The Circular Concrete Viaduct

Development of a Concept Demountable Footing to Foundation (F2F) Dowel Connection for the Application in Multiple Life-Cycles

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by

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TU Delft

Acknowledgements

This thesis marks the final achievement in order to obtain the degree of Master of Science in Structural Engineering, with the specialisation Concrete Structures, at the faculty of Civil Engineering and Geosciences at Delft University of Technology. The research has been performed in collaboration with Lievense Infra B.V. in Leeuwarden, and took place during the period from February 2020 until January 2021. In this work, the findings of my research into the topic of circular concrete viaducts, specifically focused on the development of a demountable footing to foundation (F2F) dowel connection, are presented. The goal of this research was to gain knowledge about the requirements in order to achieve circular viaduct construction as well as to propose a technical solution for the main bottleneck in traditional viaduct design which currently limits the possibilities for deconstruction and reuse of viaducts. This was motivated by the enormous challenge faced by Dutch Ministry of Infrastructure and Water Management (Rijkswaterstaat) to replace a large number of viaducts across the Netherlands in the upcoming decades.

First of all, I would like to take the opportunity to express my gratitude to the members of my graduation committee for all their (mostly online) contributions in these bizarre times marked by the global Covid-19 pandemic. I want to thank dr. ir. Yuguang Yang for helping me shape the topic in the first place, and for all his shared expertise and constructive feedback. Besides, my thanks go to dr. ir. Herbert van der Ham and dr. Francesco di Maio for sharing their knowledge, constructive feedback and inspiration which guided me through the process. Finally, I want to thank Lievense Infra B.V. and all colleagues for providing me the opportunity to conduct my research in collaboration with them and for an inspiring and supportive working environment. In particular, my thanks go to ing. Albert Rodenhuis PMSE as my daily supervisor at Lievense Infra B.V. for all his guidance and support, and for all of his shared ideas, expertise and constructive feedback.

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Abstract

The current (traditional) construction industry is characterised by a linear life-cycle model which results in huge amounts of construction and demolition waste, pollution, and loss of embodied energy. However, the awareness to protect the environment is rapidly growing on a global scale. With regards to the construction industry, this is characterised by the goal to transit from a linear to a circular construction industry in which the service life of materials and components is extended, and in which the final stage of demolition and disposal is replaced by a stage of disassembly and reuse.

On a national scale, the Dutch government has stated the ambition to achieve a fully circular economy by 2050 at the latest. This also includes a fully circular construction industry, which is currently believed to be one of the main polluting industries. In accordance with this ambition, Rijkswaterstaat has set the goal to work completely climate-neutral and circular already in 2030. Currently, their main focus is on developing circular solutions for concrete viaducts for (governmental) roads, since a large amount of the estimated 40.000 bridges and (mostly) viaducts in the Netherlands need to be replaced in the coming decades. Therefore, this has been the main problem focused on in this research.

Concretely, this research problem has been translated into the main objective to develop demountable, instead of modular, solutions for concrete viaducts in order to get closer to a circular concrete viaduct concept, which can be applied on a large scale. Besides, attention has been paid to two subparts regarding the concept of a circular concrete viaduct, which involved monitoring of the viaduct, and the life-cycle costs of a circular viaduct, compared to the same viaduct, constructed in a traditional way, characterised by a linear life-cycle model. The goal of the former was to advise on desired monitoring aspects in order to be able to incorporate this into the design process and to guarantee the circularity of the viaduct during its entire intended service lifetime. The goal of the latter was to explore the financial feasibility of a circular concrete viaduct.

In order to develop demountable solutions for concrete viaducts in a systematic way, first it was focused on three key technical action points to achieve circular bridge (viaduct) construction. These three action points concerned a redefinition of Brand's shearing layers of longevity specific for bridges, the adjustment of Design for Deconstruction (DfD) principles to the specific needs and requirements of bridges and, finally, the development of a complimentary standardisation scheme. This last action point was the most complex task since it involved the development of a so-called standard viaduct. Based on the design and layout of this standard (circular) viaduct, a demountable solution for the key bottleneck in current viaduct design, preventing it from being demountable and reusable, was developed. This key bottleneck was identified to be the connection between the (abutment) footing and the foundation, referred to as the F2F connection. Therefore, with regards to the main research objective, the development of a demountable F2F connection has been focused on.

This demountable F2F connection has been developed by means of a step-wise verification procedure, which included the development and use of two 2D finite element models created in SCIA Engineer, and of a 1D analytical semi-infinite beam on an elastic foundation model created in MAPLE. The first SCIA Engineer model involved a model of the developed standard viaduct, which was used to obtain the critical cross-sectional forces at the F2F interfaces. The second SCIA Engineer model involved a non-linear model of the concept demountable F2F dowel connection. This model was used to verify different geometries and properties of the connection by means of checking the maximum dowel deformation (w_{max}) and the maximum contact stress between dowel and concrete/mortar (σ_z). The 1D analytical beam model was used to assess the influence of changing geometries and properties of the connection. Besides, the 1D analytical model was used to perform sensitivity analyses with respect to the parameters that represent the concrete embedding surrounding the steel dowel, i.e. the foundation modulus of concrete under dowel action (k_c and k_d). Sensitivity analyses with respect to this parameter were performed, since it turned out to be the most critical parameter. Therefore, its influence on the replacing rotational spring stiffness of the dowel connection ($k_{r,con}$) and on the maximum dowel deformation (w_{max}) was investigated. Furthermore, three different execution variants for the demountable F2F connection were drafted of which the practical issues were discussed extensively.

The subpart regarding the monitoring aspects has been based on information found in literature and on engineering judgement. This part mainly focused on identifying the dominating damage and deterioration mechanisms typically observed in concrete bridges, and on determining the locations where damage and deterioration is expected to occur. The other subpart regarding the life-cycle costs of a circular concrete viaduct has been based on key cost figures, a number of assumptions and starting points, and on experience of cost calculators at Lievense. Based on this, a life-cycle cost analysis (LCCA) for both a traditional and a circular alternative was performed in Excel.

Finally, the research has resulted in both a proposal for a standard (circular concrete) viaduct (i.e. a standardisation scheme) and a concept demountable F2F dowel connection. The design and layout of the standard viaduct as well as the geometry and properties of the demountable connection have extensively been described in the report. In short, the connection consists of a steel dowel welded to an end plate which, in turn, is welded on top of steel pipe foundation piles. The steel dowel partially penetrates the (abutment) footings and is enclosed by means of a mortar filling. This is based on execution variant 3, which was believed to be the most feasible variant considering both executional and financial aspects. Furthermore, an additional result of this research with regards to the demountable connection is a design table, which can be used to quickly assess the feasibility of the same demountable F2F dowel connection with different parameter ranges (i.e. different geometry and/or properties).

Besides, the subpart regarding monitoring aspects has resulted in the creation of two draft versions of monitoring plans, advising on *where* to monitor *what*. The first monitoring plan addresses the (standard) circular concrete viaduct in general, whereas the second is specifically with regards to the demountable F2F connection. Finally, the LCCAs resulted in estimations of the life-cycle costs for the traditional and the circular alternative. Subsequently, these life-cycle costs were compared to each other and the financial feasibility of a circular concrete viaduct was explored.

The results regarding the development of demountable solutions for the concept of a circular concrete viaduct suggest that the proposed concept demountable F2F dowel connection provides a potential circular solution for the connections between (abutment) footings and foundation. Besides, it was concluded that execution variant 3 is the most feasible execution variant based on both executional and financial aspects. Furthermore, the results of the subparts regarding monitoring aspects and the life-cycle costs indicate that an intended full service lifetime of the (elements and components of the) circular viaduct of 200 years is realisable, and that the (provisional) conclusion can be drawn that a circular viaduct is financially feasible, provided that a number of starting points and assumptions are met with regards to both subparts. Generally, it was concluded that the results of this research provide requirements in order to transform the traditional (linear) design of a concrete viaduct in the Netherlands into a circular (demountable) viaduct, and therefore contribute to a future of large scale circular viaduct construction.

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List of Symbols

The next list describes several symbols that are used within the body of this document.

NB: the subscript i indicates that the symbol can take different index labels, e.g. $i = 0,1,2,\dots,$ or $i = x,y,z$, or i as a description/abbreviation.

Latin upper case letters

A (Cross-sectional) area

C_i Integration constants

E Modulus of elasticity

EI Bending stiffness

F Point load ; stress

$F_i^{+/-}$ Point load as a result of a decomposed bending moment with lever arm e (positive/negative)

I_z Area moment of inertia

L Length

M_i Bending moment

Q_i Uniform distributed load (UDL)

T Temperature

ΔT Temperature load

V_i Shear force

Latin lower case letters

a Protected dowel length

b Embedded dowel length

c_1 Bar spacing coefficient, ranging between 0,6 and 1,0

d Diameter

d_b Dowel diameter

e Lever arm/eccentricity ; Euler's number

f_b Bearing strength of concrete

f_c Compressive strength of concrete

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

$f_{M;i}^{+/-}$ Line force as a result of a bending moment (positive/negative)

$f_{V;i}^{+/-}$ Line force as a result of a shear force (positive/negative)

k_c Foundation modulus of 2D concrete members under the action of a steel dowel

k_d Foundation modulus of 1D concrete members under the action of a steel dowel of 1D members

k_h Distributed spring stiffness

$k_{r;i}$ (Replacing) rotational spring stiffness

l_b Beam length subjected to significant deformation

p Perimeter

p_b Dowel perimeter

q_i Line load

r Horizontal radius of carriageway

t_i Thickness

u Deformation

u_{c3} Compressive deformation in concrete at peak stress f_c

u_{cu3} Ultimate compressive deformation in concrete

w Carriageway width

w_1 Notional lane width

w_i (1D beam) deformation

x, y, z Coordinates

Greek upper case letters

$\Delta 1_{xy}$ Horizontal construction tolerance for execution variant 1

$\Delta 2_{xy}$ Horizontal construction tolerance for execution variant 2

$\Delta 3_{xy}$ Horizontal construction tolerance for execution variant 3

Greek lower case letters

α Ratio of applied shear force and bending moment

β Angle

γ Partial (load) factor

γ_a Self-weight of asphalt

γ_c Self-weight of concrete

γ_s Self-weight of steel

γ_{so} Self-weight of soil

ε_{c+s} Combined shrinkage and creep strain

ε_{c3} Compressive strain in concrete at peak stress f_c

ε_{cu3} Ultimate compressive strain in concrete

κ_i (1D beam) curvature

λ Coefficient in Euler-Bernoulli beam on elastic foundation equation

ϕ_i (1D beam) rotation

ψ Factors for defining representative values of variable actions
ψ_0 for combination values
ψ_1 for frequent values
ψ_2 for quasi-permanent values

Other symbols

Ø Diameter

1 Introduction

In this chapter, firstly the background and motivation for the topic are explained in section 1.1. Next, the objective and scope of the research are defined in section 1.2, and subsequently the research questions are stated in section 1.3. In section 1.4, definitions of some important key concepts are given in order to avoid possible misconceptions. Finally, the outline of the report is stated in section 1.5.

1.1. Background and Motivation

First of all, the general concepts of sustainability and the circular economy are shortly addressed. Subsequently, both global and Dutch plans for a transition from a linear to a circular construction industry are discussed, and finally the main challenge (and opportunity at the same time) for circular solutions that is currently relevant in the Netherlands is identified.

1.1.1. Sustainability and the Circular Economy

Ever since the Brundtland report in 1987, and later the launch of the Ellen MacArthur Foundation (EMF) in 2010, the importance of the ideas of ‘sustainability’ and the ‘circular economy’ have been gaining increasing attention. Both ideas more or less similarly originate from an environmental awareness and consequent call to preserve our planet [1]. Often the interpretation of these terms is unclear as a result of their, nowadays, widespread application in almost every discipline, and of the fact that every discipline tends to define them differently. Therefore, first of all, a well-known definition for ‘sustainability’ and an elaborate definition for the ‘circular economy’, based on 114 different definitions [2], is given, which are adopted in this research:

> **Sustainability**: “*Sustainable development is development that meets the needs of the present without compromising the ability of future generations to meet their own needs.*” [3]
>
> **Circular economy**: “*A circular economy describes an economic system that is based on technological advances and new business models which replace the ‘end-of-life’ concept with reducing, alternatively reusing, recycling and recovering materials and energy in production/distribution and consumption processes in order to keep products at their highest possible value, thus operating at the micro-level (products, companies, consumers), meso-level (eco-industrial parks) and macro-level (city, region, nation and beyond), with the aim to accomplish sustainable development, which implies creating environmental quality, economic prosperity and social equity, to the benefit of current and future generations.*” [1]

The relationship between both terms is defined by Anastasiades et al. [1] who state that “*sustainability is the goal, while the circular economy is a means to achieve a more sustainable economy*”.

1.1.2. Transition from a Linear to a Circular Construction Industry

Within the principles of a circular economy, the construction industry is thought to have the highest potential [4]. This is because the construction industry in general is known as a conservative industry, characterised by a linear life-cycle model, also referred to as the ‘cradle-to-grave’ model. In such a linear model, the life-cycle of a structure is characterised by the stages of initiation, design, construction, operation and maintenance, refurbishment and finally demolition. On a materials scale, a similar life-cycle can be recognised, starting with raw materials extraction, followed by materials processing, assembly and construction, operation, and finally demolition and disposal [5] (see Figure 1.1). This results in huge amounts of construction and demolition waste, pollution, and loss of embodied energy[1].

It is estimated that the global construction industry accounts for almost 40% of all carbon dioxide (CO_2) emissions, from which 11% results from manufacturing building materials and products such as steel, cement and glass. Therefore, it is clear that decarbonising the construction industry is critical to achieve the Paris Agreement commitment and the United Nations Sustainable Developments Goals [6]. Furthermore in the “2019 Global Status Report for Buildings and Construction” [6], it is emphasised that 2020 is a key year for countries under the Paris Agreement commitment as the Nationally Determined Contributions (NDCs) that are part of the agreement have to be revised. An important topic in the NDCs concerns the further actions to address energy use and emissions including embodied emissions in the construction industry. Although 136 countries have mentioned buildings in their current NDCs, only few have specified concrete actions they will take. Therefore, countries have to prioritise actions to address energy use and emissions in the essential industry which also means improving building design [6].

[1] The definitions of highlighted terms are defined in section 1.4

In order to achieve those goals, it is believed that the transition towards a so-called 'circular construction industry', a construction industry based on the principles of the circular economy, can play a major role. A circular construction industry, in contrast to the traditional linear construction industry, is characterised by a circular life-cycle model, also known as the 'cradle-to-cradle' model. In such a circular model, the stage of demolition and disposal is avoided as much as possible, and instead an alternative strategy of disassembly is implemented. That way, a closed-circuit, circular, model is being created. This results in a more sustainable model in which the production of waste and pollution, and loss of embodied energy is reduced, while the service life of materials as well as entire building components is extended [5]. The life-cycles of both a linear and a circular materials model in the construction industry are shown in Figure 1.1.

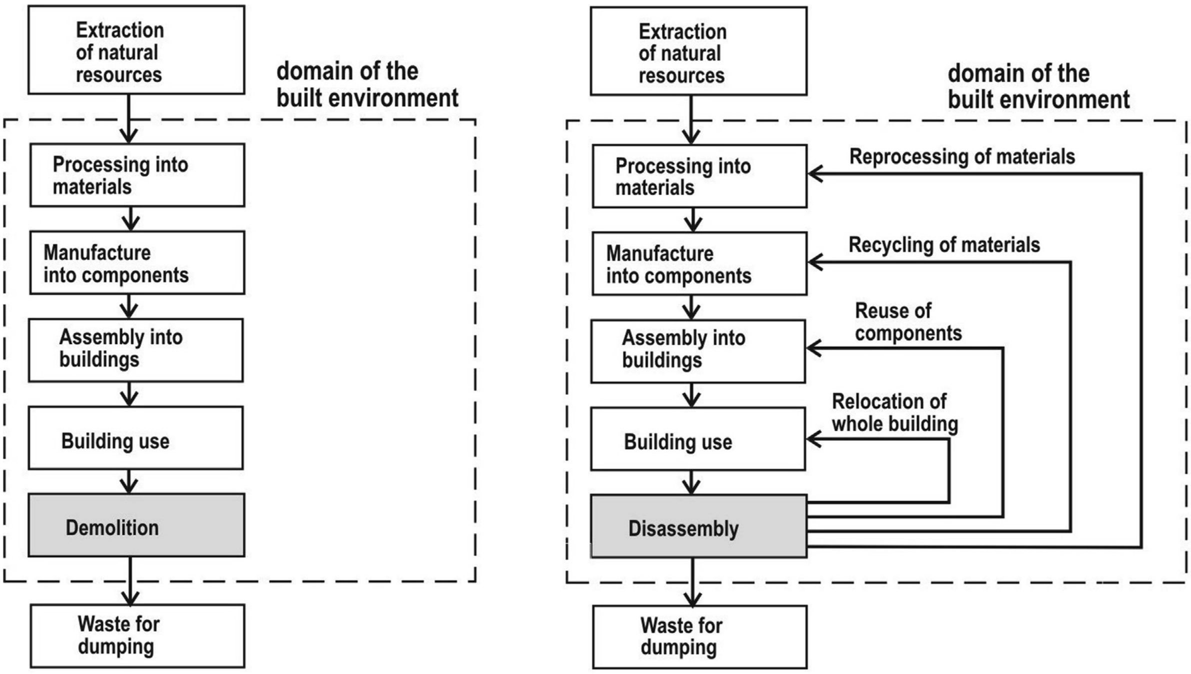


Figure 1.1: Life-cycles of both a linear (left) and a circular (right) materials model in the construction industry [7]

The situation in the Netherlands

Following the global ambitions of a future with a net zero carbon emission construction industry [6], the Dutch government has stated the ambition for a fully circular economy, which also implies a fully circular construction industry, by 2050 at the latest [8]. Besides, together with social partners, an (interim) objective of 50% less use of primary abiotic raw materials (minerals, fossils and metals) in 2030 has been set by the Dutch government. Additionally, the ambition of Rijkswaterstaat[2] is to work climate-neutral and circular in 2030 [9]. Never before has the circular economy been so prominent on the Dutch social agenda. The SER[3]'s advice [10], aiming for a circular economy, the government-wide program "The Netherlands circular in 2050" (Dutch: '*Nederland circulair in 2050*') [11], in which the construction industry is one of the five priorities, and the Raw Materials Agreement (Dutch: '*Grondstoffenakkoord*') [12] offer important frameworks. There is a broad consensus among the government both at central governmental level and at provincial and municipal level, but also among market parties and knowledge institutions [8].

As at global level, at national level too, the construction industry plays a crucial role in this process. The government-wide program "The Netherlands circular in 2050" [11] calculates that the construction

[2]Rijkswaterstaat is part of the Dutch Ministry of Infrastructure and Water Management and responsible for the design, construction, management and maintenance of the main infrastructure facilities in the Netherlands (`https://www.rijkswaterstaat.nl/english/about-us`)

[3]The SER is the Social and Economic Council (Dutch: '*Sociaal-Economische Raad*') and is the most important advisory body for government and parliament in which entrepreneurs, employees and independent experts work together to reach agreement on important socio-economic issues (`https://www.ser.nl/nl/ser/over-ser`)

industry in the Netherlands accounts for an estimated 50% of raw material consumption, 40% of the total energy consumption and 30% of the total water consumption. Besides, a large part of all waste in the Netherlands relates to construction and demolition waste and the sector is responsible for around 35% of CO_2-emissions. Though, it should be noted that around 97% of construction and demolition waste is recycled, an important part of which, however, for low-value applications in the infrastructure sector [8] such as concrete granulate as road foundation.

The transition to a circular construction industry is a complex task, but there is also an enormous potential for the development of new products and services. The larger the scale on which circular products and services are applied, the more cost reduction and effectiveness can be achieved and the faster learning experiences can be gained. Besides, circular measures can make an important contribution to the reduction of CO_2-emissions [9].

1.1.3. The Challenge for Circular Solutions in the Netherlands

At the moment, the main focus for circular solutions within the Dutch construction industry is on bridges and viaducts. The Netherlands roughly counts 40.000 bridges and viaducts, the majority of which are viaducts. Most viaducts in the Netherlands were built between 1960 and 1980. A large part will have to be replaced in the coming decades. In order to achieve the objectives in terms of circularity and climate neutrality, it is therefore crucial that new, sustainable solutions are available. Various innovations have already been developed in the GWW[4] sector to reduce CO_2-emissions and the use of primary raw materials for the construction, management, and maintenance of infrastructure. Examples of innovations in the field of circular bridges and viaducts in the Netherlands (and Germany) are [9]:

- *NTA[5] Industrial-Flexible-Demountable (IFD) construction for movable bridges (Province of Noord-Holland and NEN):*
  The design code NTA 8086:2019 aims for circular construction of movable bridges and describes the first agreements on standard interfaces between parts of movable bridges [13].

- *Design and construction of Circular Viaduct (Van Hattum en Blankevoort, Spanbeton and Rijkswaterstaat):*
  See subsection 2.2.2 for more detailed information.

- *The Circular Road (Dura Vermeer and Province of Overijssel):*
  Contractor Dura Vermeer is the owner of the road, and provides it as a service to the user. Also, they are responsible for the end-of-life scenario of the road to ensure high-quality reuse of the raw materials [14].

- *Various initiatives involving biobased bridges, including the biobased bicycle bridge Ritsumasyl:*
  The bridge has a movable, free span of 22 m and, including the approaching bridges, a total of 66 m of biobased road surface. Renewable raw materials such as flax and resin form the basis for the new product [15].

- *Circular replacement of Cruquius bridge (Province of Noord-Holland):*
  The new bridge will be built energy-neutral, circular and as low-maintenance as possible. An energy neutral bridge is self-sufficient and generates as much energy as the bridge consumes [16].

- *Innovative and Circular bridges Floriade (Municipality of Almere and province of Flevoland):*
  The two bridges that will be built on the Floriade site are largely made from materials produced in Almere from Almere's residual flows. The materials used and the lesser transport kilometers, among others, result in CO_2-savings [17].

- *Concrete innovations, such as 'Smart breaking':*
  'Smart Breaking' is an innovation that completely breaks down concrete to the original raw materials: gravel, sand, cement and cement hydrate (to be used as CO_2-free raw material for the cement industry). This makes concrete 100% circular, without residual flows or downgrading [18].

[4] GWW stands for Soil, Road and Hydraulic engineering (Dutch: '*Grond-, Weg- en Waterbouw*')
[5] NTA stands for Dutch Technical Agreement (Dutch: '*Nederlandse Technische Afspraak*')

- *3D-printed bridge (Rijkswaterstaat, Municipality of Nijmegen, Witteveen+Bos, and more):*
  A 28 m long and 3,6 m wide 3D-printed concrete pedestrian bridge, the longest in the world, has been built in Nijmegen [19].
- *Modular bridge in Germany (ARUP, North Rhine-Westphalia):*
  As part of a pilot project, Arup has developed a new modular bridge system that cut construction time on-site by more than half, down to sixteen weeks from an initial estimate of about twelve months [20].

Partly due to the replacement and renovation task, in the short term there is an urgent need for validated solutions that Rijkswaterstaat can purchase and apply through a regular tender. However, at present there is insufficient validated supply of circular viaducts. For many of the above listed solutions it applies that it is not yet possible to apply the solution on a large scale because further (technical) development is needed, or because the solution has not yet been sufficiently tested and validated [9].

1.2. Research Objective and Scope

From the foregoing, it has become evident that validated solutions are needed to be able to build and maintain viaducts with a lower environmental impact over their entire lifespan (possibly consisting of a number of life-cycles). Therefore, the main part of this research is dedicated to developing circular solutions for concrete viaducts for (governmental) roads, as is the challenge posed by Rijkswaterstaat [9]. The focus is on design innovations, which are defined as "*new, future-proof, adaptive design solutions and design methods to preserve the value of existing and future objects, parts and materials and thus reduce the use of primary raw materials*" [9]. Within the category of design innovations, it is chosen to specifically focus on demountable, instead of modular, solutions for concrete viaducts. This specific research direction has been chosen based on several reasons:

1. The personal idea of a feasible and widely applicable concept in which the general layout of a circular concrete viaduct is standardised and consists of a number of compatible components, inspired by a typical IKEA building kit.
2. Lievense's vision on a successful and feasible concept of a circular concrete viaduct [21].
3. The principle of Design for Deconstruction (DfD) which, in the last two decades, has been gaining increasing attention in the research and development fields related to the construction industry.

In order to facilitate a successful process of demounting and reusing a concrete viaduct in a next life-cycle, data such as the structural properties, history of loading, current conditions, etc. of the different components has to be available and easily accessible at all times. Therefore, a subpart of this research concerns the monitoring aspects, which is focused on the question *where* to monitor *what*.

Besides, for the concept of a circular concrete viaduct to be feasible from a financial perspective, insight has to be obtained in the life-cycle costs of a circular concrete viaduct, and even more importantly, how this relates to the life-cycle costs for the same viaduct, constructed in a traditional way, characterised by a linear life-cycle model. Therefore, a second subpart of this research focuses on (a comparison between) these costs by means of a life-cycle cost analysis (LCCA).

Finally, the demarcation of the scope of the research is graphically visualised in Figure 1.2, and the above has been summarised into the following research objective:

> The research objective is to gain knowledge about the development, monitoring, and life-cycle costs of demountable solutions for concrete viaducts by conducting a design-orientated research consisting of collecting and analysing data which is used to propose and develop solutions for bottlenecks between traditional (non-demountable) and circular (demountable) concrete viaducts, to advise on desired monitoring aspects, and give insight in the life-cycle costs of a circular concrete viaduct.

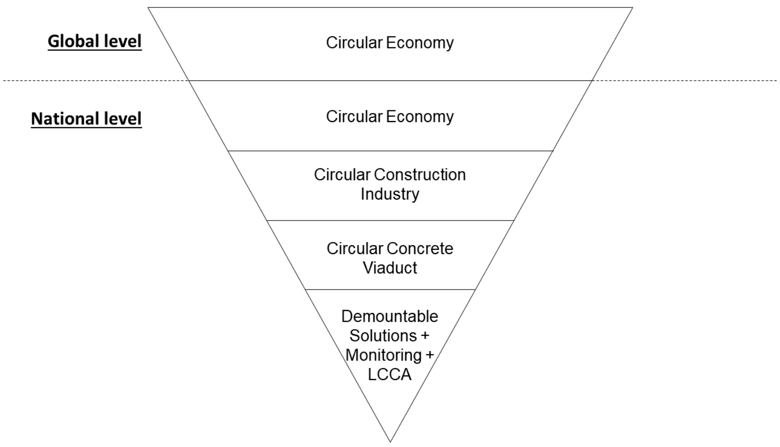


Figure 1.2: Graphical representation of the scope of the research

1.3. Research Questions

In order to achieve the research objective, a main research question and several supporting sub-research questions have been drafted. The questions have been chosen and formulated in such a way that the answers are either necessary or useful in achieving the research objective.

In summary, this research seeks to develop circular (demountable) solutions for concrete viaducts for (governmental) roads, to advise on desired monitoring aspects, and to give insight in the life-cycle costs of a circular concrete viaduct by addressing the following main research question and sub-research questions:

Main research question

> What is required in order to transform the traditional (linear) design of a concrete viaduct in the Netherlands into a circular (demountable) viaduct?

Sub-research questions

1. (a) What are the key action points for technical solutions that are needed to achieve a circular (demountable) concrete viaduct?
   (b) What are the main bottlenecks in current viaduct designs which make it unsuitable and/or impossible to be demountable and reusable?
   (c) What is/are possible technical solution(s) for the main bottleneck(s) in current viaduct design?
2. (a) What data regarding the properties and conditions of a circular (demountable) concrete viaduct and its elements and components is desired to be monitored?
   (b) How can the desire for monitoring-related data be incorporated into the design process of a circular concrete viaduct?
3. (a) What are the life-cycle costs of both a circular concrete viaduct and of the same viaduct, constructed in a traditional way, characterised by a linear life-cycle model?
   (b) Under what conditions is the concept of a circular concrete viaduct feasible from a financial perspective in comparison to the same viaduct, constructed in a traditional way, characterised by a linear life-cycle model?

1.4. Key Concepts

In order to avoid any misconceptions, the definitions of several key concepts as they are used in this research are defined below. In literature, multiple, slightly different, definitions for some of the terms exist. Here, those terms are defined as they are understood in the context of this research, most likely inspired by many of these different existing definitions in literature. Therefore, it must be emphasised that these definitions are not claimed to be original, nor definite or complete. Note that definitions of 'sustainability' and the 'circular economy' have already been given in subsection 1.1.1, and are therefore not repeated here.

- **Circular construction industry**: A construction industry, based on the principles of a circular economy, characterised by a closed loop materials cycle, also known as a 'cradle-to-cradle model', in which waste can ultimately be considered as a design mistake (see Figure 1.1). This means that structures/components/materials that have reached their end-of-life stage or are removed/demolished for any other purpose, are used as an input again in the life-cycle of a new structure, either directly or indirectly, in order to minimise the use of primary raw materials, and reduce waste, pollution and loss of embodied energy.
- **Demountable**: A structure is considered to be demountable when it can be disassembled into transportable and storable components without damaging components during disassembly and transportation (and storage) in such a way that excessive repairs and/or testing are needed before being able to reassemble it again in the same way in the same or in a similar structure, analogous to a typical IKEA building kit in which the final dimensions are fixed.
- **Design for Deconstruction (DfD)**: A principle that incorporates an end-of-life scenario in the design stage of a new to be designed structure in order to ease future disassembly and reuse, or relocation of the entire structure or parts (materials and components) of it. The principle is also known as 'Design for Disassembly', however, no explicit distinction between both terms is made in this research. The principle is explained in detail in section 2.1.
- **Embodied energy**: The amount of energy that has been consumed in all of the processes associated with the realisation of a structure such as extraction, transportation, processing, assembly and construction.
- **Modular**: A structure is considered to be 'modular' when it is 'demountable' (as defined above), but additionally it is possible to change the overall layout of the structure, i.e. dimensions and shape, by adding/removing parts, analogous to building with LEGOs in which the final dimensions and shape are variable.
- **Viaduct**: A bridge structure for road traffic crossing another road or railway, consisting of a deck supported by piers and abutments. Therefore, a bridge structure crossing water (rivers, canals, etc.) or an urban area is not considered to be a viaduct in this research. Besides, in the context of this research, when speaking of a viaduct, a concrete viaduct is referred to unless stated differently.
  - **Standard viaduct**: The term 'standard' should be interpreted in this research' context as 'most common (in the Netherlands)', i.e. a viaduct that has a very typical layout and is generally seen a lot in or crossing over Dutch (governmental) roads.

Furthermore, it is noted that the terms 'disassembly' and 'deconstruction' are most of the time used interchangeably in literature. Although separate definitions for both term have been proposed in literature (e.g. [22]), it is chosen to not make such a strict distinction between both terms in this research.

1.5. Report Outline

First, an extensive literature study related to the main topic of developing circular (demountable) solutions for concrete viaducts as well as some relevant literature with regards to monitoring of concrete structures is summarised in Chapter 2.

Subsequently, technical action points that are required in order to achieve circular bridge construction, and which are used in order to systematically determine the layout and design of a standard circular (demountable) concrete viaduct, and consequently to develop demountable solutions for such a viaduct, are elaborated upon in Chapter 3.

After establishing this framework, first of all, typically found bottlenecks preventing and/or limiting current viaduct designs to be demountable are identified in Chapter 4, and subsequently, the layout and design of a (proposal for a) standard viaduct are discussed. This step is required first before moving on to Chapter 5, in which the development of a concept demountable solution, namely a concept demountable footing to foundation (F2F) dowel connection, is discussed, which is based on and suitable for the defined standard viaduct. Besides, a design table for the same connection with different properties, dimensions, and load capacities is presented, as well as two sensitivity analyses, a discussion of the practical aspects of different execution variants, and the verification and validation of the developed solution. In summary, Chapter 4 focuses on the concept of a standard (circular) viaduct design (i.e. 'the bigger picture'), whereas Chapter 5 zooms in on the development of a demountable F2F dowel connection which is based on and suitable for application in the standard (circular) viaduct.

Next, the first subpart of this research regarding monitoring of the circular concrete viaduct in general and of the developed concept demountable F2F dowel connection is covered in Chapter 6. Subsequently, the second subpart of this research regarding the life-cycle costs of a circular concrete viaduct is discussed in Chapter 7, in which the life-cycle costs of a circular concrete viaduct are compared to those of the same viaduct, constructed in a traditional way, characterised by a linear life-cycle model.

Finally, an extensive discussion of the results, the conclusions of the research, answering the main and sub-research questions, and recommendations for future research are given in Chapter 8.

A schematic overview of the research outline, as well as the respective software used, is shown in Figure 1.3.

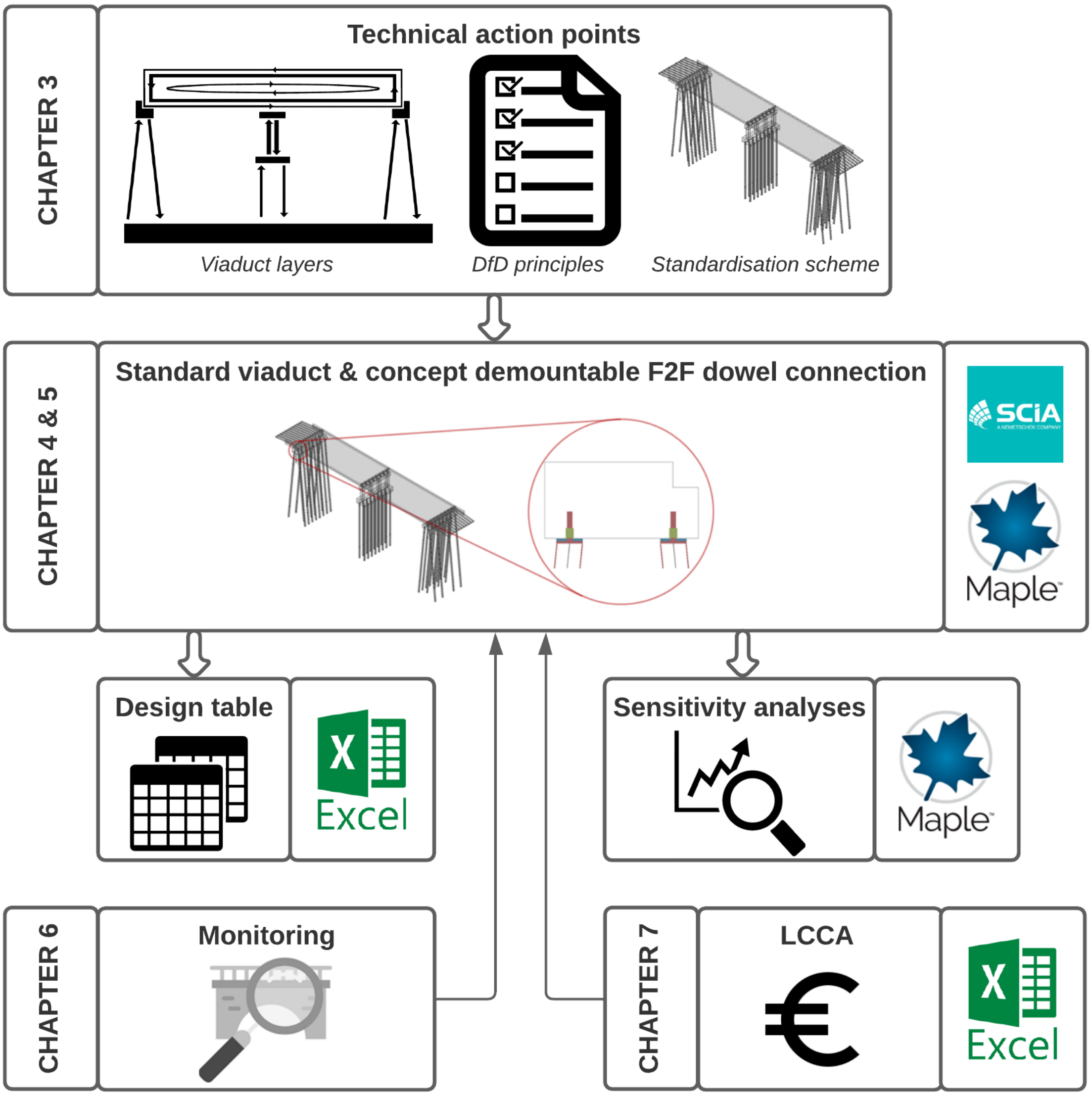


Figure 1.3: Overview of research outline

2

Literature Study

In this chapter, the literature that is relevant to this research is summarised. This literature can be roughly divided into two parts, namely that related to the main topic of developing circular (demountable) solutions for concrete viaducts, and that related to monitoring aspects. The part related to the main topic is covered in sections 2.1–2.3, which consecutively deal with the principle and theory regarding Design for Deconstruction (from this moment on referred to as 'DfD'), its application in bridge construction, and the development of concrete DfD connection methods. In section 2.4, relevant literature related to the monitoring of concrete structures is shortly addressed.

Finally, two additional sections, which don't necessarily involve literature, are dedicated to the more practical side of the situation in the Netherlands. Section 2.5 deals with the applicable building codes for the construction of a concrete viaduct in the Netherlands, and in section 2.6 the different types of (existing) concrete viaducts in the Netherlands are analysed and discussed.

2.1. Design for Deconstruction (DfD)

In this section, the principle of DfD, which was briefly discussed in section 1.4, is elaborated upon in detail. Firstly, the challenges that were, and sometimes still are being, faced by (design for) deconstruction, and its potential are highlighted. Subsequently, the development of DfD in the construction industry is explained by means of lessons learned from industrial design industry and historic and more recent examples. Finally, a framework that can assist in systematically applying DfD in the construction industry is discussed in detail as it is used to address the technical action points that are elaborated upon in Chapter 3, and to develop a standard viaduct as well as a concept demountable solution in Chapter 4 and Chapter 5 respectively.

2.1.1. Challenges and Potential of DfD in the Construction Industry

As has been pointed out in subsection 1.1.2, it is believed that the transition from a linear to a circular construction industry, in which the strategy of demolition and disposal is replaced by a strategy of disassembly, can play a major role in addressing the sustainability issues in the construction industry. However, it was already argued by Crowther [23] in 2000 that deconstruction is severely limited by the fact that the vast majority of existing buildings is simply not designed to be deconstructed. In other words, these buildings are almost all designed and constructed with a linear life-cycle model of the built environment in mind. This creates multiple technical barriers to the successful deconstruction of these buildings, i.e. the recovery and reuse of components and materials, and severely limits the end-of-life options of buildings [7]. Some other, more specific, problems in building designs that limit deconstruction were identified by Guy and Ciarimboli [24], which include:

- Increased use of composites and engineered products which are difficult to recycle because of their chemical complexity.
- Costs of labour to deconstruct and process commingled recovered materials, and the ability to use human, mechanical, thermal, optical and even sonic means of separation.
- Use of connection techniques such as pneumatically driven nails, staples and adhesives that are extremely difficult to “undo”.
- Loss of craft skills such that the labour costs to create exposed connections and details that are also aesthetic, is prohibitive.
- Reliance on coatings and encapsulation of elements with innumerable layers of finish materials instead of integral building envelope, finish and structural systems.
- The highly speculative nature of buildings, whereby there is not a long-term ownership, and therefore adaptation, renovation and demolition costs are not borne by the original owner.
- The perception that incorporation of components and systems designed-to-be-disassembled, other than those explicitly meant to have short lives (like exhibition spaces, entertainment venues, etc.) will reduce value and imply other aesthetic or safety compromises.

However, they conclude that the main obstacles for deconstruction are ‘time to deconstruct’ and ‘low disposal costs’, which, Guy and Shell [25] argued, are interrelated. It has to be noted though that their conclusion is based on a research in 2004, so these main obstacles might have changed by now.

Kibert et al. [26] identified eight challenges faced by deconstruction, which generally fit into either the category ‘design’ or ‘policy’. These challenges are:

- Existing buildings have not been designed for dismantling.
- Building components have not been designed for disassembly.
- Tools for deconstructing existing buildings often do not exist.
- Disposal costs for demolition waste are frequently low.
- Dismantling of buildings requires additional time.

- Re-certification of used components is not often possible.
- Building codes often do not address the reuse of building components.
- The economic and environmental benefits are not well-established.

According to them, these challenges can all be readily overcome provided that changes are made in both design and policy. Also here, it has to be noted that this list is based on a research in 2000, so (some of) these challenges might be less relevant nowadays.

However, most of the above mentioned problems still exist nowadays. Although in today's building design usually considerations are being made about the materials used and the embodied energy in them, and a lot of effort is put into making buildings sustainable from an operational perspective, still most buildings are constructed with the production of a building as the final goal, i.e. with a linear life-cycle model in mind, and very few building designers, contractors and clients consider the end-of-life scenario. Most buildings are cast in-situ, built-in, chemically bonded or even a combination of this, which makes deconstruction almost impossible. Therefore, the predominant end-of-life scenario still is demolition, although deconstruction of buildings in order to recover materials and components has become more common [23]. Another reason why deconstruction and the application of DfD in building design, despite the hype of the concepts among practitioners, still hasn't achieved success in today's construction industry, as suggested by Rios et al. [27], is due to its impracticality imposed by codes, standards and professional practices. Furthermore, they found that other hindrances discussed in literature dealt with time constraints, costs, contractual issues, the lack of involvement and responsibility of manufacturers to reduce waste, and the lack of methods to measure the benefits of deconstruction and the recyclability of materials and buildings. However, the main hindrance for deconstruction identified by past studies according to Rios et al. [27] is the design process. Therefore, it seems that all problems finally can be traced back to the earlier mentioned statement that the main obstacle for deconstruction of existing buildings is that they are simply not designed to be deconstructed.

Therefore, the potential of DfD as a solution to the challenges currently faced by deconstruction is widely recognised in literature [1, 5, 7, 23–30]. For example, according to Anastasiades et al. [1] the way that structures are designed becomes one of the main priorities in order to successfully transit from a linear to a circular life-cycle model of materials and components. Additionally, it is argued by Crowther [7] that a cyclic view of the built environment and the materials within it not only considers the construction process at the design stage, but also recognises that the process of disassembly has to be taken into account at this stage. Besides, it has been argued by Guy and Shell [25] that the increasing strictness of legislation concerning the disposal of building materials together with the value of recovered materials, both in environmental and economic terms, are the main opportunity factors for (design for) deconstruction. These are just few of the many reasons why the principle of DfD has been gaining increasing attention in the research and development fields related to the construction industry in the last two decades.

2.1.2. Development of DfD in the Construction Industry

According to Salama [30], in 1999 Philip Crowther was the first person to approach the application of DfD in the construction industry in a theoretical and scientific instead of experimental way by applying it on buildings. In the period that followed, until 2005, he investigated and wrote several articles regarding the application of DfD in the construction industry [5, 7, 23, 29, 31]. However, before he started investigating the application of DfD in the construction industry, he first investigated the lessons to be learned from the application of DfD in the industrial design industry [28], and from historic and more recent examples [32].

Lessons learned from industrial design industry

Whereas in the construction industry the implementation of DfD is still very limited, its application in the industrial design industry, e.g. car manufacturing and computer industry, is already common practice for many years now. In 1994, for example, General Motors, Chrysler and Ford already formed the Vehicle Recycling Partnership to develop ways to recover materials from cars for the purpose of reuse and recycling [25]. Crowther [28] pointed out that before 1999 four major strategies to reduce resource use had already been investigated and implemented in the industrial design industry by companies like Xerox, Eastman Kodak, Hewlett-Packard, and BMW. These four major strategies were 'dematerialisation' (Reduce), 'material substitution' (Reduce), 'recycling and reusing' (Recycle and Reuse), and 'waste mining' (i.e. the use of waste from one production process as resource for another; Recover). The strategies were used by researchers and developers to create guidelines for product designers on how to design for disassembly. It was found by Crowther [28] that many of these guidelines could be easily adapted for the application in the construction industry in order to assist building designers to design for disassembly. These guidelines, originating from the industrial design industry and applicable for the construction industry, are listed in Table 2.1. He realised that this list wasn't a complete list of design guidelines for the construction industry yet, and that also design guidelines existed that are only suitable to the construction industry and not to the industrial design industry. However, these guidelines turned out to be a good starting point for the development of a list of key principles for DfD in the construction industry, as is explained in subsection 2.1.3.

Table 2.1: Guidelines for DfD from industrial design industry applicable for construction industry [28]

	Guideline
1.	Minimise the number of different types of material
2.	Avoid toxic and hazardous materials
3.	Use materials compatible with standard recycling practice
4.	Do not join different materials in an inseparable way
5.	Avoid secondary finishes to materials
6.	Provide standard and permanent identification of material types
7.	Minimise the number of different types of components
8.	Use mechanical not chemical connections
9.	Minimise the number of different types of connectors
10.	Design to use common tools and equipment, avoid specialist plant
11.	Provide access to all parts and connection points
12.	Make the most reusable parts most accessible
13.	Allow for easy handling and cleaning
14.	Sustain information on location of reusable components
15.	Use modular or standard design

Guy and Shell [25] discussed a design for disassembly tool for products applied in the industrial design industry, which was named "End of Life Design Advisor" (ELDA). The tool used a list of key characteristics to determine a product's potential for disassembly and material reuse/recycling. They found that this list provided generic guidelines for the design for deconstruction of buildings. The list, differentiating between critical and non-critical end-of-life factors, consists of the following key characteristics:

Critical End-of-Life Factors

- Number of parts
- Number of materials
- Cleanliness of the product - amount of dirt accumulated by product
- Design cycle - time between new designs
- Technology cycle - time that product will be cutting edge before new technology makes it obsolete
- Replacement life - time that average user upgrades product

Non-Critical End-of-Life Factors

- Size
- Number of modules
- Hazards and hazardous materials - components that need to be removed before further recycling

- Wear-out life
- Reason for obsolescence
- Functional complexity - high level of dependence between parts with multiple functions

Because of the obvious differences between products and buildings, for example the fact that buildings are generally larger and more sensitive to gravitation than products, Guy and Shell [25] argued that the non-critical factors 'size' and 'hazards and hazardous materials' for product are in fact critical for buildings. Besides, they recognised that buildings, unlike products, additionally are subject to a certain fixed location which has consequences on all levels of design, such as the forces acting on it and the influences of weather, alterations, and different ownership. Since in essence therefore every building will have unique requirements, they argued that design for deconstruction for buildings will have to be specific for each building.

Lessons learned from historic and more recent examples

In an attempt to learn from historic and more recent examples, firstly Crowther [32] began to research why buildings are demolished at all at some point in their life. He argued that distinction can be made between the 'economic life' and the 'physical life' of a building. The economic life of a building refers to a certain predetermined period of investment within which an economic return is achieved, whereas the physical life of a building refers to the period of time within which the building can serve as a safe and functional structure. Usually, the physical life of buildings is significantly longer than the economic life. Crowther [32] believed there are potentially five main reasons for the physical lifetime of a building not to be reached, i.e. to demolish a building:

1. **Locational obsolescence**: the function of the building is no longer appropriate or needed in its current location.
2. **Functional obsolescence**: the function of the building is no longer needed within society.
3. **Technical obsolescence**: the building can no longer attain expected performance standards.
4. **Physical obsolescence**: the building or its components have fallen below acceptable standards of safety or amenity due to deterioration.
5. **Fashionable obsolescence**: the building no longer meets current standards of style and trend.

Subsequently, he highlighted several examples of both historic and more recent buildings and building traditions that revealed a number of characteristics that had been applied in an attempt to overcome or reduce one or more of these types of obsolescence:

- Light-weight, demountable and transportable tents of nomadic societies, made of appropriate materials which additionally were simple to replace.
- Traditional Japanese wooden houses consisting of primary, structural, and secondary framing members, shaping the floor plan, which could be disassembled and placed differently without interfering with the primary members.
- The Manning Portable Colonial Cottage (in 1624) which consisted of several prefabricated and pre-painted timber pieces that fitted exactly into each other, and could be (dis)assembled by an unskilled labourer using just a wrench.
- The Crystal Palace in London (in 1851) which was characterised by a simple system of prefabricated units and repetitive techniques, and which had been disassembled and re-assembled into a new building with a significantly different design in 1854.
- The demountable Nissen (Hospital) Huts of which tens of thousands were constructed during the First World War, and which showed the potential of mass production of buildings with the same design.

- The unrealised, futuristic concepts like 'The Walking City', a forty storey moving building, by a group of architects named 'Archigram' in the 1960's which highlighted the importance of the sequence of assembly and disassembly.
- The IGUS factory in Germany which was constructed in the 1990's (and still is in service) as a column free space, accommodating a completely flexible floor plan by placing all internal ancillary spaces on movable 'pods' that can be moved around through the entire factory.

The characteristics revealed by these examples, shown in Table 2.2, proved to be valuable in the creation of a list of key principles for DfD, as is explained in subsection 2.1.3.

Table 2.2: Characteristics derived from a number of historic and more recent examples of buildings and building traditions, and their similarities with guidelines from the industrial design industry [32] (adapted)

	Characteristic	Similar to guideline(s) in Table 2.1
1.	The use of appropriate, light-weight materials	2, 3
2.	The separation of structure and building envelope	(12)
3.	The use of simple connections	4, 8, 10 (9, 11)
4.	Completeness of the building system	4, 11
5.	An open system of construction	(15)
6.	A standard structural grid	15
7.	The use of a limited number of standard components	7 (1, 9)
8.	The use of a mass production process for buildings with the same design	15
9.	The sequencing of assembly and disassembly	11, 12, 14
10.	The application of disassembly potential on all levels, from recycling of materials to the reuse of whole buildings	All

Besides the lessons learned from these examples, Kibert et al. [26] reviewed the status of deconstruction in several countries worldwide in 2000. The main lesson learned from this international overview was, again, that the buildings had not been designed and constructed to ease future deconstruction. They recognised that changes and developments on a broad scale were required, such as the the development of techniques and tools to properly deconstruct buildings, research into the topic of deconstruction, and changes in policies promoting deconstruction by increasing disposal costs or even prohibiting the disposal of reusable materials.

2.1.3. Framework for DfD in the Construction Industry

According to Crowther [7], from the review of DfD in the industrial design industry and the historic and more recent examples, it can be concluded that there are two types of knowledge that are relevant in order to design for deconstruction. He distinguished between broad themes that address issues of *why*, *what*, *where*, and *when* to disassemble, and key principles on *how* to design for deconstruction, which he fit into a framework consisting of a total of four themes and principles. The three broad themes that significantly impact on the decision making process of designing a building for future deconstruction are:

- A holistic model of sustainable construction.
- The perception of a building consisting of several layers with different service lives.
- A recycling hierarchy recognising the benefits of different end-of-life scenarios.

Holistic model of sustainable construction

The potential environmental benefits of DfD are obvious and some of the first to be identified by Kibert et al. [26] include the increased diversion rate of demolition waste from landfills, the potential reuse of building components, an increase in the ease of recycling, and enhanced local and global environmental protection. Besides, they realised that deconstruction preserves the embodied energy in materials and thus reduces the input of new embodied energy in the reprocessing or (re)manufacturing of new materials, with a significant potential reduction in landfill space as a consequence. Other than just environmental benefits, Rios et al. [27] additionally identified social benefits (e.g. creation of jobs in deconstruction practice), economic benefits (e.g. creation of new markets for salvaged materials), and other benefits such as historic preservation and the earning of credits for green building certificates.

Despite these many potential environmental benefits that DfD offers, it is pointed out by Crowther [7, 29] that the principle of DfD should fit within a broader understanding of sustainable construction, and of the global environment in general. It is important to remember that it is not the goal to design a demountable structure. The goal is to achieve a sustainable construction industry, and DfD is merely a (promising) means to that end, analogously to the conclusion of Anastasiades et al. [1] that the circular economy is a means to achieve the goal of a more sustainable economy, as was mentioned in subsection 1.1.1. This means, for example, that if the life-cycle costs of a building that has been designed for deconstruction turns out to be higher than the actual potential benefits, it is undesirable to built it in this way. However, then it has to be assured that these life-cycle costs have included the full life-cycle of the building, since it is pointed out by both Crowther [7, 28] and Guy and Shell [25] that in the short term DfD might have added economic and possibly even extra environmental costs, but the benefits on a much larger scale of the life-cycle of resources are potentially much greater. This emphasises the importance to recognise and consider these consequences in a holistic model that allows the place and role of DfD to be seen within the overall picture of sustainable construction.

A widely known tool which deals with this topic is life-cycle assessment (LCA). LCA allows to understand and quantify the environmental impact of buildings and products or, moreover, the actions of humans in general. In a LCA study, all inputs (e.g. materials, energy, etc.) and outputs (waste, pollution, etc.) over the full life-cycle of a building or product are taken into account and quantified in order to identify its overall environmental performances. Usually, the result of a LCA is a two-dimensional plot or graph in which the environmental impacts in each life-cycle stage can be observed, analysed and compared easily. However, it was recognised by Crowther [7, 29] that such a two-dimensional result does not offer strategies on how to deal with these (unwanted) environmental impacts. That is why he proposed to add a third axis of 'principles' in order to create a holistic model of sustainable construction, based on an idea that had been investigated earlier by Kibert [33]. First of all, the model, shown in Figure 2.1, illustrates the large number of issues that concern a sustainable construction industry, and the relationships between them. Besides, the model can be used as a decision-making tool in a construction design process. At every point of intersection of the three axes there are decisions to be made which impact the final outcome in terms of sustainability [7].

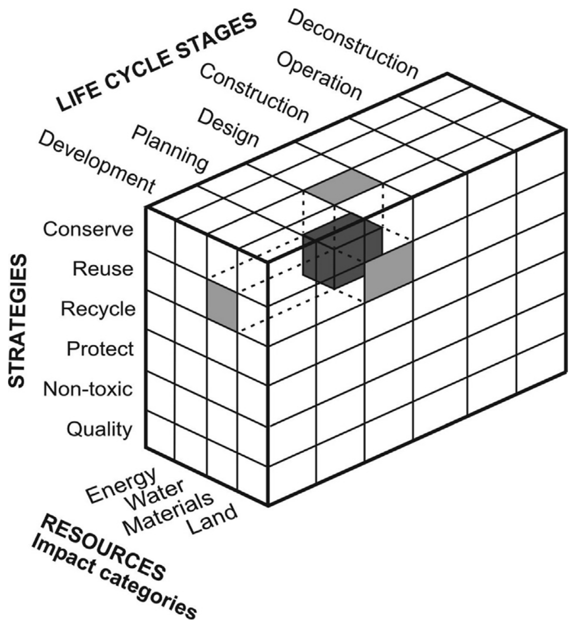


Figure 2.1: Model for sustainable construction highlighting the main area of concern of DfD [7]

Furthermore, in Figure 2.1 the main area of concern of DfD is highlighted which emphasises that DfD deals with the *design* stage of a building for the purpose of the *reuse* (preferred over recycling) of *materials*. Most of the times the reuse strategy is preferred over recycling because the aim of DfD is to apply the most sustainable strategy, which usually puts 'reuse' above 'recycling' [29], as is explained in the third theme concerning the recycling hierarchy (see subsection “Recycling hierarchies”, page 20).

Conclusively, it was stated by Crowther [7] that the model highlights the fact that the act of designing for deconstruction has to be considered in a broader understanding of sustainable construction, and that it shows the potential relationships with other environmental issues and strategies. Therefore, the model can assist designers in understanding *why* and *when* to design for deconstruction. However, Rios et al. [27] argued that despite the existence of LCA, still appropriate methods to measure the benefits of deconstruction and the recyclability of materials and buildings are lacking, which causes recycling processes to be oversimplified and prevents its costs and benefits to be measured in an efficient way. Therefore, they argued that efforts should be made to develop these methods, for example by studying successful cases of DfD. Similarly, Anastasiades et al. [1] were concerned with the question how circularity, as a result of DfD, should be measured. They pointed out that tools based on LCA principles only indicate whether one solution is more sustainable than another solution, but that this does not prove that the solution is sustainable in the broader context of sustainable construction. Therefore, they argued that certain reference values should be defined that indicate when solutions can be considered sustainable by themselves, as was proposed by Lavagna et al. [34]. Besides, they found that although LCA considers the complete life-cycle of an object, often the end-of-life phase is not or inconsistently taken into account and therefore circularity is not being measured in a clear and correct way. Therefore, the idea of performing so-called 'multi-cycle assessment' was opted, instead of LCA, based on yet to be developed circularity indicators that evaluate the meso-scale of construction, i.e. the construction or building by itself.

Theory of layers

The construction industry is still mostly characterised by the perception of a building as a singular, time-independent entity. However, it can be argued that buildings alter over their entire lifespan, and therefore can be considered as a series of different buildings over time [29], i.e. to be consisting of different time-related building layers. The earlier mentioned historic and more recent examples on page 15 can be argued to have recognised the existence of these layers.

In the 1960's, the Japanese Metabolism architects and John Habraken were the first people to write theoretically about building layers that could respond to their change over time. With his writings, Habraken made the first step in dissecting a building into layers. Later, Habraken [35] developed his 'traditions of two stage building' theory in which he separated between a primary structural frame, typically supporting the roof, and a secondary system of construction defining the internal spaces, each having different lifespans [29].

In 1961, Cedric Price, another innovative thinker who was also concerned with the way that different building layers might have different lifespans, developed an inspirational scheme for the 'Fun Palace' [36]. Although it was not realised, the Inter-action community centre, built in the 1970's and actually classified by the county as a temporary structure, followed many of the principles of the Fun Palace. One of the principles that had been included was a complete set of instructions for disassembly of the building [29].

The earlier mentioned group of architects by the name of Archigram were one of many that had been inspired by Price's work. Another one of their concepts was 'The Plug-in City', in which parts that needed most frequent service or replacement were actually made most accessible. This notion of a hierarchy of obsolescence had been incorporated by means of a list of different life expectancies for different elements in the building, ranging from a couple of months (tenancy in a shop) to 20 years (roads and civil works). At the same time, the Japanese Metabolism Group were pursuing similar ideas, also incorporating a hierarchy of different life expectancies for various elements within the built environment. Most of the works of both groups, however, has never been realised [29].

Duffy and Henney expanded upon the two layer theory by Habraken, and instead introduced a theory of four time-related building layers in 1989. This theory was developed by means of an analysis of London office buildings, but is also appropriate to other building types. An important addition to Habraken's theory is the fact that Duffy and Henney, like Archigram and the Metabolists, assigned a specific service life to each building layer, which they based on experience of two types of changes: the changing demands of users and the required upgrades or expansion of plant and equipment. The four layers and their respective assigned service life are [29]:

- **Shell**: foundation and structure of the building, including non load-bearing components; 50 years.
- **Services**: electrical, hydraulic and HVAC (Heating, Ventilation and Air-Conditioning) systems, and lifts and data; 10-15 years.
- **Scenery**: internal partitioning system, finishes, and furniture; 5-7 years.
- **Set**: freely movable items by users to suit their daily needs; several days or weeks.

