Technical feasibility of a quick bridge replacement strategy with minimal traffic hindrance

On the retainment of existing foundations and the application of Advanced Cementitious Materials in an Accelerated Bridge Construction method

by

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- LITERATURE STUDY -

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Preface

Dear reader,

Before you lies the final version of my Literature Study, which will form the basis of my graduation thesis. The aim of this report is to gain insight into the possibilities for a quick bridge replacement strategy that minimises (traffic) hindrance and can be applied in general for the large replacement task that lies in the near future.

First, in part I, a general problem description will be given (1) after which the twofold problem (a large bridge replacement task (2) and the economic impact of traffic hindrance (3)) will be further elaborated into more detail. Also, the current state-of-the-art (4) regarding quick bridge replacement methods will be covered.
The second part will start with a preliminary conclusion (5) on the information required to adapt a strategy needed to overcome the presented problem. This information will then be covered by three separate chapters, regarding Accelerated Bridge Construction (6), Advanced Cementitious Materials (7) and the reuse of existing foundations (8).
This report will be concluded in part III, where first a general conclusion (9) will be drawn on the potential application of the quick replacement methods elaborated in part II. Finally, from this a general concept for a quick bridge replacement strategy(10) will be presented that will be tested in a specific Case Study.

I would like to thank my graduation committee for the advice and remarks that I have received so far on my graduation results and process. Special thanks goes out to prof.dr.ir. D.A. Hordijk for his enthusiasm and extensive feedback in this stage of my thesis and my daily supervisor, ir. A. Reitsema, for his frequent remarks and comments. Furthermore, I would like to thank prof.ir. A.F. van Tol, ing. H.J. Everts and ing. A. Zeilmaker for their input on the reuse of existing foundations, an area of expertise that is not common knowledge to me. And finally, I would like to express my gratitude towards Heijmans, where I have felt at home since starting there in January 2017, thanks to the enthusiasm and receptivity of my direct colleagues. I am looking forward to continuing my graduation thesis within Heijmans and the TU Delft.

M.A.G. (Merijn) Stap
Delft, April 2017

Summary

Concrete bridges and viaducts are a crucial part of the Dutch national and regional infrastructural network. With increasing mobility and prosperity after the Second World War, the total length of highways has increased substantially, with the largest growth taking place between 1960 and 1980. Simultaneously with the expansion of our road network, also a large number of bridges and viaducts were constructed and almost 50% of all currently existing bridges are built in this time period. The applied design codes prescribe a design service life of 50 years and although this means that every bridge built before 1957 has reached the end of its intended life span, this does not mean that all these bridges will fail after this period. Due to maintenance throughout the years, several strengthening mechanisms and included safety factors, the bearing capacity of these bridges is often still sufficient after 50 years. It is, however, an inevitable fact that a large bridge replacement task lies ahead of us in the near future.
In addition, as it is one of the busiest highway networks of the world, the Dutch infrastructure is very susceptible to economic damage following from undesired traffic hindrance. The large replacement issue of concrete bridges must therefore be handled in such a way that the impact on traffic hindrance is as low as possible, especially since the economic impact of congestions has increased over the past few years and will increase even further in the near future.
This twofold problem calls for a smart solution regarding the reduction of on-site construction time, through the formulation of a quick bridge replacement strategy.

The on-site construction time can be reduced by limiting the amount of actions required, through eliminating groundwork, maintaining as many elements as possible and quick element erection. In general, three time-reducing measurements can be distinguished, of which the extent and depth of the analysis depends on several boundary conditions of the specific project:

A. Reuse of existing foundations:

  By reusing the existing foundations, less demolition work and less installation activities have to take place. The question is, however, whether the old pile foundation is capable of bearing the new loading situation. The lower bound bearing capacity is based on the current loading situation and an increase of load can be taken up by additional piles. This is to be expected, since the traffic load has increased and a larger part of the dead load will be transferred to the end supports if the intermediate supports are removed (see measurement C.).

  The amount of additional piles needed can be reduced or even diminished by recalculating the pile bearing capacity, improving the soil-structure interaction and including time-effects. If possible, the exact bearing capacity of the pile foundation can be measured, provided that this does not result in additional traffic hindrance.

B. Accelerated Bridge Construction (ABC) approach:

  The already common lower bound ABC approach consists of a quick construction process by making use of prefabricated elements and night-time closures. An increasing reduction of highway closure time can be obtained by placing modular systems or even entire superstructures at once, through lateral sliding or transportation by SPMT. This can ultimately result in a total closure time of only several hours to several days.

  The best approach is case-specific and depends on the boundary conditions such as road configuration, investment cost, MEAT price reductions and available construction space at a nearby location.

C. Remove intermediate supports:

  The removal of intermediate supports reduces the on-site construction time, since less elements have to be erected. Furthermore, the presence of these supports is undesirable due to the freedom of underlying road configuration and safety issues like vehicle collisions.

  By applying this measurement, however, the design will become substantially more complicated. Especially the increase of dead weight on the end supports might result in additional foundation piles needed, thus reversing the profit gained by reusing the existing foundation (see measurement A.). Also, in order to avoid extensive groundwork activities the traffic profile should remain

intact, which calls for the application of slender and lightweight UHPFRC bridge girders.
The choice whether or not to remove the intermediate supports must be based on an overall consideration of time reduction, design costs, construction costs and technical feasibility.

The overall goal is to construct a quick bridge replacement strategy that minimises the traffic hindrance during bridge construction works. It depends on the case-specific characteristics and boundary condition to which extent on-site time reduction is required and whether this is stimulated by a price reduction in a MEAT procedure.
The next step will be to determine the technical feasibility of this strategy by looking at a specific case study, that will be redesigned based on a minimal on-site reduction time. Furthermore, this case study will be used to investigate the sensitivity of the measurements to several evaluation criteria and boundary conditions. The results of this sensitivity analysis will be incorporated in a more advanced quick bridge replacement strategy, in order to facilitate a probable choice in an early design stage on the package of time-reducing measurements to be applied.

NOMENCLATURE

ABBREVIATIONS

ABC	Accelerated Bridge Construction
ACM	Advanced Cementitious Material
ADT	Average Daily Traffic
AFGC	Association Française de Génie Civil
CPT	Cone Penetration Test
CPTu	Piezo Cone Penetration Test
DEF	Delayes Ettringite Formation
EMVI	Economisch Meest Voordelige Inschrijving
FHWA	Federal Highway Association
FRC	Fibre Reinforced Concrete
GC	Geotechnical Category
GRS-IBS	Geosynthetic Reinforced Soil-Integrated Bridge System
HSC	High Strength Concrete
LWAC	Light Weight Aggregate Concrete
MEAT	Most Economic Advantageous Tender
NSC	Normal Strength Concrete
PBES	Prefabricated Bridge Elements and System
PVA	Polyvinyl Alcohol
RFG	Richtlijnen voor funderingen van gebouwen
RH	Relative Humidity
RPC	Reactive Powder Concrete
RWS	Rijkswaterstaat
SBIC	Slide-In Bridge Construction
SHCC	Strain Hardening Cementitious Composite
SLS	Serviceability Limit State
SPMT	Self-Propelled Modular Transporter
SPT	Standard Penetration Test
TGB	Technische Grondslagen voor Bouwconstructies
UHPC	Ultra-High Performance Concrete
UHPFRC	Ultra-High Performance Fibre-Reinforced Concrete
ULS	Ultimate Limit State
wcr	water-cement ratio

LIST OF SYMBOLS

Latin symbols:

A	Area *or* set-up
A_{punt}	Pile tip area
c	Cohesion
c_{min}	Minimum concrete cover
D_{max}	Maximum aggregate size
D_p	Pile diameter
e	Element thickness
$(EA)_{cs}$	Combined axial stiffness reinforced concrete
$E_s A_s$	Axial stiffness reinforcement steel
E_{cm}	Modulus of elasticity of concrete

$E_{c,eff}$	Effective modulus of elasticity of concrete
f_1	Compaction factor in group effect
f_2	Effective stress factor in group effect
f_{cd}	Concrete compressive strength
f_{ck}	Characteristic compressive cylindrical strength
$f_{ck,cube}$	Characteristic compressive cubic strength
f_{ct}	Concrete tensile strength
f_{ctfm}	Maximum mean post-cracking stress in tension
$f_{ctm,el}$	mean elastic limit stress in tension
f_s	Local friction
l	Length
l_c	Characteristic length
l_f	Individual fibre length
K	Fibre orientation factor
K_0	Neutral horizontal soil stress
K_{global}	Global fibre orientation factor
K_{local}	Local fibre orientation factor
Ø	Diameter
$p_{R;max;shaft}$	Maximum pile shaft resistance
$p_{R;max;tip}$	Maximum pile tip resistance
q_c	Cone resistance
s	Reduction factor for the pile tip shape *or* settlement
V	Volume
w	Settlement *or* crack width
w_{max}	Maximum crack width
w_{peak}	Crack opening corresponding to the local peak

Greek symbols:

α_{cc}	Long-term effects coefficient for compressive strength
α_h	Degree of hydration
α_p	Pile tip resistance factor
α_s	Pile shaft resistance factor
β	Reduction factor for the pile tip shape *or* relative rotation
δ	Friction angle
γ_c	Partial factor for concrete
γ_{cf}	Partial factor for fibre-reinforced concrete under tension
γ_f	Load factor
γ_M	Material factor
ϵ_c	Applied concrete strain
ϵ_{c3}	Maximum linear compressive strain
ϵ_{ct1}	Maximum linear tensile strain
ϵ_{cu3}	Ultimate compressive strain
$\epsilon_{lim,hard}$	Maximum tensile hardening strain
ξ	Correlation factor
λ	Slenderness ratio
ϕ	Internal angle of friction
$\varphi_{cc}(t)$	Concrete creep factor
σ_f	Applied stress

Contents

PART I:

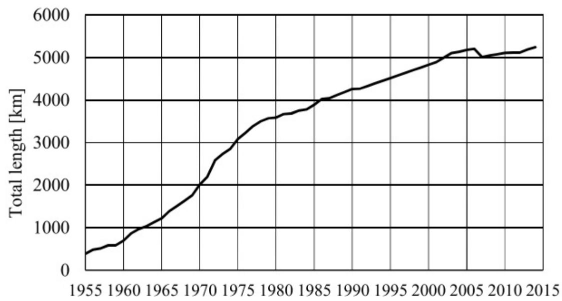
PROBLEM DESCRIPTION

1

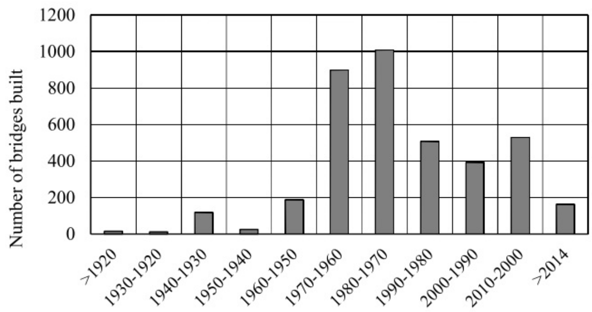
INTRODUCTION

1.1. PROBLEM DESCRIPTION

Concrete bridges and viaducts are a crucial part of the Dutch national and regional infrastructure network. With increasing mobility and prosperity after the Second World War the total length of highways has increased substantially, in which the largest growth has taken place between 1960 and 1980. Simultaneously with the expansion of our road network, also a large number of bridges and viaducts were constructed and almost 50% of the existing bridges are built in this time period, see Figure 1.1 *[Reitsema et al., 2015]*. The used design codes prescribe an intended service life of 50 years but that doesn't mean that all bridges will fail after this period. Due to maintenance throughout the years and several strengthening mechanisms, such as ongoing hydration, the bearing capacity of these bridges is often still sufficient after 50 years. It is, however, a fact that a large amount of bridges will inevitably have to be replaced in the near future.



(a) Historical overview of length of Dutch main road network.



(b) Historical overview of constructed Dutch bridges in last decades.

FIGURE 1.1: Historical overview of the Dutch infrastructure *[Reitsema et al., 2015]*

Being one of the busiest highway networks of the world, the Dutch infrastructure is very susceptible to economic damage following from undesired traffic hindrance *[Reitsema et al., 2015]*. The large replacement issue of concrete bridges must therefore be handled in such a way that the impact on hindrance such as the traffic flow is as low as possible. This calls for smart solutions regarding on-site building time and temporary rerouting of traffic.

Besides the fact that the economic impact of traffic jams has increased over the past few years, additional advantages of a shorter construction period can also be found in weather-related time delays, work site safety, quality and durability of the structure. To decrease the total project delivery time an Accelerated Bridge Construction (ABC) method can be used in combination with a slender and lightweight bridge *[Reitsema et al., 2015]*.

Although this bridge replacement task is a current issue in multiple countries in the world, this thesis will focus on the application in the Dutch infrastructure. This is due to the fact that the Dutch highway network is very dense and heavily used. Replacement of a bridge in the Netherlands will therefore have a larger impact on the hindrance from construction works than in other countries

1.1.1. Accelerated Bridge Construction (ABC) method

The ABC method consists of different technologies that have a common goal *[Federal Highway Administration, nda]* to produce safer, more durable bridges with longer service lives than conventional bridges and an on-site building time of just 48 to 72 hours. Its roots lie in the USA, where it is commonly applied using the following three technologies:

- Geosynthetic Reinforce Soil-Integrated Bridge System (GRS-IBS)
  GRS-IBS is a construction method that combines closely spaced geosynthetic reinforcement and granular soils into a new composite material. This material can be used to construct abutments and approach embankments that are less likely to settle (avoiding a 'bump' in the road) and can reach a much higher slope angle, thereby reducing the span length of the bridge.
- Prefabricated Bridge Elements and Systems (PBES)
  Prefabricated structural components can be built offsite under controlled conditions and therefore a high quality and durability can be guaranteed. Another advantage of prefabrication is that it allows for quick erection of the elements on-site, thus reducing traffic hindrance to a minimum.
- Slide-In Bridge Construction (SIBC)
  A new bridge is built on temporary supports parallel to an existing bridge and slid into place after the existing bridge structure is demolished. This replacement can take place within 48 to 72 hours as only minor adjustments have to be carried out after the bridge has lowered onto the supports.

When coupled with appropriate quality assurance, safety, project management and construction engineering practices, the ABC method can have a large beneficial influence on the total project delivery time and costs.

1.1.2. Bridge deck slenderness by application of ACM's

In the past, mainly statically indeterminate cast in-situ reinforced concrete plate bridges were built as this bridge type was, prior to the increasing use of prefabricated concrete, economically most attractive. These plate bridges were mostly built with three or four spans for a total length of 20 to 60 meters *[Reitsema et al., 2015]*.

An advantage of a plate bridge with multiple intermediate supports over a single span bridge is the possibility to achieve higher slenderness, due to the more favourable distribution of bending moments. A disadvantage of an in-situ plate bridge is that more on-site construction time is needed for the mounting of formwork, placing of reinforcement and pouring and hardening of the concrete. Since, however, these bridges were initially built in a new highway, traffic hindrance was irrelevant and thus also the construction time was not an issue.

However, when considering the future replacement task, rebuilding cast in-situ plate bridges is, due to its impact on traffic, undesirable and should be avoided. This makes the use of prefabricated girders more favourable.

Furthermore, rebuilding the intermediate supports is undesirable due to traffic hindrance and safety issues such as vehicle collisions. In addition, when building without the intermediate supports, more traffic lanes can be created in the available space under the bridge, the so-called traffic profile as shown in Figure 1.2 *[Reitsema et al., 2015]*.

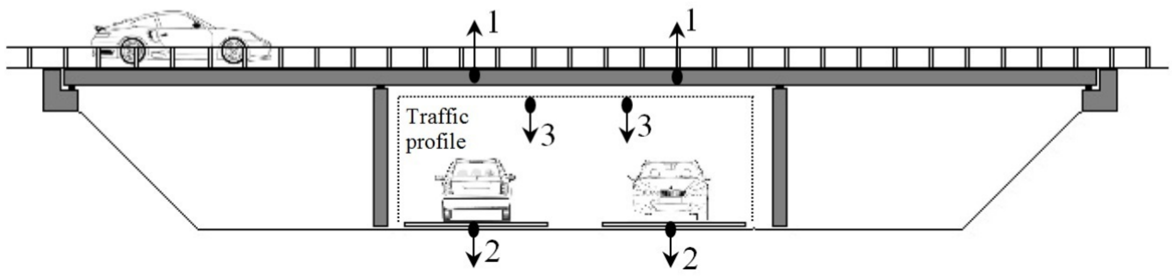


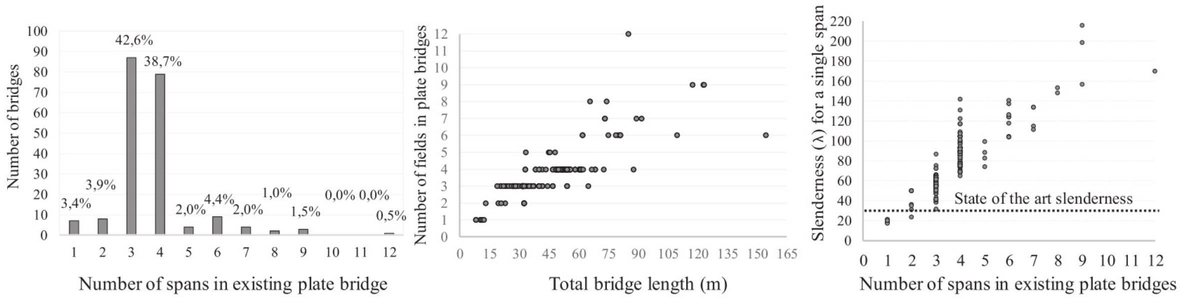
FIGURE 1.2: Replacement of an existing three span concrete plate bridge *[Reitsema et al., 2015]*

Every bridge type has a more or less linear relationship between the deck height and span length. Therefore removing the intermediate support and creating a single span bridge will result in an increase of deck height, when using the same design and concrete type as in the initial construction. This can have the following impact on the bridge profile (Figure 1.2):

1. Increasing the height of an existing highway in order to align it with the new viaduct under which the traffic profile stays the same;
2. Lowering the underlying road so that the traffic profile under the viaduct stays the same;
3. Reducing the existing traffic profile under the viaduct.

The first two options will require additional groundwork to accommodate for the increased deck depth, leading to a longer construction time and higher costs. The third option will lead to a reduced height of the traffic profile and thus a restriction on the traffic that can pass underneath. The conclusion is that all three options are highly undesirable regarding hindrance.

In order to replace an existing cast in-situ plate bridge without using the existing intermediate supports and in order to keep the current traffic profile, a high bridge deck slenderness is needed. However, the current state of the art slenderness with prefabricated prestressed concrete box girders is $\lambda = 30$ and this is not sufficient for the replacement task of three- and four-span bridges (Figure 1.3).



(a) Existing plate bridges (n=204) and the number of spans.

(b) Total length of existing plate bridges with the number of bridge fields.

(c) Slenderness required to replace existing plate bridges with a single span bridge.

FIGURE 1.3: Identification of plate bridges within the Dutch main road network *[Reitsema et al., 2015]*

A possible solution lies in the application of prefabricated beams made from so-called Advanced Cementitious Materials (ACM's) in order to reach the required slenderness. ACM is a general name for every concrete type that is not defined as Normal Strength Concrete (NSC) in the Eurocode (maximum concrete strength class is C90/105) or contains fibres *[Nederlands Normalisatie-instituut, 2005]*. Examples of ACM's are Ultra-High Performance Concrete (UHPC), Strain Hardening Cementitious Composite (SHCC) or Geopolymer Concrete (GC). These ACM's have increased mechanical and durability properties in comparison with Normal Strength Concrete (NSC) and can therefore achieve a higher slenderness and a lower self-weight. Comparison of their characteristics shows that UHPC has the best mechanical and durability properties, while NSC has the best sustainability characteristics *[van Oosten, 2015]*. Reducing the weight of prefabricated elements has a number of advantages when looking at transportation, Slide-In Bridge Construction and required foundation.

1.1.3. Keeping existing foundations

In order to reduce the traffic impact during the replacement of an existing three or four span plate bridge even further, it would be beneficial to keep the existing foundation in place. Then, only the existing bridge deck and intermediate supports need to be demolished and removed before placing the new construction. This does not only result in a considerable decrease of on-site construction time, it can also reduce the foundation costs.

However, removing the intermediate supports will cause the static scheme to change from a statically indeterminate beam to a statically determinate beam, thus causing forces in the foundation piles for which the existing foundation was not designed. Furthermore, since the entire weight of the bridge deck will now have to be carried by less supports then in the existing situation, the call for a lightweight structure (see section 1.1.2) is also required.

1.2. RESEARCH OBJECTIVE

Considering the boundary conditions for a rapid bridge replacement (or ABC) with minimal traffic hindrance, as stated in the previous sections, a solution must be sought in an innovative slender and lightweight prefabricated bridge concept. Second, more insight must be gained in the state and design of existing foundations and whether the construction is sufficiently dimensioned to carry the new bridge design.

1.3. RESEARCH QUESTION

From the problem description as described in the previous section, the following research question has been formulated:

How can slender and lightweight prefabricated UHPC bridge girders be used for a minimal (traffic) hindrance strategy regarding the infrastructural replacement task within our highway network?

1.4. SCOPE LITERATURE STUDY

The problem description as described in the previous sections has been schematised below. The fact that a large bridge replacement task awaits in the near future while the economic impact of (traffic) hindrance has increased, means that a solution must be sought in decreasing the on-site building time where possible. For this, several methods and technologies have been proposed that can all contribute to an Accelerated Bridge Construction method.

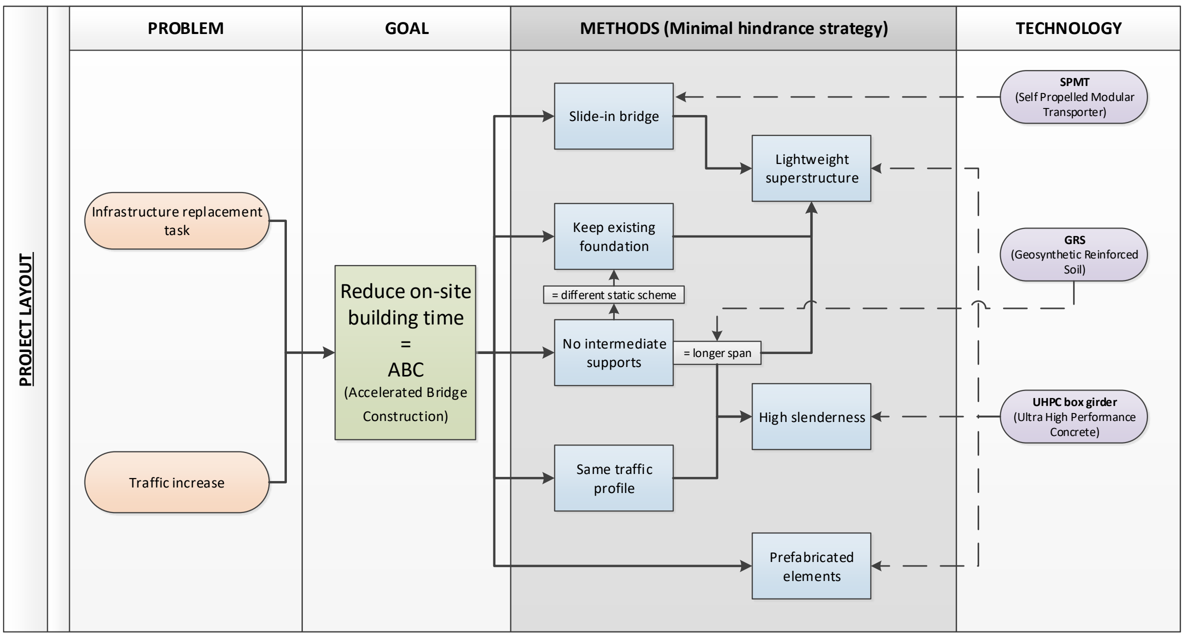


FIGURE 1.4: Schematisation of the problem description

In order to answer the research question, several sub-questions are developed that can be related to the schematisation above:

1. Problem statement:
    - What is the current situation of our highway network?
        - What current bridge types can be distinguished and when where they built?
        - What are the terms and conditions of the applied design codes?
        - What is the required slenderness when removing the intermediate supports?
    - Which aspects are of importance when replacing a bridge?
        - What kind of hindrance may occur during replacement?
        - What are the demands in term of hindrance that have to be met during construction?
        - What is the economic impact of traffic hindrance?

- How is the avoidance of hindrance incorporated in the building contract?

2. Reference projects:
   - What is the state-of-the-art regarding bridge replacement technologies?
     - What is the current development in Accelerated Bridge Construction?
     - In which circumstances has UHPC been used in bridge construction?
     - Have existing foundations been reused before?
3. Research goal:
   - In which fields of application regarding bridge replacement technologies is further improvement required?
4. Methods for a quick bridge replacement strategy:
   - What are the terms and conditions for a successful application of the Accelerated Bridge Construction method?
     - What is the application field of GRS-IBS and what are the advantages?
     - What are the advantages of prefabricated girders?
     - What is the influence of using a SIBC method on the on-site construction time and what are the conditions for maximum weight and available space?
   - To what extent can the use of ACM's increase the slenderness of prefabricated girders?
     - What are Advanced Cementitious Materials and what application do they have regarding increased slenderness?
     - What are the characteristics of UHPC and what design codes apply?
     - What is the required slenderness for a new bridge design and how can it be achieved?
   - Can the existing foundations be reused in a new bridge design?
     - What is the capacity of existing foundations and which design codes have been used?
     - What is the influence of the static scheme on the load distribution in the foundations and can the foundations be adapted to a different load configuration?
     - What mechanisms can be used in order to increase the bearing capacity of the existing foundation?
     - What is the influence of time and loading during the previous structure lifetime on the existing foundations?
5. Impact of improvements:
   - What are the advantages and disadvantages of above mentioned technologies on a rapid bridge replacement strategy with minimal (traffic) hindrance?
   - What technologies are expected to have the best technical feasibility?
   - What technologies are expected to have the best impact when looking at reduced time and costs?

These sub-questions will form the basis of the Literature Study. From this, a general concept will be developed that aims to answer the research question and this concept will be tested in one or multiple representative case studies.

1.5. GUIDELINE LITERATURE STUDY

In order to obtain a general concept that can be used for the remainder of this graduation thesis, the information in this Literature Study is presented according to the scheme given in Figure 1.5.

The problem introduction has been given in this chapter, chapter 1. From this, it follows that there are two separate problems that can be distinguished, namely the imminent infrastructural replacement task that lies in the near future (chapter 2) and the increasing economical impact of traffic hindrance (chapter 3). It is expected that a solution for these problems lies in the application of ABC, UHPC and the reuse of existing foundations. Therefore, in chapter 4 reference projects are given in which the current state-of-the art regarding these methods is presented.
From the problem description and reference projects it follows which improvements are required in chapter 5. These methods for a quick bridge replacement will be further elaborated separately in chapters 6, 7 and 8, respectively.

The potential improvements that follow from these chapters are judged in chapter 9, after which a general concept for the reduction of on-site construction time for a bridge replacement task will be given in chapter 10. This general concept will be the starting point for the remainder of this graduation study.

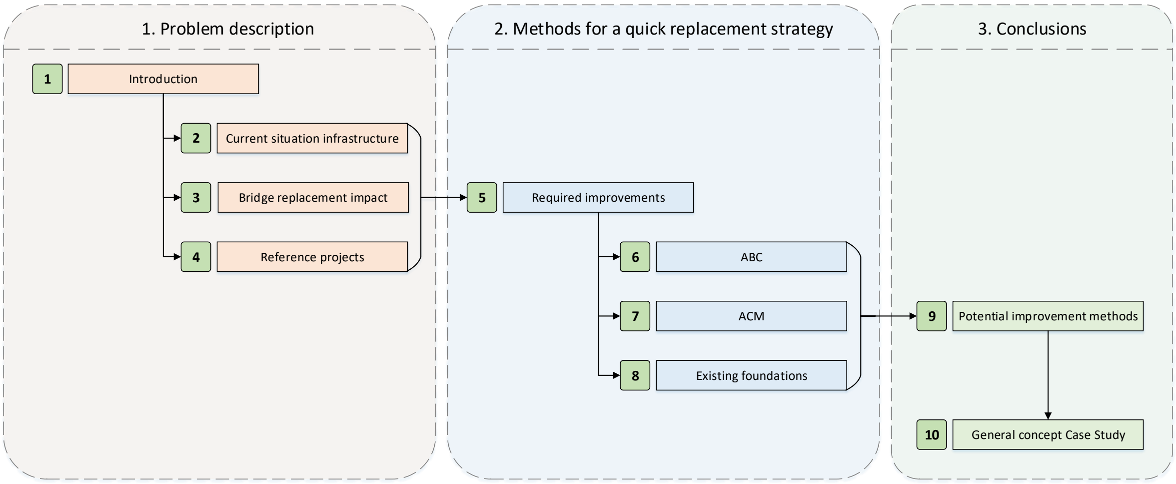


FIGURE 1.5: Schematisation of the Literature Study

Since the information given in chapters 4 and 6 to 8 is rather extensive and therefore tends to become unclear, each mentioned chapter will start with a specific reading guideline and the summary of the main outcomes.

2

CURRENT SITUATION INFRASTRUCTURE

As stated in section 1.1, it is a fact that a large amount of bridges in the Dutch highway network will have to be replaced in the near future. This chapter will elaborate on the current situation of the Dutch infrastructural network in order to gain insight into the extent of this replacement task.
Therefore, in section 2.1 a general overview will be given on the amount of bridges and their building year and section 2.2 will give more information on the several bridge types that can be distinguished. Also a small elaboration is given in section 2.3 on the required slenderness for the bridge deck if the intermediate supports are removed (see section 1.1).

It turns out that almost 50% of all current bridges in the Dutch highway network are built between 1960 and 1980, with a design service life of 50 years. This technically means that a large infrastructural replacement task is to be expected in the near future. Furthermore, the most common bridge type is the in-situ concrete plate bridge with 3 or 4 spans, accounting for about 20% of all current bridges.
In order to replace these bridges by a single span bridge by using UHPC bridge girders, a slenderness ratio of $\lambda = 30 - 90$ and $\lambda = 65 - 145$ is required for 3-span or 4-span bridges, respectively.

2.1. BRIDGES IN THE DUTCH INFRASTRUCTURAL NETWORK

The Netherlands is a small country with a high population density. Together with the large transportation hubs of the Port of Rotterdam and Schiphol Airport, there is a high demand for good accessibility. It is thus not surprising that the Netherlands has the highest density of highways in the European Union, with 57.5 km of highway length per 1000 km^2 *[Wegenwiki, 2017]*. According to *[Centraal Bureau voor de Statistiek, 2017]*, however, the total highway length in 2016 is 5340 km, which would mean a highway density of 128.5 km per 1000 km^2. This discrepancy between the two references is probably the result of different definitions for highway kilometres, but it still holds that the Netherlands has the highest highway network density of the EU.

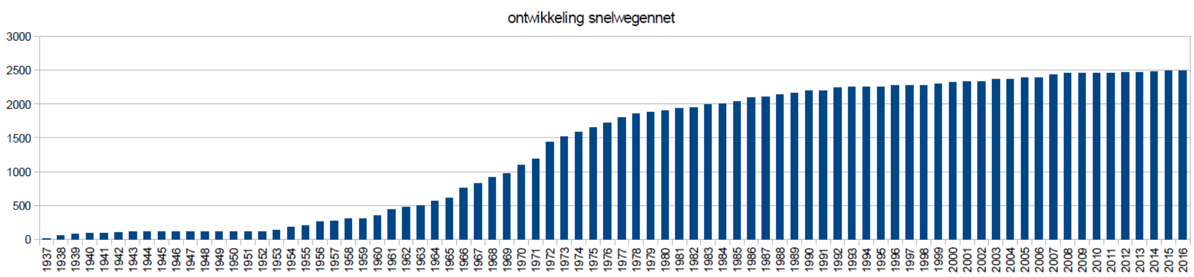


FIGURE 2.1: Historical overview of development of the Dutch highway network length *[Wegenwiki, 2017]*

The first highway, the A12 between Voorburg and Zoetermeer, was built in 1937. The largest growth occurred in the period after the Second World War, when mobility and prosperity were increasing again. This can be seen clearly in Figure 2.1, which is obtained from *[Wegenwiki, 2017]* and thus uses the lower highway network length as a definition. In the rest of this report, however, the total highway length as defined by *[Centraal Bureau voor de Statistiek, 2017]* will be defined, since Rijkswaterstaat (RWS) uses this definition as well. Figure 2.2 is obtained from RWS and presents the highway length since 1955 according to the *[Centraal Bureau voor de Statistiek, 2017]* highway length definition. Despite the differences in definition, the same trend of development in highway length can clearly be seen. In Table ..., the amount of highway kilometres according to *[Centraal Bureau voor de Statistiek, 2017]* between 2001 and 2016 is given that supports Figure 2.1.

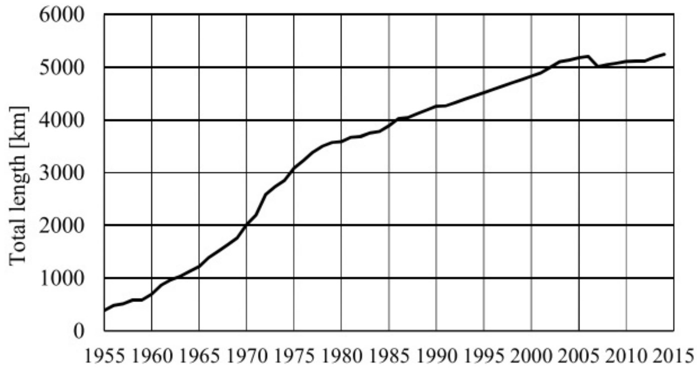


FIGURE 2.2: Historical overview of development of the Dutch highway network length *[Centraal Bureau voor de Statistiek, 2017]*

Total highway length		
Period	Length [km]	Increase km since 2001
2001	4892	0
2002	4997	105
2003	5104	212
2004	5136	244
2005	5178	286
2006	5204	312
2007	5012	120
2008	5050	158
2009	5076	184
2010	5109	217
2011	5121	229
2012	5120	228
2013	5191	299
2014	5242	350
2015	5279	387
2016	5340	448

TABLE 2.1: Overview of development of the Dutch highway network length since 2001 *[Centraal Bureau voor de Statistiek, 2017]*

Together with the development of the Dutch highway network, a growth of bridges and viaducts can be observed to accommodate this high density network. In 2007, RWS has conducted a large-scale investigation on the state of all their existing bridges, which has led to Figure 2.3. From this historical overview it follows that most current bridges (around 50%) were built between 1960 and 1980 *[Reitsema et al., 2015]*. The total amount of bridges currently under management of RWS lies just below 4000 bridges and viaducts.

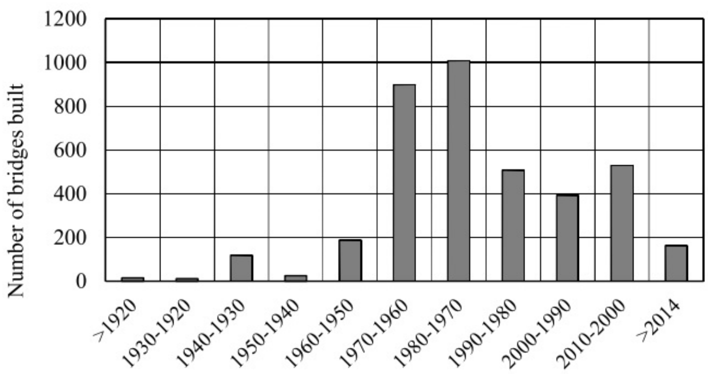


FIGURE 2.3: Historical overview of amount of bridges built in the Dutch highway network *[Reitsema et al., 2015]*

Since most bridges were constructed for a design service life of 50 years, it follows that for the majority of the current bridges this design service life will expire in the near future. Although the design service life is not a criterion for the condition of a bridge, as a certain safety is taken into account and maintenance may have been executed, it is an indication of the extent of the bridge replacement task to be expected in the near future.

2.2. DISTRIBUTION OF BRIDGE TYPES

When looking at the amount of bridges constructed over the last century, a distinction can be made in the type of bridge, as illustrated in Figure 2.4. The dataset behind this figure has been analysed, which has led to the numbers given in Figure 2.5. Although the exact numbers differ, probably due to various definitions, these two figures do indicate the distribution of present bridges in the Dutch highway network.

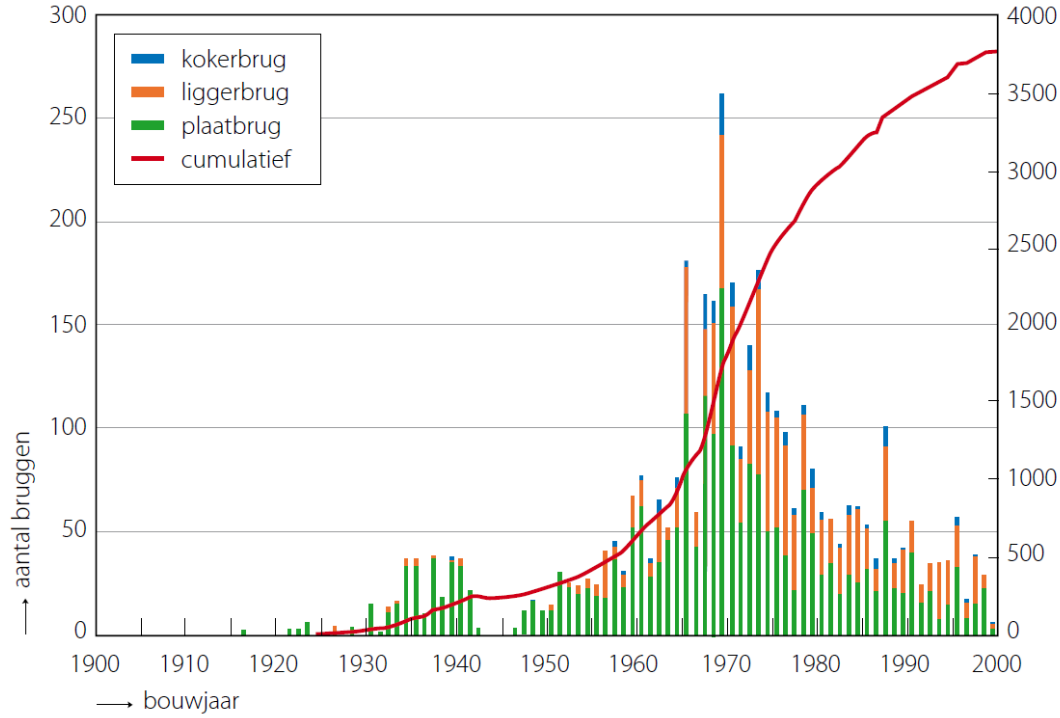


FIGURE 2.4: Historical overview of amount of bridges built in the Dutch highway network according to their type *[Gijsbers et al., 2012]*

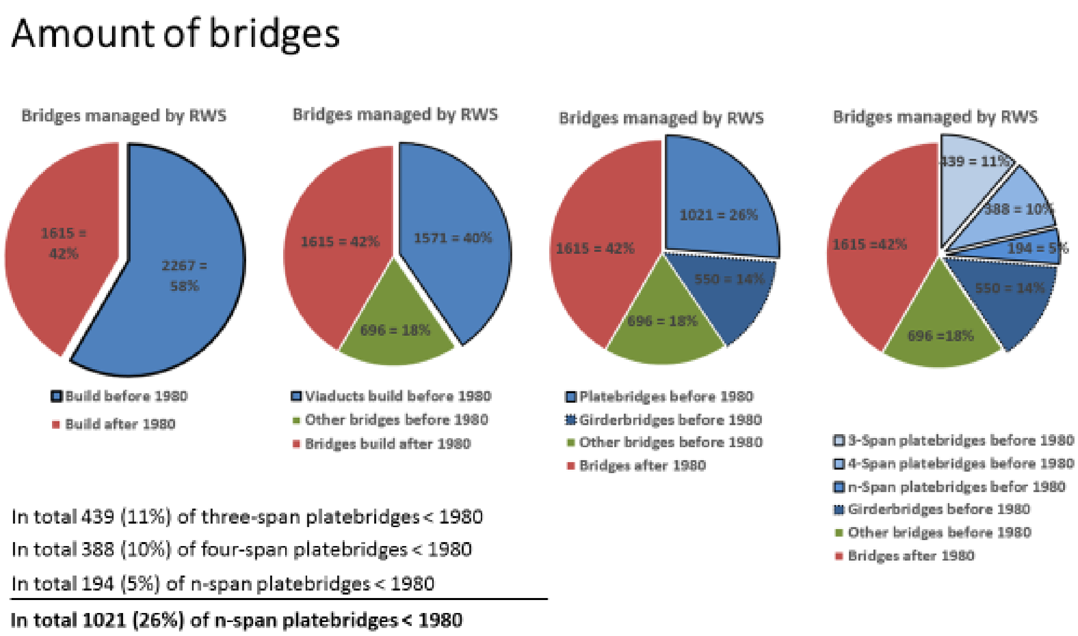


FIGURE 2.5: Distribution of bridges over building year and bridge type *[Reitsema, 2017]*

Of all viewed bridges, a clear majority is built before 1980 and is therefore interesting with respect to a future replacement task. Around 40% of all current bridges are typical viaducts situated in or over highways and of those viaducts around 50% consists of 3- or 4-span in-situ plate bridges. In other words, about one-fifth (or 20%) of all bridges currently present in the Dutch highway network is a 3- or 4-span plate viaduct and therefore this bridge type is the most relevant to look at in terms of bridge replacement strategies.

A gradual shift can however be observed in amount of plate bridges built in recent years towards the application of prefabricated girder bridges. Especially before 1980, most bridges were constructed within new highway sections and therefore they were not limited by the need for short construction times.

When looking at the design codes of concrete bridges over the last century, as listed in Table 2.2 and Figure 2.6, the largest changes can be observed with the introduction of VB 1974, in which the recommended shear capacity raised substantially (Table 2.3) *[Gijsbers et al., 2012]*.

Concrete regulations abbreviations		**Year**
Reinforced Concrete Regulations	GBV	1912 - 1974
Guideline Prestressed Concrete	RVB	1962 - present
Regulations Concrete	VB	1974 - 1990
Regulations Concrete Structural	VBC	1990 - 2012
Eurocodes	EC	2012 - present
Bridge load regulations abbreviations		
Regulations for Designing Steel Bridges	VOSB	1933 - 1995
Regulation Concrete - Bridges	VBB	1995 - 2012
Eurocodes	EC	2012 - present

TABLE 2.2: Abbreviations used for the concrete design and bridge load regulations *[Gijsbers et al., 2012]*

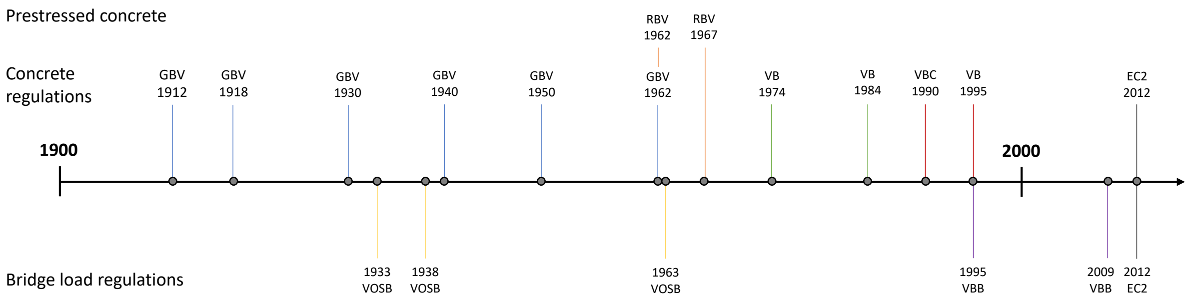


FIGURE 2.6: Introduction year of the concrete design and bridge loading regulations *[Dieteren, 2012]*

Aspect	**Regulation**			
	GBV	**VB**	**VBC**	**EC2**
Check	SLS	ULS	ULS	ULS
Safety	Implicit	Load	Load and strength	
Design value	Main tension stress $\rho = 1.5 \cdot V / (b \cdot h)$	Nominal shear stress $\tau_d = V_d / (b \cdot h)$		
Limit without shear reinforcement	K160: $\rho \leq 0.6$ Mpa K225: $\rho \leq 0.7$ Mpa K300: $\rho \leq 0.8$ Mpa	$\tau_d \leq 0.5 \cdot f_b$	Lower bound: $0.4 \cdot f_b$ Optimalisation: $0.4 \cdot k_h \cdot k_\lambda \cdot \omega_0^{0.33} \cdot f_b$	Lower bound: V_{min} Optimalisation: $C_{Rd,c} \cdot k \cdot (100 \cdot \rho_l \cdot f_{ck})^{0.33}$
Favourable effect point loads at support	None	None	At end supports	At end and intermediate supports

TABLE 2.3: Overview of shear capacity calculation differences between the concrete design regulations *[Gijsbers et al., 2012]*

2.3. REQUIRED SLENDERNESS

In section 1.1 it was mentioned that when replacing a 3- or 4-span plate bridge, it is beneficial to remove the intermediate supports, as this not only saves construction time but it also increases the flexibility of the underlying road. However, the traffic profile of the existing bridge must remain the same in order to prevent extensive groundwork activities. Therefore it is desirable to increase the slenderness of the deck to such extent that the deck height remains equal while the span lengthens.

The required slenderness to achieve this goal depends on the dimensions of the existing bridge, mainly the deck height and the total length. These dimensions have been gathered for numerous existing plate bridges and the result is given in Figure 2.7. In Figure 2.8, this has been translated into the required slenderness in order to replace these bridges by single span bridges with equal deck height.

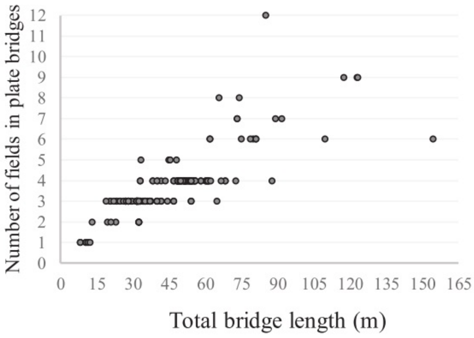


FIGURE 2.7: Relation between the number of spans in a plate bridge and the total length of the bridge *[Reitsema et al., 2015]*

FIGURE 2.8: The span number in a plate bridge related to the required slenderness for a single span *[Reitsema et al., 2015]*

It follows that 3-span bridges have lengths varying from 20 to 65 meters, with the majority between 20-40 m, while 4-span bridges have lengths varying from 35 to 90 meters with a majority between 40 to 60 m. If the current bridge deck is replaced by a single span with equal deck height, the required slenderness ranges between $\lambda = 30$–90 for the 3-span bridges and $\lambda = 65$–145 for the 4-span bridges.

3

Bridge replacement impact

As elaborated in chapter 2, there is a large bridge replacement task that lies ahead of us in the near future. Although the amount of bridges that need to be replaced and the year in which this will be done is unknown, it is imminent that this causes hindrance to both users and residents. This chapter will focus on the hindrance as a result of these large infrastructural construction works.

First, in section 3.1, an overview is given of the different types of hindrance that can occur in construction works and especially in a bridge replacement task. This will be followed up by an elaboration on current regulations to limit or prevent this hindrance in section 3.2. In section 3.3 focus will be laid on the impact of hindrance to the traffic flow and the accompanying economic damage, since this is by far the most important hindrance during bridge replacement projects. Finally, current forms of contracting are discussed in section 3.4, that will stimulate the reduction of hindrance during execution.

It turns out that through the current practice of MEAT it pays out to invest in technologies that reduce the on-site construction time substantially. Therefore it is the economic drive behind the execution of this thesis.

3.1. Hindrance during replacement

When replacing a bridge, or for that matter, building any structure, hindrance to the surroundings is an unavoidable consequence. This hindrance manifests itself in different forms and magnitudes but will always have to be minimized according to several building regulations (see section 3.2).

In general, the following effects of construction works are regarded as hindering:

- Noise;
  Almost all activities at a construction site generate noise to some extent. Large equipment, heavy traffic, warning sounds and material clattering are only a few examples. A major contributor for noise nuisance is also the installation of piles for foundations, although different gradations can be observed within the different pile types. As an alternative to driving, several screwed pile types have been developed that limit the magnitude of noise generated.
- Vibrations;
  As for noise hindrance, vibrations occur as a result of heavy traffic and most pile installation methods. This is not only experienced as being tedious, but can also damage fragile surrounding buildings.
- Dirt or dust;
  Open building pits or projects with large amounts of groundwork can be the cause of dirt or dust spreading over a larger area than the project site. The amount of type depends on the weather (rainy or dry) but can nevertheless be experienced as hindrance to surrounding residents.
- Stench;
  When the construction works are accompanied by sewer works, an annoying stench that carries over a large area can be the result.
- Light;
  If activities on the project site are carried out during night-time, these are most likely supported by

bright building lights. This can not only be experienced as annoying by residents, but can also cause disturbance within the animal night-life.

- Supply routes;
  All materials, heavy equipment and personnel have to be transported to and from the project site, thus causing an increase of traffic intensity on the surrounding infrastructural network. Depending on the size of this network and the amount of residents nearby, this can be experienced as hindering to a certain extent.
- Traffic detours;
  Especially during construction projects around or within the infrastructural network, the traffic flow can be disturbed to a more or less extent. Whether lanes are closed or the traffic is rerouted, it will imminently result in traffic delays.

The victims of this construction hindrance can be allocated to two groups: residents (people that live or work in the surroundings) and users (people that pass by). The first group experiences the largest impact, since construction works generate a lot of noise, vibrations, additional traffic and detours throughout the day and sometimes weekends or nights. For the second group, the users, only the hindrance in traffic flow as a result of detours is considered as being tedious.
It depends on the location of the construction site whether there are residents that experience hindrance from the construction works. Irrespective of this project location however, is the fact that with a bridge replacement task, it is imminent that the users and passers-by will be affected by traffic flow hindrance. The exact magnitude of delay and the duration of this hindrance does depend on the specific location and the traffic intensity of the infrastructural network around the project site.

3.2. DEMANDS FOR HINDRANCE

The allowed hindrance for residents and the allowed traffic hindrance is defined for every project in regulations and contracts between contractor and client. Before construction works can start, a Safety Plan must be presented and approved in order to prevent unsafe or unwanted situations during execution *[Bouwbesluit online, 2017]*.
The limitations on noise, vibration and dirt or dust hindrance are regulated in Bouwbesluit 2012, while limitations on stench and light will have to be handled case specifically. For the traffic delay as a result of supply routes and detours, so-called mobility management is applied in order to limit the hindrance as much as possible.

Bouwbesluit 2012 *[Bouwbesluit online, 2017]*
Up to a sound pressure level of 60 dB(A) during workdays, no exemption is needed for noisy construction works. Workdays are defined as weekdays and Saturday between 7 AM and 7 PM. For higher sound pressure levels (up to 80 dB(A)) a limitation is given on the maximum duration of the exposure, see Table 3.1.

Sound pressure level [*dB(A)*]	≤ 60	> 60	> 65	> 70	> 75	> 80
Maximum exposure duration [*days*]	Unlimited	50	30	15	5	0

TABLE 3.1: Maximum noise exposure duration depending on the sound pressure level *[Bouwbesluit online, 2017]*

Exemptions to the sound pressure level, the maximum duration of exposure and the workdays can be granted by the supervising authority.
This sound pressure level must be measured at the façade of sound-sensitive objects, according to Circulaire Bouwlawaai 2012 *[Ministerie van Infrastructuur en Milieu, 2010]*. Since this concerns a daily value, short-term noise exceedances of 5 dB(A) are allowed (for instance during pile driving). During night-time, a maximum sound pressure level of 40 dB(A) is advised.

For hindrance from vibrations, *[Bouwbesluit online, 2017]* prescribes an allowed vibration level as given in Table 3.2. Again, exemptions can be granted by the supervising authority.
For hindrance of dust and dirt, *[Bouwbesluit online, 2017]* speaks of measures to prevent visual dust distribution outside the project site. Examples are the application of covers, screens to reduce wind and wetting or dry-keeping of the terrain.

Duration D of activities during daytime (days)								
D ≤ 1			6 < D ≤ 26			26 < D ≤ 78		
A1	A2	A3	A1	A2	A3	A1	A2	A3
0.8	6	0.4	0.4	6	0.3	0.3	6	0.2

TABLE 3.2: Maximum duration of vibrating activities during daytime *[SBR, 2002]*

Mobility management *[Rijkswaterstaat, 2010] [Transumo, 2009b]*
In order to reduce the impact of traffic hindrance during construction works, use is made of so-called mobility management, which aims to influence the travel behaviour over a certain period of time. This can be done by reducing and distributing the amount of car trips in the network or by offering alternative routes and alternative transportation options. Another aspect of mobility management is the user satisfaction through clear and transparent communication.
The five most commonly applied execution options for road construction works with accompanying traffic hindrance are given by *[Rijkswaterstaat, 2010]* in Table 3.3.

