of the twelve concrete caps and the 12898 self-weight of the twelve timber parts. The twelve 3.5-meter long ∅ 480 mm concrete caps also contain a lot of cement, which results in a relatively high ECI cement contribution as confirmed by Figure 6.12. Therefore, implementing CEM III instead of CEM I results in an ECI reduction of 10.64%. It can be concluded that transport, diesel piling rig and the steel tube have the largest optimisation potential.

If it is again assumed that 577 kg CO_2/m^3 storage in the timber does not occur due to disposal or burning at the end-of-life stage, the ECI increases to €600 (CEM I) and €561 (CEM III) in Delft, as

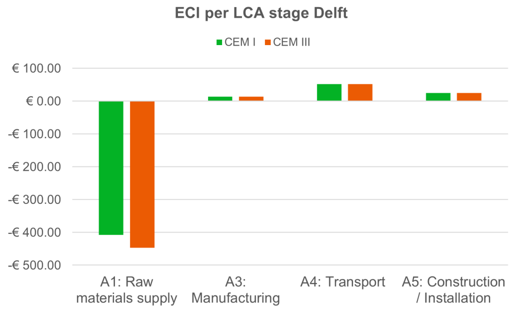


Figure 6.11: ECI value per LCA stage Delft

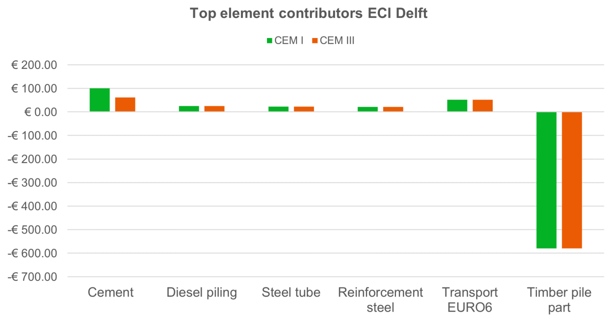


Figure 6.12: Top 4 elements contribution ECI Delft

depicted in Figure 6.13. Assuming no CO_2 by the timber part therefore results in an ECI increase of €919 to €600 for the CEM I mixture. Therefore, it can be concluded that the assumption on the CO_2 storage of timber has immense effects on the environmental impact. However, in this research, it is assumed that the piles remain in the soil after their service life and therefore have a lifetime of >100 years. In that case, the sawn dried timber has a GWP value of –577 $kgCO_2/m^3$, which includes biogenic carbon storage at 715 $kgCO_2/m^3$. However, suppose for some reason the timber piles are removed and subsequently disposed of or burned. In that case, the ECI values given in Figure 6.9 for Zwolle and Figure 6.13 for Delft are accountable.

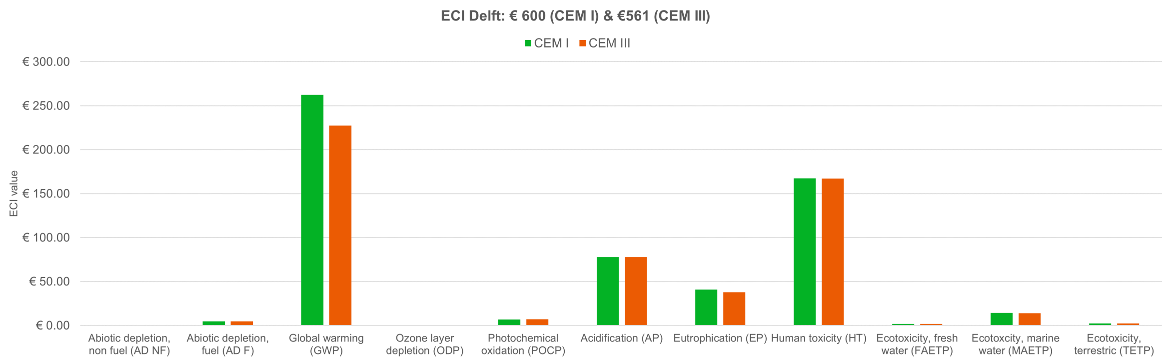


Figure 6.13: ECI value of Delft per impact category considering no CO_2 storage

The negative ECI value of C18 spruce timber in both Zwolle and Delft leads to a reduced environmental impact. Consequently, it can be inferred that incorporating extra timber through over-dimensioning timber elements is advantageous. Nonetheless, it is crucial to acknowledge that minimising the quantity of materials in a structure holds significant (environmental) importance. For instance, during procurement, including additional timber may lead to a lower ECI value and, subsequently, a higher environmental score. However, the quantity of materials should also be emphasised, as material reduction is a critical criterion. Additionally, utilising extra timber will inevitably result in higher costs. Hence, it is essential to consider the ECI value of a design and other factors, such as material quantities and costs.

6.5. ECI optimisation

From the calculated and analysed environmental impact in section 6.4, the foundation design in both Zwolle and Delft can be optimised to lower the ECI value even more. The optimisation will be performed on the ECI values with CO_2 storage, because of the >100 years lifespan. The first optimisation was using BFS instead of OPC, as seen in section 6.4. This optimisation resulted in a 15.9% ECI reduction of the foundation piles in Zwolle and a 10.64% ECI reduction in Delft. This difference is caused by the different concrete cap lengths of 1.50m and 3.50m, respectively in Zwolle and Delft. In this section, other potential optimisations will be investigated. These optimisations include solely the use of alternative transportation methods and the utilization of an electric piling rig. Alternative optimisation variants will not be considered because the concrete cap is already designed to meet the minimum required C35/45 strength class and the minimum amount of reinforcement according to the BRL 1721. Furthermore, as depicted in Figure 6.8 and 6.12, cement has a relatively small contribution to the total ECI value. Therefore, the concrete cap for both Zwolle and Delft will not be optimised in terms of concrete strength class and reinforcement. The steel tube connection between the timber foundation pile and concrete cap plays a significant role in the ECI value. However, the thickness of the tube already meets the minimum required thickness of 4mm. Alternative connections have been considered between the cap and timber pile, such as steel hairpins or pins. However, these connections often lack the capacity for larger structures and are primarily used for greenhouse structures. Consequently, the steel tube will not be optimised in the timber foundation variant.

6.5.1. Optimised transport and installation

The timber foundation piles and the concrete caps must be transported to both Zwolle and Delft. Therefore, two truck trips are required. However, because of the low 2537 kg self-weight of piles in Zwolle, the ECI value is limited during the transport phase (A4) as concluded from Figure 6.7. For Delft, on the other hand, the self-weight of the piles is 31461 kg, which results in a significantly higher ECI contribution, as depicted in Figure 6.11. In Zwolle, the construction and installation phase (A5) has the highest ECI value, as seen in Figure 6.7. This is caused by the high-polluting diesel and piling time of one hour. For Delft, the diesel piling rig has a relatively smaller ECI contribution. Therefore, it can be concluded that the transport optimisation has the largest potential in Delft, and the piling rig has the largest potential in Zwolle. As seen in subsections 2.3.5 and 2.3.4, electric transport and installation rigs are used more often nowadays. This subsection will investigate the environmental reduction due to alternative transport of the prefab piles and research the electric piling potential.

Transport foundation piles

Replacing EURO6 diesel trucks with trucks powered by alternative sources can reduce emissions and thereby the environmental impact of the transportation stage (A4). However, it is important to note that the Netherlands imposes a maximum load restriction of 50 tons for more sustainable transportation options. If both the self-weight and the freight load are below 50 tons, no additional permits are required for transportation. This requirement often plays a crucial role in terms of costs and efficiency. Because the timber elements and the concrete caps originate from different locations, the total weight for both Zwolle and Delft will remain below the 50-ton total weight restriction. Therefore, a single truck for transporting the concrete caps and timber piles to Delft and Zwolle will be sufficient. The considered transported alternatives are similar to the prestressed concrete variant. The assumptions about the alternative transport options and their benefits are discussed in section 4.5.2. Figure 6.14 shows the alternative transport variants for Zwolle and Figure 6.15 for Delft. It can be concluded that the absolute environmental impact reduction for Zwolle is minimal by using an alternative transport. This is due to the limited ECI value of the transport towards Zwolle. For Delft, on the other hand, the absolute reduction has a higher effectiveness. Using green-generated electric transport, the ECI value of foundation piles can be reduced by €26.89, equivalent to an 8.43% total ECI reduction. Moreover, it can be noticed that the first-generation bio-diesel has the smallest reduction in comparison with the EURO6 diesel truck. The hydrogen truck and second-generation bio-diesel have a similar total ECI reduction of roughly 3.62%. Figure 6.16 illustrates the ECI contribution per impact category per transport variant of Delft when only considering the transport life cycle stage (A4). The electric truck exhibits a significantly smaller ECI value than the EURO 6 diesel truck. By only considering the transport stage, this results in a reduction of 53%, as depicted in the rightmost bar chart in Figure 6.16. The ratios between the ECI

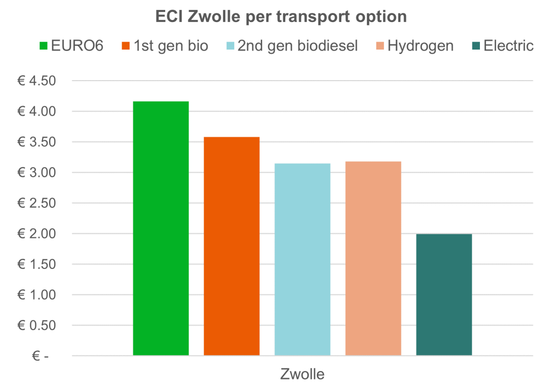


Figure 6.14: ECI value Zwolle per transport option variant

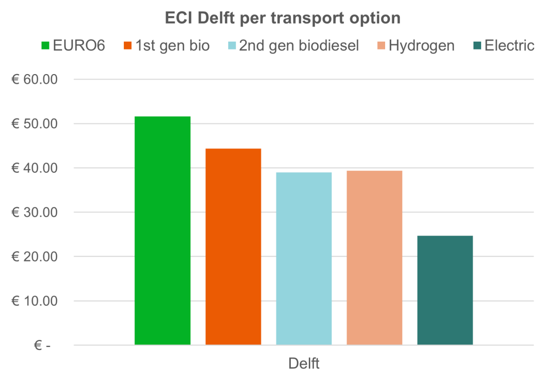


Figure 6.15: ECI value Delft per transport option variant

values remain consistent for Zwolle, but the quantities vary. What strikes is that the EURO 6 diesel truck results in a relatively high ECI value for GWP and HT. Besides that, the hydrogen truck has a relatively high HT value compared to the bio-diesel fuels and the electric transport. Overall, it can be concluded that using alternative transport like electric, hydrogen, or HVO100 bio-diesel generated fuel greatly saves on environmental impact in Delft. However, the ECI contribution for the transport phase (A4) is low compared to the ECI value of the design of concrete prestressed foundation piles (A1). Therefore, for Delft, the maximum ECI reduction by implementing electric trucks instead of EURO6 transport is €25.23, equivalent to a 6.5% reduction. For Zwolle, using electric transport results in €1.57, which means a 2.55% reduction. It needs to be noticed that these reduction values are based on the non-optimised C45/55 CEM I/III variants. By using the optimised concrete prestressed variants, the transport optimisation has a larger influence in lowering the total ECI value.

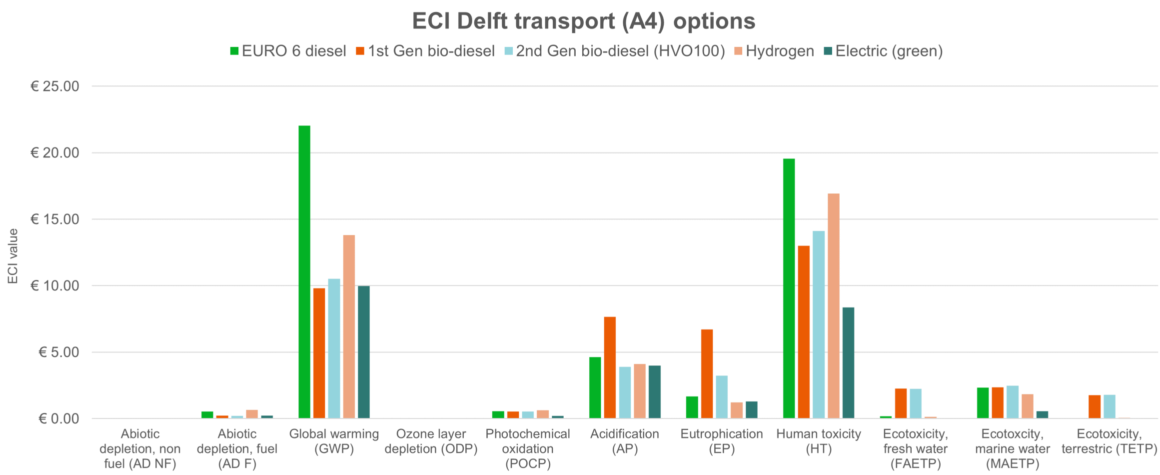


Figure 6.16: ECI value of transport variants Delft

Installation foundation piles

As seen in Figure 6.7, the construction and installation phase (A5) in Zwolle has the highest ECI contribution, which comes down to €12.37. This is caused by the heavy required diesel piling rig for the twelve foundation piles. Also, for Delft, the A5 phase has a €24.74 ECI contribution. However, because the ECI value in Delft is much higher than in Zwolle, the relative piling contribution is significantly smaller than in Zwolle. As mentioned in subsection 2.3.5, implementing an electric piling rig has significant potential. It is assumed that the piling in both Zwolle and Delft can be performed with the electric Junttan PMx2e rig. As previously mentioned in subsection 4.5.2, the Junttan has an operational weight of 68,000 kg and can handle piles with a maximum length of 20 meters. The timber part in Delft has a total length of 19.5 meters, and the timber part will be piled for the first few meters without a concrete cap. Therefore, the Junttan PMx2e has sufficient capacity for both the timber piles in Delft and Zwolle, and there is no need for installation with the heavier and more energy-demanding Woltman 90DRe. The Junttan PMx2e has a total battery capacity of 792 kWh, allowing continuous piling operations lasting 8 to 13 hours. The piling rig will be used for one hour in Zwolle and two hours in Delft. Consequently, a total of 75 kWh of electricity is required in Zwolle and 150 kWh in Delft. Once again, it is assumed that the piling rig will be charged with grey electricity due to the limited availability of green electricity resources around the construction site.

Figure 6.17 illustrates the ECI values per impact category for both the diesel and PMX2e piling rig in Zwolle and Delft. It can be concluded that replacing the diesel rig with an electric rig reduces harmful emissions, particularly in terms of AP, EP, and HT. The e-rig in Zwolle results in an ECI reduction of €8.80 to €3.57, which comes down to a total ECI reduction of no less than 38.3%. For Delft, the installation ECI will decrease from €17.59 to €7.15, which comes down to a 5.51% reduction. Besides the ECI reduction with the electric piling process, the implementation will also result in less local noise and emission hindrance, which is often an important requirement regarding the piling process. The disadvantages, however, are the high self-weight during transport and the high investment costs of the electric piling rig.

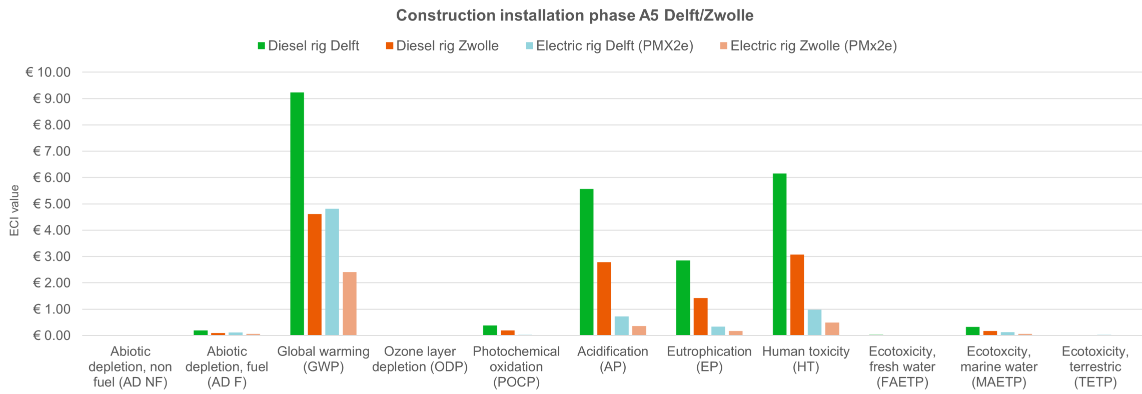


Figure 6.17: ECI value of installation with diesel and electric rig in Zwolle and Delft

6.5.2. Total optimised ECI value representation

Figure 6.18 and 6.19 depicts the total optimised values for respectively Zwolle and Delft. It can be noticed that the electric piling rig has the most considerable ECI potential in Zwolle. In Delft, using CEM III is the best optimisation regarding ECI. However, it needs to be noticed that the percentages in Zwolle are excessive due to the low ECI benchmark value because of the high negative ECI value of the C18 spruce. However, in Zwolle, the total ECI reduction due to optimisation is €14.67, which comes down to 63.25%, and in Delft, the total reduction is €83.33, which comes down to 26.2%.

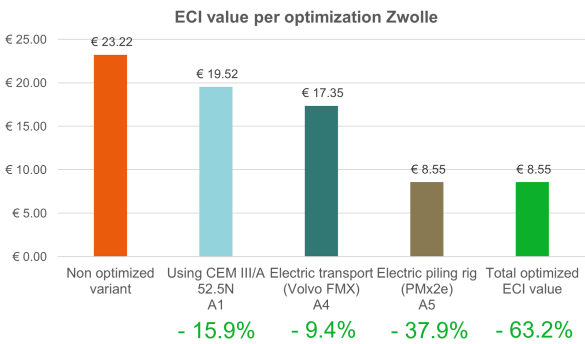


Figure 6.18: ECI reduction Zwolle timber pile, per each optimisation

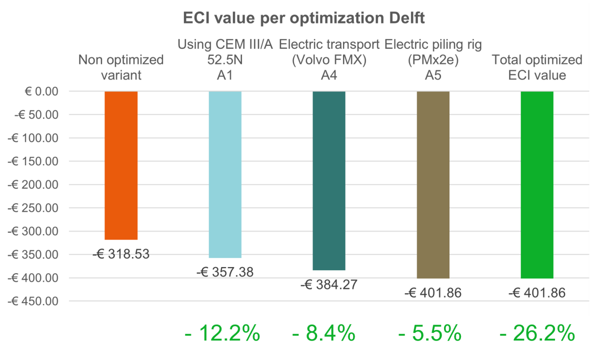


Figure 6.19: ECI reduction Delft timber pile, per each optimisation

6.6. Conclusion

Chapter 6 addresses the final foundation variant, the timber foundation pile with a concrete cap, which is investigated because of its high environmental reduction potential. The required pile lengths and diameters for Zwolle and Delft are determined while considering their bearing capacity. It is assumed that C18 spruce timber will be used for the foundation design due to its high availability and regular foundation application. Due to the relatively low stiffness (EA) in conjunction with the non-rigid superstructure of FLOW, rotational checks are critical for the timber variant. Consequently, 12 piles, each measuring 5 meters long and with a diameter of Ø 200 mm, are necessary for Zwolle. In Delft, piles measuring 23 meters long with an excessive pile top diameter of Ø 450 mm and a pile tip diameter of Ø 320 mm are needed. However, the rotation requirements could not be met in Delft due to the significant settlements of the piles and the small required distance between them. Nonetheless, it is assumed that the timber foundation design is feasible to investigate the ECI potential of longer timber piles in weaker soil conditions. Subsequently, the structural capacity of the timber piles with concrete caps and steel tube connections is analysed to ensure sufficient structural capacity. In Zwolle, the Ø 200 mm timber piles have a combined compression and bending UC of 0.82, while the Ø 450 mm piles in Delft have a UC of 0.20. The 4mm thick S235 steel tube connection between the concrete cap and timber pile provides enough capacity to transfer the required force in both Zwolle and Delft. The design of the C35/45 concrete cap with B500B regular reinforcement follows the guidelines of BRL 1721 and has been validated by the supplier. The structural calculations for the timber foundation design can be found in Appendix E.3.1. The technical foundation design drawings of Zwolle and Delft are depicted in Figure 6.4 and 6.5.

With the determined timber pile foundation design, the ECI values of both Zwolle and Delft are calculated and analysed. For the C35/45 concrete cap, both CEM I and CEM III concrete mixtures are used (Fig.4.11). For the manufacturing process (A3), the standardised 3.63 kWh electricity, 0.12 l diesel and 0.13 m^3 gas per cubic meter concrete with additional 20 kWh and 30 kWh for respectively, Zwolle and Delft are considered. The transport (A4) will involve using a single EURO6 diesel truck for the piles and the concrete caps. These trucks will cover a single distance of 82 km to reach both Delft and Zwolle. The installation (A5) of the twelve piles will be performed with diesel piling rigs for one hour straight in Zwolle and two hours in Delft. Moreover, it is assumed that the lifetime of the foundation is over 100 years, which means that a biogenic carbon storage of 715 $kgCO_2/m^3$ can be considered. Under these conditions, the total ECI value for the twelve timber piles in Zwolle amounts to €23 (CEMI concrete cap). Without considering CO_2 storage, the ECI value significantly increases to €67. In Delft, with CO_2 storage, the total ECI is €-319 (CEMI concrete cap), but it rises to €600 without storage. In both Zwolle and Delft, the timber component exhibits the most substantial ECI impact, as confirmed in Figure 6.8 and 6.12. Furthermore, it is notable that the inclusion of CO_2 storage profoundly affects the ECI value. In both Delft and Zwolle, this consideration results in a threefold increase in the ECI value.

With the calculated and analysed ECI values for the standardised dimensions, optimising the pile designs in both Zwolle and Delft is possible. Utilising an electric piling rig PMx2e in Zwolle and Delft results in ECI reductions of 38% and 5.5%, respectively. Transporting the piles and caps using the electric Volvo FMX truck leads to ECI reductions of 9.5% in Zwolle and 8.4% in Delft. Additionally, using CEM III instead of CEM I for the concrete caps results in a 16% reduction in Zwolle and a 12% reduction in Delft. Therefore, it can be concluded that even for the timber foundation variant, implementing CEM III has significant potential for reducing ECI. However, the drawback of CEM III is the slower hardening time compared to CEM I. Combining all optimisation options results in a €15 ECI reduction in Zwolle, amounting to 63% (Fig. 6.18). In Delft, this results in an ECI reduction of €83, equivalent to 26% (Fig. 6.19).

7 Foundation variant comparison

This chapter compares the three foundation variants on the ECI in section 7.1. Section 7.2 includes the foundation beams and wall. Moreover, to make a more comprehensive and all-encompassing comparison, the three variants will be compared in section 7.3 on other attributes besides the ECI.

7.1. Environmental impact comparison of the three variants

The prestressed concrete pile, the shallow strip foundation and the timber pile with concrete cap are elaborated and analysed in chapters 4, 5, and 6 respectively. However, to compare the three variants with each other, it is interesting to compare the ECI values of each variant in Zwolle and Delft. This way, the results can be compared, and a clear overview of the environmental impact can be displayed. The non-optimised and optimised foundation variants for Zwolle and Delft will be compared to achieve a fair and ambitious comparison. Figure 7.1 shows the ECI values of the three variants for the original non-optimised CEM I design, the CEM III design, and the total optimised design. In Zwolle, the twelve timber foundation piles with concrete caps have the lowest environmental impact. The six prestressed concrete piles have a significantly higher environmental impact, roughly three times higher than the timber variants. The shallow concrete strip foundation has the highest environmental footprint in Zwolle.

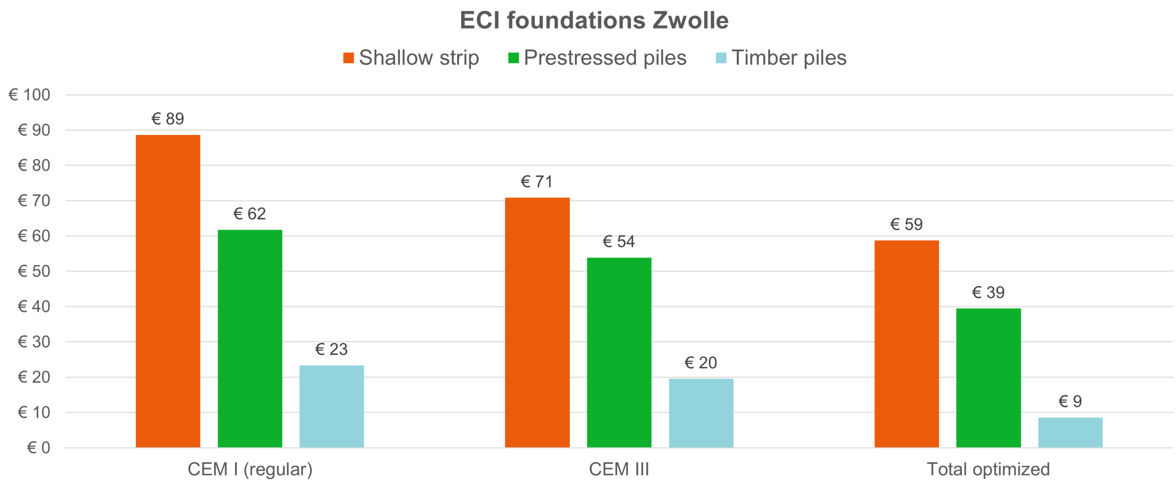
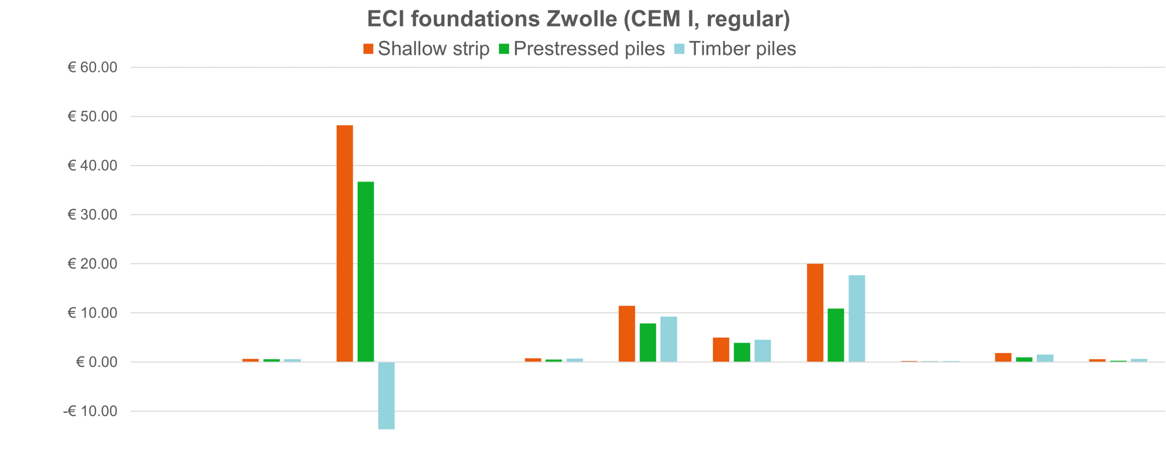


Figure 7.1: Total ECI value of the three variants in Zwolle, timber piles the lowest ECI

Figure 7.2 illustrates the ECI value of the three variants for the non-optimised CEM I designs per impact category. It can be derived that both the prestressed pile and shallow strip have roughly the same environmental impact distribution per impact category. The significant GWP impact originates from the large cement content in both concrete mixtures. The prestressed pile requires 3486 kg of concrete and 51 kg of reinforcement and prestressed steel, while the strip foundation requires 10375 kg of concrete and 120 kg of reinforcement steel. This significant difference is caused by the required 800x200 mm strip over the entire circumference to generate enough bearing capacity, resulting in a large amount of concrete. Because the forces in the strip are too large, a minimum amount of reinforcement must be applied. The six 220x220 mm prestressed concrete piles have a significant bearing capacity, resulting in a smaller amount of concrete. However, the difference between prestressed and strip ECI is limited because a C20/25 concrete mixture satisfies the shallow strip foundation, while higher concrete classes are required for the prestressed variant. The shallow strip also results in a higher HT because of the large amount of reinforcement steel, as depicted in Figure 7.2. The CO_2 storage of the timber pile variant can be seen in the negative GWP in Figure 7.2, which also results in the lowest environmental impact. The AP, EP and HT impact categories ECIs are relatively high. This is caused by the cement of the concrete cap and especially the timber sawing and treatment processes. From Figure 7.1, it can be concluded that implementing BFS instead of OPC for all variants is an effective measurement to lower the ECI value. Moreover, implementing all optimisation options results in a significant ECI decrease for all three variants. For the shallow strip, prestressed pile, and timber pile, a respectively 34%, 36%, and 63% ECI reduction can be achieved by implementing all optimisations.



	Abiotic depletion, non fuel (AD NF)	Abiotic depletion, fuel (AD F)	Global warming (GWP)	Ozone layer depletion (ODP)	Photochemical oxidation (POCP)	Acidification (AP)	Eutrophication (EP)	Human toxicity (HT)	Ecotoxicity, fresh water (FAETP)	Ecotoxcity, marine water (MAETP)	Ecotoxicity, terrestric (TETP)
Shallow strip	€ 0.00	€ 0.66	€ 48.21	€ 0.00	€ 0.79	€ 11.46	€ 4.96	€ 19.97	€ 0.19	€ 1.83	€ 0.59
Prestressed piles	€ 0.01	€ 0.56	€ 36.68	€ 0.00	€ 0.51	€ 7.86	€ 3.90	€ 10.89	€ 0.10	€ 0.97	€ 0.26
Timber piles	€ 0.01	€ 0.58	-€ 13.70	€ 0.00	€ 0.70	€ 9.21	€ 4.50	€ 17.67	€ 0.22	€ 1.51	€ 0.62

Figure 7.2: Total ECI per impact category of three variants in Zwolle

Because of the large diversity of soil conditions in the Netherlands, the three variants are also designed for the weaker soil in Delft. Figure 7.3 displays the ECI values of the pile variants in Delft.