Whereas Duffy and Henney limited their theory to the internal parts of a building, the nowadays famous theory consisting of six time-related building layers, also known as 'shearing layers of change', developed by Brand [37] in 1994 also includes external layers. Like Duffy and Henney, Brand assigned a specific service life to each layer as well. However, Brand based his lifetimes on a general understanding of how buildings change over time. Brand's layers, which are directly built on Duffy and Henney's theory, are (see Figure 2.2) [29]:

- **Site**: the building site on which the building stands; eternal service life.
- **Structure**: foundation and load-bearing components; 30-300 years.
- **Skin**: cladding and roofing system protecting the interior from external natural influences, i.e. building envelope; 20 years.
- **Services**: electrical, hydraulic and HVAC (Heating, Ventilation and Air-Conditioning) systems, and lifts and data, same as Duffy and Henney; 7-15 years.
- **Space plan**: internal partitioning system, finishes, and furniture, same as Duffy and Henney's "Scenery"; 3 years (commercial building) to 30 years (house).
- **Stuff**: freely movable items by users to suit their daily needs, same as Duffy and Henney's "Set"; several days or months.

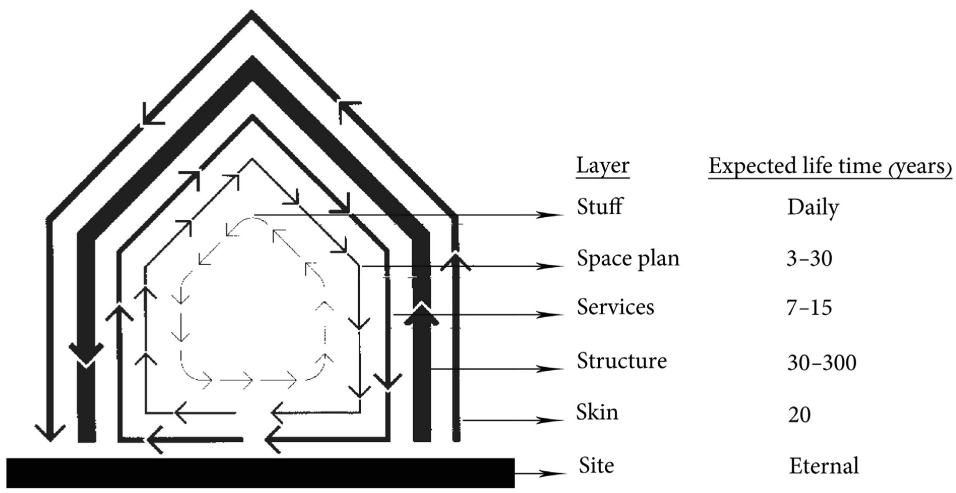


Figure 2.2: Brand's model of six time-related building layers [30]

Brand [37] wrote in detail about both the technical and social benefits of keeping these layers separate from each other when designing and constructing buildings. Also, he incorporated the lessons learned from historic buildings and building traditions, like Habraken did, and he suggested that designers should learn from these lessons. Besides Duffy and Henney, and Brand, there are a number of other people who similarly, but often from a different perspective, also defined service life expectancies to different layers of a building. These different perspectives range from particular building designs and life-cycle costs to issues of sustainable 'green' buildings and of embodied energy [29].

Crowther [29] argued that the six layers as proposed by Brand should not be seen as a definite or untouchable number. Depending on building typology and design, the building might be divided in more or less layers, especially when it is tried to link it with DfD. Besides, he argued that these theories of time-related building layers could be of great importance for DfD. He found that although several researchers, like Craven, Okraglik and Eilenberg in 1994, and Fletcher, Popovich and Plank in 2000, recognised the importance of the theory of time-related building layers in the field of DfD for buildings, none of them however indicated how this theory can or should be implemented within a strategy of DfD, nor how it interacts with other ideas like for example material recovery. Therefore, the main lessons to take away from the theories discussed above according to Crowther [29] is that the interfaces between different layers should become point of attention when designing for deconstruction. He stated that both Habraken, Duffy and Henney, and Brand agreed upon the fact that the separation of layers is the most important point of attention in order to successfully design and construct technically and socially adaptable and responsible buildings. This is exemplified by Guy and Ciarimboli [24], who state that if, for example, a change in the configuration of the *Space plan* is required, but this is prevented because the *Structure* will not allow this change to take place, this *shearing* of the layers might cause premature obsolescence or even demolition of the entire building.

This directly emphasises the main advantage of adopting the theory of time-related building layers, namely that building components are located in different separable layers and can therefore be modified or disassembled without affecting other layers [29, 38]. Therefore, it is argued by Crowther [7, 29] that an understanding of these time-related building layers will assist designers in deciding *where*, i.e. in which layer, and *when*, related to the service lifetime, to apply DfD. However, as has been pointed out by Anastasiades et al. [1], this theory does not propose any scenario for the end-of-life stage of materials, components or even an entire building.

Recycling hierarchies

As has been pointed out several times already, the life-cycle model in the construction industry is still characterised by a once-through, or an 'use-and-dispose' model of materials. The steps in such a linear life-cycle model are typically characterised by the consecutive stages of extraction, processing, manufacturing, assembly, use, demolition and disposal. The option of turning this linear life-cycle model into an actual cyclic, i.e. circular life-cycle model, has also been addressed several times already. However, the hierarchy of different recycling strategies has not been discussed yet.

Crowther [7] came up with four different end-of-life scenarios, i.e. recycling strategies, namely:

- Relocation or reuse of whole building
- Reuse of components into new buildings
- Recycling of materials into new components manufacturing
- Reprocessing (down-cycling) of materials into new materials processing

Crowther [7] argued that if the principle of DfD would be applied to the construction industry in order to replace the strategy of demolition and disposal by a strategy of disassembly, the linear life-cycle model could be turned into a circular life-cycle model by implementing these four end-of-life strategies, as can be seen in Figure 1.1. This way, the lifetime of materials, components and entire buildings can be extended, which in the end is the goal of DfD, as argued in different papers [1, 28].

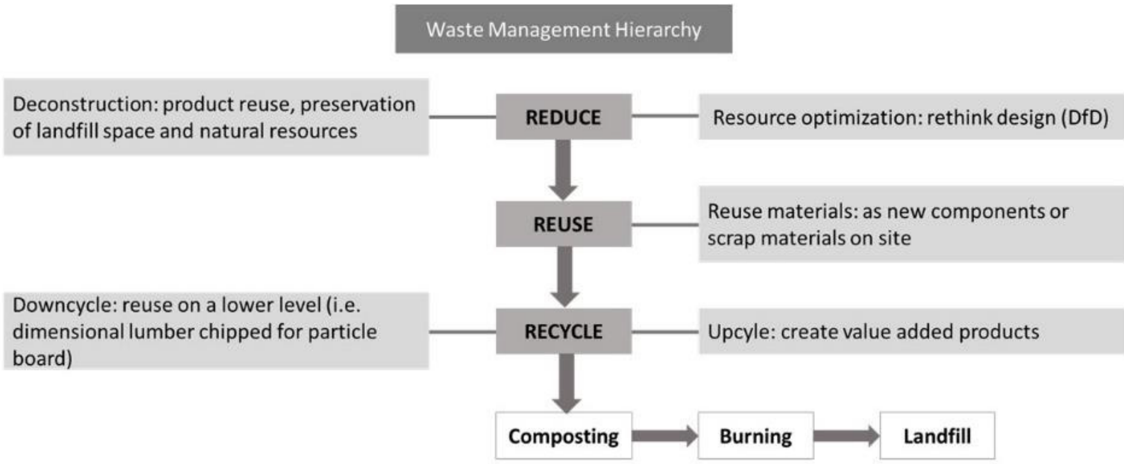


Figure 2.3: Waste Management Hierarchy [27]

Another model, proposed by Kibert et al. [26], described a similar hierarchy, however it identifies an additional strategy that is not explicitly addressed by Crowther's model [7]. Their model, also developed to address the disposal of materials from demolition and construction operations, puts the strategy of reduction at the top of the waste management hierarchy, above the strategies of reuse and recycle, and the less preferred strategies of composting, burning and land-filling (see Figure 2.3). However, it has been argued by Rios et al. [27] that the principle of DfD indirectly incorporates the strategy of reduction, since the four end-of-life scenarios of Crowther [7] independently ensure the reduction of the input of raw materials. Therefore, Anastasiades et al. [1] argued that the recycling strategies can be said to have been captured in the so-called "4R framework" (Reduce, Reuse, Recycle and Recover) of the circular economy as defined by the European Union. Basically, these 4 R's can already be recognised in the four major strategies to reduce resource use, developed in the industrial design industry in the 1990's, as was discussed on page 14. Applying the principle of DfD results in the *reduction* of natural (finite) resource extraction as a result of lifetime extension of materials and components, the *reuse* of materials, components and even entire buildings as they are easy to disassemble, the *recycling* of materials and components once their actual end-of-life stage has been reached as they are easy to separate, and the *recovery* of materials, to make new materials, and of embodied energy [1].

Based on the combination of findings of case studies and an analysis of existing hierarchies of waste recovery (amongst which was the model proposed by Kibert et al. [26]), Crowther [39] developed a new model specific to the construction industry. The model, shown in Figure 2.4, represents a taxonomy of nine strategies for construction material and component reuse and recycling. It illustrates the hierarchy of environmentally preferred reuse and recycling options in the construction and demolition industry. This taxonomy can replace the the model of four recycling strategies, as it is basically an upgraded version of it.

Furthermore, Crowther [29] emphasised that it is important to realise for which intended end-of-life scenario a material or component is designed. For example, it can be decided to design an entire building to be demountable, or just parts of a building like the facade and the rest of the building for component or material recycling. It is important to realise that such a relationship exists between the intended end-of-life scenario and DfD. Besides, Crowther [7, 28] stated that in designing a building for disassembly, the designer should always encourage the scenarios of direct reuse, either relocation of the entire building or the reuse of components in another building, over recycling or reprocessing. Although it is clear that all four scenarios (or nine when considering the model in Figure 2.4) will generate less waste than demolition, the scenario of reusing will generally also require less energy and raw materials input than recycling or reprocessing and will therefore be more sustainable. It is argued by Crowther [7] that in the end buildings should be designed for all four (or nine) recycling strategies, since the future reuse of buildings is always based on uncertain predictions.

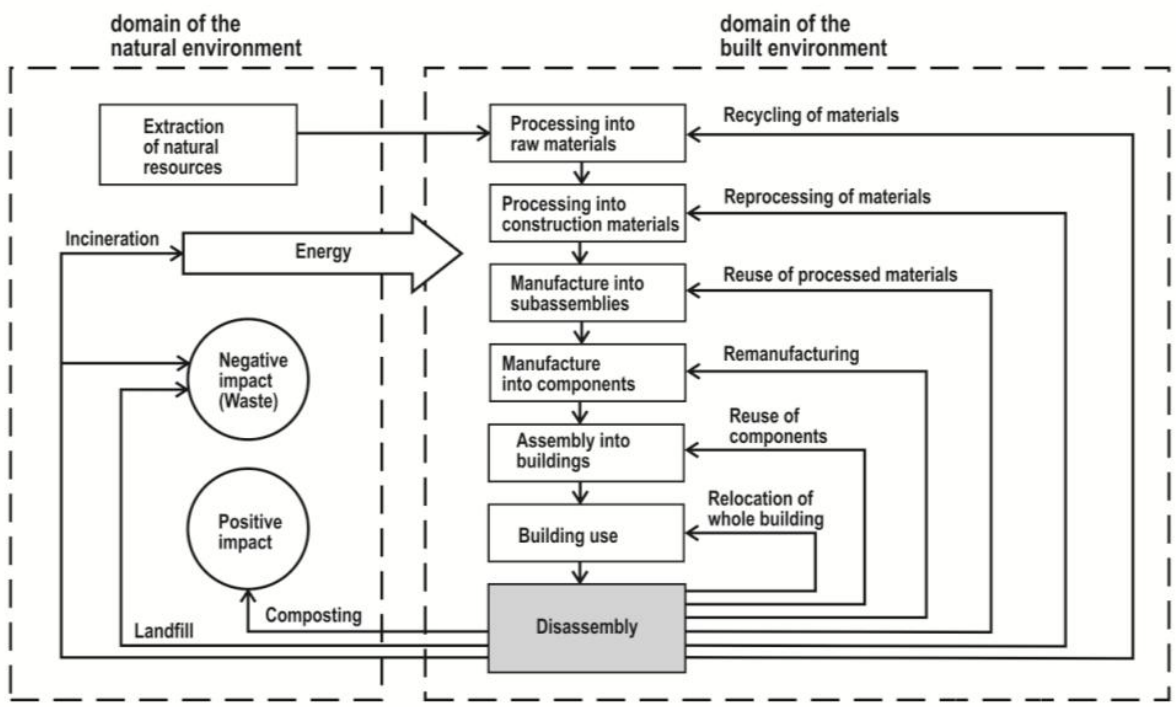


Figure 2.4: Taxonomy of hierarchical levels of waste recovery strategies [39]

An understanding of this hierarchy of recycling can assist designers in applying DfD in their designs, and can offer guidance on *what* to design for which recycling strategy. However, once again it is emphasised that the principle of DfD should fit within a holistic model of sustainable construction. It is possible, for example, that for a certain project there are other environmental issues such as autonomous energy generation or the avoidance of all toxic content that outweigh the benefits of designing the project to be demountable [7].

Key principles for DfD

Although the three broad themes (a holistic model for sustainable construction, the theory of time-related layers, and a recycling hierarchy) are important in assisting designers in successfully designing buildings for deconstruction according to Crowther [7], so far the framework does not explicitly address the topic of *how* to design for deconstruction. For that matter, a carefully composed list of 27 key principles (or guidelines, characteristics or rules) on *how* to apply DfD has been created by Crowther [7]. The list has, amongst others, been based on the guidelines for DfD from the industrial design industry as shown in Table 2.1, and on the characteristics revealed by the historic and more recent examples of buildings and building traditions as discussed on page 15 and shown in Table 2.2. Furthermore, the list is based on some realised projects, but also on conceptual ideas of buildings, and on the theory of time-related building layers. The list of key principles as proposed by Crowther [7], shown in Table 2.3, rates each principles against the four recycling strategies and ranks them either as ‘highly relevant’, ‘relevant’ or ‘not normally relevant’. Elaborated explanations of the principles are found in Appendix A where the respective pages of the paper by Crowther [7] are added.

Besides technical change, which are mainly dealt with by the list of key principles, Crowther [23] argued that it requires cultural change as well in order to successfully implement DfD into the construction industry. One of the main challenges with regard to cultural change is the fact that, as argued by Anastasiades et al. [1], it this seems to be that the construction industry in general is lacking the motivation to adapt the traditional, linear, building process to a sustainable, circular, one. Therefore, they suggested that politics should take the lead by means of legislation and well-oriented incentives in order to mobilise designers in particular, who themselves have the opportunity to create demountable designs.

Table 2.3: Principles of DfD and their relevance to the strategies of recycling [7]

	Principle	Material recycling	Component remanufacture	Component reuse	Building relocation
1.	Use recycled and recyclable materials	●	●	·	·
2.	Minimise the number of different types of material	●	●	·	·
3.	Avoid toxic and hazardous materials	●	●	·	·
4.	Avoid composite materials and make inseparable subassemblies from the same material	●	●	·	·
5.	Avoid secondary finishes to materials	●	●	·	·
6.	Provide standard and permanent identification of material types	●	●	·	·
7.	Minimise the number of different types of components	·	•	●	●
8.	Use mechanical, not chemical connections	·	●	●	●
9.	Use an open building system not a closed one	·	·	●	•
10.	Use modular design	·	·	●	•
11.	Design to use common tools and equipment, avoid specialist plant	·	•	●	●
12.	Separate the structure from the cladding to allow for parallel disassembly	·	·	●	•
13.	Provide access to all parts and connection points	•	•	●	●
14.	Make components sized to suit the means of handling	·	•	●	●
15.	Provide a means of handling and locating components during the (dis)assembly procedure	·	·	●	●
16.	Provide realistic tolerances for assembly and disassembly	·	·	●	●
17.	Use a minimum number of connectors	·	•	●	●
18.	Use a minimum number of different types of connections	·	•	●	●
19.	Design joints and components to withstand repeated use	·	·	●	●
20.	Allow for parallel disassembly	•	•	●	•
21.	Provide standard and permanent identification of component type	·	•	●	•
22.	Use a standard structural grid for set outs	·	·	·	●
23.	Use prefabrication and mass production	·	·	●	●
24.	Use lightweight materials and components	●	●	●	●
25.	Permanently identify points of disassembly	·	•	●	●
26.	Provide spare parts and on-site storage for them and parts during disassembly	·	·	·	●
27.	Retain all information of the building components and materials, construction systems, and (dis)assembly procedures	·	·	•	●

Legend of level of relevance: ● Highly relevant • Relevant · Not normally relevant

Another required cultural change that was identified by Kibert et al. [26] deals with the mentality of the public to accept second hand materials. Such a change could be achieved by realising buildings reusing second hand materials and components to show that there is no, or minimal, difference compared to a building constructed with new, first-use, materials [27]. Conclusively, it can be stated that the key design principles generally fit into two categories: 'design' or 'policy' [26].

Another important issue that is not directly addressed by the list of key principles is 'time to deconstruct', which was said to be one of the main obstacles for deconstruction by Guy and Ciarimboli [24]. Although this issue is not directly mentioned under one of the 27 key principles, it can be argued however that many of the 27 key principles indirectly address this issue, since they are all aimed at making deconstruction as easy as possible. Besides, it is suggested by different authors [24, 25, 27] that a deconstruction plan should be drawn up in the design stage. Such a deconstruction plan should contain information on how to deconstruct the building, and for example should deal with matters on how to assure stability during deconstruction. If such a deconstruction plan is made, it can be argued that it is covered by key principle 27 (see Table 2.3). Here, politics could again take the lead by creating policies that make a deconstruction plan compulsory in order to receive a building permit.

Besides, there is the issue of building codes that are not clear on how to deal with the reuse of materials and components in new building designs, as was identified by Rios et al. [27]. However, they argued that as DfD will become more common and applied, this will imply that building codes will be updated accordingly. Again, it is argued that policy and politics play an important role in achieving such changes.

A more recent study by Akinade et al. [38] in 2017 aimed at identifying, what they called, "Critical Success Factors" (CSFs), i.e. key principles, for the effective material recovery through DfD. By means of extensive literature review and conducting four focus group discussions, they managed to come up with a list of 43 DfD factors. Complex data analyses revealed five DfD factor groups, namely 'stringent legislation and policy', 'deconstruction design process and competencies', 'design for material recovery', 'design for material reuse', and 'design for building flexibility'. These five groups indeed showed that besides technical factors, non-technical factors such as stringent legislation and policy, and design process and competency for deconstruction, issues that were addressed above, are key in designing for deconstruction [38]. Finally, the list of 43 DfD factors was reduced to 38 as a result of a reliability analysis. These 38 DfD factors, classified in the five factor groups, are shown in Table 2.4. The normative weights indicate the relative importance of each factor group as identified by the data analysis. From this it becomes clear that (stringent legislation and) *policy* are even the most important success factor for DfD, over the technical *design* factors, according to Akinade et al. [38].

In another recent study (2018), Crowther [39] noticed that his key principles had, amongst others, been adopted by Akinade et al. [38] to serve as a foundation for further development of DfD strategies. However, he also noticed that until then no significant guidance on how to apply the principles with a view to the final goal of reuse and recycling had been present. Therefore, he argued that his developed taxonomy, which has been discussed in subsection 2.1.3, could be used to guide the application of DfD principles, as it better informs decision making and it can help to assess conflicting principles for their future environmental benefit.

Furthermore, Crowther [29] emphasised that a list of key principles for DfD can act as guidelines in order to assist both in designing a building to be demountable, and in the assessment of building's disassembly potential. Finally, it is noticed by Crowther [7] that, while designing with such a list of key principles, it is very likely that there will be conflicts between the application of some principles. He argued that in that case, it might be needed to evaluate and compare the impacts of these conflicting principles in the broader picture of sustainable construction, i.e. within the holistic model. Besides, he stated that the ranking of the key principles in Table 2.3 assists designers to assess the principles by their technical benefits, and that this offers a way to determine the most appropriate principle to apply based on the different recycling strategies. Although it is recognised that both lists that are described here are not claimed to be complete, such lists can assist designers in, directly or indirectly, overcoming many of the challenges that were mentioned in subsection 2.1.1 and that are still faced by DfD.

Table 2.4: Critical Success Factors for DfD classified in five factor groups and their relative weight from exploratory factor analysis [38] (adapted)

	Principle per factor group	Norm. weight [%]
	1. Stringent legislation and policy	39.15
1.	Award of more points for building deconstructability in sustainability appraisal	
2.	Government legislation to set target for material recovery and reuse	
3.	Project contractual clauses that will favour building material recovery and reuse	
4.	Legislation to make deconstruction plan compulsory at the planning permission stage	
	2. Deconstruction design process and competencies	18.32
5.	Improved education of professionals on design for building deconstruction	
6.	Effective communication of disassembly needs to other project participants	
7.	Effective pre-design disassembly review meetings	
8.	Design conformance to codes and standards for deconstruction	
9.	Early involvement of demolition and deconstruction professionals during design stage	
10.	Production of a site waste management plan	
11.	The use of BIM to estimate end-of-life property of materials	
12.	Preparation of a deconstruction plan	
13.	The use of BIM to simulate the process and sequence of building disassembly	
14.	Production of COBie[1] to retain information of the building components	
	3. Design for material recovery	15.55
15.	Use bolted joints instead of chemical joints such as gluing and nail joints	
16.	Avoid composite materials during design specification	
17.	Design foundations to be retractable from ground	
18.	Specify building materials and components with long life span	
19.	Specify lightweight materials and components	
20.	Use joints and connectors that can withstand repeated use	
21.	Minimise the number of components and connectors	
22.	Minimise the types of components and connectors	
	4. Design for material reuse	14.01
23.	Knowledge of end-of-life performances of building materials	
24.	Avoid toxic and hazardous materials during design specification	
25.	Making inseparable products from the same material	
26.	Avoid specifying materials with secondary finishes	
27.	Specify materials that can be reused or recycled	
28.	Design for steel construction	
	5. Design for building flexibility	12.97
29.	Use open building system for flexible space management	
30.	Using of interchangeable building components	
31.	Design for modular construction	
32.	Design for preassembled components	
33.	Design for the repetition of similar building components	
34.	Ensure dimensional coordination of building components	
35.	Separate building structure from the cladding	
36.	Standardising building form and layout	
37.	Use standard structural grid	
38.	Structure building components according to their lifespan	

[1] "*Construction Operations Building Information Exchange (COBie) is a non-proprietary data format for the publication of a subset of building information models (BIM) focused on delivering asset data as distinct from geometric information*" (https://www.thenbs.com/knowledge/what-is-cobie)

Conclusion on framework

Finally, Crowther [7] concluded that the framework must be used by designers as a starting point for the development of individual strategies for individual building designs. Furthermore, he recognised that both the environmental and economical benefits of DfD will be greatest for buildings that are owned by the same clients who regularly maintain or upgrade them, and for large number of buildings which are used for similar purposes. These buildings typically would be hospitals, universities and government departments such as the department of defence. In these cases, it would be most likely that the long term benefits of DfD will be appreciated and realised, even if the consequences of this are some short term extra economic costs. In the opposite cases, in which developers and building owners own their buildings for a short period, it is more likely that they are driven by short term economic, instead of environmental, costs and benefits. Anastasiades et al. [1] even suggested that the concept of ownership might need to be reconsidered, and that manufacturers might need to become the owners of a product (or building) and offer it as a service to the end user. This idea is elaborated upon in subsection 2.2.1.

Besides, it is argued by Crowther [7] that the technological principles that might be applied in order to design for deconstruction are not complex or impossible to realise in the construction industry, and that they are compatible with general good design principles and with other attempts to create a sustainable construction industry. Finally, he suggested that designers should learn by doing, i.e. simply start applying DfD in their designs.

2.2. Circular (Concrete) Bridge Construction

In this section, first of all an action plan to achieve circular bridge construction is discussed. Besides, the lessons that were learned during the development and testing of a prototype circular viaduct in the Netherlands are highlighted.

2.2.1. Action Plan to Achieve Circular Bridge Construction

In section 2.1, the application of DfD in the construction industry has merely been focused on buildings, as most (theoretical) research and literature also seems to be addressing the topic mainly in this area. However, the framework, and the key principles within it, can just as much be applied to infrastructure projects, or more specifically to the area of (concrete) bridge construction. Anastasiades et al. [1] recognised this opportunity after finding that circularity in the construction industry is extensively being investigated, but that nothing related to bridge construction had been researched so far. Therefore, they addressed the topic of the application of the circular economy in the construction industry with a special focus on bridge construction. The main reason for them was similar to the reasons mentioned in section 1.2. They also recognised the fact that a lot of (concrete) bridges are heading for their end-of-life phase and need to be replaced, and that the current practices of demolition and rebuilding involves a lot of material and energy loss. Therefore, they performed an investigation of the available circular construction principles in order to determine certain limitations and gaps for future research and to stipulate the path to be followed towards circular bridge construction, with the same goal as this research' objective of preventing a similar cycle of demolition and replacement to occur again in a few decades.

In order to come to a detailed action plan to achieve circular bridge construction, consecutively the technical solutions, the user behaviour and ownership, and the topic of circularity assessment are discussed. However, first of all it is important to explain that Anastasiades et al. [1] considered three levels of a circular construction industry (see definition of 'circular economy' on page 2). They stated that the micro-scale is the scale of materials and components by themselves, whereas the meso-scale is represented by an entire building or construction, which is an assembly of these materials and components. The highest level is the macro-scale, which is represented by eco-cities, i.e. fully self-sustaining carbon neutral cities, and in the end represents again the circular economy in itself.

Technical solutions

First of all, Anastasiades et al. [1] pointed out that until now a lot of research has been done in the field of circular construction primarily focusing on recycling. However, it turns out that the recycling of concrete (micro-scale) is not a problem with a straightforward solution and that in some cases it may not even be a sustainable solution because of the high energy demand for crushing and transporting for example. The reuse of concrete could be considered as well, however then this has to be taken into account in the earliest stages of design by means of designing mechanical joints. This is because the demounting and reuse of concrete components from currently existing buildings turns out to be practically impossible since often components are connected by means of plastic joints, as was pointed out by several other authors as well [25, 26]. Besides, Kibert et al. [26] emphasised that before concrete components retrieved from existing buildings can be reused, they have to be tested which is a rather cumbersome and time-consuming job. That is why the application of DfD is suggested by Anastasiades et al. [1]. However, it is emphasised that in order to assure that concrete components can be successfully reused, a complete knowledge of the structure over its complete lifespan is necessary, including all changes that it has undergone which may have affected the components. The suggested tool to achieve this knowledge transfer is by means of a materials passport, containing all relevant data and which is shared on a centralised platform, accessible by all stakeholders.

Related to bridge construction, Anastasiades et al. [1] argued that DfD, together with an adapted version of Brand's shearing layers (see page 18 and onwards) specific for bridges, will be a relevant design approach for bridge construction since the primary goal of a bridge is to provide a connection between two points. This primary goal will only very rarely change over its lifetime, and therefore usually a change in the layout of the bridge itself is not required in contrast to the layout of a building for example. However, one relevant argument to include the option of being able to change the layout of a bridge is the ever increasing traffic intensity, which is why the design approach "Design for Adaptability and Deconstruction" might be worth considering as suggested by Anastasiades et al. [1]. This design approach additionally allows a building or structure to adapt its layout during its lifetime, or in other words, to be modular.

Finally, three main technical action points to achieve circular bridge construction were formulated, namely that "*firstly, it will be important to consider and redefine Brand's shearing layers in order to apply them on bridges. Subsequently, specific circular design strategies for bridges should be developed, which can be based on DfD […] and DfAD but adjusted in order to meet the specific needs and requirements of bridges. Additionally, a certain degree of standardisation will be necessary to truly effectuate these design strategies.*" [1]. In addition to the third action point, it is stated that a standardisation scheme on the meso-scale has to be developed which sets certain boundaries without completely eliminating the architectural freedom.

User behaviour and ownership

Furthermore, it is argued by Anastasiades et al. [1] that the concept of ownership might need to be revised in order to increase the chances of the successful reuse of building components and materials. If the responsibility lies with the users to separate and return building components and materials at the end of their service life in order to be reused, it is very likely that this will not happen since users are mostly unaware of the environmental burden of their consumption behaviour. Therefore, it is suggested that the contractor or project developer should be the owner of the building and provide it as a service to the client or user. In that case, it becomes the responsibility of the owner to demount the building when it has reached the end of its service life and to make direct reuse possible. There are examples of several cases in which this turned out to be successful.

When considering bridges, it is pointed out by Anastasiades et al. [1] that this problem will usually not occur, since usually (local) governments are already the owners of the bridge and thus in fact it is already offered as a service to the users. However, this does imply that these governments have to set the example and take their responsibility by specifically requesting bridges that incorporate the DfD and/or DfAD principles, as well as construction and demolition waste best management practices.

Circularity assessment

In subsection 2.1.3 (specifically on page 18) it was already mentioned that according to Anastasiades et al. [1] circularity indicators should be developed in order to be able to effectively assess the eventual circularity on the meso-scale of construction, i.e. the circularity of buildings and constructions in general. It is predicted by Anastasiades et al. [1] that such circularity indicators will be the same or similar for bridges as for buildings. However, it is also recognised that bridges are very different types of structures compared to buildings, and therefore will most likely need their own set of parameters and benchmarks (reference values) that indicate when a bridge design can be considered to be sustainable. Besides, it is expected that the also earlier mentioned 4 R's (Reduce, Reuse, Recycle, Recover) will be a good starting point in the development of these meso-scale circularity indicators. It seems that a recently finished study by Coenen, Tom B.J. [40] in 2019, in which a framework to assess the circularity of bridges and viaducts has been developed, includes a first set of such indicators.

In Table 2.5 the technical solutions, the required changes in user behaviour and ownership, and the requirements for the assessment of circularity that are needed according to Anastasiades et al. [1] to achieve circular bridge construction are summarised as a set of action points.

2.2.2. Realised Prototype Circular Viaduct in the Netherlands

Dutch contractor Van Hattum en Blankevoort, precast concrete specialist Consolis Spanbeton, and Rijkswaterstaat collaborated in order to design and build the first circular, in fact modular, viaduct in the Netherlands. The prototype has been tested for a period of 9 months between December 2018 and September 2019 in the vicinity of Kampen, where it was being used by construction traffic for the Reeve sluice (Dutch: '*Reevesluis*'). The viaduct is circular (modular) since the elements it consists of can be completely reused on a different location and in a different composition. Close to zero waste is produced, almost no raw materials are needed and the parts are being reused to their highest extent. The prototype was a prove of concept and the viaduct thus contributed to the sustainability goals of the

Table 2.5: Summary of the action plan to achieve circular bridge construction [1] (adapted)

Aspect	Action point
Technical solutions	• Redefine Brand's shearing layers of longevity for bridges • Adjust the DfD and DfAD principles to the specific needs and requirements of bridges • Develop a complimentary standardisation scheme without compromising on architectural freedom
User behaviour and ownership	• Local governments as bridge owners need to be incentivised to implement DfD and DfAD strategies as well as CDW best management practices
Circularity assessment	• Bridges are situated on the meso-scale, so meso-scale circularity indicators need to be developed • Dedicated sets of parameters and benchmarks for bridges are required

Netherlands [41]. For a detailed overview of the project and the lessons learned from it, the reader is referred to the project's web page[1], where all relevant documents have been made publicly accessible with the idea to promote and stimulate circular development.

The concept

The key principles of the circular viaduct are:

- The choice for two modular elements with standard dimensions.
- The choice for unbonded prestressing in order to be able to remove the prestressing tendons from the concrete elements and to subsequently be able to reuse the tendons and elements again.
- The choice for so called "shear keys" (see Figure 2.5a) instead of large headjoints (Dutch: '*verbindingskopstukken*'). These are most prominent in the direction of the beams, but also in transverse direction smaller shear keys have been applied.

The prototype at the Reeve sluice consisted of five beams that were placed next to each other. In transverse direction, steel bars were inserted through the five beams in order to connect them together as a whole (see Figure 2.5b). Cementitious mortar was poured between the beams in order to create one solid bridge deck. The beams consisted each of eight modular elements; six hollow ones and two solid headers at the beam ends resting on the abutments (see Figure 2.5c). The total viaduct thus consisted of 40 modular concrete elements [42].

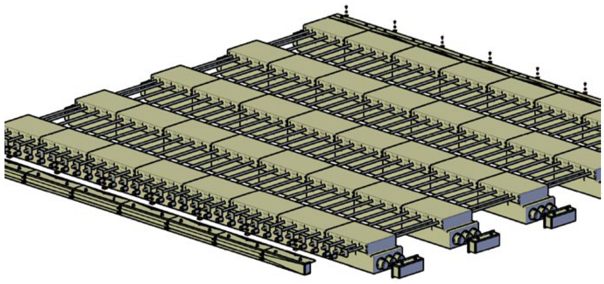
Structural design

The structural design of the circular viaduct was an intensive and challenging process for a couple of reasons. First of all, since it comprised an innovative design. The techniques used (e.g. shear keys, cold connections, external unbonded prestressing) have all more or less been applied before, however, the combination of these techniques makes it innovative [41]. Usually, the prestressing tendons are cast into the concrete after they have been tensioned. This, however, is not a circular solution, since this makes that the structure can not be demounted and therefore not be reused to its highest extent. For the circular viaduct, it was therefore decided to make recesses in the concrete elements, through which the prestressing tendons can pass afterwards (see Figure 2.5). When the elements are attached to each other and the joints are filled with grout, the tendons are tensioned. The application of unbonded prestressing in a viaduct was a deviation from the ROK[2]. This meant that extensive structural calculations had to be made to demonstrate that it is safe. The calculations for the established conditions of the viaduct have been made by engineers from both Van Hattum en Blankevoort and Consolis Spanbeton, which were subsequently approved by engineers from Rijkswaterstaat. It is important to mention that this deviation from the ROK is only approved as constructionally safe in this specific situation and can therefore not simply be applied in every viaduct from now on [42].

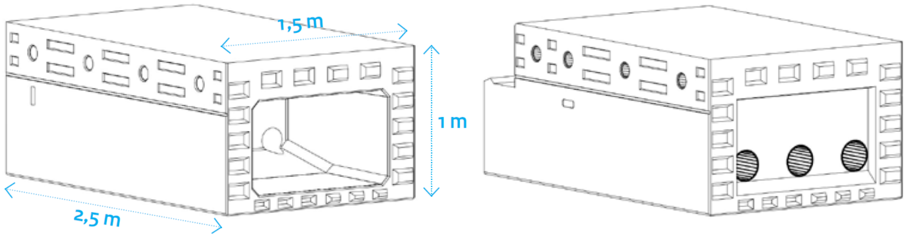
[1] https://www.rijkswaterstaat.nl/zakelijk/innovatie-en-duurzame-leefomgeving/duurzame-leefomgeving/circulaire-economie/bouw-circulair-viaduct-bij-kampen/index.aspx

[2] ROK stands for Guidelines for the Design of Civil Works (Dutch: 'Richtlijnen Ontwerp Kunstwerken'; see subsection 2.5.2)

(a) Elements in factory [43] (adapted)



(b) 3D model of prototype viaduct composition [42]



(c) 3D drawings of hollow and solid elements [42]

Figure 2.5: Details of the circular viaduct

Besides, circularity of the design was the key criterion for every choice to be made. This meant that standard solutions were not always possible. The application of epoxy between the joints would for example have been a logical solution, but from a circular perspective undesirable since it complicates disassembly and epoxy itself has a high environmental impact [41]. This added the challenge of designing a suitable joint for the circular viaduct. The joint namely had to posses two apparently contradictory characteristics. It had to be strong enough to absorb the forces that were to be placed on the viaduct, but on the other hand, the modular elements that were connected by means of the grouting material had to be demountable. Therefore, it had to be possible to break the joint, but only at moments when it was intended to be broken. Finally, the grouting material had to be so fluid that when poured, it would flow everywhere between the shear keys in order to create a homogeneous connection. In the end, it was decided to use a cementitious mortar. In order to ensure that the mortar fell out during disassembly of the elements, the surfaces of the elements had been lubricated with demoulding oil. The combination of cementitious mortar and the lubrication of the surfaces with demoulding oil was tested on an 1-to-1 scale model, which showed satisfactory results. Ultimately, these calculations were also approved [42].

Finally, the elements, that were based on the LEGOs concept, had to be applicable to function in different spans, which meant that multiple designs had to be considered to identify the critical configuration [41]. The length of the elements resulted from the fact that a length of 2,5 m was transportable in width direction and that decks varying in length from 15 to 25 m could be made in multiples of 2,5 m. The width was originally chose to be 1,25 m. Not too wide from a total weight perspective, and not too narrow because of the hollow element in combination with the webs. In the end, the width was adjusted to 1,5 m since the width of the webs was used more efficiently that way, and it made the shear keys easier to fit in terms of geometry. The construction height of 1m followed from calculations for the longest span. The location of the centre of gravity of the prestressing played an important role in this decision (see Figure 2.5c) [42].

Furthermore, it is worth mentioning that a detailed assembly and deconstruction plan was drafted as well [41].

Monitoring

The prototype was extensively monitored during the use phase. Two methods were used for this. External sensors were applied using a frame under the viaduct, and besides sensors were poured into the concrete. The following characteristics were measured: prestressing, deflection, mutual deformation of the elements (joints), temperature and type of vehicle. During the period of use, also a calibration vehicle drove over the viaduct, from which the exact weights were known. The extensive monitoring results showed that the viaduct reacted as expected. No particularities were found in the results, however, a number of aspects needed to be investigated longer or more closely in order to draw conclusions. Also, when the calibration vehicle was passing, the deflection of the viaduct was well within what was calculated in advance [41].

Besides monitoring and collecting data regarding the structural behaviour of the viaduct, it was deemed necessary to record the information required for high quality reuse of the viaduct, possibly even by other parties. That way, it should be prevented that parts or materials over time turn into waste because their ‘identity’ and properties are unknown [41]. Both aspects were translated into a ‘monitoring plan’ during preparation of construction. This plan described in detail which characteristics should be measured from a structural and from a reassembly perspective. In a summarised form, the data demand as laid down in the monitoring plan comes down to the topics in Table 2.6. Special features with respect to regular data demand are the disassembly and reassembly instructions, the production data and plan, and the management plans for both the assembled and disassembled stages [41].

Table 2.6: Data demand as laid down in monitoring plan [41]

What to monitor/collect?	**Why and what?**
• Condition of parts (damage, deformations)	• Elements fit again?
• Management/Maintenance condition • Management instructions (assembled and disassembled stage)	• Reuse of elements →(maintenance) history of elements
• Location in the deck (where are you monitoring?) • Loading of bridge deck/elements	• Reuse of elements →(loading) history of elements
• Production data and plan	• Creation of additional elements
• Assembly, disassembly and reassembly instructions	• How to reuse elements? • Required additional skills (construction, maintenance)
• Applied (raw) materials per part	• Recycling of (raw) materials

Circularity and environmental impact

An independent LCA has been carried out by NIBE[3] to determine both the circularity (as far as it can be assessed) and the environmental impact of the prototype circular viaduct. In the analyses, a 22,5 m by 7,5 m composition of the circular viaduct (the bridge deck) has been compared with a reference design (bridge deck made of box beams from Consolis Spanbeton, SKK700). In addition, NIBE also determined the circularity of both designs according to the method as described by the CB’23 Action Team Measurement (Dutch: ‘*CB’23 Actieteam Meten*’). Besides, Coenen, Tom B.J. [40] has tested his assessment method, which specifically focuses on the circularity of bridges and viaducts, on the prototype circular viaduct [41].

It turned out that the circular viaduct was initially heavier and more impactful than the reference design. LCA showed that it takes a while until this is ‘regained’ through reuse. The total environmental impact of the circular bridge deck over the entire 200 years, provided that it lasts that long with little failure, is however always lower than the reference design in various scenarios. If a regular bridge deck would be demolished and rebuilt earlier than after 61 years, the circular bridge deck would have been less environmentally impactful in every scenario. Besides, the circular viaduct also scored better in various assessment methods for circularity (analysis in accordance with CB’23) and considerably better (bridge circularity indicator [40]) than the reference design. According to these methods, the viaduct is indeed more circular than regular viaducts.

[3]NIBE is the Dutch Institute for Building Biology and Ecology (Dutch: ‘*Nederlands Instituut voor Bouwbiologie en Ecologie*’) and does research, advises and designs on the areas of environment, health and building/managing (`https://www.nibe.org/en/about-nibe/our-mission`).

Conclusions and recommendations

It was concluded that many elements of the prototype can certainly be used in other circular applications in the future, such as the shear keys, applying unbonded prestressing, and developing a demountable bridge deck with a certain degree of standardisation and modularity. However, the flexibility of this specific concept, allowing the same viaduct to be used for different spans, also causes the greatest limitation. Because of the chosen design, the application as a bicycle bridge of 15 m is namely just as robust as a bridge of 25 m for the heaviest traffic class. As a result, the circular viaduct will most likely, because of practical, aesthetic, and above all financial considerations, not be used for a large part of the bridges for which it can be used in the existing infrastructure. Further development of the underlying concept of the prototype is more obvious, for example by developing a simpler concept with less flexibility resulting in fewer joints and a lighter design and/or a few different sizes. In order to be able to compete on a broader scale with existing solutions, the design also has to become lighter and cheaper [41].

From a monitoring perspective, both substantive and process related insights were gained. The substantive insights concerned the fact that circular concepts require management plans for both assembled (here: as a whole viaduct) and disassembled condition (here: as separate elements, see Figure 2.6), as well as a standard or directly applicable method to create a materials passport for an infrastructural civil work. Considering the process related insights, the main lesson learned was that it is important to determine what you want to *know*, instead of what you want to *measure*. Involving an expert on monitoring at an early stage can assist in these decisions [41].

Finally, the studies on circularity and environmental impact revealed a lot about current assessment methods. NIBE identified two notable points of attention, namely that the effect of value retention as a result of the modular design currently is hardly included in any assessment method, certainly not in LCA based methods (like already was concluded in subsection 2.1.3). Secondly, it was found that the difference between high-quality and low-quality reuse is not yet being appreciated. However, the CB'23 Measurement Action Team have initiated research into this in 2019 [41].

(a) (b)

Figure 2.6: (a) Removing and (b) disassembly of beams [41, 44]

2.3. Concrete DfD Connection Methods

In this section, the research that has been conducted into developing connection methods to create demountable joints in concrete structures is discussed. Generally speaking, it is found that applying the principle of DfD in concrete structures is more complicated than for example in steel structures. This is mainly because of the fact that concrete structures usually require monolithic joints between structural components. Therefore, usually cast in-situ concrete is applied in areas where different structural components meet in order to connect them rigidly. As a result, however, this makes it practically impossible to disassemble components from a structure. In an attempt to overcome this problem, in the last two decades several researchers have conducted investigations and experiments on concrete connection methods which possess a certain degree of demountability. From a DfD perspective regarding concrete, such methods should be dry connections without or with very little use of cast in-situ concrete in order to be regarded as suitable for DfD purposes [45, 46].

Xiao et al. [45] and Ding et al. [46] identified three connection methods that had previously been investigated, which are dowel connections, prestressed precast connections, and hybrid-steel connections. Based on the conclusion of the studies into these three types of connections, they decided to investigate a moment-resisting beam-to-beam connection. These four connection methods are discussed below.

2.3.1. Pinned Dowel Connection

Zoubek et al. [47] investigated the cyclic failure mechanisms of concrete beam-to-column dowel connections, which are the most common connection method applied in European precast industrial buildings. Based on experimental results of monotonic and cyclic tests on realistic connections, they were able to develop a numerical model which was able to describe the inelastic cyclic behaviour of dowel connections on global and local level. Besides, the model turned out to be able to correctly predict the failure mechanism as well as the most important characteristics of the monotonic and cyclic response on component level. The test set-up is shown Figure 2.7. The failure mechanism that was observed is schematically shown in Figure 2.8.

Psycharis and Mouzakis [48] performed a similar experiment, however instead of focusing on the failure mechanisms, they were focused on determining the shear resistance of pinned dowel connections under monotonic and cyclic loading. A similar test set-up was used as Zoubek et al. [47], as is shown in Figure 2.9. The main conclusions of both researches were:

1. Standard theory assuming that the failure mechanism is initiated by flexural yielding of the dowel and crushing of the surrounding concrete has been confirmed [47].
2. The cross section of the dowels is the main parameter that determines the resistance of the joint [48]. Besides, the strength of the connection considerably depends on the depth of the plastic hinge in the dowel [47].
3. The resistance of the connection for cyclic response is less than one half the monotonic one [48]. This strength reduction is due to the smaller depth of the plastic hinge [47].
4. Neoprene bearing pad can considerably increase the strength of the connection, particularly when large relative displacements between the beam and the column are developed [47]. Also the use of high strength grout increases the resistance of the connection and improves the cyclic response by decreasing pinching and increasing ductility [48].
5. The thickness of the concrete cover on the dowels in the direction of the loading plays an important role to the response [48].
6. In the case of large rotations between the beam and the column, cyclic resistance is reduced by 15–20%, because the dowel is loaded not only in flexure but also in tension [47].
7. Failure of the dowels does not necessarily imply loss of resistance, because broken dowels usually protrude inside the opposite element and resist the horizontal movement [48].

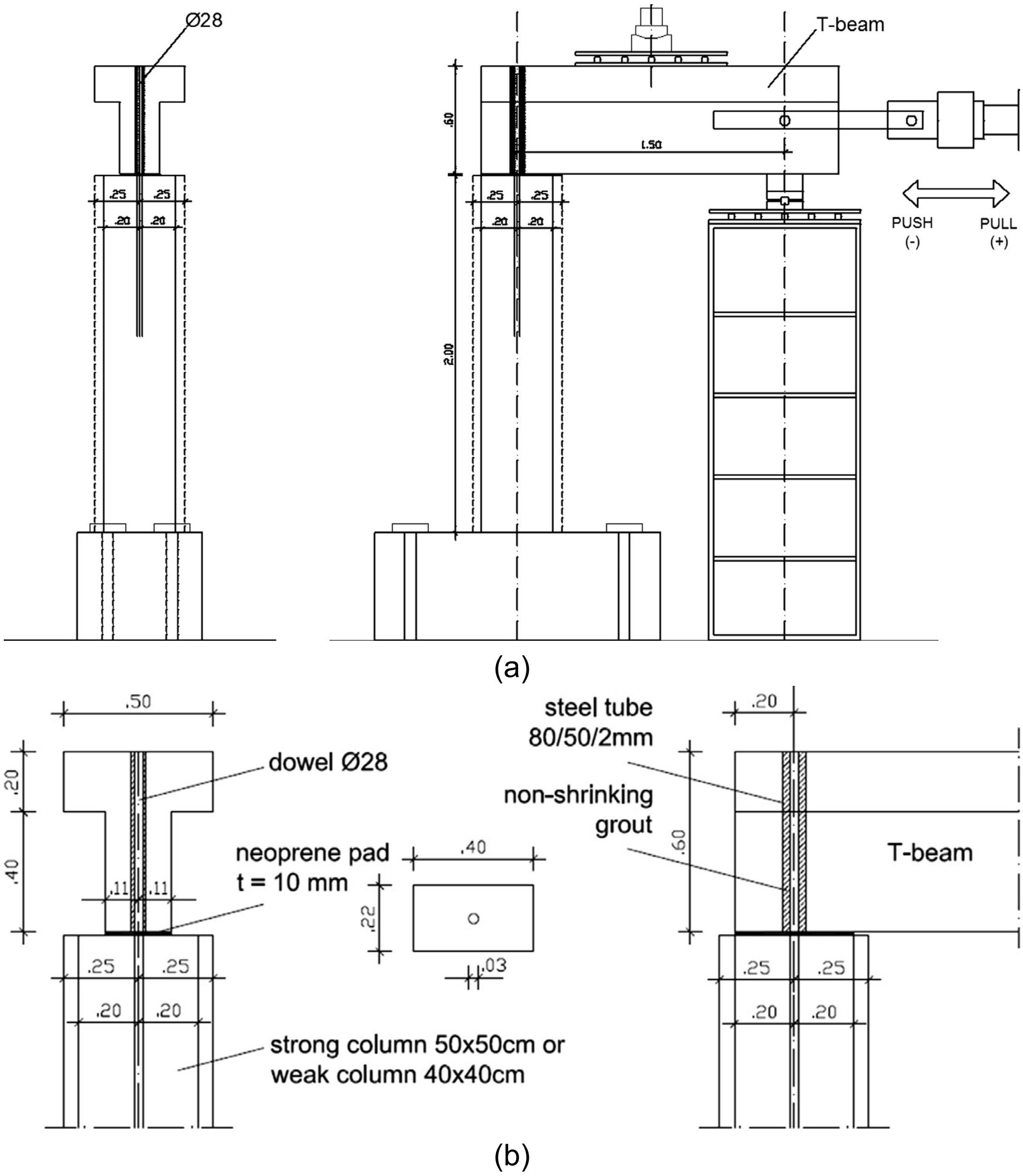


Figure 2.7: (a) Experimental set-up pinned dowel connection and (b) detail of beam-column connection [47]

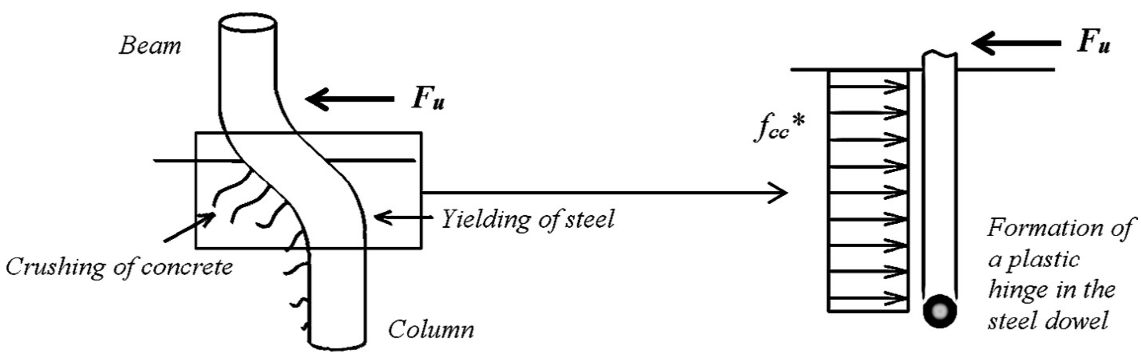


Figure 2.8: Failure mechanism of dowel connection [47]

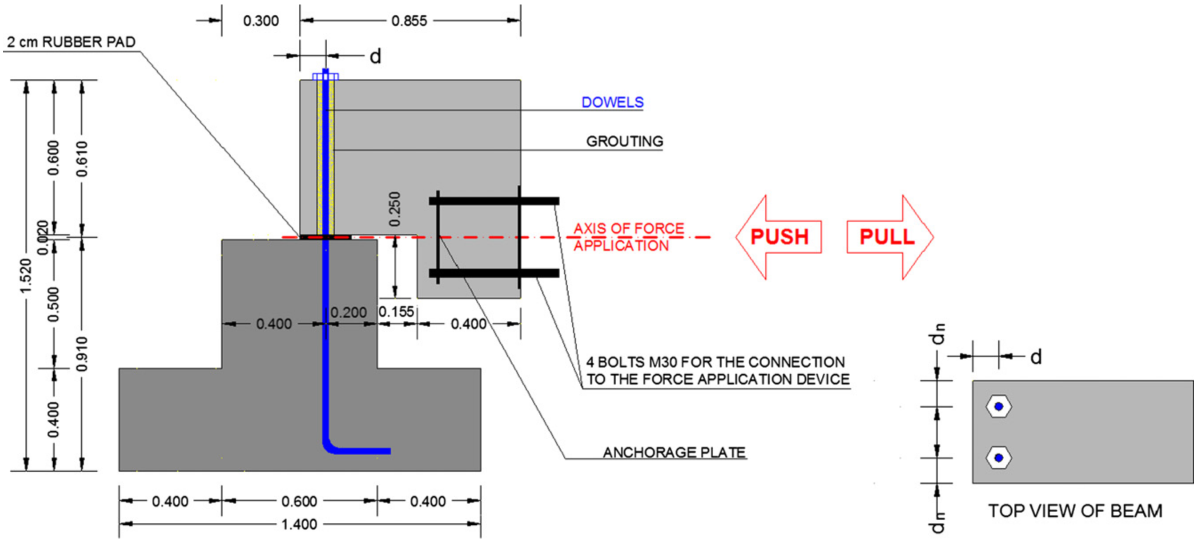


Figure 2.9: Experimental set-up pinned dowel connection [48]

In general, it was concluded by Xiao et al. [45] that the pinned dowel connections can be used in joints that do not have to transfer (large) bending moments, for example in low-rise buildings that are not being designed to withstand earthquakes.

Modelling of dowel action embedded in concrete

In an attempt to derive equations for the bearing strength and subgrade stiffness (foundation modulus) of concrete under the action of a steel dowel, Soroushian et al. [49] performed a series of tests in which the effect of concrete strength, bar diameter, and location of the bar was investigated. Based on the test results, empirical expressions were derived for the bearing strength (f_b) and foundation modulus (k_c) of concrete, of which the latter is of most interest:

$$k_c = \frac{127 c_1 \sqrt{f_c}}{d_b^{2/3}} \qquad [\text{N/mm}^3] = \left[\frac{\text{N/mm}^2}{\text{mm}}\right] \tag{2.1}$$

This expression for the foundation modulus for the concrete surrounding a dowel is of most interest, since this parameter, which governs the dowel stiffness, is of considerable complexity and importance for modelling the dowel action. Besides, it is the only known empirically derived expression in current literature [50].

The behaviour of a dowel embedded in concrete can be analysed by modelling it as a beam on an elastic foundation [50, 51] to deal with the interaction between the dowel and the surrounding concrete. According to the beam on elastic foundation theory, the foundation modulus (k_d) may be treated as a bed of Winkler springs. The relation between the the foundation moduli used in a 2D model (k_c) and in a 1D (beam on elastic foundation) model (k_d) is simply obtained by means of the dowel diameter, i.e.:

$$k_d = d_b \cdot k_c \qquad [\text{N/mm}^2] = \left[\frac{\text{N/mm}}{\text{mm}}\right] \tag{2.2}$$

For relatively small dowel deformation, and provided that none of the materials have yielded, the dowel force-deformation relation is linearly elastic, and can therefore reasonably be estimated by using the beam on elastic foundation theory. However, when the elastic limit is exceeded, the dowel action becomes plastic, and local crushing of the surrounding concrete and/or yielding of the dowel occurs [50, 51]. At this point, the force-deformation relation calculated with the beam on (linear) elastic foundation theory loses accuracy since at this point the behaviour is highly non-linear and therefore should, for example, be analysed in a finite element program.

2.3.2. Prestressed and Hybrid-Steel Connections

Based on extensive literature review, Xiao et al. [45] concluded that the prestressed and hybrid-steel connections generally showed better performances than dowel connections in terms of bending moment capacity, earthquake resistance and energy dissipation characteristics, which were satisfactory compared to monolithic connections. However, the downside of these connection methods turned out to be that they are complicated to construct and additional equipment and technology is required which limit their application. Therefore, these connection methods are not elaborated further upon here.

Typically tested connection details are shown in Figure 2.10.

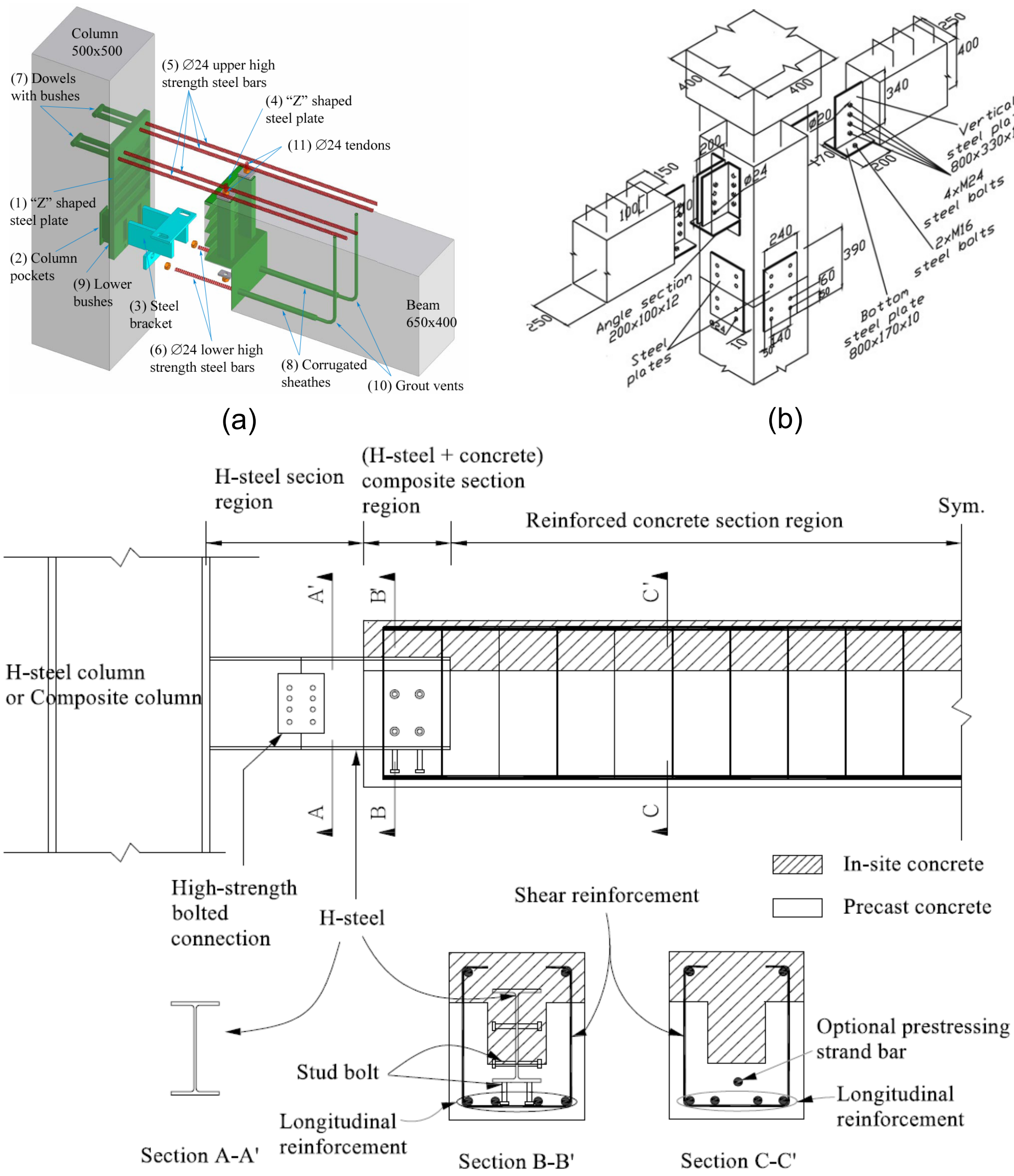


Figure 2.10: Three different types of hybrid-steel connection methods tested [52–54]

2.3.3. Moment Resisting Beam-to-Beam Connection

Based on the conclusions from the researchers that investigated the three types of connections discussed above, Ding et al. [46] argued that beam-to-beam concrete connections seem to be the best way to realise DfD joints in concrete frame structures, since continuity of reinforcement is possible to achieve without disturbing the usually complicated reinforcement layout in the joint core area. In addition, the inherent plastic hinging region can be avoided [45], although it has to be noted that all of these researches were also not directly aiming at developing concrete DfD connection methods. They were mostly focused on investigating to what extent their methods proved to be earthquake resistant [47, 48, 52, 53].