	Execution option	Time/duration	Type of measure	Speed limit	Detour	Delay
1	**Night-time construction works, at least 1 lane is accessible** The other lanes are used for construction activities	One or multiple nights	Partial closure	90 or 70 km/h	No	Slight
2	**Night-time closure** Entire road part is closed	One or multiple nights	Full closure	-	Full detour	10 to 15 minutes
3	**Weekend closure** Highway is closed between two intersections from Friday night till Monday morning 5 AM	One or multiple weekends	Full closure	-	Full detour	± 30 minutes (also delay on detour)
4	**Less, narrowed and/or moved lanes accessible** The other lanes are used for construction activities	Varying between one weekend to several months	Less, narrowed or moved lanes	90 or 70 km/h	No	10 to 15 minutes during rush hour
5	**Multiple day closure** Highway is closed between two intersections on multiple consecutive workdays, 24 hours a day	Multiple days	Full closure	-	Full detour	± 60 minutes (also delay on detour)

TABLE 3.3: Most commonly applied execution options for road construction works *[Rijkswaterstaat, 2010]*

When looking at the demands for hindrance limitations, it can be seen that the hindering effects of noise, vibrations, dust or dirt, stench and light mainly impact the construction works on the project site and can therefore be taken care of easily. Traffic hindrance, however, has an impact of much larger scale, which is not confined to the direct surroundings of the project site but can cause a backlash on the entire infrastructure network nearby. Furthermore, the delays that are almost imminently the result of traffic hindrance have a large impact on the Dutch economy.

3.3. ECONOMIC IMPACT TRAFFIC HINDRANCE

The economic damage from traffic hindrance manifests itself in different ways. Direct damage for freight transport occurs from delays in logistics, for instance when products are delayed and thus the manufacturing processes experience losses. There is also indirect damage, as a result of increased costs from detours or the use of extra equipment to limit the loss of time. Also, the additional planning activities that follow from a limitation of losses will lead to extra costs. Furthermore, not only freight transport experiences economic damage from traffic hindrance, but also passenger transport *[van Zanten and Wiesehahn, 2016]*.

Current developments in traffic jams *[Rijkswaterstaat, 2017]*
When looking at the development of heaviness of congestion over the last few years, a clear relation can be seen with the increase of traffic volume (amount of vehicle kilometres covered) and the amount of available road kilometres. This heaviness of congestion is defined as the average congestion length times the duration, measured in kilometre minutes.

In Figure 3.1, the increase of covered vehicle kilometres is presented, from 55.6 billion in 2000 to 69.9 billion vehicle kilometres in 2016. This is a total increase of 25.7% over the last 16 years, with a 3.1% increase over 2016. The development of congestion rate over the same period is given in Figure 3.2. The worst year in history was reached in 2007, when a peak value of 15.7 million kilometre minutes was measured. This was the result of an increase in traffic, but although this increase has persevered, a fall in congestion rate has been observed since 2007 due to the economic crisis and new traffic lanes. Since 2014, the congestion rate has increased again as a result of economic growth and thus an increase in covered vehicle kilometres. Over 2015, this has led to an increase of 26% in congestion rate and over 2016 to an increase of 13% to a total of 11.57 million kilometre minutes.

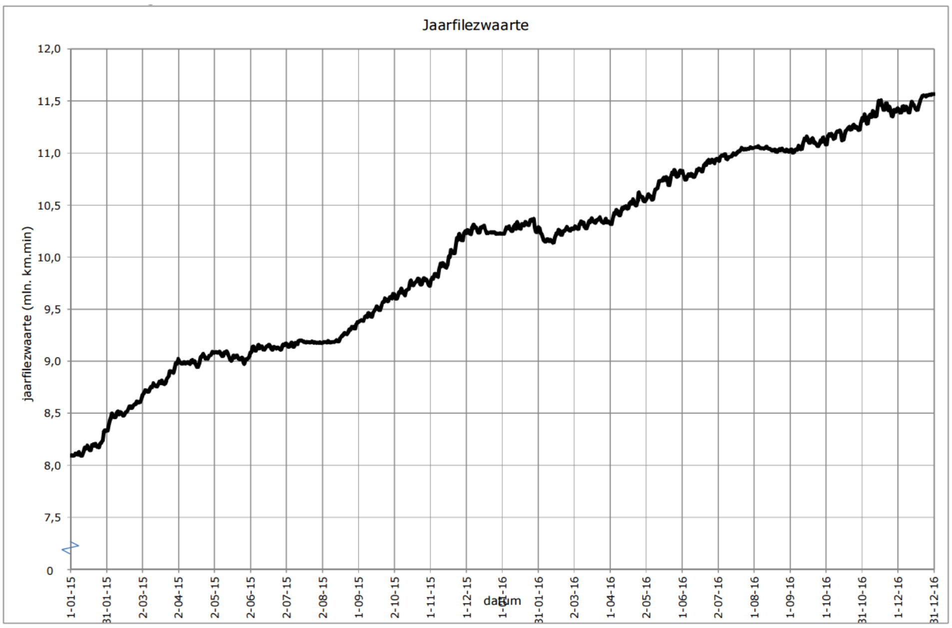


FIGURE 3.1: Heaviness of congestion measured in amount of covered kilometre minutes (in millions) *[Rijkswaterstaat, 2017]*

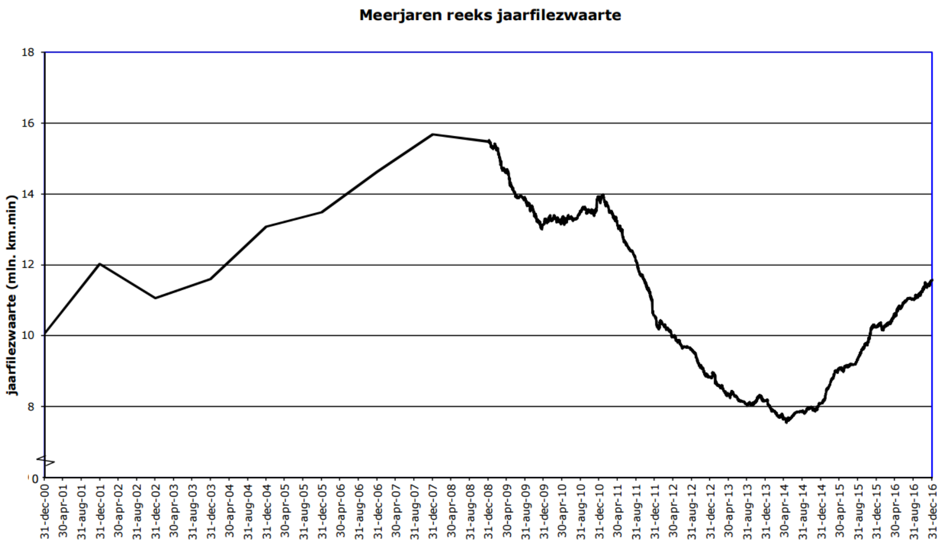


FIGURE 3.2: Congestion rate development measured in kilometre minutes (in millions) *[Rijkswaterstaat, 2017]*

Both Figures have been combined in Figure 3.3 to illustrate the relation between amount of covered vehicle kilometres and the congestion rate.

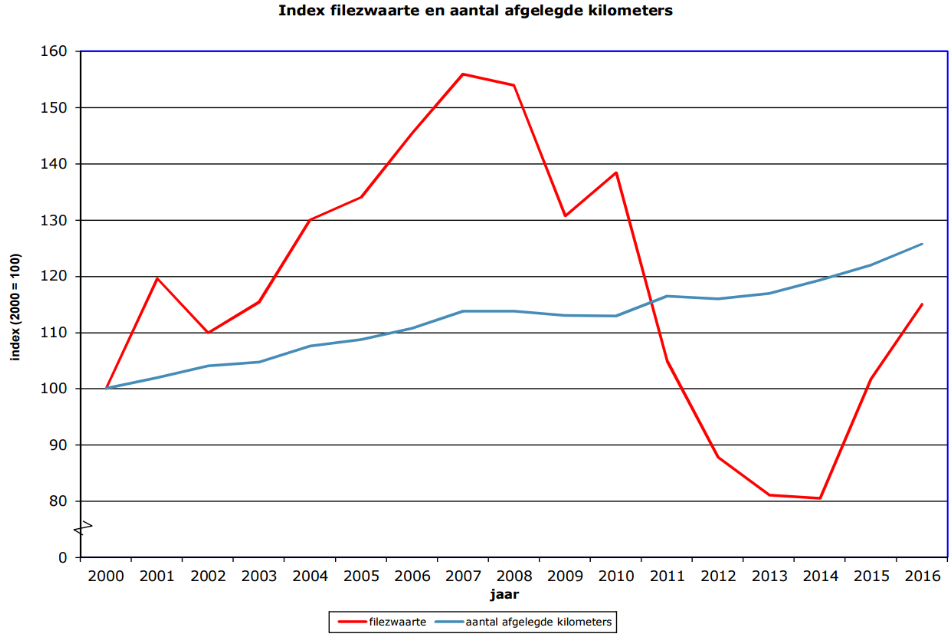


FIGURE 3.3: Relation between covered vehicle kilometres (blue) and congestion rate (red) *[Rijkswaterstaat, 2017]*

Another definition to measure the congestion rate is given by the so-called travel time loss, which is the difference between actual travel time and the theoretical travel time at 100 km/h. Although it is measured differently to the heaviness of congestion, it is also an indication of congestion rate. In Figure . . . , the travel time loss from 2000 to 2016 is presented, of which the largest part is caused by congestion (in green) and a smaller part is caused by general delayed traffic flows (blue). In 2016, a total travel time loss of 61.4 million vehicle hours was observed, which is an increase of 10.6% compared to 2015. The same trends as in Figure 3.2 for the heaviness of traffic can be observed, with a clear peak in 2007 and a low in 2013.

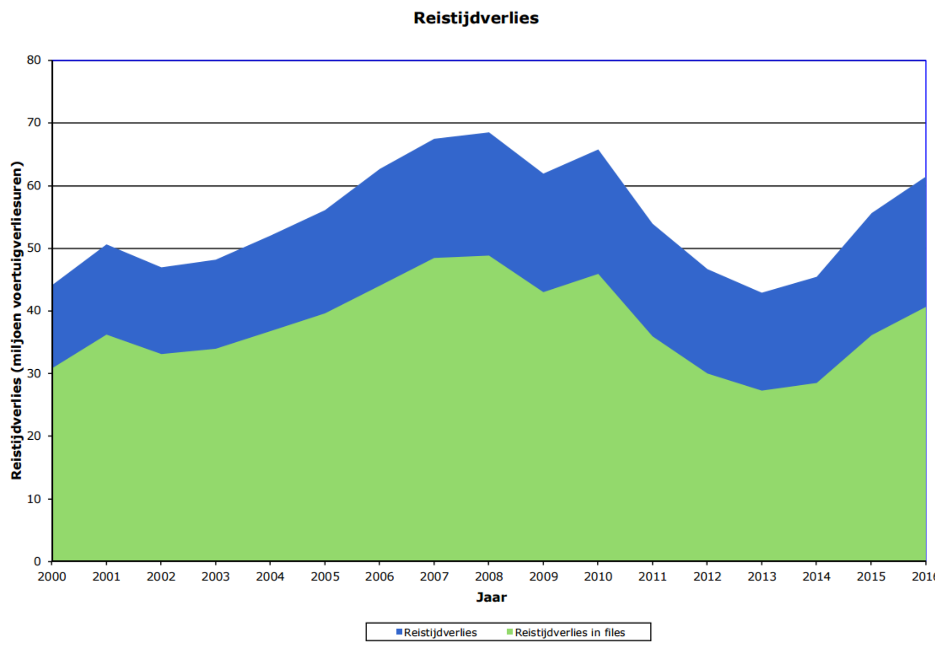


FIGURE 3.4: Congestion rate measured as loss of travel time in vehicle hours (in millions) *[Rijkswaterstaat, 2017]*

When looking at the distribution of traffic congestion over a day, it can be seen that the largest problems occur during rush hours and that the average congestion length has increased over the last few years (Figure

3.5). Furthermore, Figure 3.6 shows which highway section cause the most delay, in which a distinction is made based on the accepted travel time delay (which is 2 times higher than when travelling at 100 km/h on beltways and 1.5 times higher elsewhere).

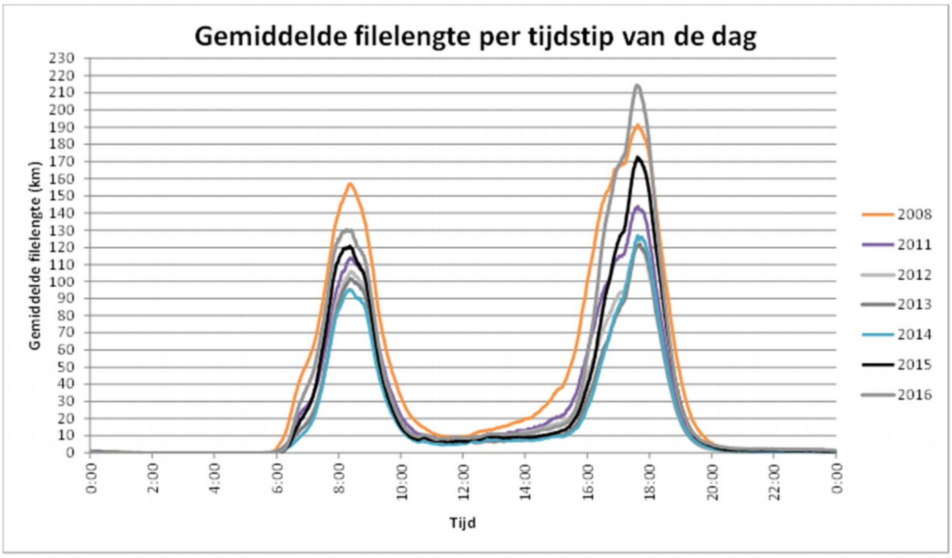


FIGURE 3.5: Average congestion length over a day *[Rijkswaterstaat, 2017]*

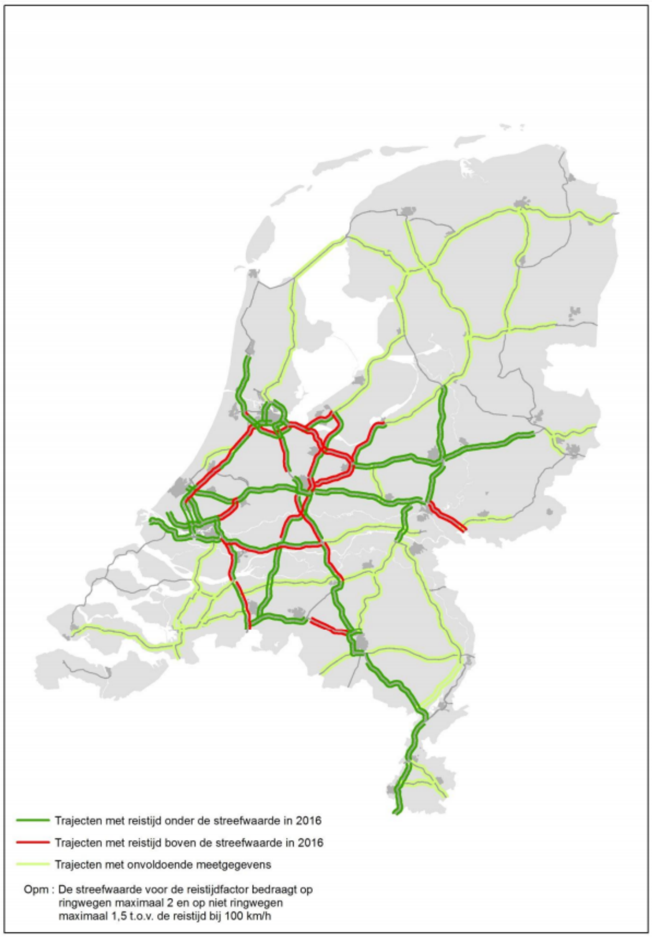


FIGURE 3.6: Overview of Dutch infrastructural network and the most heavily congested parts *[Rijkswaterstaat, 2017]*

Another interesting analysis is the cause of these congestions. From Table 3.4 it follows that two-thirds is caused by a high intensity during normal rush hours, 26% is caused by accidents or incidents and the remaining 6% is caused by, amongst others, construction works. In 2006, a maximum standard of 10% was agreed by the government, and as illustrated in Figure 3.7 this limit has clearly not been exceeded between 2010 and 2016.

Year	2011	2012	2013	2014	2015	2016
High intensity	73.3	70.0	68.3	62.5	66.5	67.4
Accident	15.0	16.5	19.6	21.8	19.4	19.1
Incident	4.7	5.7	4.8	8.0	7.9	7.4
Construction works	5.5	5.9	5.3	4.6	3.5	2.9
Event	0.2	0.3	0.3	0.3	0.3	0.3
Weather	0.1	0.2	0.2	0.1	0.5	0.1
Capacity reduction	0.3	0.5	0.7	0.8	0.6	1.0
Remaining	1.0	0.9	0.8	1.8	1.2	1.8
Total	100.0	100.0	100.0	100.0	100.0	100.0

TABLE 3.4: Distribution of different congestion causes (in percentages) *[Rijkswaterstaat, 2017]*

FIGURE 3.7: Ratio of congestions caused by construction works over the period 2010-2016 *[Rijkswaterstaat, 2017]*

Economical damage traffic jams *[van Zanten and Wiesehahn, 2016] [Kennisinstituut voor Mobiliteitsbeleid, 2016]*

As stated before, the occurrence of traffic delays causes economic damage directly to companies but also to the Dutch economy in general. An estimation can be performed on the direct damage encountered by companies, which is given in Figure 3.8, in which it can be seen that the economic impact follows the curves of congestion rate presented in Figures 3.2 and 3.4. The total economic damage in 2015 as a result of traffic hindrance is estimated to lie between €857 million and €1.114 billion, which is an increase of 30% compared to 2014 (€655 million to €852 million).

For the development of damage to the Dutch economy as a whole, the same trend can be observed in Figure 3.9, where both direct and indirect costs are covered, including the damage to companies. A total direct damage of €2.3 billion is measured, with an estimated additional damage of €0 to €0.7 billion from indirect effects. The increase of damage compared to 2014 is €0.7 billion or 20% and is the result of both an increase in amount of lost vehicle hours (80%) and a reduced reliability of travel time loss.

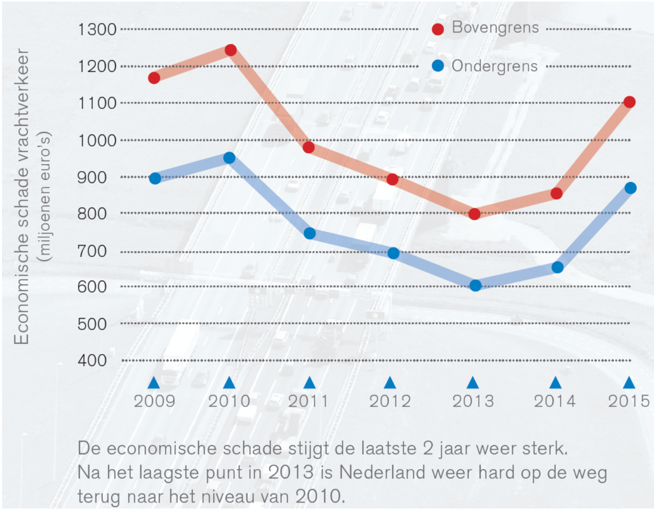


FIGURE 3.8: Direct and indirect economic damage to freight companies as a result of congestions in the period 2009-2015 *[van Zanten and Wiesehahn, 2016]*

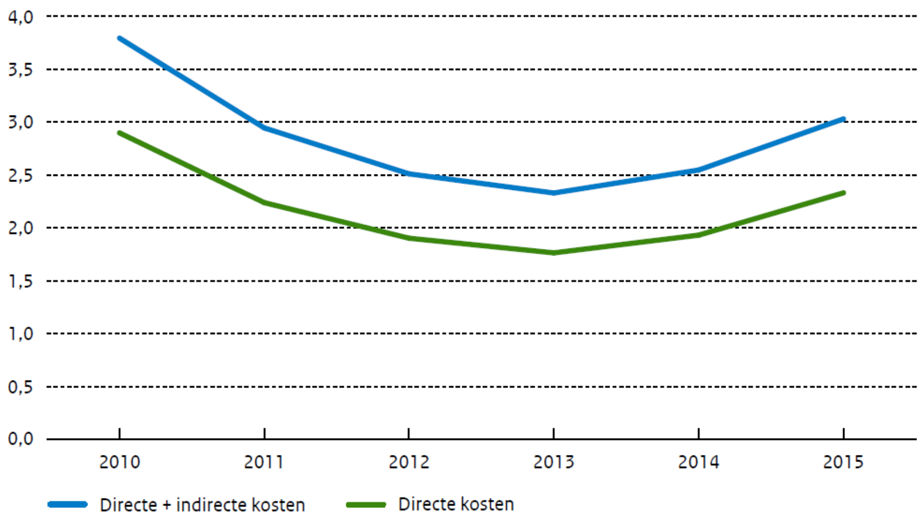


FIGURE 3.9: Total direct and indirect economic damage to the Dutch economy as a result of congestions in the period 2010-2015 *[Kennisinstituut voor Mobiliteitsbeleid, 2016]*

Forecast of traffic jams in the near future *[Francke and Wüst, 2016] [Kennisinstituut voor Mobiliteitsbeleid, 2016]*
In Table 3.5, a forecast is presented on the increase of vehicle kilometres and travel time loss over the coming five years, as calculated by *[Francke and Wüst, 2016]*, for general mobility policy purposes. It follows that an increase of 11.4% of vehicle kilometres compared to 2015 and an increase of 33.6% of travel time loss is to be expected in the period 2016-2021. This is the result of economic recovery and a lower realistic fuel price. Both increase in vehicle kilometres and travel time loss are illustrated in Figure 3.10.

	% average per year					% total
	2015	2016	2017	2018 - 2021	2016 - 2021	2016 - 2021
Vehicle kilometres	2.2	2.9	1.4	1.7	1.8	11.4
Travel time loss	22.5	8.1	4.7	4.2	4.9	33.6

TABLE 3.5: Forecast of the increase of mobility on the Dutch highways between 2016 and 2021 *[Francke and Wüst, 2016]*

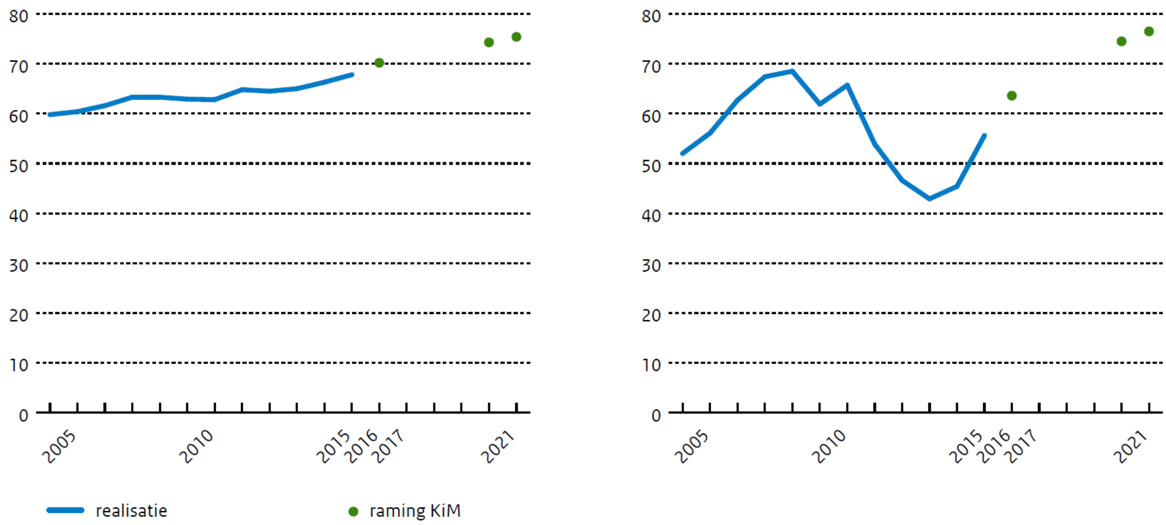


FIGURE 3.10: Illustrated increase of vehicle kilometres (left) and travel time loss (right) *[Kennisinstituut voor Mobiliteitsbeleid, 2016]*

Since there is a clear relation between the congestion rate and the encountered economic damage for companies and the Dutch economy as a whole, and it is estimated that the congestion rate will increase by 34% over the coming five years, it seems evident that the economic damage from traffic hindrance will also increase substantially over this period. It is therefore beneficial to look at execution solutions that limit the amount of traffic hindrance as much as possible.

3.4. CONTRACTING

As the economic damage as a result of traffic hindrance concerns the Dutch economy as a whole or logistic companies specifically, there is no direct economic stimulant for contractors to reduce the on-site construction time. This must therefore be incorporated in the contract between the commissioning company and the contractor.

Essence of collaboration *[de Ridder and Noppen, 2009]*
There are many different forms of building contracts or collaboration agreements that are applied nowadays and have been formed over the last decades. The type of agreement is based on its position between two extremes: the commissioning party does everything himself or everything is put out to a tender (Figure 3.11). This degree of collaboration is based on several aspects of the project, such as complexity, expected modifications and duration, see also Table 3.6.

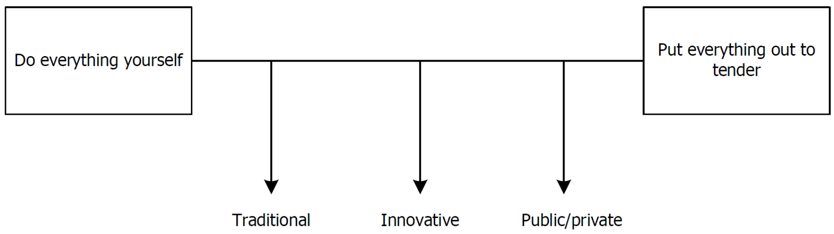


FIGURE 3.11: Two extremes in collaboration *[de Ridder and Noppen, 2009]*

Both the commissioning party (or employer) and the contractor (or supplier) have different interests but the same aim, which is in fact the essence of collaboration. The agreement is based on two major problems:

1. In order to ensure that the employer obtains the required values for the agreed price, a List of Requirements (LoR) is drawn up (Figure 3.12).
2. The agreed price must gain a profit margin on the construction costs for the supplier, while taking the risks into account. Therefore, a concept is drawn up by the supplier in which the costs are roughly determined (Figure 3.13).

Characteristics of collaboration	
Low degree	**High degree**
Temporary	Project duration is often long
Objectives have a low degree of complexity	Objectives are highly complex
One-sidedness and low complexity in points agreed upon, based on a Employer-Contractor system	Partners are equal
Activities are easily planned	Uncertainty about the progress and feasibility of the project
Agreed points are relatively simple	Points of agreement are difficult to lay down in a contract
No changes to be expected	Changes are to be expected

TABLE 3.6: Specifications for a low or high degree of agreement *[de Ridder and Noppen, 2009]*

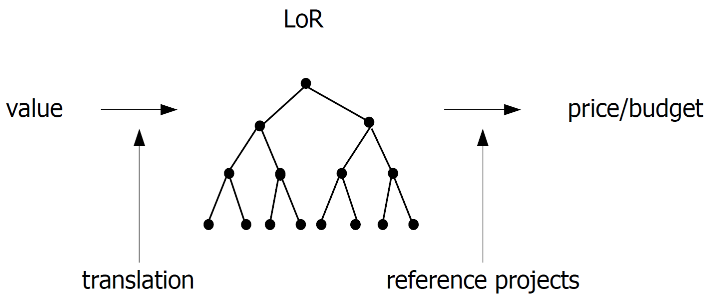


FIGURE 3.12: List of Requirements *[de Ridder and Noppen, 2009]*

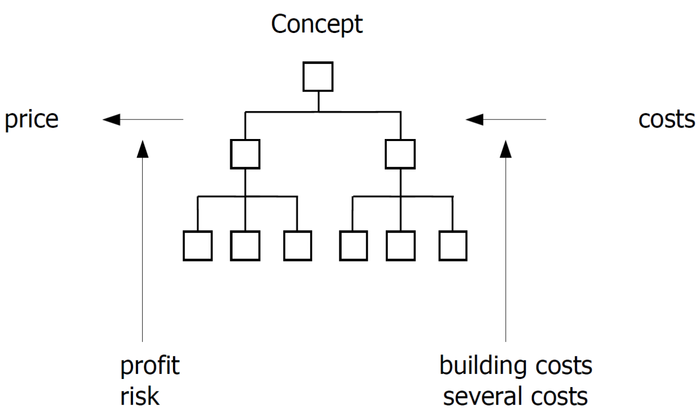


FIGURE 3.13: Concept *[de Ridder and Noppen, 2009]*

These two approaches are combined in Figure 3.14 into a process diagram for the specified project.
However, as mentioned earlier, risks have to be taken into account in the agreement on forehand. Two types of risks can be distinguished, namely the risk for employers in specifying the demands and the risk for suppliers in the production (Figure 3.15).
Overall, the main objective of composing an agreement is to determine a proper price for both parties. An employer can increase his influence on the price agreement by making use of market competition, while the supplier will want to maintain an optimum ratio between profit and risk (Figure 3.16).
The essence of collaboration as presented above can be combined with different types of contracting in order to determine the focus domains of both parties. This will be done within the following paragraphs of traditional contracting and new forms of contracting. Finally, attention is paid to the currently applied allocation procedure known as MEAT (Most Economic Advantageous Tender).

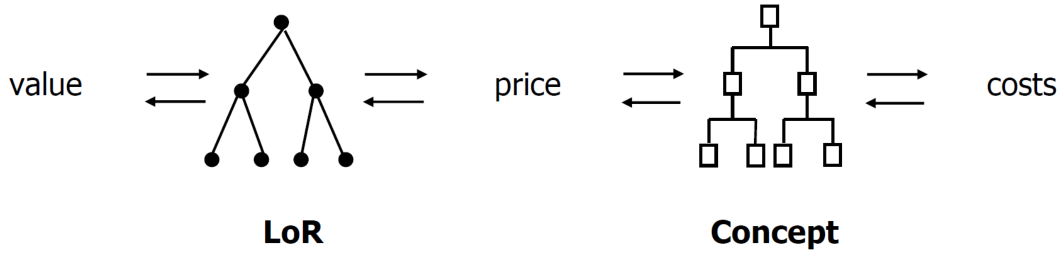


FIGURE 3.14: Combined process diagram for a project *[de Ridder and Noppen, 2009]*

FIGURE 3.15: Risks in the process diagram for a project *[de Ridder and Noppen, 2009]*

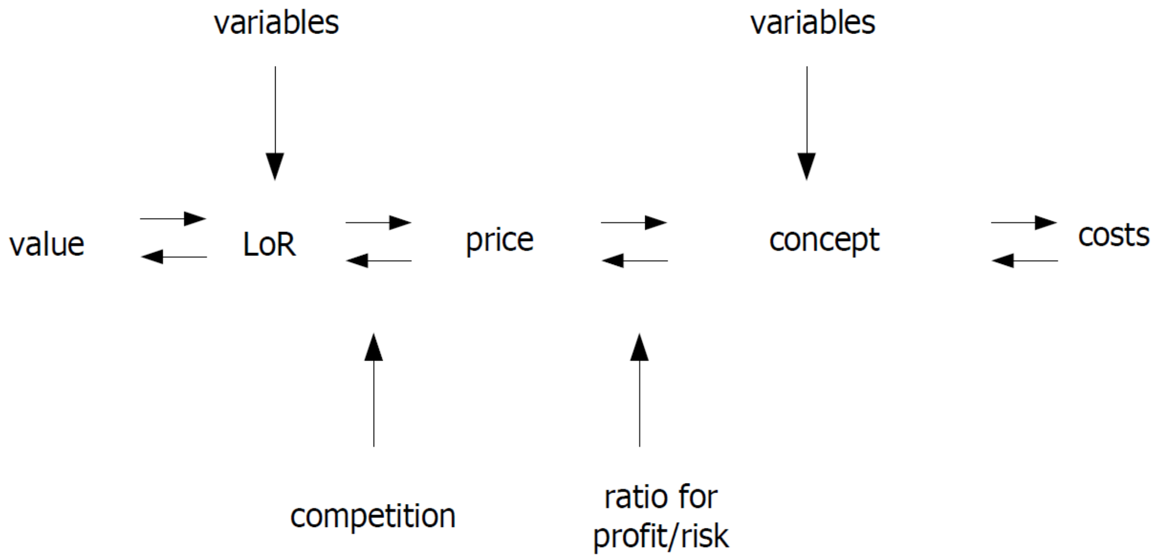


FIGURE 3.16: Influence factors on the process diagram for a project *[de Ridder and Noppen, 2009]*

Traditional contracting *[de Ridder and Noppen, 2009]*
The traditional form of contracting was very straightforward and known as a Bid-Build contract, in which the employer provides the design (perhaps with the exception of construction details) and the Contractor builds the permanent works. The responsibilities are clearly defined, since the design lies with the employer and the construction with the Contractor. This does require a high staff capacity within the employers organisation which might not be desirable. Furthermore, when dealing with complex projects the construction process is more and more an aspect that needs to be handled during the design phase in order to work efficiently and effective.

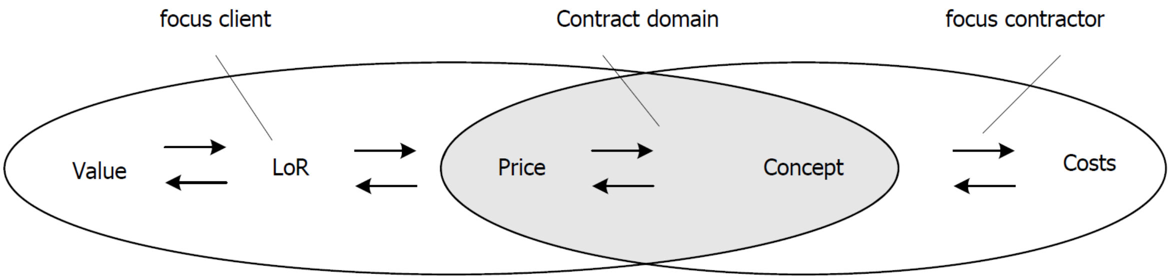


FIGURE 3.17: Focus areas of employer and supplier in a traditional contract *[de Ridder and Noppen, 2009]*

New forms of contracting *[de Ridder and Noppen, 2009] [de Ridder and Braat, 2008]*
In recent years, there has been a tendency within employers to shift responsibilities towards the supplier. Ultimately, the goal is to obtain an optimal degree of collaboration between the different parties in projects that become more and more complex. When looking at Figure 3.11, this shift in involvement and responsibility from employer to contractor is seen as movement from the first extreme ('Do everything yourself') to the second extreme ('Put everything out to tender').
The general new contracting forms and their relation to the building process is presented in Figure 3.18. It must be said that this list is far from complete, but does give a good idea of the different possibilities.

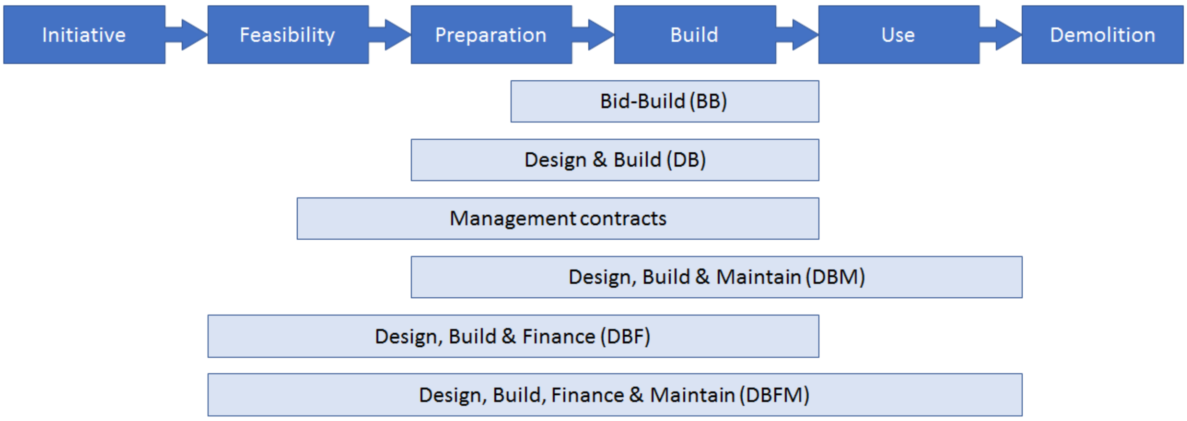


FIGURE 3.18: Different contract forms and their relation to the building process *[de Ridder and Braat, 2008]*

Most Economic Advantageous Tender (MEAT) *[CROW, nd] [Transumo, 2009a] [Transumo, nd]*
Since modification of the Aanbestedingswet 2012 on January $1^s t$ 2016, procurement is executed in Dutch practice according to the so-called MEAT-method. In fact, the already existing term MEAT is redefined as an overall term for three awarding methods:

- Lowest price (traditional method);
- Best price-quality ratio (former MEAT definition);
- Lowest costs based on cost effectiveness (or so-called Life Cycle Cost Analysis).

In this report, the term MEAT will however be based on the former definition and thus means the best price-quality ratio. In a MEAT procedure, not only the tender price but also performance and quality aspects like durability and hindrance can be taken into account and expressed as reduced costs (Figure 3.19). In this way, it can be beneficial for contractors to include investments for the required performance and quality criteria in order for the project to be assigned to them.

One of these quality criteria for infrastructural projects could be to guarantee an adequate traffic flow throughout the entire construction period. Depending on the location of the project, the traffic intensity of the surrounding network and the expected economic damage from traffic flow hindrance, a price is given by the employer that can be gained by reducing the traffic hindrance by a certain amount of time or other value.

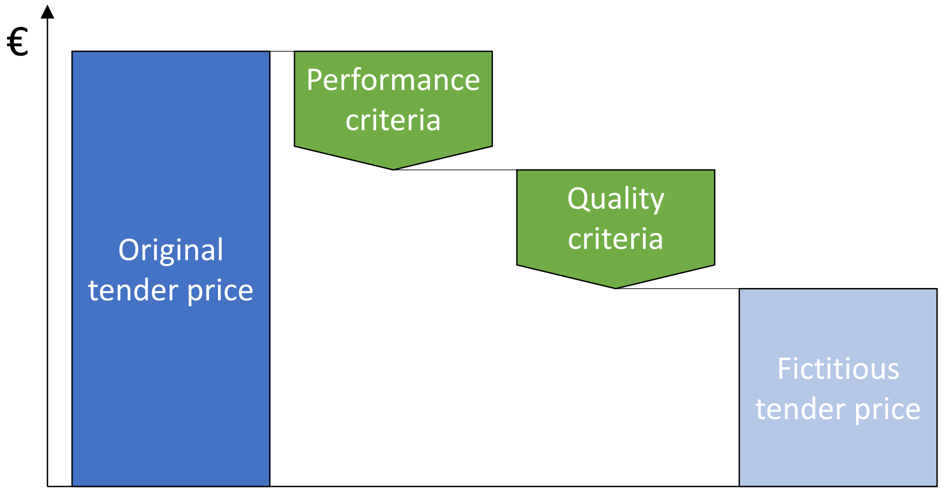


FIGURE 3.19: Fictitious tender price as the result of performance and quality criteria *[Transumo, nd]*

Although other quality or performance criteria will also effect the final fictitious tender price, when coping with a large infrastructural replacement task within a highway network the reduction of on-site construction time in order to minimise traffic hindrance will undoubtedly be governing in a MEAT procedure. Therefore, the method of assigning a project through MEAT is the economic driving force behind the objective of this thesis.

4

REFERENCE PROJECTS

The goal of this chapter is to illustrate the possibilities in the proposed reduction methods for on-site construction time as discussed in chapter 1.1. This is done according to the following subdivision:

- Section 4.1: Accelerated Bridge Construction (ABC);
- Section 4.2: Advanced Cementitious Materials (ACM's) - Ultra-High Performance Concrete (UHPC);
- Section 4.3: Existing foundations.

The outcome of these sections will be used as a starting point for chapters 5, 6, 7 and 8. In chapter 5 a conclusion will be drawn on the further improvement required, with respect to the current state-of-the-art as presented in this chapter, for the different time-reduction methods. Subsequently, the individual methods will be further elaborated in the chapters 6 to 8.

4.1. ACCELERATED BRIDGE CONSTRUCTION

Accelerated Bridge Construction, or ABC, is a bridge design and construct methodology that aims to reduce both the on-site construction time and overall project duration.

When looking at the current possibilities of ABC, it can clearly be seen that the U.S. is most experienced in applying ABC in bridge replacement tasks. Up to date, nearly all states have initiated projects in which ABC techniques have led to significant time reductions and safer work environments. To illustrate the possibilities in ABC, some reference projects are listed below that have been completed in the U.S. over the past few years. It should be noted that this is a very small selection and the range of opportunities in ABC is much larger than these projects suggest. More information on various projects in the U.S. can be found on *[Florida International University, ndb]* and *[Federal Highway Administration, ndb]*.

Geosynthetic Reinforced Soil-Integrated Bridge System (GRS-IBS):
I-70 Tower Road to Calfox Avenue *[Federal Highway Administration, 2015a]*
Parts of I-70 in Colorado were declared to be insufficient in travel safety and traffic flow. Therefore a portion had to be reconstructed and widened, including a twin bridge crossing an underlying road. The bridges, with an average span length of 135 m and a width of 13.1 m, had an average daily traffic (ADT) of 14.000 vehicles (of which 20% trucks) and were widened to 18.3 m. The major innovation in this bridge replacement task was however the application of GRS-IBS, being the first project in the U.S. to apply this technology for a multi-span and high-volume interstate highway. The choice for GRS-IBS was based on the following considerations:

- Reduced abutment construction duration from 1 month to 1 week;
- Reduction of abutment cost by 25%;
- Reduced differential settlement between bridge and approach roadway;
- Expanding the current knowledge on GRS-IBS.

The geotechnical investigation showed that the subsoil consisted of sandy clay and sandy claystone material. During construction, one bridge was replaced at a time, while the other was used for the re-routing of traffic in both directions. The overall project was completed in 2016.

FIGURE 4.1: Overpass situation prior to the I-70 bridge replacement *[Federal Highway Administration, 2015a]*

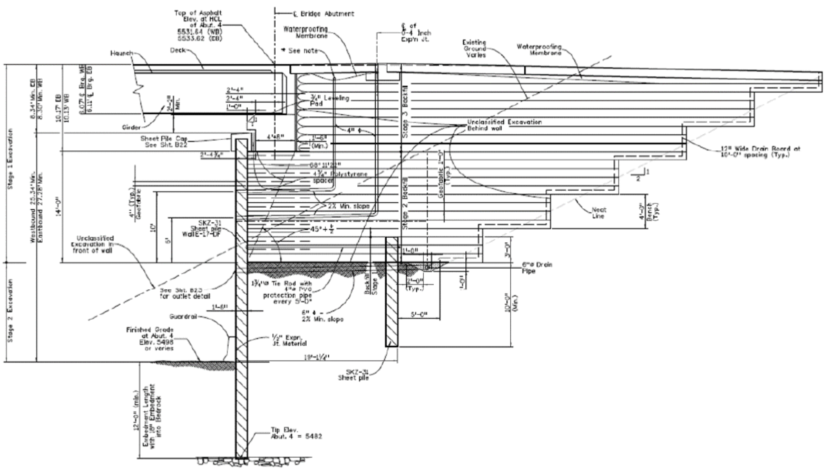


FIGURE 4.2: Application of GRS-IBS in I-70 bridge replacement task *[Federal Highway Administration, 2015a]*

Prefabricated Bridge Elements and Systems (PBES):
US 6 Bridge over Keg Creek *[Federal Highway Administration, 2013]*
To increase structural capacity, improve the roadway conditions and increase the user safety, the bridge in US 6 highway that crosses the Keg Creek in Iowa had to be widened. This bridge consists of three spans, with a total length of 33 m and a width of 8.5 m, with an ADT of 4.000 vehicles in 2009 of which 9% trucks.

Conventional construction methods for this bridge replacement task were estimated to cause 6 months of traffic hindrance. By applying prefabricated modules and UHPC joints, this impact on traffic could be reduced to 16 days. The prefabricated elements and modules were produced next to the project site to eliminate the need for heavy transportation. An additional advantage of prefabrication on-site was the possibility to properly fit the modules as they could be produced next to each other (see Figure 4.3). To further reduce the required on-site construction time no overlay was applied after erection of the modules and the joints.

The modular deck panels consist of two parallel steel beams topped with a HPC layer. All prefabricated elements were made of HPC and were produced using conventional construction equipment on the nearby farmland For the joints, UHPC was used (Figure 4.6), because of the strong and ductile properties that support flexural end tensile loads even after initial cracking.

The new bridge was completed in 2011.

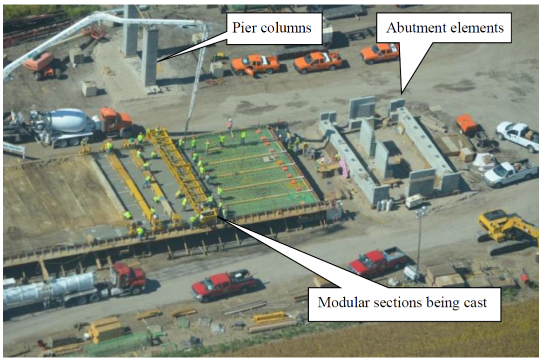


FIGURE 4.3: Overview of prefabrication site *[Federal Highway Administration, 2013]*

FIGURE 4.4: Erection of the prefabricated modules *[Federal Highway Administration, 2013]*

FIGURE 4.5: Prefabricated steel/concrete module *[Federal Highway Administration, 2013]*

FIGURE 4.6: Longitudinal joint will be filled with UHPC *[Federal Highway Administration, 2013]*

Lateral sliding:
I-84 Bridge over Dingle Ridge Road *[Federal Highway Administration, 2014a]*
The heavily used I-84, with an ADT of 75.000 vehicles, crosses Dingle Ridge Road in New York, which has an ADT of 3.000 vehicles. Two bridges are used for this crossing, with a differential elevation level of 4.6 m due to the steep slope of 15.7% in Dingle Ridge Road. Both bridges consist of three spans, with a total length of 42.7 m and a width of 10.1 m. Due to expected increase in traffic intensity over the years, the replacement bridges had to be widened to 17.4 m and as a result of the steep slope of the Dingle Ridge Road, they also had to be elevated by 0.6 m to maintain a minimal head clearance.

The high traffic intensity demanded a constant availability of two lanes in each direction, which would introduce a two year traffic hindrance period if the bridges were replaced by conventional method. To drastically reduce this hindrance period, a period of 20 hours between Saturday 5 p.m. and Sunday 1 p.m. was appointed as an allowable traffic disruption. Although the application of a SPMT would satisfy this time period, the steep slope of Dingle Road Ridge made this solution unwanted. Therefore, the choice was made to construct both replacement bridges parallel to the existing bridges and by making use of lateral sliding, these could be replaced in two separate weekend. This meant that in the period of a 20 hour closure, the old bridge had to be demolished, the new one slid into place and the approach way had to be elevated by 0.6 m.

To accelerate the overall project construction time, the replacement bridges were constructed with prefabricated elements, longitudinally connected through UHPC joints. The project was completed in 2013, with a total reduction of time, safety, costs, environmental impact and traffic hindrance.

FIGURE 4.7: Construction of two adjecent replacement bridges with differential elevation *[Federal Highway Administration, 2014a]*

FIGURE 4.8: Erection of the prefabricated superstructure elements *[Federal Highway Administration, 2014a]*

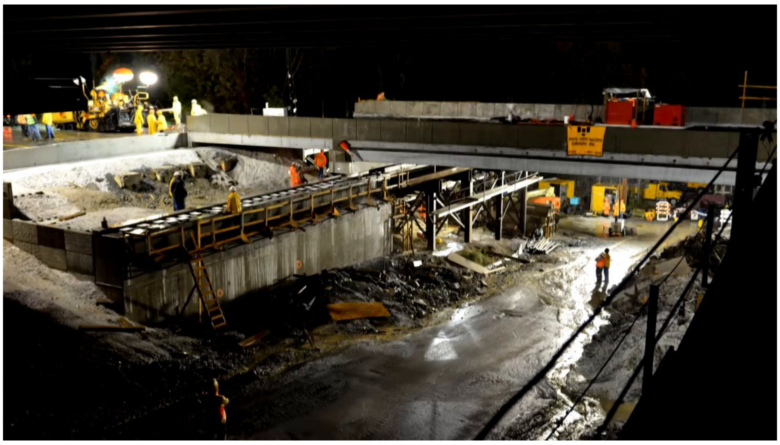


FIGURE 4.9: Lateral slide-in of replacement bridge *[Raglow, 2013]*

Self-Propelled Modular Transporter (SPMT):
Maryland Avenue Bridge over I-35E *[Federal Highway Administration, 2015b]*
Maryland Avenue Bridge, Minnesota, is situated in Highway 31 and crosses the I-35E. A combination of ageing, freeze-thaw cycles and de-icing salts resulted in such a poor structural condition (Figures 4.10 and 4.11) that the bridge had to be replaced. To reduce traffic hindrance to a minimum, which was

demanded by the high ADT of 28.000 vehicles on highway 31 and 140.000 vehicles on I-35E, the four span bridge with a total length of 64 m had to be replaced through innovative construction methods. The solution consisted of the following innovations:

- Off-site construction of a two-span superstructure, supported by falsework;
- On-site construction of the substructure, without hindrance of traffic flow on I-35E;
- Use of prefabricated elements, transported by ship;
- Use of EPS Geofoam as a light embankment fill material;
- Use of an SPMT for placement of the individual superstructure spans.

Where conventional construction methods would have result in partial lane closure for 4 months (Highway 31) and 16 days (I-35E), this ABC method resulted in a reduction of traffic hindrance to a 2 month closure of Highway 31 and a 15 hour closure of I-35E.

In the 2 month closure of Highway 31, the original bridge was demolished, the substructure was erected from prefabricated elements without impact on the traffic flow on I-35E, the new bridge was placed and the embankment and approach ways constructed. The 15 hour closure of I-35E was the result of the superstructure move by means of an SPMT. The project was completed in 2012.

FIGURE 4.10: Poor condition of the concrete cover in the Maryland Avenue Bridge *[Federal Highway Administration, 2015b]*

FIGURE 4.11: Poor condition of the concrete pier in the Maryland Avenue Bridge *[Federal Highway Administration, 2015b]*

FIGURE 4.12: Off-site erection of the prefabricated superstructure *[Federal Highway Administration, 2015b]*

FIGURE 4.13: Superstructure movement by SPMT to the bridge location *[Federal Highway Administration, 2015b]*

The possibilities in SPMT placement of superstructures are large: an example is the replacement of the Sam White Bridge (2012) situated in the I-15 in Utah, where a two span bridge was transported over 150 m, as one unit within 8 hours (Figures 4.14 and 4.15) *[Florida International University, nda]*.

FIGURE 4.14: Superstructure movement by SPMT to the bridge location *[Florida International University, ndc]*

FIGURE 4.15: Superstructure movement by SPMT to the bridge location *[Florida International University, ndc]*

Multiple ABC techniques are also very suitable for combined application, to further optimise the construction process. Two examples are presented below:

Combined application - GRS-IBS, PBES and lateral sliding:
I-84 near Salt Lake City *[Federal Highway Administration, 2014b]*
The I-84, with an ADT of 9.000 vehicles, crosses the underlying Echo Frontage Road in Utah by means of two bridges: an eastbound and a westbound bridge. In order to reduce the traffic hindrance to a minimum, these bridges are replaced by a combination of GRS-IBS, prefabricated elements and lateral sliding.

The soil investigation showed a favourable subsoil composition for the application of GRS-IBS, due to the presence of gravel, clayey sand, lean clay and silty clayey sand. With completion of the project in 2013, this was the first application of GRS-IBS in Utah, the first application of GRS-IBS in an interstate bridge in the U.S. and the first combined application of GRS-IBS and lateral sliding.