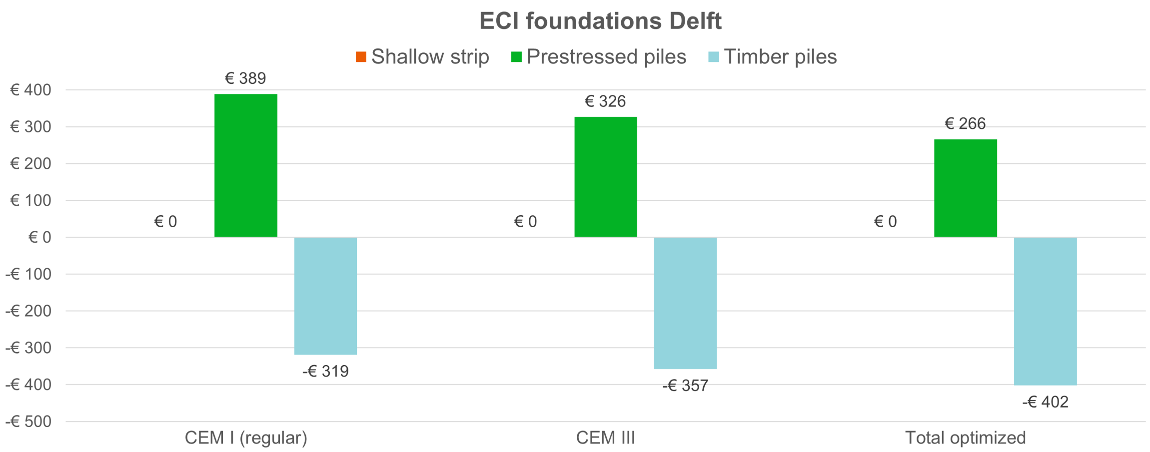
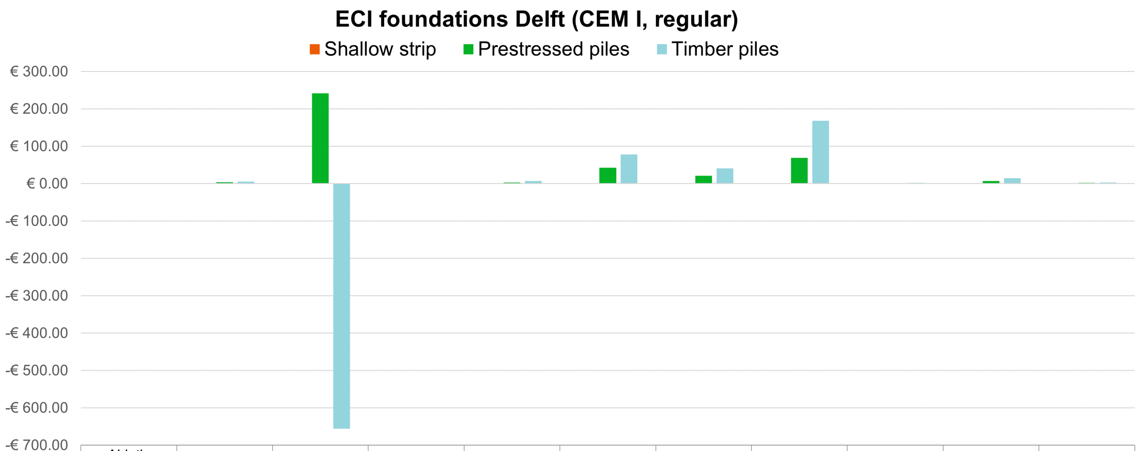


Figure 7.3: Total ECI value of pile variants in Delft

A shallow strip foundation in Delft is not feasible because of the weak soil conditions and, thereby the unallowed settlements and rotations of the strip. Even with a 1.5m width strip, the occurring settlements are too high to satisfy the maximum allowed settlements. Moreover, it must be mentioned that the timber pile foundation in Delft has an exorbitant pile diameter of 450 mm at the top and is, therefore nearly impossible to realise. From Figure 7.3, it can be seen that the ECI of the CEM I prestressed pile in Delft is €389, which is 5.3 times higher compared to Zwolle. This is caused by the required six 290x290 mm 23-meter-long piles, which have a total concrete weight of 27850 kg and reinforcement weight of 376 kg. However, by applying CEM III instead of CEM I, the ECI value can be reduced by 16%. However, for the twelve spruce timber piles with concrete caps, the ECI value is negative, which means that it positively impacts the environment. This is caused by the 15867 kg CO_2 storage in the twelve timber piles. This is also confirmed by Figure 7.4, where the large negative GWP of the timber piles can be seen. This CO_2 storage compensates for the ECI value of the other impact categories, which results in a negative ECI value. Therefore, it can be concluded that in the weaker soil locations in the Netherlands, a timber pile with a concrete cap has a large environmental potential compared to the regularly applied prestressed concrete pile foundations. Figure 7.3 also shows the potential of the optimisations in the regular applied prestressed concrete variant. Applying all optimisations given in section 4.5, the prestressed foundation in Delft can be reduced by €123, which comes down to a 31.6% reduction. For the timber piles with concrete caps, an ECI reduction of €83 can be achieved.



	Abiotic depletion, non fuel (AD NF)	Abiotic depletion, fuel (AD F)	Global warming (GWP)	Ozone layer depletion (ODP)	Photochemical oxidation (POCP)	Acidification (AP)	Eutrophication (EP)	Human toxicity (HT)	Ecotoxicity, fresh water (FAETP)	Ecotoxcity, marine water (MAETP)	Ecotoxicity, terrestric (TETP)
Shallow strip	0	0	0	0	0	0	0	0	0	0	0
Prestressed piles	€ 0.13	€ 3.30	€ 241.43	€ 0.01	€ 2.95	€ 42.40	€ 20.92	€ 68.31	€ 0.75	€ 6.83	€ 1.75
Timber piles	€ 0.09	€ 4.79	-€ 656.22	€ 0.02	€ 6.88	€ 77.96	€ 40.77	€ 167.51	€ 1.67	€ 14.16	€ 2.31

Figure 7.4: Total ECI per impact category of pile variants in Delft

Comparing the three foundation variants on ECI, it can be concluded that for both Zwolle and Delft, the timber foundation variant results in the lowest environmental impact due to the high amount of CO_2 storage. The prestressed concrete foundation piles have the mean environmental impact in both Zwolle and Delft. Surprisingly, the shallow strip foundation has the highest environmental impact in Zwolle and is not feasible in Delft. Due to the small bearing and structural capacity, a relatively large amount of concrete and reinforcement is required. Also, by optimising each variant, the timber foundation piles have the smallest environmental impact and the shallow strip foundation the highest. Moreover, the soil conditions of the foundation are of high importance because they considerably affect the bearing capacity and, thereby the dimensions and environmental impact. As discussed in section 3.2, the foundation has a minor part in the MPG value. In Zwolle, the MPG can be lowered by 3.3% to 0.56 by applying the optimised timber foundation piles. In Delft, the MPG can be reduced by 5.7% to 0.55 by using the optimised timber foundation piles. Therefore, as already expected, the MPG value requirement of 0.5 for houses in 2030 cannot be achieved by lowering the environmental impact of the foundation. Even in Delft, with the timber pile variant's negative ECI value and positive environmental contribution, the 0.5 MPG requirement can not be achieved by applying a sustainable foundation.

7.2. Foundation variants including beams and wall

This research does not include an extensive ECI analysis of the foundation beams and the strip wall because it was assumed that the foundation beams and strip wall had similar dimensions. However, the structural capacity analysis revealed a slight difference in the dimensions of the foundation beams and strip walls. Moreover, including the foundation beams and wall will result in a complete and all-encompassing assessment of the environmental impact of the entire foundation. Therefore, the environmental impact of the foundation beams and strip wall will be calculated in a simplified manner. The standard foundation beams are designed with a C30/37 concrete mixture, whereas the strip wall will be made from a C20/25 concrete mixture. The L foundation beams of FLOW have a total cross-section area of 0.22 m^2/m, while the 300x800 mm strip wall has a total area of 0.24 m^2/m. A reinforcement ratio of 1.00% is assumed for the foundation beams and 0.75% for the strip wall. The foundation beams require more reinforcement because of the larger reaction forces during transport. However, because the reaction forces are limited, a small amount of reinforcement is needed for both the foundation beams and the strip wall. It is assumed that the foundation beams are prefabricated and transported with EURO6 trucks, and the strip wall will be cast in situ, where the concrete transport will be performed by a mean-sized truck mixer. The total volume of concrete for the foundation beams will be 6.16 m^3, and for the strip wall, 6.72 m^3. The total amount of B500 reinforcement will be 485 kg for the foundation beams and 365 kg for the strip wall. It is assumed that the foundation beams in Zwolle and Delft are similar. Moreover, it is assumed that the foundation beams and strip walls will be recycled after service life because of the relatively easy removal process. The total estimated ECI value of the C30/37, CEM I foundation beams for the prestressed and timber pile variants is €165. The ECI value of the C20/25, CEM I strip wall for the shallow strip foundation is calculated at €150. Figure 7.5 shows the ECI values of the foundation beams and strip wall for the non-optimised CEM I designs, the CEM III designs, and the total optimised designs, including electric transport. It must be noted that the total optimised value of the foundation beam and strip does not include the concrete class and reinforcement optimisation.

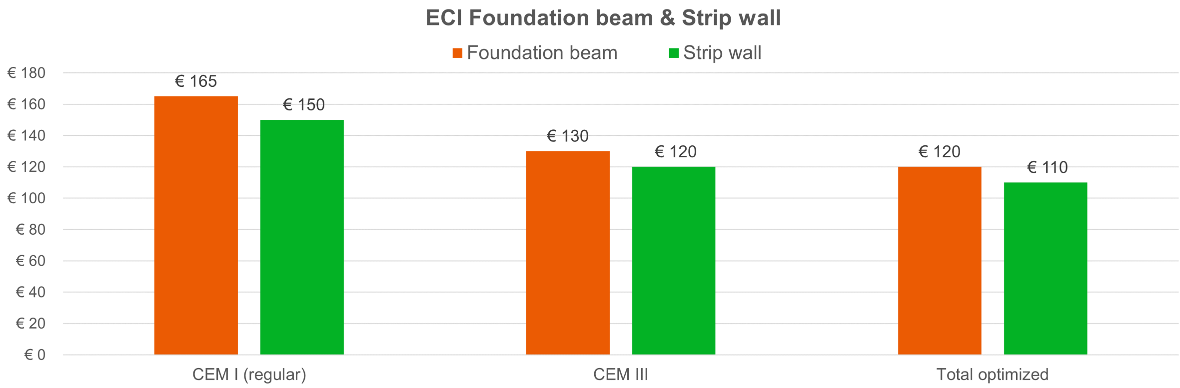


Figure 7.5: Total ECI foundation beams and strip wall in Zwolle and Delft

From Figure 7.5, it can be concluded that the difference between the environmental impact of the foundation beam and the strip wall is limited, primarily due to their similar design dimensions. It is important to note that the shallow strip wall has a slightly lower ECI value than the foundation beams. This difference is attributed to the additional required reinforcement for the prefab foundation beams. Considering the ECI values of both the foundation beams and the strip wall, the environmental impact of the total foundation can be displayed for Zwolle and Delft. The prestressed and timber pile variants include the foundation beam, whereas the shallow strip foundation includes the strip wall. Figure 7.6 shows the combined environmental impact of the foundation variants in Zwolle. It can be seen that the environmental impact of the variants, including beams and walls, are much more levelled compared to the environmental impact of the individual foundation piles and strip as shown in Figure 7.1. This is caused by the relatively high environmental impact of the strip wall and foundation beams. It can also be inferred that, for Zwolle, both the foundation beams and strip wall have a higher ECI value than the piles and shallow strip. Therefore, optimisation through the use of CEM III and electric transport for the foundation beams and strip wall results in a significant reduction in the total environmental impact of the foundation design. In conclusion, the optimised shallow strip foundation, including the wall in Zwolle, continues to have the highest ECI at €169, followed by the prestressed pile foundation, including beams at €159, and finally, the timber foundation piles, including beams at €129.

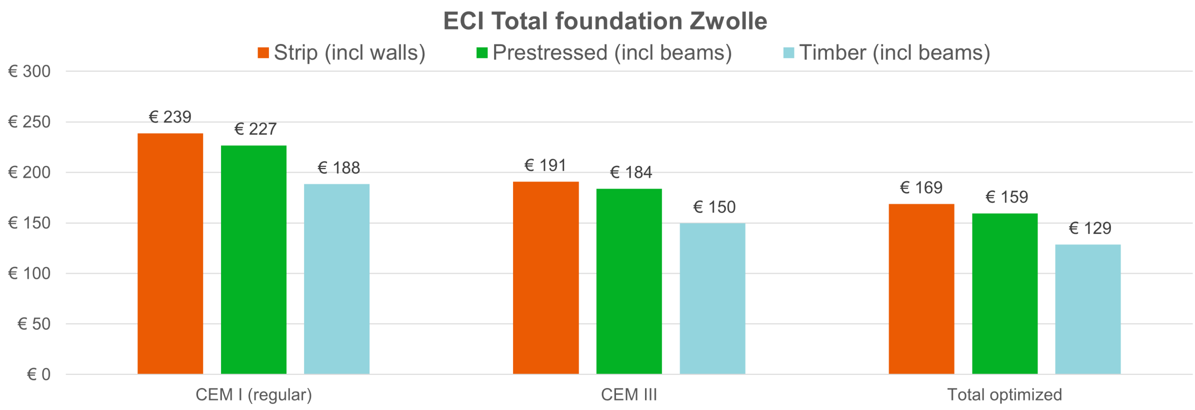


Figure 7.6: Total ECI of variants incl beams and strip wall in Zwolle

Figure 7.7 shows the total ECI value of the prestressed and the timber piles, including foundation beams in Delft. The shallow strip variants are not feasible in Delft because of unacceptable soil settlements. Because both pile variants have the same foundation beam design, the ratios between the timber and prestressed pile remain similar. Moreover, it can be concluded that in Delft, the piles have a higher ECI value than the foundation beams. The timber foundation variant, including foundation beams, still results in a negative ECI value in Delft and thereby positively contributes to the environment. The optimised ECI value of the prestressed piles including foundation beams results in €266, and the timber piles including beams in -€282.

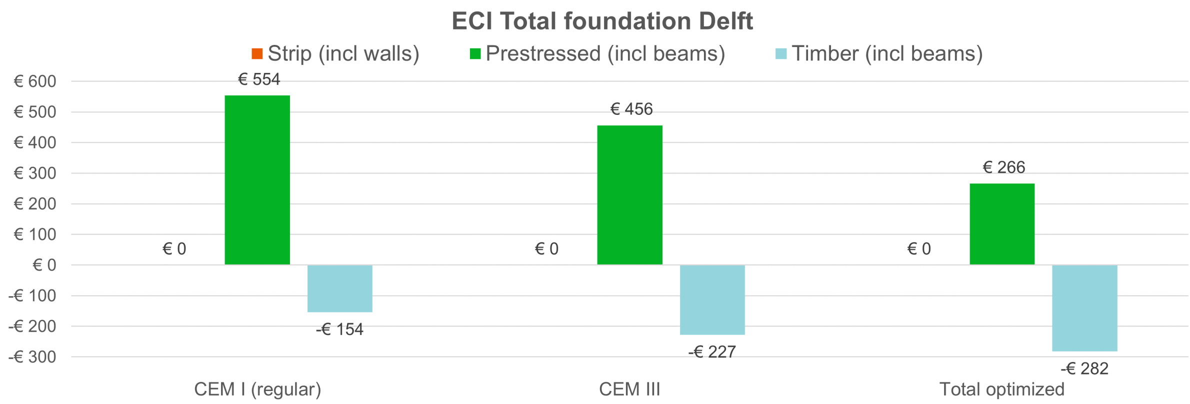


Figure 7.7: Total ECI of variants incl beams and strip wall in Delft

7.3. Foundation variants characteristics

As discussed in section 3.4, a foundation must meet several conditions and requirements. This section will, therefore, show the different characteristics of the three foundation variants and the pros and cons of each design. This way, a more comprehensive and all-encompassing comparison between the three variants can be made instead of only focusing on the environmental impact. Table 7.1 shows the score of the three variants on the essential aspects of a foundation design as discussed in subsection 2.3.1. Per each important foundation criterion, a value of - - to ++ is given to the three foundation variants. A score of - - means very bad, a - means bad, a +- neutral, a + good, and a ++ very good. The importance of the criteria depends on the specific soil conditions and situations; therefore, a similar weight factor is given to each criterion.

	1. Prestressed pile	**2. Shallow strip**	**3. Timber pile with cap**
Environmental impact	+-	-	++
Costs	+	++	-
Complexity	++	+	+
Noise	- -	++	-
Vibrations	- -	++	-
Settlement risks	++	-	-
Bearing capacity	++	-	- -
Tension capacity	++	- -	- -
Transport	- -	+	-
Construction	- -	++	+-
Recycling potential	- -	++	+
Lifetime	++	++	+
Production time	-	++	-

Table 7.1: Characteristics of the three foundation variants, ++ very good and - - very bad

The first and, for this research, the most crucial criterion is the environmental impact. The timber pile scores best because of the positive environmental impact, followed by the prestressed pile and shallow strip. Another highly importance criterion is the costs of each foundation. The shallow strip has the lowest costs because of the simplicity and lack of prefab processes and heavy machinery, followed by the prestressed prefab pile, which is highly standardised and cost-efficient. The timber pile with concrete cap has a bad score because of the high price of large required timber dimensions. Another important criterion is the complexity of the foundation, where the prestressed pile has the best score because of the frequent applications and experiences. The shallow strip foundation is more complex because of the required detailed soil conditions and more unpredictable settlements. The same counts for the timber pile, which is not often applied and, therefore, less experience and knowledge. An important advantage of the shallow strip foundation is the absence of noise and vibrations during construction, often an essential criterion in the foundations' design. The pile foundations have the score very bad and bad, because of the considerable noise and vibration pollution during the piling process. However, because the timber pile requires a less heavy piling machine, the noise and vibrations are slightly less than the prestressed pile foundation.

Another essential criterion for foundation is the risk for settlements. Because of the large bearing capacity of the prestressed foundation pile, the settlements are minor and, therefore have a very good score. For the shallow strip, the bearing capacity is limited and detailed knowledge of soil conditions is of great importance, which makes the strip more prone to settlements. Because of the timber piles' limited bearing capacity and relatively low stiffness, the settlement risk for the timber piles is also significant, especially in combination with a highly fluctuating GWT. Sometimes, tension forces appear in the foundation, which can not be transferred by both the shallow strip and (tapered) timber pile and, therefore has a very bad score. In contrast, the prestressed foundation pile has a high capacity for transferring the tension forces. The shallow strip foundation has a good score for the required transport because only a mean-size truck mixer is needed, while a heavy-weight flatbed truck is necessary for the prestressed pile. The timber piles also need a large-capacity truck. However, the weight of the piles is limited compared to the concrete piles. The pile foundations have a low score for the construction because of the required heavy piling rigs. The construction of the shallow foundation scores very good because it only required a concrete pump with a compaction needle. Because the pile foundation often remains in the soil after a lifetime or requires a heavy machinery process, the piles' recycling is less effective than the shallow strip foundation. Besides that, the soil conditions may change when removing the pile foundations because of the created cap. Moreover, the timber foundation pile needs to remain in the soil to ensure the CO_2 of the timber. Therefore, the prestressed pile has the worst recycling score and the strip foundation's highest score. The foundations are designed for 75 years; however, because of the higher degradation risk of the timber pile with a concrete cap, it may be argued that after 75 years, the lifetime of the timber pile is less compared to the concrete variants. Finally, the production time of the three variants is classified. The production time is relatively long because of the 28-day hardening time of the prestressed foundation pile and the required heavy transport. The same counts for the large dimensions timber piles, which are not harvested in the Netherlands and, therefore, often require a long ordering process. The strip foundation only requires the highly available concrete mixture and standardised reinforcement steel, which makes the production time short.

Table 7.1 shows that each foundation variant has its characteristics and pros and cons. Considering the foundation's environmental impact, the timber foundation pile with a concrete cap has the best score. Besides that, it can be concluded that the shallow strip foundation has a good score on construction, installation, noise, vibrations and recycling potential. However, it has a bad bearing/tension capacity score and high risk on settlements. Moreover, the shallow strip foundation can not be implemented in many places in the Netherlands. The often-used, and therefore non-complex prestressed pile, has a high capacity but also causes vibrations and noise pollution due to the heavy transport and piling rig. Besides that, the piles can not be recycled easily and the production time of the prefab process. The timber foundation pile with concrete cap has a high environmental reduction potential but also comes with high costs and noise and vibration pollution. Moreover, the bearing capacity is limited and is not feasible in every location. However, the timber foundation piles also have a high recycling potential without disturbing the soil conditions heavily. From this section, it can be concluded that each of the three variants has its advantages and disadvantages and that the choice for the foundation design depends on the boundary conditions and requirements of the specific project.

8 Discussion

This chapter is dedicated to the key findings and notable observations concerning the design and environmental impact of the three foundation variants: the prestressed concrete pile, the shallow strip foundation, and the timber foundation pile. The environmental impact assessment has been established through a comprehensive LCA study, utilising available environmental data. Numerous boundary conditions and assumptions have been employed in developing these design variants and the LCA study. These assumptions and scope limitations significantly affect the research outcomes and their reliability, and thus, will be thoroughly examined and discussed within this chapter.

8.1. Foundation design

Soil characteristics

The foundation variants' bearing capacity in this research is determined on a single available CPT for both Zwolle and Delft. As a result, the correlation factors ξ_3 and xi4 are 1.39 (NEN 9997-1+C2:2017 Table A.10). In all other housing projects, multiple CPTs are performed. By conducting multiple CPTs, the correlation factors can be reduced. By conducting four CPTs, the ξ_3 and ξ_4 factors reduce to 1.28 and 1.03 for non-rigid superstructures. It must be mentioned that conducting more CPTs can result in smaller pile dimensions and shallow strip widths, reducing the environmental impact. Moreover, conducting multiple CPTs instead of one is not a labour and environmental-intensive task. Therefore, it can be concluded that performing multiple CPTs is the starting point for reducing environmental impact. In this way, a more detailed soil profile can be obtained, resulting in less high correlation factors and a lower ECI value. Besides that, conducting more CPTs yields a more comprehensive and detailed soil analysis, consequently reducing technical uncertainties and risks.

For the design of the shallow strip foundation in Zwolle, a small CPT adjustment is made, as elaborated in Appendix D.1. This is because the CPT in Zwolle does not consist solely of sand layers in the top few meters. As a result, a shallow strip foundation would not be possible when considering the original CPT of Zwolle because of unacceptable settlements due to the present thick, weak loam layer. However, to investigate the environmental impact of a shallow foundation, a 0.5-meter sand layer at a 1.5-meter depth is assumed instead of the 0.5-meter loam layer. Because the prestressed and timber pile foundations are designed with the original Zwolle CPT, it may be argued that this will result in an unequal comparison between the variants. However, the CPT improvement has a negligible difference for the pile foundation designs because of the limited change in bearing capacity at the deeper-located piles. The 0.5-meter additional sand layer at a depth of 1.5 meters only influences the bearing capacity of the soil to a depth of 3.5 meters, as depicted in Figure D.3. The sand soil improvement results in similar pile dimensions, located at the same depths as the original CPT in Zwolle. Therefore, the slight change in CPT does not influence the pile foundations in Zwolle. However, it must be remembered that a shallow foundation in Zwolle would not have been feasible with the given original CPT due to unacceptable settlements caused by the relatively thick loam layers.

Another important discussion point about the soil characteristics is that the three foundation designs are based on two representative soil profiles in the Netherlands, mainly consisting of clay, sand and loam. However, the soil conditions in other parts of the world are completely different, greatly influencing the foundation designs and corresponding environmental impact. This also results in the fact that there is not one fit-for-all sustainable foundation design which can be applied in all situations. Because of the different soil conditions per location, having one reproducible and scalable ECI value for the foundation is impossible. A single prefab foundation pile value, expressed per meter length, is often used in the different MPG calculation tools. Therefore, it must be noted that this is a rough estimation and that for a more detailed MPG value for the foundation, at least the foundation piles' length and cross-section dimensions must be included. Besides that, it must be mentioned that the MPG value of the FLOW building can not be expressed in one detailed single value because of the highly different foundation dimensions per location in the Netherlands. This is also confirmed by this research, where in Delft, a 5.3 times higher ECI value for the prestressed piles is calculated compared to Zwolle. As a result, giving one single and detailed MPG value for FLOW in the Netherlands is impossible. It also must be noticed that in the vicinity of Delft, the soil is highly unfavourable for foundation designs, as depicted in Figure 2.4. Consequently, the ECI outcome in Delft represents one of the worst-case scenarios concerning soil conditions, and many locations in the Netherlands could yield a smaller environmental impact. Lastly, this research does not consider earthquakes on the foundation design and does not include the feasibility of each variant in earthquake-prone regions, such as Groningen.

FLOW house

The timber FLOW superstructure must be classified as a non-rigid structure. This is because the concrete foundation beams undergo deformations exceeding 5 mm if a single foundation pile were to be removed. Additionally, the FLOW house lacks robust walls above the foundation beams, which would otherwise be capable of transferring loads to the other foundation piles. Therefore, FLOW must be considered as non-rigid. However, designating the FLOW house as a non-rigid structure carries significant implications for the dimensions of the foundation piles. This is because if the superstructure is assumed to be rigid, discrepancies in settlement might be disregarded. Consequently, the often-critical rotations requirement could also be neglected (as specified in NEN 9997-1:2016, Article 6.6.2 (C)). Because the rotational check was often the most critical verification in determining the pile dimensions, this greatly affects the foundation designs and dimensions and the corresponding environmental impact. Therefore, it may be interesting to investigate if the FLOW superstructure can be made rigid easily. This way, the foundations' dimensions may be reduced, resulting in a lower ECI. However, this will be only effective if the additional measurement to make the superstructure rigid, will not come with many additional materials and thereby high environmental impact.

For the determination of the forces on the foundation piles and the shallow strip, it is assumed that foundation beams and strip wall connection will not re-distribute the forces on top of the foundation beam. Therefore, the point and distributed forces mentioned in section 3.1.3 will also act on the foundation. This assumption is conservative because the relatively stiff foundation beams will re-distribute the forces to a more equally distributed force. Especially for the shallow strip design, this has an effect because concentrated point loads are the reason for the relatively high moments in the strip. Therefore, it may be interesting to investigate to what extent the foundation beams or walls redistribute the forces and to look into the possible moment reductions.

Foundation beams and strip wall

In chapters 4, 5, and 6, the discussion on foundation variants omitted the consideration of foundation beams and shallow strip walls, assuming that foundation beams could also serve as strip walls due to their presumed similar dimensions and configurations. However, the structural capacity analysis later revealed slight disparities in the design of foundation beams and strip walls. Consequently, in section 7.2, an environmental impact assessment was conducted for these components in a simplified manner. The analysis indicated a slightly higher ECI for foundation beams, attributed to the additional required reinforcement compared to strip walls. Including foundation beams and strip walls in Zwolle's overall assessment of foundation variants led to a more balanced comparison, as these elements exhibited a relatively high environmental impact. Nevertheless, even with the inclusion of foundation beams and strip walls, the shallow strip foundation retained the highest ECI value. Given the limited reaction forces and the relatively high ECI value in Zwolle, there is an interest in exploring alternative, environmentally friendly materials for strip walls instead of the conventional use of concrete. However, In Delft, foundation beams and strip walls are subordinate to piles and strips due to their relatively small ECI values. Furthermore, given the approximate dimension similarity between the foundation beams and the strip wall, it becomes worthwhile to investigate whether they can serve a dual purpose as strip walls. This dual function could potentially enhance standardisation within the foundation design of FLOW. This will also enhance the reuse potential of the foundation beams and strip walls because they can be reused for both shallow strip foundations and pile foundations.

Dynamic force during piling process
An essential aspect of the piling process is the dynamic force of the piling block on the pile foundation and the capacity of the piling head. This is also why the head and base spiral reinforcement is applied to the foundation piles, according to the BRL 2352. However, with the prestressed prefab pile optimisation, the concrete strength class is reduced to C35/45 instead of C45/55. The EN 12794:2005 + A1:2007 standard mentions that the reinforced or prestressed precast foundation piles shall be at least C35/45. Therefore, it is assumed that the pile head and reinforcement can transfer the dynamic forces of the piling rig. However, additional research and calculations need to be performed to ensure that lowering the concrete class to C35/45 will not result in exceeding pile head failures. If reducing the concrete class increases pile failures during piling, the ECI optimisation will be ineffective and may even result in a higher ECI due to pile replacements.

Building materials
This research uses traditional and regularly applied building materials like concrete, timber and reinforcement steel. This is done to achieve more practical and applicable studies for the foundation industry and BAM. However, alternative and low-environmental potential materials are not considered for this research. For example, lightweight foam concrete or biobased materials in the shallow strip foundation may have a low environmental impact.

8.2. Environmental data

Another important discussion aspect concerns the utilised environmental data and the outcome. Environmental data about processes or EPDs are often not (freely) available. Additionally, when ECI data is published, it frequently lacks transparency regarding the underlying assumptions and origins. This limitation extends to data sourced from the NMD, where the single ECI values are often provided without a comprehensive description of the associated product or process. This lack of detail makes it challenging to use the data effectively, given the potential for high data inaccuracy within the specific application context. Improving the accessibility of environmental databases through increased public availability, coupled with comprehensive documentation of the underlying data and assumptions, would facilitate a more precise and detailed assessment of the ECI value for an LCA study.

LCA study
An important discussion point about the ECI results for the different variants is the LCA-study assumption. This research does not include the transport phase (A2) for all the variants. Therefore, it must be noted that this research's outcome can not be compared one-to-one with other foundations ECIs that include the A2 phase. The A2 transport is not included because determining the transport of all the raw materials towards the manufacturing fluctuates per specific foundation manufacturer.