The conclusion of Ding et al. [46] for these beam-to-beam connections was based on research by Korkmaz and Tankut [55] and Khoo et al. [56] who found that these connections can function as feasible replications of cast-in-situ moment-resisting connections in frames since similar crack propagation, failure patterns, and ductile behaviour were observed. Additionally, this type of connection allows the formation of plastic hinges at the beam-end, instead of in the DfD connection region. However, so far only one research was found by Xiao et al. [45] that verified the structural behaviour of the DfD beam-to-beam connection by testing it after deconstruction and reconstruction [57]. Ong et al. [57] proposed a DfD moment-resisting beam-to-beam connection for application in typical multi-storey reinforced concrete apartment blocks. They selected a bolted steel end-plate connection as the basis for their connection method. However, they found that during deconstruction damage to the connection might occur due to mechanical demolition work which results in a lot of noise and debris [45].

Therefore, Xiao et al. [45] decided to design and test five full-scale specimens of a moment-resisting DfD concrete connection by welding the reinforcement at both ends of the concrete beam (see Figure 2.11). The specimens were tested to failure under both static and cyclic flexural loading, and experiments were carried out to determine the structural behaviour and the demountable flexibility of the connection method (see Figure 2.12).

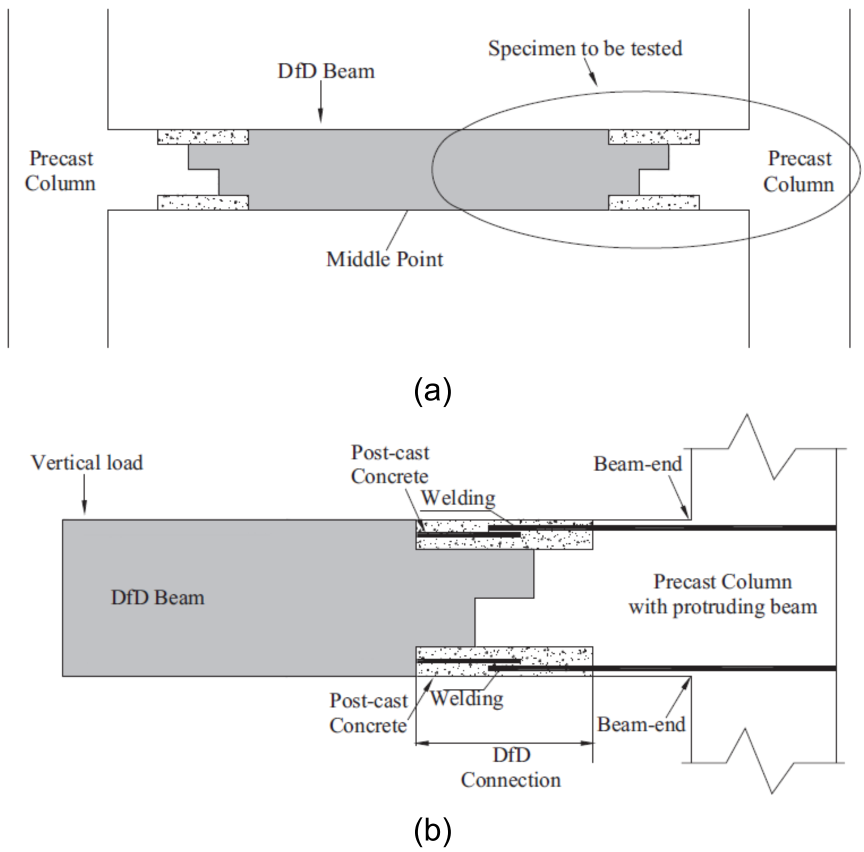


Figure 2.11: (a) Specimen and (b) details of proposed DfD concrete connection tested [45]

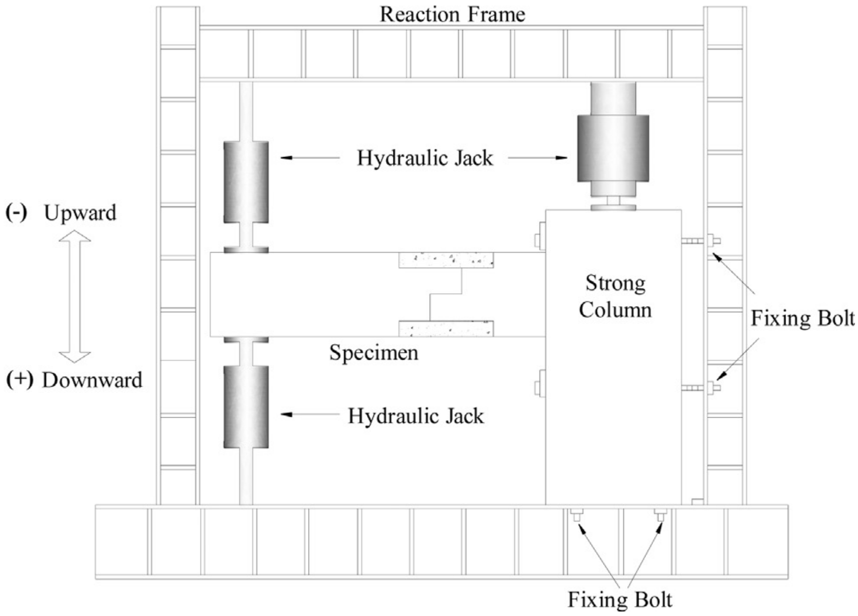


Figure 2.12: Test set-up used [45] (NB: the hydraulic jack at the left bottom corner was only used during cyclic testing)

Ding et al. [46] conducted similar experiments on a slightly different specimen (see Figures 2.13 and 2.14). Besides, they developed a finite element (FE) model, calibrated with the test results, which was suitable to predict the seismic behaviour of the joint with DfD connections. Subsequently, they performed parametric studies with the calibrated FE model to investigate the impact of different joint layouts and characteristics.

The following conclusions could be drawn from both researches:

1. The proposed moment-resisting DfD concrete connection for frame joints is capable of providing adequate moment resistance, which is feasible as a replication of cast-in-situ connection for frame structures in a seismic region [45], as was claimed by Korkmaz and Tankut [55] and Khoo et al. [56] already. Also, no significant difference in crack propagation and failure pattern were observed between monolithic specimens and DfD specimens, and even favourable ductility behaviour was observed [45] in the DfD specimens.

2. The FE model proved to be able to predict the seismic behaviour of the concrete frame joint with DfD connections with acceptable precision. The cracking pattern, lateral load versus drift ratio relationships and steel bar stress behaviour were all reasonably predicted. Besides, the model captured the failure behaviour observed during tests, in which spalling of the concrete cover occurred near the beam-column interface. Based on the overall results, it was finally concluded that the FE model can be adopted as an effective tool to acquire the basic cracking and failure mechanisms of the concrete frame joint with DfD connections, and also to evaluate the parameters controlling its overall seismic behaviour.

3. Furthermore, the FE model showed that the embedded steel section was providing shear resistance for the proposed concrete frame joint with DfD connections while it still behaved elastically even after the maximum lateral loading was reached. Besides, an analysis of the stress behaviour in the concrete column and beam showed that joint with DfD connection provides a favourable integrity for stress transfer between discontinued beam and column [46].

4. The parametric study confirmed the claim of Xiao et al. [45] that the welding of the longitudinal tension reinforcement, providing beam reinforcement continuity, significantly improved the load capacity and ductility of DfD concrete connections. Furthermore, it showed the significant influence of concrete strength on the lateral load bearing capacity. The cast-in-place concrete strength and the thickness of steel section on the other hand showed to have a minor effect on the overall seismic behaviour of the DfD concrete frame joint [46].

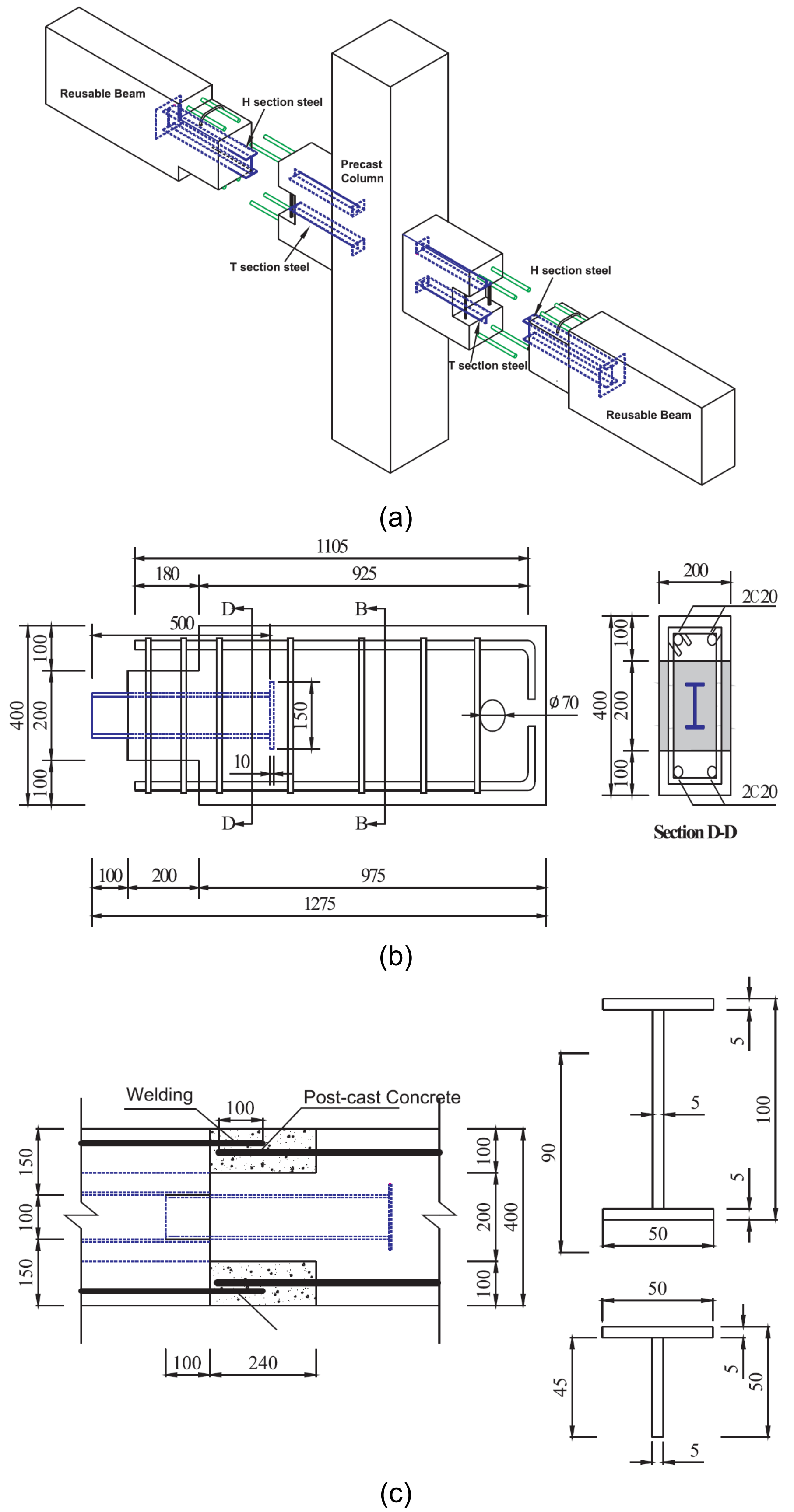


Figure 2.13: (a) Specimen, (b) section details and (c) connection details of proposed DfD concrete connection tested [46]

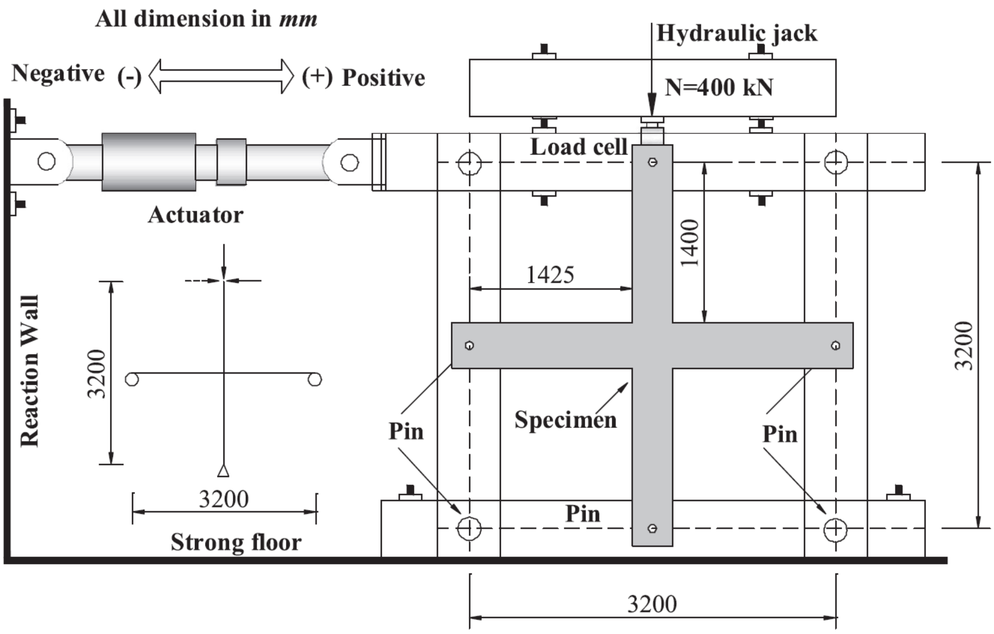


Figure 2.14: Test set-up used [46]

5. With regard to the de- and re-constructability of the connection, it was found that the small amount of post-cast concrete and the welding between the longitudinal reinforcement bars could be easily removed with mechanical tools, and very little debris was generated during the deconstruction stage. Also, little damage occurred on the DfD concrete part. As a result, the disconnected beam could be reused [46].

Furthermore, two recommendations were made with regard to de- and re-construction of the connection:

1. The surface of the concrete component should be as smooth as possible before casting the concrete encasement. Application of suitable polish or thin-film before concrete casting is suggested in order to greatly reduce the difficulty of removing the encasing concrete [45].
2. The length of the bare reinforcement steel should be sufficient in order to be able to create continuity of reinforcement again in a reuse scenario. Therefore, elaborate work should be carefully undertaken during the initial design procedure [45].

2.4. Monitoring of Concrete Structures

In this section, first of all the requirements of a monitoring plan are addressed, which are used in Chapter 6 to develop two draft versions of a monitoring plan. Subsequently, the most common deterioration mechanisms of reinforced concrete structures are shortly addressed, after which the dominating mechanisms are elaborated upon in more detail.

2.4.1. Monitoring Plan

First of all, it is noted that ‘monitoring’ in this research’ context is subdivided into two different stages (see Figure 2.15). On the one hand, it concerns real-time monitoring of a structure during the use phase, whereas on the other hand, it concerns monitoring in terms of, what has been labelled, a ‘reusability assessment’, in which both the condition of, as well as the deterioration and/or damage to, the different elements and components of the structure are evaluated at the end of a life-cycle (i.e. during demounting the structure). The former is concerned with checking the safety and reliability of the structure when it is actually being used, whereas the latter is performed in order to check and decide whether the elements and components can be reused safely in a new life-cycle (i.e. circular construction).

Besides, in order to ensure high-quality reuse of (elements and components of) a structure, it is important to document all relevant information, which helps to avoid that (parts of) elements and components over time turn into waste because their ‘identity’ (properties) and history of usage, inspection, maintenance, and repair are unknown. This information should carefully be documented in what could be called a ‘element/component passport’. Such an element/component passport then both should contain the relevant as-built (i.e. ‘birth certificate’) as well as the ongoing (through-life) updated data, focused on specific details of important durability and service life parameters of the structure [58]. With regards to the circular viaduct, this would imply that the results of the reusability assessment of elements and components should also be included in each element/component passport respectively.

In order to ensure monitoring to be performed appropriately, either real-time monitoring or for the purpose of a reusability assessment, a detailed monitoring plan should be drafted. In such a monitoring plan, the following four main questions should be answered [59]:

1. What are the relevant (parts of) elements and components to monitor?
2. What potential deterioration and/or damage is expected?
3. What physical parameter(s) can reflect each of these types of deterioration/damage?
4. How can these physical parameter(s) be monitored?

The first question is rather straightforward, and simply depends on the responsible party to determine in which (parts of) elements and components he/she is interested. With regards to monitoring of concrete structures, the term ‘damage’ in the second question can be interpreted as referring to the potential physical damage and (electro)chemical deterioration mechanisms that are expected to occur. The main deterioration mechanisms for reinforced concrete structures are therefore discussed in detail in

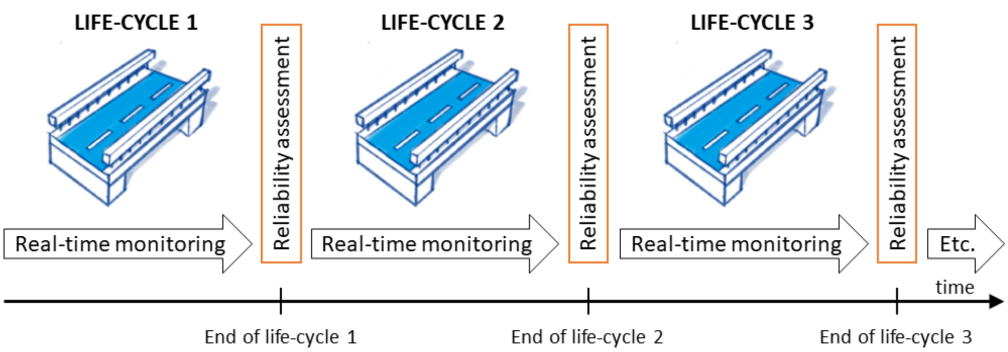


Figure 2.15: Different stages of monitoring of a circular structure

subsection 2.4.2. The third question refers to the measurable and observable progress and changes in materials and/or structural properties with time that are typical signs of certain damage or deterioration mechanisms, which is also dealt with in subsection 2.4.2. Finally, the fourth question concerns the type of systems that are available and applicable to monitor these physical parameter(s), and which are possibly also controlling the functioning of, for example, a warning system based upon these physical parameters (applied load, element deflection, etc.) [58, 59]. These different types of systems are not discussed in further detail, since this is not relevant within the scope of the research, because the focus is on advising *where* to monitor *what*, and not on *how* (i.e. with which systems) to monitor the circular viaduct.

2.4.2. Reinforced Concrete Deterioration Mechanisms

Many mechanisms exist that can potentially lead to damage of reinforced concrete structures, and these mechanisms can be ordered in many different ways. Commonly, a distinction is made between two broad categories, namely deterioration of concrete (e.g. cement matrix) itself, and deterioration (i.e. corrosion) of the steel reinforcement, which are also referred to as direct and indirect deterioration respectively [60, 61]. The most common direct and indirect deterioration mechanisms are shown in Figure 2.16 and are shortly addressed in the following subsections.

Direct deterioration mechanisms

Direct deterioration is defined as the deterioration of cement and aggregate phase of the concrete itself as a result of exposure to harmful substances. Due to these substances, the cement matrix is weakened and followed by abrasion. The most common direct deterioration mechanisms of concrete are [61]:

- Sulphate attack, leading to internal expansion of cement matrix.
- Freeze-thaw action, leading to internal expansion of frozen pore water.
- Chemical deterioration by acids/salts, leading to dissolution of concrete/cement matrix.
- Alkali-Aggregate Reaction (AAR) / Alkali-Silica Reaction (ASR), leading to internal expansion of cement matrix.

Besides, one more mechanism should be added to this list, which is deterioration due to mechanical attack. An example of this is the damage to the concrete cover of a bridge deck caused by traffic [60] or, more generally, damage due to extreme/unforeseen (i.e. not designed for) situations which cause the bridge to exceed its structural capacity.

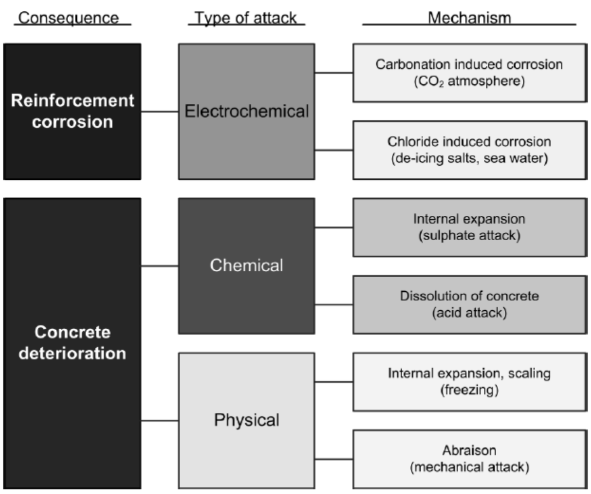
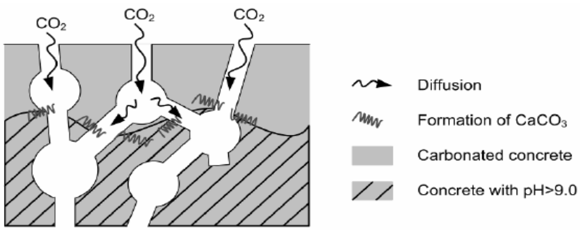
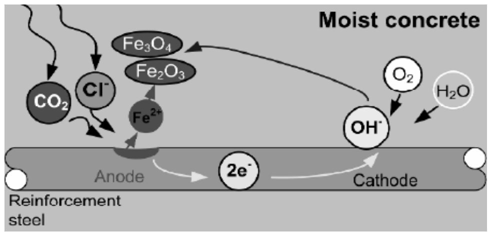


Figure 2.16: Most common direct and indirect deterioration mechanisms of reinforced concrete structures [58]



(a) Carbonation of concrete [58]



(b) Electrochemical process of corrosion [58]

Figure 2.17: Mechanisms of reinforcement corrosion in concrete

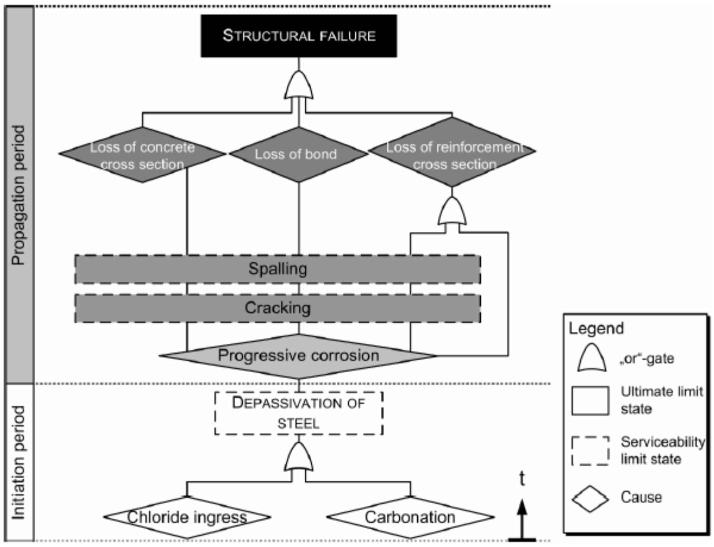


Figure 2.18: Schematic process of deterioration of reinforced concrete structures due to corrosion [58]

Indirect deterioration mechanisms

Indirect deterioration is defined as the appearance of concrete cracking and spalling due to a volume increase as a result of the electrochemical process of corrosion of the steel reinforcement. In this case, the cement matrix itself is not affected by harmful substances that penetrate into the concrete. Instead, it is the passive layer on the surface of the steel reinforcement that is destroyed by either carbonation (CO_2) of concrete or by the ingress of chlorides (Cl^-) into the concrete in combination with the presence of oxygen (O_2) (see Figures 2.16 and 2.17). After depassivation of the reinforcement, active corrosion starts and leads to a decrease of the steel reinforcement bar diameter in the anodic area (see Figure 2.17b). Besides, the volume of the corrosion products is several times higher than the original steel (range from 100% to 300%), and therefore causes expansion induced strains in the concrete which subsequently leads to cracking and spalling of the concrete cover. Finally, corrosion may also lead to degradation of the bond strength between concrete and reinforcement as a result of the loss of concrete cover or degraded ribs of the reinforcement bars [58, 61]. This process, all the way leading up to structural failure, is schematically shown in Figure 2.18.

Dominating deterioration mechanisms

Based on numerous studies that analysed the decay of concrete structures in many different countries across the world, it appears that reinforcement corrosion due to chloride ingress and carbonation are the dominating deterioration mechanisms of general (i.e. not just specifically precast prestressed) concrete bridges [61] (see Figure 2.19), which is also confirmed by Robert E. Melchers and Igor A. Chaves [60]. Both mechanisms are shortly described below.

Chloride-induced corrosion

The ingress of chlorides can lead to chloride-initiated reinforcement corrosion. The chloride ions originate from the salt sodium chloride, which is the main salt in seawater and in de-icing salts. As soon

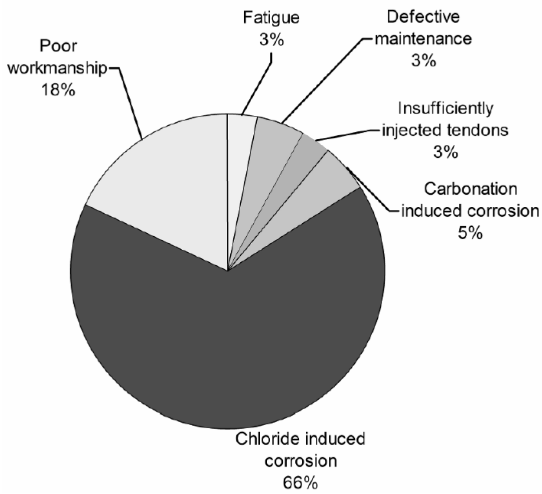


Figure 2.19: Causes of damage to bridge structures of the German motorway network [58]

as the chloride concentration at the reinforcement exceeds the critical chloride concentration, the steel surface depassivates and corrosion of the reinforcement starts. Different factors influence the time until initiation of corrosion, such as for example the concrete quality, the thickness of the concrete cover, and, clearly, the concentration of chlorides to which the concrete is exposed [58, 61].

Carbonation-induced corrosion

Carbonation of concrete takes place when concrete is located in an atmosphere which contains carbon dioxides (CO_2) and has an appropriate relative humidity (highest rate of carbonation for RH between 60% and 80%). During this process, carbon dioxide diffuses through the pore system of the concrete and reacts with hydroxides (OH^-) in the pore solution to finally form calcium carbonate ($CaCO_3$). The reduction of hydroxides results in a pH-value drop below 9,0 (see Figure 2.17a). The depth over which this happens is called the 'carbonation depth'. Once this has happened, the passive layer on the surface of the steel reinforcement is destroyed and corrosion initiates [58, 61].

2.5. Building Codes

In this section, the existing applicable and compulsory building standards and regulations, guidelines, recommendations, documents, and reports for the construction of a concrete viaduct in the Netherlands are introduced and discussed. The collective name “building codes” is introduced to refer to all of those. Firstly, all the building codes that are applicable and/or compulsory for the construction of a concrete viaduct in the Netherlands are listed. Subsequently, each of those are shortly treated separately, and their specific applications are highlighted. Finally, the degree to which these currently existing applicable and/or compulsory building codes address the topic of ‘circularity’ (‘demountability’) is discussed shortly.

The applicable and/or compulsory building codes for the construction of a concrete viaduct in the Netherlands are:

- Eurocodes/NEN-standards
- ROK
- Rijkswaterstaat guidelines
- Other documents

Besides, there is the fib Model Code for Concrete Structures 2010 which is not a legal document but rather serves as a basis for future building codes for concrete structures.

2.5.1. Eurocodes/NEN-standards

With the NEN-EN 1990 to NEN-EN 1999 series of standards and the associated Dutch National Annexes, it can be demonstrated that the structural safety of a construction (buildings, civil works, etc.) meets the requirements of Dutch building regulations. The National Annexes are inextricably linked to these standards; without them the standards cannot be properly used. In the National Annexes, choices have been laid down for options given in the standards and the applicable Dutch values for the nationally determined parameters are given [I].

The most used, applicable and/or compulsory NEN-standards and associated Dutch National Annexes (NA) for the construction of a concrete viaduct in the Netherlands are:

- **NEN-EN 1990 [I] + NA [Ia]:** Eurocode - Basis of structural design
- **NEN-EN 1991-2 [II] + NA [IIa]:** Eurocode 1: Actions on structures - Part 2: Traffic loads on bridges
- **NEN-EN 1992-1-1 [III]+ NA [IIIa]:** Eurocode 2: Design of concrete structures - Part 1-1: General rules and rules for buildings
- **NEN-EN 1992-2 [IV] + NA [IVa]:** Eurocode 2: Design of concrete structures - Concrete bridges - Design and detailing rules
- **NEN-EN 1337 [VI]:** Structural bearings

The NEN-standards and associated Dutch National Annexes fall within the system of the Eurocodes and can be applied for the construction of new structures without further adaptation [I].

2.5.2. ROK

The ROK[4] [XI], including annexes A [XIa] and B [XIb], is a framework within the Rijkswaterstaat working method. It is a collection of generic requirements that the design and execution of a new civil work has to meet. The ROK also applies to new parts of existing civil works, when these parts are replaced, or to extensions when civil works are expanded [XI].

[4]ROK stands for Guidelines for the Design of Civil Works (Dutch: ‘Richtlijnen Ontwerp Kunstwerken’)

The design of civil works is a creative process that should take place in complete freedom. However, the resulting product must be reliable, durable and functional. Requirements and boundary conditions are required to demonstrate this. The vast majority of these requirements are included in the Eurocodes with associated National Annexes. In addition to these, Rijkswaterstaat has a number of specific requirements, because the Eurocode requirements are not strict enough, or because they do not appear in the Eurocodes and National Annexes. Also, sometimes the Eurocodes and National Annexes offer options in which case the ROK provides clarity [XI].

The ROK is about the design of civil works, and not about the design of the dimensions that are required based on functional requirements. Also the design of installations, which have to be added due to the functional requirements, are not the subject of the ROK. In case a design is considered that meets deviating but equivalent requirements, permission has to be obtained from the ROK management committee before the start of the design. The goal of Rijkswaterstaat, as an expert client, is to use the ROK to provide unambiguous guidelines for the design of all its new to be built civil works [XI].

The Eurocodes distinguish three categories [XI]:

- Buildings
- Bridges
- Other structures

The categories ‘Buildings’ and ‘Other structures’ are not considered as ’civil works’ by Rijkswaterstaat and therefore are not included in the ROK. However, many other types of civil works, as defined by Rijkswaterstaat, are not mentioned in the Eurocodes. The ROK has therefore appointed the following 6 categories [XI]:

- Bridges
- Tunnels
- Hydraulic civil works (Dutch: ‘*Natte kunstwerken*’)
- Movable bridges
- Noise barriers
- Traffic engineering support structures (Dutch: ‘*Verkeerskundige draagconstructies*’)

The Eurocode parts with associated National Annexes mentioned in the ROK are applicable to and compulsory for all six categories, including the standards referred to in the Eurocodes and the additions to the Eurocodes mentioned in the ROK. All other standards, guidelines and documents mentioned in the ROK, including the additions to these, are also binding. All documents invoked in or via the aforementioned standards are also binding [XI].

In case data is conflicting, the following order of precedence applies [XI]:

1. Requirements from the contract
2. ROK provisions
3. Rijkswaterstaat guidelines (see subsection 2.5.3)
4. Eurocodes + NAs, NEN-standards, CUR- and CROW-documents (see subsection 2.5.4)

In the event of any discrepancies between binding documents that fall under the same order of precedence, the most recent document prevails over the document of an earlier date [XI].

2.5.3. Rijkswaterstaat guidelines

In addition to the Eurocodes, the ROK also references other guidelines, such as CUR- and CROW-reports and guidelines and other documents (see subsection 2.5.4), and its own Rijkswaterstaat guidelines. The Rijkswaterstaat guidelines have to be applied when designing civil works, but concern matters other than the structural safety and durability of the main supporting structure. The Rijkswaterstaat guidelines can be downloaded online[5]. The guidelines cover a variety of different topics [XI].

2.5.4. Other Documents

Besides the already discussed building codes, there are still a number of other relevant guidelines, recommendations, reports, documents, etc. Among these are the CUR-recommendations, which are publications in which agreements between parties in the construction industry are laid down. Clear communication between client, structural engineer, (sub)contractor, consultancy, supplier, construction supervisor, etc. is of the utmost importance to prevent misunderstandings and errors and to construct a high-quality building. Clear agreements also reduce the risk of cost and time exceedances in construction projects. That is why all parties in the construction industry benefit from CUR-recommendations, which are issued on behalf of CROW [62].

CROW, in turn, is one of the parties that ensures that infrastructure, public space and traffic and transport are well organised in the Netherlands. It is a non-profit organisation in which the government and businesses work together in pursuit of their common interests through the design, construction and management of roads and other traffic and transport facilities. CROW focuses on distributing knowledge products to all target groups by being active in research and in issuing regulations. Their core tasks involve research in the area of traffic, transport and infrastructure, standardisation in this sector, and transfer of knowledge and knowledge management [63].

Finally, there are still a number of other relevant guidelines and documents, and research reports and literature which are listed in sections 2.5 and 2.6 of the ROK [XI].

2.5.5. fib Model Code for Concrete Structures 2010

The aim of the fib Model Code for Concrete Structures 2010 (MC2010) is twofold. Firstly, it is intended to serve as the basis for future codes for concrete structures. Secondly, whereas existing operational codes are legal documents, based on mature knowledge, the MC2010 also takes into account new developments with respect to concrete structures, the structural material concrete, and new ideas with respect to requirements to be formulated, in order to ensure that structures achieve optimum behaviour according to new insights and ideas. Additionally, the MC2010 includes the whole life-cycle of a concrete structure, from design and construction to conservation and dismantlement, in one code for buildings, bridges and other civil engineering structures. Furthermore, besides the traditional topics regarding safety and serviceability, the MC2010 also takes into account the design criteria for durability and sustainability [XII].

Whereas the existing applicable and/or compulsory building codes like the Eurocode predominantly gives sets of application rules that should be transparent enough to be applied by professional designers while also accurate enough to be economical, the MC2010 also aims to give sufficient background information. So, although the MC2010 is not a legally binding building code like the Eurocode, nevertheless it is also meant to be an operational document for every day design situations and structures [XII].

2.5.6. Circularity in Existing Building Codes

A quick scan of the most important existing applicable and/or compulsory building codes (Eurocodes and ROK) results in the conclusion that none of them currently addresses the topic of circularity directly. No mentioning of the concepts of ‘demounting’ or ‘demountable’, ‘modularity’ or ‘modular’, ‘reuse’ or ‘recycling’, etc. were found in any of these building codes.

[5] https://www.rijkswaterstaat.nl/zakelijk/werken-aan-infrastructuur/bouwrichtlijnen-infrastructuur/

The MC2010, however, in chapter 10 discusses the act of dismantling buildings. In this chapter, however, the focus is on the dismantling of existing buildings, and is mostly focused on the processes it involves (e.g. see Figure 2.20) instead of the technical aspects. The latter are only shortly dealt with in a couple of sentences in paragraph 10.2.3 regarding the structural safety, i.e. stability, during dismantling.

Besides, the impracticality of the existing buildings codes, revealed by different authors [26, 27], was addressed already in subsection 2.1.1, and additionally it was mentioned already in subsection 2.1.3 that it is argued by Rios et al. [27], for example, that codes will be updated as soon as DfD (as well as other circular design approaches) will become more common and applied. The MC2010, as it is one of the aims of the code, could potentially inspire the developers of new building codes for concrete structures to include specific guidelines and regulations dealing with this topic.

However, the fact remains that the existing applicable and/or compulsory building codes do not yet address these topics. Therefore, this implies that the development and validation of circular (demountable) solutions in the context of this research will at least partly have to be based on engineering knowledge and skills. Subsequently, it will most likely have to be approved of by (a) different independent institution(s) (e.g. Rijkswaterstaat, similar to approving the unconventional methods used in the circular viaduct; see subsection 2.2.2) before it can be applied in a viaduct.

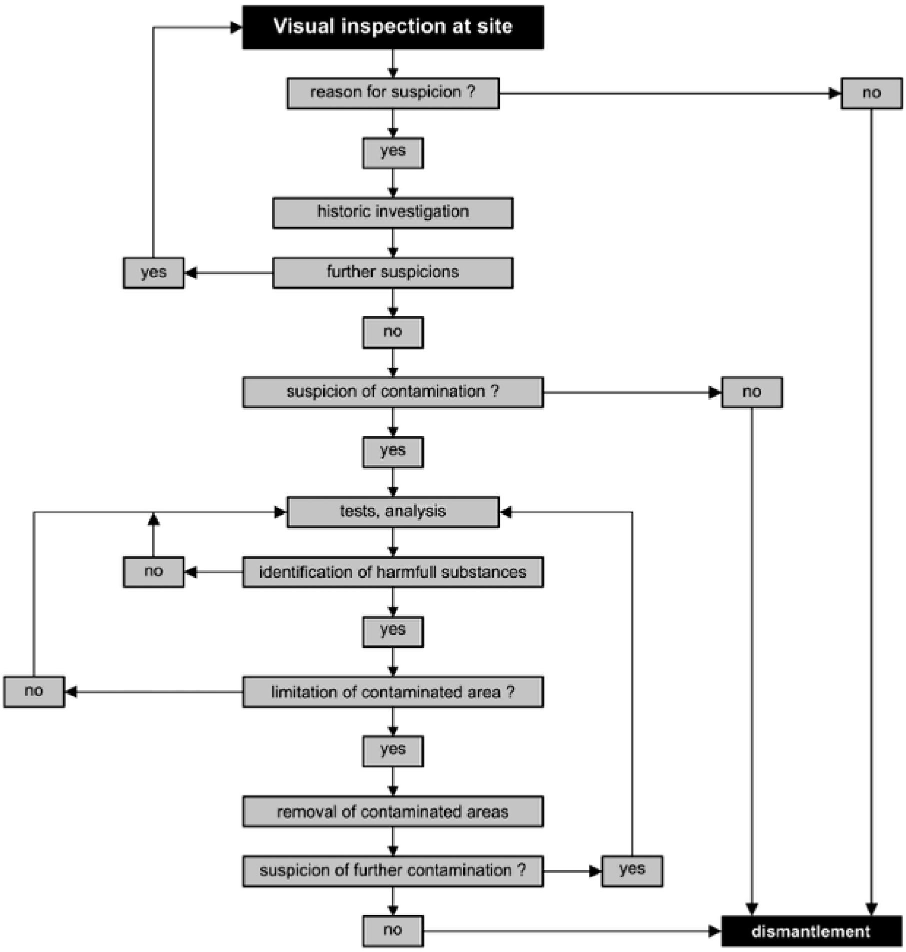


Figure 2.20: Flowchart of the inspection of a building in the preparation stage of dismantlement [XII]

2.6. Analysis of (Existing) Concrete Viaducts in the Netherlands

In this section, different types of (existing) concrete viaducts in the Netherlands and the current commonly used methods of construction are analysed. First, the typical layout and components of a viaduct in the Netherlands are briefly described. Subsequently, a distinction between prefab and cast in-situ viaducts is made. For both types, several (dis)advantages are mentioned, and the commonly used techniques and options for the different components as well as the execution methods are addressed. Next, characteristics and (dis)advantages of different structural systems of concrete bridges are discussed. Subsequently, standard dimensions of concrete viaducts are identified, based on an analysis of data of existing viaducts in the Netherlands. Finally, the current life-cycle aspects of concrete viaducts in the Netherlands are addressed.

2.6.1. Layout of a Viaduct

Most of the existing viaducts in the Netherlands were built after 1960. From this period onwards, the prestressed solid slab has been used for viaducts on a large scale. In the early years, these viaducts often consisted of three spans, with a gradient. From the seventies, a so-called umbrella version of the solid slab viaduct has been widely applied. Intermediate piers were placed in the central reservation of a highway. The slab was given a parabolic shape, with the center point being the highest. This type is particularly suitable for highway intersections with two times two or two times three lanes [64]. Nowadays, it is estimated that 90% of the viaducts in the Netherlands are constructed with prefabricated elements [65].

A viaduct can basically be split up in two subsystems: the superstructure and the substructure. The superstructure mainly consists of the deck. Other parts of the superstructure concern safety and aesthetic provisions like finishing layers, kerbs, safety barriers, parapets, etc. The substructure consists of the components that carry the superstructure and transfer it to the subsoil. These components are:

- Abutment (or bank seat)
- Intermediate pier(s) (if present)
- Capping beam(s) (in case of a prefab deck, see subsection 2.6.2)
- Foundation

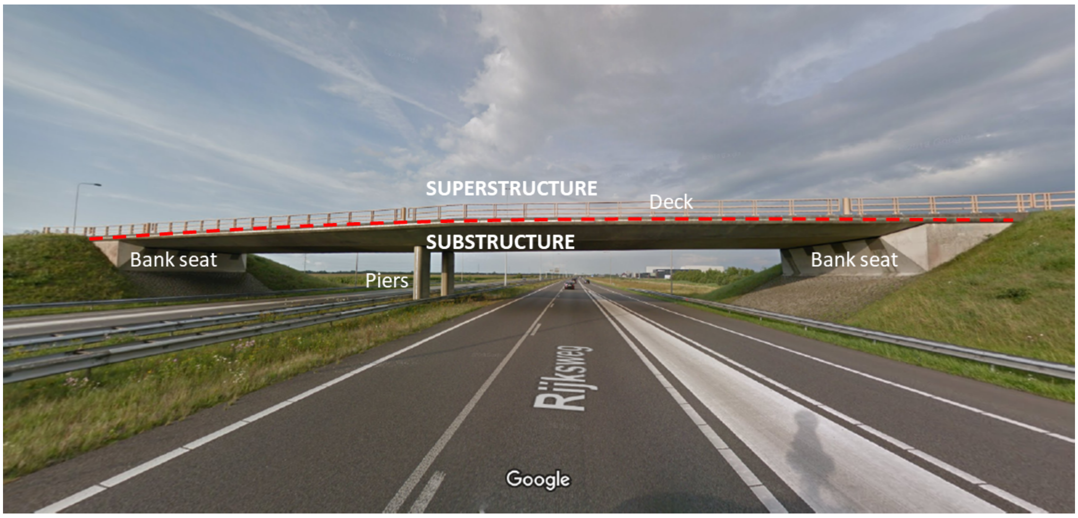
The way in which the super- and substructure are usually connected is similar for both precast and cast in-situ viaducts. The deck and supports are connected by means of bearings and transition joints [66]. However, different connection methods and different solutions in which super- and substructure are (partially) monolithically connected also exist [67].

Furthermore, a viaduct can either be perpendicular to the road/railway that it is crossing, or it can be skew or curved which can complicate the design. Besides, the deck can be horizontal, or a transversal inclination can be realised [68]. Typical cross-sections of a prefab and a cast in-situ viaduct, with transversal inclination, are shown in Figure 2.21. Typical layouts of respectively a two-span and a four-span viaduct over Dutch highways are shown in Figure 2.22.

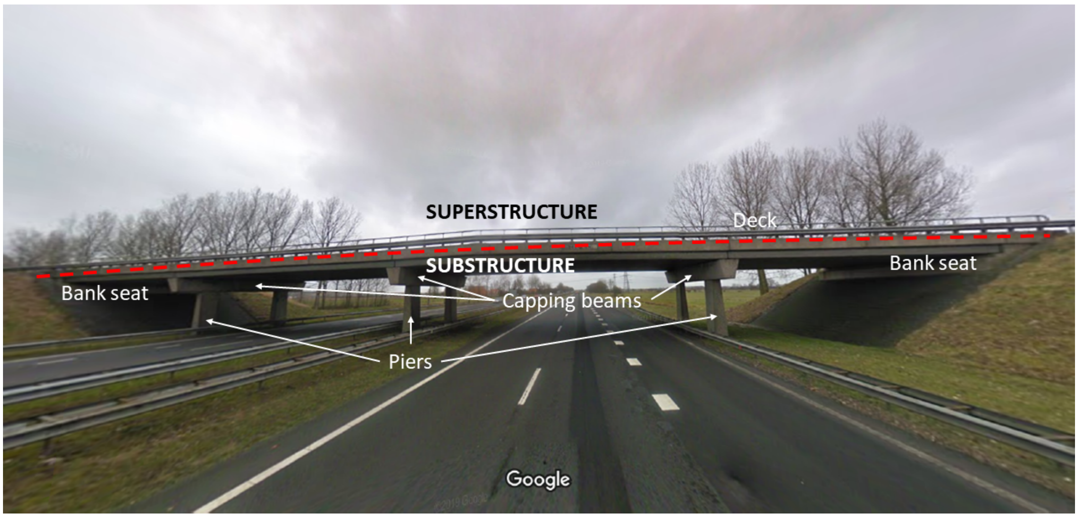
(a) Typical cross-section of prefab viaduct

(b) Typical cross-section of cast in-situ viaduct

Figure 2.21: Typical cross-sections of prefab and cast in-situ viaducts with transversal inclination [65] (NB: Note that the clearance height under both viaducts is equal, which results in a larger total height of the prefab viaduct)



(a) Two-span viaduct over Dutch highway A7 at "Knooppunt Heerenveen", Heerenveen [69]



(b) Four-span viaduct over Dutch highway A7 at Oudehaske [70]

Figure 2.22: Typical layouts of respectively (a) a two-span and (b) a four-span viaduct over Dutch highways

2.6.2. Prefab Viaducts

In a prefab viaduct, it is mainly the deck that is created by means of prefabricated concrete beams, whereas the substructure usually is still cast in-situ. However, it is also possible to construct the substructure with prefab elements [71].

A prefab viaduct has several advantages compared to a cast in-situ viaduct [66]:

- The construction time on-site is shorter.
- Traffic interruption is significantly smaller.
- No major formwork and falsework is needed.
- Higher concrete and product quality can be achieved.
- The construction stage is less dependent on weather conditions.

Disadvantages compared to an in-situ casted viaduct are [66]:

- Each prefab element needs at least one bearing.
- A capping beam is (usually) needed to support the prefab deck elements.
- (Consequently) a higher construction height is required.
- (Consequently) a higher (and longer) approaching ram is required.
- Generally, more transition joints are needed.
- Size and weight of components is limited by factory, transport, and hoisting capacities.
- Prefab viaducts are generally more expensive.

Superstructure

As mentioned before, the main component of the superstructure is the deck. For a prefab viaduct, there are mainly three prefabricated systems used in the Netherlands to construct the deck. These three systems are a solid deck bridge, a girder bridge, and a box beam bridge, which are consecutively discussed below [66].

Solid deck bridges

The solid deck bridge is suitable for short span bridges and consists of prefab elements combined with a cast in-situ infill topping. This type leads to heavy, but easy to erect structures, and therefore it is only valid for short to medium-span bridges [68]. A slenderness ratio of 20-25 is usually obtained [65].

The first type of solution is a massive slab, which can span between 4 to 13 m. Usually the prefab slab has a standard width, for example 1200 mm, and a thickness between 150 and 200 mm. Once the slabs are put in place, a structural topping varying between 150 and 200 mm is cast in-situ. The prefab slabs are usually prestressed and contain protruding rebars that ensure a good connection with the structural topping. Besides, the longitudinal joint interfaces of the slabs are provided with a longitudinal slot to form a shear key. The edge of the bridge is usually cast in-situ [68].

The second type of solution is a deck with prestressed infilled beams. These beams usually are I-shaped or inverted T-profiles, which are placed side by side, and connected by means of a cast in-situ infill topping. Usually the beams have a standard width of 990 mm or 1180 mm for example. The height of the prestressed beams typically ranges between 300 and 800 mm, and the thickness of the infill deck excluding the finishing layer is 120 mm, measured from the top of the beams (see Figure 2.23) [72]. The bottom transversal reinforcement of the cast in-situ infill runs through holes in the webs of the beams, whereas the top transversal reinforcement is placed on top of the beams. Typical span lengths run from 6 to 20 m. The edge of the bridge can either be realised with a prefab element or can be cast in-situ [68]. This system is suitable for both statically determined and statically indeterminate structures. In a statically indeterminate structure, the longitudinal reinforcement in the compression layer above an intermediate support provides the bending moment capacity [73].

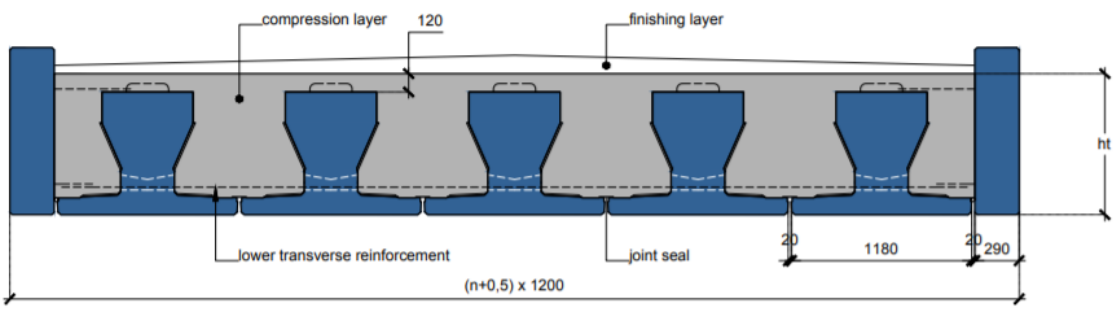


Figure 2.23: Cross-section of a solid infilled beam (SJPFlex) deck [72]

(a) Prefab inverted T-profiles put into place [68]

(b) Finished solid slab bridge [68]

Figure 2.24: Examples of solid deck bridges

Some typical examples of solid deck slab bridges are shown in Figure 2.24.

Girder bridges

The girder bridge is suitable for medium to long span bridges. The deck can either be composed of several prestressed I-shaped or inverted T-profiles, combined with edge beams for which many different profiles exist. After installation of the beams, they are usually connected by means of in-situ cast crossbeams at both beam ends, which ensures fixation and rotational stiffness of the beams after erection, and distributes loads better in transverse direction. Finally, a deck slab is cast in-situ by means of applying formwork (usually plywood shuttering planks) positioned on a notch at the top of the beams (see Figure 2.25) [65, 68].

A big advantage of this system, both from economic and from sustainability perspective, is the fact that it safes a lot of material. Usually the beams have a standard width of 1200 mm. The height of the inverted T-profiles typically ranges between 700 and 1700 mm, and the thickness of the compression layer excluding the finishing layer ranges between 230 and 260 mm (see Figure 2.25) [74]. The top of the beams are provided with protruding reinforcement in order to ensure a good connection with the deck. A specific characteristic of this system is the fact that the deck has a closed bottom, however, girder bridges composed of I-shaped beams with a certain distance in between them exist, resulting in longitudinal openings when looking from underneath the bridge. With this system, spans of 15 up to 60 m can be realised and a slenderness ratio of approximately 20-28 is usually obtained [65, 68].

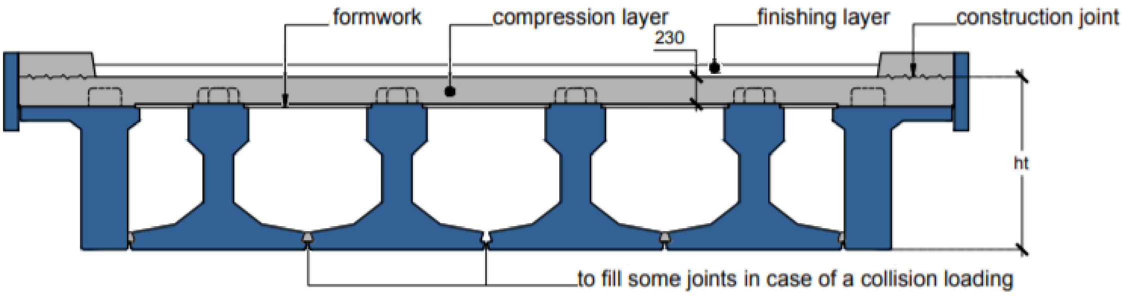


Figure 2.25: Cross-section of a girder (ZIPXL; inverted T-profile) deck [75] (adapted)

(a) Construction of girder bridge with transversal reinforcement for crossbeam at support put in place [65]

(b) Construction of girder bridge over highway with no traffic interruption [68]

Figure 2.26: Examples of girder bridges

This system is suitable for both statically determined and statically indeterminate structures. A statically determined structure is preferred, since statically indeterminate structures often result in large hogging bending moments at intermediate supports due to settlement, temperature effects, shrinkage and creep [74].

Some examples of girder bridges are shown in Figure 2.26. Besides, standard details from Rijkswaterstaat of girder decks (inverted T-profiles) are attached in Appendix B.

Box beam bridges

The box beam bridge is also suitable for medium to long span bridges. The deck is composed of prestressed box-shaped beams placed either side-by-side or at a small distance. Once the beams are placed, the onsite work is merely limited to filling of the longitudinal joints and the transversal post-tensioning of the beams [68]. Usually the beams have a standard width of 1200 or 1500 mm. The height of the box beams typically ranges between 700 and 1900 mm, and no in-situ deck needs to be casted (see Figure 2.27) [76]. It is possible to apply protruding reinforcement in the beams for connections to cast in-situ edge profiles, joint constructions, a screed, etc. With this system, spans of 15 up to 69 m (a world record-length [77]; see Figure 2.28b) can be realised and a slenderness ratio of approximately 28-32 is usually obtained [65, 68].

This system is suitable for both statically determined and statically indeterminate structures, and it can be used for both perpendicular and skewed crossings [74]. Besides, this system has some other additional advantages [65, 76]:

- Savings on the construction height resulting in slender deck structures (e.g. 1500 mm height for 50 m span).
- The two-directional prestressed deck can be available for traffic after just 1 week.

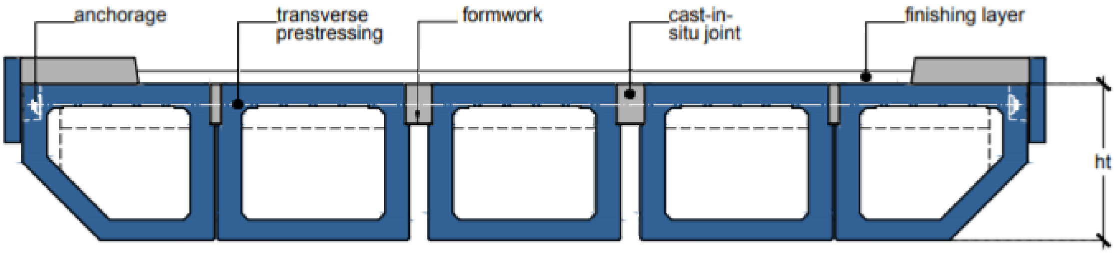


Figure 2.27: Cross-section of a box beam (SKK) deck) [78] (adapted)

(a) Construction of box beam bridge for metro rail in Amsterdam [65]

(b) Construction of Lienebrug in Wanssum (NL) with world record-length box beams of 69 m [77]

Figure 2.28: Examples of box beam bridges

- Box beams can be produced horizontally curved with an arc radius up to 100 m.
- The box beam bridge system can be made impact-resistant.
- Due to its cross-section, the box beams are torsion-stiff.
- Building with box beams is fast.
- No special edge beams are required.
- By means of smart use of transverse prestressing at intermediate supports, the need for capping beams can be eliminated (*Spanbeton®-4P-system*).

Some examples of box beam bridges are shown in Figure 2.28. Besides, standard details from Rijkswaterstaat of box beam decks are attached in Appendix C.

Finishing layers
The finishing layer (Dutch: "*deklaag*") of a prefab viaduct usually consists of a 100 to 120 mm thick asphalt layer, which is applied either on the compression layer or can be directly applied on top of the beams in case of a box beam deck [79].

Kerbs
The main purpose of kerbs (Dutch: "*schampkanten*") are providing safety provisions, such as anchorage of safety barriers and parapets (see Figure 2.29). Kerbs function to withstand impact loads and to resist against lateral collision. Therefore, kerbs are mandatory on both sides of a bridge. Besides, it provides a water barrier and the inclusion of casing (Dutch: "*mantelbuizen*") for utilities. The kerbs are usually cast in-situ, because of the high level of accuracy that is required [65, 80]. The document RTD 1010 [XIb] provides standard details for concrete bridges, amongst which are standard details for kerbs, of which a detail is shown in Figure 2.29.

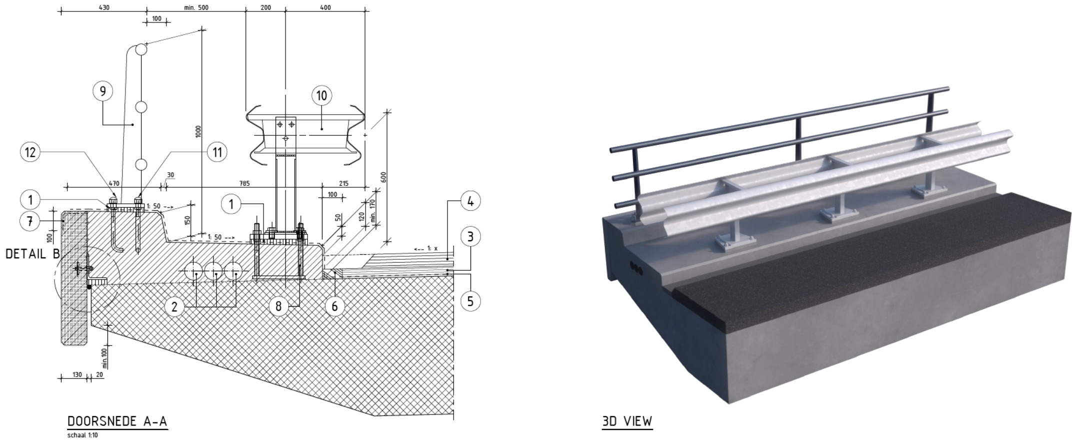


Figure 2.29: Cross-section of a standard Rijkwaterstaat kerb detail and a 3D rendering [XIb]

Substructure

As mentioned before, the substructure of a viaduct consists of abutments (or bank seats)[6], intermediate pier(s) (if present), capping beam(s) (in case of a prefab deck), and a foundation. These components are consecutively discussed below.