First, a temporary support in the median is constructed with the help of GRS-IBS, while both the east-bound and westbound traffic are undisturbed. Using prefabricated elements with a span length of 17 m, a bridge is constructed on the median supports, which will later become the eastbound bridge. After completion, the median bridge is however first use to reroute the westbound traffic from the original westbound bridge. This westbound bridge can then be demolished and the GRS-IBS substructure can be constructed. Prefabricated elements are again used to construct the new westbound superstructure. The westbound traffic is rerouted to the new westbound bridge upon completion, after which the eastbound traffic is rerouted to the median bridge. Now, the eastbound bridge can be demolished and a GRS-IBS substructure is made. The next step is the rerouting of eastbound traffic over a detour during 27 hours, in which the median bridge can be slid laterally into place as the new eastbound bridge and the approach ways can be completed. The overall road closure amounts only 34 hours and the use of GRS-IBS resulted in a total project construction time reduction from 194 to 125 days.

Due to the application of GRS-IBS substructures, the original three-span bridge with a length of 26 meters can be replaced by a single span bridge of 17 m long.

FIGURE 4.16: Underpass situation prior to bridge replacement *[Federal Highway Administration, 2014b]*

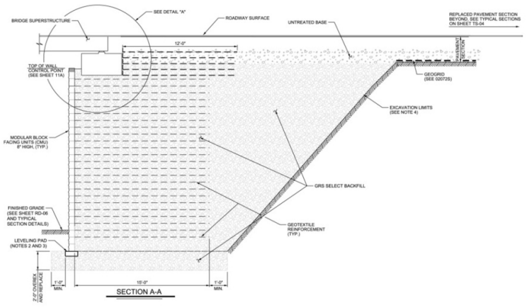


FIGURE 4.17: GRS-IBS substructure design *[Federal Highway Administration, 2014b]*

FIGURE 4.18: Construction of the median substructure with GRS-IBS *[Federal Highway Administration, 2014b]*

FIGURE 4.19: Erection of the superstructure from prefabricated elements *[Federal Highway Administration, 2014b]*

FIGURE 4.20: Lateral slide-in layout for the eastbound bridge *[Federal Highway Administration, 2014b]*

Combined application - GRS-IBS, PBES and SPMT:
East Shore Expressway Bridge over US 6 *[Federal Highway Administration, 2015c]*
The East Shore Expressway Bridge crosses Warren Avenue (US 6) in Rhode Island. The skew three-span bridge, with a length of 17-33-17 m and a width of 11.5 m, has an ADT of 23.000 vehicles. To ensure an undisturbed traffic flow on US 6, a combination of GRS-IBS, prefabricated elements and an SPMT is used in the construction method. The result is a total traffic impact of 80 hours.

To achieve this minimal traffic hindrance, the GRS-IBS substructure is constructed behind the existing interior supports while the existing bridge is still in use. The superstructure itself is constructed off-site by means of prefabricated elements, and transported to site by an SPMT after demolition of the old bridge. The project was completed in 2016.

FIGURE 4.21: Construction of the GRS-IBS substructure behind the existing interior supports *[Turn To 10, nd]*

FIGURE 4.22: Superstructure movement by SPMT to the bridge location *[WPRI, 2016b]*

FIGURE 4.23: Final situation after the bridge replacement *[WPRI, 2016a]*

When looking at the international application of ABC in bridge replacement tasks, little information is found as the definition of ABC is not commonly used outside the U.S. The techniques, on the other hand, have been used in different countries all over the world, but without the coordination of an ABC program and the combination of several techniques, the intended effect on time-reduction might be somewhat less. Nevertheless, some examples of Dutch practice in those ABC techniques are presented below.

Reinforced soil:
The application of reinforced soil is widespread in the Netherlands, but this has been limited to the construction of retaining walls with steep slopes. No projects have been found in which reinforced soil is used to construct a bridge substructure.

FIGURE 4.24: Retaining structure in reinforced soil *[Geologics, 2015]*

Prefabricated bridge elements:
It is common in Dutch bridge construction practice to make use of prefabricated elements to facilitate quick erection and thus minimal traffic hindrance. An example is the construction of the bridge in the A9, that crosses the A4 near Badhoevedorp (2016). Within five nighttime closures, the prefabricated girders with closed footing were placed on their bearings, after which traffic on the A4 had no further hindrance during the construction of the bridge *[Rijkswaterstaat, nda]*.

FIGURE 4.25: Erection of prefabricated bridge girders in A9 Badhoevedorp *[Spanbeton, nd]*

Lateral sliding:
Several projects in Holland have been completed by means of a lateral slide-in of the structure. This has for instance been done with the railway superstructure over the A9 near Gaasperdammerweg in 2015, in which the movement of the complete structure was realized within 5.5 hours *[Heijmans, 2015]*.

Another eye-catching project was the construction of a new underpass beneath the A12 near Ede. Within a weekend closure of the A12, room was made for the new underpass and a tunnel was slid laterally into the gap by means of hydraulic jacks *[Heijmans, 2016]*.

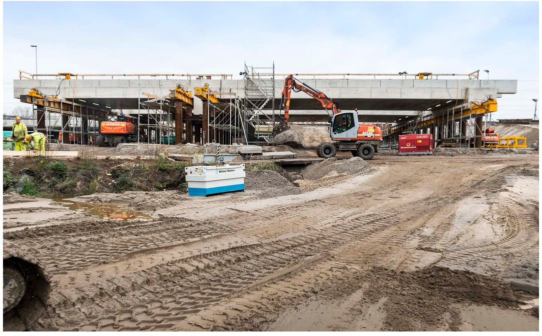


FIGURE 4.26: Lateral slide-in railway deck in A9 Gaasperdammerweg before movement *[Rijkswaterstaat, ndb]*

FIGURE 4.27: Lateral slide-in tunnel part in A12 Ede before movement *[Heijmans, 2016]*

Self-Propelled Modular Transporter (SPMT):
In 2016, the largest bridge movement by SPMT was performed in the SAA one project, in which the new Muiderberg railway bridge was moved within 7 hours. With a total length of 255 m, this is also the largest railway bridge in Holland. A remarkable fact is that the bridge was moved by SPMTs onto temporary supports next to the existing bridge. After completion of the Muiderberg bridge, the railway traffic was closed for a weekend, the old bridge was demolished and the new bridge was slid into place laterally into the original route *[Rijkswaterstaat, ndd]*.

FIGURE 4.28: Railway bridge Muiderberg during movement by SPMT's *[Rijkswaterstaat, ndc]*

4.2. ULTRA HIGH PERFORMANCE CONCRETE

UHPC is an ACM that has been increasingly applied in bridge constructions worldwide over the last years. Since the introduction of UHPC in 1995 by *[Richard and Cheyrezy, 1995]*, various countries have started research programs into the application of this new material in bridge design. Although its enhanced performance compared to NSC has been proved in many projects, application is still not widespread due to high initial costs and lack of international codes and regulations. A brief historic overview of the use of UHPC in bridge construction is given to illustrate the possibilities in UHPC design *[Voo et al., 2014]*

The first structure to be (partially) constructed out of UHPC is the Sherbrooke Pedestrian Bridge in Quebec, Canada, which was built in 1997.

Sherbrooke Pedestrian Bridge, Quebec, Canada *[VSL, 2012]*
Completed in 1997, the Sherbrooke Pedestrian Bridge is the world's first post-tensioned concrete structure in which UHPC is applied, although in this case it is referred to as Reactive Powder Concrete (RPC). The RPC applied had the following, revolutionary, properties:

- The compression strength was 4 times greater than HPC, namely 200 MPa;
- It had a tensile strength of 7 MPA and a flexural tensile strength of 40 MPa;
- The modulus of elasticity was 50 GPa and the fracture energy was 250 times greater than that of HPC.

These enhanced properties were mainly the result of a concrete mix containing small particles (< 0.5 mm) to create a dense material and of heat treatment at high temperature (90°C).
The Sherbrooke footbridge consists of a three-dimensional truss made of thin-walled stainless steel tubes filled with RPC and a RPC deck of only 30 mm thick. The post-tensioning is applied in the form of external prestressing tendons that are situated between the truss members. The bridge has a total span of 60 m and is 3.5 m deep, thus a slenderness of $\lambda = 17.1$.

FIGURE 4.29: Sherbrooke Pedestrian Bridge, Canada *[Russell and Graybeal, 2013]*

After this, the Seonyu Footbridge in Seoul, South Korea, the Sakata-Mirai footbridge in Sakata, Japan and several other Japanese segmental footbridges followed. Subsequently, other countries like France, New Zealand, Spain, Germany and elsewhere started to build UHPC bridges for pedestrian traffic.

Seonyu Footbridge, Seoul, South Korea *[Ricciotti, nd]*
The Seonyu footbridge, also known as the 'Bridge of Peace', connects Seoul with the Sunyudo Island and was competed in 2002 just in time for the World Cup. The desire for a slim design to blend in with the landscape and the long span length of 120 m resulted in the choice for UHPC, Ductal® in this case. The required mechanical properties ($f_{cd} = 180$ MPa, $f_{ct} = 8$ MPa and $E_{cm} = 50.000$ MPa) were achieved by adding short steel fibres ($l = 13 - 15$ mm, Ø = 0.2 mm) to the mix, with a volume of 2% of the total concrete volume. By applying heat treatment, the shrinkage and creep effects were almost eliminated. The arch is built up from six precast, post-tensioned π-shaped cross-sections with a depth of 1.3 m. The

deck is 30 mm thick with transverse prestressing and the webs, containing the longitudinal prestressing, are 160 mm thick. Because of the small dimensions of the webs, specially adapted small anchors were used to transfer the prestressing forces to the concrete. To eliminate potential vibrations in both horizontal and vertical direction, shock absorbing mass dampers were applied.
The Seonyu Footbridge is still the longest span UHPC bridge in the world and has a slenderness of $\lambda = 92.3$.

Figure 4.30: Seonyu Footbridge, Seoul, South Korea *[Ohio State University, nd]*

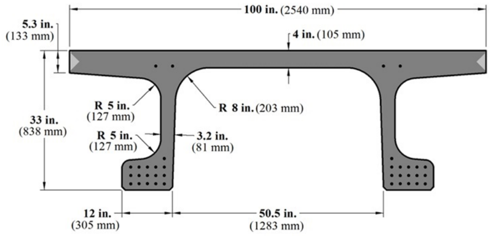


Figure 4.31: Example of a π-girder (with arbitrary dimensions) *[Russell and Graybeal, 2013]*

Sakata-Mirai Footbridge, Sakata, Japan *[Tanaka et al., 2010]*
Due to the location of this footbridge, about 3 km from the coastline, exposure to severe corrosive environment has led to construction of the first UHPC (again Ductal®) bridge in Japan in 2003. The enhanced properties of UHPC (high strength, ductility, fluidity and durability) make this material an interesting alternative, especially in earthquake countries like Japan.
For the Sakata-Mirai Footbridge, the following principles were applied to ensure a durable structure:

– A very low wcr (=0.22) to minimize defects such as microcracks and pore spaces;
– Optimization of the granular mixture by adding fine materials to increase the density;
– Application of heat treatment (90°C) to enhance the microstructure;
– Additional short steel fibres (l = 15 mm, Ø = 0.2 mm, V = 2%) increases the ductility.

This resulted in a compressive strength of 196 MPa and a flexural tensile strength of 36.5 MPa. Several requirements from the client led to a single box girder to span the 51 m river crossing. To reduce the dead weight as much as possible, the webs and slab were only 8 cm and 5 cm thick, respectively, and moreover the webs were perforated. Due to the steel fibres no passive reinforcement was needed and the required prestressing was applied externally. This led to a dead weight reduction of nearly 80% compared to a design in ordinary prestressed concrete. Since the height of the precast box girder increases from 0.55 m at the ends to 1.56 m in the middle, the obtained slenderness decreases from $\lambda = 92.7$ to $\lambda = 32.1$ *[Musha et al., 2013]*.

Figure 4.32: Sakata-Mirai Footbridge, Seoul, South Korea *[VSL, ndb]*

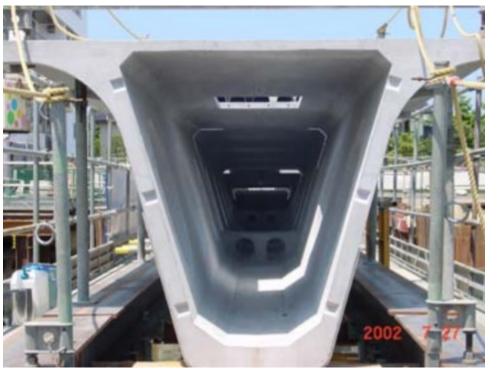


Figure 4.33: Cross-section of the precast box-girder *[Musha et al., 2013]*

It was not until 2005 that UHPC was used in the construction of road bridges. Four bridges were built at around the same time: one in Australia, two in France and one in Japan. Although these bridges consisted of different types of girders and prestressing, there was no conventional shear reinforcement present in all cases.

Shepherd's Gully Bridge, Australia *[VSL, 2008]*
This is the world's first UHPC highway bridge and was completed in 2005. It has a span of 15 m and a width of 20 m to accommodate four traffic lanes and a footpad. For the superstructure, 16 precast prestressed UHPC I-beams were used with a 170 mm in-situ deck from reinforced concrete. The formwork panels for the in-situ deck are actually thin precast UHPC panels that provide an additional environmental protection for the ordinary concrete. The I-beams have a depth of 600 mm and are spaced at 1.3 m.
In order to obtain certification for the crossing of highway traffic, the bridge was load-tested at completion and again one year later. The bridge behaviour that was expected from the design were confirmed by the tests and thus the bridge was approved for highway traffic.
The bridge has a rather small slenderness of $\lambda = 19.5$.

FIGURE 4.34: Shepherd's Gully Bridge, Australia *[VSL, 2008]*

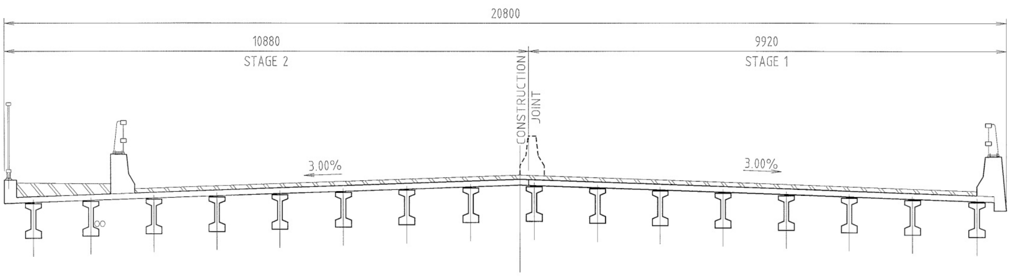


FIGURE 4.35: Cross-section of the Shepherd's Gully Bridge *[VSL, 2008]*

N34 overpass on the A51 (PS34), France *[Fehling et al., 2008]*
The N34 overpass on the A51 highway in France was built in 2005. It consists of a single 47.4 m long prestressed box girder and is made of BCV®, a UHPC type developed in France. It supports a 3 m wide regional road and has a height of 1.60 m over its entire length. Longitudinal prestressing is used to connect the 22 prefabricated segments and due to the high compression no additional watertight or paving layer is needed on the deck. Special formwork was used to achieve a high surface roughness for vehicle adhesion.
To achieve the required characteristics, BVC® uses a 2% volume fibre content, of which two-third has a length of 20 mm and one-third has a length of 12 mm. The final compressive strength is 130-150 MPa and the direct tensile strength is about 12 MPa.
In the SLS, full compression is required under longitudinal bending to avoid any durability issues from

crack formation, while at ULS small tension stresses are allowed as long as the compression zone is large enough to provide sufficient shear capacity. The transverse bending forces are however solely carried by the UHPC tensile strength capacity.
The 22 prefabricated segments were transported to the site and connected one-by-one on an assembling platform by temporary prestressing. After all segments were in place, final prestressing was applied and the entire construction was placed directly on its final bearings by a single crane.
Not only did this solution reduce the amount of material required from 200 m^3 in NSC to 80 m^3 in UHPC, it also eliminated the necessity of an intermediate pier. With the dimensions as given above, a bridge slenderness of $\lambda = 29.6$ could be achieved.

FIGURE 4.36: N34 overpass on the A51 (PS34), France *[Fehling et al., 2008]*

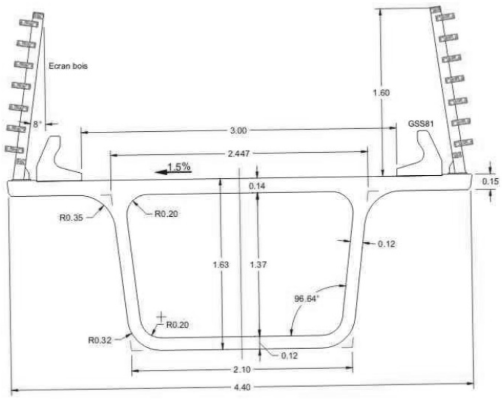


FIGURE 4.37: Cross-section of the precast box-girder *[Fehling et al., 2008]*

Saint Pierre La Cour Bridge, France *[Fehling et al., 2008]*
The deck of this railway overpass consists of 10 prefabricated prestressed I-girder beams made of UHPC (Ductal®). A reinforced concrete slab with a thickness of 200 mm is poured on top of the 25 mm thick prefabricated UHPC panels that are placed on the I-girders.
The material, with a compressive strength of 180 MPa and a direct tensile strength of 9.7 MPa, was strengthened with short steel fibres ($l = 12$ mm) and heat treatment was applied to ensure its durability. Due to the low weight of the prefabricated girders, with a length of 20 m, a single crane was sufficient for erection of the bridge. This resulted in a quick installation of the prefabricated elements, but an additional curing time for the in-situ poured traditional concrete deck was still required.
With a height of 0.95 m *[Resplendino and Toulemonde, 2013]*, the achieved slenderness of this bridge is $\lambda = 20.2$.

FIGURE 4.38: Saint Pierre La Cour Bridge, France *[Ductal, nd]*

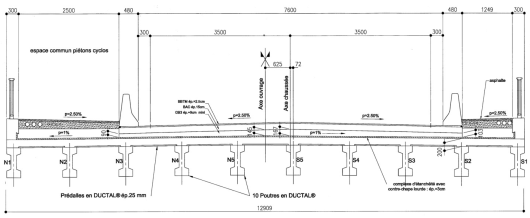


FIGURE 4.39: Cross-section of the Saint Pierre La Cour Bridge *[Fehling et al., 2008]*

Horikoshi C-ramp Bridge, Japan *[Resplendino and Toulemonde, 2013]*
This bridge, with a span of 16 m, was Japan's first UHPC bridge and was completed in 2005. The original design consisted of 11 prestressed hollow slab girders in ordinary concrete but it was changed into a composite design using UHPC girders and a cast in-situ deck in NSC. This caused a reduction in the amount of girders needed, from 11 to 4, and a total weight reduction of 30% which allowed the use of a lighter crane for erection.
The achieved slenderness of this bridge, with a girder height of 1.01 m *[Woodworth, 2008]*, is $\lambda = 15.8$.

FIGURE 4.40: Horikoshi C-ramp Bridge, Japan *[Resplendino and Toulemonde, 2013]*

As mentioned, in most countries the development of UHPC bridge construction has stagnated due to the high initial costs and perceived difficulties in design and production. In the USA, where extended research started in 2001 by the FHWA, new projects are still initiated but the technology is used only limited as a jointing material or as deck panels.

Malaysia, however, has been very successful in developing techniques for cost-effective solutions and have completed over 70 UHPC bridges between 2010 and 2015, with 32 expected to follow in 2016. Their largest savings have followed from *[Tadros and Voo, 2016]*:

- Creation of 4 standardized cross-section shapes with a maximum weight of 22 tons:
  - For short spans: pre-tensioned decked I-beams;
  - For medium spans: spliced I-beams and segmental U-girders;
  - For long spans: segmental box-girders.
- Simplification of the curing strategy caused a reduction of precast production cycle to the conventional 1-day cycle.
- Optimization of the UHPC mixture proportions made production of concrete with the required properties possible for only a small part of the initial mixture costs.

Kampung-Linsum, Malaysia *[Voo et al., 2012]*

At the time of completion, this 50 m long single span U-trough girder bridge was the longest UHPC bridge in the world. The 1.75 m high girder is combined with an in-situ poured ordinary concrete deck with a height of 200 mm. The 7 prefabricated segments are post-tensioned in two phases by 5 tendons. To achieve the required material properties (f_{cd} = 150 MPa, f_{ct} = 10.7 MPa and E_{cm} = 46.500 MPa) DURA® was used, with the addition of two types of steel fibres: straight fibres with a diameter of 0.2 mm and a length of 20 mm and end-hooked fibres with a diameter of 0.3 mm and a length of 25 mm. Due to the applied steam curing, post-production shrinkage has become negligible and the basic creep coefficient is reduced to $\varphi_{cc}(t) = 0.20$. Due to the addition of fibres, no conventional reinforcement is needed, except against bursting around the anchorage zones, some lifting reinforcement and horizontal shear reinforcement for the connection to the in-situ deck.

FIGURE 4.41: Kampung-Linsum Bridge, Malaysia *[Voo et al., 2012]*

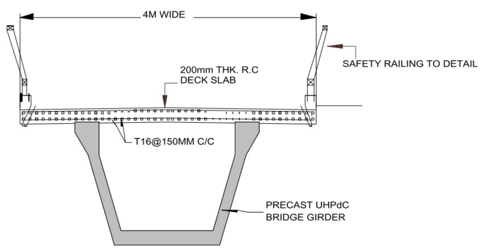


FIGURE 4.42: Cross-section of the U-trough girder *[Voo et al., 2012]*

The segments are connected using post-tensioning and then loaded onto a steel framed transfer girder by two cranes. The girder is then gradually launched over the river after which the steel frame is removed and the concrete deck is poured. To eliminate the net-deflection after two days the post-tensioning is increased to its final stress. Since it was the first UHPC road bridge in Malaysia, it was load-tested after completion. The acquired slenderness for this bridge is $\lambda = 25.4$.

Batu 6 Bridge, Malaysia *[Foster and Voo, 2015]*
Currently, this 100 m span box girder is the largest UHPC span bridge in the world. The 40 prefabricated box segments of 4.0 m height were placed on a falsework rail system and connected through post-tensioning. The largest difficulties during construction were however not the large dimensions and span, but the severe floods that occurred and their impact on the integral abutments.
The slenderness of this bridge is $\lambda = 25.0$.

FIGURE 4.43: Batu 6 Bridge, Malaysia *[DURA, 2015]*

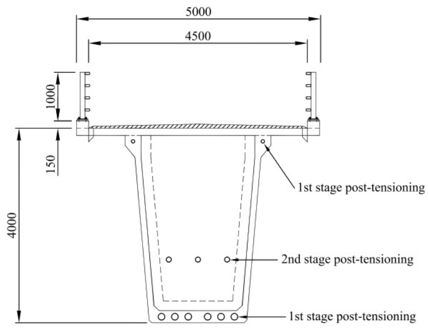


FIGURE 4.44: Cross-section of the box girder *[Foster and Voo, 2015]*

A further investigation into the precast elements developed by DURA® for the Malaysian market gives an overview of the current and near future applications of UHPC. The different elements have been listed in Table 4.1, together with their length range, weight and slenderness. It follows that the current UHPC girders that are applied by DURA® lie within a slenderness range of $\lambda = 20-30$, in which both multiple and single span bridges can be constructed. For small lengths and accessible project sites, pre-tensioned girders are possible, but most projects make use of segmental girders in combination with post-tensioning to acquire longer spans *[DURA, nd]*.

Type	Name	Height [mm]	Weight [kN/m]	Length range [m]	Slenderness range [-]
Box-girder	BBG4000	4000	66	80 - 100	20.0 - 25.0
U-girder	UBG1000	1000	6.7	16 - 23	16.0 - 23.0
	UBG1250	1250	11.8	20 - 35	16.0 - 28.0
	UBG1750	1750	22	26 - 52	14.9 - 29.7
	UBG2000S	2000	21	40 - 55	20.0 - 27.5
	UBG2250	2250	26.5	50 - 61	22.2 - 27.1
	UBG2500	2500	30	55 - 68	22.0 - 27.2
T-girder	TBG875	875	8.3	12 - 20	13.7 - 22.9
	TBG1000	1000	7.8	12 - 20	12.0 - 20.0
	TBG1325	1325	10	18 - 32	13.6 - 24.2
I-girder	IB650	650	3.4	8 - 18	12.3 - 27.7
	IB850	850	4	19 - 22	22.4 - 25.9
	IB1000	1000	4.3	23 - 25	23.0 - 25.0
U-trough	UTG1200	1200	13	9 - 30	7.50 - 25.0
	UTG1500	1500	40	9 - 30	6.00 - 20.0

TABLE 4.1: Property comparison of NSC and UHPFRC *[van Oosten, 2015][AFGC, 2013][Paskvalin, 2015]*

Over the last few years, the implementation of UHPC in bridge constructions has stagnated, as the higher costs of production and placement are not adequately countered by the enhanced properties of the material. In other words, in order for UHPC to be beneficial in a bridge design, the enhanced properties of UHPC must be indispensable. The Malaysian application of UHPC is a good example, since the remote locations in the jungle ask for easy transportation in the form of light-weight segments and a low maintenance as a result of high durability.
In this respect, the application of UHPC to develop very lightweight and slender prefabricated girders, to remove intermediate supports from a three- or four-span bridge, might be very promising.

4.3. EXISTING FOUNDATIONS

The reuse of existing foundations when replacing or expanding a structure is not new but has been applied in many cases. In fact, in early 16th century London, it was rule rather than exception to reuse foundations in order to prevent urban sprawl. With the increase of heavier structures and higher performance requirements, the amount of applications of foundation reuse has declined. In present construction projects, the accompanying cost and time savings and increased sustainability, have introduced a new upswing in using (part of) the existing foundation.

A large contributor to this upswing was the initiation of research programme RuFUS (Re-use of Foundations for Urban Sites), by the European Union in 2003. This resulted in a manual on European practices: "Reuse of Foundations for Urban Sites: A Best Practice Handbook" *[Butcher et al., 2006]*. Multiple projects were assessed throughout Europe (London (UK), France, Sweden, Greece and Germany) in which the foundation of buildings was reused. Two of these projects is elaborated below:

Empress State Building, London, UK *[Giannopoulos, 2011]*
This building was originally built in 1961 and founded on so-called under-reamed piles in London Clay. Under-reamed piles are bored cast in-situ concrete piles with bulb-shaped enlargement near the pile base. In 2003 the building was refurbished, in which 3 floors were added to the 100 m high structures, together with a 6 m southward extension. Although desired otherwise, the resulting 33% load increase could not be carried entirely by the existing foundation. In this calculations, a maximum pile capacity was assumed just below their previously experienced load to ensure the stiffness of the piles after reloading. But despite the 10% additional capacity on which piles were originally designed in that time, the existing pile foundation was found not to be sufficient to carry the new structure. New piles were added underneath the new extending part, which were coupled to the existing foundation system by means of stiff beams to reduce the probability of unacceptable settlement differences.

FIGURE 4.45: The Empress State Building *[Wikipedia Commons, 2012]*

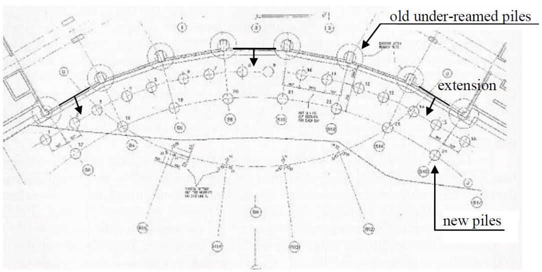


FIGURE 4.46: Existing and new foundation plan for the Empress State Building *[Giannopoulos, 2011]*

Juxon House, London, UK *[Giannopoulos, 2011]*
This building was constructed in the early 1960s and is also founded in London Clay using under-reamed piles. The existing 10 storeys were demolished after 40 years to make room for a new 8-storey building with a larger floor plan than the original. This resulted in the need for additional foundation

piles underneath the extension. Again, the existing pile capacity was based on the maximum experienced load to take into account the unknown long-term capacity behaviour and to remain stiff.

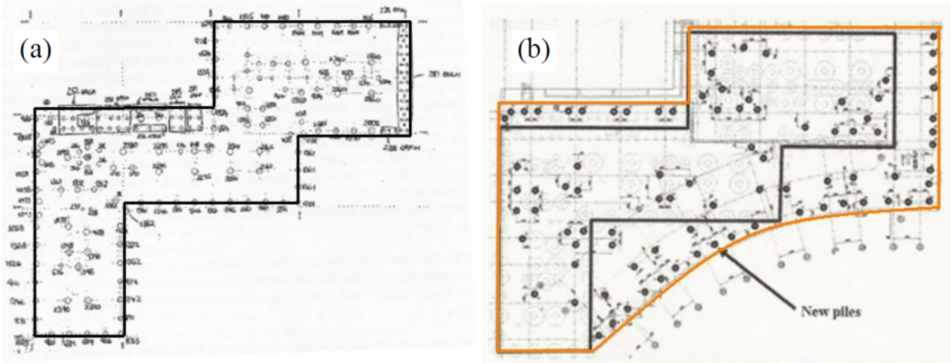


FIGURE 4.47: Existing and new foundation plan for the Juxon House *[Giannopoulos, 2011]*

When looking at the reuse of bridge foundations, not many cases are known in Europe. Within Heijmans, there are two current bridge projects that have an increased load but will still make use of the existing foundations, namely the Wilhelmina lock in Zaandam and the widening of the railway near Zevenaar. The latter has just been appointed to Heijmans and is therefore still in the preliminary design phase, but the Wilhelmina lock is discussed below:

Wilhelmina lock, Zaandam, Holland *[van Cann et al., 2017]*
The Wilhelmina lock, built in 1903, is situated in Zaandam and forms a busy junction in road and water traffic. The lock itself is confined between two bascule bridges, the Beatrix Bridge (1958) in the South and the Wilhelmina Bridge (1965) in the North, see Figure 4.48. In order to accommodate for a bigger ship size to pass the lock, the lock must be widened at the position of both bridges, which results in the need for a lengthening of both moveable bridge elements of about 2 m., from a 12 to 14 m span length.

FIGURE 4.48: Overview of the Wilhelmina lock, with at the top (south) the Beatrix Bridge and at the bottom (north) the Wilhelmina Bridge *[van Cann et al., 2017]*

This lengthening, more or less equal for both bridges, is obtained by demolishing part of the foundation on the eastern embankment, see Figure 4.49. Not all foundation piles have to be removed; the back piles are maintained and complemented by newly driven Tubex piles. The bearing capacity of the existing back piles is based on new calculations and both old and new CPT results. There is however an uncertainty in the pile depth of the existing piles that has been taken into account through the factor ξ. Furthermore, it was assumed that the piles were able to bear at least the static load from the old structure, as the foundation had already proven itself for this load.

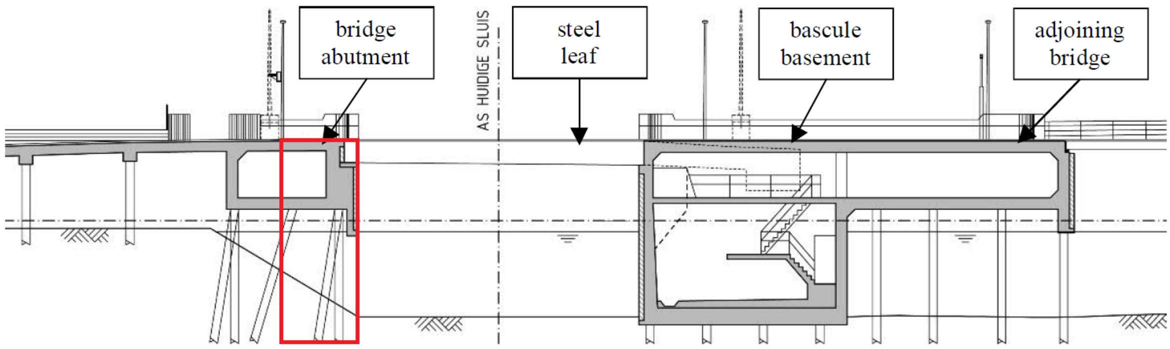


FIGURE 4.49: Cross-section of the Beatrix Bridge (viewed from north to south) with the partly demolished embankment on the left and the bascule basement on the right *[van Cann et al., 2017]*

Due to the fact that the bascule basement on the western embankment, with the exception of a small top part, did not have to be reconstructed, a solution was sought in which the existing foundation system would not have to be altered. For this, a distinction was made between the static and dynamic loading of the old and the new bridge. Since the foundation had proven over time to be able to bear the static load and the accompanying settlements had already occurred, the same static load could be carried without recalculation. For the dynamic loads, the design value of spring stiffness could be reduced, as it was proven that the dynamic loads were short-term and thus the stiffness from the soil layers is higher than for static, long-term loads.

In the U.S., where most bridges were constructed a decade earlier compared to the Netherlands, the call for foundation reuse in bridge design has led to the development of guidelines based on current practice *[Boeckmann and Loehr, 2017]*. This was preceded by the Foundation Characterization Program by the FHWA in 2013, in which issues around unknown foundations are addressed, like embedment depth, geometry, material, foundation integrity and load capacity. Multiple case histories are discussed in these guidelines, of which two representative examples are included below:

Bowker Overpass over I-90, CSX Railroad and Ipswich Street, Massachusetts, U.S.A. *[Boeckmann and Loehr, 2017]*
This four-span bridge was constructed in 1965, in Boston. The concrete slab deck was supported by steel girders and reinforced piers and the superstructure needed to be replaced in 2015. The foundation consisted of steel tubular pipes that led the loads through Boston Blue Clay onto the bedrock and were filled with concrete. The investigation whether the substructure could be reused was mainly focussed on the pile condition.

Hunt Road over I-495, Massachusetts, U.S.A. *[Boeckmann and Loehr, 2017]*
This bridge was constructed in 1969 and consisted of a concrete deck on steel beams, that rested on two abutments and four multi-column concrete piers. These piers were founded on spread footings, but the abutments were supported by cast in-situ concrete piles. It was investigated whether the substructure could be reused while the superstructure was replaced. This investigation was mainly based on historical documents, testing of substructure concrete cores and structural analyses performed with the new loading scheme. This resulted in the recommendations to reuse the pile foundation but to replace the abutment backfill by lightweight geofoam to reduce the dead load on the piles.

Overall, there is certainly experience in the reuse of bridge foundations, but due to the amount of uncertainties on current quality and increased loads this often leads to conservative designs. Also, the soil conditions on which the U.S. bridge projects were founded differs substantially from the soil conditions to be expected in a Dutch bridge replacement task.

Part II:

Methods for a quick replacement strategy

5

Required improvements

When comparing the current state-of-the-art as elaborated in chapter 4 and the problem at hand as described in chapters 2 and 3, a conclusion can be drawn on the further improvements required to reach the established objective as stated in section 1.2. In other words, what additional information is needed on ABC, ACM's and existing foundations in order to formulate a quick bridge replacement strategy with minimal traffic hindrance?

In the following chapter, all three subjects will be covered separately in order to determine the further required information. These same subjects will be further elaborated in chapters 6, 7 and 8, respectively.

Accelerated Bridge Construction (ABC)
From the reference projects in chapter 4 it follows that several ABC techniques have already been applied in Dutch practice, although they are not defined as such. A potential profit could be obtained if the extent of ABC, as applied in the US, is fully or at least partly incorporated into Dutch design and construction processes. Especially the large on-site construction time reductions that are possible when using a lateral slide-in technique or SPMT transport are very promising with regards to minimizing the traffic hindrance.

In order to use ABC to its fullest potential in a bridge replacement task, more information is needed on the different techniques and their application possibilities in the Dutch construction industry. This information will be gathered and presented in chapter 6.

Advanced Cementitious Materials (ACM's)
Although several ACM's have been mentioned in section 1.1 as possible solutions to obtain a slender and lightweight superstructure, the reference projects in chapter 4 are entirely focussed on the current applications of UHPC. This is a preliminary conclusion based on committee experience, which will be supported in the findings on several ACM characteristics as listed in chapter 7.

From section 2.3 and chapter 4 it follows that there is a discrepancy in the currently available industry slenderness and the slenderness that is required to remove the intermediate supports in a 3- or 4-span bridge. Furthermore it can be concluded from chapter 4 that the application of UHPC in bridge girders has stagnated over the last decade. The main explanation is that in order for UHPC bridge girders to be profitable, the boundary conditions must be such that the use of NSC girders is not possible.
For that same reason, use of UHPC girders in a quick bridge replacement task might be very promising, since the removal of intermediate supports and required quick erection ask specifically for a slender and lightweight structure. The question whether these specifications can be obtained by applying UHPC, will be covered in chapter 7.

To answer these questions, more information is needed on the characteristics of UHPC and their potential benefit in a bridge replacement task, together with the theoretically achievable slenderness of UHPC bridge girders.

Existing foundations

Although existing foundations have often been reused in the past (see section 4.3), the applications were mainly confined to buildings and conservative assumptions on the pile bearing capacity. In a bridge replacement task, however, an increase in foundation loading is to be expected, especially when removing the intermediate supports. In that case, no real benefit is obtained when using a conservative approach and therefore more research is needed into the behaviour of pile foundations in general and the time-effect on the pile bearing capacity of existing foundations specifically.

In chapter 8, information is gathered on research into the pile bearing capacity, applicable design codes, time-effects and other aspects that might have a positive or negative influence on the reuse of the existing foundations. Furthermore, in case the existing foundation is not sufficient for reuse in a bridge replacement task, information is given on possible strengthening methods for this foundation.

6

Accelerated Bridge Construction (ABC)

The goal of this chapter is to address the possibilities in ABC and detect potential applications of ABC in the scope of this thesis.

This is done rather straight forward and according to the scheme presented in Figure 6.1. First, a general description of ABC is given in section 6.1, after which the several ABC methods will be elaborated in section 6.2. This chapter will be concluded by section 6.3, in which the potential application of ABC in a minimal traffic hindrance strategy for bridge replacement is discussed.

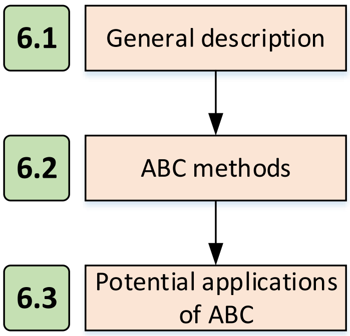


FIGURE 6.1: Schematic representation of chapter 6: Accelerated Bridge Construction (ABC)

The main outcomes of this chapter are:

- The Dutch practice already has experience in some ABC techniques, especially in using prefabricated elements;
- For the reduction of on-site construction time in a bridge replacement task, the following options can be considered:
  - Use of prefabricated elements (transported by road) and the erection by conventional cranes;
  - Use of prefabricated elements (transported by road), construction of modular systems on a location nearby and the erection by conventional cranes;
  - Use of prefabricated elements (transported by road), erection by conventional cranes parallel to the existing bridge and placement by lateral sliding;
  - Use of prefabricated elements (transported by road), erection by conventional cranes on a nearby location and placement by SPMT transport.

  The application of these options depend on the element or modular sizes and allowed closure time of underlying and crossing road.
- Reinforced soil might be used to reduce the total span length of the bridge.

6.1. General description

Accelerated Bridge Construction, or ABC, is a bridge design and construct methodology that has been largely implemented in the U.S. in bridge replacement tasks or the construction of new bridges *[HNTB, 2016]*. It is promoted by the Federal Highway Administration (FHWA), which is a part of the U.S. Department of Transportation. The FHWA is responsible for the quality and safety of all U.S. roads and railways and from that perspective they have set up an initiative in 2011 called 'Every Day Counts' (EDC). This program is used to identify and rapidly develop innovations in infrastructure projects, to reduce traffic impacts as a result of a decreased delivery time while keeping in mind the safety of the roadway and the improvement of sustainability. Although previously applied in several projects, ABC has been incorporated into EDC *[Michael and Culmo, 2011]* after which the amount of implementations has increased substantially. FHWA describes ABC as:

"ABC is bridge construction that uses innovative planning, design, materials and construction methods in a safe and cost-effective manner to reduce the on-site construction time that occurs when building new bridges or replacing and rehabilitating existing bridges." [Michael and Culmo, 2011]

The ABC method itself consists of different technologies that have one common goal *[Federal Highway Administration, nda]*: to minimize traffic hindrance, by taking as much work as possible away from the project site. The additional benefits are a shorter construction schedule, improved productivity and quality from prefabrication and improved safety for traffic and construction workers.
The reduction of on-site construction time can be realised by either a quick assembly of prefabricated elements on site or the movement of complete bridges from an adjacent location into place *[Transportation Research Board, 2013]*. The common factor is thus prefabrication of elements or systems in greater or lesser extent, resulting in a reduction of construction time.
This reduction is measured by the use five mobility impact duration classes *[Michael and Culmo, 2011]*:

- Tier (=grade) 1: Traffic hindrance during 1 to 24 hours;
- Tier 2: Traffic hindrance during less than 3 days;
- Tier 3: Traffic hindrance during 1 to 2 weeks;
- Tier 4: Traffic hindrance during less than 3 months;
- Tier 5: Traffic hindrance reduction from years to months.

The application of ABC has become widespread in the U.S. over the last few years, as is also illustrated in chapter 4 where several reference projects have been discussed. Although not defined as ABC, multiple projects in Holland have been executed by making use of these technologies.
ABC methods are not only applicable in bridge replacement projects, but has been increasingly used as design approach for repair works. Here parts of sub- and superstructures have been replaced, or the existing bridge has for instance been widened *[HNTB, 2016]*.

More extensive and detailed information on the application of ABC in the U.S. can be found in *[Khan, 2014]* and *[Transportation Research Board, 2014]*.

6.2. ABC methods

As mentioned in section 6.1, the successful application of ABC is based on removing as much work as possible away from the project site by prefabricating elements or systems in greater or lesser extent. In the following section, attention will be paid to the prefabrication of elements, methods for assembly on site and the quick construction of the substructure. Furthermore, the influence of contracting on the choice for ABC will be discussed.

6.2.1. Prefabricated Elements and Systems (PBES)

Conventional construction methods make use of in-situ poured concrete and connections, for which scaffolding and many construction workers are needed. Since the widespread introduction of prefabricated concrete elements into the construction industry in 1950, its use has increased substantially. In fact, around 2015 80% of all bridges and viaducts in the Netherlands are built with prefabricated elements. Besides the reduction of traffic hindrance, application of prefabricated elements has the following benefits *[van der Veen, 2015]*:

- A minimum disturbance of traffic and people due to the quick erection speed;
- An overall increased construction speed due to simultaneous processes of design and production;
- The quality (both strength and durability properties) of the elements is guaranteed due to the favourable working environment and absence of weather condition influences;
- Standardised cross-sections and formwork reduce the overall project costs due to decrease of scaffolding costs and labour hours in both design and construction.

Disadvantages of prefabrication are however the limited dimensions and weight as a consequence of transportation considerations and the large amount of connections that have to be manufactured on site.

Since a structure is only as strong as its weakest link, the quality of these connections is of utmost importance and often the bottleneck in a prefabricated design. A distinction is generally made between wet connections (poured in-situ) and dry connections (welded or grouted). In recent years, also the use of prefabricated connections has made progress in so-called emulation designs, that are detailed in such a manner that the properties of a conventional construction joint are obtained *[Michael and Culmo, 2011]*. Nevertheless, the manufacturing of connections of prefabricated elements will still require on-site construction works and will thus increase the total required on-site construction time.

In order to reduce the amount of connections that have to be made on-site, use can be made of modular systems, in which multiple prefabricated elements are already connected and transported as one unit to the project site (Figure 6.2). This can be done at the prefabrication plant, but in that case transportation restrictions will be governing for the system dimensions, or at a location near the project site. In the second case, it is possible to construct entire superstructures or even entire bridges off site before transporting them to the project location *[Michael and Culmo, 2011]*.

FIGURE 6.2: Prefabricated modular superstructure element placing *[Roads and Bridges, nd]*

6.2.2. Heavy lifting equipment

Where conventional construction methods and simple prefabricated elements can be erected with the use of (multiple) normal cranes, modular systems or complete superstructures will have to be placed by heavier equipment.

One of those heavy lifting equipment types is the Self-Propelled Modular Transporter, or SPMT in short (Figure 6.3). With a high loading capacity and high manoeuvrability, this transport trailer is extremely useful for the placement of complete superstructures or bridges (Figure 6.4). Multiple trailers can be connected in both transverse and longitudinal direction, thereby increasing the flexibility for different structural compositions. The trailers are self-propelled, thus do not have to be guided by tractors, and they can rotate and move in all three dimensional axes. Depending on the complexity of the structure and the route that has to be covered, the installation with the use of a SPMT can be concluded within 2 to 8 hours *[Michael and Culmo, 2011]*.

FIGURE 6.3: Self-Propelled Modular Transporter *[Mammoet, nd]*

FIGURE 6.4: Superstructure placement by means of a SPMT *[Florida International University, ndd]*

The modular structure can however also be transported to the project location by means of longitudinal launching (Figure 6.5) or transversal sliding (Figure 6.6). Launching is especially suited for bridge placement over terrains that are inaccessible for cranes. The launching pit is situated behind one of the abutments, and the bridge elements are jacked horizontally to the other abutment. This construction method generally takes between several days to several months to complete. With transversal sliding, the new bridge or superstructure is constructed next to the existing bridge and will be slid into place transversely after the old bridge has been removed. Sometimes, the new substructure can already be built underneath the existing bridge to allow the new bridge to be completed immediately *[Michael and Culmo, 2011]*.

FIGURE 6.5: Incremental bridge launching technique *[VSL, nda]*

FIGURE 6.6: Lateral sliding of a bridge superstructure *[Ohio DOT, nd]*

Some other methods of placement can also be distinguished, but are not commonly used and will thus not be treated here.

6.2.3. Rapid foundation and embankment construction

A major part of the on-site activities consists of erecting the substructure, which is mostly a foundation, abutment, approach slab and embankment.

Depending on the pile type needed in the foundation, often multiple installation actions have to be executed, especially when using drilling piles that are accompanied by the removal of soil. If the consecutive actions of drilling, injecting with a support fluid, placing the reinforcement cage and pouring of the concrete, can be replaced by a more simultaneous execution, the required installation time can be substantially reduced *[Michael and Culmo, 2011]*. Another option is to deliberately determine a pile type with low installation time, like a prefabricated driven concrete pile.

A more effective solution is creating a single composite structure that fulfils all functions of foundation, abutment, approach slab and embankment in one. This can be done by applying a Geosynthetic Reinforced Soil-Integrated System (GRS-IBS), that consists of multiple layers of soil separated and reinforced by geosynthetics (Figure 6.7). The obtained composite structure will settle as one unit and thus the need for an approach slab is cancelled *[Michael and Culmo, 2011]*. An advantage of this composite system is the increased lateral and shear strength that allows for steeper slopes to be constructed, thus possibly decreasing the span length. Other benefits are *[Michael and Culmo, 2011]*:

- Low initial and life-cycle costs;
- Fast construction (less than 30 days);
- Minimal installation labour and equipment needed.

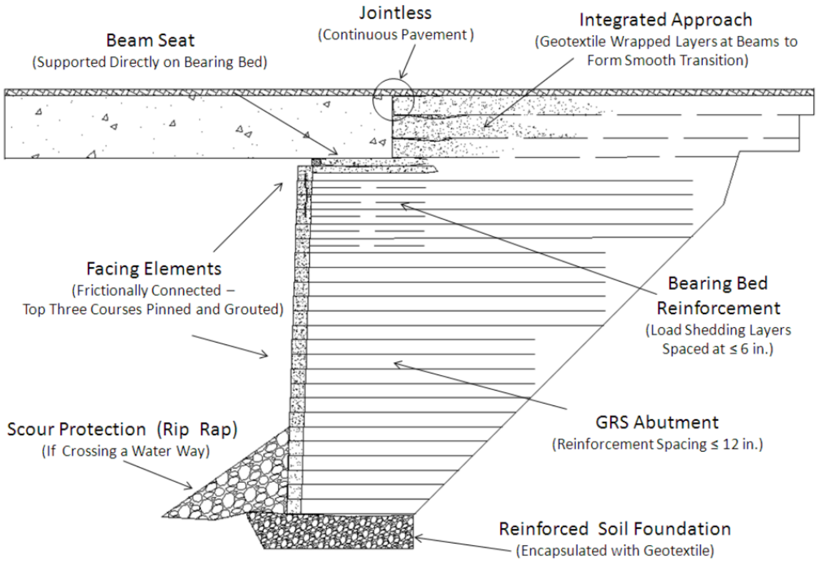


FIGURE 6.7: Geosynthetic Reinforced Soil-Integrated Bridge System (GRS-IBS) *[Michael and Culmo, 2011]*

If the substructure is located on soft subsoils like clay or loam, the expected settlements can reach substantial values. Before the superstructure is constructed, most of these settlements must have taken place to avoid damage or loss of serviceability. This, however is a slow process, especially when dealing with low permeability subsoils like clay and loam. There are several solutions to deal with this time-costly problem, of which one is the application of a low weight embankment fill system consisting of Expanded Polystyrene (EPS) Geofoam. It has no structural function, but due to the weight reduction of up to 100 times, the preload settlement times can be reduced or eliminated. There is also the possibility to apply vertical drainage systems that increase the consolidation of the soft subsoil layers with a resulting time reduction by a factor up to 4. To avoid large settlements of the embankments, use can also be made of a column supports that obtain their bearing capacity from a deeper sand layer *[Michael and Culmo, 2011]*.

6.2.4. Planning and contracting

A successful ABC application does not only depend on the design and construction techniques applied, but also on the planning and contracting of the entire project. *[Michael and Culmo, 2011]* distinguishes two main techniques:

- Accelerated Project Delivery is based on the notion that a fast project delivery not only depends on the quick construction speed, but also on an efficient design process. To accommodate this, designers and contractors should collaborate from the beginning, thus obtaining mutual insights and eliminating problems that would arise in a later stadium in a sequential design process. The overall time reduction in an entire project is especially beneficial when replacement is urgent.
- Contracting provisions are used to compensate the higher initial costs of ABC techniques in order to stimulate this investment. After all, the reduction of on-site construction time and thus the reduction of traffic hindrance will yield economic benefits for the public.

6.3. POTENTIAL APPLICATIONS OF ABC

The bridge replacement task that is imminent in the near future will have to take place in a dense infrastructural network with a high traffic intensity. Over the coming years and decades, the use and density of this network is only expected to increase.
Disruption of this network, as a consequence of construction works, will have a large impact on the traffic flow on the disrupted road itself and on the secondary road network in the vicinity of the disruption. The higher the traffic intensity, the larger the impact on the overall infrastructural network, and thus the impacts of bridge replacement on traffic hindrance will only increase more and more over future years.
Since the economic consequences of traffic hindrance will also increase over the coming years (section 3.3), the need for a bridge replacement strategy with minimal traffic hindrance is evident. In this respect, application of ABC methods in a bridge replacement task seems a promising and profitable alternative for traditional construction works.

When considering the common U.S. methods for ABC as elaborated in section 6.2 and comparing those to the current Dutch practice, it can be seen that ABC certainly has a potential application in Holland. Although some aspects are already commonly applied in Holland, there are other methods that could benefit a rapid bridge replacement project.
The use of prefabricated elements to reduce erection times is already widespread in Holland, as nearly 80% of all bridges and viaducts in 2015 are prefabricated. The use of modular systems on the other hand is not common, probably due to the tight restrictions on transportation of large dimensional units. A possible solution is the erection of modular systems on a construction site nearby the final project location, thus largely eliminating the need for special transportation. In that case, the size of the modular system has somewhat unrestricted limits, meaning that even an entire superstructure or bridge can be erected on this nearby construction site, providing the presence of adequate lifting equipment.
If conventional cranes are not sufficient for the placement of the prefabricated elements or modular systems, the use of SPMTs has the highest potential in Holland. The dense infrastructural network requires the new bridge to be at the same location as the existing bridge. Therefore longitudinal launching is not an option, since the construction space needed coincides with the current bridge route. Transverse sliding could be a potential solution, except that it would have to be built over an existing road or waterway, thus resulting in traffic hindrance for the underlying road. Transportation by a SPMT, however, means that the erection site does not cross a road or waterway and thus traffic hindrance during erection of the modular system will not occur. The question remains, however, if a suitable nearby location for erection can be found, since the infrastructural network is very dense and the SPMT must be able to move between the erection site and the final project site.
Since the subsoil in a large part of Holland consists of soft soil layers like clay and loam, mostly pile foundations are applied. There is a lot of experience on pile foundations, embankment settlements and drainage systems present in the Dutch practice, so it is likely that an appropriate solution can be found for those problems. Reinforced soil, however, is relatively new, especially the application as a composite substructure. It has been applied as retaining structures or an embankment for increased settlements, but up to date no applications in bridge foundations have been reported. Although the spatial structure of a reinforced soil system might conflict with the bridge foundation, it could be used as a method to increase the embankment slope and thus to decrease the total span length of the bridge.
The tight collaboration between designers and contractors throughout the entire project process is already common practice in Holland. Also, the so-called EMVI contracting method (section 3.4) considers traffic hindrance reduction as a criterion, thus rewarding an ABC approach with a discount on the bidding price.
It can be concluded that although many aspects of ABC are already common practice in Holland, several techniques used more frequently in the U.S. can have a high potential in a low-hindrance bridge replacement strategy, like GRS-IBS, modular systems and SPMTs.