The End-of-Life and Beyond-life cycle stages are included in the LCA for the shallow strip foundation because of the relatively easy removal and minor soil changes. This results in an ECI value of -€4.00 for the non-optimised strip. However, considering these phases results in an ECI increase of €3.50 for the optimised CEM III variant instead of a reduction as expected. This is caused by the high ECI values of the transport (C2) and waste processing (C4). Consequently, it can be deduced that recycling the optimised shallow strip foundation yields a higher environmental impact compared to leaving the entire strip foundation in the soil. As a result, it is not suitable to directly compare the ECI values of the optimised shallow foundation with those of the pile foundation, as they do not encompass phases C and D. To ensure a dependable and unambiguous comparison, the optimised design variants are compared without phases C and D. An important recommendation is to enhance the ECI efficiency of foundation recycling. This could be achieved by increasing the negative disposal values or introducing additional environmental factors for structures that remain in the soil or cannot be recycled. Another option could be to consider that the pile foundations are recycled and theoretically include the C and D phases in the ECI calculations. Particularly within a circular construction context, structures that are not or cannot be recycled should include an additional environmental impact factor. This approach would render recycling the shallow strip foundation more environmentally advantageous than the current situation, where recycling results in a higher ECI value than leaving the structure in the soil, which is not conducive to the principles of a circular building sector. It is worth noting that this research does not address the re-use of the foundation, by placing a new superstructure on top of the foundation after its service life. Given the foundation's potentially long lifetime, this re-use approach could present an interesting and environmentally optimal solution.

Reuse shallow strip foundation

The environmental impact assessment indicates that the shallow strip foundation exhibits the highest environmental impact, primarily due to the substantial volume of required concrete and reinforcement. Although a notable advantage of shallow strip foundations over pile foundations is the relative ease of removal and recycling post-service life, the environmental benefits are limited. This is attributed to the substantial energy consumption during waste processing and transport, outweighing the advantages of recycling. This research does not consider the reuse of the shallow strip foundation, which has a considerable environmental impact potential. Especially if the strip would be made from standardised smaller prefab dimensions, the shallow strip could have a considerable re-use potential for different applications on the stronger layers in the Netherlands. Given that the product stage (A1-A3) often contributes the most to the environmental impact in foundation design, focusing on reuse could be an effective strategy. Reusing foundations would involve considering only the relatively small ECI value of the removal and replacement processes (C1-C4). Shallow strip foundations could offer substantial environmental benefits for temporary or high circularity requirements projects. The feasibility of reusing a shallow strip foundation is further confirmed by a reference project involving a temporary timber school building in Amsterdam. The school, featuring a lightweight timber construction, was easily placed on a raised sand platform. Consequently, the prefabricated shallow concrete foundation construction is both movable and reusable [17]. It must be noted that the reuse potential of the shallow strip foundation is not quantified in the environmental impact of the foundation variants in this research.

However, it is essential to note that no specific reuse policy has been established for the superstructure of FLOW. Since the foundation is the only dynamic part of the standardised FLOW design, emphasising the reuse of the superstructure may be more logical. Ideally, the foundation variants would be reused after their service life by placing a new structure on top of the existing foundation. This approach minimises the environmental impact, eliminating the need for removal and replacement energy. The first crucial step is to create clear and detailed documentation about the foundation design to optimise the reuse of foundation variants after their service life.

Category 3 data

An important note regarding the environmental data used, as provided in Appendix F, is that many processes fall into the category 3 data classification. Category 3 data typically represents industry averages that lack verification, making it inherently less accurate. To account for this inherent inaccuracy, an additional 30% surcharge must be applied. Processes such as transport and the piling process prominently feature category 3 data. It is, therefore, imperative to acknowledge that the ECI results carry a certain degree of inaccuracy and emphasise the need to develop more category 1 data in the future. This would contribute to a more detailed and precise analysis and comparison. It is worth noting, however, that materials with the highest environmental impact, such as cement and reinforcement steel, are classified as category 1 data. Furthermore, in the case of the diesel piling rig, a singular category 3 data value is employed. Consequently, this data does not differentiate between various sizes or more energy-efficient piling rigs. Acknowledging that the environmental data related to the piling rig, represents an approximation and should not be construed as a detailed outcome is essential.

Transport data
The environmental data regarding the transport of the foundation piles is quantified in terms of tons per kilometer. Consequently, when the cargo weight is doubled, the environmental impact during transport also doubles. These assumptions provide a reasonable estimation for truck usage and align with a Spanish department research [26], where a truck loaded with 23,500 kg has a consumption of 38 liters per 100 kilometers, and a truck loaded with 16,000 kg has a consumption of 25 liters per 100 kilometers [26]. However, in the case of the timber foundation piles, the combined weight of the twelve timber piles amounts to 2537 kg. Due to the relatively low weight of the timber piles themselves, the ECI value attributed to their transport remains minimal. Therefore, it can be argued that the estimation based on tons per kilometer might result in a small ECI underestimating for low-weight transport due to the linear-based transport unit.

Timber data
The category 1, 2023 spruce timber data originates from Centrum Hout and is based on a Swedish-sawn spruce EPD. An important note for the timber data is that all wood is assumed to be harvested sustainably. The wood in the studied system thus fulfils the criteria of biogenic carbon neutrality over its life cycle. It is assumed that the timber foundation piles remain in the soil after the 75-year service life and therefore have a lifetime of more than 100 years. In that case, the sawn dried timber has a GWP value of –577 $kgCO_2/m^3$, which includes biogenic carbon storage of 715 $kgCO_2/m^3$. If the timber piles are removed, and the 100-year service life is not achieved, the environmental impact of the timber increases immensely. This is caused by the fact that the environmental data includes the burning and disposal values already in the A1 phase, which makes the ECI value of the timber high compared to other building materials. However, according to the LCA study, this is not allowed because the end-of-life data should be included in the C phase and not already in the raw material supply stage (A1). Therefore, considering only the LCA A-stages, the ECI data of the timber pile variants without CO_2 storage are extremely high because of the modelled CO_2 release in the raw material supply stage. Therefore, it is suggested to redistribute the environmental timber data over the right LCA stages to ensure a fair comparison between the timber and other building materials.

Finally, examining the timber foundation piles ECI values in Zwolle and Delft led to the observation that increasing the amount of timber results in a reduced environmental impact. However, clearly distinguishing the material use and environmental impact is essential. Without such clarity, there is a risk of needlessly enlarging the foundation pile's dimensions to lower the environmental impact. Nevertheless, it is crucial to minimise the utilisation of building materials to mitigate the environmental impact. However, comparing the larger required pile dimension in Delft, the timber foundation pile has a considerable negative ECI value due to the large CO_2 storage, making the use of timber significantly more beneficial than concrete.

8.3. Validity and application
This research gives an environmental impact representation of three FLOW foundation variants. However, because detailed environmental data is not (yet) widely available, specific assumptions about the foundation design and data are made, as mentioned above. These assumptions result in a detailed LCA of the foundation design of the FLOW building in Zwolle and Delft. However, this also results in the limitation that this LCA is specially made for the specific location with corresponding assumptions. Therefore, the results of this study can not be reproduced one-to-one for other lightweight house structures in the Netherlands. Because the environmental data is not widely and detailed available, the results of this research may deviate slightly from the exact environmental impact. However, making explicit and elaborate assumptions is intended to limit these inaccuracies. This study shows a clear difference between the three foundation variants for BAMs FLOW house: the prestressed timber pile, the shallow strip and the timber foundation pile. Moreover, each foundation design is optimised to lower the environmental impact. The optimisation results can be used for almost every (housing) project foundation design. Therefore, besides the ECI comparison between the three variants and the advised application of the timber foundation piles, this research can also be used as a reference by the (prefab) foundation industry to lower the environmental impact of their foundation designs.

9
Conclusion

This research aimed to develop a more sustainable foundation design for the lightweight FLOW timber house of BAM. Three foundation variants are analysed and optimised on the environmental impact for a location in Zwolle and Delft. The main research question of the research is as follows:

"How to reduce the ECI/MPG value of the timber FLOW housing units by implementing a low environmental impact foundation and installation method?"

The environmental impacts of three foundation variants are analysed and optimised for two representative soil locations, Zwolle and Delft. The foundation variants are a prestressed concrete pile, a shallow concrete strip and a timber foundation pile with a concrete cap. From the performed LCA study on the three foundation variants, the timber foundation pile with concrete cap in Zwolle results in the lowest ECI. Due to the large CO_2 storage and thereby negative GWP of the C18 spruce, the ECI value is calculated at €23 and -€402 for respectively Zwolle and Delft. The ECI can be lowered using BFS (CEMIII) instead of OPC (CEMI) as a cement type for the concrete cap. Additionally, significant ECI reductions are attainable through the utilisation of electric truck pile transport and electric piling rigs. Applying the above optimisation options, the timber foundation ECI can be reduced by 63% in Zwolle and 26% in Delft.

The commonly-applied prestressed prefab foundation has an ECI value of €62 in Zwolle and €389 in Delft. The substantial ECI contribution from high-cement content makes the application of BFS instead of OPC the most effective reduction. This is followed by using electric transport in both locations and adopting an electric installation process. Reducing the concrete strength class to C35/45 and applying an optimum reinforcement diameter slightly reduces the environmental impact. Combining all optimisation options results in a total ECI value of €39 in Zwolle and €266 in Delft.

The ECI value associated with the shallow concrete strip foundation amounts to €89 in Zwolle. Surprisingly, this relatively high ECI value can be attributed to the substantial volume of concrete required due to the extended length of the strip foundation. Due to unallowed high settlements in Delft, the shallow concrete strip foundation is only feasible in Zwolle. A non-reinforced strip design results in an increasing ECI compared to the minimum reinforcement strip due to the increasing strip height. The C20/25 strip foundation greatly benefits in not requiring heavy machinery or resulting in heavy noise and vibration hindrance. The strip ECI can be lowered by 20% by using CEM III/B 42.5N instead of CEMI 42.5N. Moreover, using electric transport, the total optimised ECI strip foundation value in Zwolle results in €59.

The LCA results show a significant ECI difference between the design in Zwolle and Delft due to the different soil characteristics. Building on stronger soil layers leads to a foundation with a significantly lower environmental impact, as evidenced by the observed differences in ECI between Zwolle and Delft. Furthermore, these results underscore that adopting the more environmentally friendly BFS instead of OPC substantially reduces the ECI. Lowering the concrete class with optimum reinforcement and implementing electric transport and piling shows a significant ECI reduction for every foundation variant. The 0.5 MPG requirements for housing projects can not be achieved by applying low-impact foundations because of the limited MPG contribution of foundations. Besides the environmental analysis, the foundation variants' characteristics showed that each variant has advantages and disadvantages and that a foundation must be chosen per specific project. However, to lower the environmental impact of a house foundation, the timber foundation piles show the highest ECI potential and even result in a positive environmental impact in Delft.

9.1. Recommendations

For the NMD, it is recommended to enhance the accessibility of their national environmental databases by making them more publicly available, accompanied by comprehensive documentation of the underlying data and assumptions. This approach would enable a more precise and detailed assessment of the ECI value for a given product or process, thus contributing to greater accuracy in environmental impact evaluations.

For the Dutch government institutions, which make the policy regarding the MPG requirement of 0.5, dividing or relocating the building services, like electricity and plumbing, from the MPG calculations, is recommended. This is advised because the building services have a >50% contribution to the MPG value, making optimising the structure elements, including the foundation, less effective and therefore encouraging. Therefore, it is recommended to subdivide those two aspects and to set MPG requirements for both the technical building services and structure elements. Because of the different soil conditions in the Netherlands, a foundation has less impact in the stronger eastern parts of the Netherlands compared to the weaker western part. Furthermore, in the eastern part, the areas have lower population density, and the soil is less expensive. Therefore, it is recommended to focus more on developing houses in the eastern part of the Netherlands.

The department 'sustainability' of NVAF is recommended to focus more on the foundation pile design instead of the transportation and installation. This research showed that the highest ECI reduction can be achieved by optimising the foundation design itself, while the NVAF focuses more on the sustainability of the installation and transportation. Moreover, for the different prefab foundation manufacturers, including the environmental impact in their foundation design, standardisation is recommended. The prefab industry has a high level of standardisation. However, this also often results in a higher ECI value than necessary. Therefore, focusing on a better balance between standardisation and environmental optimisation is advised for prefab (concrete) manufacturers.

Finally, it is advised for BAM to consider the timber foundation piles in their FLOW house building. The timber foundation piles with concrete caps show their environmental potential and even result in a positive environmental contribution in Delft. Therefore, it is recommended to investigate the technical difficulties and possibly increasing building costs. The MPG value of FLOW can not be expressed in one single value because of the highly different foundation dimensions per location in the Netherlands. Therefore, it is advised to use a weak-based soil conditions and large foundation dimensions for the MPG determination of FLOW. In this way, it can be guaranteed that all FLOW houses in the Netherlands, even with weak soil characteristics, will satisfy the 0.5 MPG requirement by 2030.

Moreover, it is advised for BAM to focus more on the reuse of the elements of the FLOW building, including the reuse potential of the shallow strip foundation in their design. A standardised prefab shallow strip configurations have a high re-use potential because of the easy removal and replacement process. By applying a re-used strip foundation, the environmental impact of the foundation can be lowered to a minimum. Therefore, for specific projects where a high amount of circularity is required, the shallow strip foundation may result in a high degree of circularity. However, because the foundation is the only dynamic part of the FLOW design, it is recommended to focus on the circularity and re-use of the standardised superstructure elements first. Clear and detailed documentation about the foundation design and soil conditions is advised and will enhance the reuse potential.

9.2. Further research

This research mainly focuses on the piles and the shallow strip. Including a detailed LCA of the foundation beams and strip wall in further research is recommended. This way, a more detailed and comprehensive foundation ECI analysis can be achieved. Especially in Zwolle, the foundation beams and strip wall significantly contribute to the total environmental impact and, therefore should be researched in detail.

The LCA study is based on the old 11 impact categories, which were widely available and contain the shadow costs. However, if the new 13 impact categories (NEN-EN 15804) are monetised and more widely available, it can be interesting to recalculate the ECI values and investigate any differences. Additionally, compliance with future European standards can be ensured by employing the most recent NEN-EN 15804 + A2 standard impact categories.

As inferred from the findings presented in subsection 3.2, it is evident that building services like electricity heating and ventilation,significantly dominate the contribution to the MPG value of FLOW. Consequently, an interesting research is to limit the ECI value of these installations. It is important to note that diminishing the ECI of electric installations holds greater promise in meeting the 0.5 requirement by 2030, as opposed to reducing the environmental impact through foundation design modifications.
This research is limited to three regularly applied foundations because of time constraints and the desired in-depth analysis of the foundation variants. However, further research can focus on alternative foundation materials like lightweight foam concrete or other bio-based materials, which may have a small environmental impact. Because all three foundation variants contain concrete and using CEM III concrete shows a high ECI reduction potential, it is interesting to investigate alternative concrete mixture compositions. Especially because many construction elements are made from concrete, an alternative low-environmental impact concrete mixture can make an enormous impact.

Another exciting follow-up research can focus on re-using foundations for housing projects. Because the foundation piles often remain in the soil, it may be interesting to investigate the re-use of the foundation by placing a new superstructure on top. In this way, the environmental impact of the foundation can be substantially reduced. Moreover, it could also be interesting to investigate alternative foundation designs, which can be easily removed and replaced in other locations. For example, steel-based foundation piles may have a high reuse potential. The prefab shallow strip foundation also has a high reuse potential because it can be relatively removed, transported and replaced at another location.

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A

Soil conditions

A.1. Physical soil properties

The physical properties need to be determined to predict and calculate the mechanical properties of the soil. This is because the physical properties of the soil underneath a building influence the mechanical properties of the soil. The physical properties of soil are [3]:

- Volumic weight
- Consistency
- Grain size
- Hydraulic properties
- Excavation restrictions

Volumic weight

Soil mainly exists in grains, water and especially above-ground-level air. The total volume of the soil v_g is equal to the sum of the separated components:

$$V_{ea} = V_w + V_s + V_a \tag{A.1}$$

The weight of the soil is equal to the sum of the weight of the separated components, which can be calculated with:

$$W_{ea} = W_s + W_w + W_a \tag{A.2}$$

By determining the total volume of the saturated soil, the volumic weight of the saturated soil can be calculated:

$$\gamma_{sat} = W_{ea}/V_{ea} \tag{A.3}$$

By subsequently drying the saturated soil and determining the weight of the solids, the volumic weight of the dry particles can be calculated with the following equation, where the V_{ea} is the original volume of the soil:

$$\gamma_{dry} = W_s/V_{ea} \tag{A.4}$$

The volumic weight of the soil, the γ_{dry}, γ_{dry} and the hydraulic head of the groundwater determine the soil, grain and water stresses in the soil. From the above-defined units, other physical properties can be calculated, which are important properties to predict soil behaviour. The porosity n of the soil is equal to the volume of the voids (V_{pr} = V_w + V_a) divided by the total volume:

$$n = V_{pr}/V_s \tag{A.5}$$

Subsequently, the void ratio e is defined as the volume of the voids divided by the volume of the grains:

$$e = (\gamma_{sat} - \gamma_{sat})/(\gamma_{sat} - \gamma_w) \tag{A.6}$$

Therefore, the relation between the porosity and the void ratio can be defined with the following equation, which can also be seen in Figure A.1:

$$e = n/(1 - n) \tag{A.7}$$

$$n = e \cdot (1 + e) \tag{A.8}$$

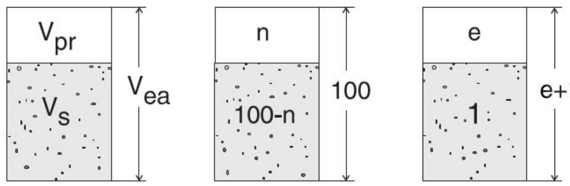

Figure A.1: Relationship between the porosity n and void ratio e [3]

The water ratio w is defined by dividing the water weight by the dry weight of the soil (A.9). Sometimes, the water ratio is also expressed in volume percentages. In that case, the water ratio can be calculated by dividing the water volume by the soil volume, times 100%.

$$w = w_w/w_s = (v_w \cdot \gamma_s)/(v_{ea} \cdot \gamma_{dr}) \tag{A.9}$$

The saturation degree s_r is the ratio between the volume of the free water and the void volume of the considered soil:

$$s_r = v_w/v_{pr} \cdot 100\% \tag{A.10}$$

With the above equations and variables, the important physical properties of different kinds of soils can be determined. In Figure A.2, some values are given for the common present soils in the Netherlands.

Grondsoort	**Poriëngehalte** [%] n	**Poriëngetal** [–] e	**Volumiek gewicht** [kN/m^3] γ_{dr}	γ_{sat}
grind	20-40	0,25-0,67	21-16	23-20
zand, grof	25-45	0,33-0,82	20-15	22-19
zand, uniform	35-45	0,54-0,82	17-15	21-19
zand, gegradeerd	25-35	0,33-0,54	20-17	22-20
zand, zeer fijn	30-45	0,43-0,82	18-15	21-18
zandige klei	35-45	0,54-1,00	17-14	21-18
klei, slap	70-80	2,33-4,00	8-5	15-13
klei, matig	50-70	1,00-2,33	13-8	18-15
klei, vast	35-50	0,54-1,00	17-13	20-18
veen	70-95	2,90-15,50	6-2	13-10

Figure A.2: Porosity, voids ratio and volumic weight of both the dry and saturated soil [3]

Consistency

The second important physical and dynamic property of the soil is consistency. The consistency of the soil can be expressed with the consistency index I_c. Consistency is the behaviour of soil under a certain stress. The soil consistency varies with the variation of soil moisture and the applied stress.

With decreasing moisture content, the soils lose their stickiness and therefore plasticity. As a result, the soil becomes friable and soft. Ultimately, when the soil is dry, it will become hard and coherent [45]. However, in practice, consistency is of subordinate interest in the design of a foundation. Only for specific problems, the consistency plays an important role [3].

Grain size

Another important property is the grain size of the soil. Especially for sand layers, the grain size plays an important role in the design of the foundation. The grain size indicates the permeability of the layer and the possibility of improving the soil by injecting cement or chemical means. Moreover, the size plays an important role in soil improvements; with a proper gradation of the grains, a high soil compaction can be achieved. The exact grain distribution can be determined with a sieve analysis in the laboratory. The grain classification according to ISO 14688-2 results in the values for spread and uniformity-coefficient.

Hydraulic properties

The final important physical aspect is the hydraulic property of the soil. Especially because the hydraulic property greatly influences the water content and water level in the soil, which subsequently influences the mechanical behaviour. In the calculations for the water flow in the soil, it is assumed that the water is non-compressible. From this, the theory of continuity is formed. Darcy's law describes that the mean flow rate v is proportional to the gradient i. With that, the mean flow in the soil can be calculated with equation A.12. The gradient i is negative if the height in the flow direction reduces, therefore the minus sign.

$$i = \Delta H / \Delta L \tag{A.11}$$

$$v = -k \cdot i \tag{A.12}$$

$$q = v \cdot 1 \tag{A.13}$$

The flow rate per square meter therefore can be calculated by multiplying the flow rate v with the surface. The volume flow can be calculated with the following equation: Figure A.3 shows the permeability coefficients of the different soil types. What can be seen is that peat has a very low permeability. The difference between coarse and fine sand is also considerable.

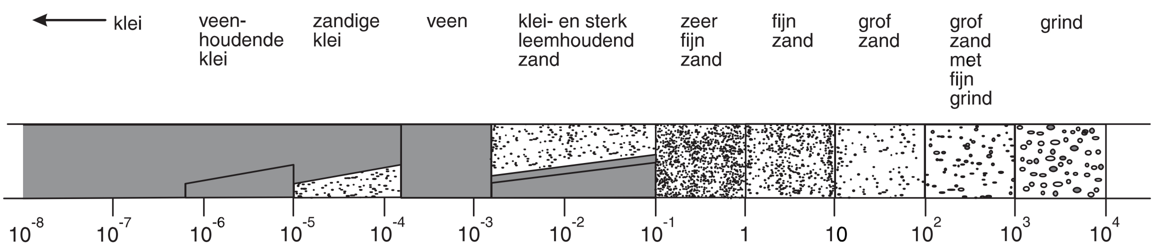

Figure A.3: Permeability coefficients k of the different soil types [3]

Excavation restrictions

In the case of an excavation, after a conducted CPT, the cone resistance in both sand and gravel also must be reduced. The reduced cone resistance for vibration piling can be calculated using equation A.14. If the piling is vibration-free or implemented before excavation, equation A.15 can be used.

$$q_{c,z;exc} = q_{c;z} \cdot \frac{\sigma'_{v,z;exc}}{\sigma'_{v,z;0}} \tag{A.14}$$

$$q_{c,z;exc} = q_{c;z} \cdot \sqrt{\frac{\sigma'_{v,z;exc}}{\sigma'_{v,z;0}}} \tag{A.15}$$

Swelling of the soil

Besides the compression of the soil due to added load, there is also ground heave in the case of relieving. However, with the same load change, the swelling of the soil will be less than the compression of the soil. Nevertheless, the ground heave occurs in an abbreviated period of time than the compression of the soil. Due to, for example, excavation, the swelling can be calculated using the following equation:

$$Z_w = [H \cdot C_{sw} \cdot \log((\sigma'_{v;z,o} + \Delta\sigma'_{v;z})/\sigma'_{v;z,o})]/(1+e) \tag{A.16}$$

If the soil is loaded again after the swelling, it will behave stiffly until the previous loading level is reached. After reaching this loading level, the soil will behave 'weaker' again. These phenomena result in opportunities to limit the settlement by excavating. By relieving the soil, swelling will occur. After re-loading, the soil behaves very stiff, which can be preferable in some cases [3].

A.1.1. Stresses in soils

Soil can be considered as three substances: grains, water and air. For the design of the foundation, the air does not have to be considered due to the subordination concerning the grains and water.

Vertical stresses

The soil can be defined as the sum of the grains and the water. Therefore, the stresses on the soil are taken by both the grains and the water, which are both completely different elements. The total vertical soil stress can be calculated using the following equation [2]:

$$\sigma = \sigma' + p \tag{A.17}$$

The pore pressure, assumed as only the water pressure, can be derived from the capillary rise of the water. This rise can be easily determined with the use of monitoring wells. The capillary rise of the water H is the sum of the pressure height h with the potential head z, as can be seen in eq. A.18. The pore pressure and the capillary rise are related to each other, as can be seen in eq. A.19.

$$H = h + z \tag{A.18}$$

$$u = (H - z) \cdot \gamma_w \tag{A.19}$$

It is important to mention the horizontal reference point with respect to the potential head. Usually, the ground level or the NAP is taken as the reference level. To determine the effective stress, the water stress needs to be subtracted from the vertical soil stress.

Horizontal stresses

Besides the vertical stresses in the soil, there will also be horizontal stresses. The horizontal stress depends on the vertical stress and the deformation in the horizontal direction. If the grain particles can extend in the horizontal direction, the stresses between grains will decrease [3]. The horizontal stress can be calculated using equation A.20, where the $K_{\gamma;a}$ is the active horizontal soil stress because of the soil pressure. However, soil is only active if the soil can displace free horizontally. Normally speaking, the horizontal soil stress is neutral, and therefore $K_{\gamma;o}$

$$\sigma_h = \sigma' \cdot K_{\gamma;a} \tag{A.20}$$

Shear stresses

The soil's failure occurs when the soil's maximum shear resistance is exceeded. The connection between the shear stress and the soil stress under normal conditions can be calculated by using the following formula:

$$\tau_f = \sigma \cdot \tan\varphi' + c \tag{A.21}$$

The cohesion factor c in the formula indicated that even if the normal stress is equal to zero, a certain shear stress is needed to produce a shear failure (Fig. A.4). This may be caused by irregularities in the surface of two different particles or molecular attraction. Sand typically has a negligible shear strength at a normal stress of zero and therefore has a cohesion value of zero. If the shear stress on a certain plane is smaller than the τ_f, then the deformations will be limited. However, if the shear stresses on the plane exceed the maximum value, then the shear deformations are unlimited, indicating a shear failure [2]. There is an equilibrium if the shear stress remains under the maximum shear value. Nevertheless, that does not imply that there will be no deformations in the soil. During the development of the shear stresses, those deformations are highly common and are unacceptable in practice. This also results in the fact that the deformation criteria are decisive in the design of the foundation.

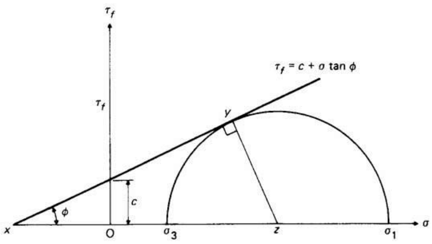

Figure A.4: Mohr's circle and Mohr-Coulomb failure criterion [25]

A.1.2. Deformation soil

For the soil deformation calculations, it is assumed that the soil behaves elastically with small loads, and when the load increases the deformation behaves plastically. The relation between the stress increment and deformation is important for the foundation calculation. This results in the parameters for a shallow foundation in the sub-grade reaction modulus k and for the pile foundation in the spring constant k. The sub-grade reaction modulus can be calculated with the load increment and the settlement of a construction part (eq. A.22). The reaction modulus k needs to be considered as a variable, which depends on the loaded surface, the stress level and the dynamic behaviour of the soil. Therefore, the sub-grade reaction modulus needs to be calculated with the on-site soil investigation. If the shallow foundation is loaded horizontally, the horizontal sub-grade reaction modulus will also be important [20].

$$k = p/s \tag{A.22}$$

To determine the soil compression with a given load, a distinction should be made between the high permeable sand and gravel layers and the low permeable cohesive layers. The deformation will occur directly with the highly permeable sand and gravel layers. The cohesive layers like clay and peat need to consolidate before they can be compressed. Soil consolidation is the process in which the volume of a saturated soil decreases due to an applied stress [40]. Especially with shallow foundations, soil consolidation plays an important role, which can result in difficulties. An elastic behaviour of the soil can be used for the small deformation. Hooke's law describes the stress/strain relationship:

$$\sigma = E \cdot \epsilon \tag{A.23}$$

$$\Delta h = \sigma \cdot h/E \tag{A.24}$$

$$E = 2 \cdot (1 + v) \cdot G \tag{A.25}$$

For the elasticity modulus E, there is a distinction between the static and dynamic E modulus. The dynamic E-modulus is much higher because of the relatively short loading time. In modern computer analysis, the E-modulus will be calculated using equation A.25, in which G is the shear modulus and v is the Poisson ratio [2]. Another important value is the Over Consolidation Ratio (OCR), which is defined as the ratio between the once-exposed and current effective stress. In the past, due to for example large ice layers, the soil became overconsolidated. Especially with sand layers, this results in higher cone resistance than, based on the actual ground profile, may be expected. Therefore, according to NEN 9997-1+C2, the cone resistance needs to be reduced if the foundation is implemented in an overconsolidated region and implemented with piling or vibrating (eq. A.26). In principle, values of OCR smaller than one are associated with an unconsolidated state. An OCR value of 1 is associated with a normally consolidated (NC) state for the soil. Values of OCR larger than 1 are associated with an overconsolidated (OC) state [59]. The OCR value can be determined by conducting an odometer test with an undisturbed soil example.

$$q_{c;z;NC} = q_{c;z;OC} \cdot \sqrt{\frac{1}{OCR}} \tag{A.26}$$

Characteristic soil parameters
Table A.1 gives representative soil values per soil type. The soil parameters are essential properties that influence the bearing capacity of the soil layers and therefore the foundation design. However, the values in the table are mean; therefore, for a detailed soil investigation, those parameters must be determined individually. However, the table properly indicates the soil parameters.