Abutments

An abutment (Dutch: "*laaggefundeerd landhoofd*") functions as a foundation for the end of the viaduct, and it connects the superstructure (the deck) with the main road. Besides, an abutment usually has an earth-retaining function provided by the front and wing walls, however a layout with no earth pressure on the front wall is possible as well (see Figure 2.30). An abutment can be founded on raking piles (Dutch: "*schoorpalen*") (slope 5:1 - 7:1) or on a spread footing, depending mostly on the local soil conditions. In case piles are chosen, these are inclined in order to resist the horizontal forces caused by breaking/acceleration forces, earth pressure, and temperature influences.

Besides, a sheet pile or combi wall (combination of tubular piles and sheet pile walls) can be applied, which has similar (structural) principles as an abutment. A foundation on a sheet pile or combi wall namely both has an earth-retaining (horizontal component) and a load-bearing (vertical component) function. Additionally, a sheet pile or combi wall functions as a foundation in itself. Particularly in the case of small spans, a foundation on sheet piles or on combi walls is often used [67].

The advantages of an abutment compared to a bank seat are a shorter total span, and a much better load distribution from the deck to the subsoil. The main disadvantage compared to a bank seat is a reduced sight under the viaduct which makes it undesirable for (highway) viaducts, however it is a suitable solution for a viaduct that, for example, spans a railway track [65, 66].

In some cases, prefab elements are used to construct abutments. The elements are full height, have a modular width, and usually have one or more top to bottom ribs on the earth-retaining backside, which implies a T or double-T cross-section (see Figure 2.31). In case of considerable heights, a precast tie can be used to form a truss structure. The elements are placed side by side and are usually completed by means of a cast in-situ foundation and top beam [68].

Bank seats

A bank seat (Dutch: "*hooggefundeerd landhoofd*") has the same function as an abutment, and, besides on piles or on a spread footing, can also be founded on a sheet pile or combi wall. A bank seat is located on top of the embankment, which implies that the embankment needs a certain slope (see Figure 2.32). This results in the advantage of more light under the bridge and simultaneously better

[6]In this context, 'abutment' is used as the general term for both terms (Dutch: "*landhoofd*"). The difference between both is explained in the following paragraphs.

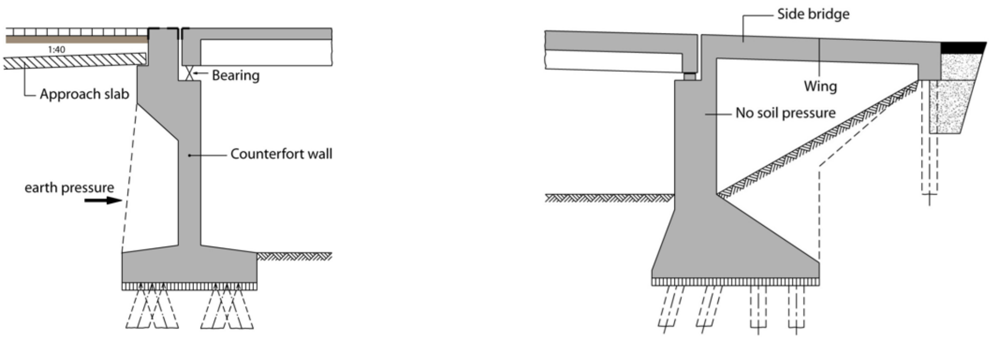
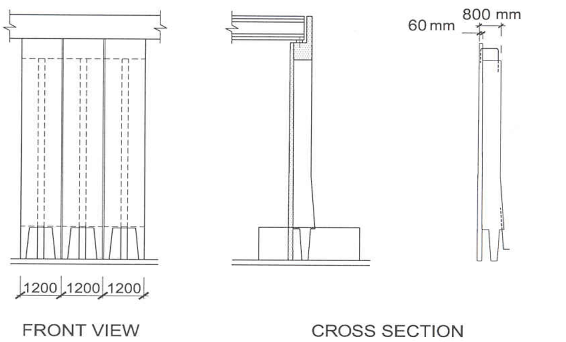


Figure 2.30: Schematic cross-sections of two types of abutments; with (left) and without earth pressure on front wall (right) [65]



(a) Schematic cross-section of prefab abutment elements [68] (adapted)

(b) Example of prefab abutment with ribbed wall [68]

Figure 2.31: Examples of prefab abutments

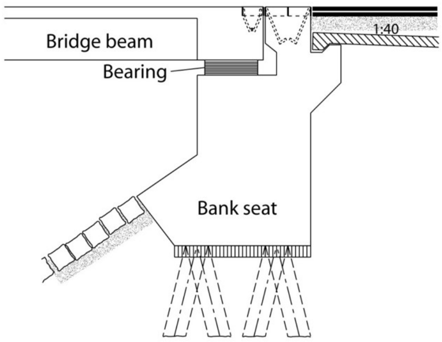
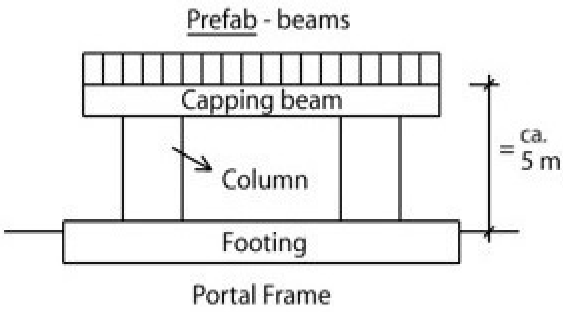
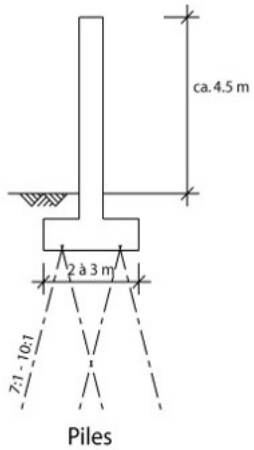


Figure 2.32: Schematic cross-section of a bank seat [65]

Figure 2.33: Examples of a viaduct with a bank seat (left) and a viaduct with an abutment, crossing a railway track (right) [65]



(a) Schematic cross-section and components of an intermediate pier [65] (adapted)



(b) Schematic cross-section of an intermediate pier [65] (adapted)

Figure 2.34: Components and rough dimensions of an intermediate pier

(a) Hoisting of a prefab bridge pier [68]

(b) Completely prefab viaduct “Groenedijk”, Rotterdam [81]

Figure 2.35: Examples of prefab capping beams and intermediate piers

sight under and beyond the viaduct, which is why a bank seat is typically used for viaducts crossing a highway. Another advantage compared to an earth-retaining abutment is that the earth pressure on a bank seat is minimal, and therefore smaller wings are required. Finally, a bank seat is simpler to construct. The largest disadvantage of a bank seat compared to an abutment, however, is the fact that a longer total span is required [65, 66].

In both solutions, an inclined transition slab (see e.g. Figures 2.30 and 2.32) is needed to smooth the transition between road and viaduct, and to redistribute settlement differences. Typical dimensions of a transition slab are 6 to 8 m by 1 m, and with a thickness of 300 to 400 mm [65, 66].

Some examples of viaducts with either a bank seat or an abutment are shown in Figure 2.33.

Intermediate piers and capping beams
Most viaducts spanning a typical Dutch highway need one or more intermediate piers. The typical layout of a pier of a viaduct consists of one or more columns, a footing, and, in case of a prefab superstructure, a capping beam on which the prefab elements are supported (see Figure 2.34a). The footing usually is founded either on raking piles (slope 7:1 - 10:1; see Figure 2.34b), or on a spread footing. The slope of the piles under a pier is generally smaller than the slope of the piles under an abutment or a bank seat, because no earth pressure is exerted on an intermediate pier. The piers are generally cast in-situ because of their relative large size and weight, however, there are several examples of prefab piers, both columns and capping beams (see Figure 2.35). Here, transport capacity is the main limiting

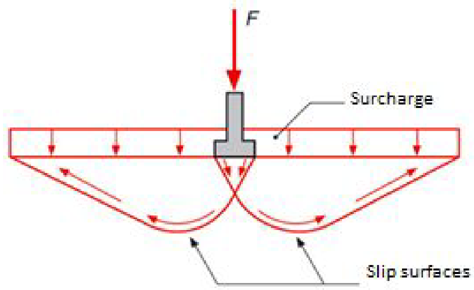


Figure 2.36: Load distribution of a spread footing foundation [66]

factor [66, 68]. As was mentioned earlier already (see “Box beam bridges”, page 54), nowadays it is even possible to eliminate the need for a capping beam in order to support prefab box beams at an intermediate pier.

Foundations
The forces on and the self-weight of a viaduct have to be transferred to the subsoil via the foundation. Generally, two methods are possible: a foundation on piles, or a spread footing foundation. A foundation on piles can be made with many different type of piles; both different in terms of material and in terms of execution method. Three commonly used types of piles are prefab concrete piles, drilled piles (concrete) (Dutch: “*boorpalen*”), and steel pipe piles (Dutch: “*buispalen*”) [80].

The soil conditions in the Netherlands are such that the use of a spread footing foundation is impossible in many parts of the country. However, in the southern part of the Netherlands it can be done. In order to be able to use a spread footing foundation, a stable soil (e.g. sand) is required, which is able to withstand and distribute the load. Alternatively, reinforced soil is commonly applied for the foundation of a bank seat [79]. An example of the load distribution of a spread footing foundation is shown in Figure 2.36 [66, 80].

Connections and joints
As was mentioned in subsection 2.6.1, the super- and substructure are usually connected by means of both bearings and transition (expansion) joints. Besides, several methods exist for connecting elements or components, such as connections by means of protruding bars and cast-in ducts (Dutch: “*stek-gain verbinding*”), and connections by means of post-tensioned bars. These connections and joints are consecutively discussed below.

Bearings
Bearings have mainly three functions. Firstly, they have to transfer forces from the superstructure to the substructure and spread the stresses. Secondly, they have to allow built-in movement of the bridge deck to accommodate small movements due to thermal transition and shear stress strains. A third function is to fill up the space between the beam and the pier. There are two main types of bearings used nowadays, namely elastomeric (rubber) bearings (see Figure 2.37), and pot or spherical bearings (Teflon) (see Figure 2.38). Elastomeric bearings are usually reinforced with steel plates, and can both be anchored or unanchored. They have load-carrying capacities ranging from 150 to 20.000 kN. Pot and spherical bearings consist of several members with different friction coefficients. Their capacity ranges from 1000 to 100.000 kN [65].

Besides, both bearing types exist in three different types regarding their degree of freedom. These types are fixed (constrained in two transverse directions), constrained in one transversal direction and free to move in the other transversal direction, or free in both transversal directions. The choice for which bearing type to use depends mainly on the following aspects [65]:

- Type of construction
- Maximum and minimum vertical loads

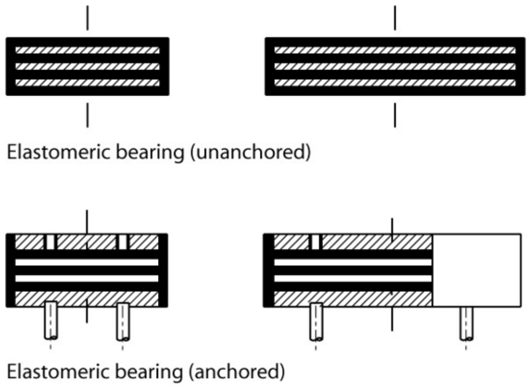
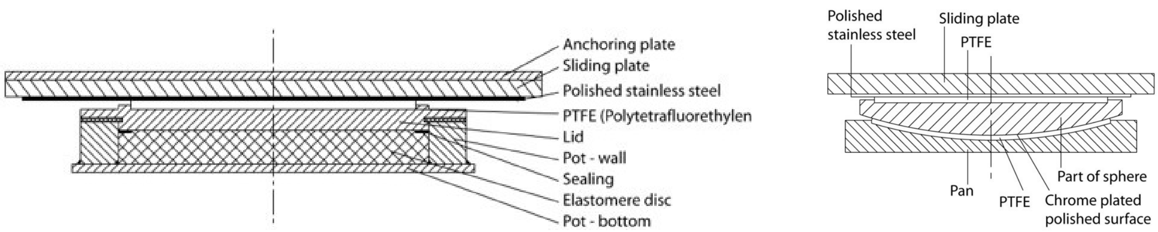


Figure 2.37: Schematic layout of anchored and unanchored elastomeric bearings [65]



(a) Members of a pot bearing [65] (adapted)

(b) Members of a spherical bearing [65] (adapted)

Figure 2.38: Schematic layout of pot and spherical bearings

- Maximum and minimum horizontal loads and direction
- Range of rotations
- Range of transversal movements
- Permissible concrete stress

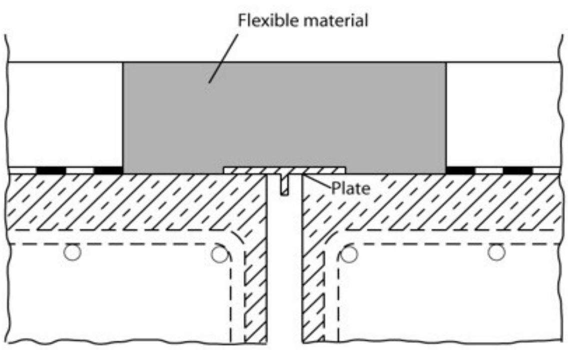
Usually bearings are placed on dumps (Dutch: "*opstorts*"). This enables to obtain a horizontal contact surface, and besides the space between the bottom of the deck and top of the contact surface can be filled partially, resulting in significantly less thicker bearings. Furthermore, the replaceability and maintainability of bearings is usually an important point of attention. This mainly concerns a sufficient available space for jacking of the superstructure and sufficient space to perform maintenance or replacement [79].

Some examples of bearings are shown in Figure 2.39. Besides, standard details from Rijkswaterstaat of decks of both girder bridges (inverted T-profiles) and box beam bridges are attached in Appendix B and C respectively, in which the bearings are indicated as well.

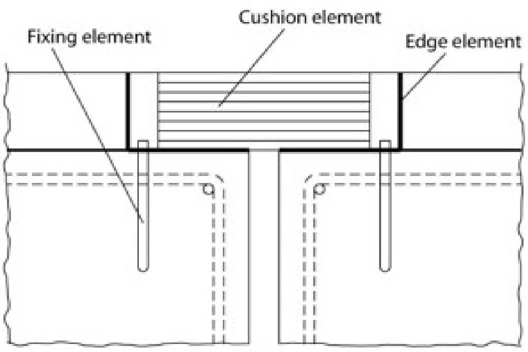
Transition (expansion) joints
The function of expansion joints is twofold. The main function is to accommodate deformations of the bridge deck due to temperature variations and shrinkage/swelling of concrete. Besides, they are used to smooth the transition between adjacent intermediate spans and between end spans and abutments. A rule of thumb is that a bridge deck moves 1 mm/m. Thus, for larger spans, expansion joints with a larger deformation capacity are required, assuming the same number of dilatations. Therefore, several types of expansion joints exist, allowing small (15 to 25 mm), medium (25 to 80 mm), or large (80+ mm) movements (see Figure 2.40). However, two main disadvantages of expansion joints are the long-term durability problems in presence of de-icing salt, and the discomfort (noise and bumps) it causes [66, 68].

An example of a commonly applied expansion joint is shown in Figure 2.41a. On the other hand, the result of not applying an expansion joint can be seen in Figure 2.41b, in which a discontinuity in the

Figure 2.39: Examples of bearings [65]



(a) Asphaltic plug joint (small movements) [65]



(b) Expansion joint for medium movements [65]

Figure 2.40: Schematic layout of expansion joints

(a) Example of finger joint (large movements) [66]

(b) Consequences of not applying expansion joints [65]

Figure 2.41: Examples of (not applying) expansion joints

deck has appeared. Besides, document RTD 1007-1 [XIa] covers a wide variety of expansion joints, including a decision matrix, supporting in which type of expansion joint to apply.

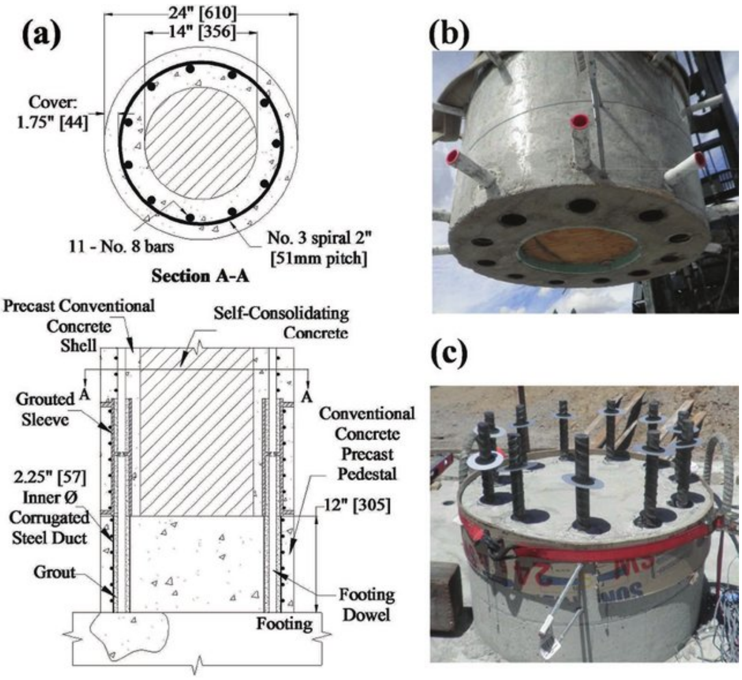


Figure 2.42: Example of a connection by means of protruding bars and cast-in ducts; (a) cross-sectional details, (b) cast-in ducts in column and (c) protruding bars in column base [82]

Protruding bars and cast-in ducts

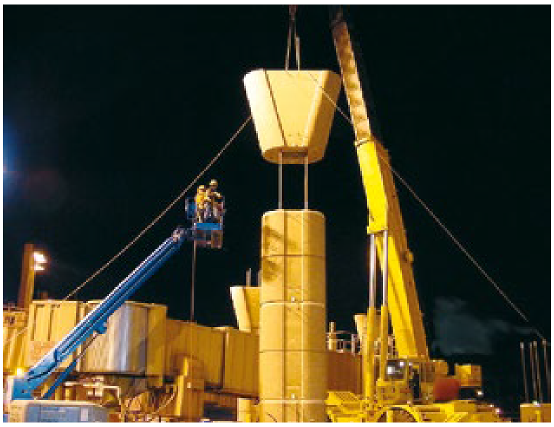
A joint by means of protruding bars and cast-in ducts (Dutch: "*stek-gain verbinding*") is a common joint type for the connection of precast elements, and is especially widely applied in the prefab building industry, however it can also be applied in bridge construction [80]. In fact, these joints were used to connect the different prefab components of the substructure of viaduct "Groenedijk" in Rotterdam (see Figure 2.35b).

In order to connect prefab elements and establish a joint that is able to transfer the acting forces, cast-in ducts of one element are made to fit precisely over protruding reinforcement bars of another element. Before casting of the elements, these ducts, which for example can be corrugated steel or PVC ducts, therefore need to be placed inside the mould in order to be casted into the concrete elements. After placing and adjusting the elements that are to be connected, the ducts are filled with grout by either pouring it into the duct from above or by injecting it from below. However, this way of connecting also results in the fact that the joint is not demountable, at least not in such a way that it can be done without damaging the elements to a serious extent. An example of such a joint is shown in Figure 2.42.

Post-tensioned bars

For temporary or permanent assembly of components (i.e. the assembly of several elements into one component, for example a pier or bridge deck; see Figure 2.43), or for connections between prefab components (pier to capping beam, or pier to footing) a post-tensioned bar system can be used to connect these elements or components. Several companies exist which provide more or less the same system (DYWIDAG-Systems International B.V., Freyssinet Nederland B.V., VSL International Ltd.), and which have proven that the system can successfully be applied by using it in practice [83–85].

The system is fairly simple. It consists of a steel bar and an anchorage system which usually consists of some sort of nut and washer. The bars can either be bonded (i.e. embedded in the concrete), or unbonded. In case of unbonded bars, they can both be applied internally (i.e. inside the cross-section) or externally (i.e. outside of the cross-section) [83–85]. It is not mentioned in any of the product manuals however (see [83–85]), if mortar needs to be applied between connecting concrete elements (see e.g. Figure 2.43a) or if the elements can be connected with a cold joint, i.e. concrete to concrete. From a durability perspective however, it is likely that it is preferred to provide at least some sort of protective



(a) Assembly of prefab elements into a pier [83]

(b) Assembly of bridge deck elements [83]

(c) Final assembled pier (see Figure 2.43a) [83]

Figure 2.43: Examples of applications of (unbonded) post-tensioned bars

layer in between connecting concrete elements to both shim any unevennesses and to protect the concrete elements against damages [79].

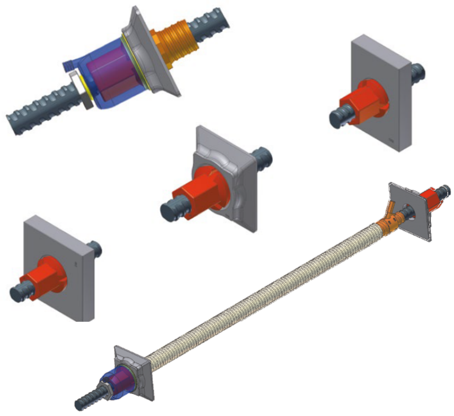
However, it is mentioned that different methods exist to protect the system itself against corrosion. Which method is applied depends mainly on the expected design lifetime and the conditions of exposure. Besides, a high quality thread as a result of the fabrication process, optimised material selection and careful detailing ensures high fatigue resistance and low susceptibility to stress corrosion [83, 84].

Some examples of both bonded and unbonded post-tensioned bar systems and applications are shown in Figure 2.44 and Figure 2.45 respectively.

Execution methods

The main procedure in order to construct a prefab viaduct consists of the following (global) steps [65, 66, 68]:

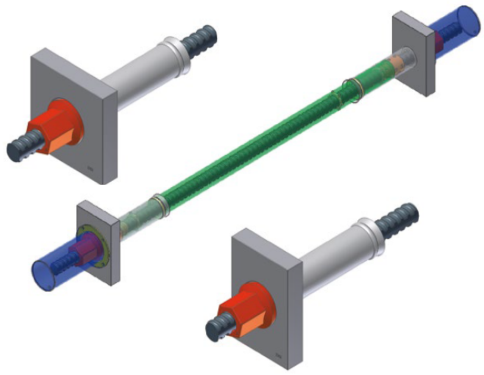
1. Preparation of the construction site.
2. Construction of the substructure: foundations, abutments or bank seats, intermediate piers and capping beams, either constructed in-situ or prefabricated and transported to the construction site.
3. Application of bearings and other provisions that are needed to connect the substructure with the superstructure.
4. Construction of the superstructure: hoisting in the prefab beams.
5. Finishing of the superstructure, depending on the used prefabricated system: casting the in-situ deck, applying the transversal prestressing, installing the transition joints, etc.
6. Finishing of the viaduct: installations, safety barriers, parapets, finishing layer (asphalt), etc.



(a) Bonded bar and anchorage system [83]

(b) Application of bonded post-tensioned bars [83]

Figure 2.44: Example of bonded post-tensioned bar system and application



(a) Unbonded bar and anchorage system [83]

(b) Application of unbonded post-tensioned bars [83]

Figure 2.45: Example of unbonded post-tensioned bar system and application

Elements that are prefabricated in a factory and that are transported to the construction site over road, often requiring special trucks, usually is done at night in order to limit the traffic disturbance. However, if water is nearby, elements can also be shipped, or in some countries they are even transported by rail. On-site, usually two mobile cranes are needed to hoist the prefab elements into their final position [68].

2.6.3. Cast In-Situ Viaducts

A cast in-situ viaduct is completely constructed on the construction site. For this type of viaducts, sufficient space on the construction site is required for both the materials and equipment that is needed for construction [66].

A cast in-situ viaduct has several advantages compared to a prefab viaduct [66]:

- Different kind of shapes and cross-sections are easier to realise.
- Monolithic connections can easily be realised.
- A capping beam is not needed.
- The construction process is more flexible.
- Less bearings and transition joints are needed, which both saves money and maintenance.

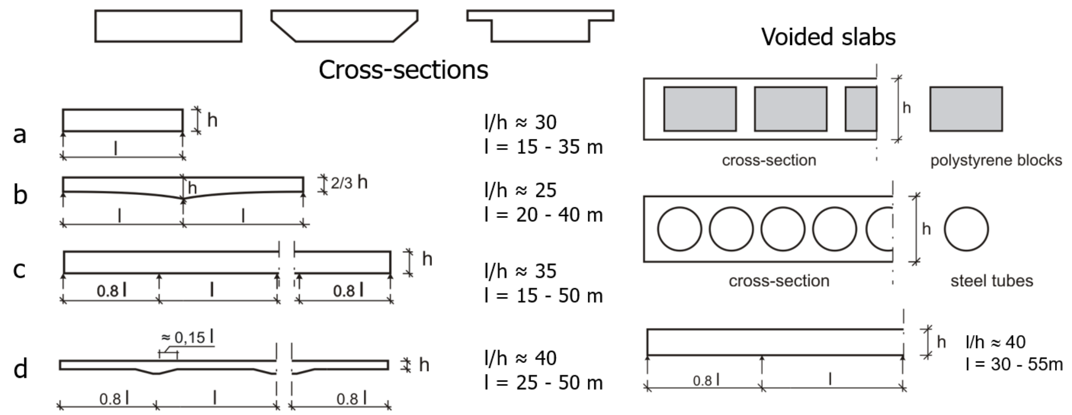


Figure 2.46: Different cross-sections and shapes of cast in-situ bridge decks [65]

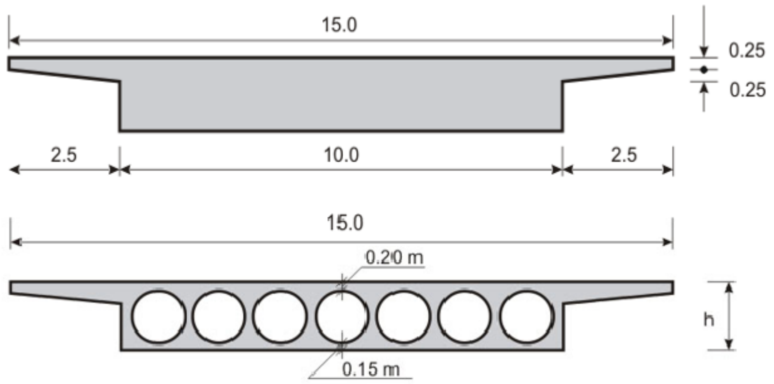


Figure 2.47: Typical cross-section of a slab bridge, including indicative dimensions (all in meter) [65] (adapted)

Disadvantages compared to a prefab viaduct are [66]:

- The construction time on-site is longer.
- The construction method is labour intensive.
- Temporary formwork and falsework is required.
- In case the viaduct is crossing an existing road, traffic interruption is usually considerably large.
- Concrete and product quality is less certain due to the outdoor working and weather conditions.

Superstructure

The deck of a cast in-situ viaduct consists of a solid slab. Different cross-sections and shapes of decks exist. Furthermore, decks can be simply reinforced, or prestressing or post-tensioning can be applied. If a viaduct consists of more than one span, also different cross-sections in longitudinal direction are possible, e.g. varying thicknesses of the deck along the span. These different cross-section and span types result in different slenderness ratios. In order to save weight, polystyrene blocks or steel tubes can be casted in the deck (see Figure 2.46). A statically indeterminate deck is preferred, since this reduces the required thickness of the deck [65, 66, 80].

A common cross-section type of a cast in-situ slab bridge is shown in Figure 2.47. The cantilevering sections can usually be thinner as these spaces are reserved for bicycle and/or pedestrian lanes, and safety barriers and parapets. Besides, this positively influences the weight of the bridge deck. The width of the thick section is usually determined by the number of prestressing tendons that is required, since a considerable amount of space is required for the anchors [65].

Finishing layers
A similar finishing layer (Dutch: "*deklaag*") for a cast in-situ viaduct as for a prefab viaduct is applied. This means that usually a 100 to 120 mm thick asphalt layer is applied, which in this case can always be applied directly on top of the solid deck slab [79].

Kerbs
The requirements with regards to kerbs (Dutch: "*schampkanten*") for cast in-situ viaduct are the same compared to prefab viaducts. The kerbs are usually cast in-situ in a second pour since the anchors for safety barriers and parapets need to be positioned with a very high level of accuracy. Besides, from a practical perspective, it is very hard to cast the kerbs and the deck at the same time [79, 80].

Substructure
The components of the substructure of a cast in-situ viaduct are comparable to those of a prefab viaduct. The same alternatives for abutments, bank seats, intermediate piers, and foundations exist. However, the main difference compared to a prefab viaduct is the fact that capping beams can always be left out. The deck then is supported directly by the piers (see for example Figure 2.22a). However, in some cases capping beams are still applied, for example for aesthetical purposes [66, 79].

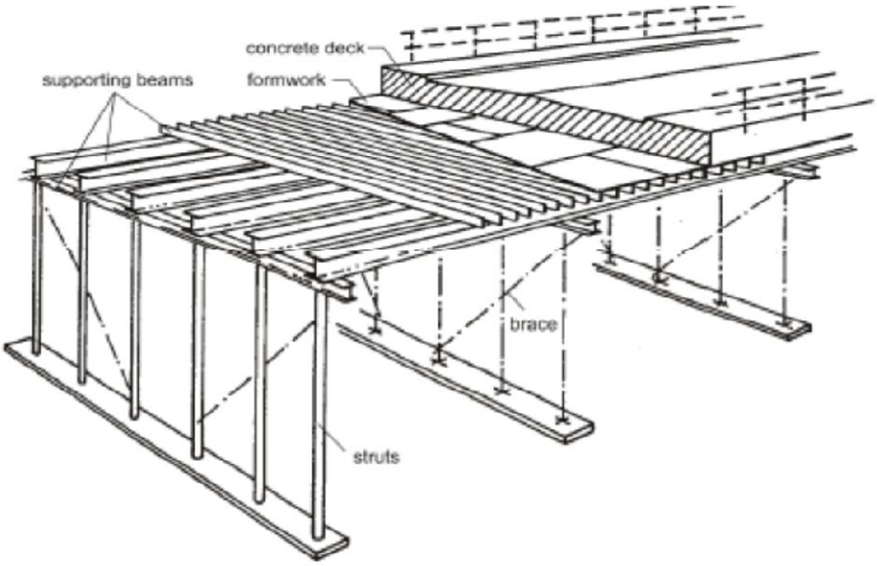
Connections and joints
Also with regards to connections and joints, the same alternatives for bearings and transition (expansion) joints exist for cast in-situ viaducts as for prefab viaducts. One main difference is that generally less bearings are needed simply because of the fact that a prefab viaduct requires a bearing at each beam end, which, for example, results in two rows of bearings on an intermediate support, whereas a single row of bearings suffices for a cast in-situ deck. Besides, it is possible to monolithically connect super- and substructure (see "Integral bridges", page 69) [66, 79].

Execution methods
The main procedure in order to construct a cast in-situ viaduct consists of the following (global) steps [65, 66]:

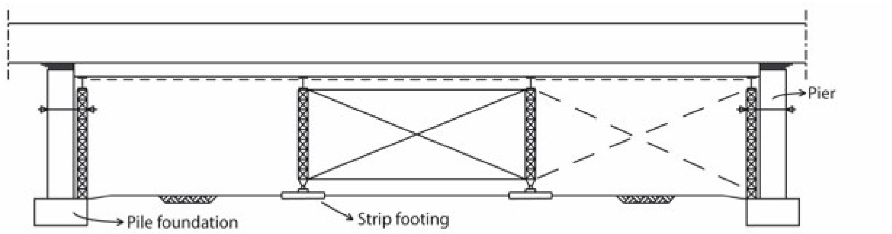
1. Preparation of the construction site.
2. Construction of the substructure: foundations, abutments or bank seats, intermediate piers and capping beams, either constructed in-situ or prefabricated and transported to the construction site.
3. Application of bearings and other provisions that are needed to connect the substructure with the superstructure.
4. Construction of the superstructure: casting of the deck slab.
5. Finishing of the superstructure: post-tensioning of the prestressing tendons, installing the transition joints, etc.
6. Finishing of the viaduct: installations, safety barriers, parapets, finishing layer (asphalt), etc.

There are several execution methods to construct a cast in-situ viaduct, amongst which the most common methods are [65, 66]:

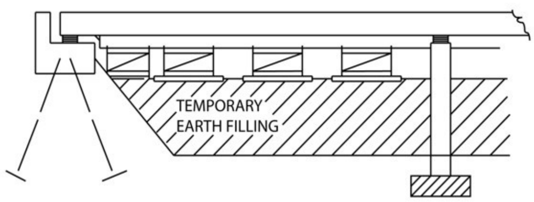
- The use of traditional formwork and falsework, as can be seen in Figures 2.48a and 2.48b.
- Constructing the deck at a higher level, and lowering it onto the supports (abutments, and intermediate piers) with jacks.
- Constructing the deck on a (temporary) soil filling (e.g. sand), and excavate the sand after completion of the deck (see Figure 2.48c).
- Construction of (part of) the deck next to the viaduct, and drive or launch the deck into its final position using, for example, horizontal jacks (see Figure 2.48d).



(a) Traditional formwork and falsework construction [66] (adapted)



(b) Side view of scaffolding layout [65] (adapted)



(c) Construction on temporary soil filling [65]

(d) Horizontal jacking of deck into final position [65]

Figure 2.48: Examples of execution methods for cast in-situ viaducts

2.6.4. Structural Systems

Several structural systems with regards to the superstructure i.e. deck structure are possible. These systems are [68]:

- Simply supported bridges
- Simply supported bridges with continuous slabs
- Continuous (statically indeterminate) bridges
- Integral (statically indeterminate) bridges

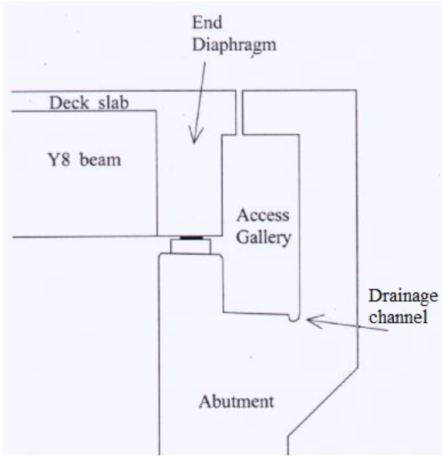


Figure 2.49: Provisions to delay durability problems [68] (adapted)

As was mentioned earlier (see "Superstructure", page 64), a statically indeterminate deck (i.e. continuous bridge) is preferred and therefore usually applied. However, for prefab viaducts, each of these structural systems are applied. Therefore, these systems are consecutively discussed below.

Simply supported bridges

The first prefab bridges were simply supported, since it was considered logical to design simply supported bridge decks with transition joints at intermediate supports and between beam ends and abutments. In case of a simply supported bridge deck, each beam is supported on an individual bearing; one at each beam end [68].

The main advantages of simply supported bridges are [65, 68]:

- Beams are directly installed on their final bearing.
- Differential settlements do not influence the structural system.
- Joints are dimensioned to allow for thermal movements, creep and shrinkage.
- Simply supported deck systems result in durable structures.

The main disadvantages of simply supported bridges are [65, 68]:

- The need for bearings at both beam ends: both expensive and need frequent maintenance or even replacement.
- The need for transition joints: discomfort to traffic and long-term durability issues in presence of de-icing salt.

Good detailing of joints can delay the durability issues (reinforcement corrosion and wear of bearings), for example by applying provisions for inspection and replacement of bearings and/or by providing drainage channels for the drainage of water (see Figure 2.49). However, the most effective solution is to eliminate the transition joints in the deck, which is done in the other systems [68].

Standard details from Rijkswaterstaat of decks of both girder bridges (inverted T-profiles) and box beam bridges are attached in Appendix B and C respectively, which also indicate details for simply supported decks at intermediate supports (see detail "met open voegconstructie").

Simply supported bridges with continuous slabs

There are two methods, relying on simple measures with a minimum of extra design and construction effort, to provide simply supported continuous decks. This is also referred to as partial continuity, which

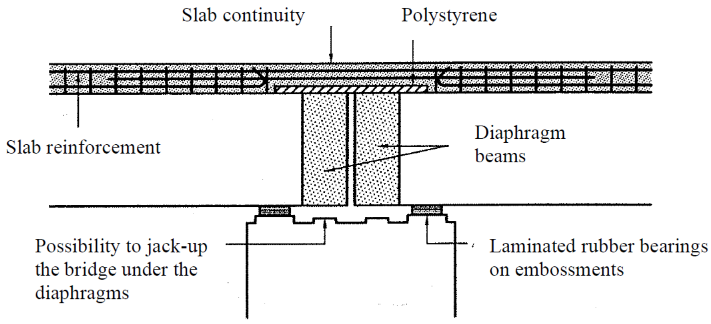


Figure 2.50: Detail of a simply supported bridge with continuous, separate, slabs (partial continuity; method 1) [68]

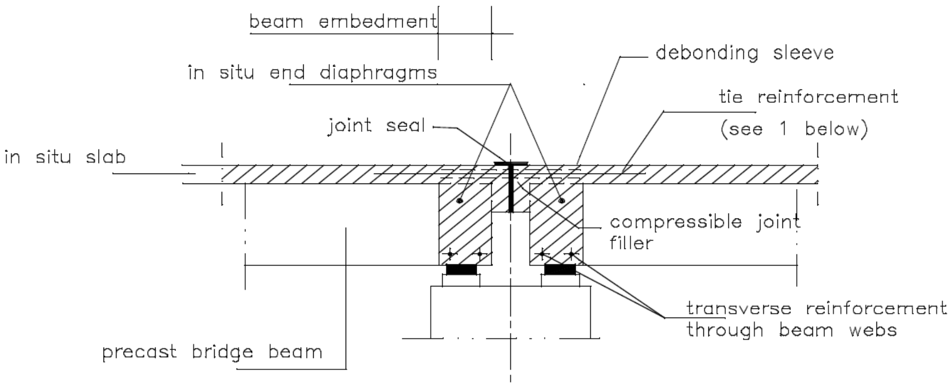


Figure 2.51: Detail of a simply supported bridge with continuous, tied, slabs (partial continuity; method 2) [68]

means that only continuity of the deck slab is provided while the beams are designed as simply supported beams. As a result, no vertical loads, both self-weight and live load, are transferred between both bridge decks [68].

The first method restricts the continuity to the slab only, which is able to deflect in order to accommodate the (different) rotations of the simply supported beams. The beams are erected in the conventional way onto individual bearings. The deck slab is separated from the crossbeams (in case of inverted T-profiles) over a length of about 1,5 m by a layer of deformable material, for example expanded polystyrene, to provide rotational flexibility. Besides, there is no continuity reinforcement between the beam ends. An example of this method is shown in Figure 2.50 [65, 68].

The second method is based on the conventional design and construction of simply supported multi-span bridges. Here, the beams are also erected in the conventional way onto individual bearings. The difference with respect to the first method is that here longitudinal reinforcement bars are incorporated in the slab in order to tie the slabs together over the intermediate support. Therefore, expansion movement at deck level is eliminated and the use of an incorporated deck rotation joint (Dutch: "*buigslappe voeg*") is required. This can be accommodated by debonding the continuity reinforcement over a certain length at both sides of the joint, while applying a compressible joint filler around the debonded surface. Besides, the slab and crossbeams usually have a reduced thickness in order to provide more flexibility and rotational capacity. An example of this method is shown in Figure 2.51 [68].

Standard details from Rijkswaterstaat of decks of both girder bridges (inverted T-profiles) and box beam bridges are attached in Appendix B and C respectively, which also indicate details for simply supported bridges with continuous deck slabs at intermediate supports (see detail "met buigslappe voegconstructie").

Continuous bridges

Continuous bridges are multi-span bridges with mechanical continuity between adjacent spans, which is realised by connecting the beams by means of an integral reinforced concrete crossbeam on top of the intermediate supports. This is realised in two steps [65, 68]:

1. First, the beams are simply supported, and carry their self-weight plus load from formwork and wet cast concrete.
2. Then, after hardening of the crossbeam and deck, the structure becomes continuous, but only for additional dead and live load.

Again, several different solutions exist, however, these are not elaborated upon further.

The main advantages of continuous bridges are [68, 80]:

- More slender decks can be realised, which has beneficial consequences for, among others, the foundation (e.g. less/smaller foundation piles) and approaching ramps.
- Horizontal curvature of the bridge can be easily accommodated by varying the width of the integral crossbeam to form a trapezium, resulting in the possibility to use prefab beams with the same length over the entire width.
- Only a single row of bearings is required.
- The intermediate piers can be more slender since bending moments applied to the intermediate piers by the eccentric position of bearings are prevented.

The main disadvantages of continuous bridges are [65]:

- The system is more complex to design (however, this should not be a reason not to consider a continuous system [80]).
- The system is more expensive to construct because of the continuity (however, overall it might result in a cheaper construction [80]).
- A rather wide in-situ crossbeam is required.

Integral bridges

An integral bridge is characterised as a bridge (or viaduct) without transition joints, neither between adjacent spans nor between end spans and abutments. A distinction between two types of integral bridges can be made, namely a (fully) integral bridge or a semi-integral bridge. In the case of a (fully) integral bridge, the abutments, including the foundation, are monolithically connected to the bridge deck. Since the abutments are connected to the bridge, they have to follow the horizontal movements of the bridge caused by breaking/acceleration forces, earth pressure, and temperature influences. Therefore, in this case, the abutments have to be designed to allow these movements to occur and at the same time be able to resist traffic loads. In the case of a semi-integral bridge, the earth-retaining function of the abutment at the level of the adjacent embankment is separated from the load-bearing function for vertical loads. In that case, the bridge deck has an earth-retaining end plate and a joint-free transition with the embankment. The bridge deck is laid on the abutment by means of bearings. The horizontal movements and rotations of the bridge deck are therefore only transferred to the foundation to a limited extent. Examples of integral and semi-integral bridges supported by either a bank seat or an abutment are shown in Figure 2.52 [67, 68].

The main advantages of (semi-)integral bridges are [67]:

- The lack of transition joints: both from a maintenance/replacement and a driving comfort perspective.
- A generally thinner bridge deck compared to simply supported bridges.

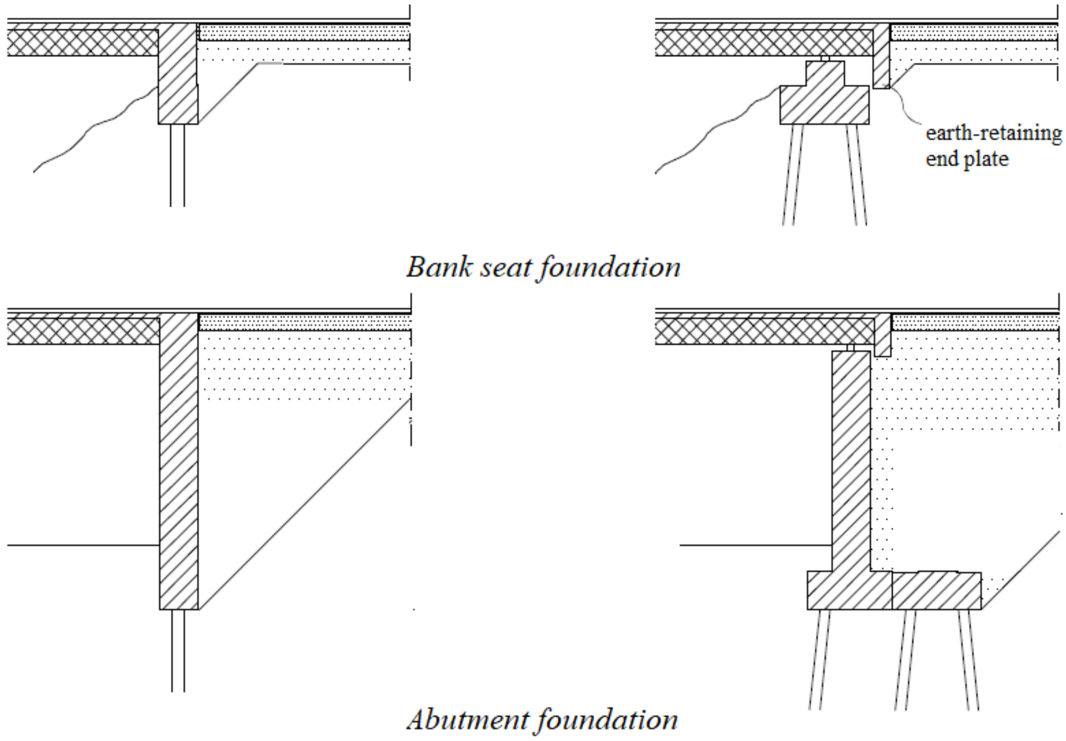
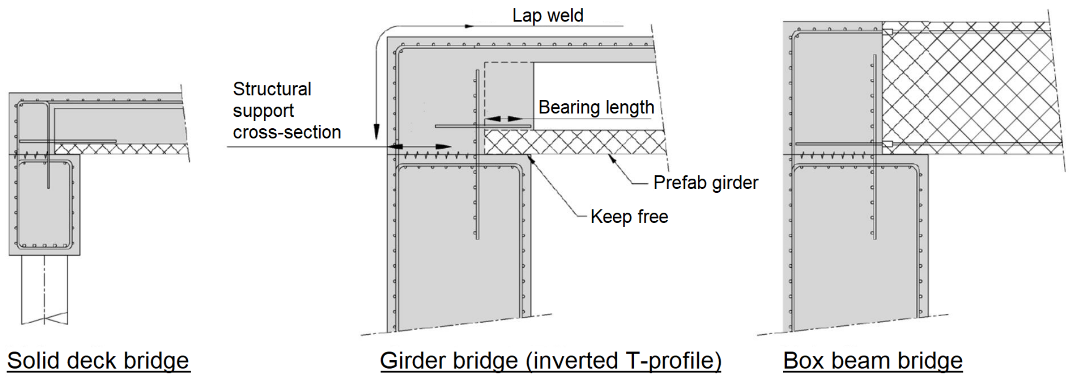
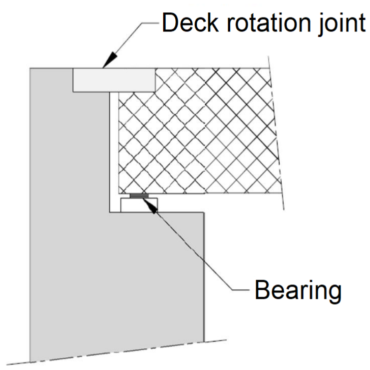


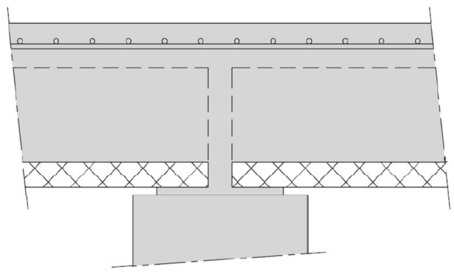
Figure 2.52: Integral bridges (left) and semi-integral bridges (right) on bank seats (top) and abutments (bottom) [67]



(a) Examples of monolithic connections between deck and abutment [67] (adapted)



(b) Semi-integral deck–abutment connection [67] (adapted)



(c) Monolithic connection at intermediate support [67] (adapted)

Figure 2.53: Examples of connection details for (semi-)integral bridges

- In case of a (fully) integral bridge, the maintenance required on bearings disappears since no bearings are applied.

- A more robust experience of this type of bridge because of the absorption of horizontal forces and the possibility of moment redistribution.

The main disadvantages of (semi-)integral bridges are [67]:

- In longer decks, the forces from the horizontal bridge deck movements become larger and less easy to determine, which can lead to large forces in the connection of the bridge deck to the abutment and the detailing of this joint to become difficult to execute.

- In case of an integral bridge, it is no longer possible to jack the bridge deck to increase the clearance height.

- Deformations in the structure are transferred to the pavement construction (asphalt layer) on the embankment due to the lack of transition joints. Except for small spans, special provisions are needed to prevent the asphalt from cracking at the end of the transition slabs, for example reinforced asphalt.

- The design and calculation of the bridge becomes much more complex.

Some examples of details at abutments and intermediate supports of (semi-)integral bridges are shown in Figure 2.53.

2.6.5. Standard Dimensions of (Existing) Concrete Viaducts

In order to be able to design a viaduct that is demountable and can be reused on other locations, it is important to have an indication of standard dimensions, such as span length and deck width, and other characteristics, like number of spans, crossing angle and location in the road layout, of (existing) concrete viaducts. The importance of this was already emphasised in subsection 2.2.1, in which one of the technical action points in order to achieve circular bridge construction was to "*develop a complimentary standardisation scheme without compromising on architectural freedom*" [1] (see Table 2.5). Therefore, based on a data set [86] provided by Rijkswaterstaat[7] containing information about nearly 3600 existing viaducts in the Netherlands, an analysis has been performed into the dimensions and characteristics of these viaducts.

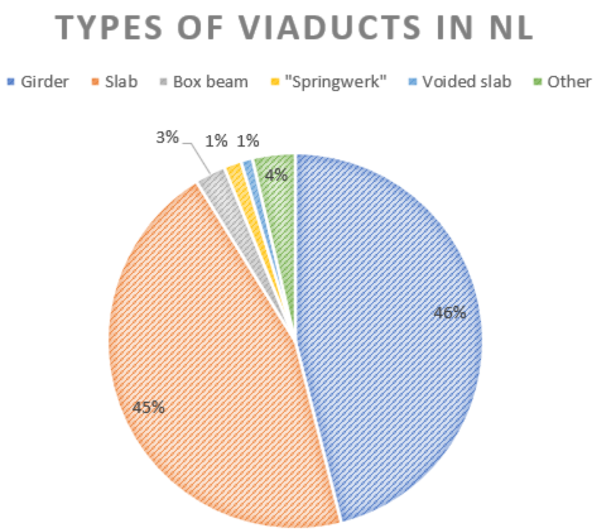


Figure 2.54: Distribution of types of existing viaducts in the Netherlands

[7] This confidential data set was carefully compiled by Rijkswaterstaat specifically for the the SBIR challenge Circular Viaducts [9] (see section 1.2) and therefore is not publicly available. For purposes of this research only, the data set was shared and permission was granted by Rijkswaterstaat to make use of this data set.

Figure 2.55: Example of a “springwerk” viaduct, characterised by its inclined piers [87]

First of all, regarding the types of existing viaducts in the Netherlands, 20 different types were identified, which were reduced to the 5 main types and a sixth category containing all other 15 types adding up to a total of 3513 viaducts. The distribution of those types of viaducts is shown in Figure 2.54. It can be seen that over 90% of the viaducts are either prefab girder (46%) or solid slab viaducts (45%). Besides, 3% are prefab box beam viaducts, and both “springwerk” (see Figure 2.55) and voided slab viaducts account for only 1% each.

Besides, the average dimensional characteristics are calculated for the five main types of viaducts (see Table 2.7). Here, it has to be explained that ‘object type’ refers the main component, i.e. girder, slab, box beam, etc. and ‘object’ refers to the total viaduct i.e. the civil work (Dutch: “*kunstwerk*”). Furthermore, other relevant characteristics, such as for example thickness of the deck, were not included in the data set. Finally, the location of the viaducts in the road layout has been analysed, i.e. whether the viaduct is located in a governmental road (GR) or crosses a governmental road (see Table 2.8).

Table 2.7: Average dimensional characteristics for the five main types of viaducts

Object type	Number	Object type avg. length [m]	Object avg. length [m]	Object avg. width [m]	Avg. number of spans	Avg. crossing angle [gon]
Girder	1618	66,3	83,7	17,1	3,0	87
Slab	1585	59,1	65,8	15,1	3,0	86
Box beam	93	140,8	150,3	13,7	4,2	86
“Springwerk”	53	52,7	52,7	13,8	3,0	87
Voided slab	35	75,8	87,2	14,9	2,3	84
Other	129	N / A	N / A	N / A	N / A	N / A
Total	3513					

Table 2.8: Distribution of the location of the five main types of viaducts in the road layout

Object type	Number	In GR		Over GR		Non GR		Unspecified	
Girder	1618	1121	69%	445	28%	27	2%	25	2%
Slab	1585	930	59%	622	39%	20	1%	13	1%
Box beam	93	47	51%	44	47%	2	2%	0	0%
“Springwerk”	53	31	58%	21	40%	0	0%	1	2%
Voided slab	35	15	43%	20	57%	0	0%	0	0%
Other	129	69	53%	56	43%	3	2%	1	1%
Total	3513	2213	63%	1208	34%	52	1%	40	1%

NB: “GR” stands for governmental road

It can be concluded that the average object length, object type length, and number of spans differ considerably depending on the type of viaduct, whereas the average object width and crossing angle[8] are rather similar for all types of viaducts. Considering the location of the viaducts, it can be concluded that for all types of viaduct, except for voided slab viaducts, and for all viaducts in general, more than 50% is located in a governmental road. Only somewhere between 30% to 45% crosses a governmental road, depending on the type of viaduct, except for the voided slab viaducts which cross a governmental road in 57% of the cases. Finally, some of the viaducts are neither located in, nor are crossing a governmental road ('Non GR'), or it is not specified. This only accounts for around 1% in both cases.

Besides the calculation of the average dimensional characteristics, an analysis of the frequency distribution of the same characteristics has been made in order to gain more insight in the most common values for each characteristic per type of viaduct.

2.6.6. Current Life-Cycle Aspects of (Existing) Concrete Viaducts

The designs of current viaducts, especially the relatively old viaducts, are characterised by the traditional, linear life-cycle model. Besides, these viaducts are usually designed for a lifetime of around 80 years, whereas since the introduction of the Eurocodes the typical design lifetime has been increased to a minimum of 100 years. Usually the fact that a viaduct no longer functionally satisfies (i.e. 'locational obsolescence', see page 15) is the reason for not reaching its design lifetime [80]. However, from the perspective of concrete as a material, a design lifetime of 100 years generally is not a problem, since, if designed properly, concrete is a very durable material. The durability of concrete structures is mainly governed by a minimum cement quantity in the concrete mix, a low water/cement ratio, the compaction and strength of the hardened concrete, and the concrete cover which has to be large enough to prevent corrosion of the reinforcement. These factors are important, since over their lifetime structures like concrete viaducts are often exposed to severe weather conditions and influences of de-icing salts [68], as has been discussed in subsection 2.4.2 for example.

The experiences of prefab bridges with regard to durability aspects are generally positive. This mainly results from the high concrete strength, low water/cement ratio and the quality of execution, which is due to the indoor manufacturing of components, repetition of work and high control level. Besides, it has been concluded repeatedly from practical examples of prefab bridges in many countries, especially in Norway, Belgium, and the Netherlands, where climatic conditions are often even worse than in more southern countries, that the costs for maintenance and repair of these bridges are much lower than for cast in-situ bridges, particularly in the case of simply supported bridges with a continuous deck slab [68].

However, there are also drawbacks to the use of prefab bridges, which mainly has to do with maintenance aspects. Considering the costs of maintenance and inspection, it seems that these are higher for prefab than for cast in-situ bridges. This mainly results from the fact that in prefab bridges generally more bearings and transition joints are required, which need regular inspection and maintenance. Here, also the contradiction between the principle of DfD, in which a lot of connections and joints are used, and the wishes from an asset manager, who prefers integral bridges and statically indeterminate decks from a maintenance perspective, becomes apparent [68, 80]. This topic of life-cycle costs is discussed in detail in Chapter 7 by means of the calculation and a comparison of the life-cycle costs of a traditional and a circular alternative for the same concrete viaduct.

[8] Note: the crossing angle is measured in gons [gon] rather than in degrees [°]; 100 gon = 90°

3
Technical Action Points

In this chapter, the technical action points that are required in order to achieve circular bridge construction according to Anastasiades et al. [1] (see Table 2.5) are elaborated upon. First, a division of a circular concrete viaduct into time-related layers is established in section 3.1. Subsequently, a list of key DfD principles specific for circular concrete viaducts is drafted in section 3.2, based on the framework for DfD in the construction industry (see subsection 2.1.3). Finally, general starting points for the design and layout of a standard (circular concrete) viaduct (i.e. standardisation scheme) are established in section 3.3. This standardisation scheme is then used in Chapter 4 to develop the layout and design of the standard viaduct, which in turn is used as the main essential starting point for the development of a demountable solution in Chapter 5.

The aspects “user behaviour and ownership” and “circularity assessment” are not elaborated further upon, since the focus of this research is on (developing) design innovations rather than on policy-related innovations. However, with respect to the aspect “user behaviour and ownership”, it can be noticed that Rijkswaterstaat, as part of the government, is actually setting the example and taking its responsibility by specifically requesting (solutions for) viaducts that incorporate the DfD principles, which can turn out to be a major incentive for achieving circular bridge construction.

3.1. Layers of a Circular Concrete Viaduct

The first technical action point in the plan to achieve circular bridge construction was formulated as to redefine Brand's shearing layers of longevity for bridges (see Table 2.5). Therefore, the different main components of a concrete viaduct have been listed and they have been assigned a layer in a similar way as was done by Brand [37], based on their expected (intended) longevity. This results in a subdivision into five different time-related layers of a viaduct (see Table 3.1). Both this subdivision into layers and the assignment of an expected (intended) longevity for each of the components has been done based on the knowledge gained about these components in section 2.6.

Table 3.1: Redefinition of Brand's shearing layers of longevity specific for concrete viaducts

Layer of viaduct	Expected (intended) longevity	Component
Site	Eternal	N / A
Superstructure	200 years (full service life)	Deck Kerbs
Substructure	200 years (full service life)	Foundation Abutments Wing walls Transition slabs Footing (pile caps) Intermediate piers Capping beams
Skin	20-50 years	Finishing layer Bearings Transition joints
Services	Multiple life-cycles (if in good condition)	Safety barriers Parapets Other

An intended full service lifetime of 200 years has been assumed as a realistic and desirable lifetime for the concept of a circular viaduct to be successfully reused in a number of different life-cycles. This assumption has been discussed in more detail by means of a life-cycle cost analysis (LCCA) in Chapter 7. Furthermore, the expected longevity of the components in the layer "Skin" has been based on the average service life of those components. Finally, no specific expected (intended) longevity has been assigned to the layer "Services" which includes safety provisions, but also all other services such as lighting, wind shields, sound barriers, electricity, etc. Instead, it was reasoned that these components could be reused in multiple life-cycles as long as they are in a good condition. A graphical representation of the redefinition of Brand's shearing layers of longevity for concrete viaducts is shown in Figure 3.1.