The different technologies of ABC, as elaborated in section 6.2, are based on three general strategies that allow optimal use of prefabrication and quick erection *[Transportation Research Board, 2013]*:

1. The bridge elements should be as light as possible:
    - Dimensions have to be manageable for transportation and installation;
    - Low weight simplifies transportation and erection;
    - Existing foundations will experience less loading.
2. The bridge system should be as simple as possible:

    - Use as little girders as possible;
    - Use as little on-site connections as possible;
    - Use as little bracing systems and temporary supports as possible.
3. The bridge system and elements should be easily erectable:
    - Less construction workers needed on site;
    - Less actions needed on site;
    - No falsework is needed on site;
    - Simple geometry increases erection speed.

When coupled with appropriate quality assurance, safety, project management and construction engineering practices, the ABC method can have a large beneficial influence on the total project delivery time and costs *[Michael and Culmo, 2011]*. Other advantages are *[Khan, 2014]*:

- Minimized impact on traffic and surroundings;
- Increased safety for travelling public and construction workers;
- Improved quality and productivity;
- Improved durability and lifetime of structures;
- Reduced environmental impacts;
- Reduced life-cycle costs.

Despite the advantages of ABC, large scale implementation will probably need some years to develop.

The high initial costs, which can reach up to 20 to 30% more than traditional building methods *[HNTB, 2016]*, and the perceived risks makes clients hesitant to apply ABC in bridge construction *[Transportation Research Board, 2013]*. When looking at the total project costs however, reductions in detours, maintenance of traffic and environmental mitigation can result in the ABC approach being the most cost-effective means of construction. On top of that, the prevention of large-scale traffic hindrance will result in substantial economic benefits (section 3.3).
For the contractor this means that by using an ABC approach a large discount on the initial bidding price can be obtained, thus having an advantage compared to other contractors. Furthermore, the reduction of on-site reduction time might result in cost savings from labour hours and equipment renting. A further reduction of contractor costs can be gained over the years by investing in standardised designs and equipment and applying ABC more frequently *[Transportation Research Board, 2013]*.

7

Advanced Cementitious Materials (ACM's)

The goal of this chapter is to gain more insight into the possibilities for slender and lightweight concrete bridge girders. This is done through the scheme given in Figure 7.1.
In section 7.1 a variety of advanced Cementitious Materials is covered, from which can be concluded that UHPFRC has the highest potential in terms of slender and lightweight solutions. Therefore, in section 7.2 this material will be discussed in more detail. However, in order to gain insight into the (enhanced) properties of UHPFRC, first the material NSC will be presented in subsection 7.2.1, after which the same is done for UHPFRC in subsection 7.2.2. Section 7.2 is concluded by a brief comparison of UHPFRC to NSC in mix design, strength, durability, sustainability and production costs.
In section 7.3 a connection is made between the material UHPFRC and the required slenderness ratio for a successful bridge replacement task. Is it possible to remove intermediate supports but maintain an equal slenderness ratio, by applying UHPFRC bridge girders?

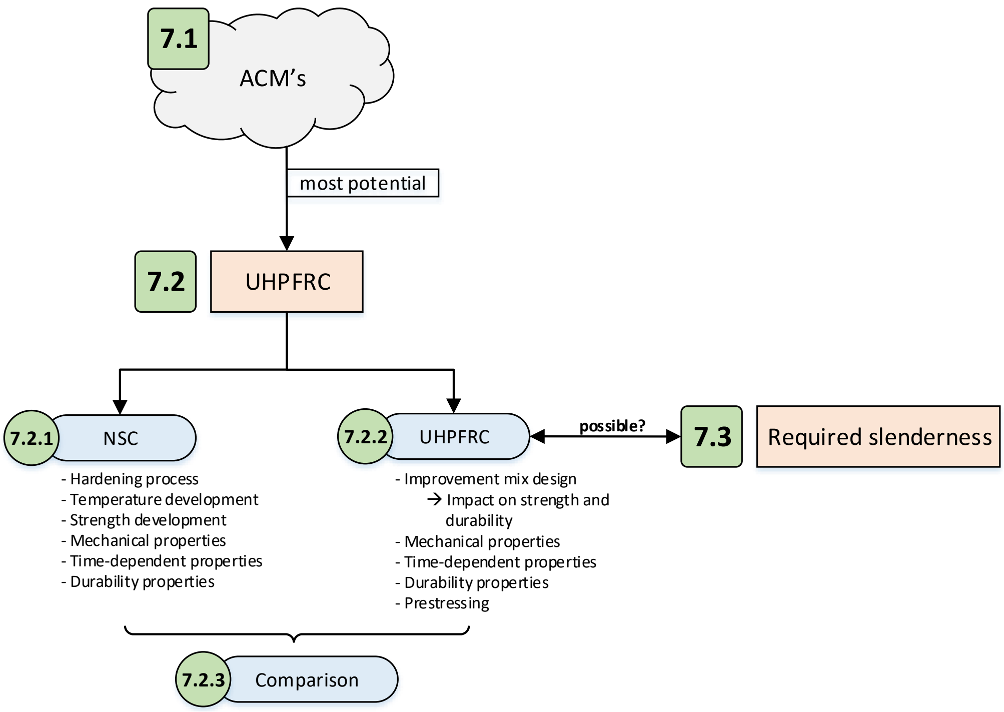


FIGURE 7.1: Schematic representation of chapter 7: Advanced Cementitious Materials (ACM's)

The main outcomes of this chapter are:

- Application of UHPFRC in bridge design can reduce the slenderness of the girder, due to:
  - The high compressive strength allows for a high prestressing force, which increases the moment capacity;
  - The shear capacity of a cross-section is increased by the application of fibres, reducing or even eliminating the amount of shear reinforcement needed.
- The overall weight of a girder in UHPFRC is reduced as the enhanced strength and durability properties result in a cross-section with less material.
- Up to date, a maximum slenderness ratio of $\lambda = 30$ has been applied in practice with UHPFRC. Calculations predict a slenderness ratio of $\lambda = 40 - 50$ to be possible.
- The required slenderness ratio of a three-span bridge that will be replaced by a single span, is $\lambda = 30 - 90$, while for four-span bridges it is $\lambda = 65 - 145$. It can thus be concluded that applying UHPFRC will only be sufficient in some cases for three-span bridges.

7.1. Advanced Cementitious Materials

Advanced Cementitious Material, in short ACM, is a general name for every concrete type that is not defined as Normal Strength Concrete (NSC) in the Eurocode (maximum concrete strength class is C50/60 *[Betonlexicon, nda]*) or contains fibres *[Nederlands Normalisatie-instituut, 2011]*. Overall, there is large amount of ACM's that can be distinguished, each with their own concrete mixture and accompanying advantage in comparison to NSC. The main properties that can be influenced are:

- Ultimate strength development;
- Ductility;
- Weight;
- Durability;
- Sustainability;
- Strength development rate;
- Pouring, compacting and curing;
- Aesthetics;

The scope of this thesis includes the search for prefabricated, slender and lightweight concrete structures. Therefore emphasis is laid on those ACM's that can enhance the ultimate strength and ductility of the material while the overall weight is reduced. Due to prefabrication of the elements the strength development rate, curing and durability can be assumed as sufficient. The aesthetics of the concrete will not be discussed any further since this is a technical feasibility study. In this respect, sustainability is also not a requirement that will be taken into account for the choice of applying a certain ACM. It is however a major disadvantage of using concrete as a building material and will therefore be regarded in the overall performance of the chosen ACM.
With respect to high ultimate strength, high ductility and low weight, the following ACM's are interesting:

- **High Strength Concrete (HSC)** *[Betonlexicon, nda] [VOBN, 2015]*
  HSC is concrete with a higher strength than NSC, that can reach strength classes between C55/67 and C90/105. It has a decreased wcr of 0.30-0.35 and coarse aggregates are replaced by finer quartz to increase the packing density of the material. Besides the higher compressive strength, HSC is therefore also more durable than NSC.
  Designing and constructing in HSC is regulated in Eurocode 2 and follows the same rules as NSC. Current application of HSC is widespread and used in construction of bridges, high-rise buildings and for large spans.

- **Fibre Reinforced Concrete (FRC)** *[Betonlexicon, ndc] [Walraven, 2011]*
  Steel or polymer fibres can be added to the concrete mixture to enhance the consistency and ductility of the material. The fibres regulate crack formation and decrease the crack widths. In certain applications the addition of fibres to the concrete mix will reduce the amount of required reinforcement steel. In order to work sufficiently, however, the fibres should be distributed homogeneously over the material.
  Over the last years, different guidelines have been developed on working with FRC and application is becoming more and more common.

- **Low Weight Aggregate Concrete (LWAC)** *[Betonlexicon, ndb] [Concrete Society, nd] [VOBN, 2016]*
  LWAC makes use of porous aggregates with a low self-weight to reduce the weight of the overall material. Since the relation for LWAC between the characteristic compressive cylindrical strength (f_{ck}) and the characteristic compressive cubic strength ($f_{ck,cube}$) differs from the relation in NSC, a different strength class is used, namely LC. Together with NSC and HSC, LWAC is regulated in Eurocode 2 for strength classes ranging from LC8/9 to LC80/88. The maximum strength of LWAC is restricted due to the relatively low strength of the porous aggregates.
  Due to the porous nature of the lightweight aggregate, its thermal insulting properties are very high and LWAC can therefore be used in housing. The low weight itself is beneficial in foundations or in renovations of existing structures.

- **Strain Hardening Cementitious Concrete (SHCC)** *[van Oosten, 2015]*
  Due to the strain hardening behaviour of fibre reinforced SHCC when loaded in tension, this material is very ductile. The fibres present in the material facilitate a high energy absorption.
  Application of SHCC is especially interesting in constructions prone to earthquake loading and in dynamically loaded joints.

- **Ultra-High Performance Concrete (UHPC)** *[van Oosten, 2015] [Schlangen, 2016]*
  UHPC is concrete with a much higher strength than both NSC and HSC, with a characteristic cubic compressive strength that lies between 150 and 200 MPa. The wcr is further decreased to 0.2 and by addition of very fine aggregate material the packing density is optimized even more. The result is a material with a high compressive strength, a higher shear capacity, a higher stiffness and better durability. Furthermore, the tensile strength is relatively large compared to NSC.
  Due to the high strength and high durability, UHPC is very suitable for the production of slender elements.

- **Ultra-High Performance Fibre-Reinforced Concrete (UHPFRC)** *[van Oosten, 2015]*
  The high strength of UHPC enables the production of very slender cross-section with thin webs and slabs. If conventional reinforcement is needed this minimal thickness can be problematic, however, due to the required concrete cover around the reinforcement steel. A solution is to combine UHPC with FRC to create Ultra-High Performance Fibre-Reinforced Concrete (UHPFRC). The addition of fibres even further increases the strength and durability properties of the material.
  The result is that in general, because nearly all UHPC is fibre-reinforced, the terms UHPC and UHPFRC are used for the same material. The application of UHPC has been growing fast over the last years due to extensive research and the development of codes and regulations in several countries.

For nearly all ACM's it holds that application is scarce due to the lack of knowledge or regulations and due to the high production costs. However, nowadays projects are becoming more complicated as a result of time and space restrictions, durability issues and sustainability demands. Therefore, the advantages that rise from the application of ACM's will, in time, weigh up more and more to the higher production costs and required extensive research.

From the ACM's listed above it follows that, in terms of mechanical optimisation, UHPFRC has the highest strength and durability properties. SHCC is more suitable for the construction of joints and LWAC, while having a low weight, has a maximum strength that lies in the HSC range. Therefore, when looking for a prefabricated, slender and lightweight structure, it can be concluded that UHPFRC has the highest potential.

7.2. ULTRA-HIGH STRENGTH CONCRETE

In order to substantiate the high potential of UHPFRC for prefabricated, slender and lightweight structures, first the general material behaviour of NSC will be discussed. Second, the modifications of UHPFRC with respect to NSC will be elaborated, to explain the enhanced properties of UHPFRC. Finally, a comparison is made between the characteristics of NSC and UHPFRC.

7.2.1. Normal Strength Concrete

NSC (concrete class C12/15 to C50/60) exists of a mix of coarse and fine aggregates (Ø = 0 – 32 mm), cement and water. In general, Portland cement is chosen with a wcr of 0.5 to obtain full hydration *[van Breugel, 2014]*.

The design of concrete structures in NSC is regulated in Eurocode 2 *[Nederlands Normalisatie-instituut, 2011]*, but a brief overview of the material behaviour is given below.

Hardening process *[van Breugel et al., 2013]*
As soon as the cement and water come into contact, a phase-boundary reaction occurs resulting in a thin layer of reaction products that forms at the surface of the cement particles. Initially, the reaction process is retarded quite significantly due to the precipitation of reaction products at the surface of the cement particles. The duration of this dormant stage depends on the presence of accelerators or retarders and ranges from several hours up to more than twenty hours.
At the end of the dormant stage, the reaction process changes into the acceleration period. In a relatively short time, a substantial part of the cement reacts with water. In this stage the layer of hydration products around the cement particles become thicker and the particles come in contact with each other. A microstructure develops, consisting of unhydrated cement particles, hydration products (gel, consisting of a solid material and water filled gel pores) and a (capillary) pore system, partly filled with water. The gel water is physically bound to the surface of the gel particles.
After a certain period, the acceleration stage changes towards the ceasing stage in which the process is diffusion controlled.

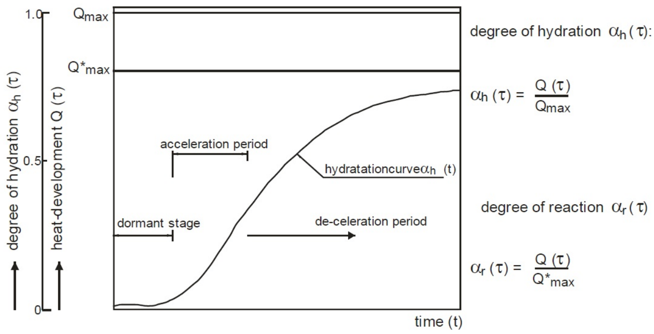


FIGURE 7.2: Hardening process of young concrete *[van Breugel et al., 2013]*

Temperature development *[van Breugel et al., 2013]*
The ratio of the amount of cement converted into reaction products and the originally available amount of cement is called the degree of hydration α_h.
In an adiabatic process (Figure 7.3), the heat development and degree of hydration have a linear relationship, as heat is liberated during the chemical reactions of the cement and water. If, however, the process is not adiabatic and heat can escape from the specimen (Figure 7.4), the heat development stagnates. The larger the volume of the specimen and the faster the hydration rate, the more heat will develop inside the specimen. This also means that under a high temperature, the hydration rate will increase and thus the stress development rate increases. A large temperature rise during hardening will however result in a larger overall temperature drop of the specimen when fully developed, which increases the probability of cracking.

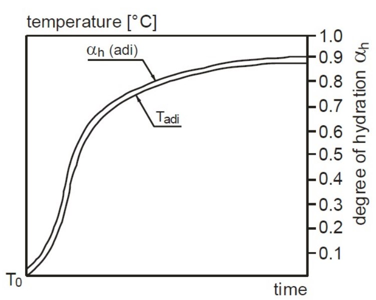


FIGURE 7.3: Temperature development in an adiabatic process *[van Breugel et al., 2013]*

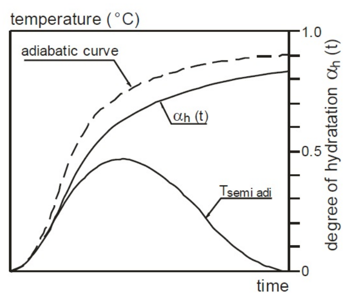


FIGURE 7.4: Temperature development in a semi-adiabatic process *[van Breugel et al., 2013]*

Strength development *[van Breugel et al., 2013]*
At increasing degree of hydration, cement particles are gradually more intensely and firmly connected to each other. Therefore the degree of hydration is an important parameter with respect to the strength development of concrete. After reaching a critical degree of hydration α_0, the strength increases almost linearly with the hardening process. The critical degree of hydration turns out to be a function of the wcr, as for a high wcr the degree of hydration must be higher before the hydrating cement particles are able to bridge the inter-particle distance and strength and stiffness start to develop. For practical purposes this linear relationship is sufficiently accurate, for the exact relation reference is made to Eurocode 2 *[Nederlands Normalisatie-instituut, 2011]*.
The same dependencies hold for the development of tensile strength and the modulus of elasticity.

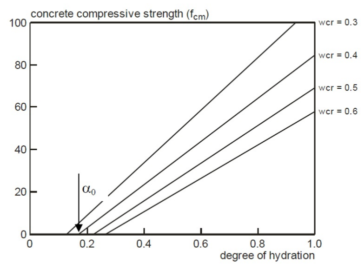


FIGURE 7.5: Compressive strength as a function of α_h and wcr *[van Breugel et al., 2013]*

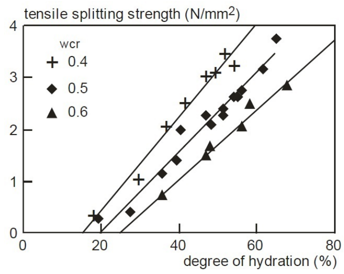


FIGURE 7.6: Tensile splitting strength as a function of α_h and wcr *[van Breugel et al., 2013]*

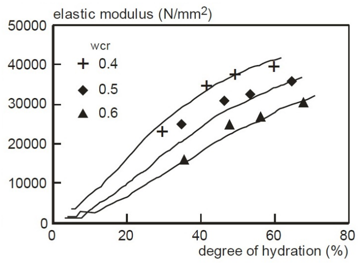


FIGURE 7.7: Modulus of elasticity as a function of α_h and wcr *[van Breugel et al., 2013]*

Mechanical properties *[Nederlands Normalisatie-instituut, 2011]*
Behaviour in compression:

After 28 days, the mechanical properties of the concrete are assumed to be fully developed. The corresponding compressive stress-strain relationship of NSC is given in Figure 7.8. For design considerations the non-linear stress-strain relation is simplified to a bi-linear relation, in which ϵ_{c3} is the strain where the maximum strength is reached and ϵ_{cu3} is the ultimate compressive strain.

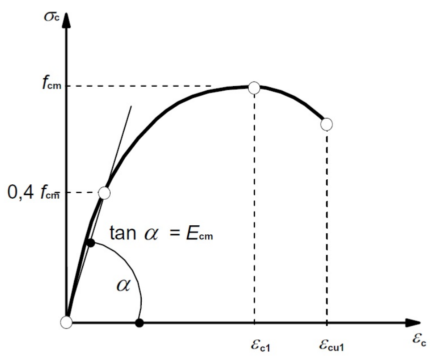


FIGURE 7.8: Theoretical non-linear stress-strain relation for concrete under compression *[Nederlands Normalisatie-instituut, 2011]*

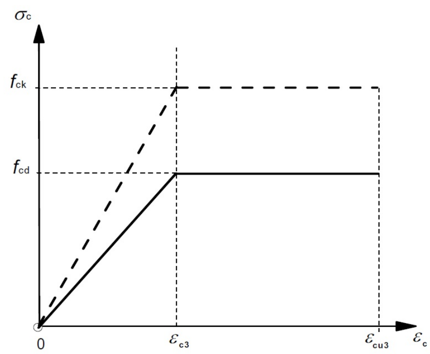


FIGURE 7.9: Simplified bi-linear stress-strain relation for concrete under compression *[Nederlands Normalisatie-instituut, 2011]*

Behaviour in tension:

Due to the small tensile strength, the tensile capacity of NSC is normally neglected in design. In order to be able to resist tensile stresses, NSC usually contains steel reinforcement bars. The accompanying combined stress-strain relationship is given in Figure 7.10. This diagram contains four stages *[van Breugel et al., 2013]*:

- In the first stage, the concrete is uncracked and the steepness of the graph is given by the combined axial stiffness $(EA)_{cs}$.
- In the second stage, the concrete has reached its tensile strength and cracks start to form.

– When the concrete cross-section is fully cracked, the third stage is reached, in which the tensile force is carried by the steel and the concrete in between the cracks (tension stiffening). The axial stiffness of the specimen is now equal to that of the steel only, $E_s A_s$.
– The final stage is when the yield strength of the reinforcement is reached and the specimen can deform freely.

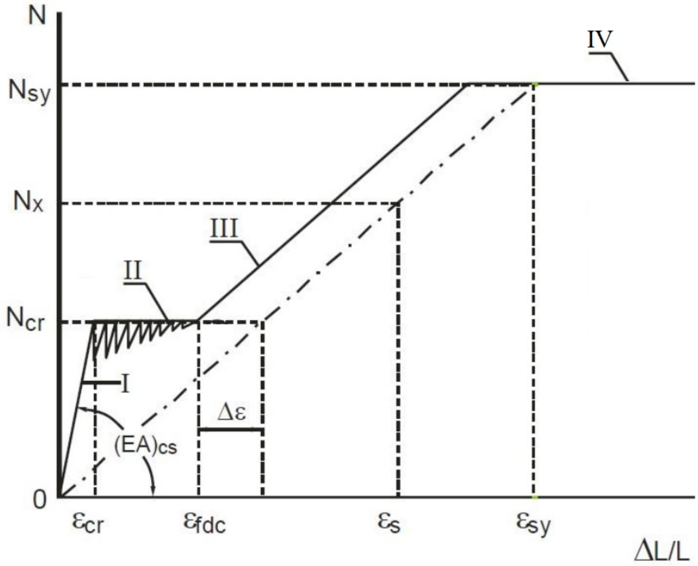


FIGURE 7.10: Stress-strain relationship for NSC under tension *[van Breugel et al., 2013]*

Time-dependent properties *[van Breugel et al., 2013]*
Over time, the concrete behaviour changes due to shrinkage and creep effects, depending on the ambient humidity, the dimensions and the concrete composition. Creep is also largely influenced by the moment, duration and magnitude of loading.
There are four different types of shrinkage, of which the first three stated below will occur during the hardening process:

- Plastic shrinkage
  In freshly cast concrete, the heaviest parts of the mixture tend to move downward under the influence of gravitational forces, which results in the water moving upwards to the surface. Depending on the surrounding conditions (wind, temperature), this water layer evaporates more or less rapid and plastic shrinkage may occur as a result of under-pressure in the fresh concrete.

- Chemical shrinkage
  The volume of the hydration products (the gel) is smaller than the sum of the volumes of the reacting cement and water. This volume reduction is defined as chemical shrinkage and manifests itself mainly as capillary pores in the cement paste and hardly as a deformational change of a concrete element.

- Autogenous shrinkage
  With progress of the hydration process, the water available in the fresh mixture will gradually be consumed by the reaction process, resulting in an 'emptying' or 'self-drying' of the system.

- Drying shrinkage
  The loss of water from the concrete to the environment causes a reduction of the relative humidity in the pore system and thus an increase of capillary forces in the pore water.
  The magnitude of shrinkage mainly depends on the amount of cement paste in the concrete, while the rate depends reversely proportional on the size and dimensions of an element. Fortunately, in general, shrinkage deformation develops very slowly and therefore the development of tensile stresses will develop very slowly as well. Stress relaxation might reduce these tensile stresses substantially, keeping them below the ultimate strain.

Durability properties *[Nederlands Normalisatie-instituut, 2011]*
A structure is considered to be durable if it fulfils its intended function, without significant loss of utility or the need for excessive unforeseen maintenance throughout the structure's lifetime. Different types of degradation mechanisms can be distinguished for the material concrete, namely:

- Mechanical effects;
  Imposed stress or strain from mechanisms such as creep, shrinkage, temperature changes and fatigue;
- Chemical attacks;
  Generally diffusion driven reactions of acids, bases and sulphates with the concrete mixture, leading to leaching of minerals or corrosion of steel components like fibres and reinforcement;
- Physical attacks;
  Examples are abrasion of the concrete surface or freeze/thaw cycles;

The chemical and physical mechanisms that are mostly responsible for the degradation of concrete are assigned to a so-called exposure class for design purposes (see Table 7.1).

Class	Risk
X0	No risk of corrosion or attack
XC	Corrosion induced by carbonation
XD	Corrosion induced by chlorides
XS	Corrosion induced by chlorides from sea water
XF	Freeze/thaw attack
XA	Chemical attack

Table 7.1: Exposure Classes *[Nederlands Normalisatie-instituut, 2011]*

In order to protect concrete against these degradation mechanisms, certain limits have to be considered when designing a structure. These limits apply to the minimum concrete cover and a maximum crack width. Furthermore, transfer mechanisms like penetration and diffusion have to be limited as much as possible by means of a low permeability.

- Concrete cover;
  The concrete cover is the distance between the surface of the reinforcement closest to the nearest concrete surface and the concrete surface itself. A minimum concrete cover c_{min} is prescribed by *[Nederlands Normalisatie-instituut, 2011]* to ensure the safe transmission of bond forces, adequate protection against corrosion and an adequate fire resistance.
- Crack control;
  The crack width is an important parameter when looking at the ease of penetration of water, air and chlorides into the concrete. Therefore, *[Nederlands Normalisatie-instituut, 2011]* has prescribed maximum crack widths w_{max}, depending on the present exposure class and type of reinforcement and prestressing, to ensure adequate durability of the design. The result is a minimum reinforcement ratio in areas that can be subjected to tension.
- Permeability;
  The permeability of a structure is defined by its porosity and the connectivity of the pores. The less permeable the material, the less diffusion and penetration of harmful liquids, gasses and chlorides can take place. The permeability is a result of the applied wcr and fineness of the aggregates.

7.2.2. Ultra-High Performance Fibre-Reinforced Concrete

The improvement of UHPFRC in comparison to NSC is based on four principles *[Paskvalin, 2015]*:

1. The application of a lower wcr;
2. The use of finer aggregate materials;
3. The addition of (steel) fibres;
4. The application of heat treatment for curing.

These measurements, explained in more detail below, result in more homogeneity, a higher packing density and a higher ductility of the concrete *[van der Linden et al., 2008]*.

1. **The application of a lower wcr** *[Paskvalin, 2015]*
   When decreasing the wcr, the amount of cement in the concrete mixture increases while the amount

of added water decreases. In fresh concrete, this results in a smaller inter-particle distance and thus in a quicker hardening process. A denser matrix is the result, since smaller and less pores are formed and also the chemical shrinkage is diminished. Furthermore, the low wcr means that full hydration cannot be obtained. The unhydrated cement will however act as a filler for the capillary pores, thereby improving the density of the matrix even more.

To restore the workability as a result of the low wcr, superplasticizers have to be added to the concrete mixture.

A disadvantage of lowering the wcr is that the autogenous shrinkage will be quite large in UHPFRC since the effect of 'self-drying' is promoted by the lower water content.

2. **The use of finer aggregate materials** *[Paskvalin, 2015][Oostra, 2015]*
Decreasing the maximum aggregate size Dmax results in a higher reaction speed, since a small particle size has a relatively large surface. By subsequently adding more fine particles like sand and silica fume, the packing density can be improved and thus less pores and a denser matrix is obtained. This optimized granular packing also improves the homogeneity of the concrete. For crack formation in UHPFRC, the weakest link is no longer the cement matrix or the interface zone, but the aggregate itself. It is thus necessary to use high-strength aggregates in order to optimize the overall concrete strength.

   As with the application of wcr, also the addition of silica fume will increase the autogenous shrinkage in the early hardening stage.

3. **The addition of (steel) fibres** *[Paskvalin, 2015][Wink, 2015]*
Although the tensile strength of UHPC is higher than that of NSC, the real beneficial tensile behaviour comes from the addition of (steel) fibres.

   Depending on the amount of steel fibres either strain hardening or strain softening occurs, but in both cases the ductility of the concrete has increased substantially. As the fibres are stronger than the concrete matrix, the formation of new small cracks in the concrete costs less energy than cracking through a fibre. This means that a higher fibre content results in smaller cracks. Together with the fact that in between these microcracks the concrete can still transmit forces, the result is that strain hardening occurs if the fibre content is at least 3% of the total mixture volume.

   Applying different fibre lengths is also important, as short fibres bridge microcracks and increase the overall strength, while long fibres bridge the macrocracks and increase the overall ductility.

   When a higher fire resistance is required, Polyvinyl Alcohol (PVA) fibres can be used. These will melt under high temperatures, enabling the heat from the dense concrete to escape and thus prevent spalling.

   In order for fibres to obtain their beneficial properties throughout the entire concrete, they should be evenly distributed and orientated. Furthermore, clusters of long fibres may occur, decreasing the workability of the concrete mixture.

4. **The application of heat treatment for curing** *[Paskvalin, 2015]*
The application of a low wcr and the use of fine aggregates result in a fast hardening process and thus a high temperature development, since the heat cannot leave the concrete in the same rate as it is generated. This increases the total temperature drop of the concrete in the hardening stage and will thus give rise to tensile strains. Due to the cracking that occurs as a result of these tensile strains, the final strength of the concrete will be lower.

   Therefore, during curing, heat treatment is applied, to accelerate the strength development of the concrete and to prevent it from early-age cracking. Another advantage of heat-treatment is that all early and drying shrinkage is accelerated and the concrete volume is stabilized in the early age. The result is a minimal creep and a negligible shrinkage in the long-term behaviour of the concrete.

Applied together, these measurements result in better durability and mechanical properties in comparison to NSC:

- Higher compressive strength - due to a denser matrix and high packing density;
- Higher shear capacity - due to the use of fibres;
- Higher modulus of elasticity - due to the use of fibres and a high packing density;
- Higher tensile strain - due to the use of fibres;
- Earlier strength development - due to the smaller inter-particle distance and heat treatment;

- Smaller crack sizes and more favourable crack distribution - due to the use of fibres;
- Smaller/no capillary porosity - due to a denser matrix and high packing density.

Recommendations for the design and verification of UHPFRC structures have first been set up by the French Association Française de Génie Civil (AFGC) *[Resplendino and Toulemonde, 2013]* and are based on the design method proposed in Eurocode 2 for NSC *[AFGC, 2013]*.

Mechanical properties *[AFGC, 2013]*

Apart from the fact that UHPFRC has a high compressive strength due to the dense matrix and high packing density, a major advantage of applying UHPFRC is that the tensile strength can be taken into account after cracking due to the presence of steel fibres in the concrete. Depending on the amount of steel fibres, either strain-hardening or strain-softening occurs, but in both cases the ductility of the concrete has increased substantially. This tensile behaviour has the advantage that no mild reinforcement or shear reinforcement is needed as UHPFRC has enough capacity to withstand shear and crack formation. Below, more insight will be given in the compressive and tensile behaviour of UHPFRC:

Behaviour in compression:

As in case of NSC, the compression behaviour of UHPFRC in design considerations is simplified to a bi-linear stress-strain relation as given in Figure 7.11 for both SLS and ULS calculations.

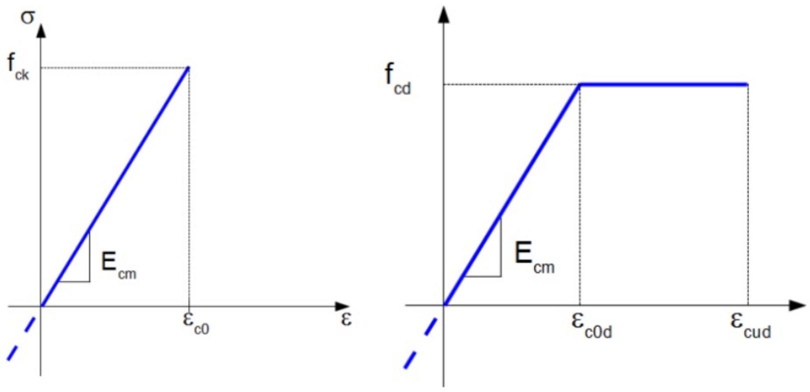


FIGURE 7.11: Compression behaviour of UHPFRC in SLS (left) and ULS (right) *[AFGC, 2013]*

For the determination of stress and strain limits, the following expressions can be used:

$\epsilon_{c0} = \frac{f_{ck}}{E_{cm}}$ $\quad f_{cd} = \alpha_{cc} \cdot \frac{f_{ck}}{\gamma_c}$ $\quad \epsilon_{c0d} = \frac{f_{cd}}{E_{cm}}$

where γ_c is the partial factor for concrete ($\gamma_c = 1.5$ *[Nederlands Normalisatie-instituut, 2011]*) and α_{cc} is a coefficient for long-term effects on the compressive strength. For NSC, it is recommended that $\alpha_{cc} = 1.0$, but for UHPFRC a value of $\alpha_{cc} = 0.85$ is recommended.

The modulus of elasticity is the ratio between stress and strain in the first part of the diagram and can be taken as $E_{cm} = 50$ GPa in preliminary design. To take into account the creep for long-term load, the effective modulus becomes:

$E_{c,eff} = \frac{E_{cm}}{1+\phi}$

For the design limit strain of concrete under compression the following value can be used:

$\epsilon_{cud} = [1 + 14 \cdot \frac{f_{ctfm}}{f_{cm}}] \cdot \epsilon_{c0d}$

in which f_{ctfm} is the maximum mean post-cracking stress in tension and f_{cm} is the maximum mean compressive stress.

Behaviour in tension:

A total of three different types of UHPFRC can be distinguished when looking at the material characteristics in tension:

1. Type I: Strain-softening FRC;
   UHPFRC's of this type show strain-softening behaviour, due to the low fibre content or the inefficient functioning of these fibres.
2. Type II: Low strain-hardening FRC;
   Although this type shows strain-hardening behaviour, when taking into account the K-factor the

corresponding behaviour is actually strain-softening. Therefore this type of UHPFRC will be treated as type I for the material properties and design considerations.

3. Type III: High strain-hardening FRC;
   Even with the K-factor reduction, this type still displays strain-hardening behaviour. This means that after cracking, the concrete in between those cracks have a favourable contribution and the ultimate tensile strength is higher than the cracking strength. This behaviour can only be obtained when a high fibre content is applied.

The tensile behaviour of type I and II is described by a so-called $\sigma_f(w)$ law, in other words a relation between the stress (σ_f) applied and the crack width (w) as a result. Type III, however, is described by a stress-strain relationship (a $\sigma_f(\epsilon)$ law).

Furthermore, for the tensile behaviour of the material, a distinction can be made between thick and thin elements. As mentioned earlier, the orientation of the fibres in the concrete has a strong influence on the tensile capacity of the element. The fibre orientation itself is largely dependent on the flow direction of the fresh concrete and in the vicinity of formwork walls, the flow of concrete is parallel to this formwork. Therefore, since fibres tend to align in the flow direction, all fibres over a small distance from the formwork will be orientated parallel to the surface. This means that the smaller the element thickness is compared to the fibre dimensions, the larger the influence on the tensile capacity will be. The definition for the limit between the two types of structures is given as:

Thick elements: $e > 3 \cdot l_f$
Thin elements: $e \leq 3 \cdot l_f$
where e is the element thickness and l_f is the length of an individual fibre.

To account for the fibre distribution scatter due to placement, an orientation coefficient $1/K$ is taken into account for ULS calculations. A distinction is made between a local (K_{local}) and a global (K_{global}) coefficient depending on the relative area in which good fibre resistance is required to withstand the local or global stresses. An example for K_{local} is the distribution of prestressing stresses and an example for K_{global} is the shear or bending strength of a slab. The following values are a good starting point when designing in UHPFRC:

$K_{local} = 1.75$	for local effects
$K_{global} = 1.25$	for all other loading
$K = 1$	for thin elements (since all fibres can be assumed parallel to the surface)

The ductility of UHPFRC in tension as a result of the addition of (steel) fibres to the concrete mixture, is obtained by the gradual slipping of the fibres in the cement matrix and the fibres not breaking. To guarantee that this slipping behaviour will occur and the required ductility is met, a minimum ductility condition holds for type I and type II materials:

$$\frac{1}{w_{lim}} \cdot \int_0^{w_{lim}} \frac{\sigma(w)}{K_{global}} dw \geq max[0.4 \cdot f_{ctm,el}; 3MPa]$$

in which w_{lim} is equal to 0.3 mm, $f_{ctm,el}$ is the mean elastic limit stress in tension and $\sigma(w)$ is the characteristic post-cracking stress.

1. *Tensile behaviour for thick cross-sections, type I and II:*
   The post-cracking peak is truncated to obtain the design curve for strain-softening or low strain-hardening behaviour (Figure 7.12). This is done by clipping the peak of the $\sigma_f(w)$ law:
   - If a maximum local value is observed, the curve is clipped as shown in Figure 7.13;
   - If no local peak can be observed (Figure 7.14), the curve is clipped at a crack width of $w = 0.3$ mm.

   To obtain a stress-strain relationship from the $\sigma_f(w)$ law, the so-called characteristic length l_c can be used: $\epsilon = \frac{f_{ct,el}}{E_{cm}} + \frac{w}{l_c}$
   in which l_c depends on the cross-section. For a rectangular cross-section it holds: $l_c = \frac{2}{3} \cdot h$.
   The characteristic values can be obtained through:
   $$\epsilon_{pic} = \frac{w_{peak}}{l_c} + \frac{f_{ctk,el}}{E_{c,eff}} \qquad \epsilon_{u,pic} = \frac{w_{peak}}{l_c} + \frac{f_{ctk,el}}{\gamma_{cf} \cdot E_{c,eff}}$$

   where w_{peak} is the crack opening corresponding to the local peak or 0.3 mm if no local peak is present. For the partial safety factor γ_{cf} for fibre-reinforced concrete under tension it holds that $\gamma_{cf} = 1.3$.

$\epsilon_{1\%} = \frac{w_{1\%}}{l_c} + \frac{f_{ctk,el}}{E_{c,eff}} \qquad \epsilon_{u1\%} = \frac{w_{1\%}}{l_c} + \frac{f_{ctk,el}}{\gamma_{cf} \cdot E_{c,eff}}$

where $w_{1\%} = 0.01 \cdot h$, with h the height of the tested prism corresponding to the thickness of the structure.

$\epsilon_{lim} = \epsilon_{u,lim} = \frac{l_f}{4 \cdot l_c}$

where l_f is the length of a fibre.

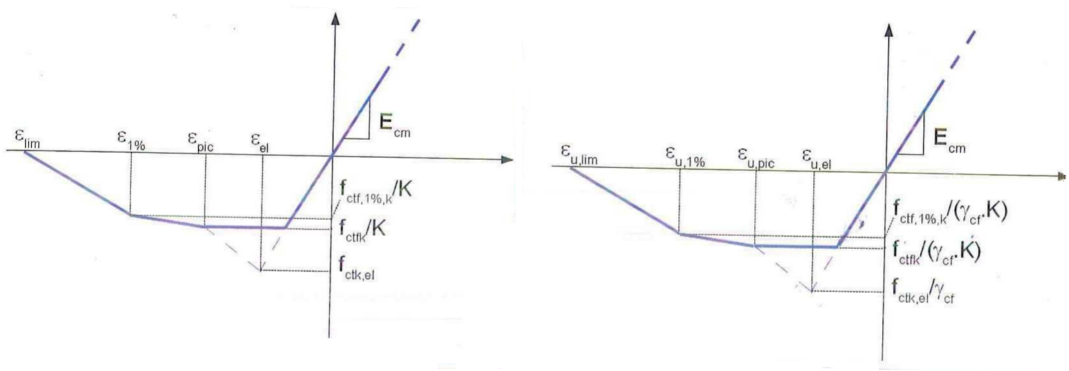


FIGURE 7.12: Strain-softening or low strain hardening law in SLS (left) and ULS (right) *[AFGC, 2013]*

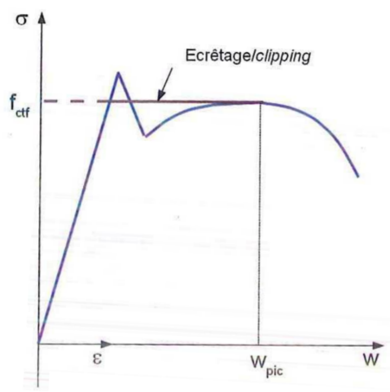


FIGURE 7.13: Clipping of the $\sigma_f w$ curve if a maximum local value is observed *[AFGC, 2013]*

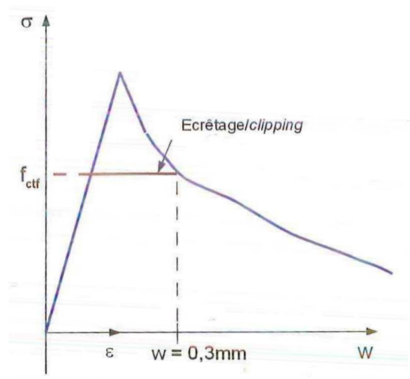


FIGURE 7.14: Clipping of the $\sigma_f w$ curve if no local peak is observed *[AFGC, 2013]*

2. *Tensile behaviour for thick cross-sections, type III:*
   For strain-hardening behaviour in thick sections the limit strain as shown in Figure 7.15 results from characterisation tests of type III UHPFRC, with a maximum of $\epsilon_{lim} = \epsilon_{u,lim} = 2.5‰$.

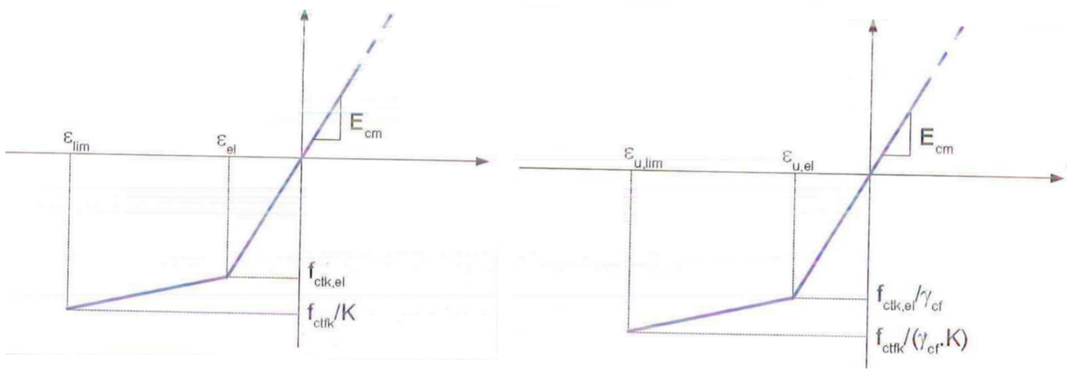


FIGURE 7.15: Strain-hardening law in SLS (left) and ULS (right) *[AFGC, 2013]*

3. *Tensile behaviour for thin cross-sections:*
   For thin plates, with an element thickness $e \leq 3 \cdot l_f$, a simplified and a realistic relationship can be distinguished, see Figures 7.16 and 7.17, as a result of the near 2D fibre orientation.

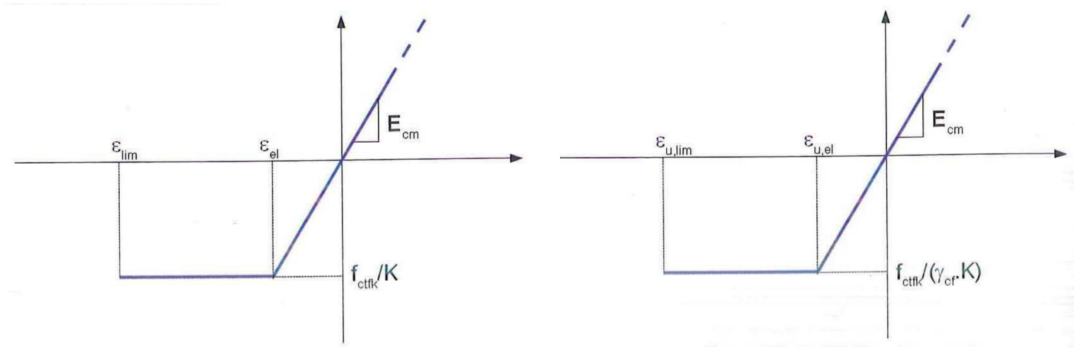


FIGURE 7.16: Simplified law for thin elements in SLS (left) and ULS (right) *[AFGC, 2013]*

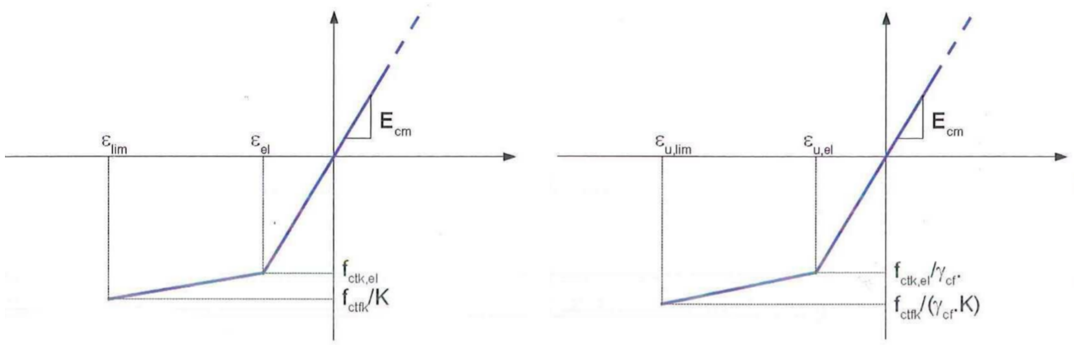


FIGURE 7.17: Realistic law for thin elements in SLS (left) and ULS (right) *[AFGC, 2013]*

- Concretes of type I can theoretically not occur in thin cross-sections, since the minimum ductility condition cannot be met.
- For concretes of type II, a distinction is made between elements that can be subjected to tension and elements in pure bending. In the first case, the same tensile behaviour as in a type II thick element can be expected, while in the second case the simplified behavioural curve from Figure 7.16 must be used.
- For a type III concrete, the simplified behaviour is only valid if the element is subjected to pure bending. In all other cases, as well as in pure bending, the realistic behaviour curve (Figure 7.17) should be used.

Time-dependent properties *[AFGC, 2013]*

As mentioned earlier, the altered concrete mixture of UHPFRC has a large influence on the shrinkage during hardening, especially on the amount of autogenous shrinkage. To reduce or eliminate shrinkage of the UHPFRC, heat treatment can be applied. Two different types of heat treatment can be distinguished: type I is applied during the first hours of hardening at moderate temperature (<65°C) to accelerate the initial hardening and type II is applied after hardening to further increase the mechanical strength and reduce delayed deformations at a high temperature (90°C). The effect on the amount of shrinkage is:

No heat treatment:	550 μm/m of autogenous shrinkage 150 μm/m of drying shrinkage (in an outdoor environment with a relative humidity of RH = 50-70%);
Heat treatment type I:	550 μm/m of total shrinkage (in an outdoor environment with a relative humidity of RH = 50-70%);
Heat treatment type II:	550 μm/m of total shrinkage before the end of heat treatment, after which no further shrinkage will occur during the concrete's lifetime.

The difference between a heat treatment of type II and no applied heat treatment is illustrated in Figure 7.18.

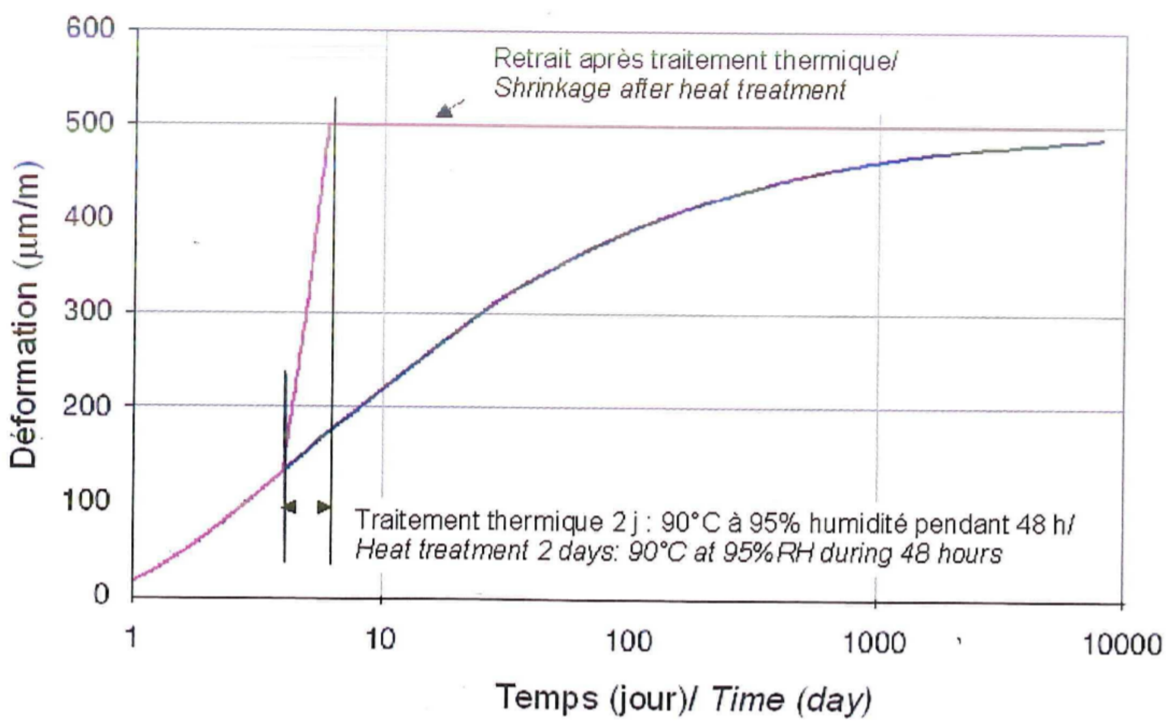


FIGURE 7.18: Influence of heat treatment on the development of shrinkage over time *[AFGC, 2013]*

If no heat treatment is applied, the creep factor of UHPFRC is similar to that of NSC. However, curing under a high temperature has a positive influence on the creep factor, the magnitude depending on the type of heat treatment applied:

No heat treatment:	$\phi = 0.8$
Heat treatment type I:	$\phi = 0.4$
Heat treatment type II:	$\phi = 0.2$

Durability properties *[AFGC, 2013]*
The durability of UHPFRC is captured by the same mechanisms as NSC. There is, however, a large difference in the degree of durability, since UHPFRC is much more resistant to degradation mechanisms than NSC. This is the result of the fine aggregates and low wcr in the concrete mixture, as they instigate a homogenous and compact matrix. This low permeability protects the (steel) fibres, ensuring the protection of the mechanical properties under tension.
Due to these enhanced durability properties UHPFRC is suitable for the manufacturing of very thin components. Although a thin element has a reduced concrete cover, this can be compensated by the increased durability of the material itself.

The minimum concrete cover for UHPFRC is obtained in the same manner as NSC, with the additional requirement following from the placement conditions, $c_{min,p}$:
$c_{min,p} = max[1.5 \cdot l_f; 1.5 \cdot D_{max}; \phi]$
with l_f the fibre length, D_{max} the maximum aggregate size and ϕ the diameter of passive reinforcement.

With respect to the overall permeability of the material, UHPFRC performs much better than NSC. An overview of transfer properties that are indicators of the durability of a material, are given in Table 7.2 as a comparison to the properties of NSC.

Some extra concerns with respect to durability arise from the relative novelty of UHPFRC. These concerns include the potential risk of:

- Possible instability of admixtures present in UHPFRC;
- Possible rehydration of unhydrated cement in hardened concrete;
- Corrosion of steel fibres;
- Chemical attack of PVA fibres;
- Possible delayed ettringite formation (DEF);
- Freeze/thaw cycles;
- Resistance to abrasion;
- Resistance to fire.

Although more research is required in some fields, generally it can be stated that these mechanisms have a small impact on the overall durability of UHPFRC.

Prestressing

To make optimal use of the high compressive strength of UHPFRC, high prestressing forces should be used. Depending on the construction method and transportation possibilities, a choice can be made between pre- or posttensioned prestressing. Pre-tensioning is applied in a prefabricated or in-situ cast structure that is fabricated (and transported if necessary) in one piece. If a segmental construction method is used, however, posttensioning is applied to connect the separate segments together.

Another distinction can be made between internal and external prestressing. In the first case, the strands are cast into the concrete material, transferring their stresses continuously to the concrete through bond forces. In UHPFRC cross-sections, however, this is often not possible due to the small web and flange thicknesses. In that case, external prestressing is a suitable solution, with the forces being transferred to the concrete at anchorage and deviation points.