Table A.1: Representative mean values for the soil parameters per type (NEN 9997-1+C2)

Grondsoort			Karakteristieke waarde [a] van grondeigenschap											
Hoofd-naam	**Bijmengsel**	**Consis-tentie** [b]	γ [c]	γ_{sat}	q_c [d g]	C'_p [g]	C'_s	$C_c/(1+e_0)$ [g]	C_α [f]	$C_{sw}/(1+e_0)$ [g]	E_{100} [g h]	φ' [g]	c'	c_u
			kN/m³	kN/m³	MPa			[-]	[-]	[-]	MPa	Graden	kPa	kPa
Grind	Zwak siltig	Los	17	19	15	500	∞	0,004 6	0	0,001 5	45	32,5	0	N.v.t.
		Matig	18	20	25	1 000	∞	0,002 3	0	0,000 8	75	35,0	0	
		Vast	19 20	21 22	30	1 200 1 400	∞	0,001 9 0,001 6	0	0,000 6 0,000 5	90 105	37,5 40,0	0	
	Sterk siltig	Los	18	20	10	400	∞	0,005 8	0	0,001 9	30	30,0	0	N.v.t.
		Matig	19	21	15	600	∞	0,003 8	0	0,001 3	45	32,5	0	
		Vast	20 21	22 22,5	25	1 000 1 500	∞	0,002 3 0,001 5	0	0,000 8 0,000 5	75 110	35,0 40,0	0	
Zand	Schoon	Los	17	19	5	200	∞	0,011 5	0	0,003 8	15	30,0	0	N.v.t.
		Matig	18	20	15	600	∞	0,003 8	0	0,001 3	45	32,5	0	
		Vast	19 20	21 22	25	1 000 1 500	∞	0,002 3 0,001 5	0	0,000 8 0,000 5	75 110	35,0 40,0	0	
	Zwak siltig, kleiig		18 19	20 21	12	450 650	∞	0,005 1 0,003 5	0	0,001 7 0,001 2	35 50	27,0 32,5	0	N.v.t.
	Sterk siltig, kleiig		18 19	20 21	8	200 400	∞	0,011 5 0,005 8	0	0,003 8 0,001 9	15 30	25,0 30,0	0	N.v.t.
Leem [e]	Zwak zandig	Slap	19	19	1	25	650	0,092 0	0,003 7	0,030 7	2	27,5 30,0	0	50
		Matig	20	20	2	45	1 300	0,051 1	0,002 0	0,017 0	3	27,5 32,5	1	100
		Vast	21 22	21 22	3	70 100	1 900 2 500	0,032 9 0,023 0	0,001 3 0,000 9	0,011 0 0,007 7	5 7	27,5 35,0	2,5 3,8	200 300
	Sterk zandig		19 20	19 20	2	45 70	1 300 2 000	0,051 1 0,032 9	0,002 0 0,001 3	0,017 0 0,011 0	3 5	27,5 35,0	0 1	50 100
Klei	Schoon	Slap	14	14	0,5	7	80	0,328 6	0,013 1	0,109 5	1	17,5	0	25
		Matig	17	17	1,0	15	160	0,153 3	0,006 1	0,051 1	2	17,5	5	50
		Vast	19 20	19 20	2,0	25 30	320 500	0,092 0 0,076 7	0,003 7 0,003 1	0,030 7 0,025 6	4 10	17,5 25,0	13 15	100 200
	Zwak zandig	Slap	15	15	0,7	10	110	0,230 0	0,009 2	0,076 7	1,5	22,5	0	40
		Matig	18	18	1,5	20	240	0,115 0	0,004 6	0,038 3	3	22,5	5	80
		Vast	20 21	20 21	2,5	30 50	400 600	0,076 7 0,046 0	0,003 1 0,001 8	0,025 6 0,015 3	5 10	22,5 27,5	13 15	120 170
	Sterk zandig	-	18 20	18 20	1,0	25 140	320 1 680	0,092 0 0,016 4	0,003 7 0,000 7	0,030 7 0,005 5	2 5	27,5 32,5	0 1	0 10
	Organisch	Slap	13	13	0,2	7,5	30	0,306 7	0,015 3	0,102 2	0,5	15,0	0 1	10
		Matig	15 16	15 16	0,5	10 15	40 60	0,230 0 0,153 3	0,011 5 0,007 7	0,076 7 0,051 1	1,0 2,0	15,0	0 1	25 30
Veen	Niet voorbelast	Slap	10 12	10 12	0,1	5 7,5	20 30	0,460 0 0,306 7	0,023 0 0,015 3	0,153 3 0,102 2	0,2 0,5	15,0	1 2,5	10 20
	Matig voorbelast	Matig	12 13	12 13	0,2	7,5 10	30 40	0,306 7 0,230 0	0,015 3 0,011 5	0,102 2 0,076 7	0,5 1,0	15,0	2,5 5	20 30
Variatiecoëfficiënt v			0,05		–	0,25					0,10		0,20	

[a] De tabel geeft van de desbetreffende grondsoort de lage, respectievelijk de hoge karakteristieke waarde van gemiddelden. Binnen een gebied, vastgesteld door de rij van het bijmengsel en de kolom van de parameter (een cel), geldt:

— als een verhoging van de waarde van een van de grondeigenschappen tot een ongunstiger situatie leidt dan de toepassing van de in de tabel gepresenteerde lagere karakteristieke waarde, moet de rechterwaarde op dezelfde regel zijn gebruikt. Is er rechts geen waarde vermeld, dan moet de waarde er recht onder zijn toegepast;
OPMERKING Dit is bijvoorbeeld het geval bij negatieve kleef op een paal waar een hogere waarde van φ', c' en c_u ook een hogere waarde van de negatieve kleef oplevert.

— voor $C_c/(1+e_0)$, C_α en $C_{sw}/(1+e_0)$ zijn in de tabel de hoge karakteristieke gemiddelde waarden vermeld.

A.2. GWT Zwolle and Delft

To simulate a realistic soil condition, an average CPT result will be used for both a location in the western and eastern parts. A CPT around Zwolle will be used for the east part of the Netherlands. A typical CPT around Delft will be used for the western part of the Netherlands. The location of both CPTs in the Netherlands can be seen in Figure A.5.

Figure A.5: The position of CPT in both Zwolle and Delft in the Netherlands

Ground water table Zwolle

Besides the CPT graphs given in Figure 3.3 of both Zwolle and Delft, more soil conditions must be considered. The first one is the GWT of both Delft and Zwolle. This is because, as seen in chapter 2.2, the GWT can largely influence the foundation design. It can influence the bearing capacity of the soil, the possible degradation mechanism of a timber foundation pile and the settlement of shallow foundations because of the lowered ground level. Figure A.6 shows the GWT last the last couple of years of the CPT near Zwolle. It must be mentioned that the groundwater level is relative to NAP and that the ground level is located at +15 cm relative to NAP. Therefore, to know the GWT relative to ground level, an additional 15 cm needs to be subtracted [1]. The mean GWT in Zwolle is 46 cm relative to NAP and 31 cm relative to ground level.

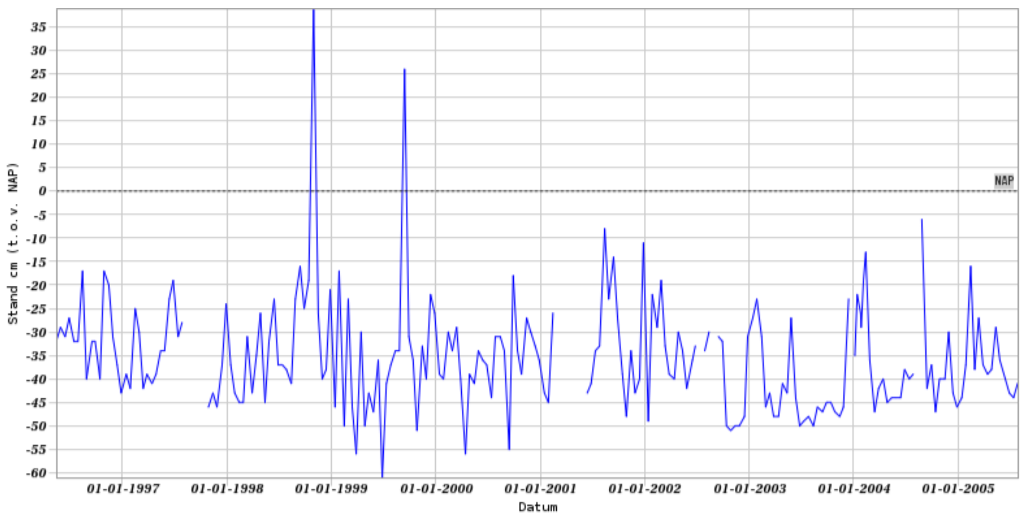

Figure A.6: GWT of CPT Zwolle from 1997 to 2005, relative to NAP (RD 208594.500, 505477.300) [1]

From Figure A.6, it can be seen that the GWT is relatively constant over ten years, with two outliers in the years 1999 and 2000. The groundwater level varies roughly from -60 to -5cm relative to NAP. The lowest groundwater level is -60cm, between 1999 and 2000. Therefore, the GWT does not exceed -105 cm relative to the ground level. The GWT fluctuates per year because of the different seasons. The maximum value is reached after the winter, and the minimum is reached after the summer. The yearly fluctuations in GWT result from the alternating wet and dry seasons. There is no GWT data available after the year 2005. However, it is assumed that the GWT remains roughly within the same range as the data in the figure.

Ground water table Delft

Figure A.7 shows the GWT of the CPT location in Delft. The mean GWT relative to NAP in the year 2018/19 is -238 cm, with a maximum level of -164cm around 1 May and a minimum level of -256.3cm around the middle of August [1]. The GWT is relative to NAP and not relative to the surface. The ground level concerning NAP can be determined by using the AHN viewer of the same location as the GWT measurement. The ground level is at -55cm relative to NAP [48]. Therefore, for the GWT relative to ground level, 55cm needs to be added, which can be seen in Figure A.8

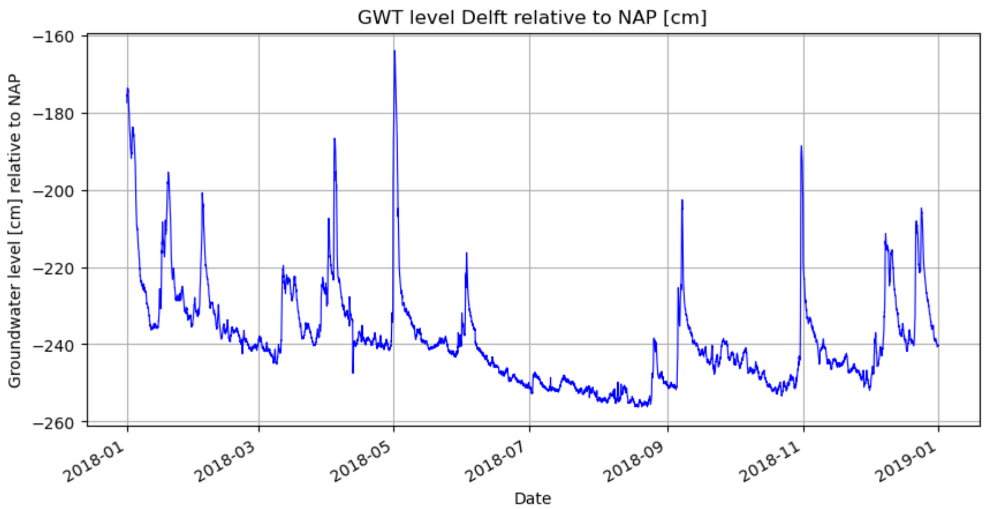

Figure A.7: GWT of CPT near Delft in 2018/19, relative to NAP (RD: 85516.900, 445880.700)

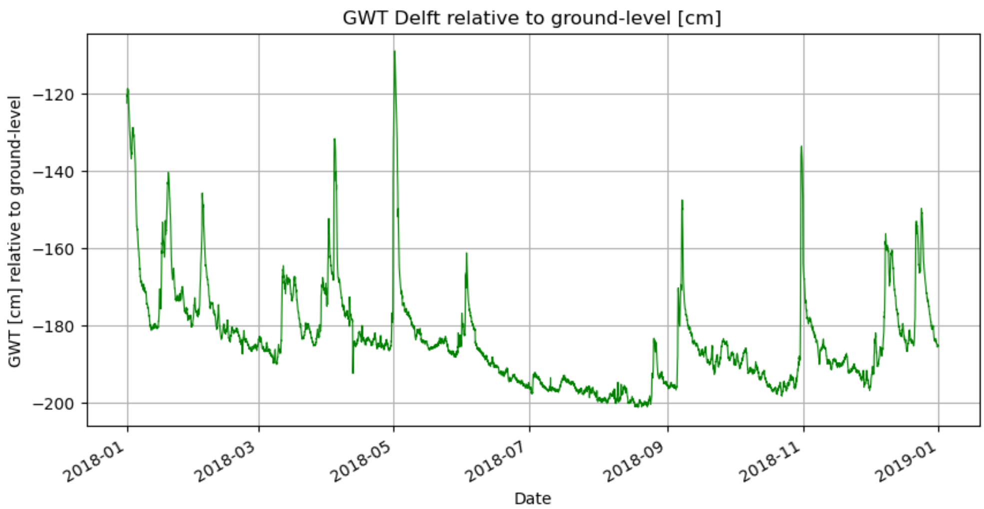

Figure A.8: GWT of CPT near Delft in 2018/19, relative to ground-level (RD: 85516.900, 445880.700)

B

Governing loads on foundation beams

B.1. Loads of FLOW

To determine the load on the foundation beams, both the permanent and variable vertical loads of all elements must be determined. Subsequently, the combination factors need to be considered to calculate the governing load case on the foundation beams

Combination factors

To determine the governing load combination, the load combination with factors needs to be investigated. Load combinations consist of design values of both permanent and variable loads. Both characteristics and reduced characteristics loads are combined for the ULS and SLS. The ULS is used to check the structural safety of a structure, and the SLS is used for the usability of a structure. According to the NEN 1990 + A1, the following load combination has to be used for the Service Limit State, as shown in Table B.1. The following load combinations must be considered for the Ultimate Limit State, as shown in Table B.2.

Combinatie	Blijvende belastingen		Veranderlijke belastingen	
	Ongunstig	Gunstig	Overheersende	Andere
Karakteristiek (6.14b)	1,0 $G_{kj,sup}$	1,0 $G_{kj,inf}$	$Q_{k,1}$	$\psi_{0,i}$ $Q_{k,i}$
Frequent (6.15b)	1,0 $G_{kj,sup}$	1,0 $G_{kj,inf}$	$\psi_{1,1}$ $Q_{k,1}$	$\psi_{2,i}$ $Q_{k,i}$
Quasi-blijvend (6.16b)	1,0 $G_{kj,sup}$	1,0 $G_{kj,inf}$	$\psi_{2,1}$ $Q_{k,1}$	$\psi_{2,i}$ $Q_{k,i}$

Table B.1: Service Limit State combination factors

	Blijvende belastingen		Overheersende veranderlijke belasting	Veranderlijke belastingen gelijktijdig met de overheersende	
	Ongunstig	Gunstig		Belangrijkste	Andere
6.10a	1,2 $G_{kj,sup}$ [a]	0,9 $G_{kj,inf}$		1,35 $\psi_{0,1}$ $Q_{k,1}$	1,35 $\psi_{0,i}$ $Q_{k,i}$ (j > 1)
6.10b	1,1 $G_{kj,sup}$ [b]	0,9 $G_{kj,inf}$	1,35 $Q_{k,1}$		1,35 $\psi_{0,i}$ $Q_{k,i}$ (j > 1)

a Bij vloeistofdrukken met een fysiek beperkte waarde mag zijn volstaan met 1,1 $G_{kj,sup}$.
b Deze waarde is berekend met ξ = 0,89.

Table B.2: Ultimate Limit State combination factors

Permanent loads

The permanent load elements are determined for the maximum load case, where all the possible building materials will be applied. However, for a complete calculation procedure, the minimum load case needs to be determined because of the lightweight character of the building and the potential upward wind force. However, the verification of BAM showed that this would not be critical due to the relatively high self-weight of the foundation beams. Therefore, only the maximum loads will be considered. The following permanent elements with corresponding weights are present in the FLOW timber house:

Elements	Density	Height		Distrubted load
	kN/m^3	m		kN/m^2
Roof				
Solar panels			0.25	0.25
Roof tiles			0.43	0.43
Tiles and battens			0.05	0.05
OSB (12 mm) – 700 kg/m^3	7.0	0.012		0.08
Steico (SJL 60x300 mm^2 hoh 600) – 550 kg/m^3	5.5	0.012		0.07
Insulation cellulose (300 mm) – 53 kg/m^3	0.53	0.3		0.16
Durelis ceiling (particle board) 10 mm – 720 kg/m^3	7.2	0.01		0.07
		1.11	/ cos(42) =	**1.49 kN/m^2**
Ground level				
Ribbed flooring (280 mm)			2.3	2.30
Finishing (cement screed) – 2000 kg/m^3	20	0.05		1.00
				3.30 kN/m^2
Storey floor				
Multiplex (18 mm) – 550 kg/m^3	5.5	0.018		0.10
OSB (22 mm) – 700 kg/m^3	7.0	0.022		0.15
Steico (SJL 45/90*39 mm^2 h=360 mm hoh 600) – 550 kg/m^3	5.5	0.020		0.11
Insulation			0.08	0.08
Rafts (22x50 mm h.t.h. 300 mm) – 420 kg/m3	4.2	0.004		0.02
Gypsum board (1x12,5 mm) – 1050 kg/m^3	10.5	0.0125		0.13
				0.59 kN/m^2
Attic floor				
Multiplex (18 mm) – 550 kg/m^3	5.5	0.018		0.10
OSB (22 mm) – 700 kg/m^3	7.0	0.022		0.15
Rafts (22x50 mm^2 h.t.h. 300 mm) – 420 kg/m3	4.2	0.004		0.02
Gypsum board (1x12,5 mm) – 1050 kg/m^3	10.5	0.0125		0.13
				0.40 kN/m^2
Bathroom unit				
Bathroom unit				2.25
Multiplex (18 mm) – 550 kg/m^3	5.5	0.018		0.10
C24 (58x138 mm h.t.h. 250 mm) – 420 kg/m3	4.2	0.032		0.13
Rafts (22x50 mm^2 h.t.h. 300 mm) – 420 kg/m3	4.2	0.004		0.02
Plumbing			0.05	0.05
Insulation			0.08	0.08
Gypsum board (1x12,5 mm) – 1050 kg/m^3	10.5	0.0125		0.13
				2.76 kN/m^2
Landing				
Multiplex (18 mm) – 550 kg/m^3	5.5	0.018		0.10
OSB (22 mm) – 700 kg/m^3	7.0	0.022		0.15
C24 (70x156 mm^2 h.t.h. 300 mm) – 420 kg/m3	4.2	0.036		0.15
Insulation			0.08	0.08
Gypsum board (1x12,5 mm) – 1050 kg/m^3	10.5	0.0125		0.13
				0.62 kN/m^2
Load-bearing facade				
Mineral stone strips			0.05	0.05
Fermacel Powerpanel H2O (12 mm)			0.13	0.13
Rafts (28x45 mm^2 h.t.h. 300 mm) – 420 kg/m3	4.2	0.004		0.02
OSB (22 mm) – 700 kg/m^3	7.0	0.022		0.15
Posts C24/LVL-R (70x320 mm^2 hoh 600 mm) – 550 kg/m^3	5.5	0.037		0.21
Bottom and top bars (2x45x150 mm^2) – 420 kg/m^3	4.2	0.004		0.02
Insulation cellulose (320 mm) – 53 kg/m3	0.53	0.32		0.17
Gypsum board (1x12,5 mm) – 1050 kg/m^3	10.5	0.0125		0.13
				0.87 kN/m^2

Load-bearing facade inc. CLT-120					
CLT-wall (120 mm) – 420 kg/m3		4.2	0.12		0.50
Mineral stone strips				0.05	0.05
Fermacel Powerpanel H2O (12 mm)				0.13	0.13
Rafts (28x45 mm² h.t.h. 300 mm) – 420 kg/m3		4.2	0.004		0.02
OSB (12 mm) – 700 kg/m³		7.0	0.012		0.08
Posts C24/LVL-R (45x150 mm² hoh 600 mm) – 550 kg/m³		5.5	0.011		0.06
Bottom and top bars (2x45x150 mm2) – 420 kg/m3		4.2	0.004		0.02
Insulation cellulose (320 mm) – 53 kg/m		0.53	0.32		0.17
Gypsum board (1x12,5 mm) – 1050 kg/m³		10.5	0.0125		0.13
					1.16 kN/m²
Seperating wall (per wall)					
Posts C24/LVL-R (75x95 mm² hoh 600 mm) – 550 kg/m³		5.5	0.012		0.07
Bottom and top bars (2x45x150 mm2) – 420 kg/m3		4.2	0.004		0.02
Glaswol Insulation (40 mm) – 19 kg/m3		0.19	0.04		0.01
Gypsum board (1x12,5 mm + 1x15 mm) – 1050 kg/m³		10.5	0.0275		0.29
					0.38 kN/m²
Longitudinal facade					
Mineral stone strips				0.05	0.05
Fermacel Powerpanel H2O (12 mm)				0.13	0.13
Rafts (28x45 mm h.t.h. 300 mm) – 420 kg/m3		4.2	0.004		0.02
OSB (12 mm) – 700 kg/m³		7.0	0.012		0.08
Posts C24 (38x245 mm² h.t.h. 600 mm) – 420 kg/m3		4.2	0.016		0.07
Bottom and top bars (2x45x150 mm2) – 420 kg/m3		4.2	0.004		0.02
Cellulose Insulation (245 mm) – 53 kg/m3		0.53	0.245		0.13
Gypsum board (1x12,5 mm) – 1050 kg/m³		10.5	0.0125		0.13
					0.62 kN/m²
CLT-wall 120					
CLT-wall (120 mm) – 420 kg/m3		4.2	0.12		0.50
					0.50 kN/m²
CLT-wall 140					
CLT-wall (140 mm) – 420 kg/m3		4.2	0.14		0.59
					0.59 kN/m²
Window-frames					
Window frames				0.50	0.50
					0.50 kN/m²
Stairs					
stairs 1 layer				0.38	0.38
					0.38 kN/m²
Steel beam		kN/m³	m²		
Profile: h = 330 mm, t = 8 mm		7.85	4.16E-03		**0.03 kN/m¹**
Fund.beam seperating	m²				
Prefab L-beam, 185/350x500 mm	0.23	25			5.81
					5.81 kN/m¹
Fund.beam head					
Prefab L-beam, 185/350x500 mm	0.23	25			5.81
					5.81 kN/m¹
Fund.beam longitudinal					
Fund.beam (0,2 x 0,81)	0.16	25			4.05
					4.05 kN/m¹
Dumping strip					
Dumping strip (0,54 x 0,25)	0.14	25			3.38
					3.38 kN/m¹

What is striking, is the limited amount of distributed load of the timber structure elements compared with the concrete foundation beams. This is due to the high density of concrete (25 $\mathrm{kN/m^3}$) in comparison with, for example, CLT timber (5 $\mathrm{kN/m^3}$), which is almost five times lower. Another interesting contributing load is the bathroom unit, which has a total load of 2.79 $\mathrm{kN/m^3}$. The total permanent loads on the foundation can be calculated with the given dimension of the FLOW building, given in section 3.1.1.

Variable loads

Besides the permanent loads, the variable loads must also be considered for the total governing load for the foundation design. Again, the maximum load case needs to be considered and therefore the psi factors need to be taken into account for the calculation according to B.2. The simplified and conservative wind load is modelled as 1.06 $\mathrm{kN/m^2}$ on both facade sides. It is also assumed that the moment generated by the wind force is insufficient to create a tension force in the foundation piles because of the relatively high self-weight of the foundation piles and the forces on top. An extensive wind analysis is out of the scope of this master thesis and is therefore simplified.

Elements	Use class	Psi	Distributed load
Roof floor			
Snow	H	0	0.34 kN/m^2
Wind	H	0	1.06 kN/m^2
Floors			
Seperation walls (1,75 + 0,5)	A	0.4	2.25 kN/m^2
Ground level			
Seperation wall (1,75 + 0,8)	A	0.4	2.55 kN/m^2
No seperation wall	A	0.4	1.75 kN/m^2
Non-common stairs	A	0.4	2.00 kN/m^2
Specialities			
Gallary	A	0.8	0 kN/m^2
Balcony	A	0.4	0 kN/m^2

B.2. Loads and combination factors on foundation beams

With the permanent and variable loads and the load combinations given in Tables B.1 and B.2, the governing vertical loads on the foundation beam can be calculated. It is assumed that the FLOW house is detached, with no connected housing units. Therefore, only the loads of the single housing units must be carried by the foundation beams, with no adjacent housing loads. As a result, a distinction can be made between two foundation beams, a longitudinal beam of 9.1 meters and a front beam of 5.1 meters. There are four foundation beams in total: two longitudinal and two front beams. Figure 3.7 shows the distributed loads on the 5.1-meter-long front foundation beams. The loads that need to be carried by the front foundation beam are the load-bearing facade elements of both the first and second floor and the self-weight of the foundation beam. This results in a maximum load of 9.5 kN/m as seen in Table B.3. It must be noted that these loads result from permanent and variable loads, including combination factors.

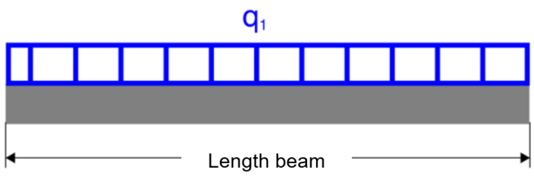

Figure B.1: Governing load on the front foundation beam of 5.1 meters

Loads

q_1	pl kN/m²	vl kN/m²	ψ	factor	width [m]	height [m]	number	pl kN/m	vl extr.	vl mom	qd 6.10a	qd 6.10b	qd max
Front facade 2nd floor	0.62	0.00	0.0	1.0	1.0	3.1	1.0	1.9	0.0	0.0			
Front facade 1st floor	0.62	0.00	0.0	1.0	1.0	3.1	1.0	1.9	0.0	0.0			
Found.beam front	4.05	0.00	0.0	1.0	1.0	1.0	1.0	4.1	0.0	0.0			
							Total:	7.9	0.0	0.0	9.5	8.7	**9.5**

Table B.3: Governing load combination on front beam

The two other foundation beams are the longitudinal beams, which have a total length of 9.1 meters. The loads on the beam can be subdivided into three different segments: the front of the house, the stairs and the back of the house. The length of the house's front and back is 3.0m, and the length of the stairs is 3.1m. Figure B.2 depicts the governing forces on the longitudinal foundation beams. The two F1 forces result from the front foundation beams placed on top of the longitudinal foundation beams. The load calculations of the longitudinal foundation beams are shown in Table B.4. The loads and calculation of the two longitudinal foundation beams are similar.

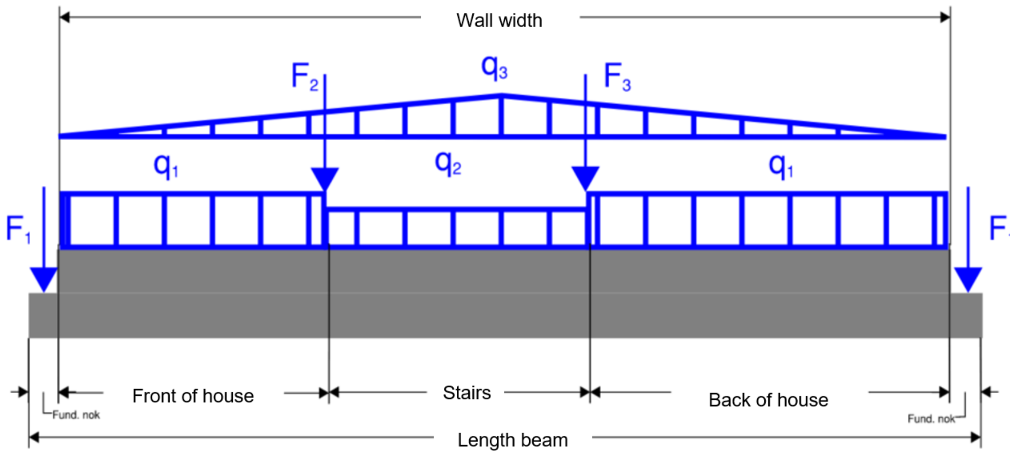

Figure B.2: Governing loads on the longitudinal foundation beam of 9.1 meters

Loads

q_1	pl kN/m²	vl kN/m²	ψ	factor	width [m]	height [m]	number	pl kN/m	vb extr.	vb mom.	qd 6.10a	qd 6.10b	qd max
Roof	1.49	0.34	0.0	0.5	5.1	1.0	1.0	3.8	0.9	0.0			
Second floor	0.59	2.25	0.4	0.5	5.1	1.0	1.0	1.5	5.7	2.3			
Load bearing façade 2nd	0.87	0.00	0.0	1.0	1.0	3.1	1.0	2.7	0.0	0.0			
First floor	0.59	2.25	0.4	0.5	5.1	1.0	1.0	1.5	5.7	2.3			
Load bearing façade 1st	0.87	0.00	0.0	1.0	1.0	3.1	1.0	2.7	0.0	0.0			
Ground level floor	3.30	2.55	0.4	0.5	5.1	1.0	1.0	8.4	6.5	2.6			
Foundation beam	5.81	0.00	0.0	1.0	1.0	1.0	1.0	5.8	0.0	0.0			
							Total:	26.4	18.8	7.2	41.4	54.5	**54.5**

q_2	pl kN/m²	vl kN/m²	ψ	factor	width [m]	height [m]	number	pl kN/m	vb extr.	vb mom.	qd 6.10a	qd 6.10b	qd max
Roof	1.49	0.34	0.0	0.5	5.1	1.0	1.0	3.8	0.9	0.0			
Façade 1st + CLT wall	1.16	0.00	0.0	1.0	1.0	3.1	1.0	3.6	0.0	0.0			
Ground level floor	3.30	2.55	0.4	0.5	5.1	1.0	1.0	8.4	6.5	2.6			
Foundation beam	5.81	0.00	0.0	1.0	1.0	1.0	1.0	5.8	0.0	0.0			
							Total:	21.6	7.4	2.6	29.5	33.7	**33.7**

q_3	pl kN/m²	vl kN/m²	ψ	factor	width [m]	height [m]	number	pl kN/m	vb extr.	vb mom.	qd 6.10a	qd 6.10b	qd max
Load bearing façade 2nd	0.87	0.00	0.0	1.0	1.0	4.1	1.0	3.6	0.0	0.0			
							Total:	3.6	0.0	0.0	4.3	3.9	**4.3**

F_1	pl kN/m²	vl kN/m²	ψ	factor	width [m]	height [m]	number	pl kN/m	vb extr.	vb mom.	qd 6.10a	qd 6.10b	qd max
Long façade 1st floor	0.62	0.00	0.0	0.5	5.1	3.10	1.0	4.9	0.0	0.0			
Long façade 2nd floor	0.62	0.00	0.0	0.5	5.1	3.10	1.0	4.9	0.0	0.0			
Foundation beam	4.05	0.00	0.0	0.5	5.1	1.00	1.0	10.3	0.0	0.0			
							Total:	20.1	0.0	0.0	24.2	22.2	**24.2**

F_2	pl kN/m²	vl kN/m²	ψ	factor	width [m]	height [m]	number	pl kN/m	vb extr.	vb mom.	qd 6.10a	qd 6.10b	qd max
CLT-wall 120	0.50	0.00	0.0	1.0	2.5	3.30	1.0	4.2	0.0	0.0			
Bathroom unit	2.76	1.75	0.4	0.5	1.5	3.00	1.0	6.2	3.9	1.6			
Second floor	0.59	2.25	0.4	0.25	3.6	3.00	1.0	1.6	6.1	2.4			
Overflow	0.59	2.25	0.4	0.25	5.1	3.30	1.0	2.5	9.5	3.8			
							Total:	14.4	19.5	7.8	27.8	42.2	**42.2**

F_3	pl kN/m²	vl kN/m²	ψ	factor	width [m]	height [m]	number	pl kN/m	vb extr.	vb mom.	qd 6.10a	qd 6.10b	qd max
CLT-wall 120	0.50	0.00	0.0	1.0	1.5	3.30	1.0	2.5	0.0	0.0			
Bathroom unit	2.76	1.75	0.4	0.5	1.5	3.00	1.0	6.2	3.9	1.6			
Second floor	0.59	2.25	0.4	0.25	3.6	3.00	1.0	1.6	6.1	2.4			
Overflow	0.59	2.25	0.4	0.25	2.2	3.00	1.0	1.0	3.7	1.5			
							Total:	11.3	13.7	5.5	20.9	30.9	**30.9**

Table B.4: Governing load combination on longitudinal beam

C

Prestressed pile calculations

The load-bearing capacity needs to be determined to investigate the dimensions and configurations of a foundation design. This way, the foundation design can be designed with the loads from the FLOW building and the soil conditions underneath the structure. For the calculation methodology, the NEN 9997-1+C2 will be used. In the NEN-norm, a distinction has been made between shallow and pile foundations regarding the bearing capacity calculation.