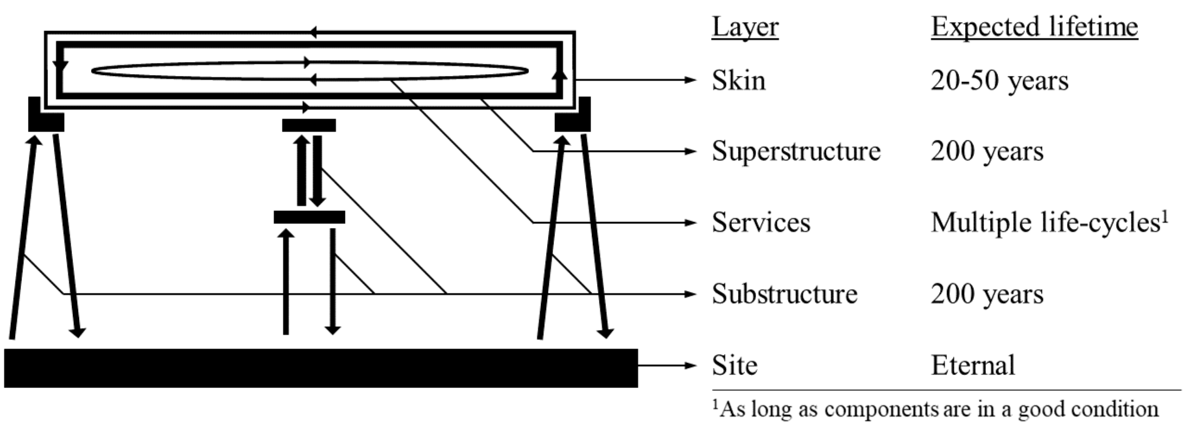


Figure 3.1: Redefinition of Brand's shearing layers of longevity for concrete viaducts

A definition of time-related shearing layers of a concrete viaduct as has been proposed here can assist in gaining insight in which components to keep separate from each other, i.e. to not connect together in such a way that it becomes practically impossible to disassemble these components from each other in the future. Furthermore, it highlights the intentions of the circular principle as it helps to understand that components in different layers could (should) be designed for a different longevity. Conclusively, keeping this idea of different layers in mind while designing a circular viaduct, it ultimately should result in a better design and prevent future problems in the process.

3.2. Key DfD Principles for Circular Concrete Viaducts

The second technical action point in the plan to achieve circular bridge construction was defined as to adjust the DfD (and DfAD) principles to the specific needs and requirements of bridges, or, in this case, concrete viaducts (see Table 2.5). In subsection 2.1.3, the framework as defined by Crowther [7] was addressed, in which a list of 27 key principles for DfD for buildings was proposed (see Table 2.3).

The list of key DfD principles is only one of the four themes and principles within this framework. However, the theme regarding the theory of layers has already been covered in section 3.1. The other two themes, a holistic model of sustainable construction and recycling hierarchies, are not elaborated upon here as the focus of this research is on developing design innovations. However, regarding the theme of recycling hierarchies, it is clear that the focus will mainly be on the strategy ‘relocation or reuse of whole building (i.e. viaduct)’ and partially on ‘reuse of components into new buildings (i.e. viaducts)’ (see Figure 2.4). Regarding the theme of a holistic model of sustainable construction, it could for example be decided to perform a comparative LCA between a traditional (non-demountable) and a circular (demountable) concrete viaduct, or to perform a MCA (multi-cycle assessment), as suggested by Anastasiades et al. [1].

Besides the list of 27 key principles proposed by Crowther [7], another list of 38 Critical Success Factors (CSFs), i.e. key principles, for the effective material recovery through DfD was proposed by Akinade et al. [38] (see Table 2.4). Mainly based on these two lists, supplemented and/or supported by a couple of other references and personal ideas, an adjusted list of 28 key DfD principles specific for concrete viaducts has been compiled, which is shown in Table 3.2.

It is important to mention that in compiling this list, only the so-called ‘design principles’ have been considered, and therefore no principles related to policy (regulations, incentives, etc.) have been taken into account. That is the reason why for example the policy-related principle ‘*Preparation of a deconstruction plan*’, which is thought to be very important within the concept of circular construction in general, has not been taken into account. This decision to only consider the design-related principles has also been made because of the focus of this research on (developing) design innovations rather than on policy-related innovations.

In compiling the list of key DfD principles specific for concrete viaducts, it has been tried to formulate each principle either very specific or rather global but self-explanatory. However, it might be useful to emphasise that the term ‘handling’ in principle 24 (‘*Make components and materials of a size that suits the intended means of handling*’) not only relates to (de)construction activities, but also includes activities such as production, transportation and storage.

Finally, it is repeated that the list of key DfD principles can both be used as a tool to evaluate the circularity of existing concrete viaducts, and as a tool to design new concrete viaducts for deconstruction, as was highlighted by Crowther [29] (see page 24).

Table 3.2: Key DfD principles specific for concrete viaducts

	Principle	Source
1.	Specify removable, durable, mechanical instead of chemical and/or cast in-situ, rigid, connections	[5, 7, 38]
2.	Design components (foundations, abutments, piers, etc.) to be retractable from ground	[38]
3.	Specify materials and components with long life span	[38]
4.	Design joints and connectors to withstand repeated use	[7, 38]
5.	Minimise the number of components	[38]
6.	Minimise the number of different types of components	[5, 7, 38]
7.	Minimise the number of fasteners or connectors	[5, 7, 38]
8.	Minimise the number of different types of fasteners or connectors	[5, 7, 38]
9.	Minimise the number of different types of material	[5, 7]
10.	Avoid toxic and hazardous materials	[5, 7, 38]
11.	Avoid specifying secondary finishes to materials or components	[7, 38]
12.	Specify materials that can be reused or recycled	[7, 38]
13.	Provide standard and permanent identification of (types of) component and materials	[5, 7]
14.	Permanently identify points of disassembly	[5, 7]
15.	Using of interchangeable components	[38]
16.	Design for prefabrication of components	[7, 38]
17.	Design for the repetition of similar components (i.e. design for mass production)	[7, 38]
18.	Separate the main load-bearing components from cladding and finishing elements	[5, 7, 38]
19.	Standardising viaduct form and layout	[5, 38, 41]
20.	Use a standard structural grid	[7, 38]
21.	Structure components according to their service life and the expected time till obsolescence to allow for parallel (dis)assembly	[5, 7, 38]
22.	Provide access to all parts and components	[5, 7]
23.	Provide realistic tolerances to allow for manoeuvring during (dis)assembly	[7]
24.	Make components and materials of a size that suits the intended means of handling	[5, 7]
25.	Reduce the number of wearing parts that may need to be serviced	[5]
26.	Use sacrificial materials and components where wear is unavoidable and allow for their easy disassembly from the whole	[5]
27.	Design to avoid permanent deformations and damage during (dis)assembly, use, and storing	[5]
28.	Minimise the use of cast in-situ concrete	

3.3. Standardisation Scheme for Circular Concrete Viaducts

The third and final technical action point in the plan to achieve circular bridge construction is to develop a complimentary standardisation scheme without compromising on architectural freedom (see Table 2.5). It was stated that such a standardisation scheme has to be developed on the meso-scale, setting certain boundaries within which all bridges (or, in this case, all concrete viaducts) have to be designed. Therefore, it was decided to draw up a draft version of a *standard*[1] concrete viaduct containing extensive starting points for the layout (e.g. dimensions) and the design (e.g. (structural) properties and characteristics) to which all other circular concrete viaduct designs should comply. In the following subsections, extensive starting points for this standard viaduct which are largely based on the analysis of (existing) viaducts in the Netherlands (see section 2.6) is explained.

First, some general starting points for the design (e.g. (structural) properties and characteristics) of the standard viaduct are explained, which can be seen as the main boundary conditions for the design. Next, a number of general parameters and variables are discussed. Finally, relevant starting points for each component are listed. It is important to realise that it is not the aim to quantify the relevant parameters and variables here, but instead to qualify them. At a later stage, when determining the actual layout (e.g. dimensions) of the standard viaduct, these parameters and variables should be quantified in order to design demountable solutions that are applicable for all viaducts that fall within the boundaries of the values of these parameters and variables (see Chapter 4).

3.3.1. General Starting Points for the Design of the Standard Viaduct

In order to come to a standardised design of a circular viaduct, it is thought to be important to establish some general starting points, or boundary conditions, for some important (structural) properties and characteristics.

Firstly, considering the (structural) properties of the standard viaduct, it is believed that a simply supported viaduct is most suitable. This is because in that way, elements and components remain relatively 'separate' from each other and can therefore easily be parallelly (dis)assembled, whereas in other structural systems elements and components become more intertwined. The main challenge here, therefore, is to design (or apply) joints that facilitate this intended structural principal. Also, this highlights the notion of keeping different layers of the viaduct (i.e. 'superstructure' and 'skin') separate from each other.

Secondly, considering the main (material-related) characteristic, it is rather clear that the viaduct should be built up from prefabricated concrete elements, and that the use of cast-in situ concrete should be limited to a bare minimum. For the superstructure, this means that prefab girders/beams should be used. The choice has been made to use prefab box beams, mainly because of the advantage that the box beams immediately form a deck without the need of in-situ concrete. Besides, the use of prefabricated elements means that the kerbs should somehow also be prefabricated. Therefore, a solution with integrated kerbs on an edge beam is suggested, as can be seen in Figure 3.2.

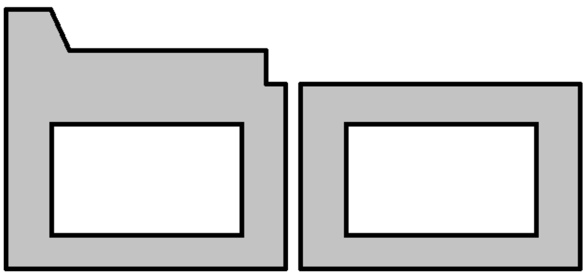


Figure 3.2: Schematic impression of prefab box (edge) beam with integrated kerb

[1] Reminder: 'Standard' in this research' context is understood as 'most common in the Netherlands' (see section 1.4)

For the substructure in turn, it means that the elements and components that make up the substructure should be prefabricated as well. Although this is not very common in current practice, it is possible as has been proven by the design of the completely prefabricated viaduct “Groenedijk” [71] in Rotterdam (see section 2.6 and Figure 2.35). Here, the main challenge is to find a solution to connect the several prefab components in such a way that they can be (dis)assembled easily, and besides to keep the components to a size that suits the intended means of handling. Furthermore, related to the foundation, it is assumed that the standard viaduct is founded on a pile foundation. In an ideal circular viaduct concept, also the foundation (piles) should be able to be reused, or at least to be retractable from the ground during disassembly of the viaduct. Usually, the foundation piles are rigidly connected with the (abutment) footings (Dutch: ‘*sloof/poer*’) by means of protruding reinforcement. In this case, however, that is undesirable and a big challenge arises here on how to connect these components in a demountable way.

Ultimately, the original idea of a standard viaduct, consisting of a number of compatible components, analogous to a typical IKEA building kit, as was described in section 1.2, becomes apparent in these general starting points. Besides, also the link to many of the key DfD principles listed in Table 3.2 becomes apparent in the above mentioned general starting points.

3.3.2. General Parameters and Variables for the Layout of the Standard Viaduct

One of the most important starting points for the concept of a circular viaduct to be successful is believed to be that the span length should be the leading parameter for all other layout choices, and should be based on standardised lengths of the box beams. The remaining variables that define the layout of the viaduct should follow from the standardised (available) span that can be realised, instead of from the underpassing road profile as is common in current practice. In a similar way, this also should hold for the deck width which should follow from a standardised width of the box beams. The main parameters and variables, which are dependent on those parameters, are listed in Table 3.3 and Table 3.4 respectively.

Table 3.3: Parameters for determination of general layout of standard viaduct

	Parameter	Value	Remarks
1.	Box beam length	…	*Standardised lengths*
2.	Box beam width	…	*Standardised widths*
3.	Crossing angle	…	
4.	Layout deck	…	*Number of lanes, type of traffic, etc.*
5.	Number of spans	…	
6.	Type of abutment	…	*Abutments and/or bank seats*
7.	Clearance height	…	*Top asphalt underpass - bottom deck*
8.	Position of viaduct	…	*In/over (governmental) road*

Table 3.4: Variables for determination of general layout of standard viaduct

	Variable (dependent on parameter(s)	[...])	Value	Remarks
a.	Span length (grid size)	[1]	…	
b.	Deck width	[2, 3]	…	
c.	Distance between embankment toes/ abutment walls	[1,4-6]	…	
d.	Slope(s) of embankment	[1,4-6]	…	
e.	Underpassing road profile	[1,4-6]	…	*Number of lanes, type of traffic, free space, etc.*

3.3.3. Starting Points per Component of the Standard Viaduct

The standard viaduct can be divided into different elements which consist of several components. This division is closely related to the division into time-dependant shearing layers (see Table 3.5). All relevant starting points for each of the components that make up these elements should be laid down. In the following subsections, these (choices between) starting points are listed, as well as some points of attention with regards to the structural design, which in the end define the main layout and design of the standard viaduct.

Table 3.5: Division of standard viaduct into elements and components

Layer of viaduct	Element	Component
Site	Location (building site)	N / A
Superstructure	Deck	Box beams Kerbs
Substructure	Abutment A	Foundation Footing Wing walls Transition slabs
	Intermediate support(s)	Foundation Footing Intermediate piers Capping beam
	Abutment B	*See components "Abutment A"*
Skin	Finishing provisions	Finishing layer Bearings Transition joints at abutments Transition joints at intermediate support(s)
Services	Safety provisions and other	Safety barriers Parapets Other

Superstructure – Deck

Box beams

In order to facilitate easy (dis)assembly and to keep the box beams separable from each other, it is proposed to not apply a structural screed (Dutch: '*deklaag*'). Furthermore, the cross-section that is applied depends on the span length. Finally, it can be chosen to apply transversal unbonded post-tensioning in the box beams, or to leave it out. This choice should be made by the designer, and the impact on the overall structural behaviour should be considered carefully.

Kerbs

As was proposed before, the kerbs could (should) be integrated on an edge beam (see Figure 3.2). However, it is undesirable for the kerbs to have a structural contribution, since this would complicate the structural behaviour. In order to prevent this, it is suggested to provide the kerbs with right-angled dilatations over the full width and height along the entire length in order to prevent the kerb from contributing to the stiffness of the edge beam. Furthermore, it is advised to execute the kerbs as much as possible in accordance with the standard details of Rijkswaterstaat (see documentation in RTD 1010 [XIb] and, for example, Figure 2.29).

Substructure – Abutment A

Foundation

For the reason mentioned earlier, namely that the foundation (piles) should be able to be reused, or at least to be retractable from the ground, it is proposed to use steel pipe piles. The structural properties, geometry, and characteristics of the foundation (piles) should be determined at a later stage in accordance with the more general parameters and variables for the layout of the standard viaduct.

Point of attention:

- The connection to the (abutment) footings.

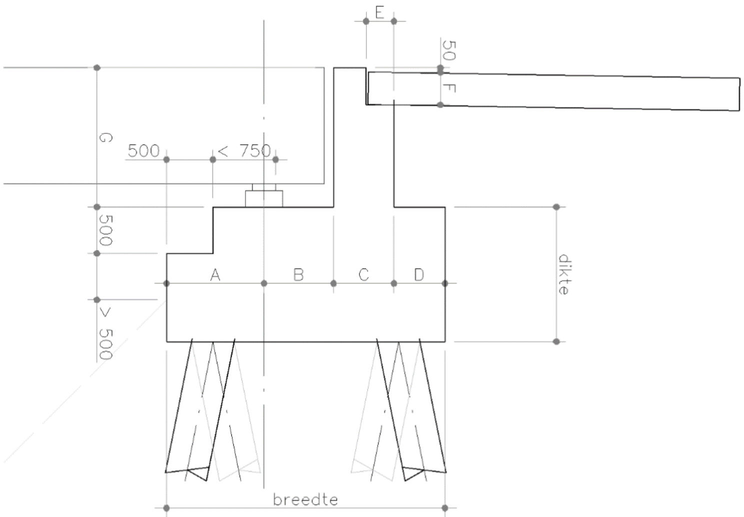


Figure 3.3: Traditional cross-section of an abutment footing (bank seat type)

Abutment footing
For the standard viaduct, it is assumed that a prefabricated bank seat (Dutch: '*hooggefundeerd landhoofd*') is applied with a constant, standard, cross-section (see for example Figure 3.3).

Points of attention:

- The connection to the foundation piles.
- Transport and lifting capacity with regard to dimensions and weight of (parts of) the abutment footing. As a result, it may prove necessary for the abutment footing to be constructed of a number of elements that can be connected to form one component.

Wing walls
Wing walls should be provided at both ends of the abutment footing to contain the embankment. It is proposed to apply a constant and simple cross-section, as well as common, average, dimensions.

Transition slabs
The abutments (bank seats) should be provided with transition slabs over the entire width of the abutment footing. It is proposed to keep the execution in accordance with RTD 1011 (see [XIb]).

Substructure – Intermediate support(s)

Foundation
See "Foundation" under subsection "Substructure – Abutment A" (page 82).

Intermediate support footing
It is suggested that a prefabricated (intermediate support) footing is applied with a constant, rectangular cross-section.

Points of attention:

- The connection to the foundation piles.
- The connection to the intermediate piers.
- Soil load on footing (> 0,5 m).
- Transport and lifting capacity with regard to dimensions and weight of (parts of) the footing. As a result, it may prove necessary for the footing to be constructed of a number of elements that can be connected to form one component.

Intermediate piers
It is proposed that prefabricated intermediate piers are applied with a constant cross-section.

Points of attention:

- The connection to the footing.
- The connection to the capping beam.
- Transport and lifting capacity with regard to dimensions and weight of (parts of) the piers. As a result, it may prove necessary for the piers to be constructed of a number of elements that can be connected to form one component, for example as is shown in Figures 2.43a and 2.43c.

Capping beam
It is suggested that a prefabricated capping beam is applied with a constant, rectangular cross-section.

Points of attention:

- The connection to the intermediate piers.
- Transport and lifting capacity with regard to dimensions and weight of (parts of) the capping beam. As a result, it may prove necessary for the capping beam to be constructed of a number of elements that can be connected to form one component.

Substructure – Abutment B

See the different components under subsection "Substructure – Abutment A" (page 82).

Skin – Finishing provisions

Finishing layer
In the end, some sort of finishing (asphaltic) layer will have to be applied, both as protection against wearing of the box beams and for driving comfort reasons. However, this layer should be applied in such a way that it can easily be removed without severely damaging the box beams.

Point of attention:

- The application of a finishing layer on top of the box beams that can be easily removed without severely damaging the box beams.

Bearings
It is proposed to support the box beams with steel plate reinforced bearings with common and constant dimensions. The bearings should both be able to transfer the forces from the superstructure to the substructure and to provide sufficient translational and rotational movement.

Transition joints at abutments
Transition joints should be applied at the transition from the abutments to the bridge deck (i.e. box beams). It is suggested to keep the execution in accordance with RTD 1007-1 (see [XIa]). More specifically, a steel/rubber sinus joint profile with detailing in accordance with standard detail RWS-VOEG-02 is proposed (see RTD 1010 [XIb]). The use of this type of joint ensures that the deck is not anchored in the abutments, resulting in a statically determined system.

Transition joints at intermediate supports
Transition joints should also be applied at intermediate supports. The application of an incorporated deck rotation joint (Dutch: '*buigslappe voeg*') is suggested, executed in accordance with RTD 1023 (see [XIb]), and with detailing in accordance with standard detail RWS-VOEG-01 (see RTD 1010 [XIb]). The use of this type of joint provides rotational freedom between the different spans, resulting in a statically determined system.

Services – Safety provisions and other

Safety barriers
Safety barriers should always be provided. Detailing in accordance with standard details RWS-SCHAMP-01 and RWS-SCHAMP-03 (see RTD 1010 [XIb]) is proposed. Besides, the safety barriers should be connected to the superstructure in such a way that they can be easily (dis)assembled and reused.

Parapets
If a pedestrian and/or cyclist lane is provided on the viaduct, parapets should also be applied. In that case, detailing in accordance with standard details RWS-LEUN-01 and RWS-LEUN-02 (see RTD 1010 [XIb]) is suggested. Similar to the safety barriers, parapets too should be connected to the superstructure in such a way that they can be easily (dis)assembled and reused.

Other
It needs to be determined by the designer if and how other provisions such as lighting, wind shields, sound barriers, electricity, etc. are incorporated in the layout and design of the standard viaduct. No specific starting points are listed here. However, it is emphasised once again that all of these provisions should be separable from, and not be intertwined with, elements and components in other layers.

4

Standard Viaduct

In this chapter, the elaboration of the technical action points which was described in Chapter 3 is firstly used in section 4.1 in order to identify key bottlenecks in current concrete viaduct designs in the Netherlands with regard to demountability issues. Subsequently, after the exact layout and design of the (adopted) standard viaduct, based on the starting points that were established in section 3.3, is explained in section 4.2, the modelling of this standard viaduct is explained in section 4.3. Next, the calculation of the loads on the viaduct as well as the applied load combinations are explained in section 4.4. Finally, verification and validation of the standard viaduct is done in section 4.5.

The result of this chapter is the establishment of a (proposal for a) standard layout and design of a circular concrete viaduct in the Netherlands. Therefore, this chapter should be considered as the first part of the main result of this research by addressing the third technical action point of the plan to achieve circular bridge construction (see Table 2.5), namely the development of a standardisation scheme. This step is required in order to move on to the second part of the main result of this research, which is the development of a (concept) demountable solution for one of the main bottlenecks in current viaduct design which prevents it from being circular (i.e. demountable), and which is addressed in Chapter 5.

4.1. Identification of Key Bottlenecks in Current Viaduct Designs

Throughout the process of collecting and analysing literature, as well as becoming familiar with the current design practice of concrete viaducts in general, and having discussions with professionals (e.g. engineers at Lievense), it has become evident that the key bottlenecks in current concrete viaduct designs with regard to demountability issues are found at the locations where different components are being connected, i.e. the connections between components of the same or different layers of a viaduct that have been identified in Table 3.1. Therefore, a 'top-down' analysis has been done to identify all relevant bottlenecks in current concrete viaduct designs, from which the results are shown in Table 4.1.

Table 4.1: Relevant bottlenecks in current viaduct designs identified by 'top-down' analysis (printed in bold are the key bottlenecks identified for further development in this research)

Bottleneck	Layer(s) of viaduct	Component 1	-	Component 2
1.	Superstructure	Box beams	-	Kerbs
2.	Superstructure - Skin	Box beams	-	Finishing layer
3.	Superstructure - Skin	Box beams	-	Transition joints
4.	Substructure	Abutment footing	-	Transition slabs
5.	Substructure	Abutment footing	-	Wingwalls
6.	Substructure	**Abutment footing**	**-**	**Foundation**
7.	Substructure	Capping beam	-	Intermediate piers
8.	Substructure	Intermediate piers	-	Footing
9.	Substructure	**Footing**	**-**	**Foundation**

First of all, it can be concluded that the connection between superstructure (i.e. box beams) and substructure (i.e. abutment footing and capping beam) is not a main bottleneck, as the connection, which is made by means of bearings, is rather simple and in fact already practically demountable.

Furthermore, bottleneck 1 (see Table 4.1) has already been shortly addressed in subsection 3.3.1, and a solution with integrated kerbs on an edge beam was already proposed (see Figure 3.2). Besides, bottlenecks 2 and 3 are thought to require a material-related innovation instead of a design innovation, which is the topic in this research. Finally, bottlenecks 4, 5, 7 and 8 are thought to be able to solve with rather 'simple' solutions. This is motivated by the design of the completely prefabricated viaduct "Groenedijk" [71]. Even though the connections in this design are not demountable, it is believed that demountable connections here could be realised with existing methods, such as the post-tensioned bars system (see subsection 2.6.2), or by means of moment resisting beam-to-beam connections (see subsection 2.3.3). Besides, it is believed that the complicating factor with regard to these bottlenecks lies more with 'handling'-related issues, mainly with regards to transportation, storage, and (dis)assembly of (parts of) components because of their size and weight (see subsection 5.5.1).

Finally, it has been concluded, supported by engineers at Lievense, that the main challenge with regard to developing a demountable solution is found at the connection between the (abutment) footing and the foundation (from now on referred to as 'footing to foundation connection', or by means of the abbreviation 'F2F'). Therefore, it has been decided to focus on this connection within this research in order to develop a (concept) demountable solution.

4.2. Layout and Design of Standard Viaduct

Before a (concept) demountable solution can be developed, first the exact layout and design of the standard viaduct for which the solution should be applicable has to be established. Once this has been established, the critical cross-sectional forces at the interface between footing and foundation can be calculated. Therefore, the different parameters, variables, and starting points which were mentioned in the standardisation scheme in section 3.3, and which were undetermined yet, are determined in the following subsections[1].

4.2.1. Specification of General Parameters and Variables

In subsection 3.3.2, it was argued that the box beam length should be the leading parameter for all other layout choices, based on different standardised lengths of box beams instead of on the underpassing road profile as is common in current practice. However, in order to determine what these standardised lengths should be, a (first) *initial length* should be calculated based on the common way in current practice, i.e. based on the profile of an underpassing road. Therefore, a fictitious road profile has been assumed consisting of two times 2 lanes plus 2 emergency lanes, which is believed to be representative for the majority of viaducts crossing the Dutch highways (i.e. the position of the standard viaduct is chosen to be *over* a governmental road, see Table 4.3). Besides, the possibility to expand to two times 3 lanes has been taken into account in order to facilitate any future changes in the road profile under the viaduct. This has been accommodated by keeping the slope of the embankment variable. Examples of such underpassing road profiles are shown in Figure 4.1 and Figure 2.22a respectively. Furthermore, several intermediate distances have been adopted from general values used in the design of road profiles, whereas for some other dimensions a representative (conservative) value has been chosen. These intermediate distances and other dimensions are listed in Table 4.2. All together, this results in a required beam length of 27,70 m, which has been adopted as being the (first) standardised length of the box beams. The Excel-spreadsheet which was used to calculate this length has been added in Appendix E.

Table 4.2: Parameter values used for calculation of required (standardised) box beam length

Widths:			**Heights:**		
Intermediate support	3,00	m	Clearance	4,60	m
Safety barrier (rail)	0,80	m	Tolerance	0,10	m
Intermediate distance	1,50	m			
Lane	3,50	m	**Abutment:**		
Road marking (line)	0,20	m	A[1]	1250	mm
Emergency lane	3,50	m	B*[2]	500	mm
			[1]See Figure 3.3; max(500 + 750)		
Intermediate support:			[2]See Figure 3.3; distance centre of bearing to end of beam		
Spacing (min. 100 mm)	100	mm			

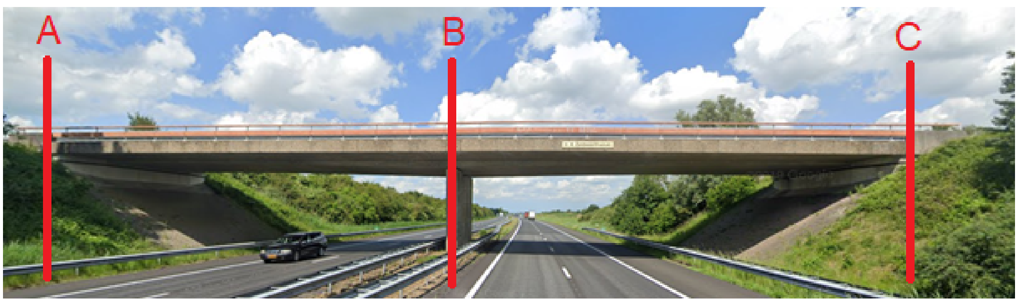
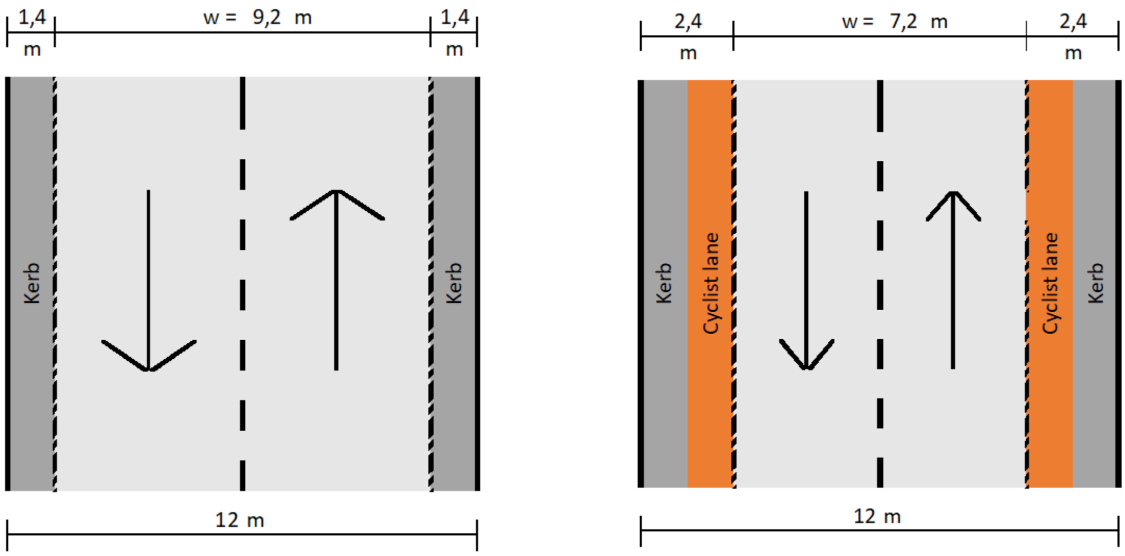


Figure 4.1: Example of a (*the*) standard viaduct over Dutch highway A32 at Idaerd

[1]The general starting points for the design of the standard viaduct that were established in subsection 3.3.1 are assumed to be known to the reader.



(a) Conservative layout of the deck used for calculation

(b) Possible layout of the deck including cyclist lanes

Figure 4.2: Schematic impression of different layouts of the deck

The final values of the general parameters and variables are shown in Tables 4.3 and 4.4. The width of the deck has been chosen to be 12 m. This is based on the standard box beam width and the layout of the deck. This is illustrated in Figure 4.2, in which a width of 1,40 m for the kerbs has been adopted. Figure 4.2a shows the (most conservative) layout of the deck, which is used for the calculation, whereas Figure 4.2b shows a possible layout including cyclist lanes. Furthermore, the crossing angle has been set to 90° in order to keep the layout simple and feasible. The values of other general parameters and variables for the layout of the standard viaduct either follow logically from the (calculation of the) box beam length, are explained by means of a short remark, or have already been explained in the foregoing. An impression of how the general layout of the standard viaduct would more or less look like according to these established parameters and variables is shown in Figure 4.1.

Table 4.3: Final parameter values for determination of general layout of standard viaduct

	Parameter	**Value**	**Remarks**
1.	Box beam length	27,70 m	
2.	Box beam width	1480 (c.t.c. 1500) mm	*Standard width of SKK beams + 20 mm spacing (see [76])*
3.	Crossing angle	90°	
4.	Layout deck	2x 1 lane	*Possibility for a cyclist lane at both sides (see Figure 4.2b)*
5.	Number of spans	2	*A symmetrical layout (symmetry plane at I.S) is assumed*
6.	Type of abutment	Bank seats	
7.	Clearance height	4,70 m	*Top asphalt underpass - bottom deck*
8.	Position of viaduct	Over GR	*GR = governmental road*

Table 4.4: Final variables values for determination of general layout of standard viaduct

	Variable (dependent on parameter(s)	**[...])**	**Value**	**Remarks**
a.	Span length (grid size)	[1,3]	27,25 m	*See Appendix E*
b.	Deck width	[2, 4]	12,00 m	*See Figure 4.2*
c.	Distance between embankment toes/ abutment walls	[1,5-7]	37,90 m (42,60 m)	*See Appendix E*
d.	Slope of embankment	[1,5-7]	1:1,77 (1:1)	*See Appendix E*
e.	Underpassing road profile	[1,5-7]	2x 2 lanes (2x 3 lanes)	*Plus an emergency lane at both sides* (see Appendix E)

4.2.2. Specification of Starting Points per Component

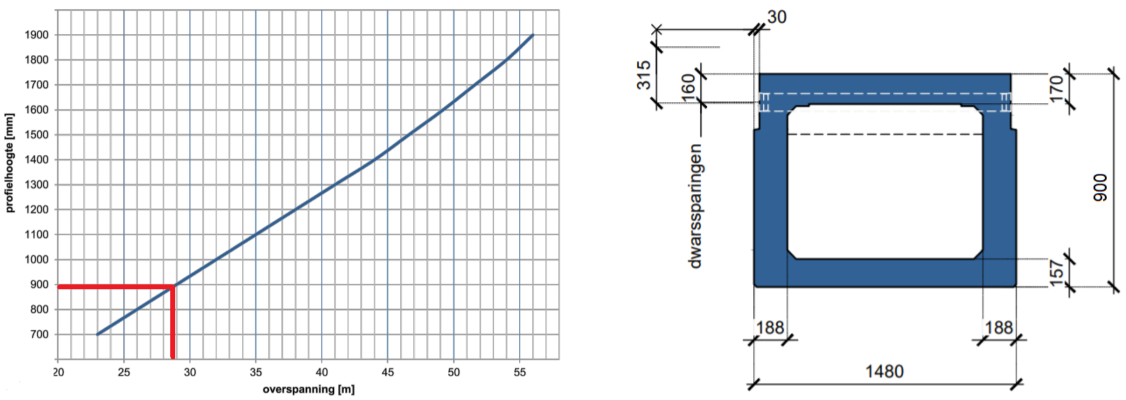
In subsection 3.3.3, the relevant starting points for each of the components that make up the standard viaduct were laid down. However, some of (the choices between) these starting points were left undetermined. Therefore, those (choices between) starting points are briefly explained in the following subsections in addition to the starting points that were already established in subsection 3.3.3.

Superstructure – Deck

Box beams

Based on the determined span length, an estimation of the required cross-section of the box beams can be made. For this purpose, the documentation of box beams produced by Spanbeton (see [76]) has been used. This results in the estimation of the required cross-section SKK 900 (see Figure 4.3a), of which the dimensions are shown in Figure 4.3b.

Furthermore, it is chosen to examine the impact of both applying transversal unbonded post-tensioning in the box beams, and of leaving it out, and thus keeping this a free variable.



(a) Estimation of cross-section based on span length [76] (adapted)

(b) Dimensions of SKK900 profile [76] (adapted)

Figure 4.3: Estimation of required box beam cross-section and dimensions

Kerbs

The width of the kerbs has been chosen to be 1,40 m in accordance with the standard details of Rijkswaterstaat (RTD 1010). The assumed (average) height has been chosen to be 0,25 m.

Substructure – Abutment A

Foundation

Since it is practically impossible to define a standard soil profile, it has been decided to base the structural properties, geometry, and characteristics of the foundation (piles) on both a fictitious soil profile and on common, average, values. The fictitious soil profile has been assumed to consist of two layers: first a 16 m thick clay layer (measured from the bottom of the I.S footing into the ground), and subsequently a sand layer which can be used to found the viaduct on. The pile tip is assumed to penetrate 4 m into the sand layer. A cross-section of this soil profile, including the layout and position of the standard viaduct, is shown in Figure 4.4. Besides, the vertical spring stiffness of the piles at the pile tip is assumed to be 100 MN/m, which is argued by engineers at Lievense to be a representative (conservative) value for this research purpose. Furthermore, a steel pipe pile circular cross-section Ø508 mm and thickness 12,5 mm (CHS 508x12.5) has been adopted for similar reasons.

Finally, the pile plan under the abutment needs to be determined. Again, this has been based on a common layout of a pile plan. As a result, it is decided to install the piles with a slope of 10:1 in a grid of 2,0 x 1,55 m, with the first row sloping forward and the second row alternately backwards and forwards in a symmetrical pattern in order to form a stable pile plan. For cross-sectional and isometric views, see subsection 4.3.3.

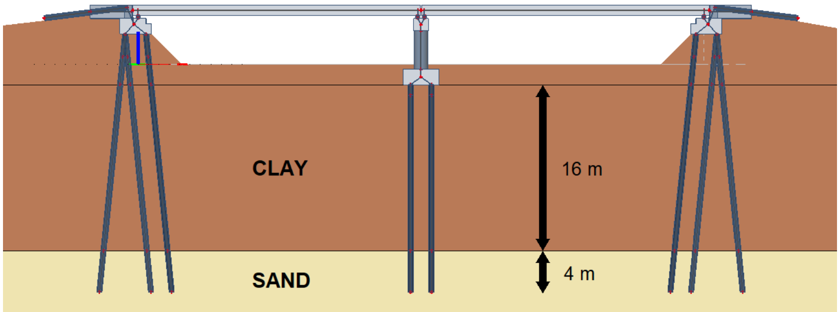


Figure 4.4: Fictitious soil profile for determination of foundation parameters

Wing walls
The length of the wing walls has been chosen to be 4,0 m (half the length of the transition slabs).

Transition slabs
The length of the transition slabs has been chosen to be 8,0 m in accordance with RTD 1011. Besides, it is chosen to apply a slope of 8:1.

Substructure – Intermediate support(s)

Foundation
The same structural properties, geometry, and characteristics of the foundation of the abutments applies to the foundation of the intermediate support. However, a different pile plan layout is used. All piles under the intermediate support footing are installed vertically (no slope) in the same grid of 2,0 x 1,55 m.

Regarding the layout of the intermediate support, it has been chosen to use four intermediate piers with a circular cross-section (Ø1300 mm) and a centre-to-centre distance of 3,5 m.

Furthermore, the layout of the entire viaduct is chosen to be symmetrical. This implies that a vertical symmetry plane exists at the intermediate support.

Substructure – Abutment B

See the structural properties, geometry, and characteristics of the different components under subsection "Substructure – Abutment A" (page 91).

Skin – Finishing provisions

Finishing layer
Since it was identified that some sort of finishing layer will have to be applied, it is decided to assume a 140 mm thick asphalt layer on top of the box beams. This value, again, is based on a common thickness used in current viaduct designs.

Bearings
The choice has been made to apply steel plate reinforced, rectangular bearings of dimensions 300x400x100 mm (length x width x thickness). According to engineers at Lievense, this is a common, average, type and geometry of bearing. No further checks for the capacity of these bearings are done.

Other
It is chosen to not take into account other provisions such as lighting, wind shields, sound barriers, electricity, etc. in the layout and design of the standard viaduct.

4.3. Standard Viaduct Model

With the (main) layout parameters and variables established, a model of the standard viaduct can be created in order to perform a structural analysis and, subsequently, to obtain the critical cross-sectional forces at the interface between footing and foundation. It has been chosen to use the finite element software SCIA Engineer 19.1.3030 (student version) for this purpose, since it is the most used software for these type of analyses (by Lievense) and because of its wide range of tools and functionalities. The structure of the standard model has been modelled in a general XYZ-environment by means of 2D and 1D members. Furthermore, a linear elastic analysis has been performed. In the following subsections, several aspects of the model are explained, and finally some 2D and 3D impressions are shown.

4.3.1. Input Parameters and Properties

General material properties

First of all, the general material properties which are applied in the model are explained. Concrete strength class C60/75 is used for the deck (i.e. prefab box beams) which is a typically used concrete strength class for prefab box beams [76]. For the remaining concrete components, cracked concrete (resulting in a reduced E-modulus) of strength class C30/37 is used, which also is a typically used concrete strength in current viaduct design. Furthermore, the steel pipe piles are assumed to be of steel grade S355. In Table 4.5, the most relevant material properties are summarised. For more details, see the separate document "SCIA Engineering Report - Standard Viaduct Model".

Table 4.5: Most relevant material properties used in SCIA model

Material	Type	ρ [kg/m^3]	E-modulus [MPa]	Remarks
Concrete	C30/37*	2500	16.450[1]	*Cracked*
Concrete	C60/75*	0[2]	39.100	*Massless*
Steel	S355	7850	210.000	
Dummy	Dummy	0	1,0e+14	*Infinite stiff, massless material; see subsection 4.3.2*

[1]The cracked E-modulus is assumed to be 50% of the uncracked E-modulus
[2]Self-weight of the deck is manually inputted (see subsection 4.4.1)

Component geometries

The general geometry of each component is given in Table 4.6 and Table 4.7. For more details, see the separate document "SCIA Engineering Report - Standard Viaduct Model".

Table 4.6: General geometry properties of 2D members

Component	Material	Thickness [mm]	Remarks
Deck (box beams)	C60/75*	1000	*Arbitrary thickness (no influence on structural model)*
Transition joint	C30/37*	500	*At intermediate support*
Transition slab	S355	500	*Slabs are assumed to be 1,0 m wide each*

Table 4.7: General geometry properties of 1D members

Component	Material	Width x height [mm]	Remarks
Abutment footing	C30/37*	*See Figure 4.5*	
Bearing	Dummy	300 x 400	*See subsection "Calculation of input parameters – Bearings" (page 95)*
Foundation pile	S355	CHS 508x12.5	
Capping beam	C30/37*	1400 x 1200	
Intermediate pier	C30/37*	Ø1300	
Footing	C30/37*	3500 x 1500	
Dump	C30/37*	300 x 400	*(Dutch: 'opstort')*
Wing wall	C30/37*	500 x 1300	

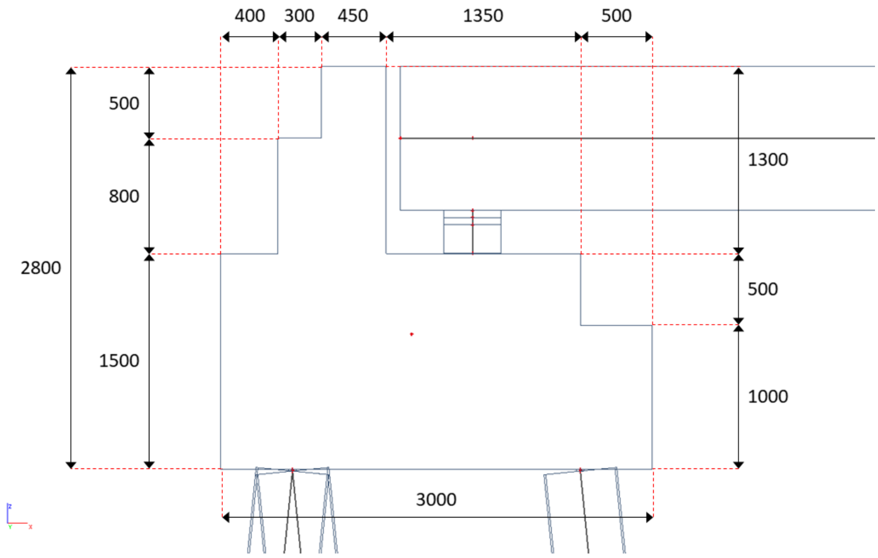


Figure 4.5: Geometry of abutment footing

Connections between elements and components

The connection between superstructure and substructure is realised by means of the bearings. The superstructure and substructure should be able to move (expand/shrink) and rotate relative to each other. This is facilitated in the way the bearings are modelled, which is explained in more detail in subsection “Calculation of input parameters – Bearings” (see page 95).

The transition joint at the intermediate support (i.e. the incorporated deck rotation joint) is hinged to one of the decks. Besides, free translation in vertical direction is modelled in the hinge in order to prevent the transfer of shear forces between the decks of both spans. Furthermore, the transition joint at the abutments is not physically modelled as this joint doesn’t prevent any relevant translations or rotations. This way, a statically determined structure is realised.

The connection between transition slabs and abutment footings is hinged as well. The (free) ends of the transition slabs are supported with fixed hinges.

Obviously, the footing to foundation (F2F) connection depends on the proposed demountable solution at this interface. This is elaborated upon in detail in Chapter 5.

Finally, the remaining connections between elements and components are assumed to be rigid. Within the concept of a completely circular (demountable) viaduct this implies that, for example, rigid, demountable, connections between the components of the intermediate support should be realised. However this is not elaborated upon in this research.

Calculation of input parameters

Deck

The deck is modelled in SCIA Engineer by means of a 2D plate member. However, in order to simulate the correct behaviour of the deck, consisting of box beams, a physical orthotropy has been calculated. This calculation, based on the dimensions of the SKK 900 profile (see Figure 4.3b), has been done by means of an Excel-spreadsheet provided by Lievense, which has been added in Appendix F.

The choice to apply or to leave out transversal unbonded post-tensioning in the box beams is taken into account in the model by means of these orthotropic parameters. The way in which this is taken into account is by means of either assuming uncracked, in case of applying post-tensioning, or cracked, in case of not applying post-tensioning, concrete. Therefore, two different types of orthotropy are inputted in the model, keeping the choice whether to apply or leave out post-tensioning a free variable.

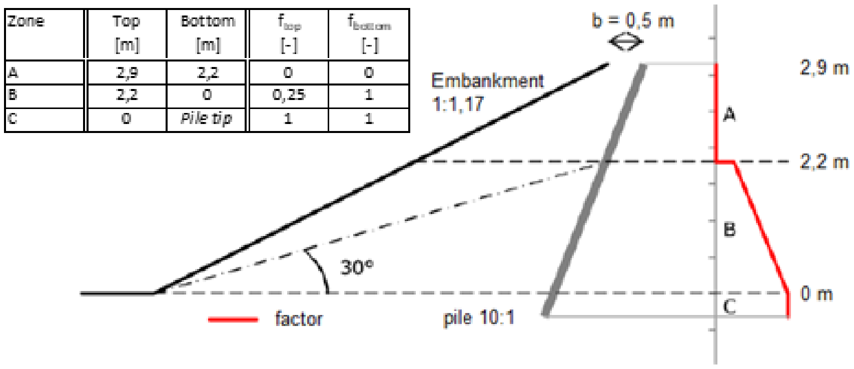


Figure 4.6: Reduction factor on horizontal modulus of subgrade reaction due to embankment

Foundation
Based on the fictitious soil profile (see subsection 4.2.2), the horizontal moduli of subgrade reaction according to Ménard are calculated by means of another Excel-spreadsheet provided by Lievense, which has been added in Appendix G. The clay and sand parameter values that are used are based on common, average, values according to engineers at Lievense.

The first meter of the horizontal modulus of subgrade reaction on the piles is not included in accordance with the ROK [XI]. Furthermore, a reduction is applied to the horizontal modulus of subgrade reaction at the embankments. The reduction is applied by means of a factor, calculated according to Figure 4.6. For zone B, an average reduction factor of 1/2 x (0,25+1,0) = 0,625 is used. The resulting horizontal moduli of subgrade reactions per zone are shown in Table 4.8.

Table 4.8: Resulting horizontal modulus of subgrade reaction on foundation piles under abutments

Zone	Trajectory[1] [m]	Horizontal modulus of subgrade reaction [MN/m^2]
A	+ 2,9 / + 2,2	0
B	+2,2 / 1,9 + 1,9 / 0,0	factor x k_h = 0,625 x 0 = 0 factor x k_h = 0,625 x 4,65 = 2,91
C	0,0 / - 16,0 - 16,0 / - 20,0	4,65 23,41

[1]Vertical distance measured from bottom of embankment (= 0,0 m); see Figure 4.6

Bearings
The bearings are modelled by means of two connected so-called 'dummy rods' (see subsection 4.3.2), each with a length of 50 mm, and thus adding up to a total of 100 mm, which is equal to the assumed thickness. Since the superstructure and substructure should be able to move (expand/shrink) and rotate relative to each other, a hinge with translational springs in X, Y and Z direction is modelled in between both dummy rods. The spring stiffnesses of those springs are calculated by means of another Excel-spreadsheet provided by Lievense, based on their assumed properties. A printout of this spreadsheet has been added in Appendix H.

The two different spring stiffnesses represent the stiffness of one bearing. It is assumed that a bearing consists of eight layers of 8 mm rubber, nine steel plates with a thickness of 3 mm and a 4,5 mm thick rubber cover (8 x 8 mm + 9 x 3 mm + 2 x 4,5 mm = 100 mm). The resulting vertical and horizontal (in two directions) spring stiffnesses are 691 MN/m and 1,48 MN/m respectively.

The top dummy rods are directly connected to the deck. The connection of the bottom dummy rods to the abutment footing, however, is made by means of a concrete dump (Dutch: '*opstort*') of 200 mm thickness in order to obtain the minimum required free space of 300 mm between the bottom of the deck and the top of abutment according to the ROK [XI].

4.3.2. Modelling Manipulations

Dummy rods

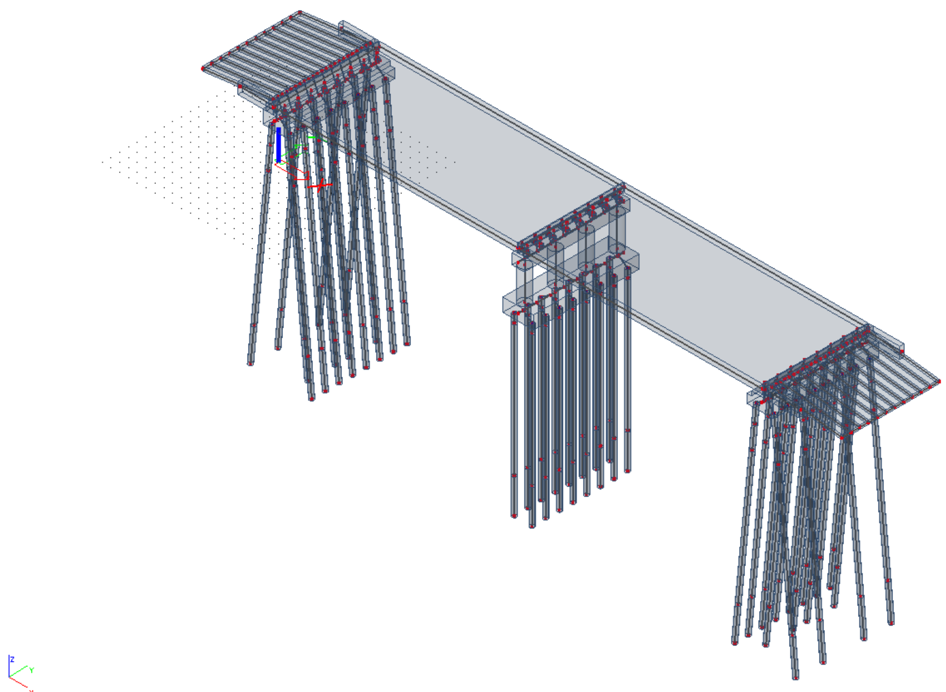
The dummy rods are massless and infinite stiff (E=1,0e+14 MPa) 1D members (see Table 4.5). They are mainly used to connect the different components to each other in the model. Besides, they are used to model the bearings, as was explained in the foregoing paragraph.

F2F interface rods

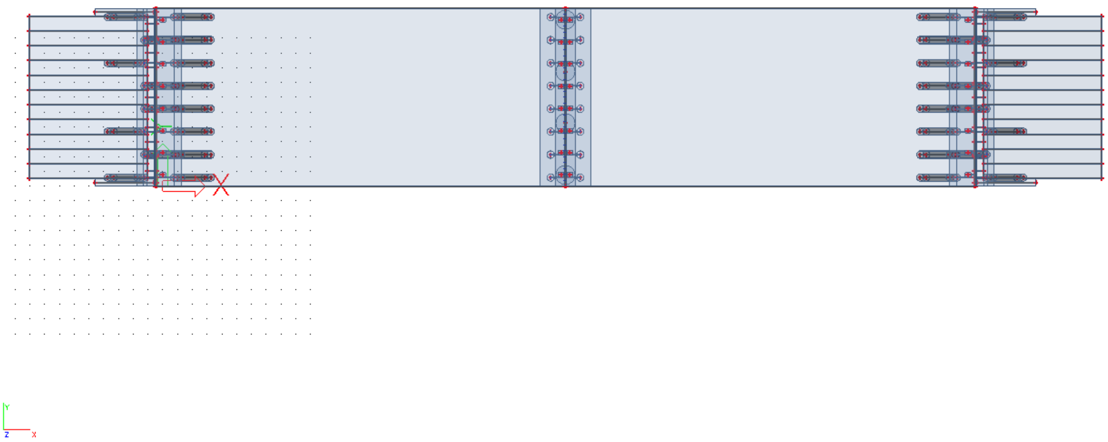
In order to be able to determine the critical cross-sectional forces at the interface between (abutment) footings and the foundation piles, which are required for the development of a demountable F2F connection, 10 mm long so-called 'F2F interface rods' are modelled vertically in between the (abutment) footings and the top of the foundation piles. These F2F interface rods are assigned the same properties as the foundation piles. By means of this modelling manipulation, the extreme 1D internal beam forces (i.e. critical cross-sectional forces) at both interfaces can be easily obtained in SCIA Engineer and can directly be compared since in this way for both interfaces the forces are shown in the global reference system, which would otherwise not have been the case for the inclined foundation piles, which would be shown in a local (rotated) reference system.

4.3.3. Final Model

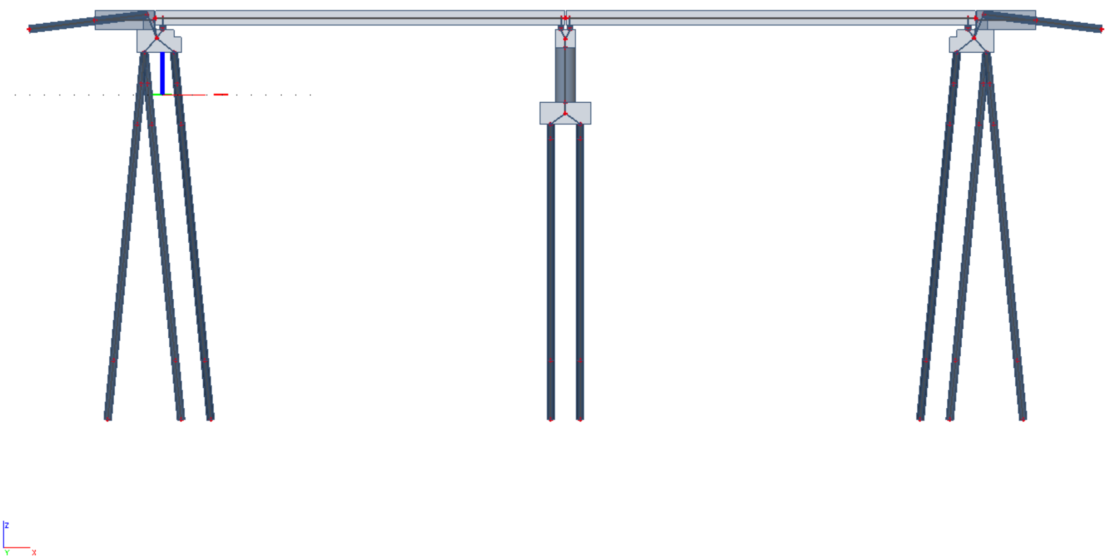
In the foregoing subsections, the main model parameters and variables are explained which determine the layout and design of the standard viaduct. Several cross-sectional and isometric views of the resulting model in SCIA Engineer of the standard viaduct are therefore shown here in Figure 4.7.



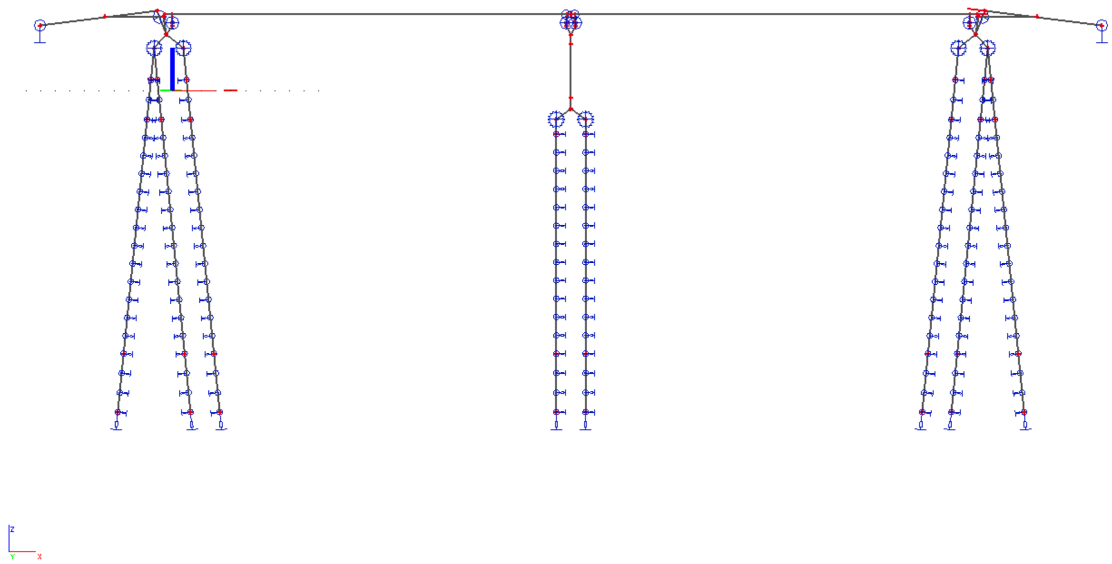
(a) Isometric view



(b) Top view



(c) Side view



(d) Side view (supports and connections)

Figure 4.7: Cross-sectional and isometric views of the final model of the standard viaduct in SCIA Engineer

4.4. Loads and Load Combinations

In order to finish the model, the loads that (can) act on the standard viaduct should first be calculated, and subsequently, relevant load combinations for both the Ultimate Limit State (ULS) and the Serviceability Limit State (SLS) should be determined. The calculation of these loads and the determination of these load combinations are done in a standard way in accordance with the applicable Eurocodes. Finally, the model is checked by means of comparing the sum of loads in the model with the calculated loads.

4.4.1. Calculation of Loads

The main starting points for the calculation of the loads on the viaduct are explained in the following subsections. However, the explanation is limited to a global level since it is a rather standardised, common, procedure which largely comes down to applying the applicable building codes.

Types of loads and load cases

In Table 4.9, an overview is shown of the numbering of the load cases with respect to their type of load as well as some examples of those loads. Note: executional, fatigue, and earthquake loads, (possibly) acting on the standard viaduct, have not been taken into account, since it is either believed that these will not be governing for the critical cross-sectional forces at the footing to foundation interfaces (executional and fatigue loads), or because they are not applicable in the Netherlands (earthquake loads).

Besides, where relevant, a reference period (i.e. design lifetime) of 100 years has been adopted for determination of the loads, since this is the longest lifetime that is considered by the Eurocodes. The National Annex to Eurocode 1-2 (EC1-2 - NA; see [IIa]), for example, does provide an expression[2] to adjust the traffic loads associated with another reference period (i.e. other than 100 years), however, this only seems to hold for reference periods smaller than 100 years. Therefore, one should be aware of this starting point that the loads are based on a reference period of 100 instead of the intended full service lifetime of 200 years (see section 3.1).