7.2.3. Comparison of UHPFRC to NSC

A short comparison between the general properties of NSC and UHPFRC, as shown in Table 7.2, shows the advantages of UHPFRC, but also the disadvantages.

It can be seen that in terms of mechanical and durability properties, UHPFRC performs better in every aspect than NSC. It is therefore possible to construct slender structures in UHPFRC that have the same strength capacities as normally designed NSC structures.

On the other hand, the emission of CO_2 and high production costs are a major disadvantage of the application of UHPFRC. The high amount of CO_2 produced is a direct result of the increased cement content in the concrete mixture, while the increased production costs can be assigned to the high amount of energy required and expensive components like silica fume and cement. Nevertheless, due to the increased slenderness that is possible in UHPFRC design, the accompanying material and weight reduction might neutralize these negative effects. After all, material reduction results in a lower amount of CO_2 emitted and a lower total production cost. Added to that the overall weight reduction of the elements can result in an additional cost saving from transportation and placement activities. Additional cost savings arise from the elimination of additional reinforcement, in both material as human labour costs. Also, the excellent durability properties of UHPFRC will result in low maintenance costs over the entire lifetime.

Although the negative characteristics of UHPFRC compared to NSC are diminished by the reduction of material, the expectation is that this is only partly sufficient to acquire a positive life cycle cost analysis. The decisive reason for choosing a structure in UHPFRC will therefore have to come from other conditions than cost savings, for instance the demand for on-site construction time reduction.

This is also the reason that, when looking at the reference projects given in section 4.2, most countries do not implement UHPFRC in bridge construction on a regular basis. The exception to this is Malaysia, where the remote location require light segmental elements for transportation purposes and an excellent durability to diminish the needed maintenance.

Properties			NSC	UHPFRC	Unity
Range f_{ck}			12 to 55	150 to 250	N/mm^2
Mix design					
Concrete class example			C30/37	C170/200	-
Cement content	CEM I		320	710	kg/m^3
Aggregate distribution	Coarse	4-32 mm	1280	0	kg/m^3
	Fine	0-4 mm	640	0	kg/m^3
	Sand	0-2 mm	0	1020	kg/m^3
	Silica fume	< 1 µm	0	230	kg/m^3
Superplasticizer			0	13	kg/m^3
wcr			0.5	0.2	-
(Steel) fibres			0	2	%
Strength properties					
Characteristic compressive strength		f_{ck}	30	170	N/mm^2
Mean axial tensile strength		f_{ctm}	2.9	12.9	N/mm^2
Shear strenght (compared to NSC)		V	1.0	15	-
Modulus of elasticity		E_{cm}	33.000	50.000	N/mm^2
Maximum linear compressive strain		ε_{c3}	1.75	2.3	‰
Ultimate compressive strain		ε_{cu3}	3.5	2.7	‰
Maximum linear tensile strain		ε_{ct1}	0.1	0.14	‰
Maximum tensile hardening strain		$\varepsilon_{lim,hard}$	-	≤2.5	‰
Creep factor	No heat-treatment		1.6	0.8	-
	Heat-treated		-	0.2-0.4	-
Autogenous shrinkage	No heat-treatment		0.8	0.55	‰
Drying shrinkage	No heat-treatment		0.44	0.15	‰
Total shrinkage	Heat-treated		-	0.55	‰
Durability properties *(numbers hold for entire strength range)*					
Water porosity			14 to 20	1.5 to 5	%
Oxygen permeability			10^{-16}	$<10^{-19}$	m^2
Diffusion coefficient chloride ions			$>10^{-11}$	10^{-13}	m^2/s
Corrosion rate passive reinforcement			1.2	<0.01	µm/yr
Electrical resistivity			16	1133	kW·cm
Mass loss from freeze/thaw cycle			3.2	0.005	kg/m^2
Sustainability properties					
CO_2 emmision (compared to NSC)			1.0	2.6	kg/m^3
Costs					
Production cost per unit			100	1000	€/m^3

TABLE 7.2: Property comparison of NSC and UHPFRC *[van Oosten, 2015][AFGC, 2013][Paskvalin, 2015]*

7.3. REQUIRED SLENDERNESS

As stated in section 2.1, three and four span cast in-situ plate bridges are the largest contributor to the bridges built between 1960 and 1980 and therefore they are the most interesting to look at in terms of the bridge replacement task.

Three span bridges have lengths varying from 20 to 65 meters, with the majority between 20-40 m, while four span bridges have lengths varying from 35 to 90 meters with a majority between 40 to 60 m. If the current bridge deck is replaced by a single span with equal deck height, the required slenderness ranges between λ = 30 – 90 for the three span bridges and λ = 65 – 145 for the four span bridges. This values are taken from Figures 7.19 and 7.20.

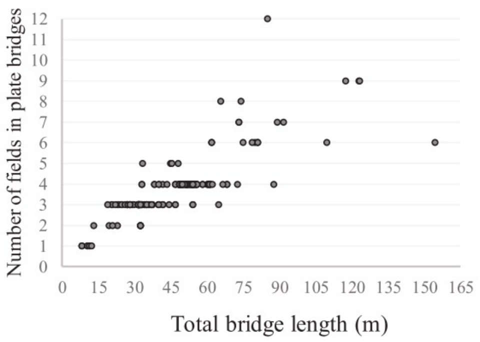


FIGURE 7.19: Relation between the number of spans in a plate bridge and the total length of the bridge *[Reitsema et al., 2015]*

FIGURE 7.20: The span number in a plate bridge related to the required slenderness for a single span *[Reitsema et al., 2015]*

The currently applied state-of-the-art slenderness, as stated in section 4.2, is obtained by prefabricated UHPFRC bridge girders with a slenderness ratio up to $\lambda = 30$. Further research and preliminary calculations have been performed by *[Ngo, 2016]* (Table 7.3) and *[Reitsema et al., 2015]*, which show that a theoretical slenderness of up to $\lambda = 50$ can be achieved.

Length [m]	Height [mm]	Slenderness ratio
60	1300	46.15
70	1600	43.75
80	2050	39.02
85	2200	38.64
90	2450	36.73

TABLE 7.3: Five UHPC bridges with their height and slenderness as designed by *[Ngo, 2016]*

It can therefore be expected that most existing three span bridges can be replaced by a single span bridge with equal slenderness, while most four span bridges will have to maintain at least one intermediate support.

8

Existing foundations

The goal of this chapter is to obtain an overview of possible positive and negative influences over the years on the foundation design of bridges, built between 1960 and 1980. This overview will result in a verdict on whether or not the existing foundation is suitable for reuse in a future bridge replacement task. Also, adequate strengthening methods will be presented in case the existing foundation is not sufficiently strong or stable.

In order to reach this goal, the following information will be presented in this chapter, of which a schematic overview is given in Figure 8.1.

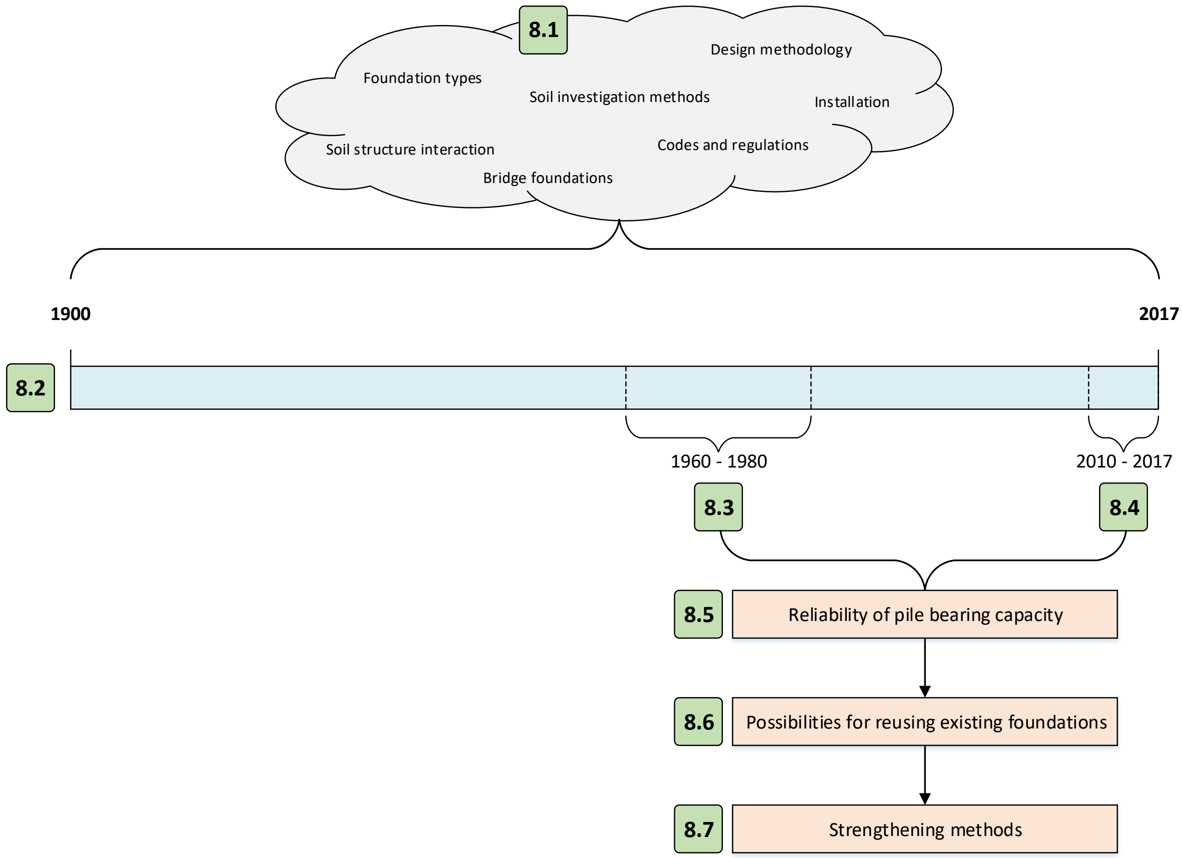


FIGURE 8.1: Schematic representation of chapter 8: Existing foundations

First, in section 8.1, an inventory is given of the aspects of geotechnical design. In order for this bulk of information to be understandable, the inventory is classified according to the generic design steps of a geotechnical design (Figure 8.2). From this inventory, a historic timeline of geotechnical design in Holland will be constructed in section 8.2. With respect to the goal of this chapter stated above, two periods are of interest: in section 8.3 emphasis is laid on foundation design for bridges in the period 1960 to 1980, in which 50% of the current infrastructural bridges were built, while in section 8.4 the same is done for the past decade and the near future. Over the last few years the reliability of the pile bearing capacity has been discussed frequently, therefore a summary of this discussion has been included in section 8.5.

The main question, whether or not bridge foundations from 1960-1980 are suitable for reuse in a near future bridge replacement task, is dealt with in section 8.6. Here, a comparison is made between the positive and negative aspects that will influence the potential reuse of the existing foundation. Finally, in section 8.7 possible strengthening methods are given in case the bearing capacity of the existing foundations are found not be sufficient.

The main outcomes of this chapter are:

- When looking at a bridge replacement task on bridges built between 1960 and 1980, several aspects have a negative influence on the potential reuse of the existing foundations. These aspects are related to:
  - Increase of the load distribution on the end supports;
  - Decrease of the pile tip resistance factor α_p.
- These negative influences might be compensated by smart design solutions:
  - Looking at differences in design methodologies of 1960-1980 and current practice;
  - Make use of a favourable soil structure interaction;
  - Take into account time-effects in soil and material.
- An overview of all influencing aspects is given in Table 8.1. The aspects presented in bold are expected to have the largest influence, both negative and positive.
- More research is needed into the time-effects, especially the set-up of soil, before application is possible.

	Effect
Bearing capacity of pile foundation	
Decrease of α_p by 30%	-
Decrease of overall safety	+
Include positive shaft friction	+
Numerical modelling pile spring constant	+
Densification from group effects	+
Lower uncertainties (γ_M and ξ)	+
Superstructure loading	
Increase of traffic load	-
Increase of overall load due to removal of supports	-
Include negative shaft friction	-
Decrease of self-weight from UHPC	+
Change static scheme of supports	+
Make use of the soil structure interaction:	
Use numerical modelling	+
Allow higher differential settlement	+
Increase stiffness pile cap for larger pile group	+
Allow higher settlements	+
Time-effects	
Include set-up of soil over time	+
Include material time-effects:	
Strength improvement from ongoing hydration	+
Strength decrease from degradation mechanisms	-
Take into account creep, shrinkage and relaxation	+

TABLE 8.1: Potential influences on the pile bearing capacity of existing foundations and the accompanying level of geotechnical analysis

8.1. INVENTORY OF GEOTECHNICAL DESIGN

Although design regulation for geotechnical structures have not been official in Holland till 1992, with the introduction of Bouwbesluit 1992 *[van Tol, 2006]*, the design steps have been very similar over the past century. The generic steps of geotechnical design are presented in Figure 8.2 and this scheme will also be the basis of this inventory of geotechnical design.

It must be stated that the emphasis of this inventory is laid on the Dutch market and the Dutch codes and regulations. The reason for this is not only the limited scope of this thesis, but also the fact that the Dutch design steps, research and rules, especially on pile foundations, have been leading in the international knowledge on geotechnical structures. Some small side notes on foreign practices will however be taken if this has influenced the Dutch practice.

For the most part, this chapter is based on the information given in *[van Tol, 2006]*. Where significant, other references are stated in the text and captions as normal.

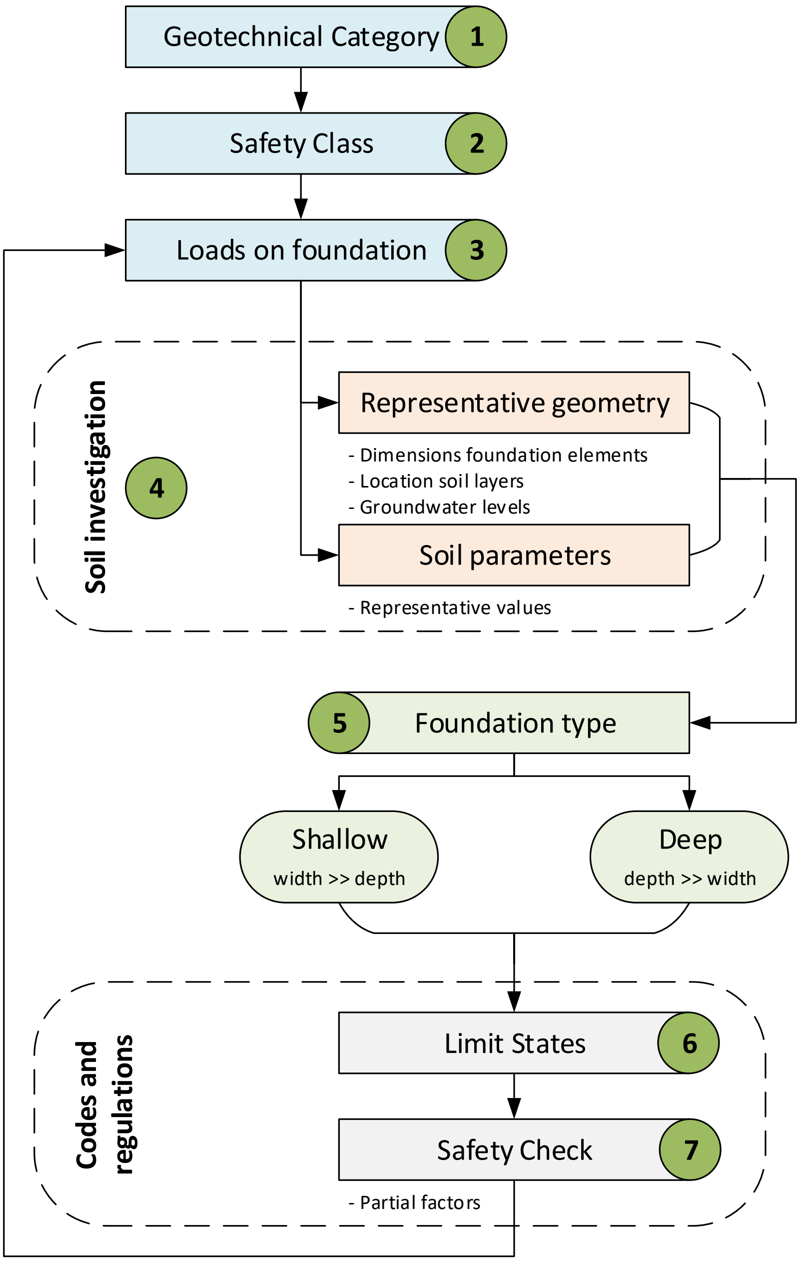


FIGURE 8.2: Schematic representation of section 8.1: Inventory of geotechnical design

8.1.1. Geotechnical Categories

Geotechnical Categories are a classification system that indicates the complexity of a geotechnical structure. This prevents unnecessary detailed calculations for relatively simple structures. The distinction is made based on the foundation type, the foundation dimensions, the load from the superstructure, the soil conditions, the groundwater level and possible additional requirements. The classification is given below:

- Category 1:
  Simple structures with a low maximum load, that can be designed based on local experience and a qualitative soil investigation, in which the soil layers are presented and the homogeneity of these soil layers is proved.
- Category 2:
  Almost all structures in Holland (about 90%) belong to this category, like for instance all spread footing foundations, raft foundations and pile foundations. The safety of the structure must be demonstrated by calculations in which the loads and soil conditions are incorporated.
- Category 3:
  This category contains all special structures that not only have to comply with the safety requirements given in the codes (Category 2), but also with additional specific requirements. Examples are dynamically loaded structures, off-shore structures or large tunnels and bridges.

The classification of a geotechnical structure should take place before the soil investigation, but must also be affirmed after the soil investigation and during construction.

8.1.2. Safety Classes

Since the introduction of an official geotechnical design code in 1992 (TGB 1990, introduced by Bouwbesluit 1992), the safety of geotechnical structures is based on probabilistic analyses. The required reliability of structures is the result of extensive risk analyses on the consequences of failure and the chance of this failure occurring. Although the reliability differs for every structure, this study has led to a classification of all structures into three Safety Classes with corresponding allowable chance of failure (Table 8.2). Geotechnical structures like embankments or sheet pilings can often be sorted into Safety Class 1, while infrastructural works like large bridges are generally gathered in Safety Class 3. With the introduction of the Eurocodes by Bouwbesluit 2012, high-rise buildings have been included in this classification system and this has instigated a couple of changes in the Safety Classes and the accompanying reliability indexes (Table 8.3) *[Nelemans, 2011]*.

Safety Class	Consequence of failure		Reliability index (β)			Failure probability (P_f)		
	Risk of fatal injury	Risk of economic damage	Ultimate limit state		Serviceability limit state	Ultimate limit state		Serviceability limit state
			If wind is normative	Otherwise		If wind is normative	Otherwise	
1	Negligible	Small	2.3	3.2	1.8	$1.07 \cdot 10^{-2}$	$6.87 \cdot 10^{-4}$	$3.59 \cdot 10^{-2}$
2	Small	Large	2.4	3.4	1.8	$8.20 \cdot 10^{-3}$	$3.37 \cdot 10^{-4}$	$3.59 \cdot 10^{-2}$
3	Large	Large	2.6	3.6	1.8	$4.66 \cdot 10^{-3}$	$1.59 \cdot 10^{-4}$	$3.59 \cdot 10^{-2}$

TABLE 8.2: Safety Classes and accompanying reliability index and failure probability *[van Tol, 2006]*

TGB 1990			Eurocodes			
Class	β	P_f	Class	β	P_f	Application
1	3.2	$6.87 \cdot 10^{-4}$	RC1	3.3	$4.83 \cdot 10^{-4}$	Living houses
2	3.4	$3.37 \cdot 10^{-4}$				Buildings < 3 floors
3	3.6	$1.59 \cdot 10^{-4}$	RC2	3.8	$7.23 \cdot 10^{-5}$	Office buildings, flats
			RC3	4.3	$8.54 \cdot 10^{-6}$	Highrise > 70 m

TABLE 8.3: Comparison between Safety Classes in TGB 1990 and the Eurocodes *[Nelemans, 2011]*

Since both the load on a foundation and the capacity of a foundation are not a deterministic value but in fact normally distributed, the required reliability must be demonstrated through probabilistic analyses. This will be elaborated further in subsection 8.1.7.

8.1.3. Load actions

The loads that act on the foundation must be determined before the appropriate foundation type can be established. The interaction between soil and structure, through a decent foundation, is very complex due to various mechanisms. Especially the stiffness differences in soil, structure and foundation elements cause difficulties in calculation, since stiffer elements attract more load and this will influence the amount of settlement, depending on the foundation type. More insight into the interaction between soil and structure will be given in subsection 8.1.5 for the different foundation types.
Because of this interaction between soil and structure, the load distribution cannot be determined exactly beforehand, since it is dependent on the foundation type and stiffness. One should, however, have an idea about the magnitude and direction of the load from the superstructure on the foundation. The exact distribution of stiffnesses and loads will have to be checked in every design loop.

8.1.4. Soil investigation

An adequate soil investigation is critical in the design of a geotechnical structure. The goal is to determine the location of different soil layers and their respective conditions and characteristics. This is done by a combination of fieldwork and laboratory tests. These soil conditions determine the required foundation type, together with the dimensions of the foundation and the expected settlement.
Before starting a soil investigation, one should determine which information is required. In general, this is the location and differentiation of the load bearing soil layers in the project site. However, since most investigation methods only give a one-dimensional representation of the soil structure over a certain depth, an infinite amount of investigation locations are possible. To reduce time and costs to a minimum, the following aspects must be assessed beforehand for a smart and adequate solution:

- The scope of the soil investigation;
- The amount of locations investigated;
- The distance between two adjacent locations;
- The method used for soil investigation;
- The depth to which the soil on each location is investigated;
- The location and differentiation of load bearing soil layers.

These aspects are all largely dependent on the soil layer structure and especially on the homogeneity of the soil. The design of the foundation thus depends on the soil investigation, but the extent of the soil investigation in turn depends on the design of the foundation. It is therefore not a straightforward design scheme, but rather a loop in which assumptions have to be made, checked and adapted until a satisfying design is obtained.

A general starting point for a soil investigation is thus to obtain a global idea of the soil conditions on site. Often, this information is available from previous projects at the same location, or from locations nearby. The information obtained from geological maps or from local experience also gives an idea about the different soil layers and the depth of the load bearing soil layer.
From this preliminary investigation, the degree of homogeneity of soil conditions has to be roughly determined in order to establish the type of soil investigation and the amount of locations investigated. Whether an area can be considered as homogeneous or whether it should be divided in smaller areas is however one of the most difficult steps in design, since a lot of experience is involved *[van Tol, 1994]*. The degree of homogeneity should therefore be checked and adapted, if necessary, after each soil investigation in a design loop.

If, from preliminary investigation, it turns out that the soil can be considered as homogeneous and the structure can be classified as Geotechnical Category 1, the determination of characteristic values for soil parameters can be based on standard tables. These values are however conservative, since it must lead to safe structure under every circumstance, and thus additional soil investigation is often more economic than applying these values.
For all other Geotechnical Categories, additional soil investigation must be performed to determine the representative values of the soil parameters. This soil investigation can consist of the following methods:

- **Cone Penetration Test (CPT)** *[van Tol, 2006] [Verruijt, 2010]*
  The CPT is the most-used soil investigation technique in the Netherlands since its introduction in the thirties. It is a simple and effective method that measures the force required to push a rod vertically into the ground as a function of its depth. The total resistance consists of a cone resistance and shaft

friction, that can be determined through a CPT and are a direct indication of the load bearing capacity of the different layers. Based on the CPT results, a visual interpretation of the different soil layers and their parameters can be obtained, as illustrated in Figure 8.3 and Table 8.4.

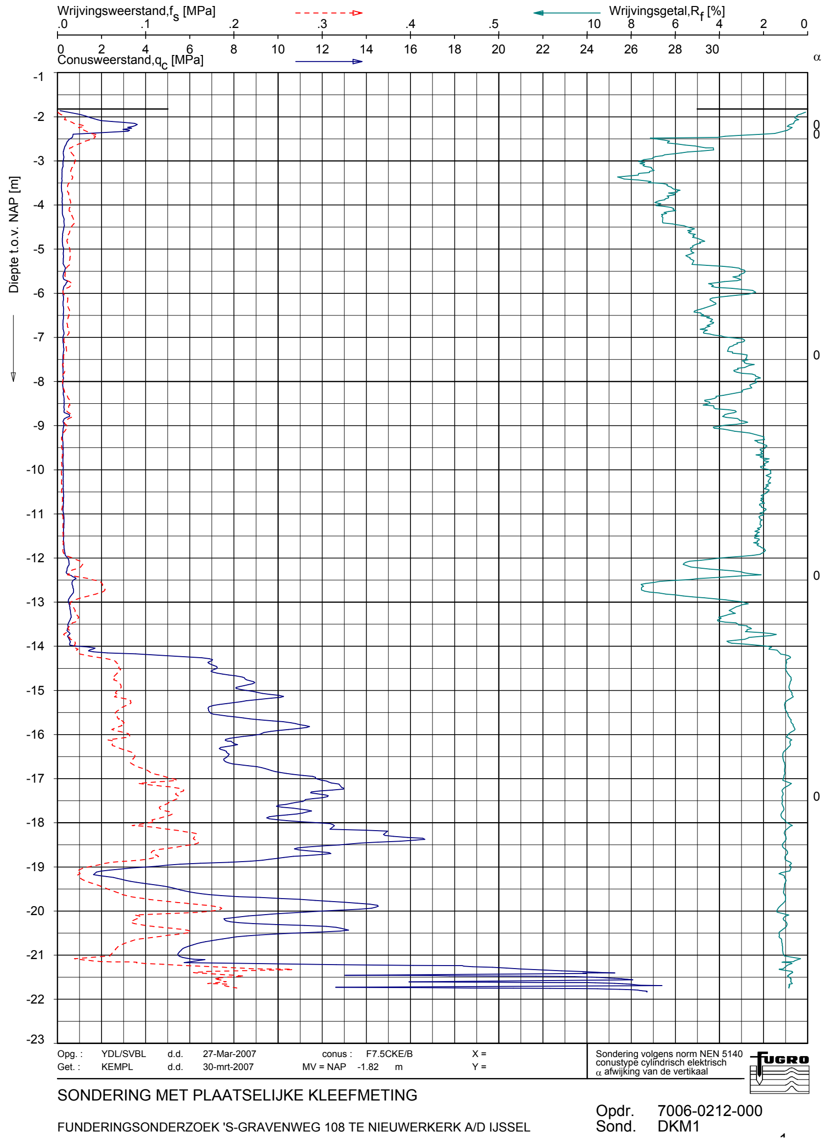


FIGURE 8.3: Example of the results from a CPT near Nieuwerkerk aan de IJssel *[Fugro, 2007]*

Soil type	Cone resistance	Friction ratio
	q_c [kN/m^2]	f_s/q_c [%]
Sand	10	1
Sandy clay	5	2
Clay	1	3-5
Loam	< 1	8-10

TABLE 8.4: General values for cone resistance and friction ratio for different soil types *[van Tol, 2006]*

– Cone resistance (q_c)
The cone resistance as measured from a CPT can be used in both a qualitative and quantitative way, for the determination of the different soil layers and the calculation of bearing capacity respectively. For non-cohesive soils, the cone resistance mainly depends on the shear strength of

the grain skeleton and the crushing strength of individual grains. This in turn depends on the vertical and horizontal effective stress, the density, the particle distribution and dimensions and the crushing strength of the grain material. For cohesive soils, the cone resistance primarily depends on the shear strength of the grain skeleton. The modulus of elasticity of the grain skeleton and individual grains will also have a, slightly smaller, effect on the cone resistance of both cohesive and non-cohesive soils.
The empirical relation between the measured cone resistance and calculated friction properties are based on the classical theories by Prandtl and Brinch Hansen for a strip foundation *[Everts, 2015]*:
$q_c = c \cdot N_c + p_0 \cdot N_q + 0.5 \cdot \gamma \cdot B \cdot N_\gamma$
with q_c the cone resistance [kN/m^2], c the cohesion, p_0 the stress (mostly taken as σ'_v, the vertical stress) and N_c, N_q and N_γ are bearing capacity factors depending on the angle of internal friction ϕ (Table 8.5)

ϕ	N_c	N_q	N_γ
0	5.142	1.000	0.000
1	5.379	1.094	0.003
2	5.632	1.197	0.014
3	5.900	1.309	0.032
4	6.185	1.433	0.060
5	6.489	1.568	0.099
6	6.813	1.716	0.151
7	7.158	1.879	0.216
8	7.527	2.058	0.297
9	7.922	2.255	0.397
10	8.345	2.471	0.519
11	8.798	2.710	0.665
12	9.285	2.974	0.839
13	9.807	3.264	1.045
14	10.370	3.586	1.289
15	10.977	3.941	1.576
16	11.631	4.335	1.913
17	12.338	4.772	2.307
18	13.104	5.258	2.767
19	13.934	5.798	3.304
20	14.835	6.399	3.930

ϕ	N_c	N_q	N_γ
20	14.835	6.399	3.930
21	15.815	7.071	4.661
22	16.833	7.821	5.512
23	18.049	8.661	6.504
24	19.324	9.603	7.661
25	20.721	10.662	9.011
26	22.254	11.854	10.558
27	23.942	13.199	12.432
28	25.803	14.720	14.590
29	27.860	16.443	17.121
30	30.140	18.401	20.093
31	32.671	20.631	23.591
32	35.490	23.177	27.715
33	38.638	26.092	32.590
34	42.164	29.440	38.366
35	46.124	33.296	45.228
36	50.586	37.753	53.404
37	55.630	42.920	63.178
38	61.352	48.933	74.899
39	67.867	55.957	89.007
40	75.313	64.195	106.054

TABLE 8.5: Values of bearing capacity factors N_c, N_q and N_γ as a function of the angle of internal friction ϕ *[Verruijt, 2010]*

The last component is negligible in pile foundations due to the small width B. This indicates that the maximum bearing capacity for a relatively small cone is more or less equal to that of a pile foundations and thus it can be concluded that the cone resistance is a measure for the maximum bearing capacity.

– Local friction (f_s)
The local friction is mainly dependent on the soil type with an inversely proportional relation to the particle size, as derived by Begemann. Together with the cone resistance, the local friction can be used to classify the soil type (Figures 8.4 and 8.5). Often, the local friction is depicted as a friction ratio, being the ratio between the cone resistance and the local friction (q_c/f_s).

Over the years, multiple CPT techniques have been developed, with variations in cone smoothness, friction jacket and driving. Due to these variations, the empirical relationships between CPT result and bearing capacity are also different and thus caution must be taken when considering those CPT results. In the early years, the cones were driven into the ground mechanically (Figure 8.6), with a system consisting of an inner and an outer rod to separate the information on cone tip resistance and shaft friction. With the introduction of an electrical CPT (Figure 8.7) the application of CPTs in soil investigation has increased even further, as not only the cone resistance is measured, but also the local friction, the hydraulic pressure (see CPTu) and the slope. These properties are measured through Gauge-strips situated directly behind the cone tip, the signal is transferred upwards through the rod. Since the eighties, a consensus has formed on the CPT technique to be used, which is an electrical driven smooth cone tip.

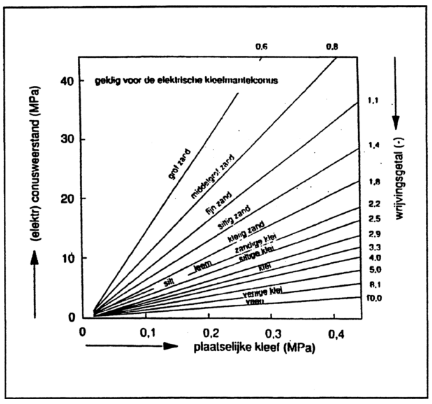


FIGURE 8.4: Soil classification from an electrical CPT *[van Tol, 2006]*

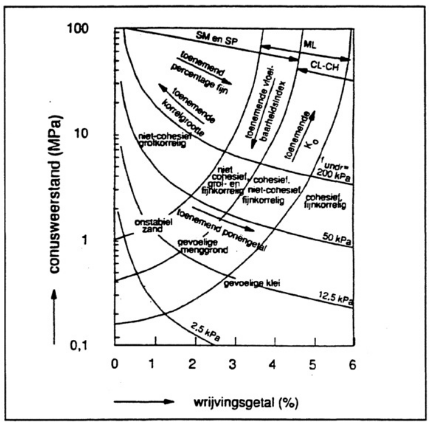


FIGURE 8.5: Soil classification from an electrical CPT *[van Tol, 2006]*

The popularity of the CPT as a soil investigation method in Holland is explained by the soft subsoil layers that are present over a large area of this country. In areas with rocks or grit present in the subsoil, the CPT may lose its value due to jamming of the cone or the disturbing effect of individual large grits on the soil behaviour. Important focus points in this respect are the maximum grain diameter (D_{max} < 10-20 mm) and the strength of individual grains, since crushing of a grain decreases the cone resistance. Other advantages of the CPT method are:

- It generates cheap and quick results, with a visual interpretation of the soil;
- Continuous information over the entire depth allows the distinction of small layers with divergent properties;
- The results are independent of human influences, although they are sensitive to errors in the equipment;
- It is especially suitable for the determination of soil parameters from incoherent layers that cannot be sampled without being disturbed.

- **Piezo Cone Penetration Test (CPTu)** *[van Tol, 2006] [Verruijt, 2010]*
  When using a CPTu instead of a CPT, together with the cone resistance and shaft friction, also the hydraulic pressure is measured. This increases the possibilities to determine the soil layers, especially thin cohesive layers. A sudden increase of hydraulic pressure indicates undrained behaviour and thus a low permeability layer. It can however also indicate the appearance of dilatancy (the appearance of an under pressure due to increasing volume of the grain skeleton) and thus the packing density of sand layers.

  The hydraulic pressure is measured through a filter behind tiny holes in the cone. In Dutch practice, the hydraulic pressure index B' is an indicative value for the classification of soil layers: B' = $\Delta u / q_c$, where Δu is the hydraulic pressure difference.

  Application of CPTu has a number of disadvantages, of which the lack of international standardisation and the difficult saturation of the filters are the main drawbacks.

- **Vane test** *[Verruijt, 2010]*
  The vane test can be used to determine the shear capacity of the soil, see Figure 8.8. The vane is inserted into the ground and through rotation of the vane, the soil at the surface of the cylindrical soil volume around the vane will be loaded until failure. The applied moment is a measure for the shear capacity through empirical relations. This type of soil investigation is mainly applied in Scandinavian countries where they have large clay layers.

- **Standard Penetration Test (SPT)** *[van Tol, 2006] [Verruijt, 2010]*
  The SPT is mainly used in Anglo-Saxon countries (United Kingdom, USA, Canada and New-Zealand). A sample tube is driven into a borehole by a standardised weight and the amount of strokes (blow count N) needed to obtain a 1 foot (300 mm) displacement is measured. Although this method is easier applicable than CPT due to the smaller equipment, and at the same time a soil sample can be retrieved, the

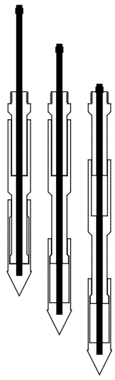


FIGURE 8.6: Mechanical CPT *[Verruijt, 2010]*

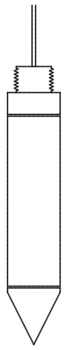


FIGURE 8.7: SElectrical CPT *[Verruijt, 2010]*

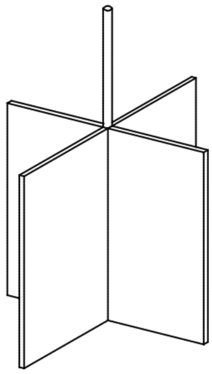


FIGURE 8.8: Vane test *[Verruijt, 2010]*

method is rather inaccurate and not reproducible. Another drawback is the relative difficult distinction between a sand and clay layer. A commonly used interpretation of blow counts and soil classification is given in Table 8.6.

Sand		Clay	
N	Density	N	Consistency
< 4	Very loose	< 2	Very soft
4-10	Loose	2-4	Soft
10-30	Normal	4-8	Normal
30-50	Dense	8-15	Stiff
> 50	Very dense	15-30	Very stiff
		> 30	Hard

TABLE 8.6: Soil classification based on blow counts *[Verruijt, 2010]*

- **Drilling and sampling***[van Tol, 2006] [Verruijt, 2010]*
  The drilling and sampling of soil is the only method so far that can directly determinate the composition and parameters of the soil. The samples, obtained from boreholes, can be analysed and tested in the lab, in a visual, chemical and mechanical way.

  Several methods for drilling and sampling are available, of which the most simple method is the driving or pushing of a 1 meter long tube into the ground and retrieving the soil within the tube. The main advantage is that no heavy equipment is needed for this type of soil investigation. For deeper samples, the borehole is supported and deepened by a special boring tube, while the sampling tube itself has a smaller diameter and fits into this boring tube.

  Independent of the method used, it is impossible to attain entirely undisturbed samples from boreholes, in which the stress state from hydraulic pressure and grain stresses is exactly the same as in the ground. Instead, the water pressure is reduced to zero, or even negative in cohesive soils, when the sample is taken from the borehole. Also, shear stresses occur at the surface of the drilling hole from inserting the drill tube into the ground. The less cohesive the soil in the sample, the higher the disturbance. All these aspects diminish the accuracy of the samples and thus the reliability of all obtained values also decreases. Since the disturbance effects are dependent on the type of drill technique, the different techniques are classified into several categories, Class 1 to 5. Class 1 is for those samples that are more or less undisturbed, Class 5 is for completely disturbed samples. The classification depends on the drill type used, the soil condition, the pipe thickness and length and the method of inserting the drill pipe into the ground. The required quality of sampling is governed by the property that is to be determined from this sample, see also Table 8.7.

  A good sample method is the Begemann sampler (Figure 8.9), which consists of two steel tubes that are pushed simultaneously into the ground. To maintain the integrity of the sample, a nylon stocking that is initially rolled up on the inner tube is gradually stripped during the downwards movement until the

Goal	Class
Stiffness properties	1
Srength properties	1
Structure, layering	5 - 1
Composition	4 - 1
Permeability	2 - 1

TABLE 8.7: Required drill class for the determination of several parameters *[van Tol, 2006]*

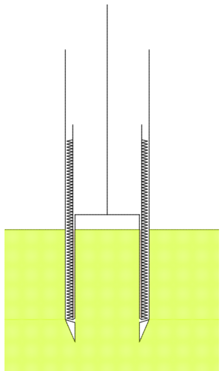


FIGURE 8.9: Begemann sampler *[Verruijt, 2010]*

sample is entirely enclosed. The sample, that can be up to 20 meters long, is supported by a heavy fluid that simulates the original lateral soil support. By making use of the Begemann sampler, a disturbance Class 1 or 2 can be reached, depending on the cohesiveness of the sampled soil.

Samples can also be taken from the bottom of the sea, which is especially interesting for offshore structures. Although it is a costly operation, both cone resistance and shaft friction can be obtained as a numerical function of the depth and therefore this soil investigation is very valuable.

Of all these soil investigation methods presented above, CPT has a number of clear advantages:

- The execution costs of a CPT are 10 to 15 times cheaper than in the case of a drilled sample. It is therefore common practice in Holland to start the soil investigation with a number of CPTs and based on the acquired results additional boring is possible at optimal locations.
- Practice has proven that in a lot of geotechnical investigations, the use of only CPTs gives an adequate representation of soil composition, homogeneity and bearing conditions.
- Empirical relations can be used to derive the cone resistance and shaft friction from the obtained results.
- The negative friction can be derived from the layer composition as a function of cone resistance and friction ratio.
- In the case of a pile foundation, the settlement of the pile tip can be determined as a function of the cone resistance, through a standardised load-displacement diagram from multiple proof loaded piles. The settlement beneath the pile tip or beneath a pile group can also be determined from the results of a CPT, through the modulus of elasticity that is derived from the cone resistance.
- In the case of other geotechnical structures, additional bored sampling is often required to execute laboratory tests on the cohesive layers. The added costs can generally be compensated by a more efficient design that follows from more economic values of the soil parameters.

An overview of advantages, disadvantages and applications for different soil investigation methods is given in Table 8.8. Especially the wide range of CPT applications is clearly illustrated.

Investigation method	Cone Penetration Test (CPT)			Drilling and sampling		Laboratory research		
Geotechnical structure	Cone	Friction	Piezo	Disturbed	Undisturbed	Classification	Strength	Stiffness
Foundation								
Piled	++	+						
Spread footing	++	++						
Construction pit								
Retaining wall	++	++			++	++	++	++
Drainage	+	+	+	+				
Infill								
Settlement	++	++			++	++		++
Stability	++	++			++	++	++	

TABLE 8.8: Overview of the application fields of different soil investigation methods *[van Tol, 2006]*

8.1.5. Foundation type

After execution of an adequate soil investigation, the soil composition and bearing conditions can be used for the foundation design. Two main distinctions can be made in foundation type, namely a shallow foundation, in which the bearing capacity is provided by the subsoil layers directly beneath ground level, or a deep foundation, in which the superstructure loads have to be directed through the soft subsoil layers to a deeper bearing layer.

Shallow foundation

If a relatively light superstructure is built on a site with relatively stiff subsoil layers, it might be possible to construct a shallow foundation. In general, this consists of a spread footing placed on or just beneath ground level that spreads the load from the superstructure over a larger surface. This increases both the overall bearing capacity as the overall stability of the soil.
The settlement and support reaction of the soil underneath the spread footing depends on the relative stiffness of the footing and the superstructure, see Figure 8.10. In a flexible footing the largest settlement will occur at the centre while the foundation pressure in the soil is evenly distributed over the entire surface. Because of the large settlement possible, the moment acting in the footing will stay limited. In a stiff footing, however, the settlement will be equal over the entire surface but the foundation pressure will be higher at the edges. Since the stiff footing is not able to deform, the maximum moment can reach high values.

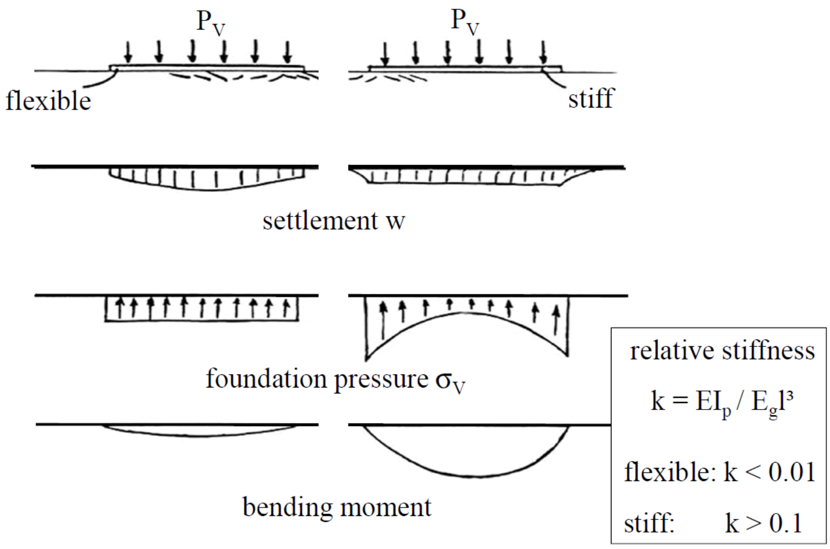


FIGURE 8.10: Settlement and reaction of a flexible and stiff shallow foundation *[Everts, 2015]*

Failure from inadequate bearing capacity can occur if the shear strength of the soil in the influenced zone is exceeded, which leads to loss of overall stability (Figure 8.11).
The maximum bearing capacity is based on Prandtl's theory, that the soil surrounding a geotechnical structure will fail by sliding surfaces with a logarithmic spiral shape. This has been adapted by Brinch Hansen and Terzaghi into the following empirical relation:
$q_c = c \cdot N_c + p_0 \cdot N_q + 0.5 \cdot \gamma \cdot B \cdot N_\gamma$

> with q_c the cone resistance [kN/m^2], c the cohesion, p_0 the stress (mostly taken as σ'_v, the vertical stress) and N_c, N_q and N_γ are bearing capacity factors depending on the angle of internal friction ϕ (Table 8.5 and Figure 8.13)

In case of an eccentric load, the width B_{eff} has to be taken into account, since the eccentricity has a detrimental effect on the bearing capacity through a decreased influence zone (Figure 8.12).
Another failure mechanism is governed by horizontal sliding of footings due to horizontal loading. Finally, excessive settlements or excessive differences in settlement can cause unwanted damage in the superstructure. The accompanying limit states will be discussed in subsection 8.1.6.

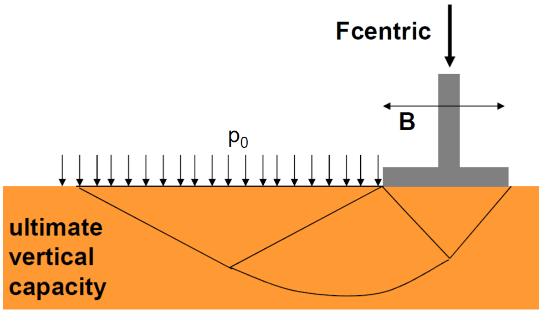


FIGURE 8.11: Maximum bearing capacity of a shallow foundation *[Everts, 2015]*

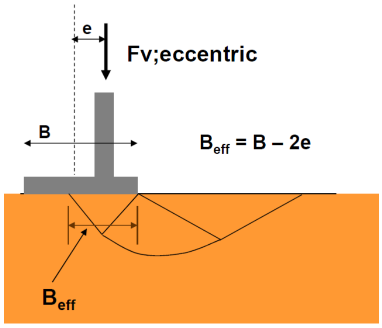


FIGURE 8.12: Influence eccentricity on overall stability of a shallow foundation *[Everts, 2015]*

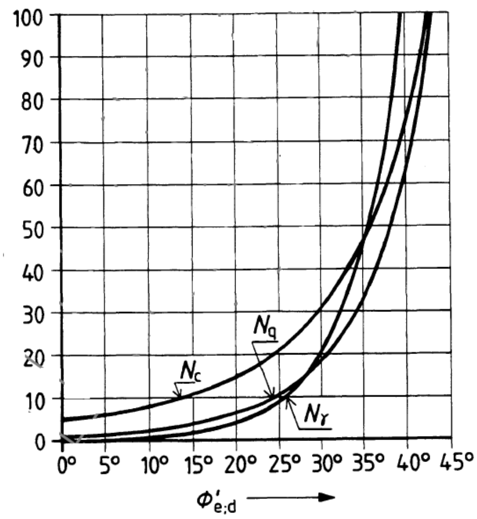


FIGURE 8.13: Values of bearing capacity factors N_c, N_q and N_γ as a function of the angle of internal friction ϕ *[Nederlands Normalisatie-instituut, 1991]*

Spread footing foundations can be applied to reduce a concentrated load from a column, but also to reduce a line load from a wall element (a so-called strip footing). Furthermore, separate footings can be connected through a beam, of which the relative stiffness determines the degree of collaboration between those footings. If an entire building is supported by a single footing, this is called a raft or slab foundation.
Spread footing foundations are usually made from reinforced concrete to withstand punching shear from the superstructure and to spread the loads evenly over a larger surface.

Deep foundation
Deep foundations are applied if the subsoil layers do not have enough bearing capacity to resist the loads from the superstructure. Pile foundations can lead these loads through the soft subsoils into a deeper bearing layer. They can be seen as a spring system in series, in which both the pile tip resistance as the shaft friction will accommodate the bearing capacity (Figure 8.14).

The interaction between pile and soil is complex and dependent on the settlements and stiffness differences. From construction mechanics it holds that the stiffer an element, the more load it will attract and thus the larger the settlement of the pile. If multiple piles are applied, settlement differences will thus occur, since there will be a difference in stiffness and thus in loading.
The soil is able to take up loads from the superstructure through the pile foundation as a result of shear resistance between individual grains. Therefore, the bearing capacity of the soil is dependent on the soil condition as well as the densification as a result of pile installation. This implies that a pile group will encounter a larger overall soil resistance as the packing density of the soil surrounding the piles has increased from the horizontal displacement of soil during installation. This also implies, however, that the pile located at the edges of

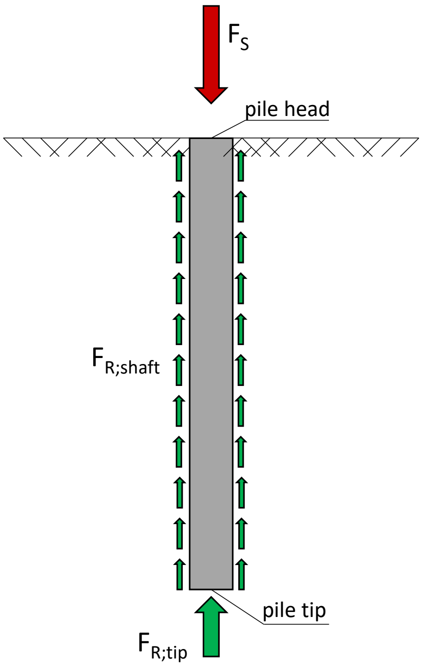


FIGURE 8.14: Bearing capacity of a pile foundation

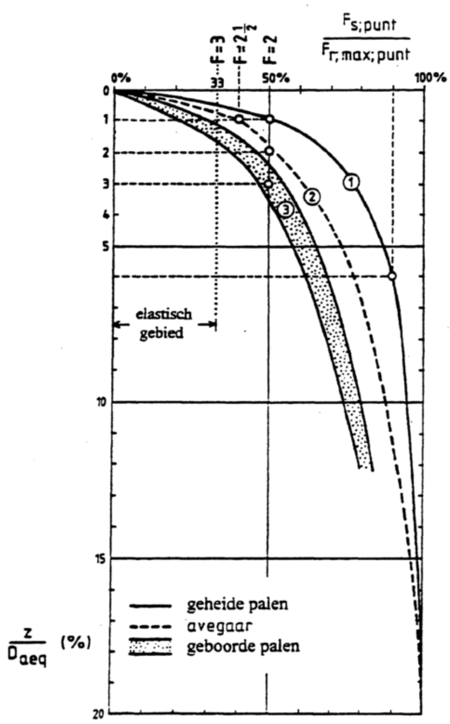


FIGURE 8.15: Load settlement diagram for different pile types *[van Tol, 2006]*

the group will encounter a relatively large settlement since these are less supported in at least one direction. The result is that also the bearing capacity of the pile foundation is not constant, but a function of the settlement: the maximum bearing capacity can only be reached if a certain displacement (about 0.1-0.2 $\cdot$ D_p) is reached (Figure 8.15). This can be explained by the fact that the soil particles will redistribute as a result of the increasing stresses into a more optimal composition, thereby increasing the packing density.

The settlement of the pile spring system, in which the pile head will be taken as a reference point, consists of three elements:

- The elastic shortening of the pile;
  If the load distribution over the shaft friction and pile resistance is known, the normal stress distribution along the length of the pile can be determined through Hooke's Law thereby giving the elastic shortening of the pile.
- The relative settlement of the pile with respect to the surrounding soil;
  The pile foundation settles due to the imposed load, but also the surrounding soil layers will settle. This can either be a result of the downward shear force exerted by the settling pile, or by the presence of cohesive layers above pile tip level. Depending on the relative direction of the settlement difference between pile and soil, this is referred to as either positive friction (the pile settles more than the soil) or negative friction (the soil settles more than the pile). Positive friction will contribute to the shaft bearing capacity, but negative friction will have to be taken into account as an additional imposed load on the pile (Figure 8.16).
- The settlement of the pile as a result of the compressed soil massive beneath the pile tip;
  If multiple piles are placed in a pile group with an intermediate distance less than 8-12 $\cdot$ D_p, additional settlement will take place. This is caused by the fact that the influenced zone reaches to a greater depth beneath the pile tip (up to 1.5 times the width of the pile group) than in the case of a single pile (up to 4 $\cdot$ D_p). The result is that, despite the densification from group installation, the total settlement for a pile group will be larger than that of a single pile.

Failure of a pile foundation can occur if the geotechnical bearing capacity is not sufficient (the shear strength of the individual grains is exceeded) or if excessive settlements or settlement differences occur that will cause damage to the superstructure. The accompanying limit states will be discussed in subsection 8.1.6.