C.1. Foundation plan and reaction forces

The first step in designing the concrete, prestressed foundation piles is determining the forces acting on the pile. The loads from the foundation beam, as described in section 3.1 and the foundation pile plan, as shown in Figure 4.2, must be considered. The ”Technosoft Balkroosters” program will be used for the force distribution calculations. The front foundation beams are placed on the longitudinal foundation beams. Therefore, the connection between the front and longitudinal foundation beam can not transfer any moments and needs to be modelled in Technosoft. For the force distribution, it is assumed that the six piles have the same stiffness, are constrained in the x-direction (torsion), and can move unconstrained in the y-direction. The stiffness of the piles in the z-direction is estimated with the following equation:

$$K = \frac{EA}{2L} \tag{C.1}$$

Considering a surface of the pile of 48400 mm^2 (220mm x 220mm), an E-modulus of 35000 MPa (EN 1992-1-1:2004+AC2) and a mean estimated length of 8 m, the stiffness of the pile is estimated on 40000 MPa. Therefore, the six piles are considered springs with a stiffness of 40000 MPa, which can only have displacement in the z-direction. Considering the above characteristics and the governing loads on the foundation beams, the following results are obtained: the moment distribution in Figure C.2 and the shear force distribution in Figure C.1. Using the moment and shear force distribution, the support reactions on the foundation piles can be seen in Table C.1. It is assumed that the total horizontal wind load of 79.6 kN is taken by the stiffness of the concrete floor and the soil against it. Therefore, the foundation piles will not be configured against horizontal (wind) load.

Pile number	Vertical ULS [kN]	Vertical SLS [kN]	Moment support [kNm]
Pile 1 (S1)	219	162	92.0
Pile 2 (S2)	195	144	61.0
Pile 3 (S3)	159	118	28.7
Pile 4 (S4)	155	115	28.7
Pile 5 (S5)	193	143	63.0
Pile 6 (S6)	224	166	92.0

Table C.1: Support reactions concrete pile foundations (V+M+H)

Because the pile positions and the loads on the foundation beams are more or less symmetric, the vertical support reactions on the foundation piles have the same magnitude on both foundation beams. The maximum sagging moment on the front facade beam is -30.9 kNm (middle), and the maximum occurring sagging moment on the longitudinal foundation beam is -60 kNm (S2-S3), as depicted in Figure C.2. The maximum supporting moment can be found on the foundation piles S1 and S6, with a magnitude of 92 kNm. Foundation piles S3 and S4 have a positive moment of 28.7 kNm, and piles S2 and S5 have a positive supporting moment of 61 and 63 kNm, respectively. The occurring moments' magnitude in the foundation beams especially influences the reinforcement content, thereby contributing to a (higher) environmental impact.

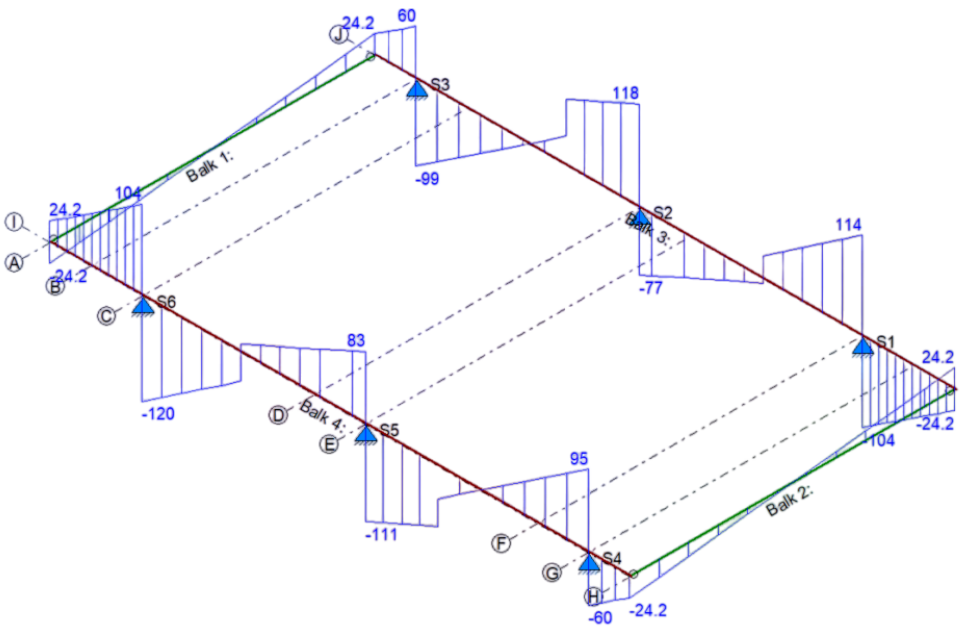

Figure C.1: Shear forces distribution on prefab concrete foundation beams/piles

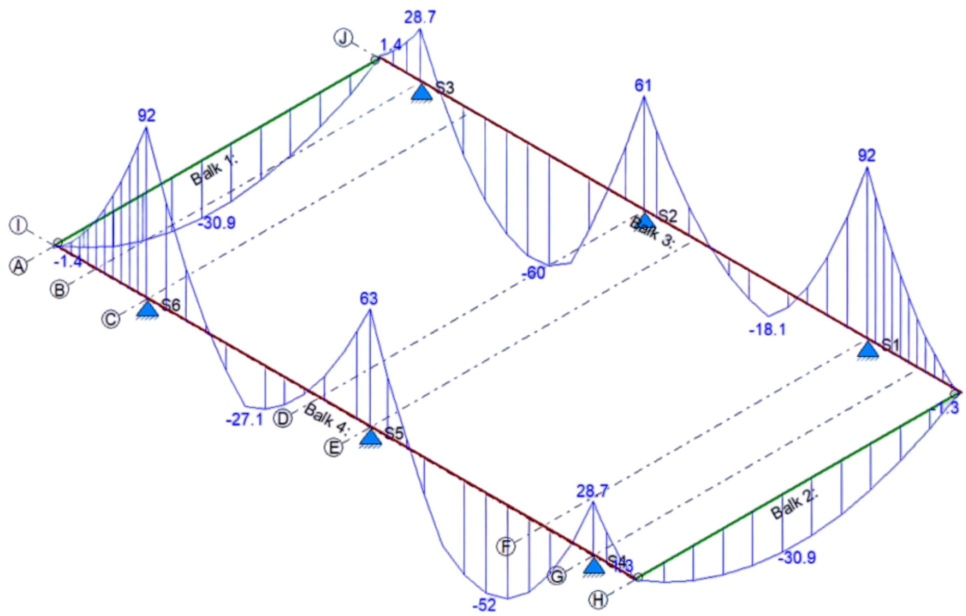

Figure C.2: Moment distribution on prefab concrete foundation beams/piles

C.2. Bearing capacity Zwolle and Delft

This Appendix section deals with the calculations of the prestressed prefab foundation piles in both Zwolle and Delft. The calculations are again subdivided into the bearing and structural capacity of the piles. For the load-bearing capacity of the pile foundation as mentioned in chapter 2.2, the following equation can be used:

$$F_{r;max;i} = F_{r;max;tip;i} + F_{r;max;friction;i} \tag{C.2}$$

For the calculation of both the end bearing capacity and the positive and negative skin friction, the flowchart in Figure C.3 can be used, where the starting point is a detailed given CPT graph:

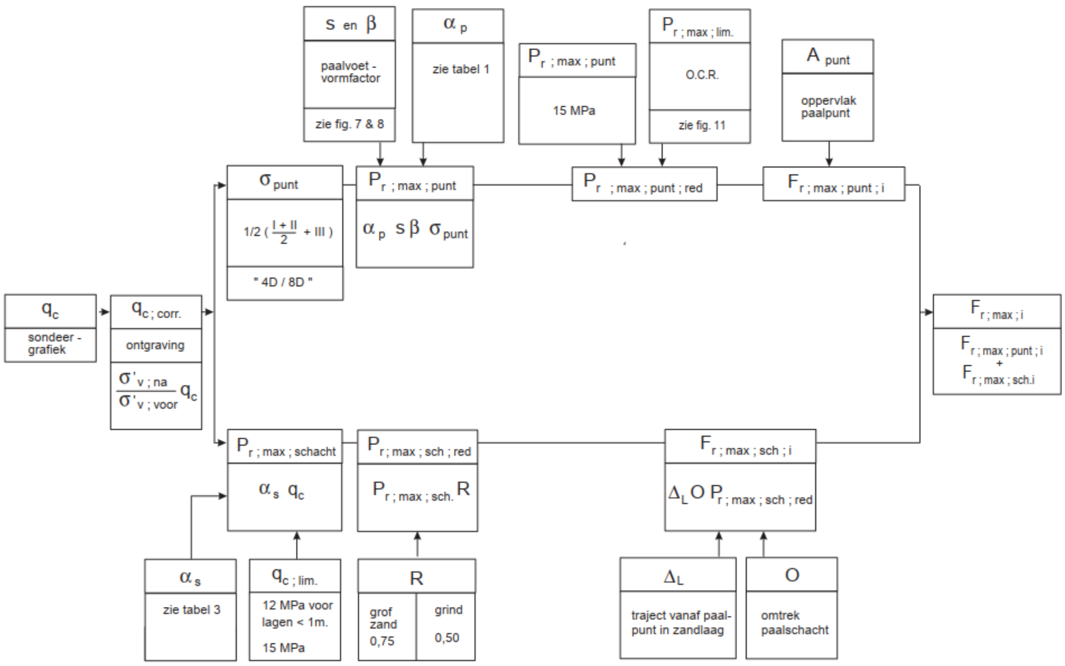

Figure C.3: Flowchart of calculation method maximum pile bearing capacity [3]

It is important to mention that the negative skin friction plays no part in the Limit State EQU as seen in equation C.3. This is because its influence is incorporated in the Limit State STR and GEO test and the serviceability Limit State (settlements and rotations). The bearing capacity verifications need to be conducted according to NEN 9997-1;2017:

Verification of Limit State EQU

The first verification is the vertical load per pile with its bearing capacity according to art 2.4.8 NEN 9997-1;2017. The following unity check needs to be satisfied according to the following equation:

$$R_{Ed} \leq R_{c;d} \tag{C.3}$$

Verification of Limit State STR/GEO

The second verification is the Limit State STR and GEO, which contains the settlement requirements and is required by the NEN 9997. Besides that, there is a recommended criterion for (relative) rotation between the different piles. The maximum required settlements is 0.15 m, and the maximum relative rotation is 0.01 (recommended values).

$$S_d \leq S_{req} \tag{C.4}$$

$$\phi_d = 1/3 \cdot S_d/min(h.t.h) \tag{C.5}$$

$$\phi_d \leq \phi_{req} \tag{C.6}$$

Verification of Serviceability limit state
Besides the checks in the ULS, the SLS checks must also be performed. It includes both requirements for the settlements and rotations. The maximum allowed settlement for houses is 0.05 m, and the proposed criteria for (relative) rotation is 0.003 m (NEN 9997).

$$S_d \leq S_{req} \tag{C.7}$$

$$\phi_d = 1/3 \cdot S_d/min(h.t.h) \tag{C.8}$$

$$\phi_d \leq \phi_{req} \tag{C.9}$$

In Table C.2 and C.3, the bearing capacity elements per level can be seen for both Delft and Zwolle. What can be seen in Zwolle is that, due to the strong sand layers at a low depth, a relatively high bearing capacity is generated. In Delft, however, a large negative skin friction is developed within the first 20 meters, resulting in a very limited bearing capacity in those layers.

Level [m R.L.]	Number/Name	Rb;cal;max [kN]	Rs;cal;max [kN]	Rc;cal;max [kN]	Rc;d [kN]	F;nsf;k [kN]	Fnsf;d [kN]	Rc;net;d [kN]
-3.00	CPT delft	22	0	22	13	11	11	2
-4.00	CPT delft	25	0	25	15	20	20	-5
-5.00	CPT delft	30	0	30	18	33	33	-15
-6.00	CPT delft	29	0	29	17	47	47	-30
-7.00	CPT delft	63	0	63	38	64	64	-26
-8.00	CPT delft	39	0	39	23	83	83	-60
-9.00	CPT delft	39	0	39	23	106	106	-83
-10.00	CPT delft	98	0	98	59	131	131	-72
-11.00	CPT delft	60	0	60	36	159	159	-123
-12.00	CPT delft	165	0	165	99	191	191	-92
-13.00	CPT delft	104	0	104	62	225	225	-163
-14.00	CPT delft	61	0	61	37	262	262	-225
-15.00	CPT delft	52	0	52	31	302	302	-271
-16.00	CPT delft	52	0	52	31	345	345	-314
-17.00	CPT delft	212	0	212	127	390	390	-263
-18.00	CPT delft	115	0	115	69	439	439	-370
-19.00	CPT delft	140	0	140	84	491	491	-407
-20.00	CPT delft	528	72	600	360	518	518	-158
-21.00	CPT delft	582	222	804	482	518	518	-36
-22.00	CPT delft	664	363	1027	616	518	518	98
-23.00	CPT delft	753	508	1261	756	518	518	238
-24.00	CPT delft	568	672	1240	743	518	518	225
-25.00	CPT delft	355	846	1201	720	518	518	202

Table C.2: Bearing capacity per level Delft

Level [m R.L.]	Number/Name	Rb;cal;max [kN]	Rs;cal;max [kN]	Rc;cal;max [kN]	Rc;d [kN]	F;nsf;k [kN]	Fnsf;d [kN]	Rc;net;d [kN]
-3.00	CPT zwolle	119	0	119	71	17	17	54
-3.10	CPT zwolle	132	0	132	79	18	18	61
-3.20	CPT zwolle	145	0	145	87	19	19	68
-3.30	CPT zwolle	151	0	151	91	20	20	71
-3.40	CPT zwolle	165	0	165	99	21	21	78
-3.50	CPT zwolle	201	0	201	121	22	22	99
-3.60	CPT zwolle	222	7	229	137	22	22	115
-3.70	CPT zwolle	226	15	241	144	22	22	122
-3.80	CPT zwolle	229	24	253	152	22	22	130
-3.90	CPT zwolle	232	35	267	160	22	22	138
-4.00	CPT zwolle	234	45	279	167	22	22	145
-4.10	CPT zwolle	238	55	293	176	22	22	154
-4.20	CPT zwolle	242	64	306	183	22	22	161
-4.30	CPT zwolle	244	73	317	190	22	22	168
-4.40	CPT zwolle	245	82	327	196	22	22	174
-4.50	CPT zwolle	247	91	338	203	22	22	181
-4.60	CPT zwolle	256	98	354	212	22	22	190
-4.70	CPT zwolle	265	106	371	222	22	22	200
-4.80	CPT zwolle	276	113	389	233	22	22	211
-4.90	CPT zwolle	284	122	406	243	22	22	221
-5.00	CPT zwolle	290	130	420	252	22	22	230
-5.10	CPT zwolle	295	138	433	260	22	22	238
-5.20	CPT zwolle	299	147	446	267	22	22	245
-5.30	CPT zwolle	301	156	457	274	22	22	252
-5.40	CPT zwolle	312	164	476	285	22	22	263

Table C.3: Bearing capacity per level Zwolle

C.3. Structural capacity and dimensions

The structural capacity of the prestressed prefab concrete piles is calculated using NEN-EN 1992-1-1+C2 and NEN 6720:1995. The methodology for both the piles in Zwolle and Delft is similar. However, the moments due to transport and lifting are much larger in Delft because of the significantly larger pile length (23m over 5m). It must be noted that the capacity calculations of the prefab prestressed foundation piles are sometimes based on assumptions and that slight capacity deviations are possible. However, the foundation design and amount of reinforcement are based on technical drawings and reports of prefab foundation manufacturers. Therefore, the below calculations need to be seen as a verification of the existing prefab prestressed foundation piles.

C.3.1. Structural capacity Zwolle

The calculation of the prestressed prefab foundation pile in Zwolle can be seen in Table C.4 below. This calculation is done for pile 6, which has the largest length and normal forces. The other piles' lengths are calculated using the same methodology. The yellow parts are the variables that must be changed and adapted for every new foundation pile design. Again, the results may deviate slightly due to some assumptions. However, the magnitude of the results is verified with existing prefab foundation calculations. From the latest part of Table C.4, the weights and volumes of all involved foundation pile materials can be seen. These values are important for the calculation of the environmental impact.

Table C.4: Calculation of prefab prestressed

Geometry paal			
Shaft height	H	220	mm
Shaft width	W (mm)	220	mm
Length	L (mm)	5000	mm
Bruto intersection	Ac (mm^2)	48400	mm^2
Netto intersection	An (Ac-Ap)	48284	mm^2
Circumference	Oc	880	mm
Moment of resistance	Wx (1/6wh^2)	1774667	mm^3

Concrete properties			
Compressive strenght (28 days)	Fck	45	N/mm^2
Compressive strenght (stressing)	Fck,i	30	N/mm^2
E-modulus (28 days) *(table EN 1992-1-1)*	Ec	36283	N/mm^2
E-modulus (stressing)	Ec,i	32985	N/mm^2
Concrete material safety factor	Yc	1.5	-
Concrete cover (XC4) *min*	c	30.0	mm

Pressting force of reinforcement (post)			
Total prestress force [Fpw] (=3.0 N/mm2)	Fpw	145	kN

Prestressed reinforcement			
Material factor	γs	1.1	-
Tension strenght	fpk (Y1860)	1860	N/mm^2
E-modulus	Ep	195000	N/mm^2
Number of reinforcement strands	n	4	-
Diameter strands	d	7.5	mm
Surface reinforcement per bar	Ap	116	mm^2
Reinforcement ratio (> Wmin)	Wop	0.24	%
Characteristic εc3	εc3	0.002	-
Prestress force mold	fpo	36.30	kN
Steel E-modulus	Es	200000	N/mm^2
Stress concrete (prestressing)	σc	3.0	N/mm^2

Creep, shrinkage and relaxation			
Creep factor *(art. 3.1.4 NEN EN 1992-1-1)*	Φk	0.9409	-
Time factor shrinkage *(depends on cement)*	βds(t, ts)	1	-
Fictive thickness [h0]	2 * Ac / Oc	110	mm
Shrinkage coefficient *(T 3.3 NEN-EN 1992-1-1)*	Kh	0.985	-
Creep concrete [εcc]	Φk*σc/Ec *1000	0.07780	-
Autogenous shrinkage strain [εca]	2,5 (fck – 10) * 10^6	0.00009	-
Basic drying shrinkage strain [εcd]	β ds (t, ts) * kh* εcd,0	0.00042	-
Total shrinkage reduction [εcs]	εcd + εca	0.00051	-
Relaxation prestresses steel [Δσpr]*	0.4% * σp,i	5.208	N/mm^2
* mean value for relaxation			

Stresses in prestressed steel			
Prestress stress [σp,max]	Fpw * 1000 / Ap	1251.72	N/mm^2
Elastic shortening prestress [Δσp,el]	Ep * [(σp,max * Ap) / (An * Ec,i)]	-17.78	N/mm^2
Initial stress [σpi] μ=70% *	fpk* μ	1302	N/mm^2
** assumed 70%, p1000 value (3.3.2 NEN 1992), which is mean value*			

Absolute value of stress change due to creep, shrinkage and relaxation:			
Initial stress concrete [σcm,0]	σpi* Ap / An	-3.13	N/mm^2
Distance from reinforcement to c.o.g [Zcp]	h/2 - c	80.00	mm
Quadratic surface moment concrete	Ic	1.95E+08	mm^4
Absolute value stress change due to creep/shrinkage/relaxation [Δσp,c+r+s] =		82.28	N/mm^2
(εcs*Ep + 0.8*Δσpr + (Ep/Ecm)*Φk*σcm,0)/(1+(Ep/Em)*(Ap/Ac)*(1 + Ac/Ic*Zcp2)*(1+0.8*Φk), NEN-EN 1992-1-1+C2			

Prestressed steel stress and max centric load			
Working stress prestressing steel [σpw]	σpmi - Δσp,c+r+s	1219.72	N/mm^2
Working stress concrete [σcw]	σpw * Ap/An	-2.930	N/mm^2
Calculating centric pile load [Nrd,max] *(only normal force)*	(Fck / Yc * Anetto) - ((σpw-(εc3*Ep))*Ap)/1000	1352.3	kN

Moments due to Transport and Lifting foundation piles			
Self weight of foundation pile [q,eq]	ρ * h * w	1.40	kN/m
Transport moment static [Mstat,T]	q,eq/2 * (0.207*L-0.60)^2	0.13	kNm
Transport moment dynamica [Mdyn,T]	1.45 * Mstat + 1.5 * W * 10^-6	2.85	kNm
Lifting moment static [Mstat,L]	q,eq/2 * (0.207*L-B)^2	0.75	kNm
Lifting moment dynamic [Mdyn,L]	1.5 * Mstat	1.13	kNm
A = 0.207 see figure, B =0.5, impact coefficient C = 1.45, D = 1.5 with truck transport, S = 1.25 (impact coefficient)			

Maximum acceptable moment Transport and Lifting*			
Mean tensile strength	Fct,eff	3.8	N/mm2
Cracking moment capacity [Mrc,x] (Mr)	Wx * (1.3* σct,eff +σcw')/1.1	12.70	kNm
Number rows of reinforcement (Top+bottom)	n1	2	-
Max working force [Pm,infitie]	σpw * (Ap)	141	kN
Eeffective height [dp]	h - c	190	mm
Concrete presurre zone [Xu]	Ap/n1[(fpk/(ys*0.95)-σpw)+pm,i] /(0.75 * fcb * w)	34.75	mm
Strain sigma prestress top [Δεp']	Ecu * (Xu-c)	0.0005	-
Stress prestress [σp']	Ep *Δεp'	93.3	N/mm^2
Prestress force top [ΔNp']	σp' * Ap / n1	5.41	kN
Strain sigma prestress bottom [Δεp]	(dp)/xu * 3.5 *10 ^-3	0.017	-
Stress prestress bottom [σp]	330 + (Δεp-1.65)/(35 - 7.60)	330.55	N/mm^2
Prestress force botom [ΔNp]	σp * Ap/n1	19.17	kN
Compression force conrete top [Nc']	(0.75 * fci * Xu * b - fci * Ap/2)	170.3	kN
Equiliubirum check:	Fpw + ΔNp - Nc' - ΔNp'	-11.32	OK
Fracture moment [Mu]	Nc' * (h/2 - 7/18 * Xu) + (ΔNp+ ΔNp')*h/2	19.13	kNm
** Determined according to VBC art 8.1.1C, Ecu = 3.5 * 10 -3 (up to C50/60), assumed equilibrium forces*			

Capacity check transport and lifting			
Unity check dynamic transport*	Mdyn,T / Mrc,x	0.204	OK
Unity check dynamic lifting *	Mdyn,L / Mrc,x	0.067	OK
** Unity check needs to be performed with BGT check and therefore cracking moment capacity (no crakcs allowed)*			

Determination quanyity steel and concrete			
Density of steel	ρs	7850	Kg/m^3
Density of concrete	ρc	2400	Kg/m^3
Prestressed Y1860 steel volume [Vsteel]	Ap * (L + 1000) / (10^9)	0.00070	m3
Prestressed Y1860 weight [Wsteel]	ρ * Vsteel	5.464	Kg
Width of head/foot reinforcemnt [Wh,f]	W - 2 * c	160	mm
Head reinforcement volume [Vsteel,h]*	$(\pi * r^2)$ * Wh * 4 * (9 * 2)	3.26E-04	m^3
Head reinforcement weight [Wsteel,h]	ρ * Vsteel	2.557	kg
Foot reinforcement volume [Vsteel]*	$(\pi * r^2)$ * Wf * 4 * 4	7.238E-05	m^3
Foot reinforcement weight [Wsteel]	ρ * Vsteel	0.5682	kg
Total Kg reinforcement steel (regular)	Whead + Wtop	3.125	Kg
Total Kg prestressed steel	Wprestressed	5.464	Kg
Total volume concrete [Vconcrete]	W * H * L - Vreinf,total	0.241	m^3
Surface of casting formwork [Vcast]	3(H * L) + 2(W*H)	3.397E-03	m^2
Total weight concrete [Wconcrete]	Pconcrete * Vconcrete	581	kg
** Head/foot reinforcement r=3mm (BRL 2352) & assumed 0.5 m add reinforcement both side for prestressing process*			

Combination between normal force and moment			
Prestres force [Fpk]	Fpk	145	kN
Selft weight of foundation pile [Wpile]	ρ * b * h * l	6	kN
Normal force on foundation pile [Fc,ed]	Fed	155	kN
Total compressive force cap [Ned]	Fed,tot = Wpile + Fed + Fpk	306	kN
Height compressive zone [Xu]	Fed*10^3/ (3/4 * w * fcd)	61.87	mm
Strain steel pressure [εs,pressure]*	((Xu-c)/Xu)*3.5*10^-3	1.80E-03	OK
Strain steel Es, [ε,s]*	((Dp-Xu)/Xu)*3.5*10^-3	7.25E-03	OK
Force steel Ns,y = Ns	Ap/n1 * 435	25.23	kN
Moment capacity with given Fed [Mrd]	Nc*10^3*(h/2 - 7/18*Xu) + Ns*(h/2-c)+Ns,y*(h/2 - 30))/10^6	26.32	kNm
Normal force + Moment capacity pile	Nrd [kN]	306	kN
	Mrd [kN]	26.32	kNm
Total excentricity pile [e]	Eimperfection + Efinishing	50	mm
Moment due to excentricity [Mtot]	Ned * e	15.31	kNm
Unity check pile reinforcement [U.C]	Mtot / Med	0.582	OK
** Assumed that the reinforcement will yield, assumed that total vertical force = W+Fed+Fpk*			

C.3.2. Structural capacity Delft

The calculation of the prestressed prefab foundation pile in Delft can be seen in the calculations below. This calculation is almost similar for every one of the six piles. Only the vertical force and therefore the moment varies per pile. The length of each pile is the same, so the transport and lifting process results in the same forces and checks. The magnitude of the results is verified with existing prefab foundation calculations. From the latest Table, the weights and volumes of all involved foundation pile materials can be seen.