Table 4.9: Overview of load cases for standard viaduct

Load cases	Type of load	Examples
LC1 - 10	Permanent loads	Self-weight, dead load, shrinkage/creep
LC11 - 30	Traffic loads	LM1, braking/acceleration loads
LC31 - 40	Pedestrian and cyclist loads	UDLs
LC41 - 50	Other live loads	Wind load, thermal load
LC51 - 60	Accidental loads	Collision impact loads

Permanent loads

LC1a – Self-weight

The self-weight of all elements is automatically generated by SCIA Engineer, except for the self-weight of the deck, which is manually calculated and inputted based on the chosen SKK 900 box beam profile (see subsection 4.2.2). The self-weight of the SKK 900 profile is 19,0 kN/m [76]. Divided over the width of the profile, this results in a self-weight surface load of 19,0 kN/m / 1,48 m = 12,84 kN/m^2.

LC2 – Dead load

The self-weight of the used materials and the (assumed) dead load of components that are taken into account are listed below:

- Concrete: γ_c = 25,0 kN/m^3
- Steel: γ_s = 78,5 kN/m^3
- Asphalt: γ_a = 23,0 kN/m^3
- Soil: γ_{so} = 20,0 kN/m^3
- Safety barriers: q_{Ek} = 0,6 kN/m
- Parapets: q_{Ek} = 1,0 kN/m

[2] See National Annex to NEN-EN 1991-2 [IIa], art. 2.2 (4)

LC3 – Shrinkage/creep
The shortening of the deck as a consequence of combined shrinkage and creep is inputted as a temperature load in the model. The combined shortening due to shrinkage and creep (ε_{c+s}) has been assumed to be 0,3‰ based on common values according to engineers at Lievense. This results in a temperature load of ΔT = -30,0 K.

Traffic loads
The traffic loads are calculated in accordance with Eurocode 1-2 (EC1-2; see [II]). The division of the carriageway into notional lanes has been done in accordance with article 4.2.3 of EC1-2, resulting in 3 notional lanes and a remaining width of 0,2 m (w = 12,0 m - 2 x 1,4 m = 9,2 m).

LC11-LC20 – Vertical load by traffic
The vertical traffic loads are determined in accordance with article 4.3 of EC1-2. Specifically, article 4.3.2 is of interest since load model 1 (LM1) has been applied. The mapping of the lane numbers has been done in two different ways: 1) mapping from left to right, the outer lane being lane number 1, and 2) mapping from the centre (lane 1) outwards on both sides of lane 1. The former way is referenced to as ‘edge’, and the latter as ‘center’ (see Appendix I). Besides, the tandem system (TS) is applied on 6 different positions of the deck. In this way, it is believed that all critical scenarios are covered.

LC21-LC24 – Horizontal load by traffic
The horizontal traffic loads are determined in accordance with article 4.4.1 of EC1-2. The calculation of the braking and acceleration forces, as defined by equation (4.6), are split up into an UDL and a TS load by dividing the UDL-related part of the equation by w_1 x L and by dividing the TS-related part of the equation by 4 axles of 0,4 x 0,4 m. Furthermore, centrifugal and other transverse forces are not required to take into account since the horizontal radius of the carriageway (r) is larger than 1500 m.

Pedestrian and cyclist loads
LC31 – Pedestrian and cyclist loads
In accordance with article 5.3.2 of EC1-2, an UDL of Q_{Ek} = q_{fk} = 5,0 kN/m^2 has been defined for footways or cycle tracks. This load has been applied on both kerbs (i.e. over a width of 2x 1,4 m).

Other live loads
For both the wind and thermal loads, values from a reference project done by Lievense are adopted. According to engineers at Lievense, these are representative, common, values. Furthermore, no more other live loads have been considered. For example, snow load on the decks has not been determined because it is not taken into account in the governing load combinations.

LC41 – Wind load
The adopted wind loads are calculated in accordance with EC1-1-4 and the ROK. This results in a wind load perpendicular to the deck of $q_{Ek,\perp}$ = 7,6 kN/m when wind is the dominant load case, and $q_{Ek,\perp}$ = 5,5 kN/m when wind load is combined with traffic loads. Similarly, wind loads parallel to the deck are calculated, which amount to 40% of the total perpendicular wind divided by the deck width (40% x $q_{Ek,\perp}$ x L_{deck} / w_{deck}). Furthermore, vertical wind load on the deck and horizontal wind load on the intermediate support is not taken into account as these are non-critical load cases.

LC42 – Thermal load
The thermal load is split up into a yearly and a daily temperature rise and drop. The adopted thermal loads are determined in accordance with EC1-1-5. This results in a yearly temperature rise and drop of respectively $\Delta T_{N,exp}$ = +23,2 K and $\Delta T_{N,con}$ = -29,5 K. The daily temperature rise has been assumed to be $T_{M,heat,bk}$ = +10,1 K on top, and $T_{M,heat,ok}$ = -2,7 K on the bottom of the deck (linear distribution). The daily temperature drop has been assumed to be $T_{M,cool,bk}$ = -4,5 K on top, and $T_{M,cool,ok}$ = +1,2 K on the bottom of the deck (linear distribution).

Accidental loads
For the calculation of accidental loads due to a collision under the bridge (EC1-1-7 + NA, art. 4.3.1) or due to a collision with the edge of the deck (EN1-1-7 + NA, art. 4.3.2), it is assumed that traffic category ‘Roads in urban areas’ applies.

LC51 – Collision under bridge
For the intermediate piers, the following loads are taken into account to simulate the impact of a collision under the bridge:

- F_{dx} = 1000 kN parallel to the underpassing road
- F_{dy} = 500 kN perpendicular to the underpassing road

The point of application of the load is at 1,2 m above road level.

LC52 – Collision with edge of deck
The load that is taken into account to simulate a collision with the edge of the deck acts on the most unfavourable location on the edge of the deck above the relevant roadway. In order to 'find' this most unfavourable location, three possible locations of the load have been taken into account. The magnitude of the applied load is F_{dx} = 1000 kN. No reduction factor has been applied (conservative approach).

LC53 – Accident on bridge
The calculation of the load due to an accident on the bridge (viaduct) has been done in accordance with the ROK, art. 5.8 - art. 4.7.1 (1)P. A tandem system (TS) in accordance with LM1 (2x $Q_{1;k}$) is taken into account, which is placed with the outer wheels on the edge of the deck, parallel to the axis of the road. The remaining area of the deck is loaded by the representative load according to EC1-2 article 4.3.2, reduced by the tandem system that is placed on the edge of the deck. Four different positions of the tandem system have been taken into account.

Calculation of loads
The calculation of the loads has been done by means of an Excel-spreadsheet, which has been added in Appendix I. Besides, a detailed overview of the inputted loads can be seen in the separate document "SCIA Engineering Report - Standard Viaduct Model".

4.4.2. Load Combinations
The load combinations for the Ultimate Limit State (ULS) and the Serviceability Limit State (SLS) are determined in accordance with EC0 - Annex A2 + NA, art. A2.2.2 and A2.2.5 (see [I] + [Ia]).

Traffic load groups
In accordance with EC1-2, art. 4.5.1 and 4.5.2 + NA, the simultaneity of the vertical (LM1) loads, the horizontal loads, and the pedestrian and cyclist loads should be taken into account by considering the groups of loads as defined in "Tabel NB.3 – 4.4a" and in "Tabel NB.4 – 4.4b" (see [IIa]), which results in the definition of traffic load group 1a (gr1a) and 2 (gr2). The resulting traffic load groups should be considered as one characteristic load for the combination with non-traffic loads.

Load factors
The load factors (γ-factors) for the ULS are chosen from "Tabel NB.16 – A2.4(B)" (see [Ia]) and "Tabel NB.22 – A2.5". The former is used for the general ULS loads whereas the latter is used for accidental loads. With regard to the general ULS load factors, the load factors belonging to consequence class CC3 have been used. The load factors for the SLS are chosen from "Tabel A2.6" (see [I]).

Combination factors
The combination factors (Ψ-factors) are chosen from "Tabel NB.12 – A2.1" (see [Ia]). NB: in "Tabel NB.19" (see [Ia]), the elaboration of the equations (6.10b) and (6.11b) (see [I]) for load combinations gr1 - gr5, W, T, S, and A1 (see "Tabel NB.12 – A2.1" in [Ia]) is given.

Load combinations
The load combinations are drawn up in an Excel-spreadsheet, which has been added in Appendix J. Besides, a detailed elaboration of the considered load combinations can be seen in the separate document "SCIA Engineering Report - Standard Viaduct Model".

Furthermore, in SCIA Engineer, every load case is assigned to a so-called “load group”. In these load groups, the relation between all load cases that are assigned to this load group is defined. By means of defining an “exclusive” relation, it is assured that load cases, even though they appear together in a load combination, are not both taken into account at the same time in that load combination. Instead, SCIA Engineer will automatically generate separate (internal) load combinations in order not to combine these load cases. These load groups and their relations are shown in Table 4.10.

Table 4.10: Definition of load groups and their respective relation in SCIA Engineer

Name	Load	Relation	Type
Permanent	Permanent		
LM1 - deck - gr1a - UDL	Variable	Standard	Traffic - gr1a - UDL
LM1 - T.slab - gr1a - UDL	Variable	Standard	Traffic - gr1a - UDL
LM1 - gr1a - TS - Lane 1	Variable	Exclusive	Traffic - gr1a - TS
LM1 - gr1a - TS - Lane 2	Variable	Exclusive	Traffic - gr1a - TS
LM1 - gr1a - TS - Lane 3	Variable	Exclusive	Traffic - gr1a - TS
Pedestrian and cycle track (gr1a - UDL)	Variable	Standard	Traffic - gr1a - Pedestrian and cycle track
Horizontal forces (gr2)	Variable	Exclusive	Traffic - gr2 - Horizontal forces
Wind - FWk	Variable	Exclusive	Wind forces - FWk - Persistent
Wind - F*W	Variable	Exclusive	Wind forces - F*W - Design
Thermal loads (yearly)	Variable	Exclusive	Thermal actions - Tk
Thermal loads (daily)	Variable	Exclusive	Thermal actions - Tk
Accidental loads	Variable	Exclusive	Traffic - gr1a - UDL

4.4.3. SCIA Model Check

In order to verify the model, the sum of loads ($\sum F_x, \sum F_y, \sum F_z$) following from the calculation protocol is compared with the calculated loads. The comparison has been added in Appendix K.

A deviation of a maximum of 5% from the manual calculation was considered permissible. However, a maximum (absolute) difference of only 0,78% was found. Therefore, the check was satisfied.

4.5. Verification and Validation

The standard viaduct as described in the foregoing sections is the main result of this chapter. Therefore, in order to complete the development of this standard viaduct, verification and validation of the final result is done. First of all, clear definitions of both concepts within the context of this research are given, formulated in a question to be answered. Subsequently, these verification and validation questions are answered.

4.5.1. Verification

Verification is understood as checking whether the system, in this case the standard viaduct, has been developed correctly. Therefore, the question to be answered here is formulated as:

> "Has a (proposal for a) standard viaduct been developed in accordance with the elaborated technical action points as was described in Chapter 3?"

This question is answered (i.e verification is done) by means of checking the developed standard viaduct within Crowther's framework. Specifically, this has been done by means of verifying to what extent the developed standard viaduct complies with the 28 key DfD principles for circular concrete viaducts (see Table 3.2), supplemented with a short explanation. An overview of this verification is shown in Table 4.11.

Table 4.11: Verification of standard viaduct by means of 28 key DfD principles for circular concrete viaducts

	Principle	**Incorporated?**	**Explanation**
1.	Specify removable, durable, mechanical instead of chemical and/or cast in-situ, rigid, connections	Yes	Emphasising need for demountable connections between prefab elements
2.	Design components (foundations, abutments, piers, etc.) to be retractable from ground	Yes	Choice for steel pipe piles
3.	Specify materials and components with long life span	Yes	(Prefab) concrete and steel
4.	Design joints and connectors to withstand repeated use	Yes/No	Mentioned, but not yet translated into design
5.	Minimise the number of components	Yes	Division of standard viaduct in minimum number of (different types of) elements and components (see Table 3.5)
6.	Minimise the number of different types of components	Yes	*Idem*
7.	Minimise the number of fasteners or connectors	No	Not specifically emphasised
8.	Minimise the number of different types of fasteners or connectors	No	*Idem*
9.	Minimise the number of different types of material	Yes	(Prefab) concrete and steel
10.	Avoid toxic and hazardous materials	Yes/No	Only (prefab) concrete and steel, but not specifically emphasised
11.	Avoid specifying secondary finishes to materials or components	Yes	Choice not to apply structural screed to box beams, and to apply removable asphalt layer
12.	Specify materials that can be reused or recycled	Yes	(Prefab) concrete and steel
13.	Provide standard and permanent identification of (types of) component and materials	No	Not specifically emphasised
14.	Permanently identify points of disassembly	No	Not specifically emphasised
15.	Using of interchangeable components	Yes	Standardisation of components' layout (e.g. dimensions) and cross-sections

Table 4.11 continued from previous page

	Principle	Incorporated?	Explanation
16.	Design for prefabrication of components	Yes	General starting point for layout and design of standard viaduct
17.	Design for the repetition of similar components (i.e. design for mass production)	Yes	Standardisation of components' layout (e.g. dimensions) and cross-sections
18.	Separate the main load-bearing components from cladding and finishing elements	Yes	Division of components of standard viaduct in five different time-related layers (see Table 3.1)
19.	Standardising viaduct form and layout	Yes	Development of standardisation scheme
20.	Use a standard structural grid	Yes	*Idem*
21.	Structure components according to their service life and the expected time till obsolescence to allow for parallel (dis)assembly	Yes/No	Components in different layers largely separated from each other, but not specifically emphasised
22.	Provide access to all parts and components	Yes/No	All considered parts and components largely accessible, but not specifically emphasised
23.	Provide realistic tolerances to allow for manoeuvring during (dis)assembly	No	Not specifically emphasised
24.	Make components and materials of a size that suits the intended means of handling	Yes/No	Mentioned, but not yet translated into design
25.	Reduce the number of wearing parts that may need to be serviced	Yes	Minimum number of bearings and transition joints required for layout and design of standard viaduct
26.	Use sacrificial materials and components where wear is unavoidable and allow for their easy disassembly from the whole	Yes	Use of bearings and transition joints, accessible for disassembly and replacement
27.	Design to avoid permanent deformations and damage during (dis)assembly, use, and storing	No	Not specifically emphasised
28.	Minimise cast in-situ components and elements	Yes	Use of (prefab) concrete components

It can be concluded that most of the 28 key DfD principles for circular concrete viaducts are incorporated into the layout and design of the (proposal for a) standard viaduct. Some of the principles, however, are not (yet) incorporated into the design, either since those principles (7, 8 and 27) are thought to be subordinate to other principles regarding the layout and design of the standard viaduct, or since it considers the detailed technical elaboration of the standard viaduct (principle 23). Besides, principles 13 and 14 are thought to be less relevant with regards to the layout and design of the standard viaduct at this stage, and can (and should) be incorporated at a later stage by means of, for example, an element/component passport, a demolition plan, and/or a monitoring plan. Finally, although a number of principles has been considered, they have not yet been incorporated into the design since this, for example, considers the detailed technical elaboration of the standard viaduct (principles 4 and 24), or they have simply been incorporated without specifically emphasising them (principles 10, 21 and 22).

4.5.2. Validation

Validation is understood as checking whether the correct system, in this case the standard viaduct, has been developed. Therefore, the question to be answered here is formulated as:

> "Has a (proposal for a) standard viaduct been developed that addresses the action point of developing a standardisation scheme as was argued to be required in section 2.2.1 (see Table 2.5)?"

This validation question can simply be answered positively, since this action point was the main motivation for the development of (a proposal for) the standard viaduct.

5

Concept Demountable Footing to Foundation (F2F) Dowel Connection

In this chapter, the development of the proposed concept demountable footing to foundation (F2F) dowel connection is discussed in detail. Firstly, the main concept of the connection is given in section 5.1. Subsequently, the design approach that has been used to develop and to check the feasibility of the concept demountable connection is described in section 5.2, as well as the SCIA Engineer model that was created to model the behaviour of the connection. After that, a more general result in terms of a design table that can be used to quickly determine a first estimation of the required connection's dimensions and properties for a range of cross-sectional forces is presented in section 5.3, and subsequently the sensitivity of several parameters and variables of the connection is checked and discussed in section 5.4. Furthermore, important practical issues regarding tolerances and phasing, including topics like (de)construction and transportability are discussed in section 5.5. Finally, verification and validation of the proposed connection with respect to the required properties of a circular (i.e. demountable) solution according to what was found in literature is done in section 5.6.

The result of this chapter is the elaborated description of a concept demountable F2F dowel connection which could be applied in the standard circular concrete viaduct as was established and described in detail in Chapter 4. Therefore, this chapter concludes the second part of the main result by proposing a demountable solution for specific application within the general concept of a circular concrete viaduct.

5.1. Main Concept

From a DfD perspective, a demountable footing to foundation (F2F) connection should, most of all, be kept as simple as possible, e.g. not consist of multiple different components. Besides, the connection should amongst others be standardised and, obviously, easy demountable and reusable. Therefore, it was decided that for every pile to footing interface only one separate connection, independent of other pile to footing interfaces, should be developed. Based on the discussed concrete DfD connection methods in section 2.3, it was decided to investigate the development of a demountable pinned dowel connection, amongst others based on what was concluded by Xiao et al. [45], namely that *standard*, i.e. non-demountable, pinned dowel connections can be used in joints that do not have to transfer large bending moments. Assuming that a demountable dowel connection would be more likely to behave like a hinge than like a rigid connection, no large bending moments were expected at the F2F interfaces, hence the motivation.

From a first look at the cross-sectional forces at the F2F interfaces, based on an assumed hinged connection, it became clear that in none of the load combinations tensile forces in the foundation piles arise. This directly simplified the challenge in such a way that for the development of a demountable F2F connection, within the boundaries of the proposed standard viaduct, it was decided to focus on a demountable pinned dowel connection that only needs to transfer bending moments and shear forces.

Finally, this resulted in a concept demountable dowel connection consisting of a circular steel end plate welded on top of the steel pipe foundation piles, and welded onto the end plat a steel dowel, which partially penetrates into the concrete footing. An impression of such a connection is shown in Figure 5.1, as well as the main parameters and variables which characterise the connection. Three different variants to execute this connection have been developed. In all three variants, it is assumed that the dowel is covered by means of a protective layer over a certain (variable) height, and that no contact between the top of the dowel and the concrete is possible. The protective layer is being applied in order to prevent the dowel from making direct contact with the concrete outside of the main reinforcement (i.e. the concrete cover), since the strength and stiffness of the concrete cover is considerably lower than the concrete that is enclosed by the main reinforcement. Therefore, it is desired to introduce the forces into the concrete that is located within the main reinforcement in order to prevent the concrete cover from being abraded (Dutch: "*afboeren*"). Besides, the hollow space between the top of the dowel and the surrounding concrete is provided in order to prevent the transfer of normal (vertical) force at this location.

Other than that, the three variants are characterised as:

1. A solution with prefabricated holes in the footings, in which the dowels including the protective layer, applied over a certain height of the dowel, fit exactly (see Figure 5.2a).
2. A double 'fitting tube' system (Dutch: "*'pasbuizen*") combined with a cast in-situ intermediate layer (e.g. non-shrinking mortar) filling in the void between both tubes (see Figure 5.2b).

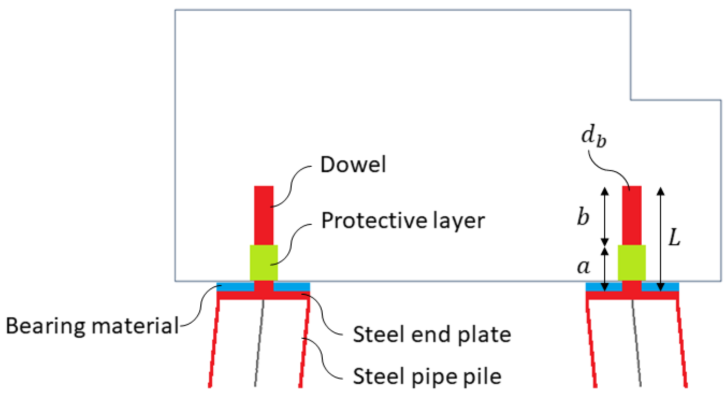


Figure 5.1: Impression of investigated demountable dowel footing to foundation (F2F) connection

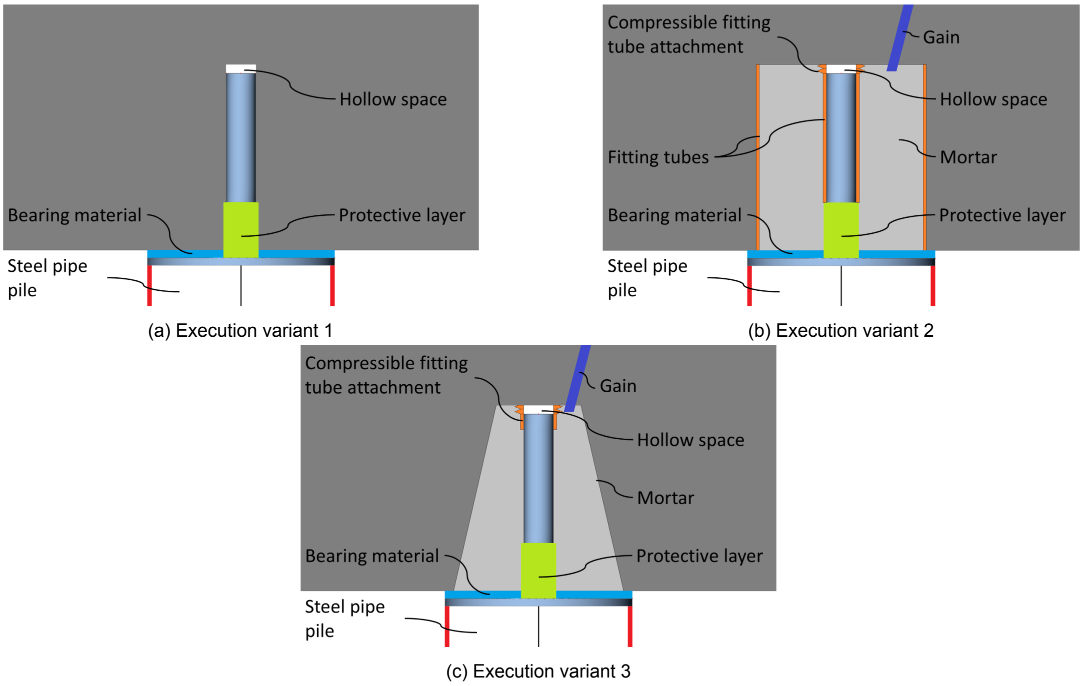


Figure 5.2: Three different variants for execution of a demountable dowel connection

3. A solution with prefabricated, oversized, cone-shaped holes in the footings, in which the dowels including the protective layer, applied over a certain height of the dowel, and a compressible fitting tube attachment at the top of the dowel are inserted in combination with a cast in-situ layer (e.g. non-shrinking mortar) filling in the void after installation of the dowels (see Figure 5.2c).

In section 5.5, these three variants, as well as some practical issues regarding each variant, are discussed in more detail.

5.2. Design Approach

The approach that was used to develop and to check the feasibility of the concept demountable footing to foundation (F2F) connection is shown in Figure 5.3 and described by the following procedure:

1. Development of an analytical model that is used to calculate the replacing rotational spring stiffness of the demountable dowel connection ($k_{r,con}$).
2. Input of rotational springs in the model of the standard viaduct in SCIA Engineer, and (re)-calculation of the cross-sectional forces at the F2F interfaces.
3. Extraction and processing of the results in order to find the critical cross-sectional forces at the F2F interface (N / V / M).
4. Conversion and input of the critical cross-sectional forces into a model of the concept demountable dowel connection in SCIA Engineer.
5. Performing relevant design checks on maximum dowel deformation (w_{max}) and maximum contact stress (σ_z). If the checks are satisfied, the procedure ends. If not, the procedure is repeated with a different layout of the connection, starting over again at step 1.

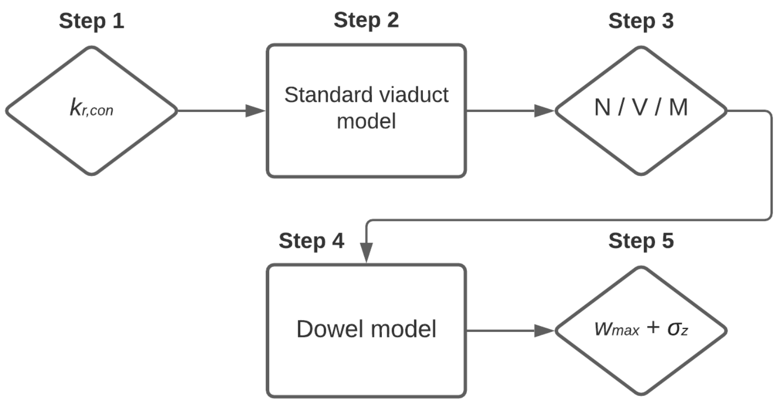


Figure 5.3: Verification process of demountable F2F dowel connection

5.2.1. Step 1. Calculation of Replacing Rotational Spring Stiffness

In order to calculate the replacing rotational spring stiffness of the connection, the connection has been schematised as a combination of a basic Euler-Bernoulli bending beam and a semi-infinite beam on an elastic foundation (see Figure 5.4), which has been based on the studies by X.G. He and A.K.H. Kwan [50] and Dei Poli et al. [51] (see subsection 2.3.1).

The ordinary differential equations (ODEs) for those cases are:

$$EI\frac{d^4w_1(x)}{dx^4} = 0 \qquad (-a \leq x \leq 0) \tag{5.1}$$

$$EI\frac{d^4w_2(x)}{dx^4} + k_d \cdot w_2(x) = 0 \qquad (x \geq 0) \tag{5.2}$$

Parameter a represents the length over which the protective layer is being applied, which is assumed to not contribute to the stiffness of the dowel connection, whereas parameter b represents the length over which the dowel is embedded in the concrete. The foundation modulus of the surrounding concrete embedding is modelled by means of distributed (Winkler) springs with a stiffness equal to k_d. Furthermore, the dowel has a constant diameter (d_b) and constant bending stiffness (EI). The cross-sectional forces that have to be transferred are modelled to seize at the dowel end, i.e. where the dowel is connected to the plate.

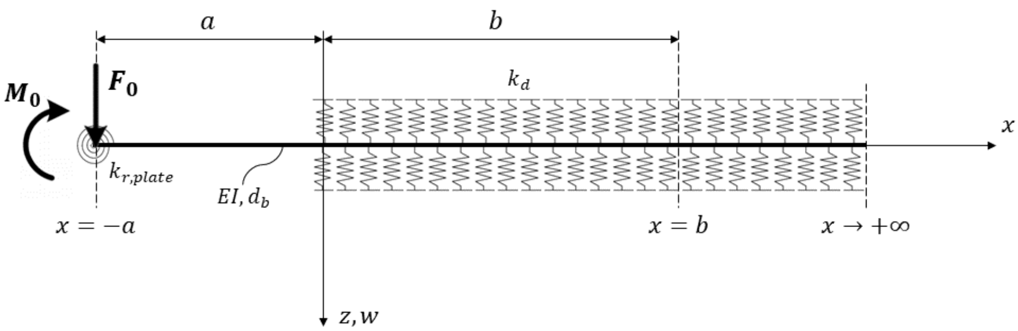


Figure 5.4: Structural scheme of dowel embedded in concrete as a semi-infinite beam on elastic foundation

Solving the ODEs in equations (5.1) and (5.2) results in the displacement fields of the respective cases, expressed in terms of 8 unknowns constants. As a result of the assumption of a semi-infinite beam on an elastic foundation, two of the four unknown constants in the solution for this beam can be set to zero (i.e. $C_7 = C_8 = 0$, see equation (5.4)), leaving a total number of 6 unknowns constants. By means of standard structural mechanics derivations, expressions for the rotation, and for the bending moment and the shear force distribution can be derived as well:

$$w_1(x) = \frac{C_1}{6}x^3 + \frac{C_2}{2}x^2 + C_3x + C_4 \tag{5.3}$$

$$\begin{aligned} w_2(x) &= e^{-\lambda x}\left(C_5 \cos \lambda x + C_6 \sin \lambda x\right) + e^{\lambda x}\left(C_7 \cos \lambda x + C_8 \sin \lambda x\right) \Rightarrow \\ w_2(x) &= e^{-\lambda x}\left(C_5 \cos \lambda x + C_6 \sin \lambda x\right) \ \text{ with: } \lambda = \sqrt[4]{\frac{k_d}{4EI}} \end{aligned} \tag{5.4}$$

$$\phi_i = -\frac{dw_i}{dx} \qquad \kappa_i = \frac{d\phi_i}{dx}$$

$$M_i = EI \cdot \kappa_i \qquad V_i = \frac{dM_i}{dx}$$

Two boundary conditions (BCs) and four matching conditions (MCs) are required to solve the system. The four MCs are found at the interface between both cases, and in fact are of the most basic form, namely equality of all four fields and distributions (displacement, rotation, bending moment, and shear force) as there is no discontinuity nor are external bending moments or forces being applied at this interface. The two BCs are found by means of the balance of external and internal shear force, and the balance of external and internal bending moments. Whereas the balance of shear forces is straightforward, the balance of bending moments is not, since the effect of the circular end plate welded to the dowel has been taken into account by means of a replacing rotational spring. The spring stiffness of this rotational spring has been calculated by means of a similar procedure as is described here, and therefore is not further elaborated upon[1]. Finally, this results in the following six BCs and MCs:

For $x = -a$:	**BC1:** $V_1(-a) = -F_0$	**BC2:** $M_1(-a) = M_0 + k_{r,plate} \cdot \phi_1(-a)$
For $x = 0$:	**MC1:** $w_1(0) = w_2(0)$	**MC2:** $\phi_1(0) = \phi_2(0)$
	MC3: $M_1(0) = M_2(0)$	**MC4:** $V_1(0) = V_2(0)$

Once the unknown constants have been solved, analytical expressions for the displacement and rotation fields, and for the bending moment and shear force distributions are obtained. The final step in order to calculate the replacing rotational spring stiffness of the demountable dowel connection ($k_{r,con}$) is to apply only a unit bending moment at $x = -a$ (i.e. $M_0 = M_{unit}$ and $F_0 = 0$), and to solve the general equation $M_i = k_{r;i} \cdot \phi_i$ for $k_{r;i}$, which results in:

[1]The calculation of the replacing rotational spring stiffness of the steel end plate has been added in Appendix L

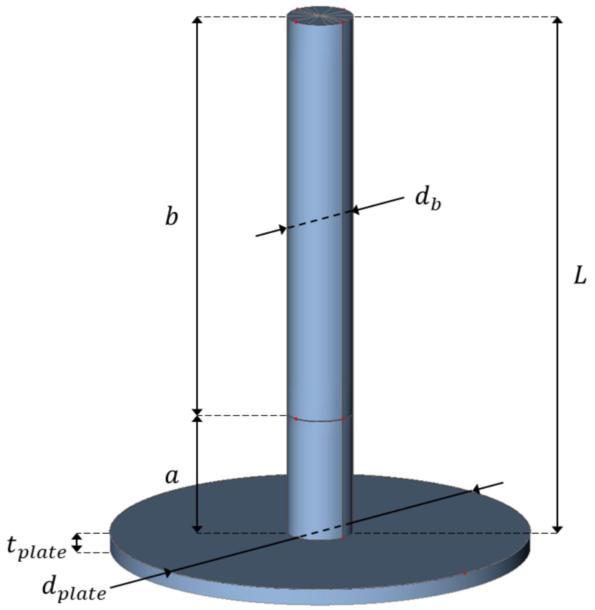


Figure 5.5: Dowel connection parameters and variables

$$k_{r,con} = \frac{M_{unit}}{\phi_1(-a)} = \frac{(a \cdot k_{r,plate} + EI)\lambda + k_{r,plate}}{a \cdot \lambda + 1} \qquad (x = -a; F_0 = 0; M_0 = M_{unit}) \tag{5.5}$$

In a similar way, $k_{r,plate}$ has been calculated, for which the end plate has been schematised as a beam supported by two translational springs, representing the edge of the steel pipe pile, and with a varying width over its length, resulting in a varying bending stiffness over its length (i.e. $EI_{plate} = EI_{plate}(x)$). For the properties of the end plate, a thickness of $t_{plate} = 20$ mm, and a diameter of $d_{plate} = \varnothing_{pile} = 508$ mm are assumed. This results in a replacing rotational spring stiffness of the plate of:

$$k_{r,plate} = 1680 \text{ kNm/rad (see Appendix L)} \tag{5.6}$$

The parameter and variable values of the final layout of the dowel connection are clarified in Figure 5.5 and listed below. The foundation modulus of the surrounding concrete embedding (k_d) is calculated by means of equations (2.1) and (2.2) (see page 35). Besides, the magnitude of length b has been based on the expression for the length of the dowel subjected to significant deformation (l_b) [50], resulting in a total dowel length of $L = a + b = 650$ mm.

$E = 210.000$ N/mm^2 $\qquad I_z = \frac{\pi d_b^4}{64} = 2{,}01 \cdot 10^6$ mm^4

$f_c = f_{ck} = 30$ N/mm^2 $\qquad EI = E \cdot I_z \cdot 10^{-9} = 422{,}23$ kNm^2

$a = 150$ mm $\qquad k_d = \frac{127 c_1 \sqrt{f_c}}{d_b^{2/3}} \cdot d_b \cdot 10^3 = 3{,}00 \cdot 10^6$ kN/m^2

$d_b = 80$ mm $\qquad \lambda = \sqrt[4]{\frac{k_d}{4EI}} = 6{,}49$ m^{-1}

$c_1 = 1{,}0$ (spacing > 25 mm) $\qquad b = l_b = \frac{\pi}{\lambda} \cdot 10^3 \approx 500$ mm

Substituting those values in expression (5.5) results in a replacing rotational spring stiffness of the demountable dowel connection of:

$$k_{r,con} = 3069 \text{ kNm/rad} \tag{5.7}$$

The entire calculation has been added in Appendix M.

5.2.2. Step 2. Input of Rotational Spring in Standard Viaduct Model

The standard viaduct contains of 48 F2F connections. In the standard viaduct model, all of these connections have been modelled by means of rotational springs as can be seen in Figure 5.6, in which also the assigned properties are shown. It is reasoned that the rotational spring stiffness of the dowel connection has the same magnitude for both vertical rotation planes (i.e. φy and φz) and that free rotation is possible on the horizontal rotation plane around the dowel (i.e. φx). Besides, the translation of both connecting elements is fixed, which implies that in the model it would be possible to transfer tensile forces between footing and foundation. Therefore, it should be carefully (and manually) checked that in none of the load cases tensile forces arise in the foundation piles (see Tables 5.1 and 5.2).

Furthermore, it was decided to assume no transversal prestressing in the deck which implies that cracked concrete in transversal direction is assumed in calculation of the orthotropy of the deck. This decision was made after evaluating the results for both cases, from which it was concluded that assuming a transversally post-tensioned deck had a negligible effect on the critical cross-sectional forces.

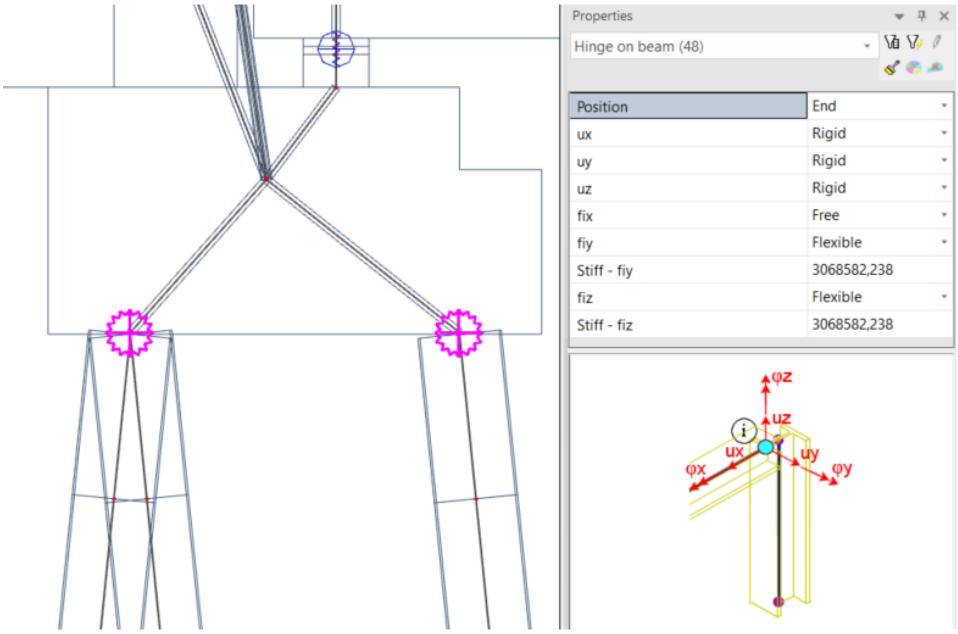


Figure 5.6: Properties of rotational springs in standard viaduct model

5.2.3. Step 3. Extraction and Processing of Critical Cross-Sectional Forces

The critical cross-sectional forces at the 48 F2F interfaces (i.e. at the location of the 48 rotational springs in the standard viaduct model) are shown in Figures 5.7a and 5.7b (also see the separate document “SCIA Engineering Report - Standard Viaduct Model”). Both the upper and lower limits of these critical cross-sectional forces (i.e. envelope results) are shown for ULS and SLS respectively. In order to be able to compare the results found in the SCIA Engineer model with the results calculated with the (1D) analytical model that was described in subsection 5.2.1 (see Figure 5.4 and Appendix N), both the shear force components (V_y and V_z) and the bending moment components (M_y and M_z) have been combined into one (critical) shear force and bending moment using Pythagoras’ theorem:

$$V = \sqrt{(V_y)^2 + (V_z)^2} \tag{5.8}$$

$$M = \sqrt{(M_y)^2 + (M_z)^2} \tag{5.9}$$

The resulting combinations of critical cross-sectional forces are shown in Tables 5.1 (ULS) and 5.2 (SLS).

Name	dx [mm]	Case	N [kN]	V_y [kN]	V_z [kN]	M_x [kNm]	M_y [kNm]	M_z [kNm]
Buispaal817	0,000	ULS 3a/1	**-1199,09**	4,03	-14,38	0,00	2,36	-0,73
Dowel3	10,000	ULS 6a/2	**-216,37**	10,66	-16,11	**0,00**	0,05	1,50
Buispaal824	0,000	CAL 2a/3	-678,74	**-55,10**	8,36	0,00	-0,29	-10,37
Buispaal831	0,000	ULS 7a/4	-615,13	**20,83**	-10,66	0,00	1,18	3,74
Dowel	0,000	ULS 2a/5	-834,91	2,01	**-108,95**	0,00	-1,75	-0,18
Dowel4	0,000	ULS 4a/6	-712,85	-1,72	**63,33**	0,00	0,88	-1,02
Buispaal816	10,000	CAL 2a/7	-581,16	9,62	-68,76	0,00	**-13,09**	1,79
Buispaal831	10,000	CAL 2a/8	-469,42	9,45	19,45	0,00	**7,69**	1,81
Dowel2	10,000	CAL 2a/9	-440,71	-49,32	-58,99	0,00	-2,84	**-11,10**
Dowel29	10,000	CAL 2a/10	-319,31	17,77	14,51	0,00	-3,44	**3,99**

(a) ULS results (NB: red boxes refer to combinations in Table 5.1)

Name	dx [mm]	Case	N [kN]	V_y [kN]	V_z [kN]	M_x [kNm]	M_y [kNm]	M_z [kNm]
Buispaal817	0,000	SLS 3a/1	**-819,22**	-1,03	-7,03	0,00	1,18	-0,97
Dowel15	10,000	SLS 3a/2	**-257,35**	-0,71	8,66	**0,00**	-2,25	-0,72
Buispaal816	0,000	SLS 4a/3	-557,84	**7,69**	-0,14	0,00	0,00	1,49
Dowel	0,000	SLS 3a/4	-582,36	-1,16	**-75,67**	0,00	-1,29	-0,65
Dowel4	0,000	SLS 3a/5	-500,93	1,32	**43,01**	0,00	-0,37	-0,16
Dowel8	0,000	SLS 2a/6	-457,87	-0,84	20,27	0,00	**-6,35**	-0,45
Dowel	0,000	SLS 3a/7	-468,52	1,24	-45,74	0,00	**2,83**	0,08
Buispaal824	10,000	SLS 4a/8	-608,55	**-8,01**	0,26	0,00	0,16	**-1,87**
Buispaal816	10,000	SLS 4a/9	-557,47	7,69	-0,08	0,00	-0,01	**1,57**

(b) SLS results (NB: red boxes refer to combinations in Table 5.2)

Figure 5.7: Critical cross-sectional forces at footing to foundation (F2F) interfaces

Table 5.1: Critical cross-sectional forces (ULS)

Combination	N [kN]	V [kN]			M [kNm]		
$N_{max;ULS}$	**-1199**	$\sqrt{4{,}03^2 + (-14{,}38)^2}$	=	15	$\sqrt{2{,}36^2 + (-0{,}73)^2}$	=	2
$V_{max;ULS}$	-835	$\sqrt{2{,}01^2 + (-108{,}95)^2}$	=	**109**	$\sqrt{(-1{,}75)^2 + (-0{,}18)^2}$	=	2
$M_{max;ULS}$	-581	$\sqrt{9{,}62^2 + (-68{,}76)^2}$	=	69	$\sqrt{(-13{,}09)^2 + 1{,}79^2}$	=	**13**

Table 5.2: Critical cross-sectional forces (SLS)

Combination	N [kN]	V [kN]			M [kNm]		
$N_{max;SLS}$	**-819**	$\sqrt{(-1{,}03)^2 + (-7{,}03)^2}$	=	7	$\sqrt{1{,}18^2 + (-0{,}97)^2}$	=	2
$V_{max;SLS}$	-582	$\sqrt{(-1{,}16)^2 + (-75{,}67)^2}$	=	**76**	$\sqrt{(-1{,}29)^2 + (-0{,}65)^2}$	=	1
$M_{max;SLS}$	-458	$\sqrt{(-0{,}84)^2 + 20{,}27^2}$	=	20	$\sqrt{(-6{,}35)^2 + (-0{,}45)^2}$	=	**6**

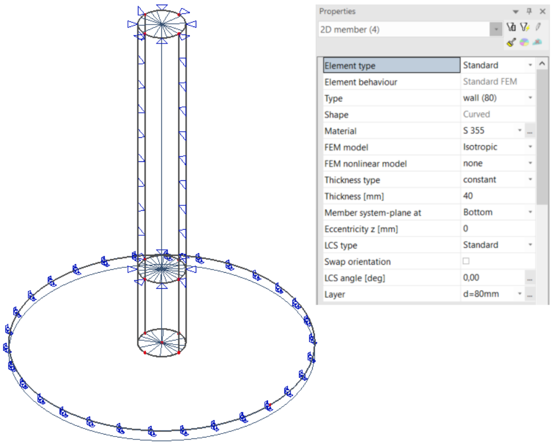
5.2.4. Step 4. Conversion and Input of Critical Cross-Sectional Forces into Dowel Model

Step 4.1. Dowel connection model

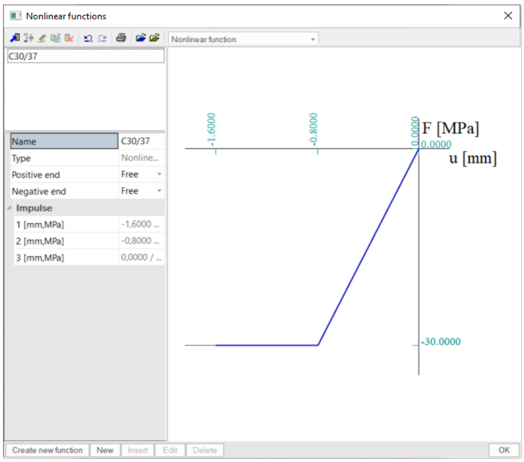
In order to be able to check the feasibility of the dowel connection, another model has been created in SCIA Engineer which has been evaluated with a non-linear analysis. The dowel has been modelled by means of curved 2D members, whereas the plate has been modelled as a circular 2D plate (see Figure 5.8a). The material used for both components is steel with steel grade S355. The plate edge is supported by means of distributed vertical springs from which the stiffness has been based on the axial stiffness of a single foundation pile:

$$k_h = \frac{E \cdot A}{L_{pile} \cdot p_{plate}} = \frac{E \cdot (p_{plate} \cdot t_{pile})}{L_{pile} \cdot p_{plate}} = \frac{E \cdot t_{pile}}{L_{pile}} = \frac{210.000 \cdot 12{,}5}{20.000} = 131{,}25 \text{ N/mm}^2 \tag{5.10}$$

The concrete embedding surrounding the dowel over the length b has been modelled as a non-linear subsoil which should illustrate the behaviour of the dowel to concrete interface. The initial linear stiffness (foundation modulus k_c) is calculated by means of equation (2.1):



(a) Analysis model including 2D dowel member properties



(b) Non-linear function of subsoil (i.e. concrete embedding)

Figure 5.8: Properties of dowel model in SCIA Engineer

$$k_c = \frac{127 c_1 \sqrt{f_c}}{d_b^{2/3}} = \frac{127 \cdot 1{,}0\sqrt{30}}{80^{2/3}} = 37{,}5 \text{ N/mm}^3 \tag{5.11}$$

The non-linearity of the embedding is inputted in SCIA Engineer by means of defining a non-linear function (see Figure 5.8b). Since the concrete can only take compression, a free displacement is inputted for the positive branch of the function, i.e. no tensile force is required to deform. For the negative branch, however, a stress-deformation (F-u) curve has to be defined. The idea is that this curve represents the properties of the typical bi-linear stress-strain diagram of concrete, where the values of ε_{c3} and ε_{cu3} are 1,75‰ and 3,5‰ respectively (see Table 3.1 and Figure 3.4 in Eurocode 2-1-1 [III]). For that reason, the upper limit value of the stress-deformation curve has been set to $F = f_{ck} = 30$ N/mm^2.

For determination of the deformation that corresponds to a strain of 1,75‰, it has been reasoned that the slope of the initial linear branch should be equal to foundation modulus k_c (similar to a typical stress-strain diagram, in which the slope represents the E-modulus), since this implies that as long as the model is in the linear branch ($F < 30$ N/mm^2), for example for a stress of $F = 20$ N/mm^2, a linear deformation of $u = \frac{F}{k_c} = \frac{20}{37{,}5} = 0{,}53$ mm should be found. Therefore, the deformation that corresponds to a strain of 1,75‰ has been determined as:

$$u_{c3}(\sim \varepsilon_{c3}) = \frac{F}{k_c} = \frac{f_{ck}}{k_c} = \frac{30}{37{,}5} = 0{,}80 \text{ mm} \tag{5.12}$$

The deformation corresponding to a strain of 3,5‰ has simply been taken as $u_{cu3} = \frac{3{,}5}{1{,}75} \cdot u_{c3} = 1{,}60$ mm. After reaching this value, no more force should be carried by the concrete, and therefore free displacement is modelled from this point onwards.

In order to check the defined non-linear subsoil (i.e. concrete embedding), a simple verification of a rectangular 2D plate element, supported by the same subsoil, has been done. The result of the obtained stress-deformation curve and the inputted stress-deformation function are shown in Figure 5.9. It can be noticed that the difference between both curves is negligible (31,5 N/mm^2 at 1,60 mm deformation instead of (the theoretical value of) 30,0 N/mm^2), and therefore the general behaviour of the defined non-linear subsoil was concluded to be correct.

For more details of the dowel model, see the separate document “SCIA Engineering Report - Demountable F2F Dowel Model”.

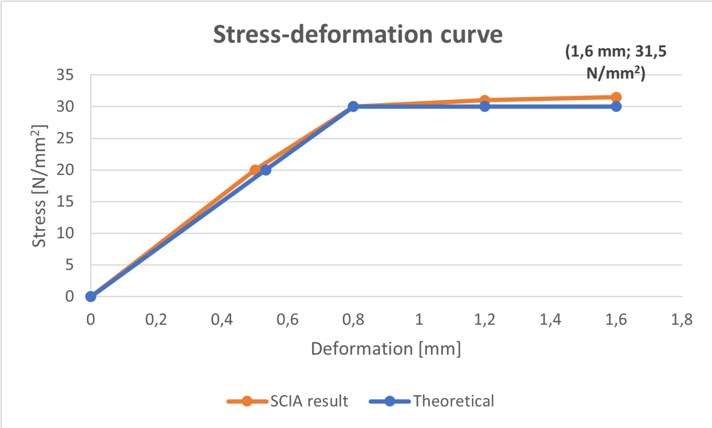


Figure 5.9: Verification of the defined non-linear subsoil (i.e. concrete embedding)

Step 4.2. Conversion of loads

Since normal forces are uncoupled from shear forces and bending moments in the analytical 1D beam (on elastic foundation) model, and since it was observed that the influence of the normal force on the dowel behaviour in the SCIA Engineer model was negligible in general, it was decided to not include the normal forces in further analyses. Therefore, the six combinations of critical cross-sectional forces shown in Tables 5.1 and 5.2 were inputted, however, each consisting of only the respective shear force and contraflexural bending moment.

The conversion of 1D point forces into line forces on the 2D member edges has been done by means of the following conversion expressions:

$$f_{V;i} = \frac{V_i}{p_b} \cdot 10^3 = \frac{V_i}{\pi d_b} \cdot 10^3 \text{ [kN/m]} \tag{5.13}$$

$$f_{M;i} = \pm \frac{\pi M_i}{2d_b^2} \cdot 10^6 \text{ [kN/m]} \tag{5.14}$$

Whereas the conversion expression for the shear forces (expression (5.13)) is straightforward (equal distribution over the perimeter of the dowel (p_b)), the conversion expression for the bending moment (expression (5.14)) is somewhat more complicated and requires explanation. The bending moment is inputted as a vertical line force on the 2D member edge with a trapezoidal distribution, i.e. on one half of the perimeter the line force is positive and on the other half negative, which results in a resulting vertical force equal to zero since the resultants of both line forces cancel each other out). However, since an eccentricity (i.e. lever arm) exists between the resultants of both line forces, a resulting bending moment is generated. It was found by means of analytical manipulation that this lever arm (e) can be expressed as:

$$e = \frac{8d_b}{\pi^2} \cdot 10^{-3} \text{ [m]} \tag{5.15}$$

Subsequently, the resulting forces (in opposite direction) on each half of the circular cross-section ($F_{M;i}^+$ and $F_{M;i}^-$) can be simply expressed as:

$$F_{M;i}^+ = -F_{M;i}^- = \frac{M_i}{e} \text{ [kN]} \tag{5.16}$$

Finally, both forces have to be linearly distributed over half of the dowel perimeter, being equal to zero at the axis of rotation (i.e. the point at which the vertical force switches from positive to negative, or vice versa) and being maximum at the outer fibre:

$$f_i^+ = -f_i^- = \frac{1}{2} \cdot \frac{F_{M;i}^+}{p_b/2} \cdot 10^3 = \frac{4F_{M;i}^+}{p_b} \cdot 10^3 = \frac{4F_{M;i}^+}{\pi d_b} \cdot 10^3 \text{ [kN/m]} \tag{5.17}$$

Substituting expressions (5.15) and (5.16) into (5.17) results in the final expression for the conversion of the 1D bending moment into a trapezoidally distributed line force on the 2D member edge:

$$f_i^+ = -f_i^- = \frac{4F_{M;i}^+}{\pi d_b} \cdot 10^3 = \frac{4M_i}{\pi d_b \cdot e} \cdot 10^3 = \frac{4M_i}{\pi d_b} \cdot \frac{\pi^2}{8d_b} \cdot 10^6 = \frac{\pi M_i}{2d_b^2} \cdot 10^6 = \pm f_{M;i} \tag{5.18}$$

The results of the conversion of the critical cross-sectional 1D point forces into line forces can be seen in Tables 5.3 (ULS) and 5.4 (SLS).

Table 5.3: Conversion of critical cross-sectional forces (ULS)

Combination	**V [kN]**	**M [kNm]**	**f_V [kN/m]**	**f_M [kN/m]**
N_{max}	15	2	59,68	±490,87
V_{max}	**109**	2	**433,70**	±490,87
M_{max}	69	**13**	274,54	**±3190,68**

Table 5.4: Conversion of critical cross-sectional forces (SLS)

Combination	**V [kN]**	**M [kNm]**	**f_V [kN/m]**	**f_M [kN/m]**
N_{max}	7	2	27,85	±490,87
V_{max}	**76**	1	**302,39**	±245,44
M_{max}	20	**6**	79,58	**±1472,62**

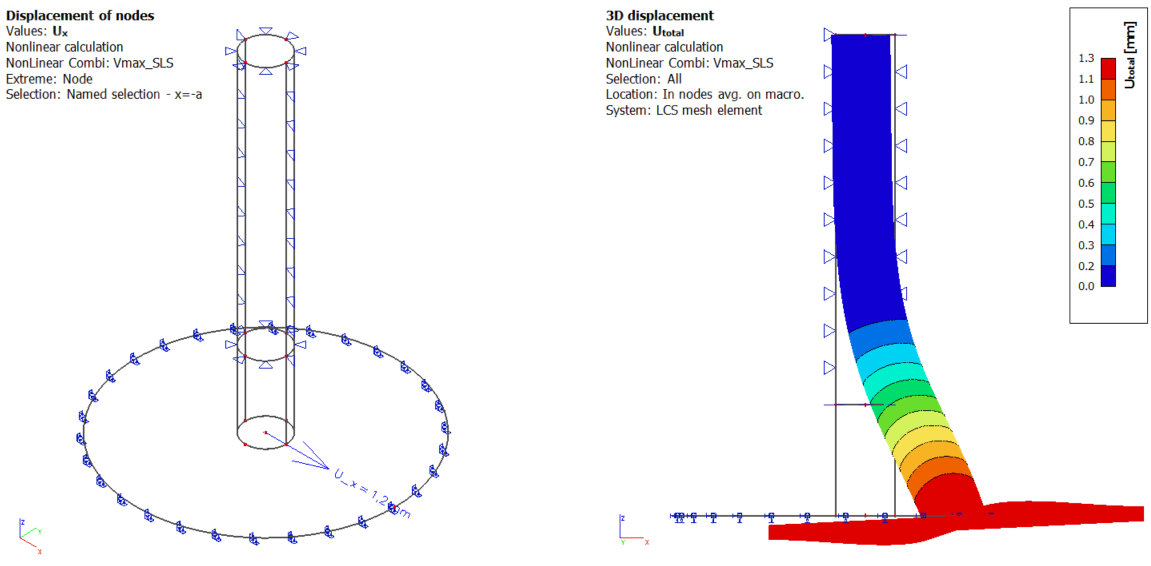
5.2.5. Step 5. Performing Relevant Design Checks

In order to verify the feasibility of the proposed demountable F2F dowel connection, two design checks are done. Firstly, it is verified that the deformation of the dowel (in SLS) doesn't exceed the predefined maximum allowed deformation of $w_{max} = 2{,}0$ mm, which has been determined based on an assumed thickness of the protective layer around the dowel over the length $a = 150$ mm. This check is done in order to ensure that the dowel end (i.e. at $x = -a$) doesn't make contact with the concrete as a result of the protective layer being completely compressed locally.

Secondly, it is checked that the contact between dowel and concrete doesn't result in large regions in the surrounding concrete over the length $b = 500$ mm, at which the contact stresses (in ULS) become equal to or larger than the concrete compressive strength of $f_c = 30$ N/mm^2. Ideally, the contact stresses remain below the concrete compressive strength for all load combinations, in which case the concrete remains to behave elastically, resulting in less or none local damage, such as crushing or abrasion of the concrete.

The maximum deformation (1,2 mm) and the maximum contact stress (28,4 N/mm^2) for the critical load combinations (V_{max} in SLS and in ULS respectively) are shown in Figures 5.10 and 5.11. It can be concluded that both checks are satisfied for the dowel connection with the properties and dimensions as stated in subsection 5.2.1 (page 110).

For more details of the dowel model, see the separate document "SCIA Engineering Report - Demountable F2F Dowel Model".



(a) Maximum deformation of dowel end

(b) Maximum (total) deformation of dowel connection

Figure 5.10: Maximum deformation of dowel connection in 2D and 3D (in SLS)

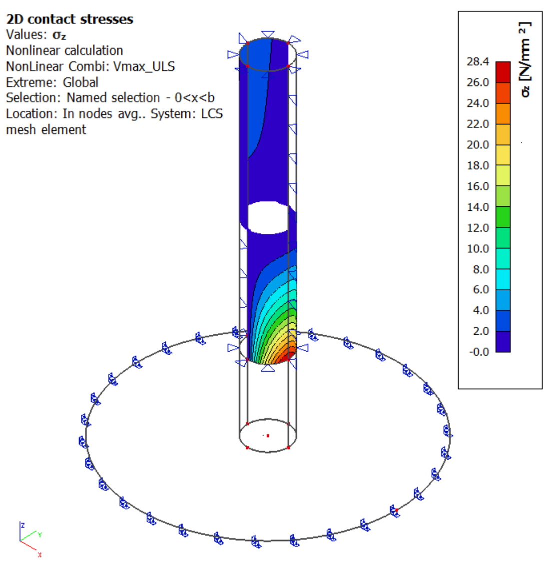


Figure 5.11: Maximum contact stress distribution at dowel to concrete interface (in ULS)

5.2.6. SCIA Model Check

In developing the dowel model, a number of assumptions and simplifications have been done which are important to be aware of, since these can potentially influence the (accuracy of the) results. Besides, SCIA Engineer has its limitations with regards to the type of non-linear interface modelling that is required to model and predict the behaviour of the demountable F2F connection more accurately. Therefore, the assumptions and simplifications included in the model as well as the limitations of the model are addressed below.

Foundation modulus of concrete under dowel action (k_c and k_d)

Most likely the most important variable is the foundation modulus of the surrounding concrete embedding under dowel action. In literature, only limited (experimental) research into this parameter and how to model the dowel action was found (see [49–51]), and therefore, before more test data are available, the empirical expression proposed by Soroushian et al. [49] (see equation (2.1)) is the best option to be used, according to X.G. He and A.K.H. Kwan [50].

Besides, sensitivity analyses of the replacing rotational spring stiffness of the dowel connection ($k_{r,con}$) and of the maximum dowel deformation (w_{max}) with respect to the foundation modulus of the surrounding concrete embedding (k_d) have been performed and are described in section 5.4.

Replacing rotational spring stiffnesses ($k_{r,plate}$ and $k_{r,con}$)

Even though the calculation of the replacing rotational spring stiffness of both the steel end plate and the demountable dowel connection has been executed conform the rules of basic structural mechanics (see e.g. [88]), it will always remain a simplification. However, comparing the results of the dowel model in SCIA Engineer with the calculated replacing rotational spring stiffness of the dowel connection ($k_{r,con} = 3069$ kNm/rad, see equation (5.7)), it can be concluded that the estimation is very close to the results obtained by the non-linear dowel model (see Tables 5.5 and 5.6), in which $k_{r,con,SCIA}$ is calculated as $k_{r,con,SCIA} = M/(\phi_{y,SCIA} \cdot 10^3)$. Therefore, it is concluded that the calculated replacing rotational spring stiffness of the dowel connection is sufficiently accurate.