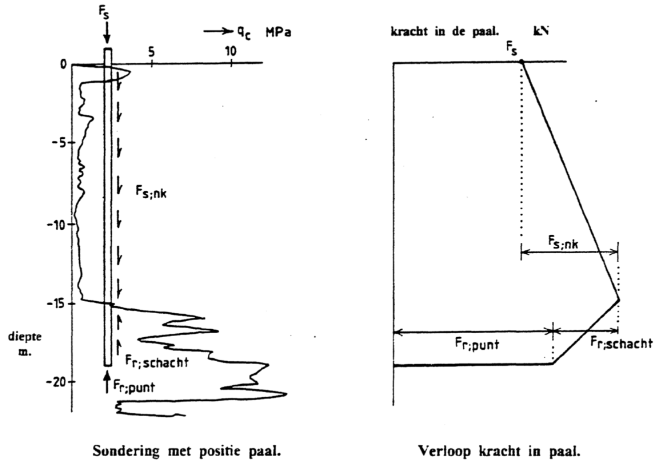


FIGURE 8.16: Load distribution over the length of a piled foundation, with distinction in positive and negative friction *[van Tol, 2006]*

- Maximum bearing capacity;
  The bearing capacity of a pile foundation in Holland is calculated using the CPT results. As mentioned earlier, the bearing capacity is generated by the pile tip resistance and the shaft friction. The accompanying formulae are given below:
  $F_{R;max} = F_{R;max;tip} + F_{R;max;shaft}$
  $F_{R;max;tip} = A_{tip} \cdot p_{R;max;tip}$
  $F_{R;max;shaft} = O_p \cdot \Delta L \cdot p_{R;max;shaft}$
  The maximum pile tip resistance $p_{R;max;tip}$ and shaft resistance $p_{R;max;shaft}$ are obtained from the CPT results.

  The basis for the determination of the maximum pile tip resistance is the method of Koppejan, which was developed in the fifties and has since been accepted as the valid calculation method in most countries. It is based on CPT results and aims on giving a direct empirical relation between the found cone resistance and shaft friction on the one hand and the pile bearing capacity on the other hand. The theory behind the method of Koppejan is analogue to the Prandtl-theory: the soil surrounding the geotechnical structure will fail by sliding surfaces with a logarithmic spiral shape. Koppejans assumption was that not only the soil beneath the pile tip but also in a certain zone above pile tip will encounter shear stresses, thus contributing to the bearing capacity. It should be noted that this assumption has been opposed by other researchers. From Figure 8.17 it follows that in soil with an angle of internal friction equal to ϕ = 30-40° this zone reaches from 2 to 3 times the pile diameter beneath pile tip to 8 to 20 times the pile diameter above pile tip. Although it is known that soil failure will not follow these logarithmic spirals exactly, it is used as an approximation. Since pile tips will be driven into bearing sand layers, that have an internal angle of friction equal to ϕ = 30-40°, Koppejan has proposed an influence zone of 0.7 to 4 times the pile diameter beneath pile tip to 8 times the pile diameter above pile tip (Figure 8.18). This proposal was accompanied by several assumptions:
  - Influence zone is divided into three sections: section I and II lie below pile tip level, section III lies above pile tip level;
  - The contribution of section I and II combined is equal to that of section III;
  - The cone resistance in sections II and III should always be lower than the underlying values since failure will always occur in the weakest layer.

  This has led to the following formula that describes the pile tip bearing resistance according to Koppejan:
  $p_{R;max;tip} = 0.5 \cdot (0.5 \cdot (q_{c;I;av} + q_{c;II;av}) + q_{c;II;av}) = 0.25 \cdot q_{c;I;av} + 0.25 \cdot q_{c;II;av} + 0.5 \cdot q_{c;III;av}$
  This empirical method is based on piles that have been installed in a similar manner to the CPT, thus round, smooth piles with full soil displacement. The accompanying increased stress state in the soil has a positive influence on the bearing capacity of the pile tip, and therefore a reduction will have to be applied for all piles with other installation effects or dimensional properties. This is done by applying appropriate reduction factors:

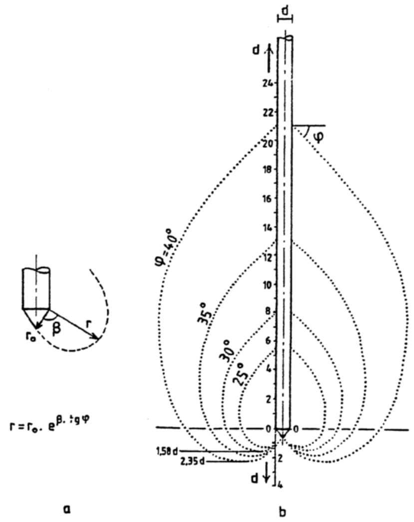


FIGURE 8.17: Influence zones around pile tip according to internal angle of friction ϕ *[van Tol, 2006]*

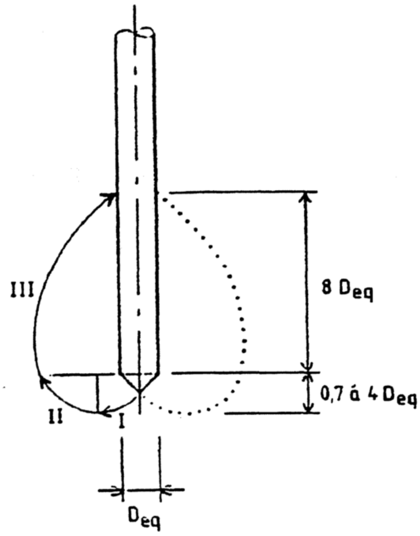


FIGURE 8.18: Influence zone around pile tip in method of Koppejan *[van Tol, 2006]*

- The installation effects of different pile types are taken into account by a so-called pile tip resistance factor α_p, that is equal to 1.0 for full displacement piles but less for all other pile types;
- As the influence of a square cross-section compared to a round cross-section is negligible, using the equivalent diameter $D_{eq} = \sqrt{4/\pi} \cdot D_p = 1.13 \cdot D_p$ is sufficient;
- If the cross-section is not square, but rectangular, a smaller bearing resistance is obtained, which is taken into account by the reduction factor s, see Figure 8.19;
- The shape of the pile tip is also of influence on the bearing capacity and is covered by the reduction factor β, see Figure 8.20.
- A limit has been set for the maximum cone resistance at 15 MPa. This is the maximum pile tip resistance measured in the small amount of proof loadings executed but it also limits the possibility of crushing of sand grains. Another consideration that supports this limit value is the fact that higher cone resistances are often the result of previous horizontal stresses, that will diminish when disturbed by pile installation.

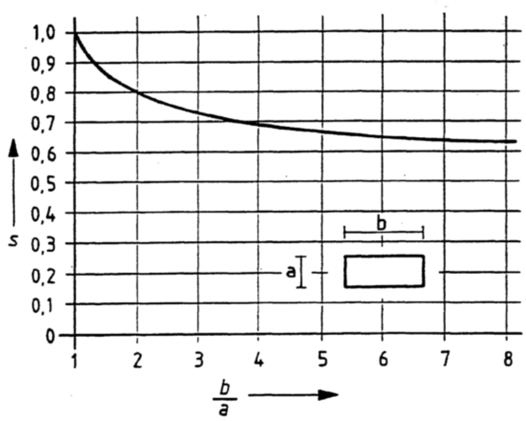


FIGURE 8.19: Pile shape reduction factor s *[van Tol, 2006]*

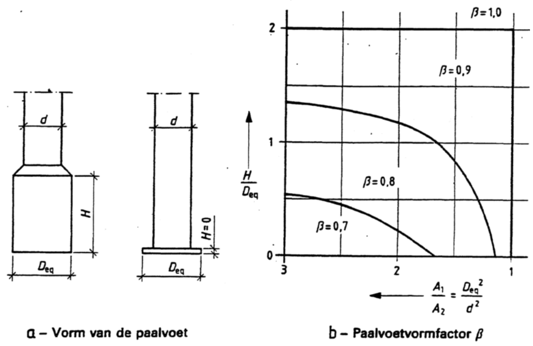


FIGURE 8.20: Pile tip shape reduction factor β *[van Tol, 2006]*

This leads to the final expression for the maximum pile tip bearing resistance:
$p_{R;max;tip} = \alpha_p \cdot \beta \cdot s \cdot 0.5 \cdot (0.5 \cdot (q_{c;I;av} + q_{c;II;av}) + q_{c;II;av})$

The maximum shaft resistance is the summation of both positive and negative friction and can also be obtained from the CPT results. Since positive friction is obtained from layers that settle less than the

pile and since the pile is only allowed to settle slightly, positive friction can only be obtained from sand layers with no underlying compressible clay layers ($q_c > 2$ MPa).
The shaft resistance is a function of shear capacity within the soil and of the shear capacity of the pile-soil boundary. The latter is dependent on the horizontal stress at this boundary and the friction angle between pile and soil. Therefore, the method of installation has a large influence on the shaft friction, since it determines the horizontal stress in the soil. The friction angle is taken as a direct relation with the angle of internal friction of the soil and is therefore a function of the cone resistance. Together, these effects are taken into account through a shaft resistance factor α_s, which is different for various pile types and their respective installation method. The factor α_s can be seen as another notation of the friction ratio f_s/q_c that is taken as 1% for sand layers (Figure 8.4).
The resulting final expression for the maximum shaft resistance is given as:
$p_{R;max;shaft} = \alpha_s \cdot q_c$
Another method for the determination of shaft friction is the so-called slip method, in which the positive friction is based on the stress state analysis and friction characteristics at the boundary between pile and soil:
$p_{R;max;shaft} = K_s \cdot \sigma_v \cdot tan(\delta)$
In this respect, the horizontal soil stress K_s differs from the neutral horizontal stress K_0 due to the effect of installation on the degree of densification of the ground. Also the friction angle δ depends on the installation method.

When looking at negative friction, it appears that at small settlements not all soft layers influence the shaft friction of the pile, but that the top layers exert more load on the pile than the bottom layers. It is generally accepted that at a settlement of 100 mm all soft layers are mobilised and can be taken into account for the calculation of negative friction. This calculation is based on the slip method for an individual pile and on the so-called Gevette Beer method for pile groups. The relative distinction between those two methods is illustrated in Figure 8.21.

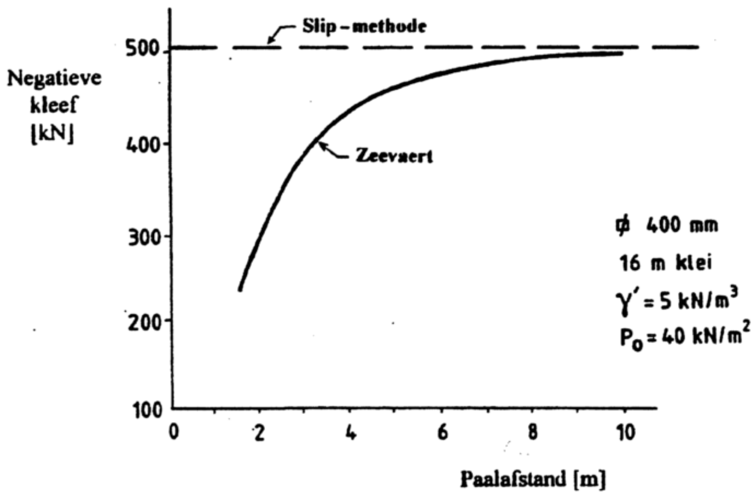


FIGURE 8.21: Comparison of the slip method and Gevette Beer method for negative shaft friction *[van Tol, 2006]*

It must be said that in other countries, not only different influence zones are used in the application of Koppejans method *[van Tol et al., 2010]*, but also different calculation methods are used for both pile tip resistance as shaft resistance *[van Tol, 2006]*. For instance, the influence zone in Belgium is variable and in France it ranges from $-1.5 \cdot D_p$ to $+1.5 \cdot D_p$.

- Excessive settlements;
  The settlement of a pile foundation is a function of the imposed load and the pile bearing capacity. As mentioned before, there is an interaction between settlement of a single pile and the load distribution of the superstructure. This has resulted in load-settlement diagrams that are standardized for different pile types, as illustrated in Figure 8.15.

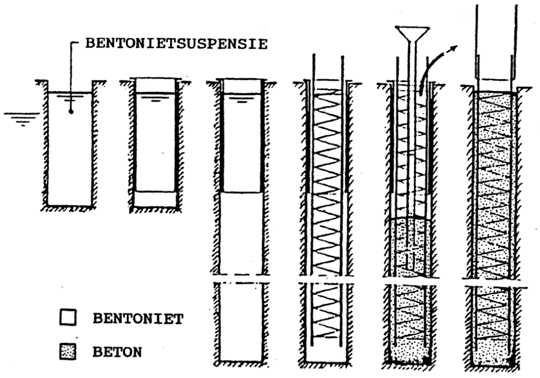


FIGURE 8.22: Installation method for drilled concrete piles *[van Tol, 2006]*

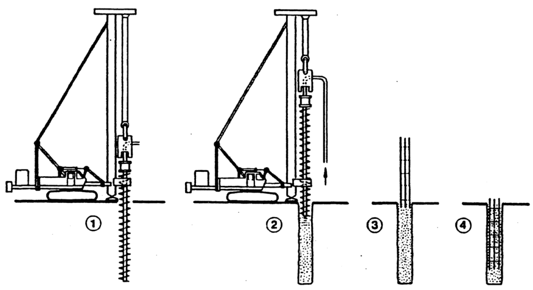


FIGURE 8.23: Installation method for augered concrete piles *[van Tol, 2006]*

The most commonly used pile types in Dutch practice are distinguished based on their installation method, since this has a large impact on the stress state of the soil surrounding the pile foundation and thus on the bearing capacity. In general, there are three different types:

- Piles that result in full displacement of soil;
  All driven piles belong to this category, since the installation will result in a full horizontal displacement of soil thus in an increased stress state of the soil around the pile. This densification increases the bearing capacity of the pile, but it can also result in secondary effects, like horizontal loading or run up, on previously installed piles in the vicinity.
- Piles that result in little displacement of soil;
  Installation of this pile type, of which sheet piling and open tubular steel piles are an example, cause no or negligible changes in the soil stress sate.
- Piles that result in the removal of soil;
  Installation of this pile type will decrease the soil stress state since soil is removed before the pile is installed. This not only causes a decreased bearing capacity of the pile itself, but may also have detrimental effects on previously installed piles in the vicinity.

An overview of different pile types, together with their respective characteristics and applications, is given in Table 8.9. It must be said, however, that this overview is far from complete, since most suppliers of pile foundation have developed their own pile types. Nevertheless, as these specific types have all been derived from the general pile types listed above, the coverage of this overview will be considered as adequate.

In the early days of pile foundations, driven wooden piles were used. These were adequately strong for smaller imposed loads and it had to be ensured that the pile cap would stay beneath the groundwater level to prevent rotting of the pile. Over the years, in-situ and prefabricated concrete piles replaced the wooden specimens, since the possibilities for reinforcement and prestressing allowed a definite increase in bearing capacity and material integrity. Steel piles were also introduced, but the costs of these are rather high and thus application is mostly limited to tension piles or piles under high horizontal loading.

8.1.6. Limit States

Limit states are certain performance requirements, that follow from the safety considerations introduced by the codes and regulations that have been in place for geotechnical structures since Bouwbesluit 1992. Whether geotechnical design fulfils these requirements has to be proven in so-called Safety Checks (subsection 8.1.7) in which the uncertainties of all parameters have been taken into account. The limit states are based on the possible failure mechanisms for a structure and thus depend on the chosen foundation type. The failure mechanisms for shallow and deep foundations have shortly been discussed in subsection 8.1.5, but will be elaborated further below.

As mentioned, the limit states are defined in the codes that apply for geotechnical structures. In Holland, two main codes have been valid since foundations have first been regulated by Bouwbesluit 1992: namely the 'Technische Grondslagen voor Bouwconstructies (TGB) 1990' which included the geotechnical norm NEN 6740 and the Eurocodes, of which Eurocode 7 covers the geotechnical aspects. In the remainder of this chapter, these codes will be indicated by TBG 1990, NEN 6740 and EC 7 respectively.

Pile type	Material	Installation	Length [m]	Diameter [m]	α_p	α_s	Remarks for application
Full displacement piles							
Wooden piles	Wood	Driven	16 - 18	0.26 - 0.28 *(top)* 0.13 - 0.15 *(bottom)*	1.0	0.012	- Concrete head to ensure all wood is below groundwater level - Tapered shape increases positive friction - Limited penetration in sand layer (1 - 2 m) to stay below strength capacity wood - Large settlements occur in old piles due to shallow placement and negative friction
Prefab concrete pile	Concrete	Driven	16 - 35	0.18 - 0.50 *(square)*	1.0	0.010	- Prestressing required to prevent crack formation during transport and installation - High quality of concrete pile is ensured through prefabrication
Driven in-situ pile	Concrete	Driven/vibrated	< 40	0.30 - 0.70	1.0	0.014 *(driven pull up)* 0.012 *(vibrated pull up)*	- Steel tube with seperated footing is driven, concrete is poured while steel is pulled - Reinforcement cage and prestressing can be applied before pulling - Installation process cannot cause damage, but quality can only be checked acoustic - Presence of fresh concrete is troublesome in soft layers and for nearby pile driving - Pulling of the steel pipe through vibration causes a small decrease of stress state
Vibro-piles	Concrete	Driven/vibrated	< 40	0.30 - 0.70	1.0	0.014 *(driven pull up)* 0.012 *(vibrated pull up)*	- Same installation principle as the driven in-situ pile, but with prefab concrete pile - Space between steel pipe and prefab pile is filled with liquid concrete - High quality of concrete pile is ensured through prefabrication - Large possibility of corrosion of reinforcement in tension piles
Screwed in-situ pile (Fundex, Tubex)	Concrete	Screwed	< 30	0.38 - 0.52	0.9	0.009 *(if grouted)* 0.006 *(if else)*	- Steel tube with spiral-shaped screwed tip is screwed into the ground - Reinforcement is placed, concrete is poured and the steel tube is pulled or vibrated - Specially suited for vulnarable locations: full displacement and little vibration - 50% more expensive than a driven prefab concrete pile - Full displacement pile, but small decrease of stress state due to pulling of steel tube
Screwed prefab pile	Concrete	Screwed	?	0.50	0.8	0.006	- Hollow prefab HSC pile with a screw-thread over lower 6 m is screwed into ground - Soil is transported upwards thus a decrease in stress state will occur
Little displacement piles							
Sheet piling	Steel	Driven/pushed	< 40	\	1.0	0.0075	- Steel sheeting and piles are in general twice as expensive as concrete piles - Application is limited to tension piles or piles subjected to high moments
Open tubular steel pile	Steel	Driven/pushed	< 40	< 0.80	1.0	0.0075	- See remarks sheet piling
Replacement piles							
Bored pile	Concrete	Drilled	< 100	< 2.50	0.5	0.006	- Large range in diameters is especially suitable for high loading - Stability of borehole has to be ensured by support tubes or support fluid (bentonite) - Bentonite is a clay, that reacts with water into a sealing fluid - Decrease of stress state
Augered pile	Concrete	Drilled/screwed	30 - 40	< 0.90	0.8	0.006	- Steel tube is screwed into the ground while transporting soil upwards - Expansive concrete is poured into the borehole when steel tip is at depth - Surrounding soil will have a decreases stress state - Reinforcement can be placed after pulling out the steel tube
Pulse pile	Concrete	Pulsated	2 - 36	0.17 - 0.50	0.5	0.005	- Borehole is obtained through pulses - Costly and lengthy installation method, is rarely used anymore - Specially suited for strengthening of existing foundations with limited height

TABLE 8.9: Overview of pile foundation types and their characteristics *[van Tol, 2006] [Backhausen and van der Stoel, 2014] [BV, nd]*

It must be stated that before the introduction of geotechnical code in 1992, several regulations were already commonly used in Dutch practice. The first was known as the RFG 1985: "Richtlijnen voor funderingen van gebouwen", which summarised the applied design philosophies of different municipalities at that time.

In general, the differences between the NEN 6740 and EC 7 are small. Since EC 7 is valid for multiple European countries, the basis is kept universal, while most specific information is stated in the National Annex and in Holland this is based on current practice, thus the NEN 6740. The largest differences can be found in the safety approach, as was shown in Table 8.3 and the notion that the resistance factors for pile tip and shaft resistance will be changed in 2016 (see subsection 8.1.7 for more information). With respect to the limit states, EC 7 is much more comprehensive in addressing the different failure mechanisms than NEN 6740.

Both codes, however, have in common that they distinguish an Ultimate Limit State (ULS), in which breaching the limit state results in structural failure, and a Serviceability Limit State (SLS), in which breaching the limit states results in an unwanted loss of serviceability or the need for excessive maintenance.
An overview of the limit states for both shallow and deep foundations, as mentioned in the codes, is given below:

NEN 6740
In NEN 6740 a distinction is made between two ultimate states (designated as limit state 1) and one serviceability state (designated as limit state 2):

1A A failure mechanism appears in the soil within the influence zone or at the boundary between soil and foundation. This is a strength requirement and is based on the design values of strength and capacity. The corresponding failure mechanisms are:
- For spread footing: vertical bearing capacity, tilting, overall stability and horizontal sliding
- For piles: vertical bearing capacity

1B Excessive deformations cause breaching of the safety requirements for the structure. This is a deformation requirement and is based on the design values of strength and capacity. The corresponding failure mechanisms are:
- For spread footing: excessive settlements and differential settlements
- For piles: excessive settlements and differential settlements

2 Excessive deformations cause unwanted loss of serviceability or the need for excessive maintenance. This is also a deformation requirement but in this case is based on the representative values of strength and capacity. The corresponding failure mechanisms are equal to that of limit state 1B.

In Figure 8.24 the coherence of all three limit states is presented. Although it is not possible to compare limit state 1A to limit states 1B and 2, due to the different calculation models, it can be seen that limit state 2 is always decisive. This is demonstrated by the fact that the settlement requirement for limit state 2 is three times more stringent than that of limit state 1B, while the difference in representative values is never more than 1.7 (material factor γ_M = 1.3 and load factor γ_f = 1.3, see also subsection 8.1.7).

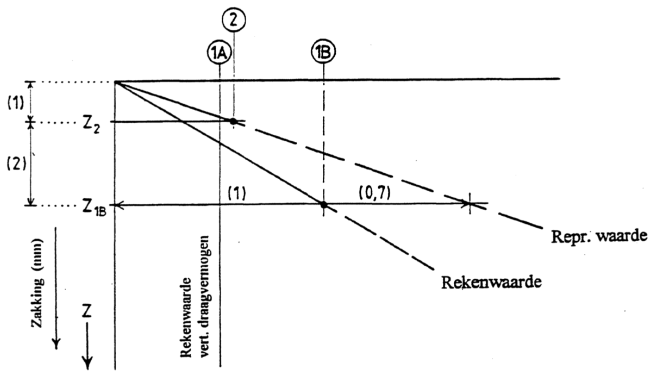


FIGURE 8.24: Comparison of the limit states for spread footings given in NEN 6740 *[van Tol, 2006]*

Eurocode 7
EC7 speaks of five limit states that have to be checked in geotechnical design:

EQU Loss of equilibrium of the structure or the ground
STR Internal failure or excessive deformation of the structure or structural elements
GEO Failure or excessive deformation of the ground

UPL Loss of equilibrium of the structure or the ground due to uplift by water pressure or other vertical actions

HYD Hydraulic heave, internal erosion and piping in the ground caused by hydraulic gradients

Of these limit states, GEO is often critical for the dimensions of the structural elements involved in foundations and sometimes it is also critical for the strength of those elements.

The failure mechanisms mentioned in EC7 for both shallow and deep foundations are:

- For spread footing:
  - Loss of overall stability (ULS)
  - Bearing resistance failure, punching failure, squeezing (ULS)
  - Failure by sliding (ULS)
  - Combined failure in the ground and in the structure (ULS)
  - Structural failure due to foundation movement (ULS)
  - Excessive settlement (SLS)
  - Excessive heave due to swelling, frost and other causes (SLS)
  - Unacceptable vibrations (SLS)
- For piles:
  - Loss of overall stability (ULS)
  - Bearing resistance failure of the pile foundation (ULS)
  - Uplift or insufficient tensile resistance of the pile foundation (ULS)
  - Structural failure of the pile in compression, tension, buckling or shear (ULS)
  - Combined failure in the ground and in the foundation (ULS)
  - Combined failure in the ground and in the structure (ULS)
  - Excessive settlement (SLS)
  - Excessive heave (SLS)
  - Excessive lateral movement (SLS)
  - Unacceptable vibrations (SLS)

More information on the use of these limit states is given in *[Nederlands Normalisatie-instituut, 2005]*.

8.1.7. Safety Checks

As stated in subsection 8.1.6, a limit state is a functional requirement condition following from the design criteria as given in the codes and regulations. With the help of Safety Checks, it must be proven that the structure meets these requirements.
In general, two limit states are distinguished, the Ultimate Limit State (ULS), in which breaching the limit state results in structural failure, and a Serviceability Limit State (SLS), in which breaching the limit states results in an unwanted loss of serviceability or the need for excessive maintenance. In this respect, the Safety Check for ULS can be considered a strength check, while SLS is often based on a deformational limit.
If a design does not meet the ULS requirement, the structure fails. Therefore, the values used in a ULS Safety Check have to be on the safe side, such that the probability of structural failure during the lifetime is diminished to a certain excepted level.

Probabilistic analysis *[Jonkman et al., 2015]*
Since the introduction of an official geotechnical design code in 1992 (TGB 1990, introduced by Bouwbesluit 1992), the safety of geotechnical structures is based on probabilistic analyses. The required reliability of structures is the result of extensive risk analyses on the consequences of failure and the chance of this failure occurring. Due to the fact that both strength and resistance are not a deterministic value, but are distributed according to a Gaussian or normal distribution (Figure 8.25), probabilistic calculations are needed to determine the risk of failure.

If the distribution functions of the load S and resistance R are known, the probability of failure is given by:
$P_f = P(S > R)$
When formulated as a limit state Z, in which Z = R – S, failure occurs when Z < 0 and thus the failure probability is given by:
$P_f = P(Z < 0)$
This means that the probability of success, or the so-called reliability, is given as:

$P_s = 1 - P_f$ $P_s = 1 - \Phi(\beta)$ or $P_f = \Phi(-\beta)$

In which β is the reliability index and Φ is a cumulative normal distribution.

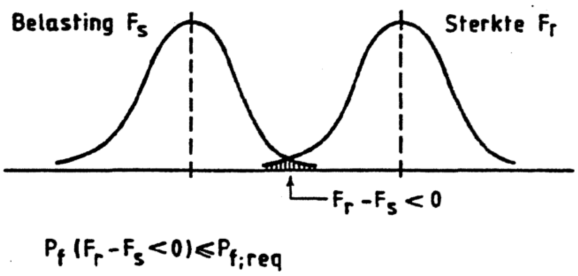


FIGURE 8.25: Normal distribution functions of load and capacity *[van Tol, 2006]*

Cornell (1962) has introduced the reliability index β as a safety measure, that gives a direct relation to the failure probability (Figure 8.26). For a linear limit state with normally distributed variables, β is defined as the ratio between the expected value μ_Z and the standard deviation σ_Z: the larger this ratio, the smaller the failure probability and thus the higher the reliability (Figure 8.27).
In order to determine the reliability of a structure, five methods can be used, each with decreasing calculation difficulty:

- Level IV methods (risk-based)
  In this method both probability of failure and the consequences of this failure are taken into account for the determination of the reliability, thus comparing different designs on an economic basis.
- Level III methods (numerical)
  All distribution functions of the variables are taken into account, using numerical integration to obtain the failure of probability exactly.
- Level II methods (approximation)
  The stochastic variables are assumed to be normally distributed, modelled by their mean values and standard deviations.
- Level I methods (semi-probabilistic design)
  Uncertain parameters are modelled as one representative or characteristic value for both load and resistance, in which partial factors are applied to obtain the reliability.
- Level 0 methods
  Deterministic calculations, in which no deviations are taken into account.

To avoid lengthy calculations and the need for an extensive probabilistic analysis for every design, codes and regulations are based on a Level I method with partial factors.

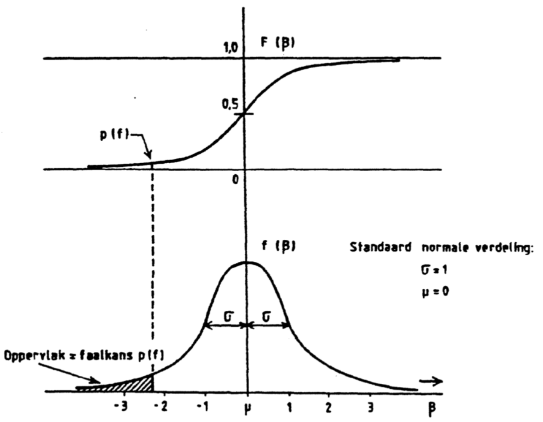


FIGURE 8.26: Relation between the reliability index and probability of failure in a standard normal distribution *[van Tol, 2006]*

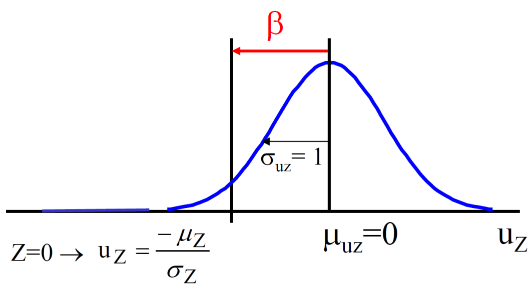


FIGURE 8.27: Reliability index in a normal distribution function *[Jonkman et al., 2015]*

Partial factors *[Jonkman et al., 2015]*
The characteristic values for stochastic variables are taken as a certain non-exceedance probability in the theoretical distribution with infinite samples. For strength properties, the 5-quantile is normally taken, while for stiffness parameters the mean value suffices. Partial safety factors are applied to these characteristic values to obtain a safe design with an adequate reliability. The relation between design values, characteristic values and partial safety factors is given below and illustrated by Figure 8.28:
$R_k = \mu_R + k_R \cdot \sigma_R$ and $S_k = \mu_S + k_S \cdot \sigma_S$
In a ULS Safety Check, it must be proven that the design meets the following requirement:
$R_d > S_d$, thus: $\frac{R_k}{\gamma_R} > S_k \cdot \gamma_S$
The values of the partial safety factors for most common load and material factors are standardised and based on a Level II calculation method.

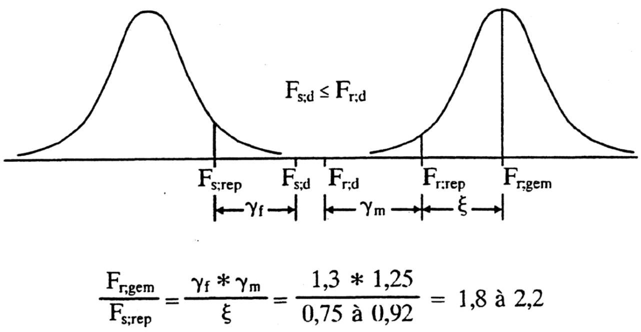


FIGURE 8.28: Partial factors in a semi-probabilistic calculation for geotechnical structures *[van Tol, 2006]*

Uncertainties in pile bearing capacity *[van Tol, 1994]*
With the introduction of codes and regulations for geotechnical structures in TGB 1990, probabilistic analyses have been performed on the loads and bearing capacity of geotechnical structures. A required reliability has been stated in the Safety Classes and partial safety factors have been determined to eliminate the need for lengthy calculations for every design.
For piled foundations it turns out, however, that if all uncertainties are taken into account correctly in this semi-probabilistic analysis, the safety margin for the required failure probability is much higher than the normal Dutch practice up to that date. As there was no indication of this Dutch practice to be unsafe, it was decided to reduce the partial safety factors to such extent that the final safety before and after code introduction would be more or less the same.
The question remains whether this adaption has adequately taken into account the uncertainties in geotechnical design. According to a rough estimation made by *[van Tol, 1994]* on the amount of damaged foundations in the period between 1965 and 1994, the probability of failure is of the same order of magnitude as prescribed in TGB 1990. It can thus be concluded that the safety margin already applied in Dutch practice was sufficient and that either the uncertainties are smaller than expected or there are some hidden safeties present in the system.

Penetratie		*Gemiddelde α_p-waarde*	*Standaard afwijking (σ)*	*Variatie coëfficiënt (v)*
< 8 D	Zonder limiet	0,989	0,27	0,27
	Met limiet $q_{b;max}$ < 15 MPa	0,992	0,27	0,28
> 8 D	Zonder limiet	0,635	0,08	0,12
	Met limiet $q_{b;max}$ < 15 MPa	0,834	0,35	0,42

Tabel 3 Gemiddelde α_p voor ondiepe en diepe palen met en zonder limietwaarden.

TABLE 8.12: Average values of $alpha_p$ for different depths and with or without cut-off limits *[van Tol et al., 2010]*

Further research into these uncertainties and hidden safeties has given more insight in the bearing capacity of a piled foundation. This topic is further elaborated in section 8.5.
It turns out that for certain situations, the pile tip bearing capacity is overestimated due to a too high value for α_p. This has led to two recommendations regarding the calculation of the pile bearing resistance of prefabricate driven concrete piles:

1. If the soil can be considered as homogeneous over an influence zone of $-4 \cdot D_p$ to $+8 \cdot D_p$, the French method may be applied, in which the influence zone is taken as $-1.5 \cdot D_p$ to $+1.5 \cdot D_p$, with the adaptation of α_p into $\alpha_p = 0.6$.
2. The Dutch method is applied, with an influence zone of $-4 \cdot D_p$ to $+8 \cdot D_p$, and a distinction is made between shallow piles ($<8 \cdot D_p$ into the sand layer) and deep piles ($>8 \cdot D_p$ into the sand layer), with $\alpha_p = 1.0$ and $\alpha_p = 0.6$ as the respective pile resistance factors.

8.2. TIMELINE OF GEOTECHNICAL DESIGN IN HOLLAND

The inventory of all aspects regarding geotechnical design, has been presented in section 8.1 according to the generic design scheme (Figure 8.2). There is, however, also a distinction to be made when looking at the Dutch practice over time. In this section, a historic overview will be given, which results in Figure 8.30.

From section 8.1 and Figure 8.2 it follows that the developments in time can roughly be covered by three subjects:

- Soil investigation methods;
- Foundation types (installation methods and materials);
- Codes and regulations.

These subjects will be included in a general historic description and are clearly distinguishable in the overview given in Figure 8.30.

According to *[van Tol, 1994]*, the Dutch geotechnical practice can be divided into three time periods, namely from before 1900 till 1940, from 1945 till 1992 and from 1992 onwards. Since this article has been written in 1994 and some crucial developments have taken place in recent years, a fourth period will be distinguished in this thesis, regarding the period from 2017 till the near future. The third period will be extended to 2017.

Period 1: < 1940

This period is characterised by the little knowledge available on the bearing capacity of the soil layers and pile foundations. Drilling was performed to obtain a global indication of the soil composition, but there were no adequate tools to derive the bearing capacity from these samples. Therefore, it was common practice in private projects to use the same pile length as was already present in nearby building structures. For the larger, public projects, often test piles were driven into the ground at location to obtain some knowledge on the depth of the bearing soil layer. Although the uncertainty of this indicative method was large, more insight and experience over the years led to an increase of quality in foundation design. A small research performed by the municipalities of Amsterdam and Rotterdam in the seventies illustrates this increase of quality (Figure 8.29).

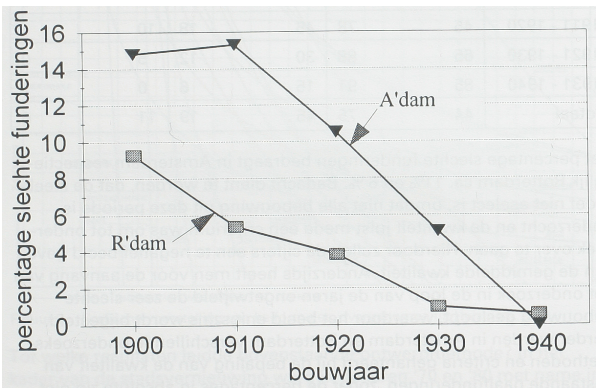


FIGURE 8.29: Decreasing number of damaged pile foundations in Amsterdam and Rotterdam between 1900 and 1940 *[van Tol, 1994]*

Before 1925, all piles applied were wooden piles with a larger diameter at the pile head than at the pile tip *[Klaassen, 2015]*. Due to this shape and the fact that negative friction was not taken into account before the thirties *[van Tol, 2006]*, many wooden piles have been loaded beyond the bearing capacity, resulting in large settlements. Another disadvantage was the decay of the pile had as a result of fluctuating groundwater levels. Therefore, the pile head had to be constructed below groundwater level, or a concrete top part had to be applied. The latter pile type was dominant in Holland between 1925 and 1950 *[Klaassen, 2015]*.
Steel piles have been used for piling as well from the beginning of the twentieth century *[Warrington, 2012]*.

Period 2: 1945 - 1992
The second period is taken between 1945, at the end of World War 2, and 1992, when the first geotechnical code was introduced in the Dutch practice. Several important developments took place in this period, like the increasing application of CPTs as a soil investigation method. Not only does a CPT give direct insight into the bearing capacity as a function of depth, it is also easy and cheap to apply. Although the CPT was already introduced by Barentsen in 1932 *[Geo-Equipment B.V., nd]*, the first CPTs were only used to determine the depth of the bearing soil layer. With the increasing possibilities in hydraulic driving and especially the introduction of the friction jacket cone by Begemann in 1965 *[Geo-Equipment B.V., nd]*, it was considered as a test pile, giving rise to the development of several empirical design rules. One of those design rules was developed by Koppejan (1952), in which a direct relation was given for the determination of pile bearing capacity from CPT results. In 1969, Begemann complemented this application by conducting a research into the shaft resistance as a function of the measured friction ratio in a CPT. Other design rules were based on different empirical relations from different CPT systems. The first electrical CPTs were introduced in 1965 as an outcome for the large horizontal deviations at greater depths with mechanical CPTs *[Geo-Equipment B.V., nd]* and have completely replaced the mechanical CPTs nowadays.
At the same time, the amount of different pile types increased substantially, like the prefabricated concrete pile and multiple in-situ concrete pile systems. The first classical reinforced concrete piles were developed in France, in 1897, by Hennebique *[Warrington, 2012]* but they were not used in Holland until around 1950, after which they gradually replaced the wooden piles *[Klaassen, 2015]*. Not much later, at the end of the fifties, pre-tensioned concrete was used more and more in civil engineering and was also introduced in pile foundations. It turned out that the increase of tensile capacity of the concrete pile through prestressing had numerous advantages in transportation and installation of the pile *[BFBN, nd]*.

Although numerous pile systems, soil investigation methods and design rules have been developed in this period, a consensus was gradually reached through practical experience and more research. This generally accepted design method was based on a smooth cone tip for the CPT in combination with the method of Koppejan to determine the pile bearing resistance. Nevertheless, there were a number of uncertainties in the pile design that had yet to be constrained by further research, like the influence of installation on the soil stress state and the negative friction.

An indication of the quality of the pile systems in this period has been obtained by *[van Tol, 1994]*, by considering insurance claims and experienced geotechnical engineers. Through rough calculations and extrapolation of the obtained results, it was concluded that the probability of failure for pile foundations constructed between 1965 and 1979 is in the same order of magnitude as later prescribed in the codes.

For shallow foundations, design rules were developed, based on the analytical theory of Prandtl (1921) for the bearing capacity of soil, by Terzaghi (1943) and Brinch Hansen (1961). They extended Prandtl's theory by taking into account the own weight of the soil and reduction factors for the foundation dimensions, the load direction and slope of the ground *[Backhausen and van der Stoel, 2014]*.

Period 3: 1992 - 2017
The third period is initiated by the introduction of the first geotechnical code, NEN 6740, as part of TGB 1990 and Bouwbesluit 1992. The TGB, in which the technical basis is laid for building structures, has previous versions, but until 1990 no geotechnical aspect was included. The design codes and regulations are based on the consensus in design methods that was accepted at that time, thus the method of Koppejan in combination with a CPT. New in this respect is however the application of statistics in geotechnical design and the use of semi-probabilistic analysis through partial factors.

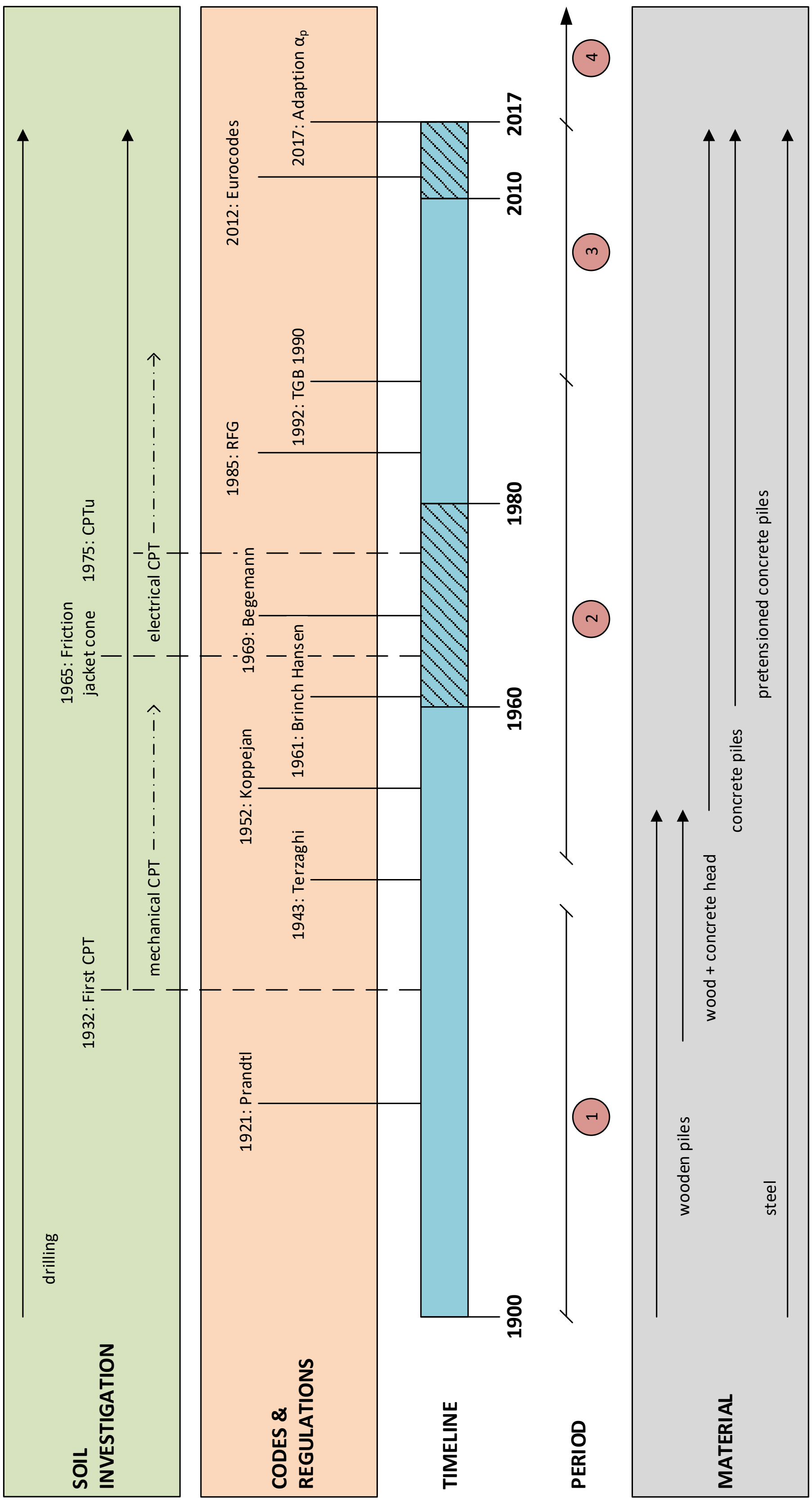


FIGURE 8.30: Timeline of geotechnical design in Holland between 1900 and the present

With the implementation of statistics in geotechnical design, many risk analyses and probabilistic analyses were conducted to obtain the partial safety factors. When compared to the common Dutch practice until that time, however, the safety margins from probabilistic design were several factors larger. Since there was no indication of structural errors in the design practice, it was decided to adapt the partial safety factors in such a way that the overall safety of the structures before and after introduction of the code would stay the same.
This has however given rise to an extensive discussion on uncertainties and hidden safeties within the bearing capacity and the codes, of which the outcome is still not settled.

Extensive research is conducted into these uncertainties and hidden safeties, of which the outcomes are presented in section 8.5. Many studies have led into more insight into the mechanisms behind pile bearing capacities, but often not into adaptations of the codes and regulations. An exception is the research into the value of pile resistance factors α_p and α_s, in which it is demonstrated through proof loading results that for certain situations a reduction of α_p is recommended.

With the introduction of Bouwbesluit 2012, the TGB 1990 was replaced by the Eurocodes, in which it is attempted to summarise all European design practices in civil engineering into one universal code. The difference between the 'old' NEN 6740 and the 'new' Eurocode 7 for geotechnical structures is small, as it was decided to postpone the adaptation of α_p until January 1^{st} 2016 (see section 8.5).

Period 4: > 2017
The fourth period starts with the official adaptation of α_p in Dutch practice on January 1^{st} 2017, a year later than initially planned. The conditions on which the adaptation was delayed in the first place, namely the chance for every pile supplier to perform proof loading tests in order to analyse the discrepancy in pile resistance factor α_p, has however not taken place. The discussion on uncertainties, hidden safeties and code application in pile foundations is thus still a hot topic and is further elaborated in section 8.5.

8.3. FOUNDATION DESIGN FOR BRIDGES: 1960 - 1980

As stated in section 2.1, almost 50% of all bridges currently present in our infrastructure network were constructed between 1960 and 1980. From the geotechnical timeline presented in Figure 8.30, a general idea can be obtained of the practice in geotechnical design of that period.
Although the RFG 1985 had not been introduced yet, this regulation does give an accurate idea on the design methodology commonly applied for foundations between 1960 and 1980. After all, the RFG 1985 was based on the Dutch practice in this period and since no information proving otherwise has been found, it is assumed that the RFG describes the design methodology applied in the time-span reviewed.

Below, the general design steps and considerations will be presented. These are based on *[Commissie Overleg Bouwconstructies, 1988]* and personal experience of several experts consulted, of which *[van Tol, 2017]* is one.

1. **Load from the superstructure**
   The load distribution and magnitude coming from the superstructure was generally provided by the structural engineer. The following major load actions were taken into account *[Nederlands Normalisatie-instituut, 1963]*:

   - Self-weight (vertical, permanent)
   - Traffic load (vertical, variable)
   - Snow load (vertical, variable)
   - Wind load (horizontal, variable)
   - Break load (horizontal, variable)

   The load distribution was obtained by hand calculations and several simplified static schemes. The same goes for the schematic representation of the pile foundations. These were schematised as being simply connected to the abutment at pile head and to the soil at pile tip, thus only being able to bear normal loads.
   Negative friction was usually neglected in the load actions on the pile foundation, but this was compensated by the fact that often the positive friction was also not taken into account.

2. **Soil investigation**
   An adequate soil investigation is required, in which mechanical or electrical CPTs have to be performed according to NEN 3680: Soil investigation - Static cone penetration tests. In the period 1960-1980, most CPTs were however performed manually, together with a certain amount of drilling tests.
   RFG 1985 prescribes a CPT every 20 m, of which about 10% are equipped with a friction jacket. However, the timeline in Figure 8.30 shows that this was not introduced until 1965. Furthermore, when a pile system is applied that does not cause soil displacement, 1.5 times more CPTs have to be executed. Also, one drilling test is prescribed for every 15 to 20 CPTs. For shallow foundations, the CPTs have to reach up to a depth of 3 m below construction level, while for pile foundations this minimum depth is $10 \cdot D_p$ beneath pile tip level, with at least 10% of the CPTs up to a depth of $15 \cdot D_p$.
   If large differential soil conditions are observed over a relatively short distance, these can lead to settlement differences and thus a settlement calculation will be required.

3. **Foundation type and length**
   In general, the foundation type and construction level are not based on the worst or average CPT result, but on one of the poor CPT results. RFG 1985 speaks of a pile foundation when the length is more than 6 to 10 times the diameter of the foundation. In this period, wooden piles were still used, but also steel tubular piles and reinforced or prestressed concrete piles that were mainly cast in-situ but prefabricated piles were gradually applied as well. Overall, the reinforced and prestressed concrete piles were mostly used.

4. **Dimensioning**
   The pile tip bearing capacity P_{pu} for compression piles is obtained by the method of Koppejan, using static CPT results and an influence zone of $0.7 \cdot D_p$ to $4 \cdot D_p$ below pile tip level and $8 \cdot D_p$ above pile tip level. Also, a reduction factor β is applied for the shape of the pile tip and a cut-off limit of 15 MPa is prescribed for the cone resistances.

   Calculations: $P_{pu} = A_{pu} \cdot \beta \cdot p_g$
   with p_g according to Koppejan: $p_g = 0.5 \cdot (\frac{I+II}{2} + III)$

   For the positive shaft friction P_p, either the slip method can be used, or the shear resistance between soil an pile shaft measured as a percentage of the cone resistance. For relatively short piles or relatively long piles, there can be a large difference between the two methods, in which case the two outcomes must be averaged.

   Calculations:
   - Slip method: $P_p = \Sigma A_m \cdot f \cdot \sigma_{vk} \cdot K_e$
     with: $f = 0.85 \cdot tan(\phi)$
   - Percentage of the cone resistance: $P_p = \Sigma A_m \cdot f_c \cdot \frac{q_c}{100}$
     Both K_e and f_c are pile dependent and can be found in *[Commissie Overleg Bouwconstructies, 1988]*.

   The negative shaft friction P_n is obtained by using the slip method and is expected to apply over the entire length of the soft subsoil layers.

   Calculations:
   - Slip method: $P_N = \Sigma A_m \cdot (f \cdot \sigma_{vk} \cdot K_e + c)$

5. **Design checks**
   In the RFG 1985 it is recommended to applied a safety factor of 1.4 on both strength and load action, obtaining an overall safety of 1.96 = 2. Before the RFG however, common practice was to use a somewhat higher safety: 2.5 for pile foundations and 3.0 for shallow spread footings.

   Designs should be checked on the following three criteria:
   - Characteristic shaft bearing capacity $\geq Q_k + n_n \cdot P_n$;
   - Characteristic equilibrium bearing capacity: $Q_k \leq P_{pu}/n_{pu} + P_p \cdot r/n_p - n_n \cdot P_n \cdot r$;
     - n_{pu}, n_p and n_n are uncertainty factors for the pile bearing capacity and negative friction, based on the uncertainty factor n that depends on the pile type and calculation method:
       - $n_{pu} = n_p = 1.4 \cdot n$
       - $n_n = \frac{n}{1.4}$

  - r is a reduction factor that takes group effects into account for negative shaft friction and tension piles, and is a function of $\frac{a}{D_{pu}}$ (figure 8.31).
- Deformation bearing capacity: deformation ≤ acceptable limits:
  - Compression of deeper layers beneath pile tip level (z);
  - Compression of layer directly beneath pile tip (z_{gr});
  - Elastic shortening of the pile (ΔL);
  - Horizontal displacement of the pile head.

Although negative friction was often neglected in design, this was compensated for by the simultaneous neglection of positive friction. Therefore, piles that were driven relatively deep into the sand actually have a higher capacity than calculated, while for shallow driven piles the opposite holds. The latter is however often not problematic, since the shallow piles are mostly concrete piles with a widened pile tip, which were commonly applied between 1960 and 1970. Due to the widened pile tip, the maximum depth to which these piles could be driven in the sand was about 0.5 m. Nevertheless, due to their massive bearing capacity the effect of negative friction will not be determinative for the overall bearing capacity.