Table C.5: Calculation of prefab prestressed

Geometry paal			
Shaft height	H	290	mm
Shaft width	W (mm)	290	mm
Length	L (mm)	23000	mm
Bruto intersection	Ac (mm^2)	84100	mm^2
Netto intersection	An (Ac-Ap)	83788	mm^2
Circumference	Oc	1160	mm
Moment of resistance	Wx ($1/6wh^2$)	4064833	mm^3

Concrete properties			
Compressive strenght (28 days)	Fck	45	N/mm^2
Compressive strenght (stressing)	Fck,i	30	N/mm^2
E-modulus (28 days) *(table EN 1992-1-1)*	Ec	36283	N/mm^2
E-modulus (stressing)	Ec,i	32985	N/mm^2
Concrete material safety factor	Yc	1.5	-
Concrete cover (XC4) *min*	c	45.0	mm

Pressing force of reinforcement (post)			
Total prestress force [Fpw] (=5.0 N/mm2)	Fpw	421	kN

Prestressed reinforcement			
Material factor	γs	1.1	-
Tension strenght	fpk (Y1860)	1860	N/mm^2
E-modulus	Ep	195000	N/mm^2
Number of reinforcement strands	n	6	-
Diameter strands	d	9.3	mm
Surface reinforcement per bar	Ap	312	mm^2
Reinforcement ratio (> Wmin)	Wop	0.37	%
Characteristic εc3	εc3	0.002	-
Prestress force mold	fpo	70.08	kN
Steel E-modulus	Es	200000	N/mm^2
Stress concrete (prestressing)	σc	5.0	N/mm^2

Creep, shrinkrage and relaxation			
Creep factor *(art. 3.1.4 NEN EN 1992-1-1)*	Φk	0.9409	-
Time factor shrinkage *(depends on cement)*	βds(t, ts)	1	-
Fictive thickness [h0]	2 * Ac / Oc	145	mm
Shrinkage coefficient *(T 3.3 NEN-EN 1992-1-1)*	Kh	0.74	-
Creep concrete [εcc]	Φk*σc/Ec *1000	0.12966	-
Autogenous shrinkage strain [εca]	2,5 (fck – 10) * 10^6	0.00009	-
Basic drying shrinkage strain [εcd]	β ds (t, ts) * kh* εcd,0	0.00031	-
Total shrinkage reduction [εcs]	εcd + εca	0.00040	-
Relaxation prestresses steel [Δσpr]*	0.4% * σp,i	5.208	N/mm^2
* mean value for relaxation, interpolation with Kh table 3.3 NEN-EN 1992			

Stresses in prestressed steel			
Prestress stress [σp,max]	Fpw * 1000 / Ap	1347.76	N/mm^2
Elastic shortening prestress [Δσp,el]	Ep * [(σp,max * Ap) / (An * Ec,i)]	-29.67	N/mm^2
Initial stress [σpi] μ=70% *	fpk* μ	1302	N/mm^2
** assumed 70%, p1000 value (3.3.2 NEN 1992), which is mean value*			

Absolute value of stress change due to creep, shrinkage and relaxation:			
Initial stress concrete [σcm,0]	σpi* Ap / An	-4.85	N/mm^2
Distance from reinforcement to c.o.g [Zcp]	h/2 - c	100.00	mm
Quadratic surface moment concrete	Ic	5.89E+08	mm^4
Absolute value stress change due to creep/shrinkage/relaxation [Δσp,c+r+s] =		53.53	N/mm^2
(εcs*Ep + 0.8*Δσpr + (Ep/Ecm)*Φk*σcm,0)/(1+(Ep/Em)*(Ap/Ac)*(1 + Ac/Ic*Zcp2)*(1+0.8*Φk), NEN-EN 1992-1-1+C2			

Prestressed steel stress and max centric load			
Working stress prestressing steel [σpw]	σpmi - Δσp,c+r+s	1248.47	N/mm^2
Working stress concrete [σcw]	σpw * Ap/An	-4.649	N/mm^2
Calculating centric pile load [Nrd,max] *(only normal force)*	(Fck / Yc * Anetto) - ((σpw-(εc3*Ep))*Ap)/1000	2245.8	kN

Moments due to Transport and Lifting foundation piles			
Self weight of foundation pile [q,eq]	ρ * h * w	2.04	kN/m
Transport moment static [Mstat,T]	q,eq/2 * (0.207*L-0.60)^2	17.62	kNm
Transport moment dynamica [Mdyn,T]	1.45 * Mstat + 1.5 * W * 10^-6	31.65	kNm
Lifting moment static [Mstat,L]	q,eq/2 * (0.207*L-B)^2	23.07	kNm
Lifting moment dynamic [Mdyn,L]	1.5 * Mstat	34.60	kNm
A = 0.207 see figure, B =0.5, impact coefficient C = 1.45, D = 1.5 with truck transport, S = 1.25 (impact coefficient)			

Maximum acceptable moment Transport and Lifting*			
Mean tensile strength	Fct,eff	3.8	N/mm2
Cracking moment capacity [Mrc,x] (Mr)	Wx * (1.3* σct,eff +σcw')/γs	35.43	kNm
Number rows of reinforcement (Top+bottom)	n1	3	-
Max working force [Pm,infitie]	σpw * (Ap)	390	kN
Eeffective height [dp]	h - c	245	mm
Concrete presurre zone [Xu]	Ap/n1[(fpk/(γs*0.95)-σpw)+pm,i] /(0.75 * fcb * w)	68.09	mm
Strain sigma prestress top [Δεp']	Ecu * (Xu-c)	0.0012	-
Stress prestress [σp']	Ep *Δεp'	231.4	N/mm^2
Prestress force top [ΔNp']	σp' * Ap / n1	24.07	kN
Strain sigma prestress bottom [Δεp]	(dp)/xu * 3.5 *10 ^-3	0.011	-
Stress prestress bottom [σp]	330 + (Δεp-1.65)/(35 - 7.60)	330.36	N/mm^2
Prestress force botom [ΔNp]	σp * Ap/n1	34.36	kN
Compression force conrete top [Nc']	(0.75 * fci * Xu * b - fci * Ap/2)	439.6	kN
Equiliubirum check:	Fpw + ΔNp - Nc' - ΔNp'	-8.80	OK
Fracture moment [Mu]	Nc' * (h/2 - 7/18 * Xu) + (ΔNp+ ΔNp')*h/2	60.57	kNm
** Determined according to VBC art 8.1.1C, Ecu = 3.5 * 10 -3 (up to C50/60), assumed equilibrium forces*			

Capacity check transport and lifting			
Unity check dynamic transport*	Mdyn,T / Mrc,x	0.89	OK
Unity check dynamic lifting *	Mdyn,L / Mrc,x	0.98	OK
** Unity check needs to be performed with BGT check and therefore cracking moment capacity (no crakcs allowed)*			

Determination quanyity steel and concrete			
Density of steel	ρs	7850	Kg/m^3
Density of concrete	ρc	2400	Kg/m^3
Prestressed Y1860 steel volume [Vsteel]	Ap * (L + 1000) / (10^9)	0.00749	m3
Prestressed Y1860 weight [Wsteel]	ρ * Vsteel	58.781	Kg
Width of head/foot reinforcemnt [Wh,f]	W - 2 * c	200	mm
Head reinforcement volume [Vsteel,h]*	(π * r^2) * Wh * 4 * (9 * 2)	4.07E-04	m^3
Head reinforcement weight [Wsteel,h]	ρ * Vsteel	3.196	kg
Foot reinforcement volume [Vsteel]*	(π * r^2) * Wf * 4 * 4	9.048E-05	m^3
Foot reinforcement weight [Wsteel]	ρ * Vsteel	0.7103	kg
Total Kg reinforcement steel (regular)	Whead + Wtop	3.906	Kg
Total Kg prestressed steel	Wprestressed	58.781	Kg
Total volume concrete [Vconcrete]	W * H * L - Vreinf,total	1.926	m^3
Volume of casting formwork [Vcast]	3(H * L) + 2(W*H)	2.018E-02	m^3
Total weight concrete [Wconcrete]	Pconcrete * Vconcrete	4642	kg
** Head/foot reinforcement r=3mm (BRL 2352) & assumed 0.5 m add reinforcement both side for prestressing pro*			

Combination between normal force and moment			
Prestres force [Fpk]	Fpk	421	kN
Selft weight of foundation pile [Wpile]	ρ * b * h * l	48	kN
Nomal force on foundation pile [Fc,ed]	Fed	224	kN
Total compressive force cap [Ned]	Fed,tot = Wpile + Fed + Fpk	693	kN
Height compressive zone [Xu]	Fed*10^3/ (3/4 * w * fcd)	106.19	mm
Strain steel pressure [εs,pressure]*	((Xu-c)/Xu)*3.5*10^-3	0.00202	OK
Strain steel Es, [ε,s]*	((Dp-Xu)/Xu)*3.5*10^-3	0.00458	OK
Force steel Ns,y = Ns	Ap/n1 * 435	45.24	kN
Moment capacity with given Fed [Mrd]	Nc*10^3*(h/2 - 7/18*Xu) + Ns*(h/2-c)+Ns,y*(h/2 - 30))/10^6	71.86	kNm
Nomal force + Moment capacity pile	Nrd [kN]	693	kN
	Mrd [kN]	71.86	kNm
Total excentricity pile [e]	Eimperfection + Efinishing	50	mm
Moment due to excentricity [Mtot]	Ned * e	34.64	kNm
Unity check pile reinforcement [U.C]	Mtot / Med	0.48	OK
** Assumed that the reinforcement will yield, assumed that total vertical force = W+Fed+Fpk*			

C.3.3. Available prestressed steel diameters in NL

The commonly used qualities and diameters in the Netherlands, as mentioned in NEN 3868:2001, can be seen in Table C.6. All the prestressed strands have a tensile capacity of 1860 MPa, denoted as Y1860. It is worth mentioning that the only available option for 3-wire strands is a diameter of 7.5 mm. The limited diversity in strand diameters may lead to over-dimensioning the prestressed reinforcement and environmental impact. The difference between the 3 and 7-wire strands can be seen in Figure C.4. Figure C.4 shows the determination of the total reinforcement surface A_p of the 3 and 7 wires prestressed reinforcement. The 3-wire strand is composed of three galvanised wires of a similar diameter, which are spirally wound around a theoretical axis with a pitch of 14 to 22 times the nominal diameter of the strand. The 7-wire strand, in contrast, consists of six same-diameter wires which are spirally and tightly wound around one core wire, with a diameter which is at least 3% larger than the diameter of the twist wires. The winding pitch is between 12 and 18 times the nominal strand diameter [8].

Strengtype	**$\varnothing_k$** mm	**A_p** mm^2	**ρ_1** g/m
7-draadsstreng	15,7	150	1177,5 ± 24,0
	15,2	139	1091,2 ± 22,0
	12,9	100	785,0 ± 16,0
	12,5	93	730,1 ± 15,0
	9,3	52	408,2 ± 8,0
	6,9	29	227,7 ± 4,6
3-draadsstreng	7,5	29	227,7 ± 4,6

Table C.6: Commonly used qualities and diameters prestressed steel NL (NEN 3868:2001, Table 5)

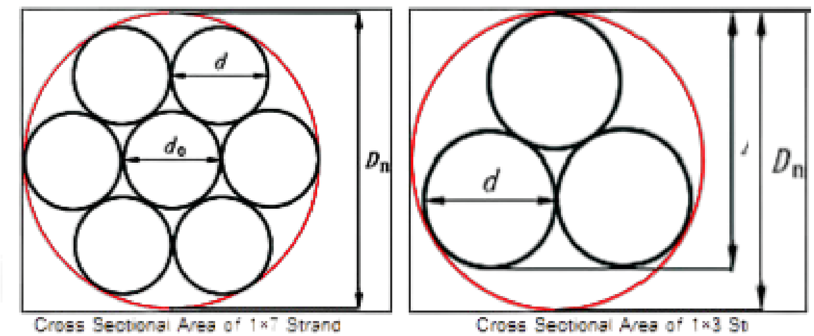

Figure C.4: Cross-section of 3 and 7 wires prestressed steel

C.3.4. Cement and concrete strength development CEMI/III

The hardening time of prefab concrete elements is a significant factor in the efficiency of the moulding process. Implementing the more sustainable CEM III instead of CEM I result in a lower ECI value of the foundation design. However, a significant disadvantage of CEM III is the slower hardening time. Table C.7 shows the strength of the cement after n days of both OPC and BFS. It can be concluded that the cement strength of BFS needs two days to reach 28 MPa instead of 1 day with OPC to reach 29 MPa. However, it must be mentioned that the values in Table C.7 are determined for the optimum conditions. With raw materials and composition precisely defined in the standard, the mortar is made with 40 x 40 x 160 mm rectangular beams. The sand/cement ratio of the mortar mortar is 3:1 (m/m). The water-cement factor is 0.5. The beams are stored at 20 °C and high RH [67]. The strength values may deviate because the (boundary) conditions in the prefab factory are less optimised. However, Table C.7 shows a proper indication between the different OPC and BFS hardening times. However, more detailed and elaborate research must be conducted on the precious hardening time of both mixtures to investigate possible strength reduction.

Table C.7: Guideline values for the average standard strength (N) of cement commonly used in the Netherlands [67]

Cement type	codes	norm strength *N* in N/mm²			
		1 day	2 days	3 days	28 days
Ordinary Portland Cement	CEM I 42,5 N	10	17	25	48
	CEM I 42,5 R	19	30	35	58
	CEM I 52,5 R	29	39	44	63
Blast Furnace Slag	CEM III/A 32,5 N	7	14	19	46
	CEM III/A 42,5 N	8	17	22	59
	CEM III/A-52,5 N	18	28	35	74
	CEM III/B 32,5 N	5	10	14	48
	CEM III/B 42,5 N	8	17	25	58

The values given in Table C.7 indicate the strength of the cement after n-days. However, this is not similar to concrete mixture strength. To determine the strength of the concrete mixtures after n-days, equation 4.6 and Table 4.11 can be used. By substituting the given parameter in equation 4.6, the mean concrete strength values after n-days are determined and given in Table C.8. It can be concluded that the strength development of the CEM III concrete mixture is slower in the first couple of days but that it reaches a higher-end strength in comparison with the CEM I mixture. Besides that, to reach a 30 Mpa strength, only two days are required with the BFS mixtures instead of 1 day with the OPC mixture. However, again, it needs to be mentioned that also the concrete strength values are based on the experimental conditions and that the real concrete strength values may deviate from those given in Table C.8

Table C.8: Concrete mixture strengths after n days for both CEM I and CEM III

CEM I 52,5 R f,cm,j			
Days, i	**C45/55**	**C40/50**	**C35/45**
1 day	29	22	17
2 days	37	30	25
3 days	41	34	29
28 days	56	49	45

CEM III 52,5 R f,cm,j			
Days, i	**C45/55**	**C40/50**	**C35/45**
1 day	26	16	9
2 days	34	24	17
3 days	39	29	22
28 days	70	61	54

D
Shallow strip calculations

D.1. Soil characteristics

The NEN 9997-1 Table 2.b displays either the low or high values according to the influence of parameters. For the bearing piles, the soil weight negatively influences the capacity. Therefore, the high values must be chosen, as depicted in Figure 3.14. However, for the shallow strip foundation, the soil weight has a beneficial effect on the capacity of the foundation. Therefore, the low characteristic soil values must be considered. The material parameters that need to be considered for the shallow strip foundation are shown in Figure D.1.

Soil name	Soil type [-]	Gamma-unsat [kN/m³]	Gamma-sat [kN/m³]	Friction angle (phi) [degree]	Cohesion (c') [kN/m²]	Cu (F_undrained) [kN/m²]	Compression ratio (Cc) [-]	Ca [-]	Initial void ratio (e0) [-]
Clay, clean, moderate	Clay	17.00	17.00	17.50	5.00	50.00	0.1535000	0.0061000	0.001001
Clay, clean, stiff	Clay	19.00	19.00	17.50	13.00	100.00	0.0921000	0.0037000	0.001001
Clay, clean, weak	Clay	14.00	14.00	17.50	0.00	25.00	0.3289000	0.0131000	0.001001
Clay, organ, moderate	Clay	15.00	15.00	15.00	0.00	25.00	0.2302000	0.0115000	0.001001
Clay, organ, weak	Clay	13.00	13.00	15.00	0.00	10.00	0.3070000	0.0153000	0.001001
Clay, sl san, moderate	Clay	18.00	18.00	22.50	5.00	80.00	0.1151000	0.0046000	0.001001
Clay, sl san, stiff	Clay	20.00	20.00	22.50	13.00	120.00	0.0768000	0.0031000	0.001001
Clay, sl san, weak	Clay	15.00	15.00	22.50	0.00	40.00	0.2302000	0.0092000	0.001001
Clay, ve san, stiff	Clay	18.00	18.00	27.50	0.00	0.00	0.0921000	0.0037000	0.001001
Gravel, sl sil, loose	Gravel	17.00	19.00	32.50	0.00	0.00	0.0058000	0.0000000	0.256082
Gravel, sl sil, moderate	Gravel	18.00	20.00	35.00	0.00	0.00	0.0029000	0.0000000	0.256082
Gravel, sl sil, stiff	Gravel	19.00	21.00	37.50	0.00	0.00	0.0024000	0.0000000	0.256082
Gravel, ve sil, loose	Gravel	18.00	20.00	30.00	0.00	0.00	0.0073000	0.0000000	0.256082
Gravel, ve sil, moderate	Gravel	19.00	21.00	32.50	0.00	0.00	0.0048000	0.0000000	0.256082
Gravel, ve sil, stiff	Gravel	20.00	22.00	35.00	0.00	0.00	0.0029000	0.0000000	0.256082
Loam, sl san, moderate	Loam	20.00	20.00	27.50	1.00	100.00	0.0512000	0.0020000	0.001001
Loam, sl san, stiff	Loam	21.00	21.00	27.50	2.50	200.00	0.0329000	0.0013000	0.001001
Loam, sl san, weak	Loam	19.00	19.00	27.50	0.00	50.00	0.0921000	0.0037000	0.001001
Loam, ve san, stiff	Loam	19.00	19.00	27.50	0.00	50.00	0.0512000	0.0020000	0.001001
Peat, mod pl, moderate	Peat	12.00	12.00	15.00	2.50	20.00	0.3070000	0.0153000	0.001001
Peat, not pl, weak	Peat	10.00	10.00	15.00	1.00	10.00	0.4605000	0.0230000	0.001001
Sand, clean, loose	Sand	17.00	19.00	30.00	0.00	0.00	0.0144000	0.0000000	0.256082
Sand, clean, moderate	Sand	18.00	20.00	32.50	0.00	0.00	0.0048000	0.0000000	0.256082
Sand, clean, stiff	Sand	19.00	21.00	35.00	0.00	0.00	0.0029000	0.0000000	0.256082
Sand, sl sil, moderate	Sand	18.00	20.00	27.00	0.00	0.00	0.0064000	0.0000000	0.256082
Sand, ve sil, loose	Sand	18.00	20.00	25.00	0.00	0.00	0.0144000	0.0000000	0.256082

Figure D.1: Characteristic material parameters for shallow foundations

As already mentioned in section 5.1, the CPT of Zwolle will be adapted slightly because of the high settlement of the shallow strip foundation due to the large loam layer. Figure D.2 depicts the small adjustment in CPT in Zwolle to investigate the ECI of a shallow strip foundation. The adjustment barely influences the pile foundations design. Therefore, the variants can still be compared unambiguously, as seen in Figure D.3. The small soil improvement will not influence the prestressed and timber foundation pile at -5 m depth. It is also assumed that the first 0.4 meters of clay will be excavated for workability reasons. This has a negative result on the bearing capacity of the shallow foundation; therefore, the clay layers are not included in the bearing capacity calculations. Removing the clay layers in the capacity calculations makes it possible to implement a small crawl space for ventilation and plumbing.

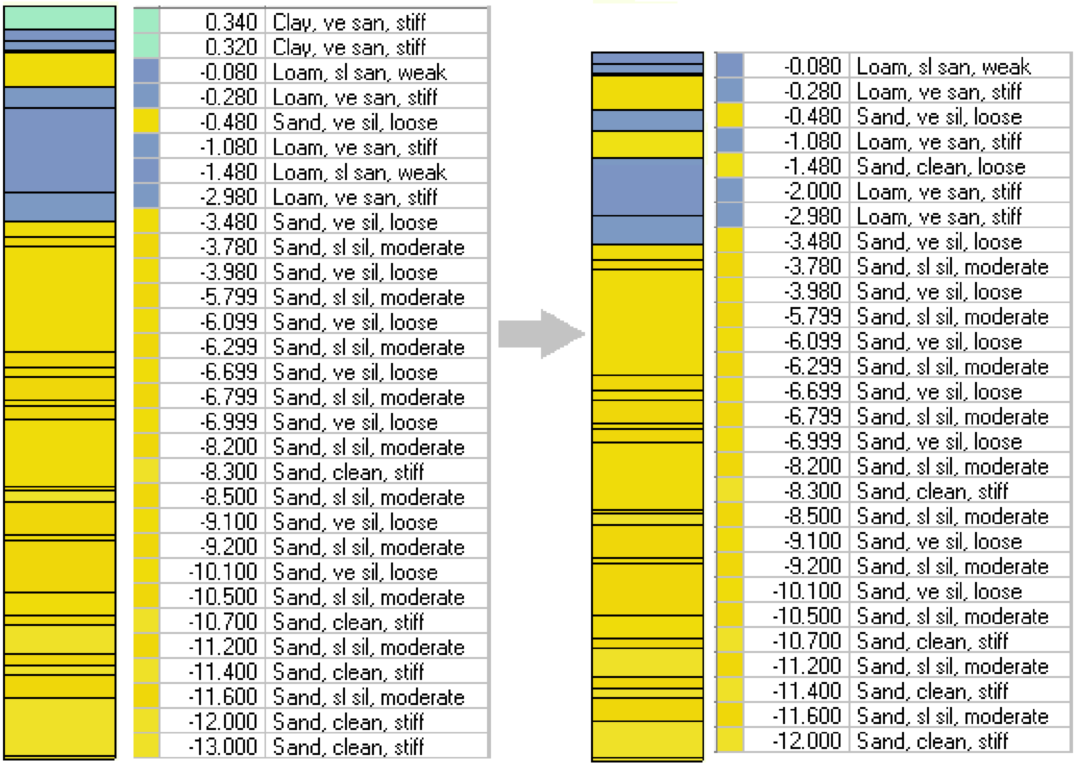

Figure D.2: Small CPT adaption in Zwolle, with additional 0.5m sand layer and no clay layer on top

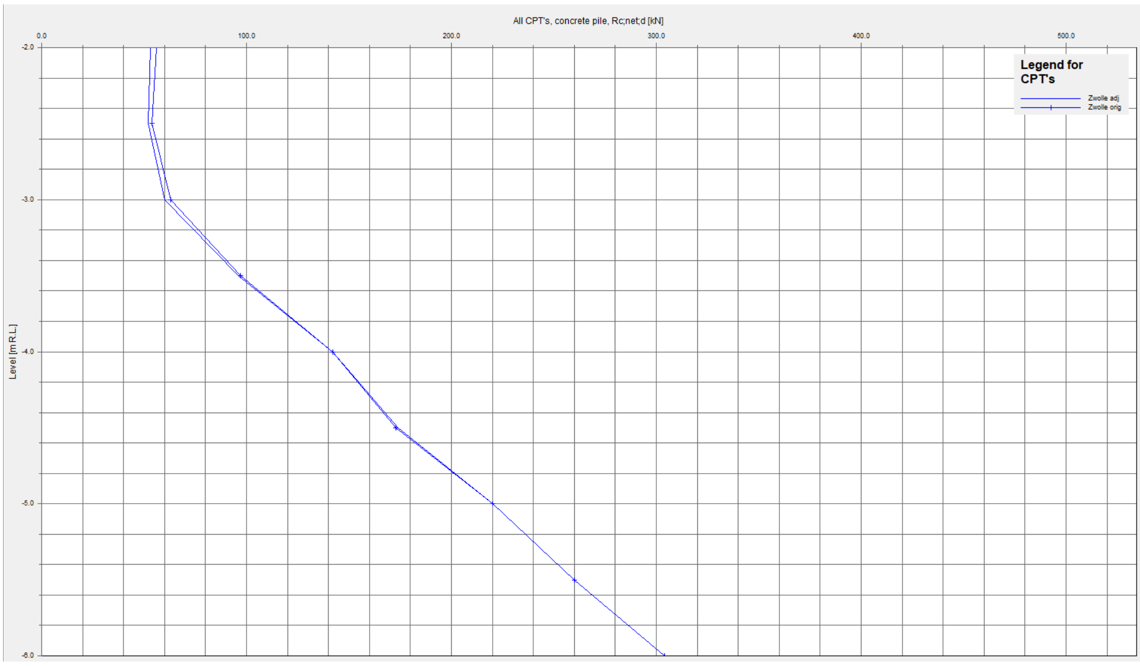

Figure D.3: Limited bearing capacity change for pile foundations due to the CPT improvement for shallow strip in Zwolle, after 3.5m no influence for pile foundation

D.2. Foundation plan and forces

As mentioned in section 5.1, it is assumed that loads from the superstructure will be transferred towards the strip foundation, according to the GTB 2010, Table 8.3a. A two-directional concrete ground floor will enable this load distribution. Figure D.4 shows the foundation plan, divided into the longitudinal (red) and front (blue) shallow strips. Considering both 8.3 and 5.1 meters lengths, the total vertical load in points 1 and 2 is 590 kN and in points 3 and 4, 90 kN (Tab. D.1). The vertical points load are required for the bearing capacity calculation in D-foundation.

Loads	Values
F1,2	590 kN
F3,4	90 kN

Table D.1: Loads on shallow foundation strip

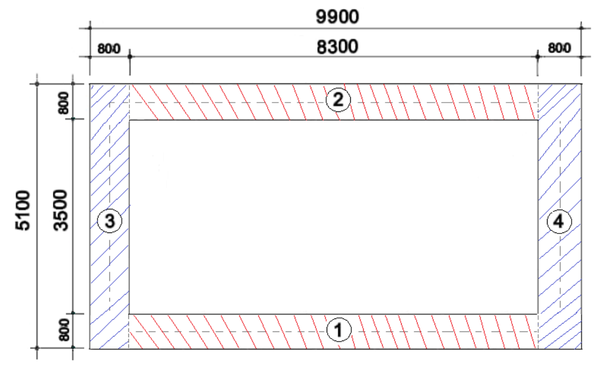

Figure D.4: Foundation plan of the shallow concrete strip foundation with simplified loads

The distribution of the forces on the longitudinal and front strip foundation are depicted in respectively Figure D.5 and D.6. Assuming a soil load of 20 kN/m^3, an effective strip width of 0.5m and a strip level at -0.8m, a total distributed soil load of 8 kN/m needs to be considered. Therefore, the difference between the load distribution, given in Appendix B.2, is that the loads shown in Figures D.5 and D.6 includes the soil loads on the strip foundation. It is assumed that the wall on top of the strip foundation will not redistribute the forces on top of the wall. Therefore, the point loads will also act on the strip foundation.

Loads	Values
q1	62.5 kN/m
q2	41.7 kN/m
q3	4.3 kN/m
F2	42.4 kN
F3	30.9 kN

Table D.2: Loads on long shallow foundation strip

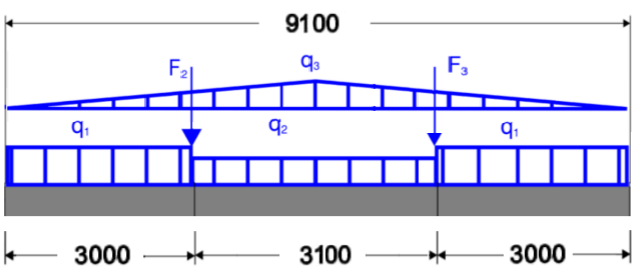

Figure D.5: Loads on shallow longitudinal strip foundation

Loads	Values
q1	17.6 kN/m

Table D.3: Design load on front foundation beam

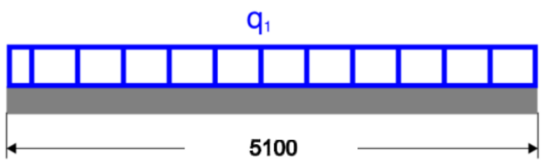

Figure D.6: Loads on front shallow foundation strip

The reaction forces can be determined from the load distribution in Figure D.5 and D.6. It is assumed that the corners in the strip foundation can transfer moments, vertical forces and torsion. A subsoil constant k of 10.000 kN/m^3 is considered for the soil in Zwolle. This constant is a relatively conservative value because the subsoil constant for sand varies around 30.000-50.000 kN/m^3 [37]. However, considering a conservative subsoil constant can compensate for the possible soil deviations under the entire strip foundation. By considering the loads on the shallow strip foundation, the self-weight of the strip foundation and the boundary condition in Technosoft. It needs to be mentioned that the forces on the shallow strip foundation are for Zwolle. In Delft, the reaction forces will differ because of the lower subsoil constant. However, the reaction forces will not be calculated because a shallow strip foundation is not feasible in Delft. The reaction forces include the self-weight of the 0.8 x 0.2m strip. From Figure D.7, it can be concluded that the maximum sagging moment is 12.2 kNm and the maximum hogging moment is 8.1 kNm. Both maximum moments occur on the longitudinal foundation strip. On the front foundation strip, the hogging moment is only 5.0 kNm. The maximum shear force is 30.5 kN, as shown in Figure D.8. The concentrated point loads on the strip especially cause the moments. The shear forces at the four corners are limited to a maximum of 13.0 kN. The small differences in reaction forces result from the different point loads F2 and F3. From the reaction forces analysis, it can be concluded that the forces on the 0.8 x 0.2-meter shallow strip foundation in Zwolle are very limited and that a minimum amount of reinforcement will be necessary.