Table 5.5: Check of replacing rotational spring stiffness from dowel model results (ULS)

Combination	M [kNm]	$\varphi_{y,SCIA}$ [mrad]	$k_{r,con,SCIA}$ [kNm/rad]	Difference [%]
N_{max}	2	0,6	3333	8,6
V_{max}	2	0,6	3333	8,6
M_{max}	13	4,0	3250	5,9

Table 5.6: Check of replacing rotational spring stiffness from dowel model results (SLS)

Combination	M [kNm]	$\varphi_{y,SCIA}$ [mrad]	$k_{r,con,SCIA}$ [kNm/rad]	Difference [%]
N_{max}	2	0,6	3333	8,6
V_{max}	1	0,3	3333	8,6
M_{max}	6	1,9	3158	2,9

Besides, the sensitivity of the replacing rotational spring stiffness of the dowel connection ($k_{r,con}$) with respect to the foundation modulus of the surrounding concrete embedding (k_d) is checked in section 5.4.

General assumptions and simplifications

Some more general assumptions and simplifications include:

- The geometry of the end plate ($t_{plate} = 20$ mm and $d_{plate} = Ø_{pile} = 508$ mm) are assumed values.
- The influence of the normal force on the dowel behaviour, which has not been taken into account in the model.

Limitations of the dowel model

The main limitation of the model results from the fact that it consists of 2D members which makes it practically impossible to model the actual interaction between the steel dowel and the surrounding concrete. It would require the use of a different finite element program such as DIANA FEA in order to accurately model this interaction by means of a 3D analysis with structural solid elements and structural interface elements.

Instead, the interaction between the steel dowel and concrete has been modelled by means of a subsoil with non-linear properties related to the surrounding concrete properties. Even though the behaviour itself of this non-linear subsoil has been verified to correspond to the intended input of the model, to a certain extent it has limited applicability. This simply results from the limits in SCIA Engineer regarding the possibilities in modelling this type of interactions between different interfaces.

5.3. Design Table for Parameter Ranges

Although the concept connection discussed in the previous section is intended to be applicable for a wide range of viaducts (hence the term 'standard viaduct'), one could imagine that the same connection with a slightly different geometry or different properties, or for a different range of cross-sectional forces could be interesting for further development. For that reason, it was decided to draft a design table for a range of five parameters.

5.3.1. Parameter Ranges and Combinations

The two parameters that influence both the design (geometry) as well as the results of the concept demountable connection the most are the the length over which the protective layer is being applied (a), and the dowel diameter (d_b). Besides, it has been chosen to include the concrete compressive strength (f_{ck}) as a variable in the design table as well, since it is the main material-related parameter which both directly (upper limit of the bi-linear stress-deformation curve) and indirectly (foundation modulus of the surrounding concrete embedding (k_c and k_d); also applies for dowel diameter (d_b)) influences the results. The fourth and fifth variables are the intended ranges of shear forces and bending moments that the connection should be able to withstand. These five parameters and their specified ranges and step sizes are shown in Table 5.7. As can be seen, the total number of combinations equals 138.908.

Table 5.7: Parameter ranges and number of combinations

Parameter	Lower limit		Upper limit		Step size		Number	
f_{ck}	30	N/mm²	60	N/mm²	5	N/mm²	7	
a	0,1	m	0,25	m	0,05	m	4	
d_b	50	mm	100	mm	5	mm	11	
F_0	0	kN	200	kN	5	kN	41	
M_0	0	kNm	50	kNm	5	kNm	11	×
			Total number of combinations:				138.908	

Because of this huge number of combinations, it was obvious that checking all of these manually in SCIA Engineer would practically be impossible. Therefore, a MAPLE script has been created which runs all these combinations, and which stores the maximum dowel deformation for each combination. The maximum dowel deformation was chosen as the indicator, since it provides a reasonable estimation for the feasibility of the connection with the specific geometry and properties despite the fact that the analytical model in MAPLE comprises of a linear calculation. However, as was stated by X.G. He and A.K.H. Kwan [50] (also see subsection 2.3.1), for relatively small dowel deformation, and provided that none of the materials have yielded, the dowel force-deformation relation is linearly elastic, and can therefore reasonably be estimated by using the beam on (linear) elastic foundation theory. A simplified version of the MAPLE script (calculation of one specific combination) has been added in Appendix N.

Finally, a sixth variable, which does not influence the number of combinations however, is the limit value for the maximum deformation of the dowel (w_{max}) that should be specified.

5.3.2. Design Table

The results (i.e. maximum deformations) of all combinations are stored in several Excel sheets, making up one big database. Subsequently, these results are filtered and arranged by means of an interactive tool developed in Excel, which is designed into a table. For a user-specified range of cross-sectional (design) forces and a specific concrete compressive strength, the maximum deformations for the four combinations of maximum and minimum cross-sectional (design) forces are filtered and compared to the specified limit value for the maximum dowel deformation for the 44 combinations of parameters a and d_b (respectively 4 × 11). If all four values are equal to or below the limit value for the maximum dowel deformation, it is reasoned that a dowel connection with that particular geometry and properties has the potential to be a feasible solution.

V = 0 - 125 kN		d_b																					
M = 0 - 40 kNm		50		55		60		65		70		75		80		85		90		95		100	
f_{ck} = 30 N/mm²		mm		mm		mm		mm		mm		mm		mm		mm		mm		mm		mm	
a	0,1 m	0,0	2,0	0,0	2,0	0,0	1,9	0,0	1,8	0,0	1,7	0,0	1,6	0,0	1,5	0,0	1,4	0,0	1,3	0,0	1,2	0,0	1,1
		2,2	0,7	2,0	0,5	1,7	0,3	1,6	0,3	1,4	0,3	1,3	0,3	1,2	0,3	1,1	0,3	1,0	0,3	0,9	0,3	0,9	0,2
	0,15 m	0,0	2,6	0,0	2,6	0,0	2,5	0,0	2,4	0,0	2,3	0,0	2,1	0,0	2,0	0,0	1,8	0,0	1,7	0,0	1,6	0,0	1,4
		3,8	1,6	3,2	1,1	2,8	0,7	2,5	0,5	2,2	0,4	2,0	0,3	1,8	0,3	1,7	0,2	1,5	0,2	1,4	0,2	1,3	0,2
	0,2 m	0,0	3,2	0,0	3,1	0,0	3,0	0,0	2,9	0,0	2,8	0,0	2,6	0,0	2,5	0,0	2,3	0,0	2,1	0,0	2,0	0,0	1,8
		6,2	3,2	5,1	2,2	4,3	1,5	3,7	1,1	3,3	0,8	2,9	0,6	2,6	0,5	2,4	0,4	2,2	0,3	2,0	0,3	1,8	0,3
	0,25 m	0,0	3,8	0,0	3,7	0,0	3,6	0,0	3,5	0,0	3,4	0,0	3,2	0,0	3,0	0,0	2,8	0,0	2,6	0,0	2,4	0,0	2,2
		9,3	5,7	7,5	4,0	6,3	2,8	5,3	2,1	4,7	1,6	4,1	1,2	3,7	0,9	3,3	0,8	3,0	0,6	2,7	0,5	2,5	0,5

Figure 5.12: Screenshot of the design table for a limit of the maximum dowel deformation of w_{max} = 2,0 mm

A screenshot of the design table for a range of $V = F_0 = 0 - 125$ kN and $M = M_0 = 0 - 40$ kNm, and $f_{ck} = 30$ N/mm^2 is shown in Figure 5.12. Besides, the limit value for the maximum dowel deformation has been chosen equal to the value adopted in the previous section, i.e. $w_{max} = 2{,}0$ mm.

It can for example be seen that, according to the analytical model, and for the specified range of cross-sectional forces and concrete compressive strength, a dowel diameter (d_b) of 55 mm or larger is sufficient when the length over which the protective layer is being applied (a) equals only 0,10 m, and besides that even the largest considered dowel diameter of 100 mm is not sufficient when a equals 0,25 m.

5.4. Sensitivity Analyses

In the foregoing sections, it has been noted several times already that the parameter that represents the concrete embedding surrounding the steel dowel, i.e. the foundation modulus of concrete under dowel action (k_c and k_d), is of considerable complexity and importance. Additionally, since a lot of uncertainty still exists with regards to this parameter, two sensitivity analyses have been performed with respect to this foundation modulus by means of the 1D analytical beam model (see Figure 5.4).

In Figure 5.13, the influence of the foundation modulus on the different steps of the verification process of the demountable F2F dowel connection is shown. From the flowchart, it becomes clear that the foundation modulus directly influences the replacing rotational spring stiffness of the dowel connection ($k_{r,con}$), which then potentially could result in considerable changes in the critical cross-sectional forces found in the SCIA Engineer model of the standard viaduct (N, V, and M). Therefore, the sensitivity of the replacing rotational spring stiffness of the dowel connection with respect to the foundation modulus is firstly investigated.

Besides, it can be seen in Figure 5.13 that the foundation modulus (obviously) has impact on the SCIA Engineer model of the dowel connection, since it is one of the inputs in the model, which is then used to check the maximum dowel deformation (w_{max}) and the contact stresses between steel dowel and concrete (σ_z). Since the contact stress can not directly be calculated by means of the 1D analytical beam model, the sensitivity of the maximum dowel deformation with respect to the foundation modulus is investigated secondly.

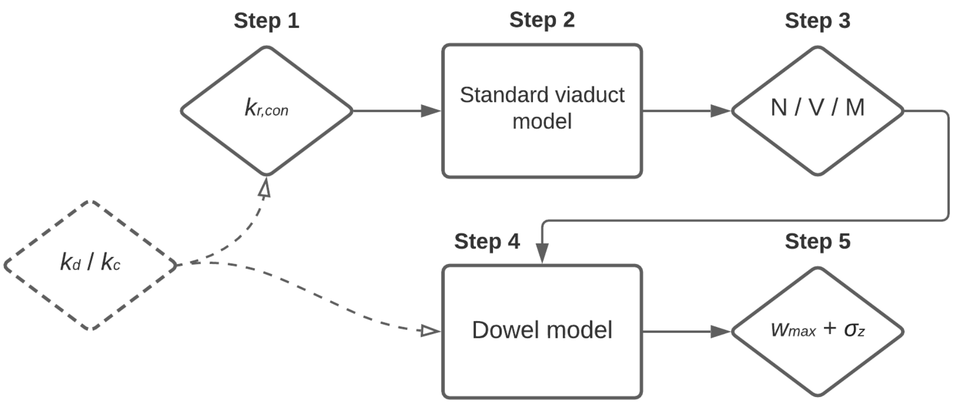


Figure 5.13: Influence of foundation modulus on verification process of demountable F2F dowel connection

5.4.1. Replacing Rotational Spring Stiffness

In order to investigate the sensitivity of the replacing rotational spring stiffness of the dowel connection, expression (5.5) has been plotted for a range of values for the foundation modulus of the concrete embedding, ranging from an upper and lower limit equal to a factor 10^2 times larger and smaller respectively compared to the reference value as was calculated in subsection 5.2.1, i.e. $k_{d,0} = k_d = 3{,}00 \cdot 10^6$ kN/m^2. Similarly, the same parameter and variable values were adopted as were introduced in subsection 5.2.1, keeping only k_d (included in expression (5.5) by means of λ) variable. This results in the following expression for the replacing rotational spring stiffness:

$$k_{r,con} = \frac{(a \cdot k_{r,plate} + EI)\lambda + k_{r,plate}}{a \cdot \lambda + 1} = \frac{105{,}17\sqrt[4]{k_d} + 1680}{0{,}023\sqrt[4]{k_d} + 1} \tag{5.19}$$

Subsequently, the expression is normalised with respect to the replacing rotational spring stiffness corresponding to the reference value of the foundation modulus (i.e. $k_{r,con}(k_d = k_{d,0}) = 3069$ kNm/rad; see equation (5.7)), resulting in the final expression for the relative replacing rotational spring stiffness of the dowel connection with respect to the foundation modulus:

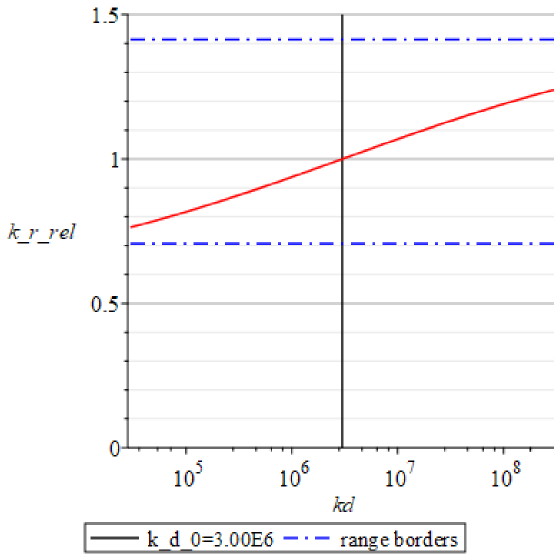


Figure 5.14: Relative replacing rotational spring stiffness with respect to foundation modulus of concrete under dowel action

$$k_{r,rel} = \frac{k_{r,con}(k_d)}{k_{r,con}(k_d = k_{d,0})} = \frac{105{,}17\sqrt[4]{k_d} + 1680}{3069\left(0{,}023\sqrt[4]{k_d} + 1\right)} = \frac{105{,}17\sqrt[4]{k_d} + 1680}{71{,}80\sqrt[4]{k_d} + 3069} \tag{5.20}$$

The graph of expression (5.20) is shown in Figure 5.14 with the foundation modulus (on the x-axis) plotted on a logarithmic scale. The two horizontally dotted blue lines indicate the upper and lower range borders equal to $\sqrt{2}$ and $\sqrt{2}/2$ respectively. Since the graph stays within both horizontal ranges, it can be concluded that the replacing rotational spring stiffness for the upper limit of k_d is less than 2 times larger than for the lower limit of k_d. In other words, it shows that for a range of a factor $10^2 \cdot 10^2 = 10^4 = 10.000$ for k_d, the maximum and minimum resulting rotational spring stiffness differ less than a factor 2.

From this analysis, it has therefore been concluded that the sensitivity of the replacing rotational spring stiffness ($k_{r,con}$) with respect to the foundation modulus of the surrounding concrete embedding is negligible. Besides, the influence of a different foundation modulus on the critical cross-sectional forces obtained from the standard viaduct model (N, V, and M; see Figure 5.3) was investigated. Concretely, the stiffness of the rotational springs in the model was changed to both the upper and lower limit value corresponding to the upper and lower limit values of $k_{r,rel}$, and for both limit cases, the critical cross-sectional forces were compared to the forces shown in Figure 5.7. A deviation of less than 5% was observed on average, and therefore it was concluded that the sensitivity of the critical cross-sectional forces with respect to the foundation modulus are also negligible.

5.4.2. Maximum Dowel Deformation

The sensitivity of the maximum dowel deformation[2] (w_{max}) with respect to the foundation modulus of the concrete embedding has been investigated by comparing the relative maximum dowel deformation for different ratios of applied shear force and bending moment. This ratio is expressed with parameter α and yields $\alpha = \frac{F_0}{M_0} \geq 0$. Similar to the analysis of the replacing rotational spring stiffness, the variable of interest (i.e. the maximum dowel deformation) has been normalised with respect to the maximum dowel deformation corresponding to the reference value of the foundation modulus (i.e. w_{max} for $k_d = k_{d,0}$):

$$w_{rel} = \frac{w_{max}(k_d)}{w_{max}(k_d = k_{d,0})} \tag{5.21}$$

Again, the same parameter and variable values are adopted as were introduced in subsection 5.2.1, keeping only k_d variable. However, different from the analysis of the replacing rotational spring stiff-

[2]The maximum dowel deformation is assumed to be found at $x = -a$ (see Figure 5.4), i.e. at the dowel end where it is connected to the plate, which (theoretically) holds for most of the cases

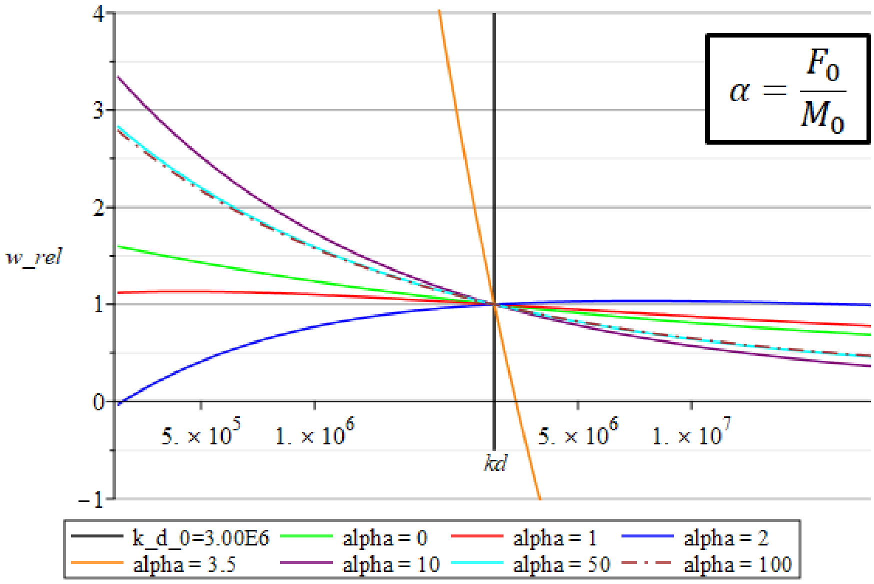


Figure 5.15: Relative maximum dowel deformation with respect to foundation modulus of concrete under dowel action

ness, the sensitivity has been investigated for a range of values for the foundation modulus, ranging from an upper and lower limit equal to a factor 10^1 times larger and smaller respectively compared to the reference value $k_{d,0} = 3{,}00 \cdot 10^6$ kN/m^2.

The graphs for different values of α are shown in Figure 5.15. From studying the behaviour for a continuous range of values of α between 0 and 100 (i.e. by means of an animation), it was observed that the largest influence is found for values of α ranging between 1 and 10, i.e. between $F_0 = M_0$ and $F_0 = 10M_0$. For values of α smaller than 1 and larger than 10 hardly any variation in sensitivity is observed anymore, except for values of k_d close to the lower limit, i.e. $k_{d,min} = k_{d,0} \cdot 10^{-1} = 3{,}00 \cdot 10^5$ kN/m^2. For values of k_d larger than $k_{d,0}$ and close to the upper limit in general, i.e. for $k_{d,max} = k_{d,0} \cdot 10^1 = 3{,}00 \cdot 10^7$ kN/m^2, it can be observed that the sensitivity is much lower for all values of α.

A particular result, however, is found for a value of α (close or) equal to $\alpha = 3{,}5$, which appears to approach an asymptotic value. The explanation for this result is relatively simple, as it represents a ratio of shear force and bending moment which results in a maximum dowel deformation (close or) equal to zero for the reference value of the foundation modulus, i.e. w_{rel} is a division by (a value close to) zero which results in an asymptote. Similarly, it can be reasoned that the opposite can occur as well, i.e. $w_{max} = 0$ for a certain value of α, and a value of k_d different than $k_{d,0}$, which results in $w_{rel} = 0$. In Figure 5.15, an example of this can be observed for the case wherein $\alpha = 2$ and $k_d \approx k_{d,min}$.

The explanation above can be visually clarified by means of Figure 5.16, in which the absolute value of the dowel deformation for the earlier specified range of shear forces and bending moments has been plotted for different values of the foundation modulus. It can be seen that for a certain combination of shear force and bending moment (i.e. a certain value of α) and foundation modulus, the deformation equals zero. These particular α-values can be graphically estimated from the points of intersection of each plane with the F_0- and M_0-axis:

$$\alpha_{kd,min} = \frac{F_0}{M_0} \approx \frac{100}{50} = 2{,}0 \tag{5.22}$$

$$\alpha_{kd,0} = \frac{F_0}{M_0} \approx \frac{170}{50} = 3{,}4 \tag{5.23}$$

$$\alpha_{kd,max} = \frac{F_0}{M_0} \approx \frac{200}{40} = 5{,}0 \tag{5.24}$$

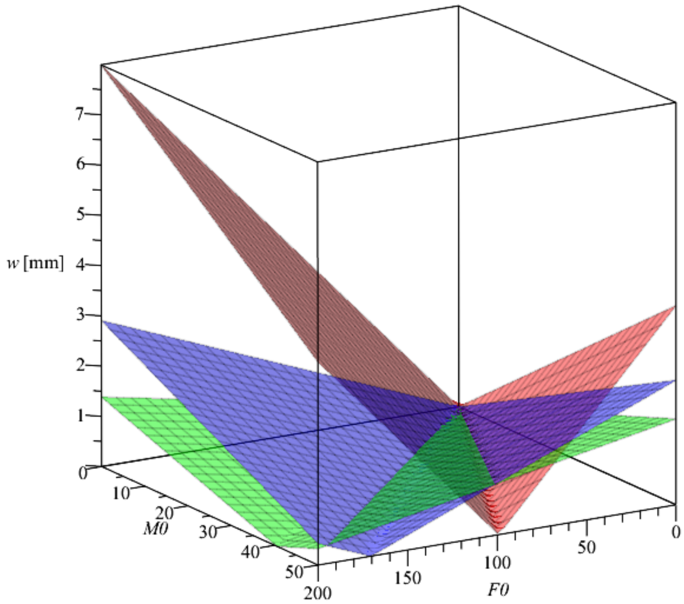


Figure 5.16: Absolute dowel deformation for range of shear forces and bending moments, for $k_{d,min}$ (red), $k_{d,0}$ (blue), and $k_{d,max}$ (green), and limit deformation w_{max} = 2 mm as a reference value

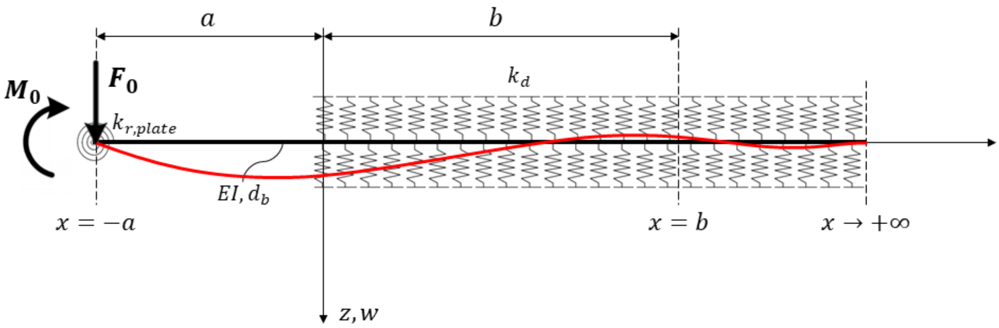


Figure 5.17: Schematic impression of scenario for which maximum dowel deformation is not found at $x = -a$

Note that the α-values approximated by equations (5.22) and (5.23) can be recognised from the α-values that were discussed two paragraphs above (i.e. $\alpha = 2$ and $\alpha = 3{,}5 \approx 3{,}4$).

Besides, it can now more easily be understood that for certain values of α the maximum dowel deformation is not found at $x = -a$, but somewhere else on the interval $-a \leq x \leq 0$. An example of such a scenario is shown in Figure 5.17. It has been verified however that the cases for which the maximum deformation is not found at $x = -a$ only occur for a very small range of α-values.

Finally, from this second analysis, it can be concluded that for the lower limit range of the foundation modulus (i.e. $k_{d,min} \leq k_d \leq k_{d,0}$), the (relative) maximum dowel deformation is sensitive, especially for values of α between 1 and 10. For the upper limit range of the foundation modulus (i.e. $k_{d,0} \leq k_d \leq k_{d,max}$), however, a substantially lower sensitivity was observed generally. This result is not unexpected, since this type of sensitivity had been observed already in a (limited) variation study with respect to the (non-linear) subsoil (i.e. foundation modulus) in the dowel model in SCIA Engineer. Therefore, despite the fact that the 1D analytical beam model is limited to a linear elastic calculation, and therefore loses accuracy when deformations get larger, it turns out that the beam on (linear) elastic foundation theory can reasonably be used in order to assess the sensitivity of the maximum dowel deformation. Besides, the particular results (i.e. the asymptotic values) have been discussed and their occurrence was logically explained.

5.5. Execution Variants and Practical Issues

The development, but potentially even more the realisation, of a circular concrete viaduct, and in particular of the concept demountable footing to foundation (F2F) dowel connection, comes with a number of important practical issues with regards to the (sequence of) (de)construction activities. Therefore, a proposed (dis)assembly sequence is drafted first for each execution variant (see Figure 5.2), and besides the most important and obvious practical issues related to these activities have been tried to identify and potential solutions are proposed. Subsequently, the individual tolerances (i.e. permitted deviations) to be considered in the design of the demountable F2F connection are identified, and for each variant, the critical scenario for the accumulation of these deviations into a construction tolerance is calculated. Finally, a number of advantages and disadvantages of each execution variant are listed.

5.5.1. (De)construction

In order to ensure a safe and correct process of (de)construction for both the circular viaduct as a whole as well as the demountable F2F dowel connection, the regulations for the execution of concrete structures (see NEN-EN 13670 [IX]) should be followed in general. However, since no regulations are known to exist with regards to the specific topic of deconstruction, and since the construction activities for a circular viaduct are also likely to differ at least to a certain extent from the construction activities for a traditional viaduct, it is discussed in this subsection how to possibly deal with these differences.

In section 5.1, three different variants for the execution of the demountable F2F dowel connection were introduced. In Tables 5.8 and 5.9, for each of these variants, the proposed sequence of activities for respectively construction and deconstruction are listed, in which the focus is on the (de)construction (sequence of) activities related to the demountable F2F connection. Essentially, Table 5.9 can therefore be seen as a first draft version of a deconstruction plan (specific to the demountable F2F dowel connection), of which the importance was emphasised on in particular in subsection 2.1.3.

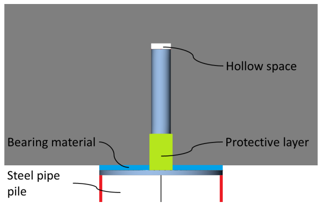
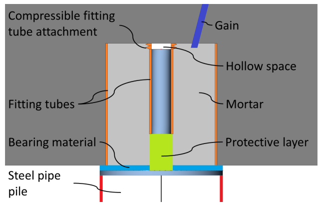
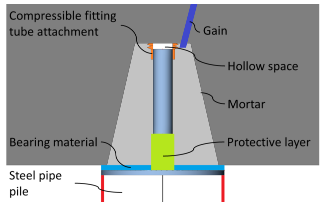
General practical issues

The most obvious general practical issues have to do with the so-called ‘handling’-related activities. The collective term ‘handling’ in this context is used to refer to the processes of production, transportation, storage and (dis)assembly (e.g. hoisting) of elements and components of the circular concrete viaduct in general. These issues mainly relate to the dimensions and weight of the different elements and components of which the circular viaduct exists and which were identified in subsection 3.1 (see Table 3.1). The dimensions and weight of each of these elements and components have to be within specified maximum allowed limits and capacities of, for example, (mobile) cranes, both on and off the building site. Regulations for handling (mainly related to storage and assembly) of precast concrete elements are, for example, listed in NEN-EN 13670 [IX]. Besides, there are several other standards and guidelines related to the topic of handling precast concrete elements.

As mentioned, the issues mainly relate to the dimensions and weight of elements and components. With regards to transportation regulations in the Netherlands, it broadly yields that the maximum load on a truck equals 300 kN, the maximum width that can be transported by road without special measures is 3,5 m, and the maximum height is 4,20 m, whereas the length in principle is unlimited. With regards to production and assembly, the main limiting factor is the hoisting capacity in the factory and on-site respectively [89]. All of these limits should therefore be carefully taken into account in the overall design of the circular concrete viaduct.

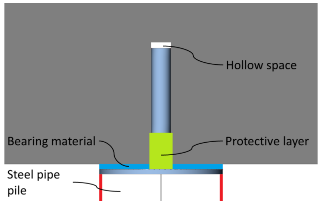
Furthermore, in terms of constructability, it becomes clear that execution variant 1 could solely be applied in a (theoretical) perfect situation, which in real life obviously is not the case. The only sequence of construction that might allow it to be possible to construct would require the dowels to be inserted in the mould during casting of the (abutment) footings to ensure a perfect fit, and to be welded on-site to the end plates, which then already should have been welded beforehand onto the installed foundation piles. This issue is discussed in more detail in “Horizontal tolerances – Variant 1” under subsection “Construction tolerances” (see page 130). However, this procedure would imply that welding of the dowels to the end plates would need to be done below the (abutment) footing, which not only would involve practical issues, but (maybe even more importantly) also safety issues.

Table 5.8: Assembly sequence specified for each execution variant of demountable F2F dowel connection

Variant 1	**Variant 2**	**Variant 3**
		
	Construction activities	
1. Installation of steel pipe foundation piles	1. Installation of steel pipe foundation piles	1. Installation of steel pipe foundation piles
2. Placing of prefabricated end plates with welded dowels including protective layer	2. (a) Placing of prefabricated end plates with welded dowels including protective layer and (‘inner’) (compressible) fitting tubes (b) Welding of end plates to piles after aligning of (oversized) holes and dowels to a certain level of accuracy[1]	2. (a) Placing of prefabricated end plates with welded dowels including protective layer and compressible fitting tube attachment (b) Welding of end plates to piles after aligning of (oversized, cone-shaped) holes and dowels to a certain level of accuracy[1]
3. Evening out vertical differences by means of non-shrink mortar	3. Evening out vertical differences by means of non-shrink mortar	3. Evening out vertical differences by means of non-shrink mortar
4. Placing of bearing material	4. Placing of bearing material	4. Placing of bearing material
5. (a) Hoisting and aligning of prefabricated (abutment) footing, and at the same time aligning end plates with welded dowels accordingly[1] (b) Welding of end plates to piles once holes and dowels are aligned (c) Lowering of (abutment) footing over dowels onto bearing material into its final position	5. (a) Hoisting, aligning, and lowering of prefabricated (abutment) footing including (‘exterior’) fitting tubes over dowels onto bearing material into its final position (b) Filling voids between both fitting tubes with mortar through gains	5. (a) Hoisting, aligning, and lowering of prefabricated (abutment) footing over dowels onto bearing material into its final position (b) Filling (cone-shaped) voids with mortar through gains
6. Construction of remaining (parts of) viaduct	6. Construction of remaining (parts of) viaduct	6. Construction of remaining (parts of) viaduct

[1]Diameter of end plate should be larger than diameter of pile for this to be possible (see “Horizontal tolerances – Variant 2” under subsection “Construction tolerances” (page 130))

Table 5.9: Disassembly sequence specified for each execution variant of demountable F2F dowel connection

<table>
<tr><th>Variant 1</th><th>Variant 2</th><th>Variant 3</th></tr>
<tr>
<td>
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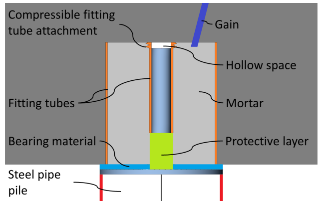
<td>
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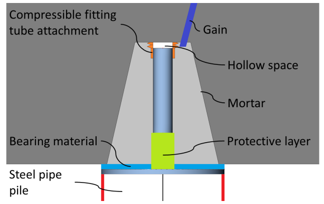
<td>
</td>
</tr>
<tr><th colspan="3">Deconstruction activities</th></tr>
<tr>
<td>1. Deconstruction of (components of) viaduct until (abutment) footings
2. Digging up (abutment) footings
3. Lifting and removing of (abutment) footing
4. Removing of bearing material
5. Crushing and removing of mortar layer used for evening out vertical differences
6. Cutting of welds between end plates and piles, and removing end plates with welded dowels and bearing material
7. Retract foundation piles from ground</td>
<td>1. Deconstruction of (components of) viaduct until (abutment) footings
2. Digging up (abutment) footings
3. (a) Lifting and removing of (abutment) footing
(b) Removing of mortar infill
4. Removing of bearing material
5. Crushing and removing of mortar layer used for evening out vertical differences
6. Cutting of welds between end plates and piles, and removing end plates with welded dowels and bearing material
7. Retract foundation piles from ground</td>
<td>1. Deconstruction of (components of) viaduct until (abutment) footings
2. Digging up (abutment) footings
3. (a) Lifting and removing of (abutment) footing
(b) Crushing and removing of mortar infill
4. Removing of bearing material
5. Crushing and removing of mortar layer used for evening out vertical differences
6. Cutting of welds between end plates and piles, and removing end plates with welded dowels and bearing material
7. Retract foundation piles from ground</td>
</tr>
</table>

Besides, with regards to execution variant 3, it has to be assured that during deconstruction the mortar infill releases smoothly and effortlessly from the (abutment) footing without damaging the concrete surface of the cone-shaped holes. Therefore, it is suggested to investigate the possibility to lubricate the surface of the cone-shaped holes with demoulding oil during construction, similar to what was done in the realised prototype circular viaduct (see subsection 2.2.2). Similarly, with regards to execution variant 2, it is suggested to investigate the necessity and possibility to lubricate the fitting pipes in order to assure a smooth and effortless release of the mortar infill from the (abutment) footing.

Finally, a general practical issue that should be considered, especially during deconstruction, is the careful handling of elements and components in order to prevent (excessive) damaging of these. Regarding the demountable F2F connection, damage to different parts of the connection is, for example, especially prone to occur while crushing off the mortar from the dowels (in execution variant 3) and while cutting the welds between the end plates and foundation piles. Besides, one might want to inspect or test some of the elements and components of the viaduct or parts of the demountable connections after deconstruction, which also requires an additional level of attention during deconstruction (also see Chapter 6).

Installation and retraction of foundation piles

With regards to the installation of foundation piles, in particular of raking piles, a solution should be provided to realise a horizontal plane at the pile head in order to be able to weld the end plates with the welded dowels onto the piles. The most obvious solution seems to be to cut the raking piles at an angle, however, one could also think of providing the end plates with a specific supplementary steel part, aligned with the inclination of the raking pile.

Besides, for deconstruction purposes related to the circular viaduct in general, it is relevant to have an idea of the force that is needed to retract the foundation piles from the ground. It is reasoned that this force depends on the way of retraction, e.g. by means of pulling, vibrating or screwing out the piles. Of these three examples, it is expected that the first would require the most force since both the weight of the pile and the pulling resistance has to be overcome, whereas in the other two examples, the pulling resistance would be (largely) taken away by the vibrations/screwing rotations, resulting in the (expected) required force just needing to be slightly larger than the weight of the pile.

Horizontal and vertical alignment

Another issue with regards to the installation of the foundation piles are the inevitable horizontal deviations that will exist between the theoretical and the real alignment of the pile heads. In order to be able to compensate (the largest part of) these deviations, it is suggested to manufacture oversized end plates in order to provide tolerance to align the holes and dowels to a certain allowable maximum deviation that can be incorporated.

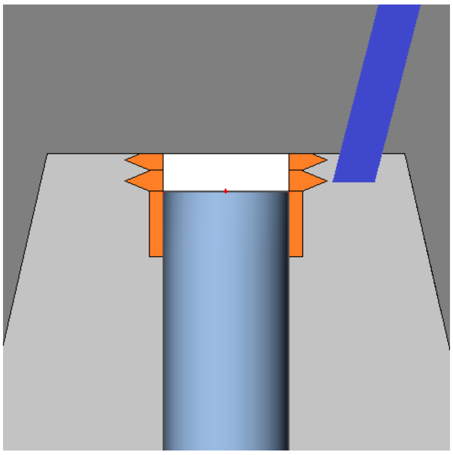
Besides horizontal deviations, inevitably also vertical differences will exist between the pile head levels. In order to realise a horizontally levelled supporting plane, a straightforward solution seems to be to level out these differences by means of applying non-shrink mortar on top of the end plates with welded dowels, after these have been welded onto the foundation piles. Then, after hardening of the mortar, the bearing material can be applied on top of the mortar.

The horizontal and vertical deviations that should be taken into account are addressed in subsection 5.5.2 as well as the way how these deviations in each execution variant can potentially accumulate.

Filling mortar for voids

With regards to the mortar used to fill the voids in execution variants 2 and 3, it is recommended to apply a mortar with a compressive strength of a similar magnitude as the compressive strength of the concrete used, since otherwise this could potentially influence the foundation modulus of the surrounding concrete embedding under dowel action, and subsequently the behaviour of the demountable F2F connection significantly (see equations (5.11) and (5.12), and Figure 5.3). As long as the compressive strength of the applied mortar is within a ±20–25% range compared to the concrete compressive strength, the deviation in the value of the foundation modulus is approximately ±10–15%, the influence

(a) Concept of compressible fitting tube attachment [91]



(b) Detail of concept solution

Figure 5.18: Impression of solutions to provide hollow space at top of dowel

of which on the relevant design checks (see subsection 5.2.5) is expected to be negligible, based on the sensitivity analysis of the maximum dowel deformation with respect to the foundation modulus (see Figure 5.15). In case the compressive strength of the mortar deviates significantly (> ±25%) from the concrete compressive strength, one should take this into account during the verification of the connection by adopting the compressive strength of the mortar in the calculation (i.e. $f_c = f_{c,mortar}$). Values for the compressive strength of different mortars are directly related to their European designation (e.g. M 2,5, M 5, M 20, etc.), and range from 2,5 to 20 N/mm^2 and higher [90].

Besides, it has to be assured that the top of the dowel (i.e. at $x = b$, see Figures 5.4 and 5.5) in none of the execution variants makes contact with concrete or mortar, since this would imply that normal (vertical) force would be transferred here, which has not been accounted for. Instead, all normal forces are supposed to be transferred at the concrete to steel end plate interface. In order to assure no contact between the top of the dowels and concrete/mortar, a potential solution for execution variants 2 and 3 is thought to be to provide the top of the dowels with a compressible (plastic/PVC) tube (see Figure 5.18a). In this way, a sealed space would be created when lowering the (abutment) footing over the dowels, and no mortar could flow over the dowels (see Figure 5.18b). Considering execution variant 1, the obvious solution would be to assure the holes to be deep enough in order to make it physically impossible for the top of the dowels to make contact with the concrete above.

5.5.2. Tolerances

In the foregoing subsection, the most important and obvious practical issues related to the (de)construction activities for each execution variant have been discussed. Most of these issues directly relate to the tolerances that should be incorporated in the design of each variant, as well as how deviations potentially do or do not accumulate depending on the (dis)assembly sequence. Therefore, all tolerances that should be incorporated in the design of the demountable F2F dowel connection should be identified and their (individual) magnitude quantified. Subsequently, based on the proposed (dis)assembly sequence, the critical scenario for each variant regarding the potential accumulation of deviations can be identified in order to calculate the overall tolerances (i.e. construction tolerances) that should be incorporated in the design.

Individual tolerances

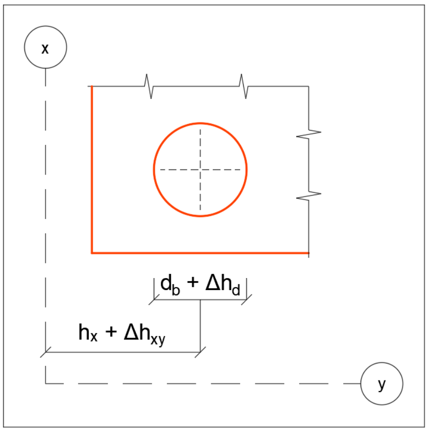
Requirements with regards to tolerances are established in different building codes, depending on their origin. Related to the demountable F2F dowel connection, four main tolerances have been identified which are shown in Table 5.10. In Figure 5.19, the types of deviations are depicted for clarification.

Regarding the tolerances on the holes in the (abutment) footing, it is worth mentioning that two different values for the permitted deviation of the location of the holes in the (abutment) footing were found in two different building codes. Furthermore, it can be noted that most regulations are specific for horizontal

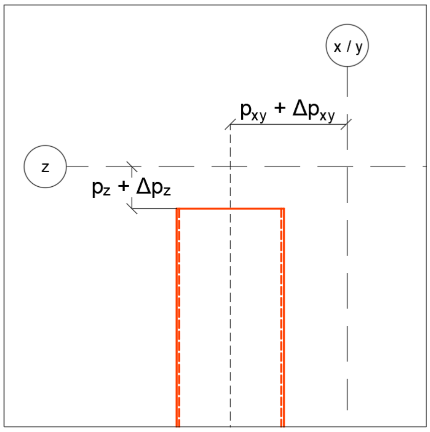
Table 5.10: Identification of (individual) tolerances related to demountable F2F dowel connection

Fig. 5.19	Description	Type of deviation	Permitted deviation	Reference (NEN-EN #)
(a)	Tolerances on holes in (abutment) footing	Deviation of location of hole	$\Delta h_{xy} = \pm 25$ mm[1] $\Delta h_{xy} = \pm 30$ mm	13670, Figure G.6 [IX] 15050, art. 4.3.1.2 [X]
		Deviation of hole diameter	$\Delta h_d = \pm 10$ mm[1]	13670, Figure G.6 [IX]
(b)	Tolerances on installation of foundation piles	Deviation of location of pile head at working level	$\Delta p_{xy} = e \leq 0{,}1$ m	12699, art. 8.2.1 [VIII]
		Deviation of pile head level	$\Delta p_z \leq 25$ mm	N / A
(c)	Tolerances on dowel to plate connection	Deviation of location of dowel on plate	$\Delta d_{xy} = \pm 3{,}0\text{–}5{,}0$ mm[2]	1090-2, Table B.15 [V]
		Skewness of dowel on plate	$\Delta d_s = \pm L/300 = \pm 2{,}2$ mm[3]	1090-2, Table B.17 [V]
(d)	Tolerances on dowel dimensions and shape	Deviation of dowel diameter	$\Delta d_d = \pm 0{,}5\text{–}1{,}0$ mm	10060, Table 1 [VII]
		Deviation of dowel length	$\Delta d_L = \pm 25$ mm	10060, Table 4 [VII]
		Deviation of dowel straightness	$\Delta d_q = q \leq 0{,}4\%$ of $L = 2{,}6$ mm	10060, Table 3 [VII]

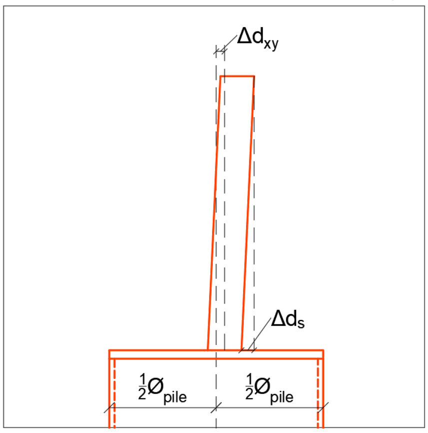
[1] Unless otherwise stated in the execution specification
[2] Based on column slice connection in buildings
[3] Based on inclination of columns in single-storey buildings



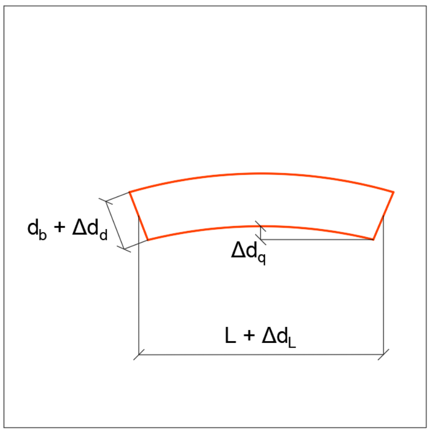
(a) Tolerances on holes in (abutment) footing



(b) Tolerances on installation of foundation piles



(c) Tolerances on dowel to plate connection



(d) Tolerances on dowel dimensions and shape

Figure 5.19: Types of deviations

deviations, since the deviations of the pile head level and of the dowel length are the only two identified types of deviation that relate to a vertical deviation. Besides, requirements for the deviation of the pile head level were not even found in the different building codes. Therefore, a permitted deviation for this deviation has been established based on experience by engineers at Lievense for cutting of steel pipe pile heads. Finally, it is emphasised that the permitted deviations with regards to the tolerances on the dowel to plate connection are based on regulations with respect to buildings. However, regulations for deviations with respect to (the very specific application of) dowel to plate connections are not known to exist, and therefore the regulations with respect to buildings have been adopted.

Construction tolerances

According to NEN-EN 13670 [IX], construction tolerances are the tolerances that are a combination of production, site construction, and erection tolerances, or, in other words, the accumulated (overall) tolerances to be incorporated in the design. It depends mainly on the assembly sequence if and how deviations (i.e. tolerances) can possibly accumulate, and therefore, the way how this is translated into construction tolerances that should be included in the design of the demountable F2F dowel connection also depends directly on the assembly sequence. Therefore, for each execution variant, relevant values for the construction tolerances to be considered are proposed, based on the sequence of construction activities described in Table 5.8.

Horizontal tolerances – Variant 1

As was already discussed in subsection 5.5.1, variant 1 as described in Table 5.8 could solely be executed in a (theoretical) perfect situation, since it is practically impossible to both be able to align all dowels and holes and to guarantee a perfect fit. Therefore, the construction tolerances to be included in the design are based on the description of the only practically possible way of executing variant 1, namely by means of welding of the dowels to the end plates on-site below the (abutment) footing.

In this case, however, it can be concluded that accumulation of the possibly occurring horizontal deviations is not an issue, since the permitted deviation of the location of the pile heads at working level (Δp_{xy}) and the deviation of the location of the holes (Δh_{xy}) maximally result in a horizontal construction tolerance ($\Delta 1_{xy}$) at the dowel to plate interface of:

$$
\begin{array}{rcll}
\Delta h_{xy} & = & 25\text{–}30 \text{ mm} & \text{(Deviation of location of hole)} \\
\Delta h_{d} & = & \text{N / A} & \text{(Deviation of hole diameter)} \\
\Delta p_{xy} & = & 100 \text{ mm} & \text{(Deviation of location of pile head at working level)} \\
\Delta d_{xy} & = & \text{N / A} & \text{(Deviation of location of dowel on plate)} \\
\Delta d_{s} & = & \text{N / A} & \text{(Skewness of dowel on plate)} \\
\Delta d_{d} & = & \text{N / A} & \text{(Deviation of dowel diameter)} \\
\Delta d_{q} & = & \text{N / A mm} \quad + & \text{(Deviation of dowel straightness)} \\
\hline
\Delta 1_{xy} & = & 125\text{–}130 \text{ mm} &
\end{array}
$$

This deviation could easily be accommodated on the end plate and would merely result in a dowel-to-plate/dowel-to-pile eccentricity of magnitude $\Delta 1_{xy}$ (see Figure 5.20a). Therefore, no additional measures would have to be taken to compensate for this horizontal construction tolerance.

Besides, regarding the horizontal deviations of the hole diameter (Δh_d) and of the dowel dimensions and shape (Δd_d and Δd_q), it can be concluded that in this (theoretical) scenario these deviations also wouldn't require any additional measures since the dowels would be included in the mould (i.e. match-casting). Finally, regarding the horizontal deviations of the dowel to plate connection (Δd_{xy} and Δd_s), it is obvious that these tolerances are not relevant, since this welded connection would be made in-situ.

Horizontal tolerances – Variant 2

With regards to execution variant 2, it is reasoned that any deviation of the location of the pile heads at working level within the permitted boundary value (i.e. $\Delta p_{xy} \leq 0{,}1$ m) can be compensated in-situ by means of applying an oversized end plate, as was proposed already in subsection 5.5.1, in order to be able to align the (oversized) holes and dowels to a certain level of accuracy before welding the plates to the piles (see Figure 5.20b). This implies, however, that the diameter of the end plate should (at least) be increased with $\Delta d_{plate} \geq 2 \cdot \Delta p_{xy} = 0{,}2$ m, and besides, it would again merely result in a dowel-to-pile eccentricity with a maximum magnitude equal to Δp_{xy}.

In this case, the accumulation of the horizontal deviations that are permitted maximally results in a horizontal construction tolerance ($\Delta 2_{xy}$) of:

$$
\begin{array}{rcll}
\Delta h_{xy} & = & 25\text{–}30 \text{ mm} & \text{(Deviation of location of hole)} \\
\frac{1}{2}\Delta h_d & = & 5 \text{ mm} & \text{(Deviation of hole diameter)} \\
\Delta p_{xy} & = & \text{N / A} & \text{(Deviation of location of pile head at working level)} \\
\Delta d_{xy} & = & \text{N / A} & \text{(Deviation of location of dowel on plate)} \\
\Delta d_s & = & 2{,}2 \text{ mm} & \text{(Skewness of dowel on plate)} \\
\frac{1}{2}\Delta d_d & = & 0{,}25\text{–}0{,}5 \text{ mm} & \text{(Deviation of dowel diameter)} \\
\Delta d_q & = & 2{,}6 \text{ mm} \quad + & \text{(Deviation of dowel straightness)} \\
\hline
\Delta 2_{xy} & = & 35{,}1\text{–}40{,}3 \text{ mm} &
\end{array}
$$

This has to be compensated by providing sufficient space in the oversized holes (see Figure 5.20b). Therefore, it is proposed to fabricate holes with a diameter of:

$$h_d^* \geq (d_b + 2 \cdot t_{fp}) + 2 \cdot \Delta 2_{xy} = (80 + 2 \cdot 2) + 2 \cdot 40{,}3 = 164{,}6 \text{ mm} \rightarrow h_d^* = 165 \text{ mm}$$

Here, h_d^* is the diameter of the hole measured between the exterior fitting tube, and t_{fp} is the thickness of the (interior) fitting pipe, which is assumed to be 2 mm. Besides, it is noted that this diameter is smaller than the minimum diameter of the end plate, and therefore the holes are completely closed off at the bottom at all times which makes it possible to fill the voids of the oversized holes with mortar without having to take additional measures (see Figure 5.20b).

Horizontal tolerances – Variant 3

The reasoning for the horizontal construction tolerances to be incorporated for execution variant 2 also applies to execution variant 3. However, one additional point of attention concerns the angle of the cone-shaped hole. Since the influence of the angle has not been structurally assessed, here solely the limit value is calculated, based on the requirement to have the holes at all times closed off at the bottom by the steel end plate in order to assure that the voids of the oversized holes can be filled with mortar without having to take additional measures. This results in a minimally required angle of (see Figure 5.20c):

$$\beta_{min} = \tan^{-1}\left(\frac{L + \Delta p_z + \Delta d_L}{(\text{Ø}_{pile} + 2 \cdot \Delta p_{xy} - h_d^*)/2 - \Delta h_{xy}}\right) = \tan^{-1}\left(\frac{650 + 25 + 25}{(508 + 2 \cdot 100 - 165)/2 - 30}\right) \approx 70^\circ \tag{5.25}$$

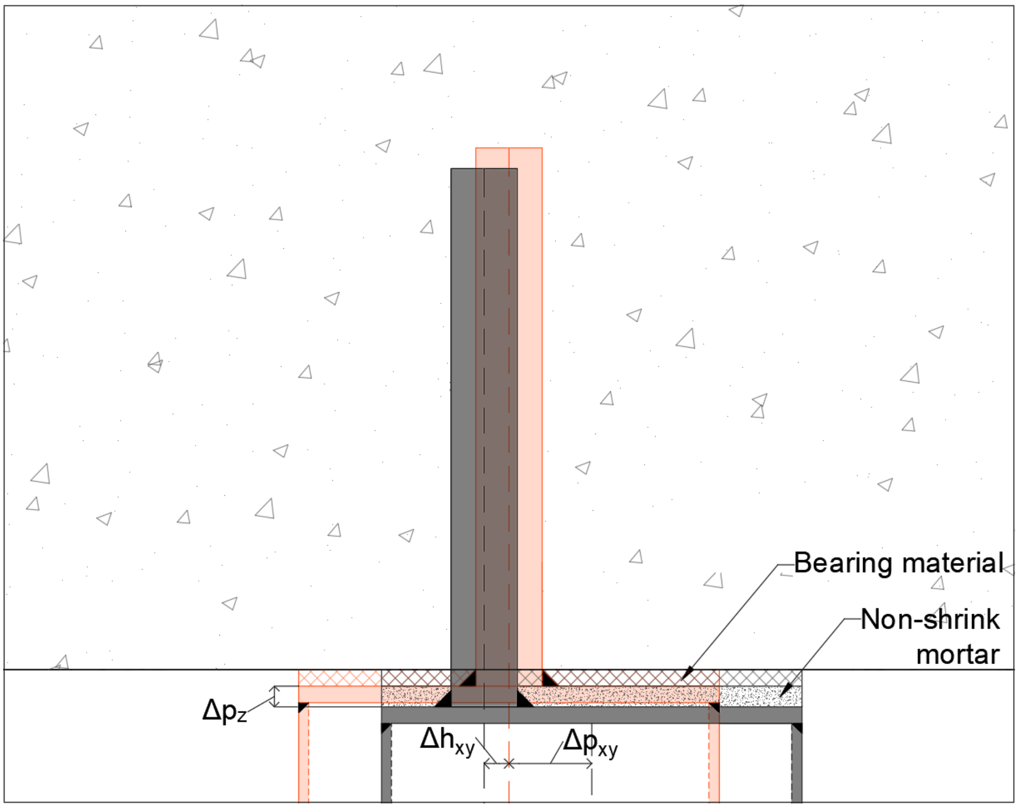
From equation (5.25) it follows that the angle of the cone-shaped holes should fall within the range $70^\circ \leq \beta \leq 90^\circ$.

Vertical tolerances

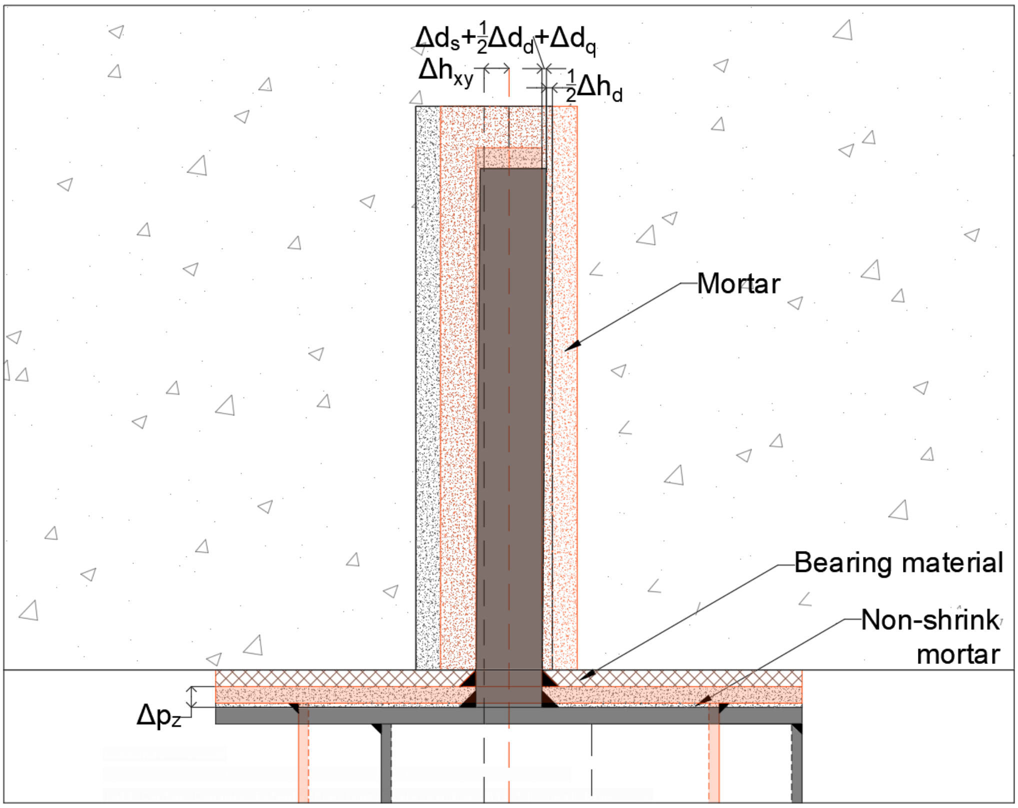
Regarding the vertical construction tolerances, the same yields for all execution variants, namely that a tolerance for the depth of the holes in the (abutment) footing should be included of:

$$\Delta 1_z = \Delta 2_z = \Delta 3_z \geq \Delta d_L + t_{bearing}$$

This is because the critical scenario would be the scenario in which the deviation of the pile head level is equal to zero (i.e. pile head level at exact height), the dowel length would be equal to $L + \Delta d_L = a + b + \Delta d_L = 150 + 500 + 25 = 675$ mm, and the layer of bearing material would be completely compressed (which is an extreme assumption). This would result in an excess length of $t_{bearing} + 25$ mm (see Figure 5.20d), which should therefore be able to be compensated in the hollow space above the top of the dowel.