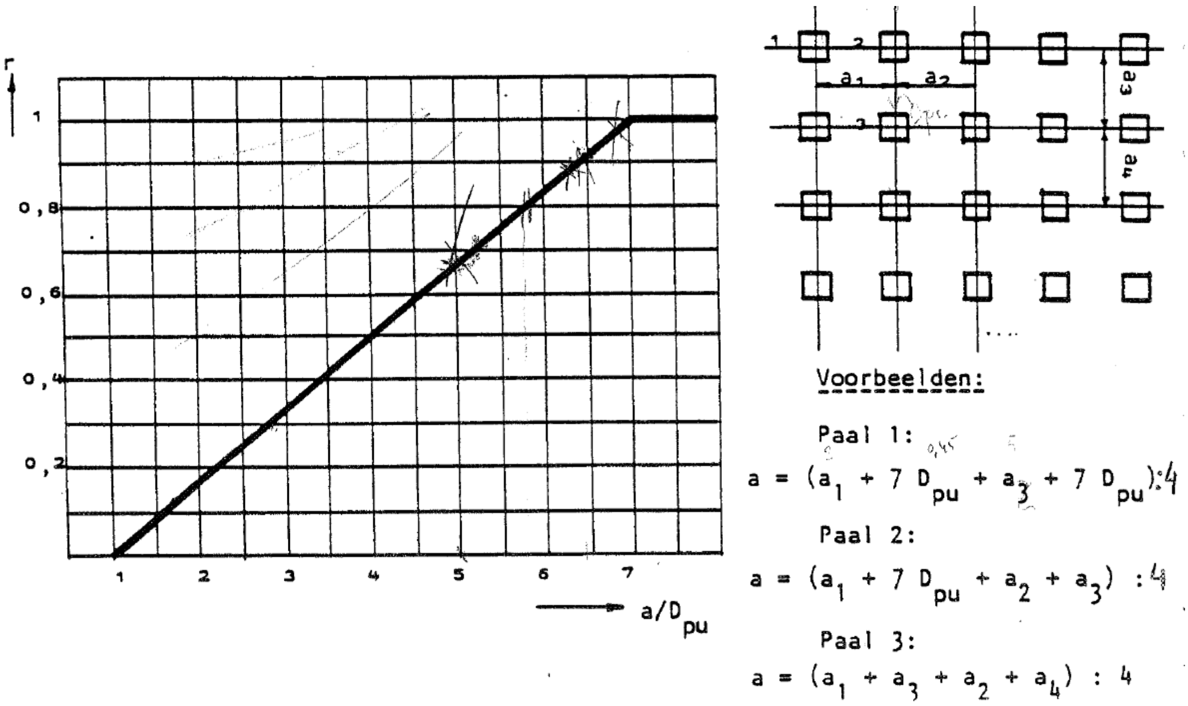


FIGURE 8.31: Group effect r as taken into account by *[Commissie Overleg Bouwconstructies, 1988]*

6. **Monitoring**
   Although the intended pile tip level is obtained from the CPT results, the method of confirmation was based on blow counting. Beforehand, a maximum amount of blows per fixed length was determined, at which the pile tip was expected to have reached its final position.

   After installation, the following data must be gathered into a report:

   - The soil investigation report with CPT results, drilling test results and settlement calculations;
   - A calculation based pile plan with accompanying construction levels;
   - Drawings which include: the positions of soil investigation, the dimensions and quality of the foundation, height of the ground level, ground water level and construction level and the position of obstacles;
   - The calculated bearing capacity.

   Although a finalising report is required in RFG 1985, it will probably not be possible for many projects executed in 1960 to 1980 to obtain this information, either because it is lost or because it was never made.

8.4. FOUNDATION DESIGN FOR BRIDGES: 2010 - PRESENT

In current practice, bridge foundation design is based on Eurocode 7 *[Nederlands Normalisatie-instituut, 2005]* and the SBR guidelines *[SBR, 2012]*. The generic design steps are schematised in Figure 8.32 and are further elaborated below.

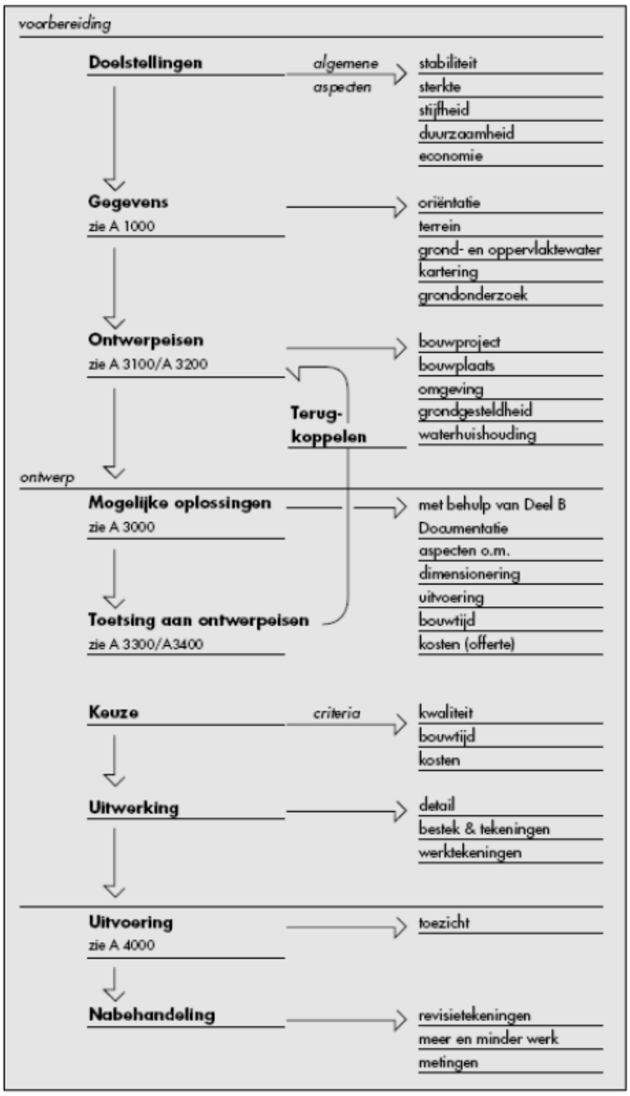


FIGURE 8.32: Generic design steps as prescribed by the SBR regulations (in Dutch) *[SBR, 2012]*

1. **Objectives**
   At first, the objectives of foundation design must be determined. This entails that the foundation has adequate strength and stiffness, the durability must be sufficient and the overall costs of the project must be minimised. These objectives will be further elaborated in step 3: Design requirements and step 4: Possible solutions.

2. **Data acquisition**
   For the acquisition of sufficient data regarding the project, the surroundings and the soil, there is no solid approach but it requires some form of creativity from the geotechnical engineer.

   - Project data:
     As there is a close interaction between foundation and superstructure, a adequately elaborated superstructure design must be present before the foundation can be designed. At first, the interaction taken into account will be limited: the geotechnical engineer will require data from the

structural engineer regarding the superstructure load and the distribution of these loads over the several elements. It is not until a later stadium that the interaction in element stiffness and redistribution of loads will be taken into account.
Information must be known on the following subjects for the foundations:
  - Structural requirements;
  - Acceptable deformations;
  - Magnitude and distribution of the load;
  - Construction schedule.
- Data on surroundings:
  At first, this will entail information that is already available through historic documentation, project site exploration, known underground obstacles and pipelines, surrounding buildings or other obstacles and a global inventory on the soil layers, ground water level and environment. In general, the required depth of the soil investigation can be based on these data.
- Soil investigation:
  A soil investigation is not only executed on behalf of the foundation design, but will also be used to check whether the soil conditions after installation are still sufficient.
  In current practice, electrical CPTs are performed in order to obtain knowledge on the soil conditions and the bearing capacity of the different soil layers. As the costs of a CPT are relatively low, clearly more CPTs are performed than in former times, like in 1960-1980. The depth of the soil investigation will also be defined by the Geotechnical Category appointed to the structure.

3. **Design requirements**
   A distinction is made between general requirements and design requirements. The general requirements consist of demands regarding the surroundings, execution, quality assurance, financial and juridical aspects.

   The design requirements follow from:

   - The deformational capacity of the superstructure;
   - The interaction between foundation and superstructure;
   - The loads from the superstructure;
   - The soil conditions;
   - The surroundings;
   - The project site and execution;
   - Building physics.

4. **Possible solutions**
   When all necessary data is required, possible solutions are formulated and dimensioned. For piled foundations, the design methodology has not really changed over the years.
   For the pile tip bearing capacity of compressive piles, the method of Koppejan is still applied, based this time on electrical CPT results but still with an influence zone of $0.7 \cdot D_p$ to $4 \cdot D_p$ below pile tip level and $8 \cdot D_p$ above pile tip level. Several reductions have to be applied on the found pile resistance, regarding the shape of the pile tip (β and s) and the method of pile installation (α_p).
   The positive shaft friction is determined based on the electrical CPT results, in which positive friction is obtained from sandy layers that are exposed to little settlement. A reduction factor (α_s) is applied to take the installation effects and pile material into account.

   The negative friction in soft subsoil layers is assumed to be negligible with a settlement less than 0.02 m and to be fully developed when a settlement of more than 0.1 m has occurred. In between, a detailed analysis must be performed on the interaction between pile and soil.

   For single piles or piles in a single line, the negative friction is obtained by the slip method. For pile groups that are approximately evenly distributed the method of Zeevaert-de Beer is used. A reduction in the determined negative friction is possible by treating the pile shaft with a material having a low internal angle of friction with the soil.

   The pile head displacement is determined based on three mechanisms:

   - The settlement of pile tip in order to activate the pile tip bearing resistance and positive shaft friction (s_{tip});
   - The elastic behavior of the pile (s_{el});

- The settlement following from the compression of soft soil layers beneath pile tip level (s_2).

In order to obtain the load-settlement curve of a pile foundation, the pile is schematised as a elastic spring system of which the spring constant depends on:

- The pile load in relation to the bearing capacity;
- The amount of soil displacement;
- The elastic shortening of the pile;
- The (local) presence of soft layers beneath pile tip level;

Furthermore, the spring constant of a pile depends largely on the duration of the load applied. Since soil tends to act stiffer when loaded during a short period of time, the effect of short-term variable loading is much smaller on the settlement of the pile than the effect of static loading.

5. **Design checks**

The uncertainties of calculation and the required safety of the structure are taken into account through multiple safety factors: ξ, γ_M and γ_f. A total safety range of 1.0 – 1.67 is the result.

The design is checked on multiple failure mechanisms, that are either categorised as an ultimate failure criterion regarding the strength of the system (ULS), an ultimate failure criterion regarding the deformation of the superstructure (ULS type B) and a serviceability failure criterion regarding the deformation of the superstructure (SLS).

A general examination procedure can be distinguished for the pile foundation design checks:

1. Determine the Geotechnical Category (see subsection 8.1.1);
2. Determine the representative foundation geometry;

   Both points 1 and 2 are based on the data acquisition for the surroundings and soil investigation (see step 2. Data acquisition).
3. Determine the structural features and load distribution from the superstructure;

   Point 3 is also based on step 2. Data acquisition, but in this case on the project data.
4. Determine the pile type, pile dimensions and construction level;
5. Determine the design value of the pile load;
   This is given by the structural engineer, including the load uncertainty factor γ_f.
6. Determine the design value of the negative shaft friction;
   The design value is given by: $F_{nk;d} = \gamma_{f;nk} \cdot F_{nk;k}$,in which $F_{nk;k}$ is determined by:
   - Single piles or piles in a single line:
     Slip method: $F_{nk;k} = O_{s;gem} \Sigma h_i \cdot K_{0;ik} \cdot tan(\delta_{ik}) \cdot \frac{\sigma_{v;i-1;rep}+\sigma_{v;i;rep}}{2}$
   - Pile groups:
     Method of Zeevaert-de Beer: $F_{nk;k} = A \cdot \Sigma(\sigma_{v;i;sur;rep} + \sigma_{v;i;m;rep})$
7. Determine the maximum pile bearing capacity from a CPT;
   This is based on both the pile tip resistance as the positive shaft friction: $R_{c;cal;i} = R_{b;cal;max;i} + R_{s;cal;max;i}$, with:
   - Pile tip resistance: $R_{b;cal;max;i} = 0.5 \cdot \alpha_p \cdot \beta \cdot s \cdot (0.5 \cdot (q_{c;I;gem} + q_{c;II;gem}) + q_{c;III;gem}) \cdot A_{punt}$
   - Positive shaft friction: $R_{s;cal;max;i} = O_{s;\Delta;gem} \cdot \int_{\Delta L} \alpha_s \cdot q_{c;z;a} \,\mathrm{d}z$
8. Determine the representative pile bearing capacity; The representative pile bearing capacity $R_{c;k}$ is the minimum of:
   - $\frac{(R_{c;cal})_{gem}}{\xi_3}$
   - $\frac{(R_{c;cal})_{min}}{\xi_4}$
9. Determine the design bearing capacity;
   $R_{c;d} = \frac{R_{c;k}}{\gamma_M}$
10. Unity check according to ULS;
    Design check: $V_d < R_{c;d}$
11. Determine the design value for deformations for limit state ULS type B;
    The pile head displacement has three components:

- The pile tip settlement mobilises the shaft resistance and pile tip bearing resistance and is known as s_{punt}. The value of s_{punt} is determined from the load-settlement curve.
- The elastic shortening of the pile s_{el} is given by:
  $s_{el} = \frac{L \cdot F_{gem}}{A_{schacht} \cdot E_{p;mat}}$
- The compaction of deeper soft layers as a result of group effects, s_2 has to be taken into account when the distance between piles is less than $10 \cdot D_p$:
  $s_2 = \frac{0.9 \cdot m \cdot \sigma_{v;4D} \cdot \sqrt{A_{4D}}}{E_{ea;gem}}$

The total pile head displacement is given by: $s = s_1 + s_2 = s_{punt} + s_{el} + s_2$

12. Unity check according to ULS type B;
    Design check: $s_d < s_{req}$ and $\beta < 1:100$
13. Determine the design value for deformations for limit state SLS;
    The pile head displacement is given by $w = w_{punt} + w_{el} + w_2$
14. Unity check according to SLS;
    Design check: $\beta < 1:300$

Points 4 to 14 will have to be considered during step 4. Possible Solutions.

This procedure has been included in the design steps from Figure 8.32 and combined into the overview presented in Figure 8.33.

6. **Choice of solution**
   As can be seen in Figure 8.32, the generic design scheme is not linear, but consists of multiple loops in which the design is more and more elaborated every time. It is divided into the following phases, that itself consist of several loops as well:
   - Preliminary design: a choice is made between a shallow or deep foundation, an indication is given for the construction level and a preliminary cost estimation is included.
   - Final design: the soil investigation is executed and the bearing capacity is calculated.
   - Bill of quantities: this results in a pile plan with the load working on each pile and the amount of reinforcement required.
   - Execution design: elaboration of the foundation design in a detailed manner, in order to obtain detailed execution drawings and instructions.

   After every phase, the amount of possible solutions is reduced based on the obtained quality, required construction time and estimated costs.

7. **Detailed elaboration**
   As mentioned before, the chosen foundation type, material and dimensions that follow from the bearing capacity calculations performed in the final design must be further elaborated in order to enable execution. This means that the details of every structural element must be designed and calculated, the exact costs can be obtained from the bill of quantities and detailed drawings are produced for the execution phase.

8. **Execution**
   During construction and installation of the foundation, it is of utmost importance to maintain oversight of the execution. When deviations are observed, these must be processed into new as-built designs in which the safety and reliability of the deviations is proven. If necessary, additional actions must be taken in order to guarantee the quality of the structure.

9. **Follow-up**
   In a follow-up, the details of the execution must be gathered into a report. In this report, information is presented on the real situation after completion, thus including revised drawings and revised calculations.

Whilst in 1960-1980 the calculations for designing a foundation were performed entirely by hand, in current practice it is common to make use of computer programs that automatically asses the imported CPTs, determine the required pile tip level and calculate the bearing capacity of the pile. Within Heijmans, the program D-foundations is used for this application.

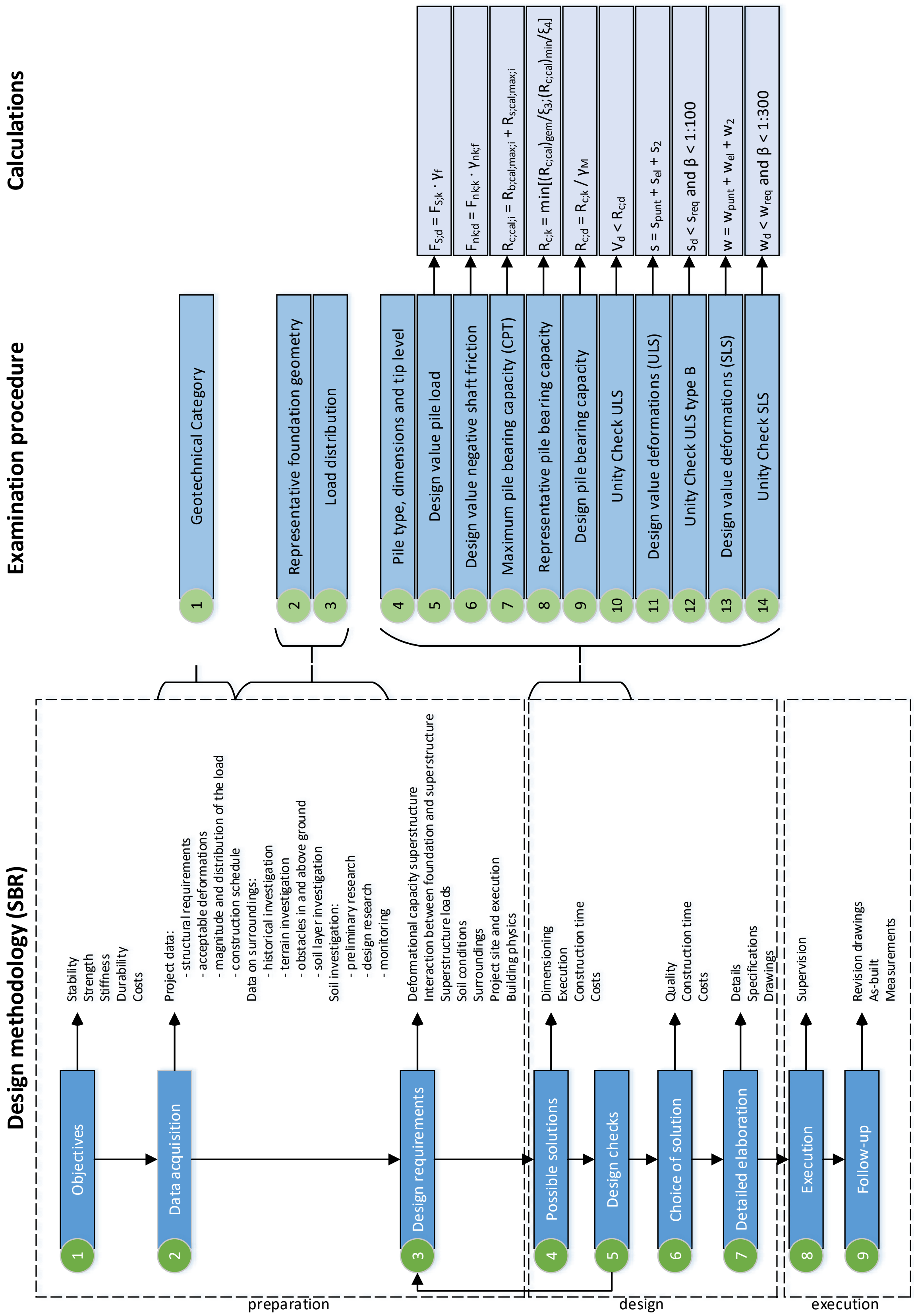


FIGURE 8.33: Overview of design steps, examination procedure and calculations in a pile foundation design *[SBR, 2012]*

8.5. RELIABILITY OF PILE BEARING CAPACITY

In subsection 8.1.7 it was mentioned that there are some uncertainties in the semi-probabilistic approach of pile bearing capacity since introduction of TGB 1990. The discussion started with the adaption of partial factors, such that they agreed with the safety margin from Dutch practice up to then, and is still going on nowadays. It not only includes the several uncertainties in geotechnical design, but also the presence of so-called hidden safeties and the relation between proof loading tests and design rules. In the following section a chronological overview of this discussion is given, in which *[van Tol, 1994]* is the starting point.

Reliability, uncertainties and hidden safeties

With the introduction of codes and regulations for geotechnical structures in TGB 1990, probabilistic analyses have been performed on the loads and bearing capacity of geotechnical structures. A required reliability has been stated in the Safety Classes and partial safety factors have been determined to eliminate the need for lengthy calculations for every design.

For piled foundations it turns out, however, that if all uncertainties are taken into account correctly in this semi-probabilistic analysis, the safety margin for the required failure probability is much higher than the normal Dutch practice up to that date (1.96, as elaborated in section 8.3). As there was no indication of this Dutch practice to be unsafe, it was decided to reduce the partial safety factors to such extent that the final safety before and after code introduction would be more or less the same.

The question remains whether this adaption has adequately taken into account the uncertainties in geotechnical design. According to a rough estimation made by *[van Tol, 1994]* on the amount of damaged foundations in the period between 1965 and 1994, the probability of failure is of the same order of magnitude as prescribed in TGB 1990. It can thus be concluded that the safety margin already applied in Dutch practice was sufficient and that either the uncertainties are smaller than expected or there are some hidden safeties present in the system.

Geotechnical design is not an exact science, but is mostly based on empirical relations and experience. This means that there are several known uncertainties that will have influence on the design values and that have to be taken into account as complete as possible. The uncertainties can be found in:

- Scatter of soil conditions over the project location;
- Empirical models and parameters;
- Dispersion in load actions;
- Execution on-site.

Through extensive probabilistic analyses these uncertainties have been taken into account by the application of partial safety factors (Figure 8.28):

- Correlation factor ξ takes into account the scatter in soil conditions from the heterogeneity of the soil and limited amount of samples taken at the project location. Depending on the amount of samples taken, the statistical factor ξ varies from 0.75 to 0.92 (coefficient of variation is 0.1);
- Material factor γ_M covers the empirical nature of the design models and parameters, and thus also deficiency of execution. For driven concrete prefabricated piles, $\gamma_M = 1.25$. . It must be said however that the variation coefficient of 0.15 that is taken into account for γ_M is lower than reality. Further research must be conducted to reach an agreement between the probability of failure and current practice. Regarding the execution, it allows for correction of errors made in the design but there are also some implementation flaws that have to be prevented during construction, like pile fracture or insufficient concrete cover;
- Load factor γ_f takes into account the dispersion in load action and is taken as $\gamma_f = 1.2$ and $\gamma_f = 1.5$ for permanent and variable actions respectively. The characteristic value is defined as the 5-percentile of the load distribution.

The expectation is, however, that there are some hidden safeties present in geotechnical design that are not taken into account, but increase the safety of the system irrespective of the uncertainties taken into account. These are:

- Redistribution of loads and interaction between foundation and superstructure;
- There is a discrepancy between the method of Koppejan, based on a mechanical CPT and the current practice in which electrical CPTs are used;
- Presence of residual stresses at pile base;
- Ultimate capacity of pile is defined at $w = 0.1 \cdot D_p$;
- Group effects:

  - Increase of vertical effective stress from superstructure load;
  - Installation effects increase capacity of driven piles in groups.
- Increase of bearing capacity in time (set-up);
- CPT cut-off limit;
- base effect from positive friction;
- Influence of test methods;
- Combination of wind load with negative friction.

These hidden safeties will be further elaborated later on in this section.

The tendency arises to execute research into these hidden safety as to obtain more economic designs, but unless this is accompanied by a decrease in uncertainties, the overall safety of the system will be diminished. Research must therefore be conducted on both uncertainties as hidden safeties, if a more economical design is desirable, without loss of reliability.

Reduction of pile tip bearing capacity factor α_p

The method of Koppejan for the determination of the pile bearing capacity, was introduced by A.W. Koppejan in 1952 as a result of a study into the failure behaviour of pile foundations. In this study, the considered piles were driven through the weak Holocene subsoil layers into the bearing sand layer over just a small distance of several times the pile diameter. The failure behaviour was then compared to the results obtained from a mechanical CPT. The failure limit was taken as the vertical asymptote of the accompanying load-settlement curve of the pile tip. This failure criterion has been altered in the course of time; nowadays, current practice defines pile failure if the pile head has a displacement of 10 times the equivalent pile diameter.

The shaft resistance has been introduced by Begemann in 1969, as the result of comparing the tensile tests and mechanical CPT results with the pile shaft resistance factor α_s. This factor is dependent on the pile type and can be used as a direct function of the measured skin friction and the expected shaft resistance.

Following the introduction of the TGB 1990, Grondmechanica Delft *[Geerling and Stoevelaar, 1993]* has performed a new research into the pile bearing capacity of prefabricated concrete piles by means of proof loading tests. The intended goal was to evaluate the common design rules through a separate measurement of pile tip behaviour and shaft resistance. Although the amount of proof loading tests was limited since there were only three suitable samples, it could be concluded that the pile behaviour for long piles differs from the calculations as prescribed by the TGB 1990. Depending on the failure criterion taken into account, only 60% ($w_{ult} = 0.1 \cdot D_p$) to 90% ($w_{ult} = 0.5 \cdot D_p$) of the calculated pile tip bearing resistance can be obtained. A possible cause for this abnormality is the difference in CPTs performed, since the method of Koppejan and Begemann were based on a mechanical CPT but electrical CPTs were used for this research.

Naturally, the reached conclusion caused some alarm within the geotechnical community and a discussion on the value of pile resistance factors and the reliability of piled foundations followed that is still not settled. Several studies have been performed on the possible reduction of uncertainties and utilisation of hidden safeties.

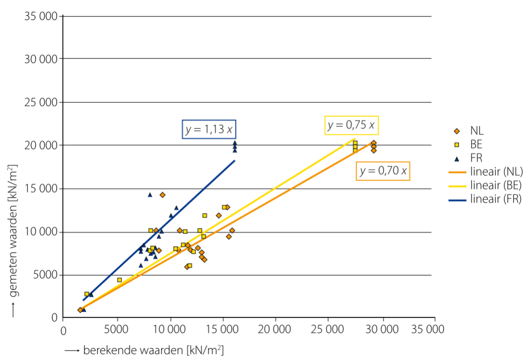


FIGURE 8.34: Measured versus calculated pile tip bearing capacity according to the Dutch, Belgium and French design codes *[van Tol, 2015]*

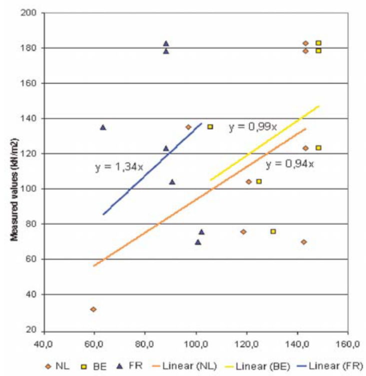


FIGURE 8.35: Measured versus calculated pile shaft bearing capacity according to the Dutch, Belgium and French design codes *[van Tol et al., 2010]*

More research has also been conducted on the correct values of the pile bearing resistance factors α_p and α_s. In 2010, a study has been conducted by the CUR-committee into the bearing capacity of axially loaded piles based on previously executed proof loading tests *[van Tol et al., 2010]*. A comparison was made between the results of different calculation methods as applied in Holland, France and Belgium, based on CPTs. The biggest difference can be seen in the influence zone taken into account for the determination of the cone resistance and the pile bearing resistance factors. Although proof loading tests have been performed on several pile types, it turned out that only the tests performed on soil displacement pile were usable and of sufficient quality. The requirements for this were a separately measured pile tip and shaft bearing capacity, a sufficient pile tip settlement of at least $0.1 \cdot D_p$ and suitable soil investigation performed in the vicinity of the proof loaded pile.
From this comparison between measured and calculated pile tip and shaft bearing capacity, it follows that the Dutch code overestimates the pile tip resistance by 30% (Figure 8.34), while the shaft resistance is slightly underestimated by 6% (Figure 8.35). Furthermore, a clear dependency can be distinguished between the ratio of measured and calculated pile tip bearing resistance and the depth to which the pile has been driven into the sand. It shows (Figure 8.36) that for piles with a length of less than $8 \cdot D_p$ in the sand, this ratio lies just below 1.0, while for piles with a length of more than $8 \cdot D_p$ in the sand, this ratio has decreased to 0.6. The results have been summarised in Table 8.13. Another aspect that stands out is the (much) larger coefficient of variation that was found for the pile tip bearing capacity in different situations.

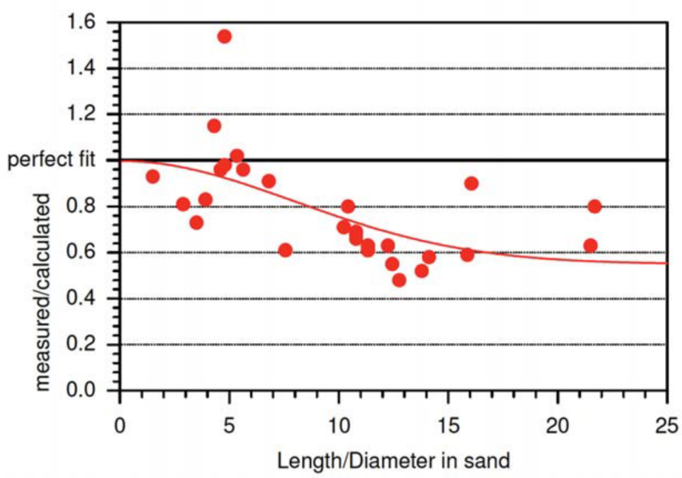


FIGURE 8.36: Ratio between measured and calculated pile tip bearing resistances as a function of the pile depth in the sand *[van Tol, 2012]*

Penetratie		*Gemiddelde α_p-waarde*	*Standaard afwijking (σ)*	*Variatie coëfficiënt (v)*
< 8 D	Zonder limiet	0,989	0,27	0,27
	Met limiet $q_{b;max}$ < 15 MPa	0,992	0,27	0,28
> 8 D	Zonder limiet	0,635	0,08	0,12
	Met limiet $q_{b;max}$ < 15 MPa	0,834	0,35	0,42

Tabel 3 Gemiddelde α_p voor ondiepe en diepe palen met en zonder limietwaarden.

Schachtdraag-vermogen		*Gemiddelde α_p-waarde*	*Standaard afwijking (σ)*	*Variatie coëfficiënt (v)*
	Met limiet $q_{b;max}$ < 150 kPa	0,0099	0,0036	0,36

Tabel 4 Gemiddelde α_p met limietwaarden.

TABLE 8.13: Average values of $alpha_p$ and α_s for different depths and with or without cut-off limits *[van Tol et al., 2010]*

An additional result was found with regards to the application of the French codes, in which and influence zone of $-1.5 \cdot D_p$ to $+1.5 \cdot D_p$ around pile tip level is taken to determine the average cone resistance. It appears that this averaging, when compared to the Dutch codes where and influence zone of $-4 \cdot D_p$ to $+8 \cdot D_p$ is taken, is not suited for a layered soil composition. The study by *[van Tol et al., 2010]* proposes two possible improvements for the Dutch code regarding prefabricated concrete piles or closed steel tubular piles:

1. For homogeneous soil over an area of $-4 \cdot D_p$ to $+8 \cdot D_p$ around pile tip level, it is allowed to apply the French code, in which α_p is reduced to $\alpha_p = 0.6$.
2. The Dutch code is applied, but a distinction is made between shallow piles ($< 8 \cdot D_p$ in the sand) and deep piles ($> 8 \cdot D_p$ in the sand), in which α_p must be taken as $\alpha_p = 1.0$ and $\alpha_p = 0.6$, respectively.

Although the reduced pile factors are in theory only based on soil displacement piles, a lack of further research into the reduction for different pile types has led to a general adaptation of the Dutch code on January 1st 2017. In the new code, the pile tip resistance factor α_p has been reduced by 30% for all pile types *[van Tol, 2015]*.

There is a discrepancy however between the required adaption of pile bearing capacity calculation and the experiences from Dutch practice. After all, although this capacity has been overestimated by 30%, there are no known cases in which the foundations have turned out to be insufficient. Therefore, there must be hidden safeties present in the system that should be identified and quantified In order to (partly) compensate the reduced pile tip resistance factor *[van Tol, 2012]*.

Further research into hidden safeties

As mentioned earlier, several hidden safeties have been proposed that could increase the bearing capacity of the pile foundation. In the past decades, multiple studies have been conducted into the nature and potential of these mechanisms. A short elaboration is given below:

- Redistribution of loads and interaction between foundation and superstructure *[van Tol, 1994]*;
  If a single pile has an insufficient bearing capacity, it will experience a relatively large displacement compared to the surrounding piles. The result is that a redistribution of forces will take place where the surrounding piles will take over a part of the load and thus no (or less) damage will occur to the structure. More insight into this mechanism could mean a more efficient design, but also a reduction of the unknown scatter in the load.

  The increasing possibilities in numerical methods over the last years have resulted into more complex load distribution calculations in which the load on individual foundation elements, depending on their relative stiffness and the stiffness differences in the superstructure.

- Mechanical and electrical CPT results *[van Tol, 1994][Geerling and Stoevelaar, 1993][SBR, 2012]*;
  There is a discrepancy between the method of Koppejan, based on a mechanical CPT and the current practice in which electrical CPTs are used. Higher cone resistances are obtained when measuring with an electrical CPT, which can reach up to 25% higher than the mechanical CPT results depending on the soil conditions.

  On the other hand, thin layers with low cone resistance are normally not registered in mechanical CPTs, which more or less compensates the difference in measured cone resistance between a mechanical and electrical CPT. This does mean, however, that a overestimation of cone resistance might occur in the case that there are (almost) no setbacks from small layers.

- Presence of residual stresses at pile base *[Everts, 2015][CUR-commissie C193, 2012]*;
  After driving soil displacement piles into the bearing layer, residual stresses may occur in the pile tip due to rebounding of the soil and an accompanying elongation of the pile. Since these residual stresses are not taken into account when determining the pile bearing capacity, the presence of residual stresses might lead to an overestimation of the bearing capacity.

  It appears, however, that the influence of residual stresses is not the cause for a decreased pile capacity as measured in the proof loading tests by the CUR-committee. These piles were driven into the bearing sand layer up to a length of $25 \cdot D_p$, whilst the influence of residual stresses is only substantial for piles with a length of $> 30 \cdot D_p$ in the sand.

- Ultimate capacity of pile is defined at $w_{ult} = 0.1 \cdot D_p$ *[Everts, 2015]*;
  The failure criterion of $0.1 \cdot D_p$ that has been defined in the current design codes, is an arbitrary limit

based on a practical consensus. From the study performed by *[Geerling and Stoevelaar, 1993]* in which proof loading test on prefabricated concrete piles were conducted, it follows however that a larger displacement is needed to approximate the maximum pile bearing capacity conform the method of Koppejan.

The expectation is however, that the failure criterion as defined in the codes ($w_{ult} = 0.1 \cdot D_p$) cannot be altered in order to obtain a more reliable method that agrees more with the method of Koppejan. The study by *[Geerling and Stoevelaar, 1993]* speaks of a displacement of $0.5 \cdot D_p$ that is needed to obtain a pile bearing capacity that is 90% of the capacity determined through the method of Koppejan. As most prefabricated concrete piles will fail around or even before reaching a displacement of $0.1 \cdot D_p$, as stated by *[van Tol, 2017]*, an increased failure criterion of $0.5 \cdot D_p$ will in practice never be reached.

- Group effects *[Everts, 2015][CUR-commissie C193, 2012]*;
  The current regulations for determining the bearing capacity of piles is based on the installation of a single pile. Research has shown however, that there are two positive effects on the bearing capacity of piles installed in a pile group:
  - Installation effects increase capacity of driven piles in groups:
    When driven piles are installed, the soil displacement causes compaction of the soil by reducing the volume of the soil skeleton around the pile shaft. This implies a reduction of pore volume and thus an increase of relative density, which is directly related to an increase in cone resistance. This direct relation is defined as the compaction factor f_1, which is based on the soil composition within an arbitrary influence zone of $6 \cdot D_{eq}$. For tension piles, the factor f_1 has already been included in the regulations, in which the effect depends on the initial relative density of the soil as give in Table 8.14. Although this has not yet been incorporated in the regulations for compression piles, based on practical experiences it seems justified to apply the compaction factor f_1 as calculated in CUR-publication 2001-4 Trekpalen to compression piles as well.

Relatieve dichtheid R_e zand	Verdichting door heien	Volumeafname door verdichting	Horizontale opspanning door grondverdringing	Effect op schachtwrijving
Laag	++	++	- tot +	+
Matig	+	+	+	+
Hoog	- [1])	- tot - -	++	++

Legenda
++ Veel; - Weinig; [1]) eerder verbrijzeling
\+ Matig; - - Zeer weinig

TABLE 8.14: Expected effect of installation of a soil displacement pile in a pile group *[CUR-commissie C193, 2012]*

  - Increase of vertical effective stress from superstructure load:
    A part of the superstructure load that is present on compression piles is transferred to the subsoil layers through shaft friction. This leads to an increase of vertical stress on the bearing layer, defined as the factor f_2, which is not explicitly taken into account in the determination of bearing capacity. Again, it is already incorporated in the regulations for tension piles, but in order to apply it to compression piles, further analyses is needed of the behaviour of compression piles in a pile group.

- Increase of bearing capacity in time (set-up) *[CUR-commissie C193, 2012] [van Tol, 2012] [van Tol, 2017]*;
  A lot of research has been performed on the increase of bearing capacity over time of drive pile foundations in sand, the so-called set-up. Although most researches were based on steel tubular tension piles, *[Axelsson, 2000]* has performed a study on prefabricated concrete compression piles (Figure 8.37). A substantial increase of bearing capacity up to 50% can be seen for a failure criterion of $0.1 \cdot D_p$, with D_p = 235 mm. From Figure 8.38 it follows, however, that this increase largely depends on the increase of shaft resistance, since the increase of pile tip resistance is only responsible for a 10% increase of total bearing capacity. In turn, the increase of shaft resistance is partly the result of an increase of horizontal effective stress in unloaded state (25%) and partly the result of ageing in loaded state (75%). Ageing is defined as a change of soil conditions at constant effective stress as a result of friction, mechanical or chemical effects. In soil, the increase of shear stress causes a higher viscosity and thus an increased

shear resistance. Therefore, the 75% increase caused by ageing is not time-dependent but a result of the increase in load.

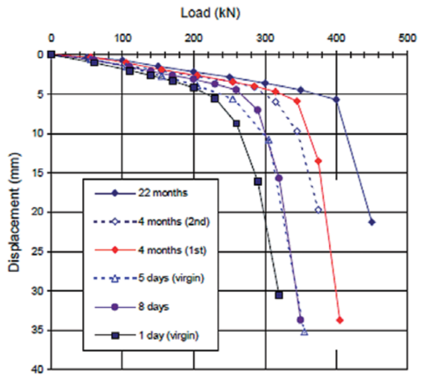


FIGURE 8.37: Pile head load-displacement curve from static tests *[van Tol, 2012]*

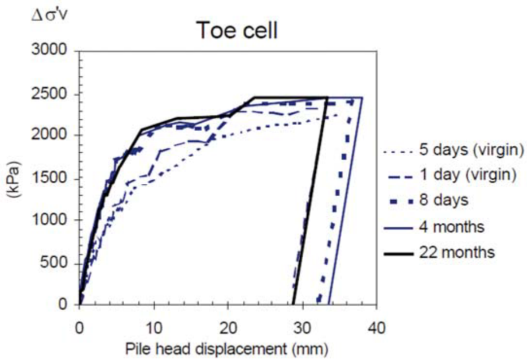


FIGURE 8.38: Measured pile tip resistance as a function of head displacement from static tests *[van Tol, 2012]*

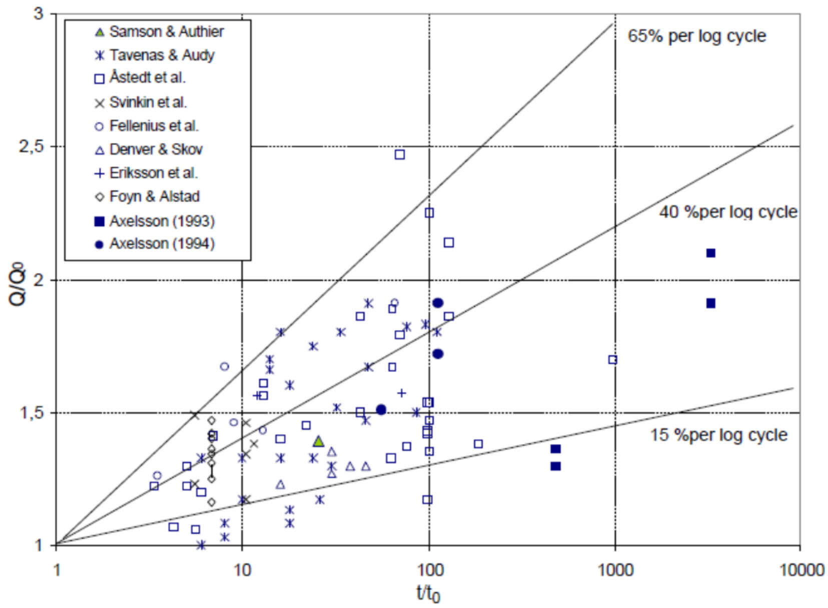


FIGURE 8.39: Case histories of long-term pile set-up *[Axelsson, 2000]*

It has been attempted to formulate an empirical expression that gives the set-up as a function of time and type of soil. The results of multiple studies have been plotted in Figure 8.39, from which the following formula is derived:

$Q_t = Q_0 \cdot (1 + A \cdot log_{10}(t/t_0))$

in which Q_0 is the initial bearing capacity, A is a function of the soil type and t_0 is the time at which Q_0 is obtained.

The set-up depends on the following parameters:

- The relative density and stiffness of the soil;
- The grain distribution;
- The grain strength;
- The moisture content;
- The stress level;
- The installation process;
- The pile diameter.

However, the exact dependence of these parameters is unknown and a consensus on the value for A and t_0 has not been reached yet. At the moment, the lower and upper bound limit as presented in Figure 8.39 are taken as 15% and 65%, respectively. This leads to a conservative approach that can be used in practice, in which the parameters A and t_0 are taken as A = 0.1 for sand and t_0 = 1 day. Another unknown is the influence of varying loads on the set-up behaviour. Studies have shown that varying loads decrease the development of set-up over time, but for instance it is not known to what extent this depends on the ratio between varying load and static load.

- CPT cut-off limit *[CUR-commissie C193, 2012] [van Tol, 2012]*;
  In the Dutch regulations, a cut-off limit for the maximum cone resistance is prescribed at 15 MPa for the pile tip resistance and 150 kPa for the shaft resistance. This cut-off limit is based on proof loading tests performed in Holland, but from the available literature it follows that there is no reason to increase this limit for the pile tip resistance. For the shaft resistance, it might be an option, but this has to be elaborated in further research.

- Base effect from positive friction *[CUR-commissie C193, 2012]*;
  A high positive friction can lead to fixture of the soil under the pile tip when installing a soil displacement pile. This is known as base-effect, but it is unknown whether this has a positive or negative influence on the bearing capacity, or whether it has any influence at all.

- Influence of test methods *[CUR-commissie C193, 2012]*;
  When performing proof load tests, the reaction forces are measured at a prescribed distance from the sample. In the current calculation model, the influence of vertical stress decrease from the reaction piles through the factor f_2 is not taken into account but would lead to a underestimation of the actual bearing capacity. Therefore, a study should be performed into this effect that results in a correct prescription of distance between sample and reaction piles.

- Combination of wind load with negative friction *[CUR-commissie C193, 2012] [van Tol, 2012]*;
  In current regulations, wind loading and negative friction are both taken into account as loading on the pile foundation. The question is, however, whether a reduction can be applied for the combined activity. It appears that when the negative friction has developed over all layers, the wind load is partly transferred to these weak layers and not entirely to the bearing sand layer. The distribution depends on the differences in strength and stiffness of pile, soil and the interaction zone but can result in a reduction of 30-50%. Since the wind load is not substantial (>10% of total load on buildings with a height lower than 40 m and the negative friction should be fully developed, this is not a generic hidden safety.

It follows that in terms of utilising hidden safeties, in order to reduce the impact of the 30%α_p factor reduction, the group effects and the set-up of soil have the highest potential.

8.6. POSSIBILITIES IN REUSING EXISTING FOUNDATION FOR BRIDGE REPLACEMENT

In the previous sections, information is gathered on geotechnical foundation design in general (sections 8.1 and 8.2) and more specifically the design methodologies in the period 1960-1980 (section 8.3) and the last decade (section 8.4). Furthermore, the reliability of pile bearing capacity and he accompanying uncertainties and hidden safeties were discussed in section 8.5. With this information, the most potential influences, both positive and negative, on the possibilities for reuse of bridge foundations can be identified and judged.

It must be said, however, that up to now this chapter has only focussed on aspects of geotechnical design, while also structural aspects influence the potential reuse of foundations. Despite the fact that background information is not given in this Literature Study, they will be included in the following section.

8.6.1. Current practice in reusing foundations

Before a conclusion can be drawn on the potential mechanisms that can be profitable for foundation reuse, it is beneficial to look at the current practice of a replacement design task. In other words, which differences in the design will decrease the amount of additional new piles needed in a bridge replacement task?

Design philosophy of a bridge replacement task

For this deduction, a bridge replacement task is reviewed. The existing bridge has a certain load, $F_{1;S}$, that has successfully been carried over the last 50 years by a pile foundation consisting of n_1 piles. Each pile of this existing foundation is assumed to have a calculated pile capacity, $P_{1;R;cal}$, according to the ' old' code and an overall safety defined by γ_1. The following relation thus holds for the existing situation (designated by the subscript 1):

$$P_{1;R;cal} \cdot n_1 = \gamma_1 \cdot F_{1;S}$$

The existing bridge has to be replaced and the question is whether the existing foundation has sufficient bearing capacity to carry the new load.

The new loading situation is denoted by $F_{2;S}$, and if this new situation would be obtained by building an entirely new foundation, according to the current codes, the amount of piles and the pile bearing capacity of a single pile would be given by n_2 and $P_{2;R;cal}$, respectively. As this foundation is calculated according to the current code, the overall safety of the system is given by γ_2. Thus, for the new situation (designated by the subscript 2), the following would hold:

$$P_{2;R;cal} \cdot n_2 = \gamma_2 \cdot F_{2;S}$$

However, in a bridge replacement task it is assumed that the existing piles will be reused. Therefore, the difference in loading between the old and new situation, $F_{dif;S}$, must be accounted for. If additional piles are needed, these are defined as m_i.

Before analysing the different options, several starting points are defined. It is assumed that:

- The original load on the foundation, $F_{1;S}$, is known;
- The original amount of foundation piles, n_1, is known;
- The new load on the foundation, $F_{2;S}$, is known;
- The difference in loading between the old and new situation, $F_{dif;S}$, is known: $F_{dif;S} = F_{2;S} – F_{1;S}$;
- The existing foundation has proven to be adequate to resist the original load $F_{1;S}$;
- The current code is more reliable than the old code.

Three situations can occur with regards to the bearing capacity of the existing piles:

1. The bearing capacity of the existing piles is based on the proven load resistance, namely $F_{1;S}$.
2. The bearing capacity of the existing piles is based on new calculations based on CPT results and the current design code.
3. The bearing capacity of the existing piles according to the original calculations is known.

The difference in load between the new and the existing situation will then lead to the following amount of additional piles needed:

A. The new bridge has a lower or equal load that has to be transferred to the foundation:

$$F_{2;S} \leq F_{1;S} \text{ or } F_{dif;S} \leq 0$$

Since the existing foundation has already proven to be adequate for load $F_{1;S}$, it will be adequate for load $F_{2;S}$.

B. The new bridge has a higher load that has to be transferred to the foundation:

$$F_{2;S} > F_{1;S} \text{ or } F_{dif;S} > 0$$

The increase of load will have to be carried by additional piles:

1. The easiest way is to base the bearing capacity of the existing foundation on the proven resistance: $F_{1;S}$. This means that the increase of load must be carried by additional piles, which are dimensioned based on the current codes:

   $$m_1 = \frac{F_{dif;S} \cdot \gamma_2}{P_{2;R;cal}}$$

   This will be known as the baseline.
2. A profit in required additional piles can be found by assessing the bearing capacity on the existing piles based on a new calculation. The theoretical allowed loading on the existing foundation then is:

   $$F_{1;S;cal} = \frac{n_1 \cdot P_{2;R;cal}}{\gamma_2}$$

   - If the existing foundation has an overcapacity, $F_{1;S;cal} > F_{1;S}$, this means that the difference in loading $F_{dif;S}$ decreases and thus the amount of additional piles decreases:

     $$m_2 = \frac{F_{dif;S} \cdot \gamma_2}{P_{2;R;cal}} < m_1$$

   - If the existing foundation has a theoretical undercapacity, $F_{1;S;cal} < F_{1;S}$, this means that the difference in loading $F_{dif;S}$ increases and thus the amount of additional piles increases:

$$m_2 = \frac{F_{dif;S} \cdot \gamma_2}{P_{2;R;cal}} > m_1$$

However, since the existing foundation has already proven to be sufficient for load F1;S, it can also be assumed to be sufficient for the smaller load F1:s:cal. Therefore:

$$m_2 = m_1$$

A footnote must be placed, however, that if the exact pile tip level of the existing pile is not known, an extra uncertainty factor ξ must be taken into account, thereby decreasing the reduction of needed additional piles:

$$F_{1;S;cal} = \frac{n_1 \cdot P_{2;R;cal}}{\gamma_2 \cdot \xi}$$

3. If the originally calculated bearing capacity is known from historical documents, a profit in required additional piles can also be found, now by assessing the overall safety applied on the bearing capacity calculation of the existing piles. The applied safety on the existing foundation is:

$$\gamma_1 = \frac{n_1 \cdot P_{1;R;cal}}{F_{1;S}}$$

   - If the original applied safety is larger than the currently prescribed safety, $\gamma_1 > \gamma_2$, this means that the allowable load actually is higher than the originally applied load, $F'_{1;S} > F_{1;S}$. Thus the difference in loading $F_{dif;S}$ and therefore the amount of additional piles increases:

$$m_3 = \frac{F_{dif;S} \cdot \gamma_2}{P_{2;R;cal}} < m_1$$

   - If the original applied safety is smaller than the currently applied safety, $\gamma_1 < \gamma_2$, the actually allowable load is smaller than the originally applied load and the foundation would have collapsed or damage would have occurred. Since this did not happen, the amount of additional piles needed is:

$$m_3 = m_1$$

The analysis has led to the scheme as presented in Figure 8.40.

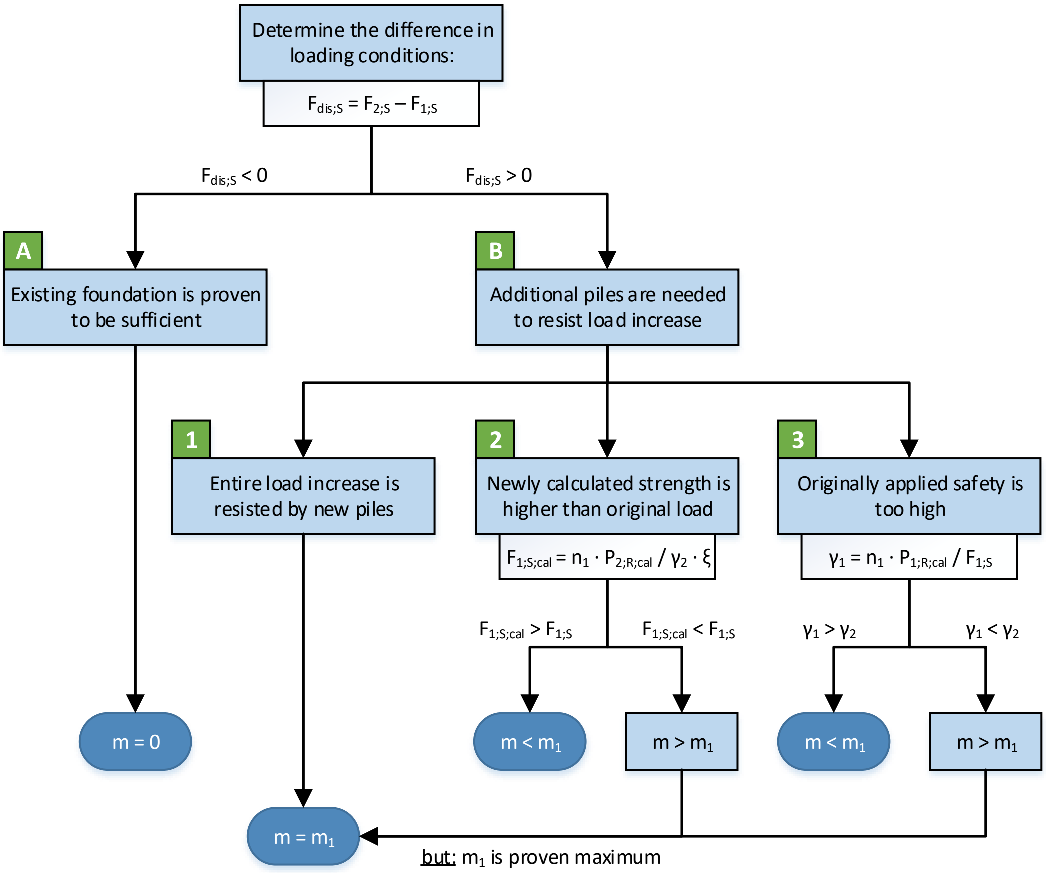


FIGURE 8.40: Scheme of the possible profit obtained when comparing design methodologies from 1960-1980 and current practice

The following conclusions can be drawn:

- If the new load is equal to or smaller than the original load, the existing foundation is sufficient and no additional piles are needed;
- If the new load is higher than the original load, $m_1 = F_{dif;S} \cdot \gamma_2 / P_{2;R;cal}$ additional piles will have to be placed.
- The amount of additional piles needed can be decreased if the following can be proven:
  - The newly calculated strength of the existing foundation is higher than the original load. In other words, prove that:
    $F_{1;S;cal} > F_{1;S}$ thus $\frac{F_{2;R;cal}}{\gamma_2 \cdot \xi} > \frac{F_{1;R;cal}}{\gamma_1}$
  - The safety applied in the original calculation is too high and a lower safety will suffice. In other words, prove that: $\gamma_1 > \gamma_2$

When comparing this analysis to the current practice, in which the existing foundation would completely be replaced by a new foundation, it can be seen that the design phase will require additional work, therefore increasing the project costs. The reduction of additional piles needed, however, a profit expressed as $p = n_2 - m_i$, does not only lead to a reduction of material costs, but also to a reduction of execution costs. This profit will almost certainly weigh up to the increased design costs.