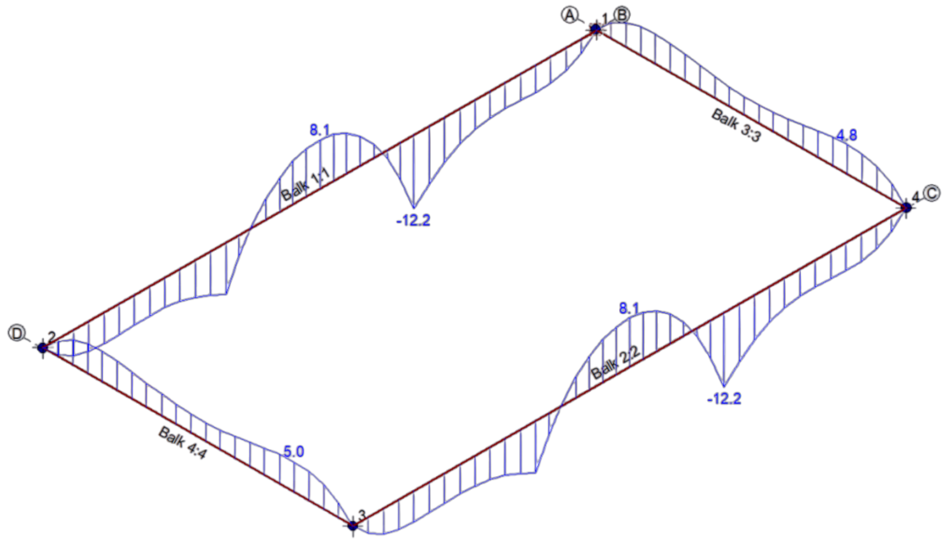

Figure D.7: Moment distribution shallow strip foundation Zwolle

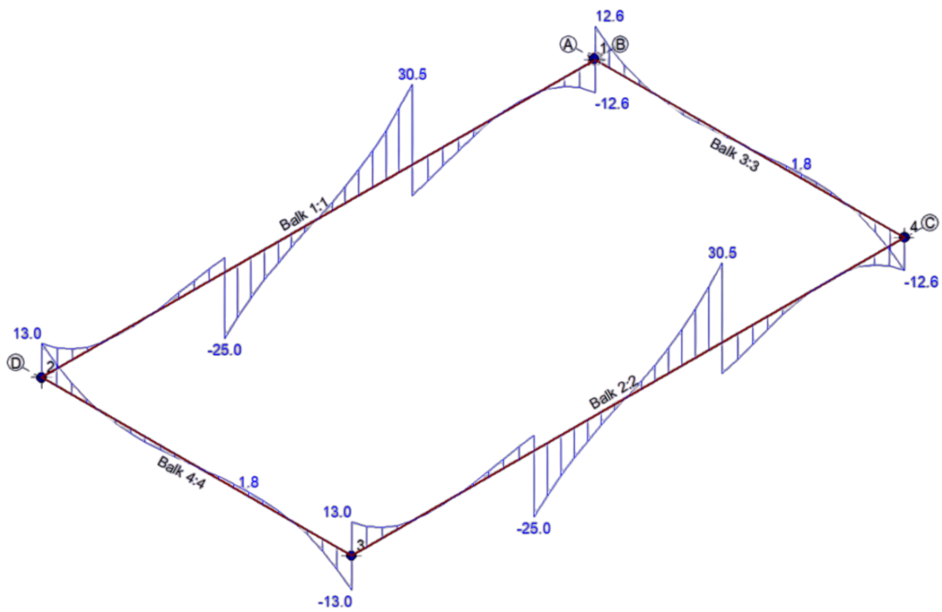

Figure D.8: Shear distribution shallow strip foundation Zwolle

D.3. Structural capacity and dimensions

The structural analysis of the concrete shallow strip foundation in Zwolle is conducted following the NEN 1992-1-1 standard. Since the strip foundation will be cast in place, there are no transportation requirements to consider. It is assumed that both the longitudinal and front foundation strips have identical concrete and reinforcement configurations. The maximum sagging moment is 12.2 kNm, and the maximum hogging moment is 8.1 kNm. Consequently, reinforcement is required on the strip foundation's upper and lower sides. This is necessary because the design tension capacity of C20/25 concrete is 0.83 N/mm^2, and the tension stress at the top is 1.5 N/mm^2, while at the bottom, it is 2.29 N/mm^2. The design tension strength can be calculated following NEN 1992-1-1 Article 12.3.1, and the equation for this calculation is provided in Equation D.1. The recommended value for $\alpha_{\text{ct,pl}}$ is 0.8.

$$f_{ctd,pl} = \alpha_{ct,pl} \cdot f_{ctk;0,05} / y_c \tag{D.1}$$

In summary, for the given C20/25 concrete with a cross-section of 200x800 mm, it is not permissible to have no reinforcement. Table D.4 illustrates the verification of the longitudinal reinforcement of the shallow strip foundation on the longitudinal strip.

Table D.4: Longitudinal reinforcement strip foundation Zwolle

NEN-EN 1992-1-1 (6.1 & 7.3)		Buigwapening + scheurwijdte					v12	
Betongegevens:								
Betonkwaliteit	C	20	/ 25	f_{ck} =	20	N/mm^2		
Staalkwaliteit	B	500		f_{cd} =	13	N/mm^2		
Milieuklasse		XC2		f_{ctm} =	2.2	N/mm^2		
Ontwerplevensduur		75	jaar	f_{yd} =	435	N/mm^2		
Kwaliteitsbeheersing		ja		$f_{ct,pl}$ =	0.83	N/mm^2		
Stortwijze		op werkvloer		$\sigma_{,t}$	2.29	N/mm^2		
Oppervlaktebehandeling beton		geen		$U.C_{;no\ reinforcement}$	2.77	-		
Constructie element		balk						
Beugeldiameter		8	mm					
Afmetingen:								
Hoogte	h =	200	mm					
Breedte	b =	800	mm	$c_{nom,beugel}$ =	40	mm		
Dekking	c =	30	mm	$c_{nom,trekwap}$ =	40	mm		
Nuttige hoogte	d =	158	mm					
Drukzonehoogte ULS	x_u =	10	mm	x_u / d =	0.063	mm		
Belastingen:								
Moment	M_{Ed} =	12.2	kNm	$A_{s;req}$ =	182	mm^2		
Moment	M_{freq} =	8.5	kNm					
Scheurmoment	M_{cr} =	11.8	kNm					
Wapening:	aantal	diam.					*voldoet*	
Buigtrek wapening	4 –	Ø 8	– mm	$A_{s;prov}$ =	201	mm^2		4.0
Buigtrek wapening		Ø 8	mm	ρ_l =	0.16	%		0.0
	$A_{s;min}$ =	176	mm^2	$A_{s;max}$ =	5056	mm^2		
Scheurvorming:							*voldoet*	
Staalspanning	$\sigma_s = M_{freq}/(d-x_?/3)/A_{s,prov})$ =	280.2	N/mm^2					
Dekking op trekwapening	=	38	mm					
Factor	$k_x = c_{toegepast}/c_{nominaal}$? 2,0 =	0.75					(NEN-EN 1992-1-1 art. 7.3.1 (5) & NB)	
Maximale scheurwijdte norm	w_{max} =	0.230	mm				(NEN-EN 1992-1-1 tabel 7.1N & NB)	
Elasticiteitsmodulus beton	$E_{cm} = 22*(f_{cm}/10)^{0,3}$ =	29962	N/mm^2				(NEN-EN 1992-1-1 tabel 3.1)	
Factor	$\alpha_e = E_s / E_{cm}$ =	6.68					(NEN-EN 1992-1-1 art. 7.3.4 (2))	
Drukzone SLS	$x_? =(-\alpha_e*\rho_l+?((\alpha_e*\rho_l)^2+(2*\alpha_e*\rho_l)))*d$ =	21	mm				Zie cement 2010 nr. 7 (en GTB 2010 14.2a)	
Effectieve hoogte 1	$h_{c,ef} = 2,5*(h-d)$ =	105	mm				(NEN-EN 1992-1-1 tabel 7.1 a) ligger)	
Effectieve hoogte 2	$h_{c,ef} = (h-x_?) / 3$ =	60	mm				(NEN-EN 1992-1-1 tabel 7.1 b) vloer)	
Effectieve hoogte maatgevend	$h_{c,ef}$ =	60	mm				(NEN-EN 1992-1-1 art. 7.3.2 (3))	
Wapeningsverhouding	$\rho_{p,eff} = A_{s;prov} / b*h_{c,ef}$ =	0.004					(NEN-EN 1992-1-1 vgl. (7.10))	
Scheurafstand	$s_{r,max} = k_3 * c + k_1*k_2*k_4* Ø / \rho_{p,eff}$ =	451.3	mm				(NEN-EN 1992-1-1 vgl. (7,11))	
Scheurafstand	$s_{r,max}$? max((50-0,8*f_{ck})Ø en 15Ø)) =	272	mm				(NEN-EN 1992-1-1 art. 7.3.4 (3) NB2016	
Wapeningsrek - betonrek	$\varepsilon_{sm} - \varepsilon_{cm}$ =	0.000324					(NEN-EN 1992-1-1 vgl. (7.9))	
Minimum criteria	minimum = $0,6 * \sigma_s / E_s$ =	0.00084					(NEN-EN 1992-1-1 vgl. (7.9))	
Scheurwijdte	$w_k = s_{r,max} (\varepsilon_{sm} - \varepsilon_{cm})$ =	0.229	mm				(NEN-EN 1992-1-1 vgl. (7.8))	

Based on the verification, it can be concluded that using 4 Ø 8 mm longitudinal reinforcement bars satisfies the requirements for the 12.2 kNm sagging and the 8.1 kNm hogging moments. The minimum required diameter for the longitudinal reinforcement is 8 mm, so it cannot be reduced. The total reinforcement area is 201 mm², which exceeds the minimum required reinforcement requirement of 176 mm². This verification is based on a concrete mixture of C20/25 and B500B reinforcement steel. The environmental class is XC2, which results in a concrete cover of 40 mm. The maximum shear force acting on the strip foundation is 30.5 kN. The C20/25 concrete with an 800x200 mm cross-section has a total shear force capacity of 55.3 kN, as indicated in Table D.5. Therefore, no additional stirrups are required for the strip foundation to resist shear forces. However, it is essential to position the longitudinal reinforcement bars correctly. To achieve this, 8 mm stirrups need to be installed at intervals of 500 mm. It's important to note that these stirrups are intended solely to maintain the proper positioning of the longitudinal reinforcement and are not designed to bear the shear forces. The reinforcement steel and concrete quantities of the strip foundation in Zwolle are listed in Table D.6.

Table D.5: Shear force verification strip foundation Zwolle

Concrete - Shear force rectangular cross sections								*v11*	
conform EC 1992-1-1, Hoofdstuk 6.2									
Strip width (b) =	800 mm		ρ_l:	0.00 %					
Strip height (h) =	200 mm		$A_{s;provided}$	0 mm²					
Cover + bgl + 1/2ø$_{;langs}$ =	44		hoek Θ | =	21.8 °	$\rho_{w,min}$	0.00072			
Effective height d =	156 mm		cot Θ =	2.50	$V_{Rd,c,bovengrens}$:	0.0 kN			
Concrete class =	C20 /25		υ_1: =	0.55	$V_{Rd,c,ondergrens}$:	55.3 kN			V_{ed} [kN]
Reinforcement =	B500 A		schaalfactor k =	2.0	$V_{rd,max}$:	285.0 kN			30.5
Stirrups				A	$V_{Rd,s}$	$V_{Rd,s}$		V_d	
	ø	h.o.h.	sneden	[mm²/mm]	[N/mm²]	[kN]		[kN]	u.c.
	No stirrups:							55.3	0.55

Table D.6: Element quantities strip foundation Zwolle

Determination quantity steel and concrete strip Zwolle			
Length of long strip	Llong	8300	mm
Length of front strip	Lfront	5100	mm
Density of steel	ρs	7850	Kg/m³
Density of concrete	ρc	2400	Kg/m³
Number of long bars [top + bottom]	n	8	-
Concrete cover	C	40	mm
Height strip	d,strip	200	mm
Width strip	w,strip	800	mm
Diameter long reinforcement	d,long	8	mm
Diameter stirrup	d,str	8	mm
Spacing stirrups	S,stirrup	500	mm
Number of stirrups [N,stirrups]	(2*Llong + 2*Lfront)/S,stirrups	54	-
Total length long reinforcement, N bars	L,total	214400	mm
Volume longitudinal reinforcement [Vlong]	1/4*(π * d,long²) * L,total	0.0108	m³
Weight longitudinal reinforcement [Wlong]	Vlong * ρs	84.60	kg
Circumference one stirrup [C,stir]	2*(w,strip - 2*c) + 2*(d,strip-2*c)	1680	mm
Volume stirrup reinforcement [Vstirrups]	1/4*(π * d,str²) * C,stir* N,stirrup	0.00453	m³
Stirrups reinforcement weight [Wstirrup]	ρ * Vsteel	35.53	kg
Total reinforcement steel [Wtotal]	(Wlong + Wstir)	120.13	kg
Total volume concrete [Vconcrete]	(d*w)*(2*Llong+2*Lfront)- Vstir - Vlong	4.273	m³
Total weight concrete [Wconcrete]	Pc * Vconcrete	10254	kg
Total weight strip foundation Zwolle	Wtotal	10375	kg

Table D.7: Non reinforced strip foundation verification C20/25, C25/30 and C30/37

NEN-EN 1992-1-1 (6.1 & 7.3)		**No reinforcement calculation C30/37**					
Betongegevens:							
Betonkwaliteit	C	30	/ 37	f_{ck} =	30	N/mm^2	
Staalkwaliteit	B	500		f_{cd} =	20	N/mm^2	
Milieuklasse		XC2		f_{ctm} =	2.9	N/mm^2	
Ontwerplevensduur		75	jaar	f_{yd} =	435	N/mm^2	
Kwaliteitsbeheersing		ja		$f_{ct.pl}$ =	1.08	N/mm^2	
Stortwijze		op werkvloer		$\sigma_{;t}$	1.02	N/mm^2	M_{Fd} / (1/6*b*h^2)
Oppervlaktebehandeling beton		geen		U.C$_{;no\ reinforcement}$	0.94	-	$\sigma_{;t}$ / $f_{ct.pl}$
Constructie element		balk		$H_{F.min}$	291	mm	M_{Ed} / ((1/6*b*$f_{ct.pl}$)^0.5)
Beugeldiameter		0	mm	0.99	≥	0.22	0.85*H_f /a ≥ √3*σ_{gd} /$f_{ctd.pl}$
Afmetingen:							
Hoogte	h =	300	mm	σ_{gd}	18	kN/m^2	
Breedte	b =	800	mm	$c_{nom.beugel}$ =	40	mm	
Dekking	c =	40	mm	$c_{nom.trekwap}$ =	40	mm	
Nuttige hoogte	d =	260	mm	a =	250	mm	
Drukzonehoogte ULS	x_u =	4	mm	x_u / d =	0.015	mm	
Moment	M_{Ed} =	12	kNM	Scheurmoment=	13.0	kNm	

NEN-EN 1992-1-1 (6.1 & 7.3)		**No reinforcement calculation C25/30**					
Betongegevens:							
Betonkwaliteit	C	25	/ 30	f_{ck} =	25	N/mm^2	
Staalkwaliteit	B	500		f_{cd} =	17	N/mm^2	
Milieuklasse		XC2		f_{ctm} =	2.6	N/mm^2	
Ontwerplevensduur		75	jaar	f_{yd} =	435	N/mm^2	
Kwaliteitsbeheersing		ja		$f_{ct.pl}$ =	0.96	N/mm^2	
Stortwijze		op werkvloer		$\sigma_{;t}$	0.87	N/mm^2	M_{Fd} / (1/6*b*h^2)
Oppervlaktebehandeling beton		geen		U.C$_{;no\ reinforcement}$	0.90	-	$\sigma_{;t}$ / $f_{ct.pl}$
Constructie element		balk		$H_{F.min}$	309	mm	M_{Ed} / ((1/6*b*$f_{ct.pl}$)^0.5)
Beugeldiameter		0	mm	1.05	≥	0.24	0.85*H_f /a ≥ √3*σ_{gd} /$f_{ctd.pl}$
Afmetingen:							
Hoogte	h =	325	mm	σ_{gd}	18	kN/m^2	
Breedte	b =	800	mm	$c_{nom.beugel}$ =	40	mm	
Dekking	c =	40	mm	$c_{nom.trekwap}$ =	40	mm	
Nuttige hoogte	d =	285	mm	a =	250	mm	
Drukzonehoogte ULS	x_u =	4	mm	x_u / d =	0.015	mm	
Moment	M_{Fd} =	12	kNM	Scheurmoment=	13.5	kNm	

NEN-EN 1992-1-1 (6.1 & 7.3)		**No reinforcement calculation C20/25**					
Betongegevens:							
Betonkwaliteit	C	20	/ 25	f_{ck} =	20	N/mm^2	
Staalkwaliteit	B	500		f_{cd} =	13	N/mm^2	
Milieuklasse		XC2		f_{ctm} =	2.2	N/mm^2	
Ontwerplevensduur		75	jaar	f_{yd} =	435	N/mm^2	
Kwaliteitsbeheersing		ja		$f_{ct.pl}$ =	0.83	N/mm^2	
Stortwijze		op werkvloer		$\sigma_{;t}$	0.75	N/mm^2	M_{Fd} / (1/6*b*h^2)
Oppervlaktebehandeling beton		geen		U.C$_{;no\ reinforcement}$	0.91	-	$\sigma_{;t}$ / $f_{ct.pl}$
Constructie element		balk		$H_{F.min}$	333	mm	M_{Ed} / ((1/6*b*$f_{ct.pl}$)^0.5)
Beugeldiameter		0	mm	1.13	≥	0.26	0.85*H_f /a ≥ √3*σ_{gd} /$f_{ctd.pl}$
Afmetingen:							
Hoogte	h =	350	mm	σ_{gd}	18	kN/m^2	
Breedte	b =	800	mm	$c_{nom.beugel}$ =	40	mm	
Dekking	c =	40	mm	$c_{nom.trekwap}$ =	40	mm	
Nuttige hoogte	d =	310	mm	a =	250	mm	
Drukzonehoogte ULS	x_u =	5	mm	x_u / d =	0.016	mm	
Moment	M_{Ed} =	12	kNM	Scheurmoment=	13.5	kNm	

E

Timber pile calculations

E.1. Foundation plan and forces

The regular foundation plan for the FLOW housing unit with six foundation piles does not satisfy timber piles because of the limited bearing capacity. The maximum ULS load of a timber foundation pile is roughly 200 kN (pressure), which is lower than the maximum occurring normal force of 224 kN on foundation pile six. Therefore, additional piles need to be implemented into the foundation plan given in subsection 4.2.1. The loads on the foundation beams remain similar. This results in the following foundation plan in figure E.1 is assumed, where the grid lines remain constant and similar to the concrete variant. There will be 12 timber foundation piles, which are symmetric and equally divided at a length of 0.65m, 2.20m, 3.75m, 5.30m, 6.85m, and 8.40m.

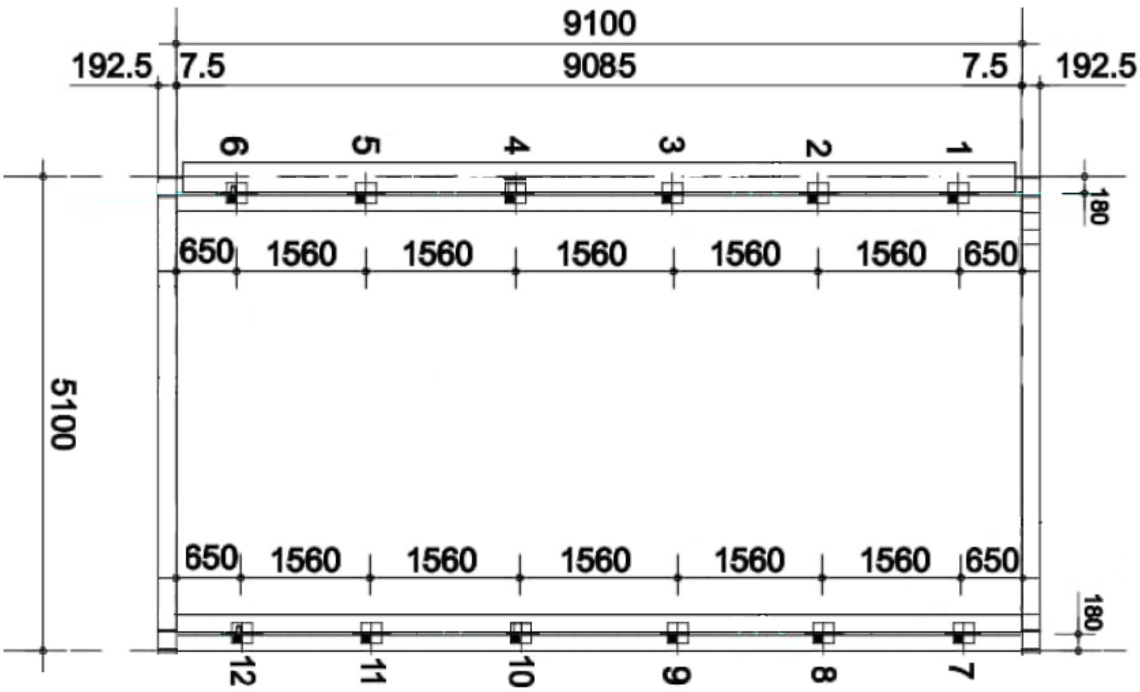

Figure E.1: Foundation plan for 12 timber foundation piles (boxes)

The stiffness of the timber piles in the z-direction will deviate from the concrete piles and can be calculated using the following formula:

$$K = \frac{EA}{2L} \tag{E.1}$$

Considering the surface of the pile of 20106 mm^2 ($\varnothing$ 160 mm), the E-modulus of 10000 MPa and a mean estimated length of 8 m, the stiffness of the pile is estimated on 12500 MPa. Therefore, the six piles are considered springs with a stiffness of 12500 MPa, which can only have displacement in the z-direction. By considering the above characteristics and the governing loads on the foundation beams, the following results are obtained; the shear force distribution in figure E.2 and the moment distribution in figure E.3.

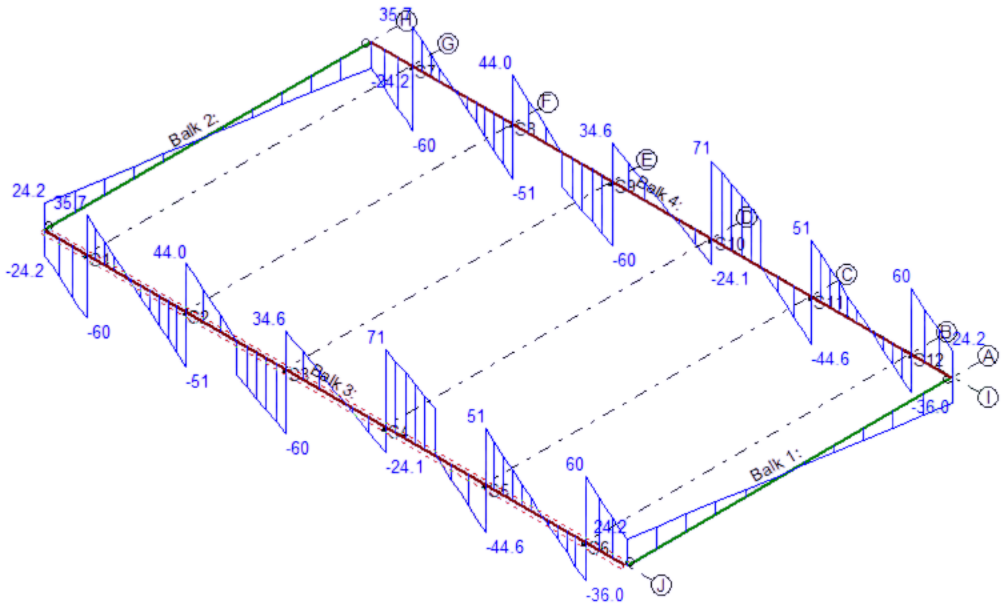

Figure E.2: Shear forces distribution on timber foundation beams/piles

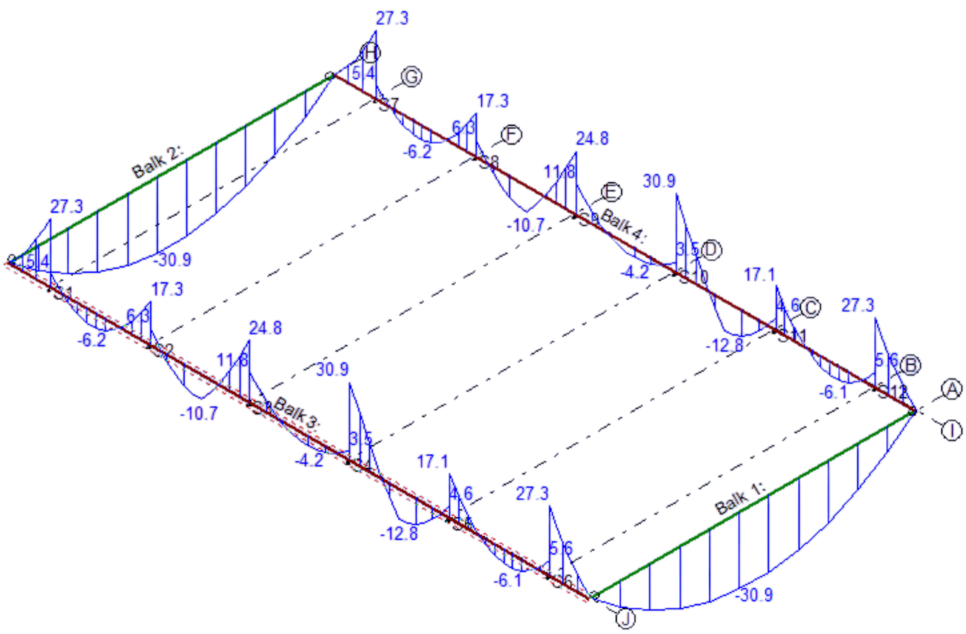

Figure E.3: Moment distribution on timber foundation piles

E.2. Bearing capacity Zwolle and Delft

Table E.1 and E.2 below shows the bearing capacity of the timber foundation piles in respectively Zwolle and Delft. It can be concluded that the bearing capacity in Delft is very low, because of the large negative skin friction and thereby low net bearing capacity.

Level [m R.L.]	Number/Name	Rb;cal;max [kN]	Rs;cal;max [kN]	Rc;cal;max [kN]	Rc;d [kN]	F;nsf;k [kN]	Fnsf;d [kN]	Rc;net [kN]
-3.00	CPT Zwolle	76	0	76	46	11	11	35
-3.10	CPT Zwolle	84	0	84	50	12	12	38
-3.20	CPT Zwolle	94	0	94	56	12	12	44
-3.30	CPT Zwolle	98	0	98	59	13	13	46
-3.40	CPT Zwolle	104	0	104	62	14	14	48
-3.50	CPT Zwolle	128	0	128	77	14	14	63
-3.60	CPT Zwolle	148	5	153	92	14	14	78
-3.70	CPT Zwolle	155	11	166	100	14	14	86
-3.80	CPT Zwolle	154	17	171	103	14	14	89
-3.90	CPT Zwolle	155	25	180	108	14	14	94
-4.00	CPT Zwolle	157	32	189	113	14	14	99
-4.10	CPT Zwolle	160	39	199	119	14	14	105
-4.20	CPT Zwolle	163	45	208	125	14	14	111
-4.30	CPT Zwolle	166	52	218	131	14	14	117
-4.40	CPT Zwolle	167	59	226	135	14	14	121
-4.50	CPT Zwolle	169	65	234	140	14	14	126
-4.60	CPT Zwolle	175	70	245	147	14	14	133
-4.70	CPT Zwolle	181	75	256	153	14	14	139
-4.80	CPT Zwolle	188	81	269	161	14	14	147
-4.90	CPT Zwolle	192	87	279	167	14	14	153
-5.00	CPT Zwolle	196	93	289	173	14	14	159
-5.10	CPT Zwolle	198	99	297	178	14	14	164
-5.20	CPT Zwolle	199	105	304	182	14	14	168
-5.30	CPT Zwolle	199	111	310	186	14	14	172
-5.40	CPT Zwolle	205	117	322	193	14	14	179
-5.50	CPT Zwolle	208	123	331	198	14	14	184
-5.60	CPT Zwolle	209	130	339	203	14	14	189
-5.70	CPT Zwolle	217	136	353	212	14	14	198
-5.80	CPT Zwolle	228	143	371	222	14	14	208
-5.90	CPT Zwolle	227	150	377	226	14	14	212

Table E.1: Bearing capacity per level Zwolle timber pile

Level [m R.L.]	Number/Name	Rb;cal;max [kN]	Rs;cal;max [kN]	Rc;cal;max [kN]	Rc;d [kN]	F;nsf;k [kN]	Fnsf;d [kN]	Rc;net;d [kN]
-18.00	CPT delft	214	0	214	128	535	535	-407
-19.00	CPT delft	262	0	262	157	598	598	-441
-20.00	CPT delft	873	36	909	545	651	651	-106
-21.00	CPT delft	1012	218	1230	737	651	651	86
-22.00	CPT delft	1134	390	1524	914	651	651	263
-23.00	CPT delft	1415	567	1982	1188	651	651	537
-24.00	CPT delft	870	767	1637	981	651	651	330
-25.00	CPT delft	672	979	1651	990	651	651	339
-26.00	CPT delft	677	1167	1844	1106	651	651	455

Table E.2: Bearing capacity per level Delft timber pile

E.3. Structural capacity and dimensions

Table E.3 shows the characteristic strength values of the timber strength classes C14 to C35, which can be used for the structural calculations.

	f_{mo}	E_{ser}	f_{to}	$f_{t,90}$	$f_{c,0}$	$f_{c,90}$	$f_{v,0}$	$E_{o,u}$	$f_{90,ser}$	G_{ser}	
C18	18	9000	11	0,5	18	2,2	2,0	6000	300	560	MPa

Table E.3: Characteristic strength values C18 spruce timber

E.3.1. Structural capacity Zwolle

The timber pile part is subjected to bending (due to eccentricity) and axial compression. Therefore, the following verification calculations are conducted, which can be seen in table E.4. It is assumed that the soil is stiff enough to prevent buckling of the foundation pile, therefore a buckling length of 1 mm is considered. The climate class is 3, corresponding with the outside environment without cover. The load duration is middle long because of the balance between permanent and variable. The ∅ 210 mm timber pile is modelled with a height and width. However, the characteristic moment of resistance is calculated for the circular shape and therefore, corresponds with the design given in Figure 6.4. The normal force of 96 kN and a bending moment of 4.8 kNm must be considered, as seen in Table 6.3. Therefore, the critical UC from equations 6.19 and 6.20 given in Table E.4, results in 0.82. Therefore, it can be concluded that the timber foundation element in Zwolle satisfies the structural requirements.