Levels of geotechnical analysis for reuse of foundations

From the design philosophy of bridge replacement tasks as presented above and in Figure 8.40, it can be concluded that there are different levels at which a foundation reuse can be analysed. These levels can be schematised as being a pyramid, see Figure 8.41, in which the scope of the research increases with every level in order to decrease the amount of additional piles needed.

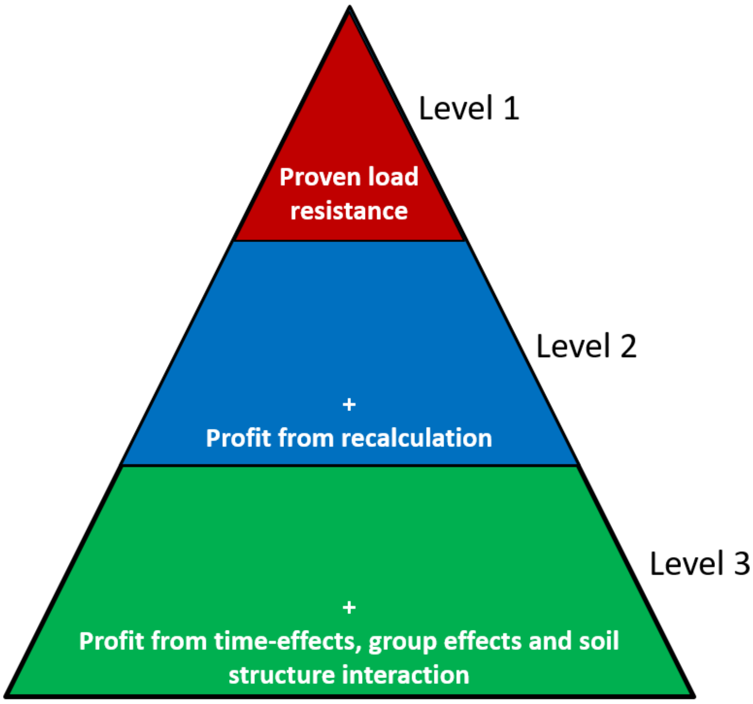


FIGURE 8.41: Different levels of geotechnical analysis for the reuse of foundations

- The first level assumes a proven load resistance from the original superstructure loading. If the new load distribution results in a smaller load than that was first present on the foundation, the foundation has already proven itself and thus the foundation can be reused. If, however, the new superstructure loading is larger than that was first present, the difference in loading will have to be taken up by newly added additional piles that are designed according to the current codes.
- The second level assumes the same baseline as level 1; that of the proven load resistance, but it aims to decrease the amount of additional piles needed through recalculation of the existing foundation.

Profit can for instance be obtained if a favourable spring constant can be assumed from short-term variable loading (as a result of traffic load) *[van Cann et al., 2017]* or if the bearing capacity increases from applying the new design codes.

- The third level aims to further increase the profit gained in level 2, by including time-effects, including group effects and obtaining a more favourable soil-structure interaction. This level is currently not implemented in practice and making use of this level would thus increase the potential of reusing existing foundations even further.

It follows that the deeper the extent is of the applied analysis on the reuse of foundation, the larger the potential profit on the amount of additional piles can become.

8.6.2. Fields of potential profit

When taking into account the information given in sections 8.1 to 8.5, in particular the sections on design methodology and reliability of the pile bearing capacity, three fields can be distinguished in which potential profit in foundation reuse can be obtained:

1. The calculation of pile bearing capacity;
2. The load distribution from the superstructure;
3. Time-effects.

These fields will be discussed into more detail below:

1. The calculation of pile bearing capacity;
   First of all, a clear reduction is observed from the adaptation of the pile tip resistance factor α_p since January 1^{st} 2017. This might be compensated by the fact that there was a higher overall safety in foundation design in 1960-1980 compared to current practice. Furthermore, the positive shaft friction was often neglected and simplified models were used to determine the piles spring behaviour. At last, if the group effects can be taken into account and the uncertainties $gamma_M$ and ξ can be lowered, this will also give rise to an increased pile bearing capacity.

2. The load distribution from the superstructure;
   When looking at the loads of a new superstructure on the existing foundation, an increase of traffic load and self-weight can be observed. The latter is mainly the result of the removal of intermediate supports, thereby transferring a larger part of the total load to the end supports. This increase can be somewhat confined by applying a UHPC superstructure that has an overall weight reduction. However, since the negative shaft friction has to be taken into account as well, there is a high possibility of overall load increase.
   By making use of soil structure interaction, it might be possible to obtain a more favourable load distribution. Numerical modelling can be used to spread the load more evenly and by allowing a higher overall settlement or differential settlements the bearing capacity of the pile foundation will increase. Also, weaker piles can be supported by stronger piles if the stiffness of the pile cap is such that it allows redistribution of loads. Another method to obtain a more favourable load distribution on the foundation piles, is by altering the static scheme of the end supports. In the old situation, a moment was generated and thus both tension and compression piles had to be applied. If, by eliminating this moment, it is possible to convert the tension piles into compression piles, the overall bearing capacity of the existing foundation will increase substantially.

3. Time-effects;
   If it can be proven that the set-up over time has increased the bearing capacity of the soil, large profits can be gained on the pile bearing capacity. For example, if the pile bearing capacity is given by both shaft resistance and pile tip resistance for 50% each, a set-up of 10% per decade is needed to entirely compensate the reduction of α_p, which is lower than the currently proposed lower boundary.
   Although the material strength is probably not governing for the bearing capacity of the existing foundation, time has had some effects on the prefab concrete piles. For instance, ongoing hydration of the concrete will have led to a higher compression strength, while deterioration effects might have reduced the durability of the pile. Furthermore, time-dependent effects like creep, shrinkage and relaxation will all have reached more or less stable values and will not change substantially any more.

The aspects listed above are summarized in Table 8.15, according to their influence field and the level of geotechnical analysis.

	Effect	Level
Bearing capacity of pile foundation		
Decrease of α_p by 30%	-	2
Decrease of overall safety	+	2
Include positive shaft friction	+	2
Numerical modelling pile spring constant	+	2
Densification from group effects	+	3
Lower uncertainties (γ_M and ξ)	+	3
Superstructure loading		
Increase of traffic load	-	1
Increase of overall load due to removal of supports	-	1
Include negative shaft friction	-	1
Decrease of self-weight from UHPC	+	(1/3)
Change static scheme of supports	+	(1/3)
Make use of the soil structure interaction:		
Use numerical modelling	+	3
Allow higher differential settlement	+	3
Increase stiffness pile cap for larger pile group	+	3
Allow higher settlements	+	3
Time-effects		
Include set-up of soil over time	+	3
Include material time-effects:		
Strength improvement from ongoing hydration	+	3
Strength decrease from degradation mechanisms	-	3
Take into account creep, shrinkage and relaxation	+	3

TABLE 8.15: Potential influences on the pile bearing capacity of existing foundations and the accompanying level of geotechnical analysis

8.6.3. Conclusions on the potential of reusing foundations

To conclude this section on the possibilities of reusing existing foundations in a bridge replacement task, the pyramid level structure from Figure 8.41 and the list of potential influences from Table 8.15 are combined into a decision diagram for geotechnical and structural analysis. This decision diagram is given in Figures 8.42 and 8.43 and it gives a stepwise approach for the feasibility of foundation reuse.

The first step is to analyse the load distribution in the new situation. If the load has not increased compared to the old situation, the existing foundation can be assumed to be sufficient, since it has already proven to have this strength. If the load increases compared to the old situation, however, the existing foundation might not be strong enough. The easiest solution, without additional designing labour, is to strengthen the existing foundation by adding extra piles. The amount of piles, m_1, is based on the new calculations codes and will bear the entire load increase of the new situation: $m_1 = \gamma_2 \cdot \frac{F_{2;S}}{P_{2;R;cal}}$.
If, however, it is decided that this amount of additional piles, m_1, is too much, profit can be obtained by looking at the second step of Figure **??**, by determining the bearing capacity of the existing foundation. This can either be done by measuring the real bearing capacity (path a) or by a theoretical calculation of the pile resistance (path b). If possible, measuring the existing foundation capacity is clearly preferred above calculating, because this gives real-time information on the strength of the piles. In either case, however, depending on the determined pile bearing capacity, a conclusion can be drawn on the amount of extra piles needed, m_2, in case additional piles are needed at all. If m_2 is higher than m_1, the amount can be corrected as m_1 is an already proven minimum baseline. If m_2 is however lower than m_1, a profit has been obtained in the amount of piles needed and thus in the on-site construction time and additional costs.

Again, a decision must be made if the amount of additional piles is satisfactory, or if further profit is required. If the latter is decided, this leads to the third step of geotechnical analysis, namely the overall analysis of the new situation. In case the strength of the existing foundation was measured, and thus path a is followed, this step consists of redesigning the superstructure in such a way that the load acting on the foundation is lowered as much as possible. Depending on the profit gained with this recalculation, the amount of required additional piles (m_3) can be decreased and maybe even diminished. The same goes for path b, in case measuring the foundation capacity in Level 2 was not an option. However, in this case the overall analysis is more extensive, as not only the load distribution, but also the pile bearing capacity is reconsidered and time-effects are taken into account.

Overall, a clear increase of designing labour can be observed with every step and level of geotechnical analysis. Between every step, a decision must be made based on designing costs, required reduction of the on-site construction time and installation costs of additional piles.

8.7. STRENGTHENING METHODS FOR FOUNDATIONS

In case the existing foundations are not capable of resisting the increased load from the superstructure, the existing foundation must be strengthened in order to increase the overall bearing capacity. For this, several methods can be distinguished, that will be briefly discussed below:

- **Additional piles**
  The shortage on foundation bearing capacity can be solved by installing additional piles next to or in-between the existing piles. These additional piles will have to be dimensioned according to the present design codes and will therefore have to take the reduction in pile tip bearing resistance into account. On the other hand, if the new piles are installed in the influenced zone of the existing foundation, use can be made of an increased cone resistance from group effects that is measured in new CPTs.

  The location of the new piles depends on the available on-site construction time and the spacing between the existing piles. If time is stringent, the choice can be made to install the piles next to the existing foundation without hindrance to the traffic, however, the increased cone resistance will be lower due to a smaller influence from adjacent piles. To increase this influence, the new piles can be installed in-between the existing piles, provided that these piles are not too close to each other. In both cases, however, it must be examined how the new piles can be coupled to the existing piles though the existing or a new pile cap, again depending on the available on-site construction time.

  Another question is whether it is possible to perform CPTs within the influence zone of the existing foundation, regarding the existing pile cap and the unhindered traffic flow.

- **Densification of soil** *[van Tol and Everts, 2006]*
  The pile bearing capacity of the existing pile foundation could In theory also be increased by further densification of the soil in the influence zone of the pile tip and shaft.

  This is automatically achieved when driving additional piles into the influence zone, but can also be obtained by vibrations or dynamic methods. The densification is the result of a decrease in pore volume from the dissipation of pore water and is thus a function of the permeability of the soil. During densification the soil particles must shift relatively to each other, thereby breaking contact and thus a temporary relaxation of effective stress will take place. This means an increase in water pore pressure, thus instigating a pressure gradient that pushes the pore water away to a low pressure zone.

  However, since the effective stress (temporarily) decreases during densification, this method might have an undesirable influence on the bearing capacity of the existing foundation. As the magnitude and long-term effect of this relaxation is unknown, it is not recommended to apply vibration or dynamic densification as a strengthening method. Nevertheless, the densification as a result of additional pile driving is of course still an option.

- **Grouting** *[van Tol and Everts, 2006]*
  Another method to increase the soil bearing capacity is grouting, through reduction of permeability and an increased shear strength or stiffness. Different types can be distinguished:

  – Permeation grouting;
    Here the pore water is replaced by a fluid that hardens after injection, but the soil structure itself is left intact. This method decreases the permeability of the soil but also increases the shear

strength and stiffness of the soil. It is mainly suitable for strengthening and supporting foundations on sand, creating horizontal or vertical water barriers and limiting settlements as a result of tunnelling.

– Displacement grouting and jet grouting;
These are grouting methods that do affect the soil structure. In jet grouting, the soil is cut up and mixed with a hardening injection fluid, often cement. As this method does not depend on the permeability of the soil, the application is much more widespread. It is mainly used to strengthen and support shallow foundations, creating water retaining layers and strengthening the soil when excavating underneath an existing building. Displacement grouting is also called fracture grouting, which displaces the soil somewhat. A distinction is made between compaction grouting, in which the soil is stressed and densified, mainly to restore an unwanted relaxation of the soil, and compensation grouting that compensates or prevent settlements to occur.

In general, however, all these grouting methods disturb the soil to a certain extent before increasing the bearing capacity. The magnitude and effect on the already present piles is unknown and thus this method is not suitable as a strengthening measure for the existing foundation. Furthermore, the initial stress level will be rather high from the thick piles that have been present and loaded for over 50 years and it is expected that this high initial stress level can never be reached again, let alone be improved, after grouting.

- **Stiffening of the pile cap**
  Instead of increasing the pile tip or pile shaft resistance, the existing foundation can also be strengthened by increasing the amount of co-operating piles in a pile group. This can be achieved by creating a higher stiffness of the pile cap, thereby enabling a more favourable load distribution among the separate piles. The result is that the low bearing capacity of weaker piles can be compensated by the stronger piles, thereby increasing the overall bearing capacity of the existing foundation.

  A stiffer pile cap can be obtained by increasing the concrete cross-section dimensions or applying a concrete type with a higher modulus of elasticity.

Although stiffening of the pile cap can increase the overall bearing capacity of the foundation, the execution aspect of stiffening a pile cap in combination with a minimised on-site reduction time decreases the potential of this strengthening method substantially. Therefore, it follows that the highest potential in strengthening of the existing foundation must be sought in the installation of additional piles.

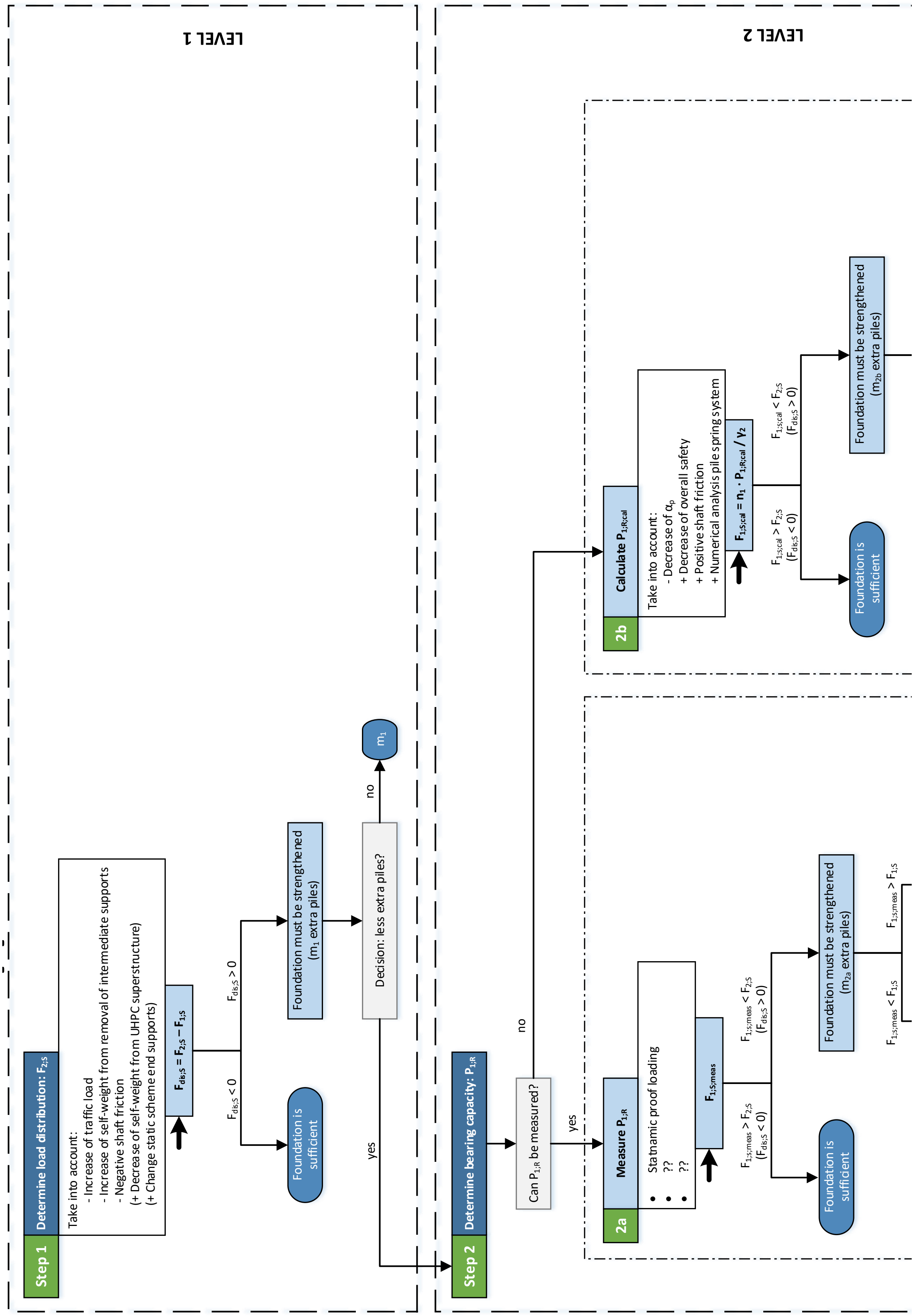


FIGURE 8.42: Schematic approach for reuse of foundations (part 1)

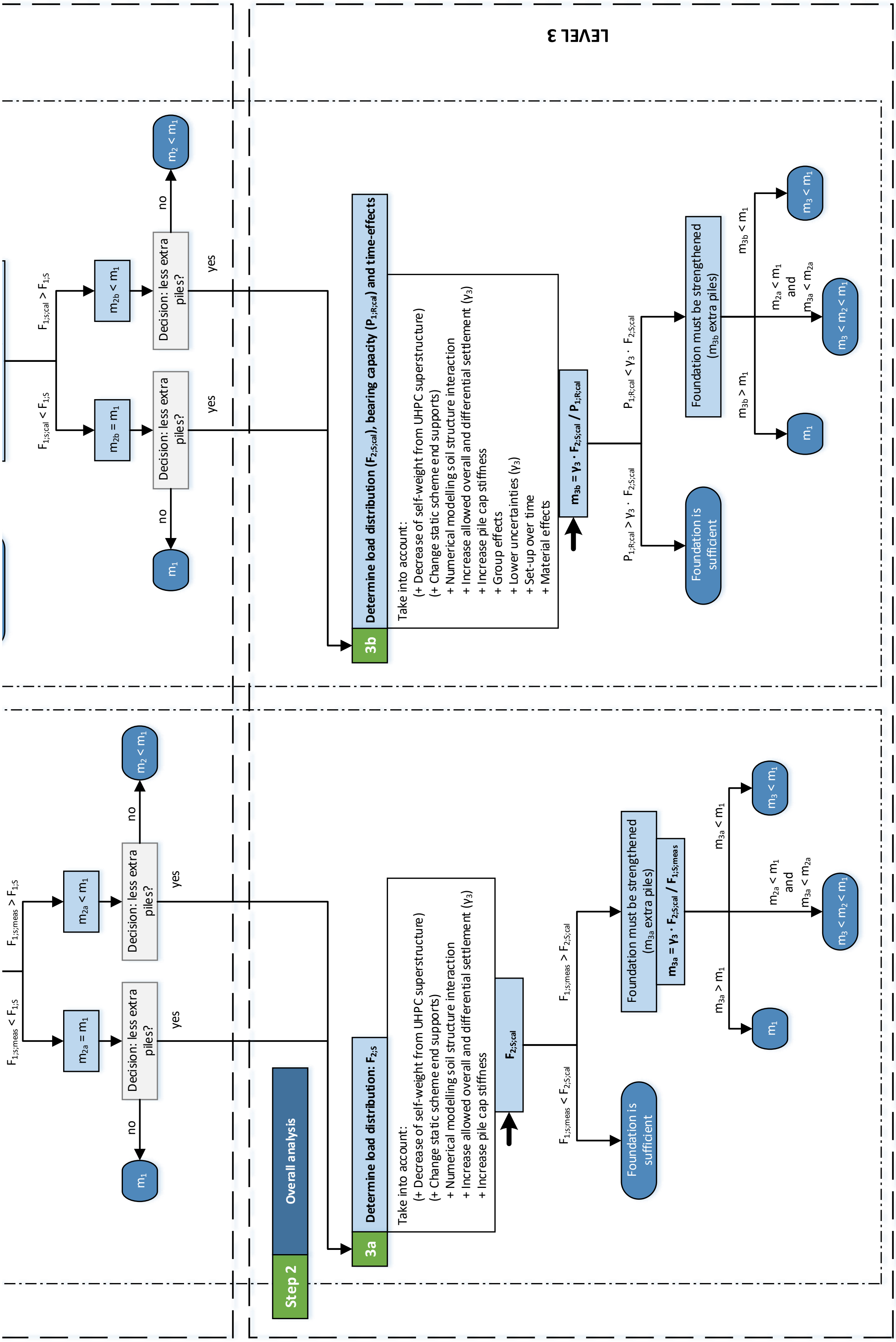


FIGURE 8.43: Schematic approach for reuse of foundations (part 2)

Part III:

Conclusions

9

Potential of improvement methods

In part II of this Literature Study, in the chapters 6 to 8, the theoretical background is given for proposed time-reducing methods in a bridge replacement task as introduced in chapter 1. The combination of a large infrastructural replacement task in the near future and a growing economic impact from traffic hindrance ask for solutions with a minimum on-site construction time. In this chapter, the potential of the different improvement methods will be discussed and compared, in order to formulate these possible solutions.

First, however, several boundary conditions for time-reduction in a bridge replacement project will be given in section 9.1. This is followed by a summation of the main outcomes of the theoretical background on ABC, ACM's and existing foundations (section 9.2), after which these outcomes are discussed in relation to a quick bridge replacement strategy (section 9.3). A first preliminary design of this strategy is then given in section 9.4.

9.1. Boundary conditions for time reduction

When looking at a quick bridge replacement strategy in which the on-site construction time is reduced as much as possible, the following boundary conditions apply:

- There should be a minimum of groundwork activities:
  In general, groundwork activities take up a lot of time and therefore it is desirable that no (large) groundwork activities are required. This leads to the requirement of maintaining the same traffic profile while removing intermediate supports (section 1.1) and thus the need for a high deck slenderness.

- There should be a minimum of demolition and construction works:
  As these activities take up time, they should be limited to a minimum. This means that, if possible, elements should be reused and elements that cannot be reused but are not hindering should be left in place. Therefore, the existing foundations and abutments must be reused if possible and otherwise left in the ground. This boundary condition also means, that it is favourable to combine separate elements to modular systems before placing in their final position, to reduce the amount of actions that need to be executed.

- New elements should be placed as quickly as possible:
  Inevitably, new elements have to be placed in a bridge replacement task, but the goal is to place them as quickly as possible. This asks for prefabricated and lightweight elements to ease transportation and erection.

9.2. Main outcomes theoretical background

In chapters 6 to 8 the theoretical background is given for the application of ABC, ACM's and existing foundations, respectively, in order to come to a quick replacement solution. Although already summarised in each respective chapter, the main outcomes are listed below:

ABC

- The Dutch practice already has experience in some ABC techniques, especially in using prefabricated elements;
- For the reduction of on-site construction time in a bridge replacement task, the following options can be considered:
  - Use of prefabricated elements (transported by road) and the erection by conventional cranes;
  - Use of prefabricated elements (transported by road), construction of modular systems on a location nearby and the erection by conventional cranes;
  - Use of prefabricated elements (transported by road), erection by conventional cranes parallel to the existing bridge and placement by lateral sliding;
  - Use of prefabricated elements (transported by road), erection by conventional cranes on a nearby location and placement by SPMT transport.

  The application of these options depend on the element or modular sizes and allowed closure time of underlying and crossing road.
- Reinforced soil might be used to reduce the total span length of the bridge.

ACM's

- Application of UHPFRC in bridge design can reduce the slenderness of the girder, due to:
  - The high compressive strength allows for a high prestressing force, which increases the moment capacity;
  - The shear capacity of a cross-section is increased by the application of fibres, reducing or even eliminating the amount of shear reinforcement needed.
- The overall weight of a girder in UHPFRC is reduced as the enhanced strength and durability properties result in a cross-section with less material.
- Up to date, a maximum slenderness ratio of $\lambda = 30$ has been applied in practice with UHPFRC. Calculations predict a slenderness ratio of $\lambda = 40 - 50$ to be possible.
- The required slenderness ratio of a three-span bridge that will be replaced by a single span, is $\lambda = 30-90$, while for four-span bridges it is $\lambda = 65 - 145$. It can thus be concluded that applying UHPFRC will only be sufficient in some cases for three-span bridges.

	Effect
Bearing capacity of pile foundation	
Decrease of α_p by 30%	-
Decrease of overall safety	+
Include positive shaft friction	+
Numerical modelling pile spring constant	+
Densification from group effects	+
Lower uncertainties (γ_M and ξ)	+
Superstructure loading	
Increase of traffic load	-
Increase of overall load due to removal of supports	-
Include negative shaft friction	-
Decrease of self-weight from UHPC	+
Change static scheme of supports	+
Make use of the soil structure interaction:	
Use numerical modelling	+
Allow higher differential settlement	+
Increase stiffness pile cap for larger pile group	+
Allow higher settlements	+
Time-effects	
Include set-up of soil over time	+
Include material time-effects:	
Strength improvement from ongoing hydration	+
Strength decrease from degradation mechanisms	-
Take into account creep, shrinkage and relaxation	+

TABLE 9.1: Potential influences on the pile bearing capacity of existing foundations and the accompanying level of geotechnical analysis

Existing foundations

- When looking at a bridge replacement task on bridges built between 1960 and 1980, several aspects have a negative influence on the potential reuse of the existing foundations. These aspects are related to:
  - Increase of the load distribution on the end supports;
  - Decrease of the pile tip resistance factor α_p.
- These negative influences might be compensated by smart design solutions:
  - Looking at differences in design methodologies of 1960-1980 and current practice;
  - Make use of a favourable soil structure interaction;
  - Take into account time-effects in soil and material.
- An overview of all influencing aspects is given in Table 9.1. The aspects presented in bold are expected to have the largest influence, both negative and positive.
- More research is needed into the time-effects, especially the set-up of soil, before application is possible.

9.3. POTENTIAL OF IMPROVEMENT METHODS

Together with the boundary conditions for a quick bridge replacement as presented in section 9.1, these main outcomes of the theoretical background can be judged on their potential in composing a quick replacement strategy. Below, the different methods are discussed by considering the following evaluation criteria:

- Technical feasibility;
- On-site time reduction;
- Ease of application;
- (Global) cost indication;
- Relevant design codes or regulations;
- Extent of the theoretical background.

ABC

As the term Accelerated Bridge Construction already entails, ABC methods are especially suitable for bridge (replacement) projects. Various techniques of an ABC approach are already common in Dutch building practices, such as the use of prefabricated elements to speed up the process of erection. Therefore, it is not the question whether ABC should be used in a quick bridge replacement strategy, but to which extent it will be applied. For every project, the targeted time reduction, additional construction costs and boundary conditions such as project site location and available space must be taken into consideration.
From the theoretical background, it follows that four different ABC approaches can be applied, with increasing on-site construction time reduction:

1. Use of prefabricated elements (transported by road) and erection by conventional cranes;
   This approach can be seen as the lower boundary of ABC, since the application of prefabricated elements is a prerequisite. The element dimensions are limited due to road transportation and therefore a minimum amount of elements is needed. These elements will be placed one by one on the supports and abutments with the use of conventional cranes, during which the underlying road will have to be closed. Common practice in Dutch regulations is a night-time closure of 15 minutes for each element that has to be placed. After placement of the prefabricated bridge girders (with closed bottom flanges) the construction works can continue without disturbance to the underlying road. The crossing road will however be closed during the entire construction process of demolition, element erection, pouring of the in-situ concrete deck and finishing. A case reference, in which two bridges have been replaced within 85 days, speaks of a 40 day closure time *[Dinnissen, 2010]*.

2. Use of prefabricated elements (transported by road), construction of modular systems on a location nearby and the erection by conventional cranes;
   If the capacity of conventional cranes allows the erection of modular systems, consisting of several prefabricated elements, this approach can reduce the closure time of the underlying road since less elements have to be erected. The elements will still have to be transported by road, though, and therefore their dimensions are limited. Another option is to combine the modular systems at a location next to the project site.

The total time reduction of this approach, compared to the lower boundary, is quite small since the only profit comes from the combination of several elements into one modular system, thereby reducing the number of erection activities.

3. Use of prefabricated elements (transported by road), erection by conventional cranes parallel to the existing bridge and placement by lateral sliding;
   If the bridge lies within a highway, the closure time of this road must be minimised. This can be done by building the entire superstructure on a lateral sliding support next to the existing bridge. During the construction period, the highway traffic is undisturbed, while the underlying road only experiences traffic hindrance from the night-time closures needed to erect the prefabricated elements or modules, as explained in the previous two approaches. Also, most of the finishing work can already be performed. If the superstructure is completed, the existing bridge is closed and demolished, after which the new superstructure is slid into place and can be connected to the highway. If necessary, a new foundation can be constructed beneath the existing bridge while it is still being used, otherwise additional closure time is needed in order to install the foundation piles and abutment.
   These activities can take place within a weekend closure of both the highway and the underlying road.

4. Use of prefabricated elements (transported by road), erection by conventional cranes on a nearby location and placement by SPMT transport;
   This approach can be chosen if the closure time has to be reduced even further, or if the traffic hindrance to the underlying road when using a horizontal launching approach is too high. Instead of constructing the entire superstructure adjacent to the existing bridge, this construction will take place on a separate project site, thus causing no traffic hindrance during erection of the prefabricated elements and other activities.
   Upon completion, both roads will be closed for demolition and placement of the superstructure by means of SPMTs, during a period of several hours to 1 day.

In general, it can be concluded that the best approach depends on the configuration of the road network, since the main decision parameter is the highway traffic hindrance. If a local road crosses over a highway, approaches one or two have the highest potential, since these lead to short-time closures during the night, when traffic is light. If, however, the bridge lies within a highway, either approach three or four will be chosen, as this limits the closure period of the highway traffic to several days. Furthermore, investment costs, heavy equipment costs and the available space near the project site are governing aspects in choosing the best ABC approach.

UHPFRC
The application of UHPFRC for the prefabricated bridge girders does not lead to on-site time reduction directly, but it does increase the technical feasibility of other time-reducing methods.

The enhanced strength and durability properties of UHPFRC compared to NSC result in a cross-section with less material and thus an overall weight reduction. This weight reduction is not only beneficial for the erection of the elements and the placement of entire superstructures, but also increases the chance that the bearing capacity of the existing foundations are sufficient for the new bridge. The production costs of UHPFRC are however substantially higher than for NSC and thus a design consideration must be executed based on reduced construction time and increased material costs.

Furthermore, by using UHPFRC for the girders, a higher slenderness can be obtained, thus possibly enabling the removal of the intermediate supports while the traffic profile is kept constant. Whether it is possible to remove all intermediate supports depends on the original deck height and the total length of the bridge. Removing the intermediate supports results in an increase of dead load on the end supports, since the span length has increased. Although UHPFRC girders will have an overall lower weight than NSC girders, due to a cross-section with less material, the increase at the end supports will be substantial, thus increasing the need for additional foundation piles to support the existing foundation.
Therefore, it must be considered whether the removal of intermediate supports is beneficial for the overall reduction of construction time. This decision must be based on the technical feasibility of a single span bridge, the design requirements from the client, the costs and the profit in construction time.

Existing foundations
The reuse of existing foundations is a time-reducing measurement since less demolition work and less installation activities have to take place, especially since piled foundations are not easily accessible.

Since the foundation has already proven to be capable of bearing the current load situation over the last 50 or more years, this loading is assumed to be the lower boundary of the pile bearing capacity in the new design. Therefore, as long as the new loading situation does not constitute higher loads than before, the existing foundation can be reused without hesitation, providing that the integrity of the pile foundations is still sufficient.

However, when redesigning this bridge the current codes have to be applied, giving rise to an increase in traffic load compared to the old load configuration. Furthermore, the possible removal of intermediate supports will result in a substantial increase of dead load on the end supports and thus on the foundations. It therefore seems unlikely that the new load situation is favourable or equal to the old load situation and thus the bearing capacity of the existing foundation might not be sufficient. A logical conclusion would be to strengthen the existing foundations by adding extra piles, but the accompanying increase of on-site construction time asks for a different solution.

The amount of additional piles needed can be reduced or even diminished by including several aspects regarding bearing capacity and load distribution in the design considerations. These aspects have been listed in Table 8.15 and will increase or decrease the amount of additional piles needed to a certain extent. Also, it depends on the depth of the design analysis which aspects will be taken into account. Although less piles will result in a larger reduction of on-site construction time, it must be considered to which extent the pile bearing capacity or load configuration is analysed in the design phase. If additional piles are needed anyway, due to the large increase of load on the existing piles, it might be beneficial to reduce the design activities and accept the relatively small increase in construction time. In this consideration, the construction costs, design costs, time reduction and possibilities in regulations must be taken into account. Therefore, more information is required on the sensitivity of the aspects listed in Table 8.15 with regards to pile bearing capacity and load configuration. This would enable a quick but plausible decision in an early design stage on the feasibility of the existing foundation.

9.4. GENERAL QUICK BRIDGE REPLACEMENT STRATEGY

These considerations lead to a general bridge replacement strategy in which the design principles and boundary conditions of the specific project result in the extent of the time reducing measures taken into account in the design phase. A first preliminary design of this quick replacement strategy is presented in Figure 9.1.

In general, three time-reducing measures can be distinguished, of which the extent and depth of the analysis depends on several boundary conditions. These measures are:

A. Reuse of existing foundations
The lower boundary bearing capacity is based on the currently applied foundation load. An increase in load can be taken up by additional piles, or (partly) by recalculating the pile bearing capacity, including time-effects and improving the soil structure interaction. If possible, the exact bearing capacity of the pile foundation can be measured, provided that this does not result in additional traffic hindrance. A schematic approach of this analysis is given in Figure 8.42.

B. ABC approach
A lower boundary for time-reduction is given by approach 1, in which prefabricated elements are erected during night-time closures using conventional cranes. Depending on the boundary conditions such as road configuration, investment costs, MEAT price reductions and available construction space an approach with more time reduction can be chosen.

C. Remove intermediate supports
The removal of intermediate supports reduces construction time, since less elements have to be erected, but it also complicates the design substantially. Especially the increase of dead weight on the end supports might result in additional foundation piles needed, thus reversing the earlier mentioned profit of needing less elements. The choice whether or not to remove the intermediate supports must be based on an overall consideration of time reduction, design costs, constructions costs and technical feasibility.

It must be stated that the decisions made regarding a quick bridge replacement do not depend solely on the reduction of on-site construction time, but also includes the price reduction obtainable from a MEAT procedure, the total project costs and the technical feasibility of the measurements.

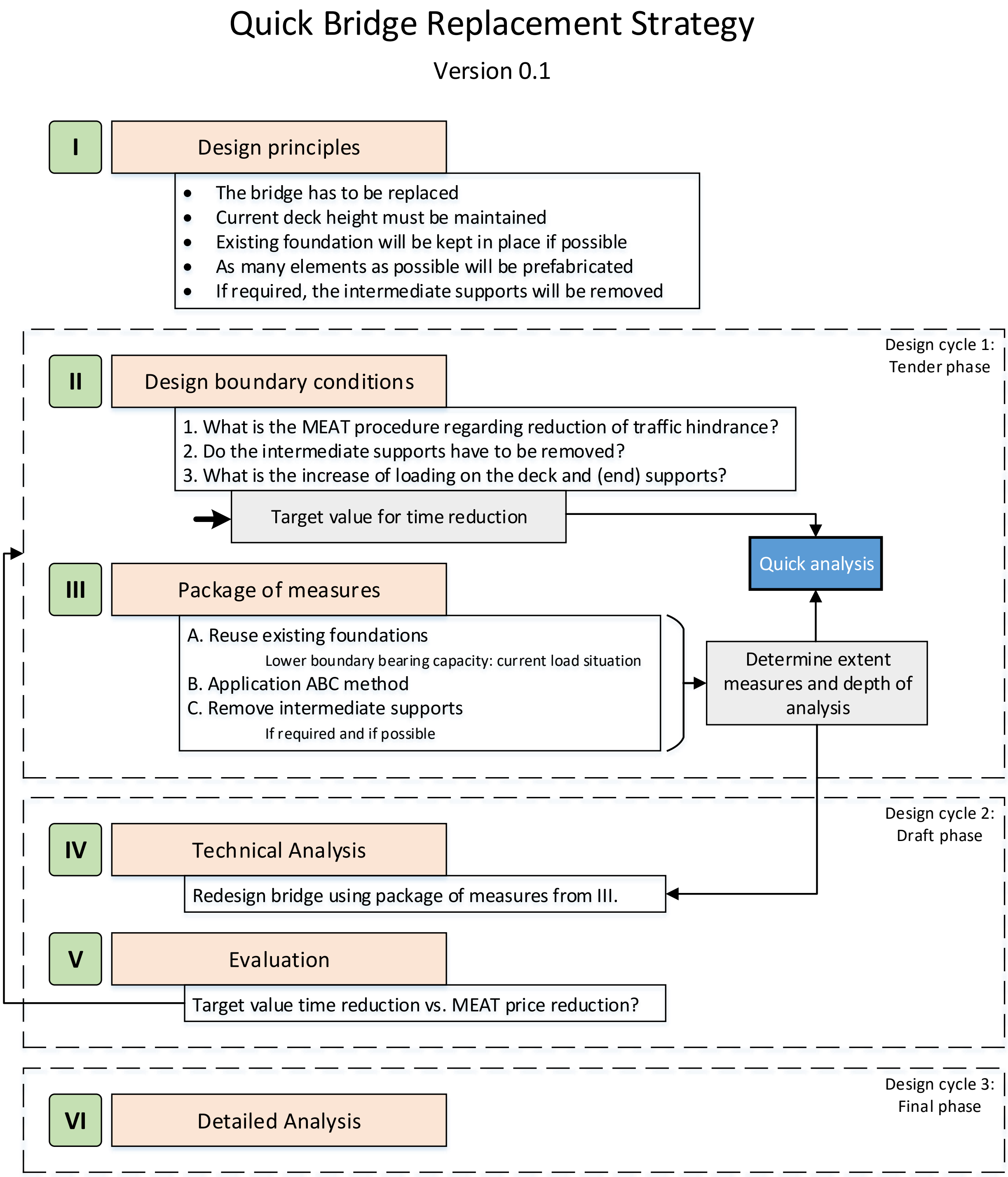


FIGURE 9.1: First preliminary design scheme for a general quick bridge replacement strategy

To give an indication of the time reduction possible when applying a quick bridge replacement strategy, a reference case is reviewed *[Dinnissen, 2010]*. In this case, two bridges have been replaced in a period of only 85 days, which can be considered as fast in the current building practice. Every bridge had a construction time of about 40 days, divided as indicated in Table 9.2 in different construction activities. The total construction time possible when using a quick replacement strategy has been added, and although these numbers are not substantiated yet, it does give an adequate indication of the possibilities in on-site construction time reduction.

Case reference	Days	Ideal quick replacement	Days
Demolition works	4	Only deck	1 - 2
Foundation	4	Nothing	0
Abutments	11	Nothing	0
Deck	10	Only placement	0.5 - 1
Finishing	12	Partly done before placement	1 - 2
Total	**41**		**2.5 - 5**

TABLE 9.2: Comparison of construction times in a reference case and an indication for a quick bridge replacement method *[Dinnissen, 2010]*

10

General concept Case Study

The overall goal of this graduation thesis is to construct a quick replacement strategy that minimises the traffic hindrance during bridge construction works. As multiple bridges will have to be replaced in the near future and every bridge has its own characteristics and boundary conditions, this quick replacement strategy must consist of a generalised action plan. The intention of this action plan is to support decisions that have to be made regarding the execution method and therefore also the design phases, with a focus on the on-site construction time reduction.

In the previous chapters of this Literature Study, a technical background has been outlined on different methods to obtain a reduced execution period. This is concluded by chapter 9, in which the potential of time reduction, (technical) feasibility and global cost impacts for all methods has been discussed. What follows is a quick replacement strategy that reduces the required on-site construction time to a minimum.

The next step will be to determine the technical feasibility of this strategy by looking at a specific case study. After demonstrating the potential of the proposed strategy, the obtained feasibility and time reduction will be analysed in a broader point of view. This ultimately leads to a generalised quick replacement strategy that can be implemented in multiple bridge replacement projects.

10.1. Proposed strategy

It depends on the specific characteristics and boundary condition of a project to which extent on-site time reduction is required and whether this is stimulated by a bonus in a MEAT procedure. Therefore, the amount of time reduction measurements and the depth of analysis is also project specific. Since only one specific case study is used to prove the technical feasibility of the proposed strategy and the intention is to generalise these outcomes, the depth of the proposed strategy must be chosen wisely. In order to cover the technical feasibility of all methods, the starting point of the case study will be a quick replacement method that minimises the on-site construction time as much as possible. When generalising the outcomes, this will mainly have impact on the design principles and conditions, which lead to the composition of the exact package of measures to be chosen in each case.
Furthermore, this case study will be used to investigate the sensitivity of the several methods to the evaluation criteria and boundary conditions as discussed in section 9.3. With the results from this sensitivity study, a probable choice can be made in a quick analysis on the package of time-reducing measurements to be applied.

Proposed strategy Case Study
The case study is based on the following design principles:

- The bridge must be replaced;
- The new bridge will have to be constructed at the same location of the existing bridge;
- The reduction of on-site reduction time is leading in decision making;
- The current deck height must be maintained;
- The current foundation is kept in place;
- Where possible, new elements will be prefabricated;

- The new deck elements will be made of UHPFRC;
- The intermediate supports must be removed.

The goal of the case study is to design a bridge replacement that has a minimal on-site construction time and thus a minimal traffic hindrance. This is obtained by:

1. An ABC method that involves the quickest structure placement;
2. Application of UHPFRC bridge girders to achieve a high slenderness and thus a single span bridge;
3. Reuse of the existing foundation in such extent that a minimum of additional piles is needed.

In order to design this bridge replacement, a sensitivity analysis is needed into the boundary conditions of:

- ABC methods like horizontal launching, SPMT placement and conventional cranes;
- Maximum slenderness and possible weight reduction of NSC and UHPC bridge girders;
- Profit in bearing capacity or load distribution of piled foundations from the aspects given in Table 9.1.

With the redesign of the bridge in the case study and the sensitivity analysis of the separate time reducing methods the basis is laid for the composition of a general quick bridge replacement strategy.

10.2. CASE STUDY

In order to execute a technical feasibility study on the time-reducing measurements as defined in section 9.4, several requirements for a suitable case study arise. This is also the case for an adequate execution of the sensitivity analysis (section 10.1) and the requirements are listed below:

- Case study requirements for time-reducing measures:
  - The bridge must lie within a highway;
  - The original design and drawings should be available;
  - The (end) supports will have to be founded on pile foundations;
  - The required slenderness for a single span must lie within the possible slenderness range of UHPC box girders.
- Case study requirements for adequate sensitivity analysis:
  - The bridge must cross another road;
  - The bridge is an in-situ plate bridge with 3 or 4 spans;
  - The bridge is built between 1960 and 1980.

Within Heijmans, multiple bridge projects have been executed over the last years or will be executed soon. One of these bridges is viaduct Zeldert (see Figure 10.1), which is part of the A27/A1 widening project that will start execution in July 2017. In reality, this bridge will be maintained and widened, but for the sake of this thesis a replacement task is assumed.

FIGURE 10.1: Google street view of viaduct Zeldert

Viaduct Zeldert *[Polhaar, 2016]* lies within highway A1, between junction Eemnes and junction Hoevelaken (see Figure 10.2). It consists of two separate bridges (Figure 10.3), a northern bridge that was built in 1970 and a southern bridge that was constructed in 1963 and widened in 1970. Both bridges consist of an in-situ reinforced plate deck with a height of 500 mm and have three intermediate supports which are founded on spread footings, while the end supports have piled foundations. A longitudinal cross-section of the existing bridge is given in Figure 10.4. The bridge is skew, with an angle of 79.26° compared to the underlying N199. The span length perpendicular to the N199 is 9.80 – 11.76 – 11.76 – 9.80 m, and parallel to the bridge it is 10.0 – 12.0 – 12.0 – 10.0 m.

In Table 10.1, the requirements as stated above are checked for this case study. It follows that although the intermediate supports are founded on spread footings and the required slenderness for a single span is too

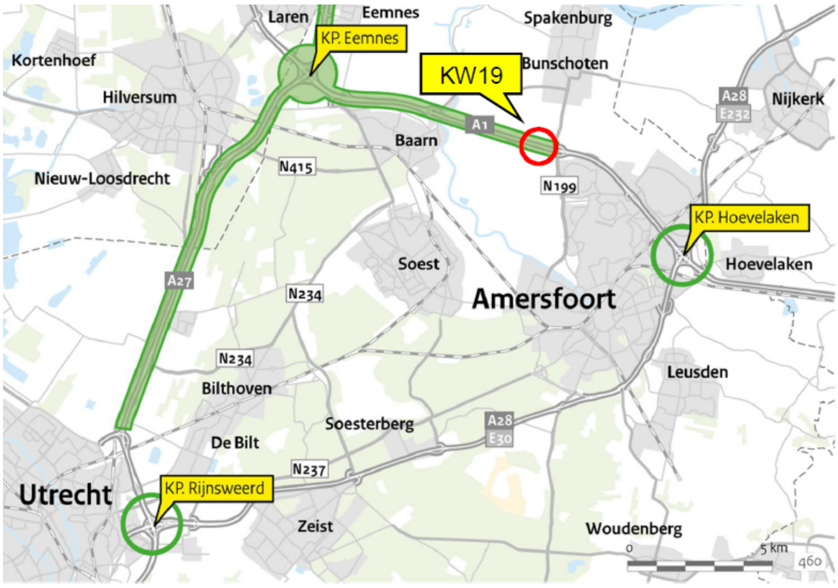


FIGURE 10.2: Position of viaduct Zeldert (KW16) within highwa A1 *[Polhaar, 2016]*

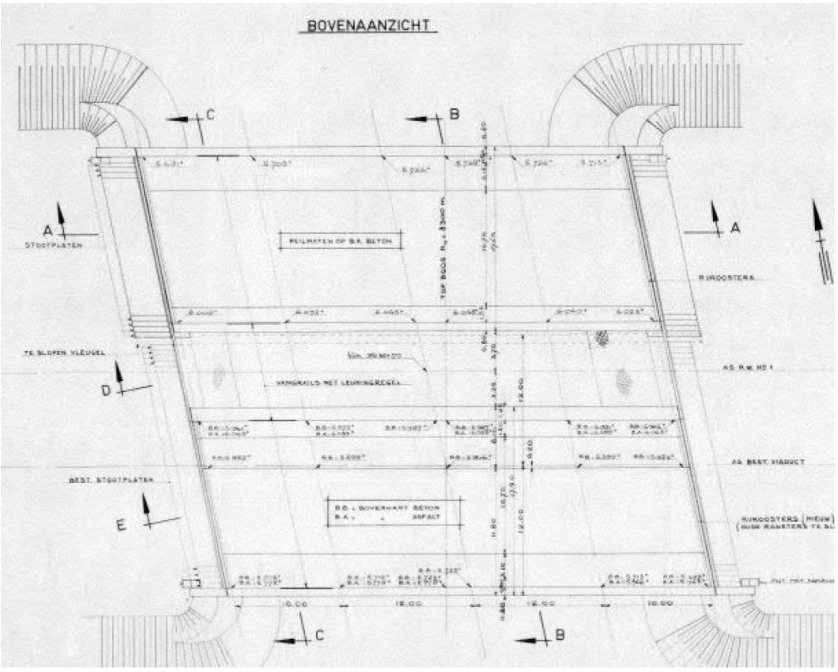


FIGURE 10.3: Top view of viaduct Zeldert *[Polhaar, 2016]*

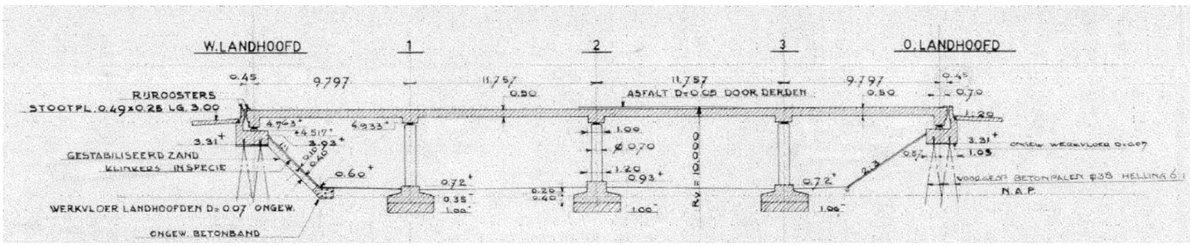


FIGURE 10.4: Longitudinal cross-section of viaduct Zeldert *[Polhaar, 2016]*

high, viaduct Zeldert is still a suitable case study to research the technical feasibility and sensitivity of the proposed quick bridge replacement strategy in Figure 9.1.

Requirements	Viaduct Zeldert	Suitable?
Technical feasibility		
Within a highway	Within A1	Yes
Original design and drawings	Available	Yes
(End) supports on piles	End supports: piles Intermediate: spread footings	Yes No
Required slenderness for single span	h = 0.5 m, L = 44 m Thus λ = 88	No
Sensitivity analysis		
Cross another road	Crosses N199	Yes
In-situ plate bridge	Reinforced in-situ plate bridge	Yes
3 or 4 spans	4 spans	Yes
Built between 1960 and 1980	Built in 1963 / 1970	Yes

TABLE 10.1: Requirement check for viaduct Zeldert as a suitable case study

For the intermediate supports it holds that it is an interesting elaboration to explore the possibilities of reusing spread footings as well as piled foundations. For the removal of intermediate supports and the required slenderness that follows, two options will be studied:

1. The middle support is removed, while the other two are maintained (Figure 10.5). The highest slenderness is in this case $\lambda = 24/0.5 = 48$, which is possible in theory.
2. The middle support is maintained, while the other two are removed (Figure 10.6). The highest slenderness is in this case $\lambda = 22/0.5 = 44$, which is possible in theory.

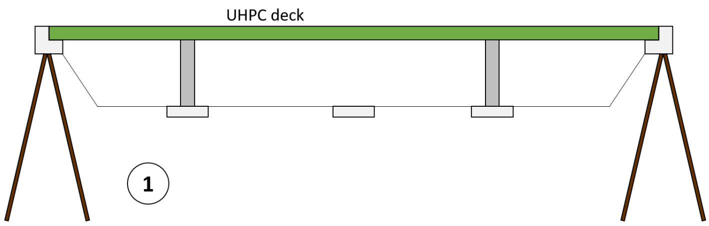


FIGURE 10.5: Option 1: Removal of the middle support

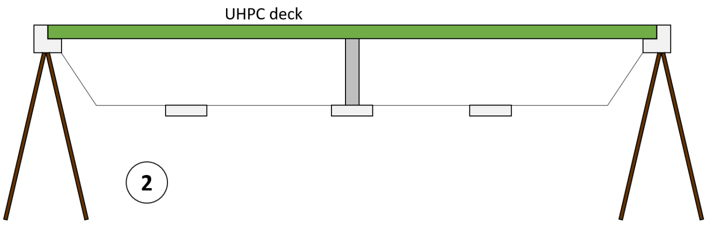


FIGURE 10.6: Option 2: Removal of the left and right intermediate supports

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