BAM Advies & Engineering				**Column bending + compression**							
Timber Foundation pile Zwolle								b	200		
Diameter: 200 mm								h	200		
1								N_{Ed}	96.0		
Strenght class	naaldhout C18							$M_{y,Ed}$	5.0		
Materiaal	Sawn timber							$M_{z,Ed}$	0.0		
formula	6.19:	0.82	formula	6.20:	0.82	formule 6.1	6.23:	n.v.t.	formule 6.15:	6.24:	n.v.t.
Note: Width and height are transformed to circulair shape											
Timber width b		200	mm	Climate class		3		γ_M	strength	1.30	-
Timber heigth h		200	mm	Load duration class		middellang		k_h	bending	1.00	-
Compressive force N_{Ed}		96.0	kN	factor volume-effect s		0.12		k_{mod}	strength	0.50	-
moment $M_{y,Ed}$		5.0	kNm	Cross section		niet rechthoekig		k_{mod}	tension	0.40	-
moment $M_{z,Ed}$		0.0	kNm	Buckling length ly		1	mm	k_{def}	deformation	2.00	-
				Buckling length lz		1	mm	W_y		785	10^3mm^3
$f_{m;d}$		9.00	N/mm^2	$E_{0;mean;d}$		9000	N/mm^2	W_z		785	10^3mm^3
$f_{c;0;d}$		9.00	N/mm^2	$E_{0;u;d}$		6000	N/mm^2	A		314	10^2mm^2
$f_{c;90;d}$		1.10	N/mm^2	I_y		7853	10^4mm^4	i_y		50.0	mm
$f_{v;d}$		1.70	N/mm^2	I_z		7853	10^4mm^4	i_z		50.0	mm
								$k_{c;y}$		1.06	-
$\sigma_{c;0;d}$		3.1	N/mm^2	$\lambda_{rel;y}$		0.000	-	$k_{c;z}$		1.06	-
$\sigma_{m;y;d}$		6.4	N/mm^2	$\lambda_{rel;z}$		0.000	-	k_y		0.47	-
$\sigma_{m;z;d}$		0.0	N/mm^2	k_m		1.0	-	k_z		0.47	-
results											
6.19		$\sigma^2_{c;0;d}$	+	$\sigma_{m;y;d}$	+	k_m	$\sigma_{m;z;d}$		=	0.8	-
		$f^2_{c;0;d}$		$f_{m;y;d}$			$f_{m;z;d}$				
6.20		$\sigma^2_{c;0;d}$	+	k_m	$\sigma_{m;y;d}$	+	$\sigma_{m;z;d}$		=	0.82	-
		$f^2_{c;0;d}$			$f_{m;y;d}$		$f_{m;z;d}$				
6.23		$\sigma_{c;0;d}$	+	$\sigma_{m;y;d}$	+	k_m	$\sigma_{m;z;d}$		=	n.v.t.	-
	$k_{c;y}$	$f_{c;0;d}$		$f_{m;y;d}$			$f_{m;z;d}$				
6.24		$\sigma_{c;0;d}$	+	k_m	$\sigma_{m;y;d}$	+	$\sigma_{m;z;d}$		=	n.v.t.	-
	$k_{c;z}$	$f_{c;0;d}$			$f_{m;y;d}$		$f_{m;z;d}$				

Table E.4: Structural calculation of timber pile in Zwolle

Besides the structural timber calculation, the concrete cap and steel tube must be calculated. Table E.5 shows the steel tube connection verification of Zwolle. Verifying the steel tube results in a UC of 0.193 in Zwolle. However, it must be mentioned that the calculation of the steel tube connection is simplified, and often, the critical failure is the timber instead of the steel tube [58]. With the above structural capacity checks, the final element quantities of the timber foundation design in Zwolle (Fig. 6.4) can be calculated, which is required to calculate the environmental impact subsequently. In Table E.6, the total weights of the timber, reinforcement steel, steel tube and concrete are given for the 12 foundation piles in Zwolle.

Calculation steel tube connection			
Density of steel	ρs	7850	Kg/m^3
Density of concrete	ρc	2400	Kg/m^3
Density of C18 spruce	ρt	469	Kg/m^3
Steel material safety factor	γM0	1	-
Steel quality	S	235	-
Tensional strength steel	f,u	360	N/mm^2
Yielding strength steel	f,yd	235	N/mm^2
Diameter steel tube	d	210	mm
Thickness steel tube	t	4	mm
Steel quality	S	235	-
Moment of resistance [W,pl]	$(D^3 - (d-2*t)^3)/6$	169765	mm^3
Tensional strength steel	f,u	360	N/mm^2
Yielding strength steel	f,yd	235	N/mm^2
Moment capacity steel tube [M,	Wpl * f,yd / γ,M0	39.89	kNm
Normal force on pile	Ned	96.00	kN
Calculation value bending mom	Ned * 0.08	8	kNm
Unity check steel tube [UC]	Med/M,rd,pl	0.193	-

Table E.5: Steel tube verification Zwolle

Determination quantity steel, concrete and timber			
Number of piles	Npiles	12	-
Density of steel	ρs	7850	Kg/m^3
Density of concrete	ρc	2400	Kg/m^3
Density of C18 spruce	ρt	469	Kg/m^3
Number of long bars	n	6	-
Diameter concrete cap	d,cap	230	mm
Thickness steel S235 tube	t	4	mm
Steel tube diamter	d,tube	210	mm
Length steel tube	L,tube	400	mm
Diameter spruce timber pile C18	d,timber	200	mm
Length timber part	L,timber	3500	mm
Length concrete cap	L,cap	1500	mm
Diameter long reinforcement	d	6	mm
Volume longitudinal reinforcement [Vlong]	$1/4*(\pi * d^2) * n * L$	2.54E-04	m^3
Weight longitudinal reinforcement [Wlong]	Vlong * ρs	1.998	kg
Diameter head/foot spiral reinforcement	D,spiral	4.5	mm
Surface of spiral reinforcement [Aspiral]	$1/4*(\pi * d^2)$	1.59E-05	m^2
Total number of windings spiral (head+foot)	N	16	-
Circumference one spiral reinforcement [C]	π*D	0.534	m
Spiral reinforcement volume [Vspiral]	C * Aspiral * N	1.36E-04	m^3
Spiral reinforcement weight [Wspir,steel]	ρ * Vsteel	1.067	kg
Total reinforcement steel [Wtotal]	(Wlong + Wspiral) * Npiles	3.064	kg
Total reinforcement steel [Wtotal], N piles	(Wlong + Wspiral) * Npiles	36.77	kg
Volume S235 tube steel [Vsteeltube]	(1/4*(π * d1) - 1/4 *(π * d2)) * L,tube	1.04E-03	m3
Total weight steel tube [Wsteeltube]	Vsteeltube * ρs	8.13	kg
Total weight steel tube, N piles	Wsteeltube	97.54	kg
Total volume concrete [Vconcrete]	$1/4*(\pi * d,cap^2)$ * L,cap- Vspir - Vlong	0.062	m^3
Total volume concrete n pile	N * Vconcrete	0.743	m^3
Total weight concrete [Wconcrete]	Pc * Vconcrete	148.6	kg
Total weight concrete [Wconcrete], N piles	Pc * Vconcrete * Npiles	1784	kg
Total volume timber [Vtimber]	$1/4*(\pi * d,timber^2)$ * L,timber	0.110	m^3
Total weight timber	ρt * Vtimber	51.57	kg
Total weight timber, N piles	ρt * Vtimber * Npiles	619	kg
Total weight of all N piles	Wtotal	2536.76	kg

Table E.6: Calculation element quantities timber foundation pile Zwolle

E.3.2. Structural capacity Delft

Also, the timber pile part in Delft is subjected to both bending (due to eccentricity) and axial compression. The same assumptions as in Zwolle are applied. The ∅ 450 mm to 320 mm tapered timber pile will be modelled as a straight timber pile with a diameter of 320 mm. However, the characteristic moment of resistance is calculated for the circular shape and therefore, corresponds with the design given in Figure 6.4. The normal force of 96 kN and a bending moment of 4.8 kNm must be considered, as seen in Table 6.3. Therefore, the critical UC from equations 6.19 and 6.20 in Table E.4, results in 0.19. It can be concluded that the timber foundation element in Delft also satisfies the structural requirements.

BAM Advies & Engineering				**Column bending + compression**							
Timber Foundation pile Delft								b	320		
Diameter: 320 mm								h	320		
1								N_{Ed}	96.0		
Strenght class	naaldhout C18							$M_{y,Ed}$	5.0		
Materiaal		Sawn timber						$M_{z,Ed}$	0.0		
formula	6.19:	0.19	formula	6.20:	0.19	formule 6.1	6.23:	**n.v.t.**	formule 6.15:	6.24:	**n.v.t.**
Note: Width and height are transformed to circulair shape											
Timber width b		320	mm	Climate class		3		γ_M	strength	1.30	-
Timber heigth h		320	mm	Load duration class		middellang		k_h	bending	1.00	-
Compressive force N_{Ed}		96.0	kN	factor volume-effect s		0.12		k_{mod}	strength	0.50	-
moment $M_{y,Ed}$		5.0	kNm	Cross section		niet rechthoekig		k_{mod}	tension	0.40	-
moment $M_{z,Ed}$		0.0	kNm	Buckling length ly		1	mm	k_{def}	deformation	2.00	-
				Buckling length lz		1	mm	W_y		3217	10^3mm^3
$f_{m;d}$		9.00	N/mm^2	$E_{0;mean;d}$		9000	N/mm^2	W_z		3217	10^3mm^3
$f_{c;0;d}$		9.00	N/mm^2	$E_{0;u;d}$		6000	N/mm^2	A		804	10^2mm^2
$f_{c;90;d}$		1.10	N/mm^2	I_y		51468	10^4mm^4	i_y		80.0	mm
$f_{v;d}$		1.70	N/mm^2	I_z		51468	10^4mm^4	i_z		80.0	mm
								$k_{c;y}$		1.06	-
$\sigma_{c;0;d}$		1.2	N/mm^2	$\lambda_{rel;y}$		0.000	-	$k_{c;z}$		1.06	-
$\sigma_{m;y;d}$		1.6	N/mm^2	$\lambda_{rel;z}$		0.000	-	k_y		0.47	-
$\sigma_{m;z;d}$		0.0	N/mm^2	k_m		1.0	-	k_z		0.47	-
results											
6.19		$\sigma^2_{c;0;d}$	+	$\sigma_{m;y;d}$	+	k_m	$\sigma_{m;z;d}$		=	0.2	-
		$f^2_{c;0;d}$		$f_{m;y;d}$			$f_{m;z;d}$				
6.20		$\sigma^2_{c;0;d}$	+	k_m	$\sigma_{m;y;d}$	+	$\sigma_{m;z;d}$		=	0.19	-
		$f^2_{c;0;d}$			$f_{m;y;d}$		$f_{m;z;d}$				
6.23		$\sigma_{c;0;d}$	+	$\sigma_{m;y;d}$	+	k_m	$\sigma_{m;z;d}$		=	n.v.t.	-
	$k_{c;y}$	$f_{c;0;d}$		$f_{m;y;d}$			$f_{m;z;d}$				
6.24		$\sigma_{c;0;d}$	+	k_m	$\sigma_{m;y;d}$	+	$\sigma_{m;z;d}$		=	n.v.t.	-
	$k_{c;z}$	$f_{c;0;d}$			$f_{m;y;d}$		$f_{m;z;d}$				

Table E.7: Structural calculation of timber pile in Delft

Besides the structural timber calculation, the steel tube must also be verified. Table E.5 shows the steel tube connection verification in Delft. The verification of the steel tube results in a UC of 0.039. However, it is assumed that the thickness of 4 mm can not be reduced because of the minimum required thickness according to the NEN-EN 1993.

Calculation steel tube connection			
Density of steel	ρs	7850	Kg/m^3
Density of concrete	ρc	2400	Kg/m^3
Density of C18 spruce	ρt	469	Kg/m^3
Steel material safety factor	γM0	1	-
Steel quality	S	235	-
Tensional strength steel	f,u	360	N/mm^2
Yielding strength steel	f,yd	235	N/mm^2
Diameter steel tube	d	460	mm
Thickness steel tube	t	4	mm
Steel quality	S	235	-
Moment of resistance [W,pl]	$(D^3 - (d-2*t)^3)/6$	831765	mm^3
Tensional strength steel	f,u	360	N/mm^2
Yielding strength steel	f,yd	235	N/mm^2
Moment capacity steel tube [M,pl,rd]	Wpl * f,yd / γ,M0	195.46	kNm
Nomal force on pile	Ned	96.00	kN
Calculation value bending moment [Med]	Ned * 0.08	8	kNm
Unity check steel tube [UC]	Med/M,rd,pl	0.039	-

Table E.8: Calculation of steel tube in Delft

The verification of the timber foundation piles during transport and lifting can be seen in Table E.9. As expected, it can be concluded that the timber foundation pile can easily bear the moments during transport and lifting. This is caused by the low self-weight of the timber and relatively large dimensions and strength properties. Because the foundation pile in Delft satisfies the requirements amply, verifying the much shorter timber pile in Zwolle will also meet the requirements.

Moments due to Transport and Lifting timber foundation piles			
Diameter pile	d	320	mm
Strength class C18	f,md	18	N/mm^2
Bending strength [f,mk]	f,md / jm3	13.85	N/mm^2
Pile length	L	19500	mm
Moment resistance [W]	pi() * R^3 / 4	3.22E+06	mm^3
Self weight of foundation pile [q,eq]	ρ * A * g	0.378	kN/m
Transport moment static [Mstat,T]	q,eq/2 * (0.207*L-0.60)^2	2.23	kNm
Transport moment dynamic [Mdyn,T]	1.45 * Mstat + 1.5 * W * 10^-6	8.06	kNm
Lifting moment static [Mstat,L]	q,eq/2 * (0.207*L-B)^2	3.08	kNm
Lifting moment dynamic [Mdyn,L]	1.5 * Mstat	4.62	kNm
Maximum bending stress [σ,ed]	Mmax / w	2.51	N/mm^2
Critical Unity check dynamic transport [UC]	σ,ed / f,mk	0.18	-

Table E.9: Transport and lifting verification timber pile element

With the above structural capacity checks, the final element quantities of the timber foundation design in Delft can be calculated, which is required to calculate the environmental impact subsequently. In Table E.10, the total weights of the timber, reinforcement steel, steel tube and concrete are given for the 12 foundation piles in Delft. In Table E.10, the quantities of the foundation design pile in Delft as shown in Figure 6.5) are shown. These quantities are required for the calculation of the ECI values.

Determination quantity steel, concrete and timber Delft			
Number of piles	Npiles	12	-
Density of steel	ρs	7850	Kg/m^3
Density of concrete	ρc	2400	Kg/m^3
Density of C18 spruce	ρt	469	Kg/m^3
Number of long bars	n	6	-
Diameter concrete cap	d,cap	480	mm
Thickness steel S235 tube	t	4	mm
Steel tube diamter	d,tube	460	mm
Length steel tube	L,tube	400	mm
Pile tip diameter	d,tip	320	mm
Diameter spruce timber pile C18 top	d,timber	450	mm
Length timber part	L,timber	19500	mm
Length concrete cap	L,cap	3500	mm
Diameter long reinforcement	d	8	mm
Volume longitudinal reinforcement [Vlong]	$1/4*(\pi * d^2) * n * L$	1.06E-03	m^3
Weight longitudinal reinforcement [Wlong]	Vlong * ρs	8.286	kg
Diameter head/foot spiral reinforcement	D,spiral	6.0	mm
Surface of spiral reinforcement [Aspiral]	$1/4*(\pi * d^2)$	2.83E-05	m^2
Total number of windings spiral (head+foot)	N	16	-
Circumference one spiral reinforcement [C]	π*D	1.319	m
Spiral reinforcement volume [Vspiral]	C * Aspiral *N	5.97E-04	m^3
Spiral reinforcement weight [Wspir,steel]	ρ * Vsteel	4.686	kg
Total reinforcement steel [Wtotal]	(Wlong + Wspiral) * Npiles	12.972	kg
Total reinforcement steel [Wtotal], N piles	(Wlong + Wspiral) * Npiles	155.66	kg
Volume S235 tube steel [Vsteeltube]	(1/4*(π * d1) - 1/4 *(π * d2)) * L,tube	2.29E-03	m3
Total weight steel tube [Wsteeltube]	Vsteeltube * ρs	17.99	kg
Total weight steel tube, N piles	Wsteeltube	215.92	kg
Total volume concrete [Vconcrete]	$1/4*(\pi * d,cap^2)$ * L,cap- Vspir - Vlong	0.632	m^3
Total volume concrete n pile	N * Vconcrete	7.580	m^3
Total weight concrete [Wconcrete]	Pc * Vconcrete	1516.1	kg
Total weight concrete [Wconcrete], N piles	Pc * Vconcrete * Npiles	18193	kg
Total volume timber [Vtimber]	1/3*π*l,timber*(Rtip^2+Rtip*Rd,timber+	2.292	m^3
Total weight timber	ρt * Vtimber	1074.80	kg
Total weight timber, N piles	ρt * Vtimber * Npiles	12898	kg
Total weight of all N piles	Wtotal	31461.87	kg

Table E.10: Calculation element quantities timber foundation pile Delft

F Environmental data LCA

An essential aspect of conducting a comprehensive LCA method is properly presenting the utilised data with the given source. Most of the environmental data used in this research originates from the NMDv3.5 and Ecoinvent3.6 databases. The data processing is conducted using the Ontwerp-Tool Groen Beton platform and Excel. It is important to note that the data provided in this appendix is presented per individual unit rather than for the entire foundation design and associated processes. For category 3 data, an additional surcharge of 30% must be considered. This surcharge is necessary due to the high inaccuracy related to category 3 data, resulting in a larger error margin. If there is any inconvenience or if you require a more detailed environmental impact assessment for each element or LCA stage, you are welcome to submit an additional request.

F.1. Materials

Product name / impact category	Unity	CEM I 52.5 R (G3). ENCI / HeidelbergCement	CEM III/A 52.5 N (G6). ENCI / HeidelbergCement	CEM I 42.5 N (G1). ENCI / HeidelbergCement	CEM III/B 42.5 N (G8), ENCI / HeidelbergCement	Gravel. NVLB	Tap water (Europe) production. conventional treatment	Sand. concrete zand
Category data		1	1	1	1	3	3	3
Unity		kg	kg	kg	kg	kg	kg	kg
Source		NMDv3.5	NMDv3.5	NMDv3.5	NMDv3.5	NMDv3.5	Ecoinvent 3.6	NMDv3.5
ECI	€	5.317E-02	3.104E-02	4.562E-02	1.733E-02	1.500E-03	2.963E-05	1.485E-03
AD NF	kg Sb eq	5.280E-07	5.680E-08	6.800E-08	3.710E-08	4.350E-08	2.800E-09	3.645E-07
AD F	kg Sb eq	2.110E-03	1.320E-03	1.210E-03	7.620E-04	8.420E-05	1.947E-06	8.068E-05
GWP	kg CO2 eq	8.570E-01	4.610E-01	7.820E-01	2.700E-01	1.180E-02	2.673E-04	1.132E-02
ODP	kg CFC eq	1.090E-08	1.090E-08	9.900E-09	8.100E-09	1.700E-09	0.000E+00	1.500E-09
POCP	kg C2H4	7.580E-05	8.710E-05	5.330E-05	3.560E-05	8.660E-06	9.560E-08	9.734E-06
AP	kg SO2 eq	1.100E-03	1.040E-03	6.080E-04	4.020E-04	7.250E-05	1.488E-06	7.497E-05
EP	kg PO4-eq	3.160E-04	1.350E-04	1.490E-04	6.930E-05	1.200E-05	1.637E-07	1.301E-05
HT	kg 1.4-DB eq	2.400E-02	2.240E-02	2.290E-02	1.400E-02	4.880E-03	8.312E-05	4.746E-03
FAETP	kg 1.4-DB eq	1.140E-03	4.560E-04	6.680E-04	3.030E-04	1.030E-05	1.848E-06	9.852E-05
MAETP	kg 1.4-DB eq	3.340E+00	1.850E+00	2.700E+00	1.150E+00	3.950E-01	7.571E-03	3.773E-01
TETP	kg 1.4-DB eq	9.830E-04	3.210E-04	1.540E-03	3.580E-04	2.140E-05	7.619E-07	1.955E-05

Table F.1: Environmental data of the used materials 1

The data regarding the C18 spruce timber given in Table F.2 originates from Centrum Hout and has been roughly rounded for confidential reasons. Detailed data for the year 2023 can be requested directly from Centrum Hout. However, for the calculations and comparisons, the detailed data is utilised.

Product name / impact category	Unity	Powdered fly ash (1997) (residual product)	Plasticiser, sulfonated melamine formaldehyde	Regular reinforcement steel (17.8% primair. 82.2% secundair) VWN	Prestressed steel Nedri (51.32% secundair)	Construction steel. hot rolled. profiles (4.2% primair. 95.8% secundair)	C18 timber spruce. 469 kg/m3 2023. assumed CO2 storage	C18 timber spruce. 469 kg/m3 2023. no CO2 storage
Category data		3	2	1	1	3	1	1
Unity		kg	kg	kg	kg	kg	m3	m3
Source		NMDv3.5	Ecoinvent 3.6.	NMDv3.5	NMDv3.5	NMDv3.5	Centrum Hout	Centrum Hout
ECI	€	0.000E+00	1.954E-01	1.418E-01	1.905E-01	1.045E-01	-2.000E+01	-2.000E+01
AD NF	kg Sb eq	0.000E+00	1.282E-04	9.660E-06	1.176E-05	3.206E-05	2.000E-03	2.000E-03
AD F	kg Sb eq	0.000E+00	1.574E-02	7.500E-03	1.286E-02	5.308E-03	5.000E-01	5.000E-01
GWP	kg CO2 eq	0.000E+00	1.296E+00	9.920E-01	1.785E+00	7.389E-01	-5.770E+02	9.100E+01
ODP	kg CFC eq	0.000E+00	1.305E-03	8.870E-08	1.168E-07	6.010E-08	2.000E-05	2.000E-05
POCP	kg C2H4	0.000E+00	1.305E-03	1.030E-03	1.291E-03	4.317E-04	9.000E-02	9.000E-02
AP	kg SO2 eq	0.000E+00	7.787E-03	4.620E-03	5.088E-03	3.054E-03	5.000E-01	5.000E-01
EP	kg PO4-eq	0.000E+00	6.882E-04	6.420E-04	8.746E-04	5.401E-04	1.000E-01	1.000E-01
HT	kg 1.4-DB eq	0.000E+00	9.138E-01	6.250E-01	6.590E-01	4.392E-01	5.000E+01	5.000E+01
FAETP	kg 1.4-DB eq	0.000E+00	1.687E-02	2.190E-02	2.976E-02	3.434E-02	1.000E+00	1.000E+00
MAETP	kg 1.4-DB eq	0.000E+00	5.192E+01	4.210E+01	4.640E+01	3.955E+01	3.000E+03	3.000E+03
TETP	kg 1.4-DB eq	0.000E+00	1.928E-03	6.010E-02	5.948E-02	7.043E-02	4.000E-01	4.000E-01

Table F.2: Environmental data of the used materials 2

F.2. Transport and installation

Table F.3 shows the environmental data of the transport and installation of the foundation variants. It can be seen that all the transport data is category 3 data. However, this is almost inevitable because of the many involved parameters in transport emissions. For example, the specific truck or engine, the truck's age, if the road is more on highways or small roads. No elaborated and detailed EPD or ECI was available for the electric truck transport. Therefore, the required (green) electricity use of a Volvo FMX truck is assumed. This truck has a total capacity of 540 KWH and an action radius of 300 km (fully loaded). Therefore, it is considered that the Volvo truck uses 1.8 kWh per km. In this way, a proper estimation is given for the environmental impact of the electric transport.

Product name / impact category	Unity	Diesel piling driver. both driving and lifting	Transport. freight. lorry >32 metric ton. euro6 Diesel truck	Transport freight lorry >32 metric hydrogen	Transport freight lorry >32 metric 1st gen biodiesel EURO6	Transport freight lorry >32 metric HVO100 2nd gen EURO6	Mean size truck mixer (10m3) >32 ton
Category data		3	3	3	3	3	3
Unity		hr	tkm	tkm	tkm	tkm	m3km
Source		NMDv3.5	NMDv2.1	NMDv3.5	NMDv3.5	NMDv3.5	NMDv3.5
ECI	€	1.237E+01	1.011E-02	7.717E-03	8.697E-03	7.717E-03	3.138E-02
AD NF	kg Sb eq	1.431E-04	1.550E-06	2.021E-06	2.058E-07	1.911E-07	3.630E-07
AD F	kg Sb eq	6.090E-01	6.716E-04	7.947E-04	2.886E-04	2.510E-04	1.545E-03
GWP	kg CO2 eq	9.233E+00	8.639E-02	5.413E-02	3.849E-02	4.124E-02	2.342E-01
ODP	kg CFC eq	1.599E-05	1.700E-08	1.040E-08	6.100E-09	5.600E-09	4.060E-08
POCP	kg C2H4	9.400E-02	5.528E-05	6.205E-05	5.273E-05	5.362E-05	2.385E-04
AP	kg SO2 eq	6.958E-01	2.272E-04	2.010E-04	3.758E-04	1.909E-04	1.765E-03
EP	kg PO4-eq	1.582E-01	3.668E-05	2.681E-05	1.461E-04	7.056E-05	4.013E-04
HT	kg 1.4-DB eq	3.418E+00	4.262E-02	3.686E-02	2.833E-02	3.073E-02	8.672E-02
FAETP	kg 1.4-DB eq	4.758E-01	1.160E-03	8.733E-04	1.474E-02	1.460E-02	1.207E-03
MAETP	kg 1.4-DB eq	1.654E+03	4.562E+00	3.597E+00	4.623E+00	4.852E+00	4.197E+00
TETP	kg 1.4-DB eq	5.626E-02	1.376E-04	1.734E-04	5.809E-03	5.836E-03	1.427E-04

Table F.3: Environmental data of the transport options

F.3. Energy, processes and re-use

Tables F.4 and F.5 show the used environmental data of the energy, processes and the re-use processes. Important to note is that almost all data is category 3 data. This is caused by significant differences in applications. Moreover, the processes are very manufacturing and situation dependent, which makes it challenging to specify a single value for one process or reuse potential.

Product name / impact category	Unity	Energy - Gas (Heat. district or industrial. natural gas heat Europe production	Energie Green Electricity. low voltage.	Energie Electricity. low voltage. grey {NL}	Energie - Diesel . burned in building machine	Concrete pump, incl vehicle, mean size	Compaction needle 0.33 kWh/m3 , grey electricity
Category data		3	3	3	3	3	3
Unity		m3	kWh	kWh	L	m3	m3
Source		Ecoinvent 3.6	NMDv3.3	Ecoinvent 3.6	Ecoinvent 3.6	NMDv3.5	NMDv3.5
ECI	€	1.334E-01	2.876E-02	4.766E-02	4.358E-01	8.890E-01	1.573E-02
AD NF	kg Sb eq	5.855E-07	6.383E-06	2.620E-06	5.042E-06	1.028E-05	8.644E-07
AD F	kg Sb eq	1.935E-02	3.542E-04	4.827E-03	2.146E-02	4.377E-02	1.593E-03
GWP	kg CO2 eq	2.204E+10	1.157E-01	6.414E-01	3.253E+00	6.636E+10	2.117E-01
ODP	kg CFC eq	2.777E-07	7.400E-09	3.170E-08	5.633E-07	1.149E-06	1.050E-08
POCP	kg C2H4	3.228E-04	1.434E-04	9.554E-05	3.312E-03	6.756E-03	3.153E-05
AP	kg SO2 eq	1.446E-03	1.774E-03	1.201E-03	2.452E-02	5.001E-02	8.643E-07
EP	kg PO4-eq	1.909E-04	3.503E-04	2.471E-04	5.573E-03	1.137E-02	8.156E-05
HT	kg 1.4-DB eq	1.262E-01	1.208E-01	7.217E-02	1.204E+00	2.457E+10	2.382E-02
FAETP	kg 1.4-DB eq	1.176E-03	3.055E-03	1.990E-03	1.676E-02	3.420E-02	6.567E-04
MAETP	kg 1.4-DB eq	5.015E+01	8.612E+00	8.398E+01	5.829E+01	1.189E+02	2.771E+00
TETP	kg 1.4-DB eq	4.053E-04	9.243E-03	3.269E-03	1.983E-03	4.044E-03	1.079E-03

Table F.4: Environmental data of energy resources and processes

Product name / impact category	Unity	Demolition process, digging + breaking concrete	Waste treatment concrete, semi-intensive	Waste treatment Ferrous metals (reinforcement)	Material equivalent-Granulate in new concrete mixture	Material equivalent-Granulate road foundation (sand)	Material equivalent-Granulate road foundation (Gravel)	Material equivalent-WVN reinforcement steel
Category data		3	3	3	2	2	2	3
Unity		kWh	kg	kg	kg	kg	kg	kg
Source		NMDv3.5	Ecoinvent 3.6	Ecoinvent 3.6	NMDv3.1	NMDv3.1	NMDv3.1	NMDv3.5
ECI	€	2.876E-02	1.164E-03	7.296E-04	2.278E-04	1.452E-04	5.272E-04	8.075E-03
AD NF	kg Sb eq	6.383E-06	1.350E-08	4.820E-08	5.100E-09	2.100E-09	2.091E-07	1.133E-07
AD F	kg Sb eq	3.542E-04	5.730E-05	7.039E-05	1.410E-05	9.150E-06	2.768E-05	1.217E-04
GWP	kg CO2 eq	1.157E-01	8.688E-03	5.168E-03	1.890E-03	1.250E-03	4.098E-03	1.085E-01
ODP	kg CFC eq	7.400E-09	1.500E-09	1.700E-09	2.000E-10	2.000E-10	4.000E-10	2.600E-09
POCP	kg C2H4	1.434E-04	8.845E-06	5.505E-06	1.010E-06	8.560E-07	3.030E-06	2.572E-05
AP	kg SO2 eq	1.774E-03	6.547E-05	3.780E-05	8.290E-06	6.900E-06	2.328E-05	6.995E-05
EP	kg PO4-eq	3.503E-04	1.488E-05	7.291E-06	1.890E-06	1.570E-06	3.794E-06	6.995E-05
HT	kg 1.4-DB eq	1.208E-01	3.216E-03	7.291E-06	8.250E-04	3.920E-04	1.898E-03	9.721E-03
FAETP	kg 1.4-DB eq	3.055E-03	4.477E-05	5.544E-05	1.080E-05	5.890E-06	2.943E-05	9.145E-03
MAETP	kg 1.4-DB eq	8.612E+00	1.557E-01	1.982E-01	3.600E-02	2.110E-02	1.222E-01	9.179E+00
TETP	kg 1.4-DB eq	9.243E-03	5.294E-06	5.867E-06	1.100E-05	4.150E-06	9.872E-06	1.768E-05

Table F.5: Environmental data of processes and materials equivalents