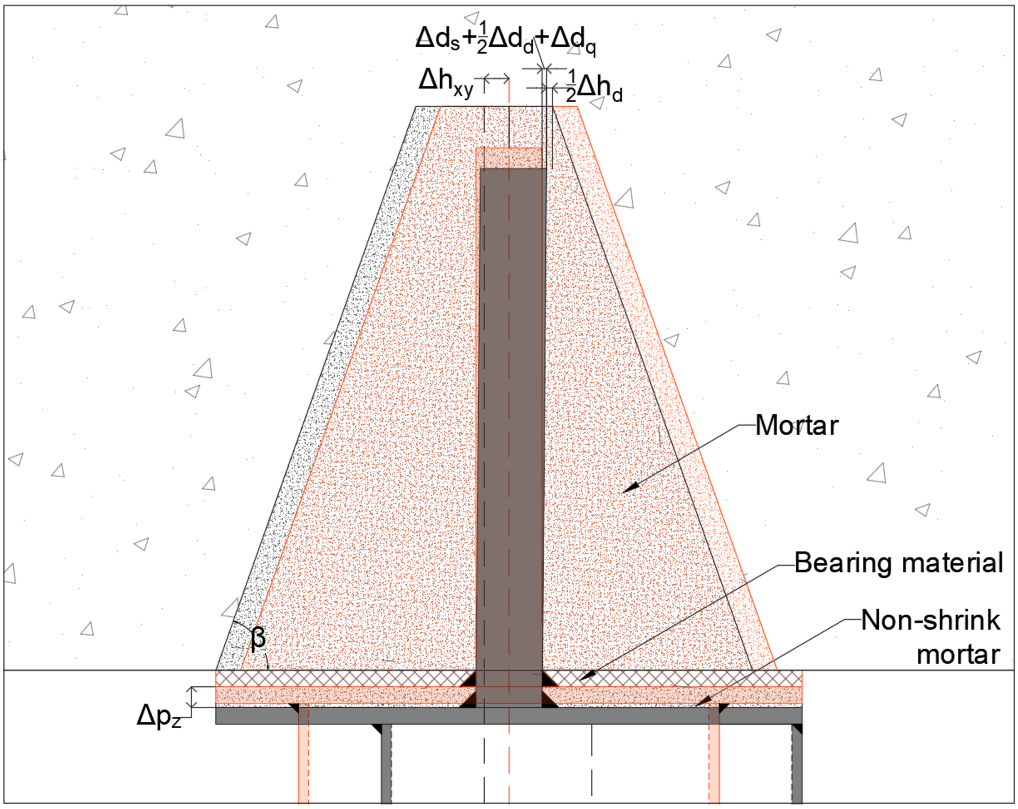


(a) Relevant horizontal deviations for execution variant 1 (NB: vertical deviation Δp_z is included for better visibility of the different situations, and, at the same time, to indicate the functioning of the mortar to eliminate this deviation)

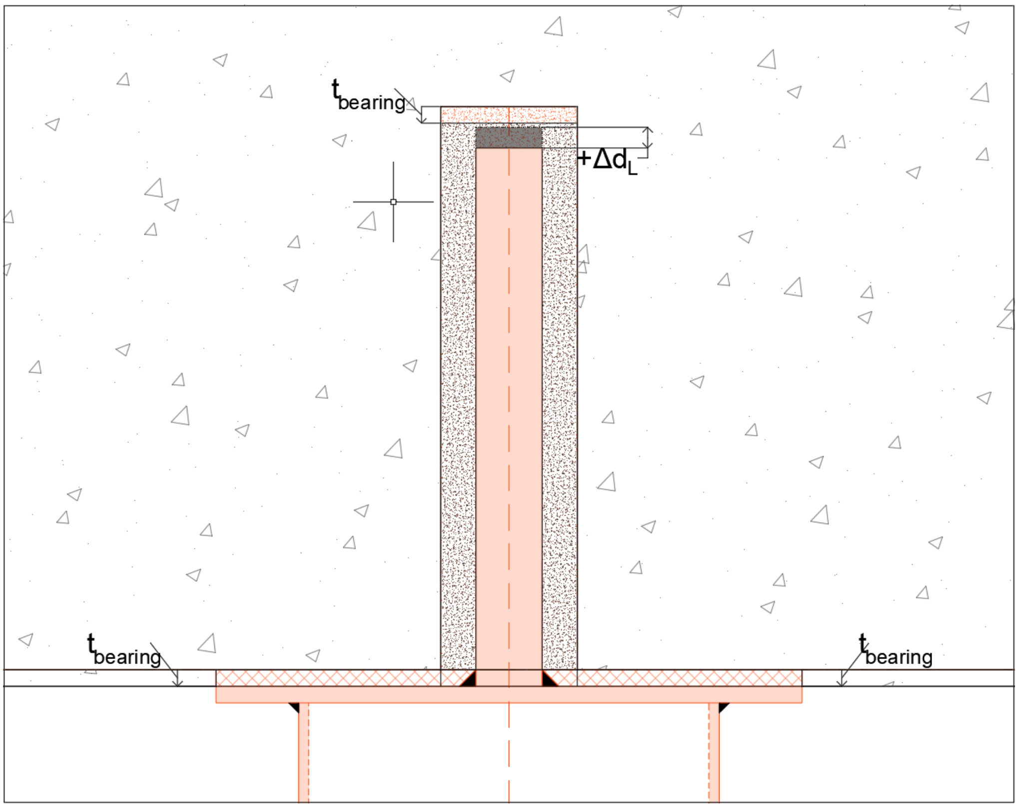


(b) Relevant horizontal deviations for execution variant 2 (NB: vertical deviation Δp_z is included for better visibility of the different situations, and, at the same time, to indicate the functioning of the mortar to eliminate this deviation)

Figure 5.20: Accumulation of horizontal and vertical deviations into construction tolerance, with theoretical (perfect) situation in red, and most unfavourable (critical) situation in grey (NB: protective layer around dowel and fitting tubes are not drawn)



(c) Relevant horizontal deviations for execution variant 3 (NB: vertical deviation Δp_z is included for better visibility of the different situations, and, at the same time, to indicate the functioning of the mortar to eliminate this deviation)



(d) Relevant vertical deviations (same for all variants), in which the bearing material is assumed to be completely compressed, and the dowel length is increased with Δd_L

Figure 5.20: Accumulation of horizontal and vertical deviations into construction tolerance, with theoretical (perfect) situation in red, and most unfavourable (critical) situation in grey (NB: protective layer around dowel and fitting tubes are not drawn) (cont.)

5.5.3. Advantages and Disadvantages

The three execution variants have been discussed in detail in the foregoing subsections. Based on this discussion, some of the main advantages and disadvantages of each variant have been identified, which are listed below for each variant separately.

Variant 1

The main advantages and disadvantages that have been identified with respect to execution variant 1 are:

Advantages

+ Most ideal ('the perfect') solution in terms of dowel to concrete interface and modelled interaction
+ Minimum use of cast in-situ mortar

Disadvantages

– From a constructability perspective practically impossible to execute (**theoretical solution only!**)

Variant 2

The main advantages and disadvantages that have been identified with respect to execution variant 2 are:

Advantages

+ Most likely to cause the least damage to dowel and concrete because of protection provided by fitting tubes
+ Most likely to be the most easy solution to demount and reuse

Disadvantages

– Most likely to be the most tedious and expensive solution
– Uncertainty regarding demountability, i.e. functioning of fitting tubes and releasing of mortar lump

Variant 3

The main advantages and disadvantages that have been identified with respect to execution variant 3 are:

Advantages

+ Most 'simple', feasible, solution
+ Most likely to be the cheapest, feasible, solution

Disadvantages

– Maximum use of cast in-situ mortar
– Introduction of new possible failure mechanism at inclined mortar to concrete interface which has to be investigated
– Uncertainty regarding demountability, i.e. releasing of cone-shaped mortar lump without damaging precast concrete

5.6. Verification and Validation

Since the development of the concept demountable footing to foundation (F2F) dowel connection will not be elaborated upon in more detail, at this stage, verification and validation of the developed solution is done. Therefore, firstly, the definitions of both concepts within the context of this research are repeated (see section 4.5). Subsequently, verification and validation is done consecutively.

5.6.1. Verification

Verification is understood as checking whether the system, in this case a demountable F2F connection, has been developed correctly. Therefore, the question to be answered here is formulated as:

> "Has a (concept) demountable F2F dowel connection been developed in accordance with the elaborated technical action points as was described in Chapter 3?"

This question is answered (i.e verification is done) by means of checking the developed connection within Crowther's framework. Specifically, this has been done by means of verifying to what extent the developed connection complies with the 28 key DfD principles for circular concrete viaducts (see Table 3.2), supplemented with a short explanation. Although these 28 key principles are specific for circular concrete viaducts in general, most of them can also reasonably be applied to the developed connection. However, principles 2 and 18–21 are believed to be not applicable. An overview of the verification is shown in Table 5.11.

Table 5.11: Verification of demountable F2F dowel connection by means of 28 key DfD principles for circular concrete viaducts

	Principle	**Incorporated?**	**Explanation**
1.	Specify removable, durable, mechanical instead of chemical and/or cast in-situ, rigid, connections	Yes	Main characteristic of (concept) demountable F2F dowel connection
2.	Design components (foundations, abutments, piers, etc.) to be retractable from ground	N / A	
3.	Specify materials and components with long life span	Yes	Steel and (non-shrink) mortar
4.	Design joints and connectors to withstand repeated use	Yes/No	Mentioned, but not yet translated into design (e.g. fatigue loads)
5.	Minimise the number of components	Yes	Limited to the minimum number of (different types of) components required to make the connection
6.	Minimise the number of different types of components	Yes	*Idem*
7.	Minimise the number of fasteners or connectors	Yes	One connection at each F2F interface
8.	Minimise the number of different types of fasteners or connectors	Yes	One type of connection used
9.	Minimise the number of different types of material	Yes	Steel and (non-shrink) mortar
10.	Avoid toxic and hazardous materials	Yes/No	Only steel and (non-shrink) mortar, but not specifically emphasised
11.	Avoid specifying secondary finishes to materials or components	Yes/No	No secondary finishes specified, but not specifically emphasised
12.	Specify materials that can be reused or recycled	Yes	Steel components and (non-shrink) mortar can be reused and/or recycled
13.	Provide standard and permanent identification of (types of) component and materials	No	Not specifically emphasised
14.	Permanently identify points of disassembly	No	Not specifically emphasised
15.	Using of interchangeable components	Yes	Standardisation of connection's geometry and cross-sections

Table 5.11 continued from previous page

	Principle	Incorporated?	Explanation
16.	Design for prefabrication of components	Yes	Steel components can be prefabricated
17.	Design for the repetition of similar components (i.e. design for mass production)	Yes	Standardisation of connection's geometry and cross-sections
18.	Separate the main load-bearing components from cladding and finishing elements	N / A	
19.	Standardising viaduct form and layout	N / A	
20.	Use a standard structural grid	N / A	
21.	Structure components according to their service life and the expected time till obsolescence to allow for parallel (dis)assembly	N / A	
22.	Provide access to all parts and components	No	Physically impossible (located below (abutment) footing)
23.	Provide realistic tolerances to allow for manoeuvring during (dis)assembly	Yes	See subsection 5.5.2
24.	Make components and materials of a size that suits the intended means of handling	Yes/No	Connection is manageable by hand, but not specifically emphasised
25.	Reduce the number of wearing parts that may need to be serviced	Yes	One minimally required bearing per connection
26.	Use sacrificial materials and components where wear is unavoidable and allow for their easy disassembly from the whole	Yes	Bearings and (non-shrink) mortar easy to disassemble during deconstruction
27.	Design to avoid permanent deformations and damage during (dis)assembly, use, and storing	Yes/No	Mentioned, but not yet translated into design
28.	Minimise cast in-situ components and elements	Yes/No	Certain minimum volume of cast in-situ (non-shrink) mortar is required

Especially with regards to principles 15, 16 and 17, it is noticed that these principles have been incorporated in the developed solution to a high extent. This is because one single design has been proposed which, in principle, can be applied in a large number of viaducts since the design of the connection has been based on the design of a standard circular concrete viaduct. However, principles 13 and 14 are again (see subsection 4.5.1) not (yet) incorporated into the design, since it is believed that these can (and should) be incorporated at a later stage by means of, for example, an element/component passport, a demolition plan, and/or a monitoring plan. Besides, principle 22 can not be met because of the simple reason that the connection is located under the (abutment) footing and covered by soil during the use phase. Finally, although the relevance of some principles has been mentioned in the design process, this has either not yet been translated into the design and should therefore be considered in further development of the connection (principles 4 and 27), or it turned out to be impossible to (fully) meet the principle due to constructability reasons (principle 28). Furthermore, principles 10, 11 and 24 seem to have been incorporated without specifically emphasising on those principles.

5.6.2. Validation

Validation is understood as checking whether the correct system, in this case a demountable F2F connection, has been developed. Therefore, the question to be answered here is formulated as:

> "Has a (concept) demountable F2F dowel connection been developed for application in the standard circular concrete viaduct as was established in Chapter 4?"

This validation question can simply be answered positively, since the established standard circular concrete viaduct has been used as the main starting point for the development of the concept demountable F2F dowel connection.

6 Monitoring

In this chapter, an advice with regards to monitoring of both the (standard) circular concrete viaduct in general and the demountable footing to foundation (F2F) dowel connection specifically is given by means of two draft versions of monitoring plans. Firstly, the general content of the monitoring plans is addressed in section 6.1. Subsequently, the determination of *what* physical parameters to monitor and *where*, as well as the monitoring plans for the (standard) circular concrete viaduct and the demountable F2F connection themselves are explained in sections 6.2 and 6.3 respectively. It has been left unaddressed *how* to monitor these physical parameters.

The result of this chapter concludes the first of the two subparts of this research, namely an advice on desired monitoring aspects, which has been discussed with respect to the general concept of a circular concrete viaduct, and with respect to the specific demountable F2F dowel connection within this concept.

6.1. General

In the transition from a linear to a circular construction industry, a key aspect concerns the monitoring of circular (demountable) structures. As was mentioned in section 2.4.1, it can be said that monitoring of a circular structure can be subdivided into two categories, namely the process of real-time monitoring of a structure during the use phase, and monitoring in terms of a reusability assessment at the end of a life-cycle, evaluating the condition of the different elements and components of the structure. The former is interesting from a structural behaviour verification perspective, and besides, it can be used to track down and predict possible future deterioration and/or damage, whereas the latter, i.e. the reusability assessment, is crucial for guaranteeing that the (elements and components of the) structure can safely be reused by means of inspecting the demounted components and, if necessary, repairing or maintaining them, in order to ensure high-quality reuse, and therefore guaranteeing the circularity of the structure.

In this section, however, both categories will be touched upon simultaneously by means of creating a draft version of a monitoring plan, based on the requirements for a monitoring plan on the one hand, and the deterioration mechanisms of reinforced concrete on the other hand, which were discussed in section 2.4.1 and 2.4.2 respectively. In fact, two draft versions of monitoring plans are created, of which the first plan is with respect to the (standard) circular concrete viaduct in general, whereas the second plan is specific to the developed concept demountable footing to foundation (F2F) dowel connection. Besides, both monitoring plans are limited to addressing only the first three questions that were stated in subsection 2.4.1 and need to be answered in order to draft a monitoring plan, which are:

1. What are the relevant (parts of) elements and components to monitor?
2. What potential deterioration and/or damage is expected?
3. What physical parameter(s) can reflect each of these types of deterioration/damage?

The fourth question ("*How can these physical parameter(s) be monitored?*") is not relevant within the scope of this research and is left to be answered by the party responsible for providing a suitable monitoring system. Therefore, the draft versions of the monitoring plans in sections 6.2 and 6.3 can be considered as an advice for anyone who is interested in proceeding with the future development of a (standard) circular concrete viaduct and/or the proposed concept demountable F2F connection.

6.2. Monitoring Plan for the (Standard) Circular Concrete Viaduct

First of all, it is clarified that 'the (standard) circular viaduct' refers to the standard viaduct as was established and explained in Chapter 4. This implies that the monitoring plan established in this section is both based on and applicable for the layout and design of this standard viaduct. Consecutively, the three questions stated in section 6.1 are addressed in the subsections below, and finally this is compiled in a comprehensive monitoring plan (see Table 6.1).

6.2.1. Determination of Monitoring Plan

Question 1. What are the relevant (parts of) elements and components to monitor?

The goal of this monitoring plan is to obtain the best possible overview of the behaviour during, as well as the condition at the end of, a life-cycle of the (elements and components of the) standard viaduct as a whole. The elements and components of which the standard viaduct consists are shown in Table 3.5. The relevant elements and components to monitor are those that have the largest influence on the structural behaviour and that are most sensitive and most likely to experience deterioration and/or damage over time. It is thought that this concerns the elements and components in the layers "Superstructure" and "Substructure" as well as in "Site", i.e. the building site itself. Regarding the components in the layer "Skin", those are not expected to be able nor intended to be reused. Finally, the components in the layer "Services" do not contribute to the structural behaviour of the viaduct nor is any relevant deterioration and/or damage expected to occur to these components. Therefore, these components are not considered in the monitoring plan.

This results in the following list of elements and components that are thought to be most relevant to monitor:

- Location (building site)
- Deck
  - Box beams
- Abutments
  - Foundation
  - Footing
- Intermediate support
  - Foundation
  - Footing
  - Intermediate piers
  - Capping beam

Besides, a special point of attention concerns the zones where different (parts of) components joint by means of (demountable) connections.

Question 2. What potential deterioration and/or damage is expected?

As was stated in subsection 2.4.2, reinforced concrete deterioration mechanisms can broadly be divided into two categories, namely concrete deterioration (direct deterioration) and reinforcement corrosion (indirect deterioration) (see Figure 2.16). Besides, it was found that the dominating deterioration mechanisms for general concrete bridges are chloride-induced and carbonation-induced corrosion. Therefore, these two electrochemical mechanisms are expected to be the (indirect) deterioration mechanisms with the highest probability of occurrence in the elements and components of the circular viaduct with respect to deterioration caused by environmental influences.

Besides, since a fully circular concrete viaduct will consist of a number of new types of (demountable) connections, it is also likely to expect physical-induced (direct) deterioration and/or damage in these zones, for example resulting from certain physical limits or unforeseen structural behaviour, which affects the concrete itself. This is for example dealt with in more detail in section 6.3 with regards to the demountable F2F dowel connection. With regards to other, similar, types of demountable connections elsewhere in the standard circular viaduct, it is practically impossible to say something about the potential deterioration and/or damage to be expected in these zones, since it is yet unknown what these connections will look like. However, it seems likely to expect potential local deterioration and/or damage at these demountable joints as well as at, for example, supports.

Furthermore, some form of fatigue-related deterioration and/or damage could be expected, which could both affect the concrete itself as well as the reinforcement and/or prestressing. This could, for example, be caused by excessive loading of the viaduct, either as a result of higher (traffic) loads or a higher number of vehicles than anticipated (see subsection 4.4.1). Finally, damage to (parts of) components could occur during deconstruction or, more generally, during handling[1] of the components of the viaduct.

Summarising the above, the following specific types of deterioration and/or damage are expected:

- (Indirect) deterioration and/or damage (e.g. cracking and spalling of concrete) as a result of electrochemical attack (i.e. reinforcement corrosion) (see Figure 2.18)
- (Direct) deterioration and/or damage (e.g. excessive cracking or abrasion of concrete) as a result of physical 'attack'
- Local damage at demountable joints and/or at supports
- Fatigue-related damage
- Deterioration and/or damage during handling of the components

Question 3. What physical parameter(s) can reflect each of these types of deterioration/damage?
As was explained in section 2.4.1, this question seeks to identify measurable and/or observable physical parameters revealing progress and change in materials and/or structural properties and behaviour that are typical signs of certain damage or deterioration mechanisms. Typical examples of these parameters are deformations, strains, stresses, (traffic) loads, vibrations, etc. However, also more specific parameters can be measured, such as carbonation depth, chloride concentration, electric potential of reinforcement or prestressing tendons, prestressing force, etc.

Besides, one could think of general physical parameters that can potentially provide relevant additional information, such as weather conditions (e.g. humidity) and temperature (e.g. of air, concrete, or steel), an exact survey (Dutch: '*inmeting*') of the location of the viaduct or, more generally, of the entire building site (e.g. x, y, and z coordinates), and CCTV footage.

6.2.2. Monitoring Plan

The answers to the questions in the previous subsections have been translated into a comprehensive monitoring plan for the (standard) circular concrete viaduct, which is shown in Table 6.1. In the first column of the monitoring plan ('*What (and where)?*'), it is indicated what physical parameter is advised to be monitored (i.e. answer to question 3), and, if applicable, at which specific location in/on the viaduct to monitor this parameter (i.e. answer to question 1). In the second column ('*Why?*'), the potential deterioration mechanism(s) and/or damage(s) that is/are expected and might be indicated by the physical parameter in the first column is/are listed.

It is emphasised again that, besides (the mostly automated) real-time monitoring, the (manual) reusability assessment is believed to be the most important part of monitoring with respect to guaranteeing the circularity of the structure. In addition to this reusability assessment at each life-cycle end, it therefore seems obvious to also periodically perform visual inspections in order to discover possible general concrete deterioration and damage such as cracking and abrasion.

[1]The term 'handling' in this research' context refers to the processes of a.o. production, transportation, storage, and (de)construction

Besides, depending on the applied monitoring system, it should be determined *when* the physical parameters are being monitored, i.e. continuously, periodically, or, for example, during specific conditions or circumstances.

Table 6.1: Monitoring plan for the (standard) circular concrete viaduct

What (and where)?	Why?
1. Deflection of box beams	• Direct deterioration and/or damage • General structural behaviour
2. (Traffic) loads on deck	• Direct deterioration and/or damage • Fatigue-related damage • General structural behaviour
3. Stress variation in prestressing tendons in box beams[1]	• Direct deterioration and/or damage • General structural behaviour
4. Chloride concentration in components exposed to de-icing salts and/or seawater	• Indirect deterioration and/or damage → specifically chloride-induced corrosion
5. Carbonation depth in components exposed to CO_2	• Indirect deterioration and/or damage → specifically carbonation-induced corrosion
6. Electric potential of reinforcement and/or prestressing	• Indirect deterioration and/or damage → degree of corrosion
7. CCTV footage of building site	• Deterioration and/or damage during use phase • Deterioration and/or damage during handling of components
8. Force needed to retract foundation piles from the ground	• Deterioration and/or damage during handling of components
9. (Environmental) conditions during storage of components	• Deterioration and/or damage during handling of components
10. Support reaction force	• Local damage at demountable joints and/or supports
11. Stress variation in prestressing bars in demountable connections (if applied)	• Local damage at demountable joints and/or supports
12. Exact survey (x, y, and z coordinates) of building site	• Local damage at demountable joints and/or supports (e.g. (relative) displacement of components) • Settlement of viaduct or embankments (e.g. for compensation of deflections)
13. Weather conditions	• Additional information regarding local circumstances
14. Temperature (air, concrete, steel)	• Additional information regarding local circumstances

[1] E.g. in order to check that the required prestressing force, depending on the type of prestressing (full, limited, or partial) is reached

6.3. Monitoring Plan for the Demountable F2F Connection

The monitoring plan that is established in this section is specific for execution variant 3 (see section 5.5) of the demountable F2F dowel connection which is assumed to be applied in the standard viaduct. It has been chosen to base the monitoring plan on execution variant 3 of the demountable connection, since it is believed that this is the most feasible variant based on a trade-off between both executional and financial aspects (also see subsection 5.5.3). Besides, it is noted that the same procedure as has been applied in section 6.2 is followed in order to establish the monitoring plan for the demountable F2F connection (see Table 6.2). However, the size of the monitoring plan is limited compared to the monitoring plan for the (standard) circular concrete viaduct, since this monitoring plan can be considered as an extension of the monitoring plan shown in Table 6.1.

6.3.1. Determination of Monitoring Plan

Question 1. What are the relevant (parts of) elements and components to monitor?

Since this monitoring plan is specific for the demountable F2F connection, it is obvious that the different parts of the connection are relevant to be monitored. Besides, also the (parts of) components that interact with the demountable connections, such as the concrete of the (abutment) footings and the mortar directly surrounding the dowel, and the steel pipe foundation piles, might be relevant to monitor.

This results in the following (parts of) components that are thought to be most relevant to monitor:

- Foundation piles
- Demountable F2F dowel connections
  - Dowels
  - Mortar infills
  - Welds
- Concrete and mortar at the interfaces

Question 2. What potential deterioration and/or damage is expected?

Except for indirect deterioration and/or damage as a result of electrochemical attack (i.e. reinforcement corrosion), the same deterioration and/or damage is expected to potentially occur at the relevant (parts of) components as was identified for the (standard) circular concrete viaduct in subsection 6.2.1. Reinforcement corrosion is not considered as a potential deterioration mechanism for the demountable F2F connection, since it is expected that this will not directly affect the connection.

Furthermore, it is emphasised that direct deterioration and/or damage as a result of physical ‘attack’ in this case is considered to be the same as local damage, since the demountable F2F connections concern a very specific (i.e. local) component, to which damage is mainly expected to occur as a result of mechanical behaviour (i.e. physical movement). Typical observable direct local damage that is expected to occur concerns (local) abrasion of the concrete cover and (local) crushing of mortar surrounding the dowels.

Besides, a potentially expected form of damage concerns fatigue-related damage, mainly regarding the steel components and welds, and the mortar infills, caused by cyclic loading of the dowel. This could, for example, result in crushing of the mortar surrounding the dowel, in crack initiation in the dowel, or in fractures in the welds.

Finally, potential deterioration and/or damage during handling is taken into account, mainly related to the process of releasing of the mortar infills from the (abutment) footings. This could, for example, result in abrasion of the concrete surface of the precast (abutment) footings due to a non-smooth release of the (abutment) footing from the cone-shaped mortar infill surrounding the dowel. Besides, it is conceivable that during hoisting, (un)loading, and transportation, the elements and components might get damaged due to collisions, vibrations, etc. which are inherent to the handling process.

Summarising the above, the following specific types of deterioration and/or damage are expected:

- (Direct) local deterioration and/or damage (e.g. local abrasion of concrete cover or local crushing of mortar surrounding the dowels) as a result of physical 'attack
- Fatigue-related damage
- Deterioration and/or damage during handling of components

Question 3. What physical parameter(s) can reflect each of these types of deterioration/damage?
The same typical examples of physical parameters as were mentioned in 6.2.1 apply in this case. Besides, it is interesting to monitor the (relative) deformation of the dowels with respect to the (abutment) footings in order to compare the actual behaviour of the demountable F2F connection to the modelled behaviour in SCIA Engineer, which was used as a first design check. Furthermore, if possible, it could be interesting to measure the contact stresses at the dowel to mortar interface, since this directly relates to the other design check that was done during the development of the connection (see subsection 5.2.5). Similarly, also the contact (shear) stresses at the mortar to concrete interface might be interesting to measure. Finally, again it might be interesting to monitor general physical parameters which can potentially provide relevant additional information.

6.3.2. Monitoring Plan

The answers to the questions in the previous subsections have been translated into a monitoring plan for the demountable F2F connection, which is shown in Table 6.2. The layout and context of the monitoring plan is equal to the monitoring plan for the (standard) circular concrete viaduct as was explained in subsection 6.2.2.

Similar to monitoring of the (standard) circular concrete viaduct in general, here it is also emphasised that the reusability assessment at the end of a life-cycle is thought to be the most important part of monitoring with respect to guaranteeing a circular life-cycle of the demountable F2F connections. This implies that the connections should be inspected thoroughly during/after deconstruction, since it is practically impossible to perform visual inspections of the demountable connections during their life-cycle, as the connections are located below the (abutment) footings and covered with soil.

Table 6.2: Monitoring plan for the demountable F2F connection

What (and where)?	Why?
1. Relative deformation of dowel with respect to (abutment) footing	• Direct local deterioration and/or damage • Fatigue-related damage • General structural behaviour of connection
2. Contact stress at dowel to mortar interface	• Direct local deterioration and/or damage • Fatigue-related damage • Deterioration and/or damage during handling • General structural behaviour of connection
3. Contact (shear) stress at mortar to precast concrete interface	• Direct local deterioration and/or damage • Deterioration and/or damage during handling • General structural behaviour of connection
4. Weather conditions	• Additional information regarding local circumstances
5. Temperature (air, concrete, steel)	• Additional information regarding local circumstances

7

Life-Cycle Cost Analysis

In this chapter, the life-cycle costs of a traditional and a circular alternative for the standard viaduct are estimated and compared in order to investigate the feasibility of a circular concrete viaduct from a financial perspective. First of all, the demarcation of the scope of the analysis and the main starting points are explained in section 7.1. Subsequently, the build-up of the life-cycle costs of the traditional and of the circular alternative are addressed in sections 7.2 and 7.3 respectively. Finally, a comparison between the life-cycle costs of both alternatives is discussed in section 7.4, including different scenarios and assumptions.

The result of this chapter concludes the second subpart of this research, namely a comparison between the life-cycle costs of a traditional and a circular alternative for the standard viaduct. By means of this comparison, it is explained under which circumstances (i.e. starting points and assumptions) the circular alternative is feasible from a financial perspective (i.e. cheaper than the traditional alternative) over the full service lifetime.

7.1. General

In order for the concept of a circular viaduct to be feasible, besides the practical issues, it should preferably be attractive as well from a financial perspective. The potential environmental benefits of a circular viaduct are rather straightforward, and can be quantified by means of, for example, a life-cycle analysis (LCA) or multi-cycle assessment (MCA) as was discussed in subsection 2.1.3. However, in order to check whether the concept is (or can be) competitive on the market, it has been decided to perform a life-cycle cost analysis (LCCA). In fact, a comparison between the life-cycle costs of the standard circular viaduct, and a viaduct with the same layout and design, constructed in a traditional way and characterised by a linear life-cycle (i.e. standard linear viaduct) has been made.

7.1.1. Demarcation of Analysis Scope

The LCCAs for the traditional alternative (i.e. linear life-cycle) and the circular alternative have been based both on key cost figures (Dutch: '*kengetallen*') for concrete viaducts and on experience and estimations made by cost calculators at Lievense. In the analyses, only the costs related to production and execution (construction costs), maintenance, and removal (deconstruction/demolition costs) of the load-bearing elements and components are taken into account. Besides, the residual value of recovered materials are considered where relevant. Finally, both analyses comprise of several assumptions, which are addressed in the respective sections.

Costs that are excluded from the LCCAs are costs related to:

- Preparatory works: preparation of the building site.
- Ground works: a.o. excavation, heightening, transportation and processing.
- Pipe works: installation of e.g. drainage.
- Road pavement: applying asphalt layer.
- Steel constructions: production, installation, preserving and maintaining of steel provisions.
- Abutment slope works (Dutch: '*talud werkzaamheden*'): e.g. applying protection and anti-graffiti coating.
- Engineering works: e.g. general calculations and drawings.
- Other: e.g. traffic measures during execution, additional work, etc.

A key argument for leaving these costs out of the analyses is that no large differences in these costs for both alternatives are expected, and therefore, the influence on the outcome of the comparison is thought to be negligible.

7.1.2. Main Starting Points

Some of the main starting points that apply to both analyses are listed below:

- The layout and design of the standard viaduct, described in detail in Chapter 4, has been used for determination of the dimensions, surface areas, volumes, etc. For more details, see the separate document "SCIA Engineering Report - Standard Viaduct Model".
- An intended full service lifetime of 200 years has been assumed. This implies an increase of 100% compared to the design lifetime of viaducts in the Eurocodes (see subsection 2.6.6). This has been chosen in order to allow the elements and components of the circular viaduct to be reused at least two or more times with relevant service lifetimes per life-cycle, similar to what was assumed during the development of the prototype circular viaduct (see subsection 2.2.2).

  At the same time, a maximum of 5 life-cycles has been adopted in order to assure a minimum lifetime per life-cycle of 40 years. It is believed that the main load-bearing elements and components (i.e. super- and substructure) can be designed to be reusable for a period of 200 years by means of careful design, and extensive monitoring and maintenance.

- An important starting point is the fact that both alternatives are compared to each other based on the same (assumed) service lifetime per life-cycle. This has been chosen because it is reasoned that in the decision-making process, the service lifetime for the viaduct, whether the traditional or the circular alternative is chosen, should be agreed upon beforehand, and therefore be equal for both alternatives. It does not make sense to compare two traditional viaducts with each a service lifetime of 100 years to a circular viaduct with 4 life-cycles of 50 years, since it should concern one and the same service (i.e. either two times 100 years or 4 times 50 years for both alternatives).

7.2. Traditional Alternative

In this section, the build-up of the construction, maintenance, and deconstruction costs and of the value of the recovered materials of the traditional alternative is discussed consecutively. Finally, the net costs (i.e. life-cycle costs) of the traditional alternative are determined. A detailed overview of the calculation of these costs can be found in Appendix O. The explanation of the build-up of the costs in the following subsections is directly related to the spreadsheet in Appendix O, and both should therefore be read in conjunction.

Whereas the estimation of the life-cycle costs for the circular alternative involves quite some assumptions (see section 7.3), it is believed that the calculation of the life-cycle costs of the traditional alternative is relatively accurate. Nevertheless, it should be remembered that it concerns a limited and simplified estimation, taking into account the demarcation of the analysis scope and the starting points as were mentioned in subsections 7.1.1 and 7.1.2 respectively.

7.2.1. Construction Costs

As was mentioned in subsection 7.1.1, the build-up of the life-cycle costs, in particular of the construction costs, has been based on key cost figures for concrete viaducts. These figures were obtained from comparable viaduct designs done by Lievense, and include both the costs for the material/production as well as the installation on-site of elements and components in one figure by means of a unit price.

This results in the following build-up of the construction costs for the traditional alternative:

- Foundation
  - Material/production and installation of 48 steel pipe foundation piles with a unit price of €3.000,- per piece.
- In-situ concrete works
  - Material/production and installation of two abutments, consisting of a.o. 121,8 m^3 concrete and 36.540 kg of reinforcement (based on the assumption of 300 kg/m^3 concrete), resulting in a unit price of €45.217,- per abutment.
  - Material/production and installation of four wing walls, consisting of a.o. 10,4 m^3 concrete and 3120 kg of reinforcement (based on the assumption of 300 kg/m^3 concrete), resulting in a unit price of €2.848,- per wing wall.
  - Material/production and installation of the complete intermediate support, consisting of a.o. 102,8 m^3 concrete and 30.840 kg of reinforcement (based on the assumption of 300 kg/m^3 concrete), resulting in a unit price of €80.505,- for the entire intermediate support.
- Prefab concrete works
  - Material/production and installation of 22 transition slabs with a unit price of €1.500,- per piece.
  - Material/production and installation of 2 deck spans consisting of eight 1,5 m wide and 27,7 m long box beams with a unit price of €300.000,- per span.
  - Material/production and installation of 36 m of transition joints with a unit price of €1.000,- per meter.
- Execution costs
  - Based on custom cost estimation by cost calculators at Lievense, an amount equal to 20% of the construction costs is added to the total construction costs, which represents costs for equipment and execution on-site.

Obviously, the estimation of the construction costs of the deck involves a key assumption. The unit price has been based on the price of a comparable deck designed by Lievense. This design consisted of eight 1,5 m wide box beams with a span length of roughly 24 m, which was assigned a unit price per span of €250.000,-. Therefore, for the standard viaduct, the unit price was proportionally increased and rounded up to €300.000,- per span.

7.2.2. Maintenance Costs

The maintenance costs have been subdivided into small and large maintenance. Small maintenance consists of, for example, surface cleaning, weeding, etc. and is estimated to be done once every two years, whereas large maintenance consists of, for example, replacement of bearings and transition joints, repair jobs, etc. and is estimated to be done once every 25 years. For both, a budget (i.e. unit price) of respectively €1.000,- and €25.000,- is reserved, again based on estimations by cost calculators at Lievense.

It is emphasised that the maintenance costs are the only costs that depend on the adopted lifetime per life-cycle, implying that these are the only costs taken into account during the use phase of the viaduct.

7.2.3. Deconstruction Costs

The deconstruction costs of the traditional alternative in fact refer to the costs to demolish the viaduct and to process the waste by a certified company. Demolition of the viaduct consists of crushing and sawing of the elements and components, and subsequently transporting the waste by means of dump trucks. An estimation for the total process of deconstruction was made by cost calculators at Lievense, and was estimated to be €50.000,- of which the division between demolition costs and costs for processing the waste was estimated to be 50/50, resulting in a unit price of €25.000,- for each process.

7.2.4. Residual Value

After demolition of the traditional viaduct, most of the waste (mainly concrete and reinforcement) can be recycled. For example, recycled concrete often is reused for road foundation, as was mentioned in subsection 1.1.2. For both materials, a recovery rate of 90% was assumed [92], and unit prices of €5,- per ton of concrete [92] and €100,- per ton of scrap steel [92, 93] were adopted.

7.2.5. Life-Cycle Costs

The calculation of the net life-cycle costs for the traditional alternative is summarised in Table 7.1, and results in an amount of €1.290.319,- per life-cycle. A detailed overview of the build-up of these costs can be found in Appendix O.

Table 7.1: Overview of build-up of life-cycle costs for traditional alternative

COSTS						
Construction costs	€	995.332,-				
Execution costs (20%)	€	199.066,-	+			
Total construction costs	€	1.194.399,-		€	1.194.399,-	
Total maintenance costs				€	60.000,-	
Total deconstruction costs				€	50.000,-	+
Total costs				€	1.304.399,-	
RESIDUAL VALUE						
Total materials recovery value				€	14.079,-	+
Total residual value				€	14.079,-	
LIFE-CYCLE COSTS OF TRADITIONAL ALTERNATIVE						
Total costs				€	1.304.399,-	
Total residual value				€	14.079,-	–
Net costs				€	1.290.319,-	

7.3. Circular Alternative

In this section, the build-up of the construction, maintenance, and deconstruction costs and of the value of the recovered materials for the circular alternative is discussed consecutively. Subsequently, the net costs (i.e. life-cycle costs) of the circular alternative are determined, which are divided into net costs for the first life-cycle and net costs for every next life-cycle. Finally, an upper and lower limit estimation is established which is used in the comparison between the life-cycle costs of the traditional and the circular alternative in section 7.4. A detailed overview of the calculation of these costs can be found in Appendix P. The explanation of the build-up of the costs in the following subsections is directly related to the spreadsheet in Appendix P, and both should therefore be read in conjunction.

The calculation of the different costs are based on several assumptions which are addressed in the respective subsections. Generally, it has been attempted to use the same way of reasoning as was used in the calculation of the life-cycle costs for the traditional alternative. Nevertheless, it is believed that the accuracy of the estimation of the life-cycle costs for the circular alternative is less exact compared to the traditional alternative, because of many unknown, and therefore estimated (distributions of) costs.

7.3.1. Construction Costs

The key cost figures for concrete viaducts, which were used to calculate the construction costs of the traditional alternative, include both the costs for material/production as well as installation of elements and components in one figure by means of a unit price. However, related to the circular alternative, a separation between the (one-time) production and material costs and the (recurring) installation costs is required in order to account for the effect of reusing the elements and components.

Therefore, it has been assumed that the (one-time) material/production costs and the (recurring) installation costs for the elements and components of the circular alternative account for 85% and 15% of the construction costs for the same elements and components of the traditional viaduct respectively. This distribution has been estimated by cost calculators at Lievense. Besides, it is expected that the (one-time) material/production costs for the circular alternative are higher than for the traditional alternative, and are therefore increased by 50% to account for potential different kinds of innovative solutions to be provided and produced. This distribution and this assumed increase of material/production costs are the two main variables in the comparison between the life-cycle costs for the traditional and the circular alternative which is discussed in section 7.4. As an example, this results in the following build-up of the construction costs for the foundation of the circular alternative:

- Foundation
  - Material/production costs of 48 steel pipe foundation piles: €3.000,- x 85% x 150% = €3.825,- per piece (one-time).
  - Installation costs of 48 steel pipe foundation piles: €3.000,- x 15% = €450,- per piece (recurring).

The calculation of the construction costs for the remaining prefab concrete elements and components is done in the same way. Besides, for calculation of the total construction costs of the circular alternative, the construction costs are increased by the same percentage of 20% to account for the execution costs, similar to what was done in calculation of the total construction costs for the traditional alternative.

7.3.2. Maintenance Costs

Similar to the maintenance costs considered for the traditional alternative, a division into small and large maintenance costs has been made for the circular alternative too. Furthermore, the same frequency of small and large maintenance, i.e. once every 2 and every 25 years respectively, has been adopted. However, similar to the (one-time) material/production costs, the maintenance budgets (i.e. unit prices) are increased by 50% compared to the maintenance budgets for the traditional alternative, resulting in €1.500,- and €37.500,- reserved for small and large maintenance respectively.

Besides, an additional budget is reserved for the reusability assessment at the end of each life-cycle, the importance of which was addressed in Chapter 6, and the resulting need for maintenance and/or repairs. It was estimated that these recurring costs are equal to 50% of the large maintenance costs, resulting in a budget (i.e. unit price) of €18.750,- per life-cycle.

7.3.3. Deconstruction Costs

Once again, similar to the (one-time) material/production costs, the deconstruction costs for the circular alternative have assumed to be equal to 150% (i.e. increase of 50%) of the deconstruction costs of the traditional alternative. This results in a budget (i.e. unit price) of €37.500,- reserved for demounting and transporting the circular viaduct.

After deconstruction of the circular viaduct, ideally it is directly transported to and reused at a new location. However, if this is not the case, the elements and components need to be stored somewhere. However, since there are many unknowns with regards to these storage costs, such as the (average) storage time and the associated costs for example, it has been decided not to consider the storage costs in this analysis. Besides, it has been verified that these costs in the end will not be decisive.

7.3.4. Residual Value

It was stated in section 3.2 that the concept of the circular viaduct is mainly based on the recycling strategies of ‘relocation or reuse of whole building’ (i.e. viaduct) and partially on ‘reuse of components into new buildings’ (i.e. viaducts) (see subsection 2.1.3 and Figure 2.4). Therefore, it is assumed that all elements and components are being reused one-on-one, which implies that practically no material is available for recovery. The relatively small volume of material that nevertheless turns into waste such as the mortar used to fill the oversized holes in the (abutment) footings and to even out vertical differences between the pile head levels is not taken into account. Therefore, this results in zero value of recovered materials considered for the circular alternative.

7.3.5. Life-Cycle Costs

The calculation of the net costs for the first life-cycle of the circular alternative is summarised in Table 7.2, and results in an amount of €1.848.268,-. A detailed overview of the build-up of these costs can be found in Appendix P.

Table 7.2: Overview of build-up of life-cycle costs for first life-cycle of circular alternative

COSTS						
Construction costs	€	1.418.348,-				
Execution costs (20%)	€	283.670,-	+			
Total construction costs	€	1.702.018,-		€	1.702.018,-	
Total maintenance costs				€	108.750,-	
Total deconstruction costs				€	37.500,-	+
Total costs				€	1.848.268,-	
RESIDUAL VALUE						
Total materials recovery value				€	0,-	+
Total residual value				€	0,-	
LIFE-CYCLE COSTS OF CIRCULAR ALTERNATIVE (FIRST LIFE-CYCLE)						
Total costs				€	1.848.268,-	
Total residual value				€	0,-	–
Net costs				€	1.848.268,-	

The net costs for every next life-cycle, however, are substantially reduced due to the fact that (in the most optimistic scenario) no material/production costs need to be taken into account. This results in an amount of €325.410,- for every next life-cycle (see Table 7.3). A detailed overview of the build-up of these costs can be found in Appendix P.

Table 7.3: Overview of build-up of life-cycle costs for every next life-cycle of circular alternative

COSTS			
Construction costs	€ 149.300,-		
Execution costs (20%)	€ 29.860,-	+	
Total construction costs	€ 179.160,-		€ 179.160,-
Total maintenance costs			€ 108.750,-
Total deconstruction costs			€ 37.500,- +
Total costs			€ 325.410,-
RESIDUAL VALUE			
Total materials recovery value			€ 0,- +
Total residual value			€ 0,-
LIFE-CYCLE COSTS OF CIRCULAR ALTERNATIVE (OTHER LIFE-CYCLES)			
Total costs			€ 325.410,-
Total residual value			€ 0,- –
Net costs			€ 325.410,-

7.3.6. Upper and Lower Limit Estimations

Since the estimation of the life-cycle costs for the circular alternative is based on several assumptions, resulting in a greater uncertainty about the accuracy compared to the traditional alternative, it is decided to establish both an upper and lower limit estimation of the the life-cycle costs for the circular alternative.

For determination of the upper and lower limit estimations, the influence of the different costs have firstly been investigated by means of increasing/reducing these by a certain percentage. Finally, the upper and lower limits have been established by means of increasing the (one-time) material/production costs, the maintenance costs and the deconstruction costs with 100% and 10% (instead of 50%) respectively. Besides, the distribution between (one-time) material/production costs and the (recurring) installation costs has been modified, resulting in a 70/30 and 92,5/7,5 distribution (instead of 85/15) for the upper and lower limit respectively.

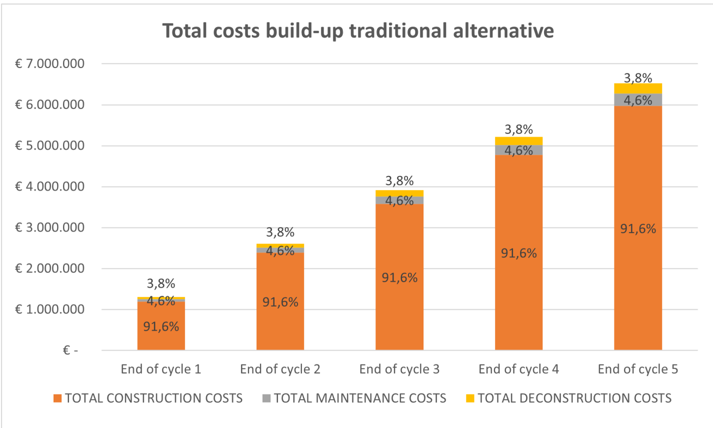
7.4. Life-Cycle Costs Comparison

After establishing the life-cycle costs for both the traditional and circular alternative, a comparison between both costs over the lifetime of the viaduct can be made. As was already stated in section 7.2, it is believed that the calculation of the life-cycle costs of the traditional alternative is relatively accurate, and therefore it is decided not to investigate the influence of changes in certain costs for this alternative. However, with regards to the circular alternative, it is believed that the accuracy of the estimation of the life-cycle costs is less exact compared to the traditional alternative, as was stated in section 7.3, and therefore an upper and lower limit estimation has been included in the comparison. The main variables that are used for making these upper and lower limit estimations are:

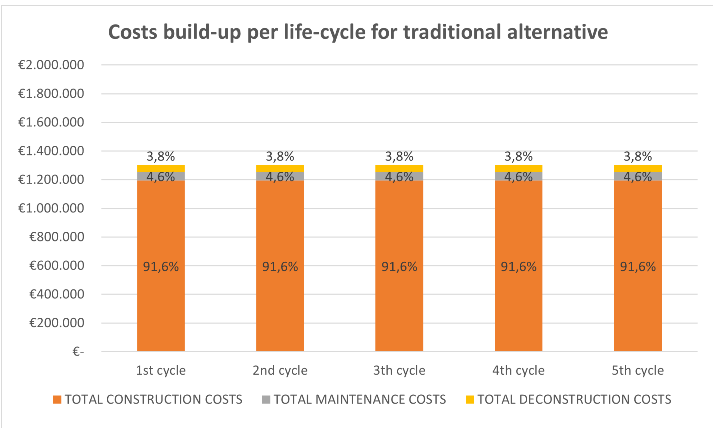
- The assumed increase of the (one-time) material/production costs, the maintenance costs and the deconstruction costs (i.e. either 10%; 50%; or 100% for respectively lower limit, reference scenario, and upper limit)
- The distribution between (one-time) material/production costs and the (recurring) installation costs (i.e. 92,5/7,5; 85/15; or 70/30 for respectively lower limit, reference scenario, and upper limit)

7.4.1. Costs Build-Up Comparison

First of all, insight is given by means of the graphs in Figures 7.1 and 7.2 how the costs for both alternatives build-up differently over the full service lifetime of 200 years (assuming 5 life-cycles). It can be clearly observed in Figure 7.1 that the traditional alternative is characterised by a linear life-cycle model, since the total costs at the end of a life-cycle are simply calculated by means of multiplying the number of passed life-cycles by the life-cycle costs of the traditional alternative, i.e. €1.290.319,- (see

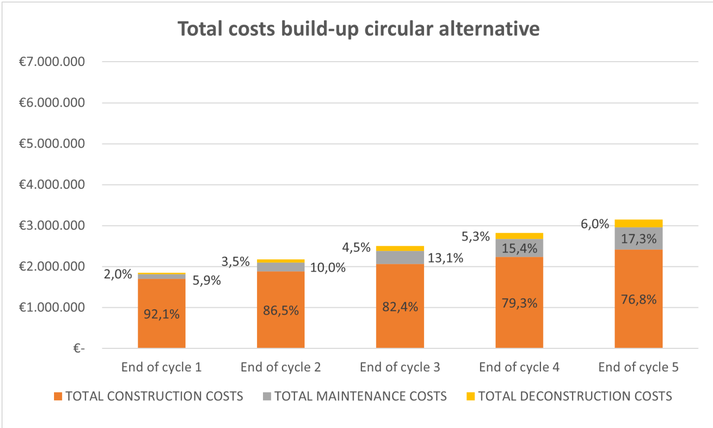


(a) Overview of the total costs at the end of each life-cycle of the traditional alternative

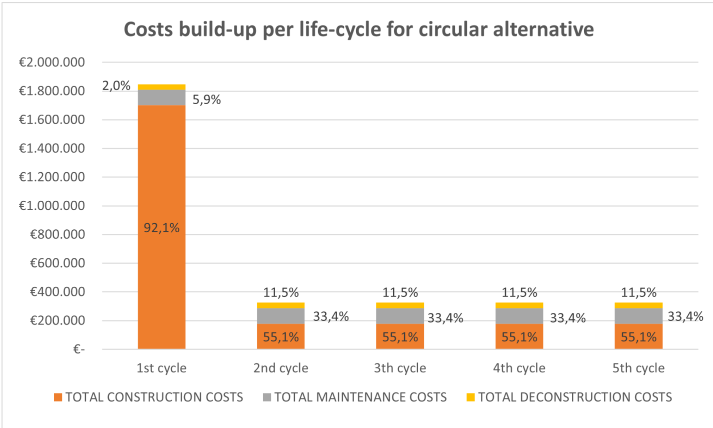


(b) Overview of the costs for each life-cycle of the traditional alternative

Figure 7.1: Costs build-up of traditional alternative (NB: the value of recovered materials has not been included)



(a) Overview of the total costs at the end of each life-cycle of the circular alternative



(b) Overview of the costs for each life-cycle of the circular alternative

Figure 7.2: Costs build-up of circular alternative

subsection 7.2.5). This is emphasised by means of the overview of the costs related to each life-cycle of the traditional alternative (see Figure 7.1b), in which it can be seen that the costs for every life-cycle are constant. Besides, the constant distribution of the construction, maintenance, and deconstruction costs in the total costs also emphasise the linear life-cycle model of this alternative.

On the contrary, in Figure 7.2a it can be observed that the total costs at the end of a life-cycle of the circular alternative are not equal to the multiplication of the number of passed life-cycles by the life-cycle costs. Besides, the distribution of the construction, maintenance, and deconstruction costs in the total costs changes as the number of life-cycles increases, which indicates a circular life-cycle model. In Figure 7.2b, it is particularly interesting to observe the drastic drop in costs for every additional life-cycle of the circular alternative compared to the costs for the first life-cycle (i.e. initial costs).

7.4.2. Life-Cycle Costs Comparison

The most interesting question is whether the circular alternative can compete with the traditional alternative over time with respect to the life-cycle costs. Therefore, the net total costs of both alternatives at the end of each life-cycle are projected in one graph, as well as the difference between both alternatives, shown in Figure 7.3. It can be seen that after 2 life-cycles, the circular alternative is cheaper than the traditional alternative, and that the difference is increasing with every additional life-cycle.

Upper and lower limit estimation

In Figure 7.4, the same data as in Figure 7.3 is presented in a slightly different way, and besides, the upper and lower limit estimations have been included. It can be seen by means of the shaded, dotted lines how the costs develop cascadingly over time (i.e. during the life-cycles). The solid lines indicate the average costs at each moment in time, and can be used to predict whether the circular alternative is

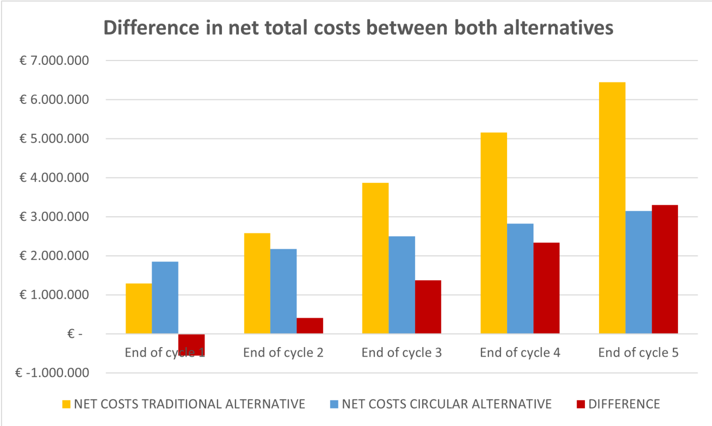


Figure 7.3: Comparison of net total life-cycle costs of traditional and circular alternative

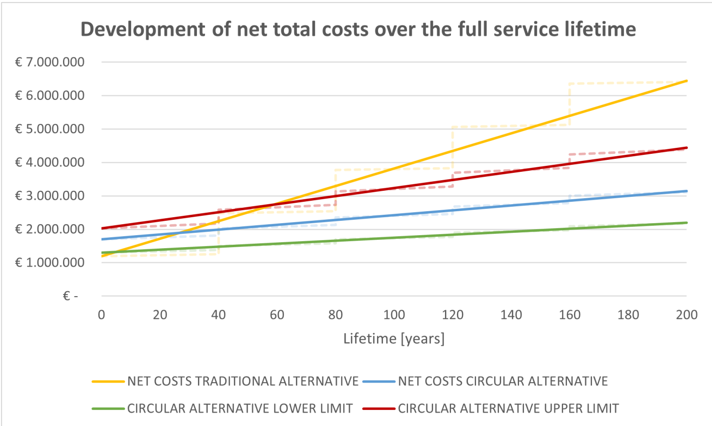


Figure 7.4: Development of net total costs for reference and upper and lower limit estimations over full service lifetime

becoming cheaper than the traditional alternative. For the reference estimation (i.e. blue line in Figure 7.4), it has already been observed that this happens after 2 life-cycles. From the graph, however, it becomes clear that in fact this is the case even earlier, namely already at the beginning of the 2nd life-cycle.

With regards to the lower limit estimation, it can be seen that it implies that the circular alternative would also just be cheaper at the beginning of the 2nd life-cycle. However, more interestingly, it can also be seen that according to the upper limit estimation, it turns out that the circular viaduct still is cheaper when 3 or more life-cycles are realised. This has been made insightful by means of Table 7.4, in which either the additional costs or the reduction in costs for the circular alternative compared to the traditional alternative are shown for the three scenarios considered and for a different number of life-cycles. However, these results indicating that a circular viaduct would be feasible from a financial perspective compared to a traditional alternative after 2 (reference estimation) or 3 (upper limit estimation) life-cycles should be verified by means of a more detailed LCCA.

Table 7.4: Additional costs (+) or reduction (-) in costs for circular alternative compared to traditional alternative for different scenarios and different number of life-cycles (LCs)

Scenario	Start of 1st LC	After 1 LC (0x reused)	After 2 LCs (1x reused)	After 3 LCs (2x reused)	After 4 LCs (3x reused)	After 5 LCs (4x reused)
Reference scenario	43%	43%	-16%	-35%	-45%	-51%
Lower limit	9%	9%	-38%	-53%	-61%	-66%
Upper limit	70%	72%	8%	-14%	-25%	-31%

8

Discussion, Conclusions and Recommendations

In this chapter, first an extensive discussion of the results is presented in section 8.1. Subsequently, in section 8.2, the main and subresearch questions are answered, and main conclusions are drawn. Finally, recommendations for future research are given in section 8.3.

8.1. Discussion

The main motivation for this research originated from two ambitions, which involve many challenges for which circular solutions have to be developed. First, the ambition of the Dutch government to achieve a circular economy, which also includes a circular construction industry, in the Netherlands by 2050 at the latest. Secondly, in line with the first, the ambition of Rijkswaterstaat to work climate-neutral and circular in 2030. At this moment, the main focus in the Dutch construction industry is on developing circular solutions which can be implemented and applied during the enormous replacement and renovation task of many of the almost 40.000 bridges and (mostly) viaducts in the Netherlands. Therefore, the main focus of this research has been on developing circular solutions for concrete viaducts for (governmental) roads. This has resulted in the development of a concept demountable footing to foundation (F2F) dowel connection which has been based on, and is suitable for application in, the proposed design of a standard (circular concrete) viaduct. Besides, attention has been paid to desired monitoring aspects regarding such a circular viaduct, and to the life-cycle costs of a circular viaduct compared to those of the same viaduct, constructed in a traditional way, characterised by a linear life-cycle model.

In general, the results of this research demonstrate the complexity of this topic due to the fact that it comprises a relatively new concept within the construction industry in which many factors are still unknown and relatively little experience is available. At the same time, however, it also reveals the potential and possibilities for the development of new, circular, solutions. Concretely, the main result of this research demonstrates the steps that need to be taken to achieve circular bridge construction. Besides, more specifically, it demonstrates the potential of a demountable connection to solve one of the main bottlenecks that limit current viaduct designs from being circular (i.e. demountable and reusable), which is the footing to foundation connection, by addressing technical, monitoring-related, and life-cycle costs-related aspects. This indicates how this research can help to transform from a linear to a circular construction industry, and therefore, how it can add its value for practice.

In the following subsections, the main results of this research are discussed separately from each other in more detail. An overview of the research outline of which the steps have been followed in order to obtain these results has been shown in Figure 1.3. Besides, the specific assumptions and limitations of these results are addressed.

8.1.1. Standard Viaduct

From the literature study (see subsection 2.2.1), it has become clear that, apart from a change in user behaviour and ownership, and the development of circularity indicators, technical solutions need to be developed for three key action points in order to achieve circular bridge (viaduct) construction. These technical action points are [1]:

- Redefine Brand's shearing layers of longevity for bridges.
- Adjust the DfD and DfAD principles to the specific needs and requirements of bridges.
- Develop a complimentary standardisation scheme without compromising on architectural freedom.

Of these three, the third action point is the most complex task since this is directly related to the actual design of the circular viaduct. Therefore, in Chapters 3 and 4, a standard layout and design of a circular viaduct (i.e. standardisation scheme) has been determined, and a model of this standard viaduct has been developed in SCIA Engineer. The establishment of the former, i.e. a standardisation scheme, is relevant with regards to the development of (circular solutions for) the standard circular concrete viaduct since a certain level of standardisation is required for the concept of circular viaduct construction to be feasible and successfully applied on a large scale. This is demonstrated by the reasoning that if all viaducts consist of standardised elements and components which are compatible and have the same structural properties, these elements and components can, for example, be interchanged, prefabricated and produced in mass. That way similarities start to appear with the demountability properties of a typical (demountable) IKEA product, which was the analogy used for arguing to focus on demountable solutions. In turn, the SCIA Engineer model of the standard viaduct has been developed in order to obtain the critical cross-sectional forces at the F2F interfaces. These are used to design and verify the concept demountable F2F dowel connection. With regards to the development of both the standardisation scheme and the model of the standard viaduct, a number of assumptions have been made, of which the main ones are:

- A simply supported structural system of the viaduct is assumed.
- It is assumed that both superstructure and substructure are constructed with prefabricated concrete elements and components.
- The design of the standard viaduct is based on the layout of, and assumed to cross over, a typical Dutch highway consisting of two times 2 lanes plus 2 emergency lanes, with the possibility to expand to two times 3 lanes plus 2 emergency lanes, resulting in two spans of the standard viaduct equal to 27,70 m (box beam length).
- A deck width of the viaduct of 12 m, suitable to facilitate two times 1 lane and the possibility to include cyclist lanes at both sides is assumed.
- A clearance height of 4,70 m is assumed.
- A crossing angle of 90° is assumed.
- The loads that are taken into account in the model of the standard viaduct are based on a reference period of 100 instead of the intended full service lifetime of 200 years.

The main limitation of the standard viaduct, which directly follows from these assumptions, is that the results of the standard viaduct model (and potentially even more importantly, the results of the developed concept demountable F2F dowel connection) are only applicable for viaducts that are designed within the boundaries of the defined standard viaduct. The influence of varying these assumptions (i.e. different dimensions and/or properties) on the above mentioned results has not been investigated, and is therefore unknown. On the other hand, the standard viaduct is supposed to represent the most common type of viaduct seen in the Netherlands, and therefore, the results are supposed to be representative for the majority of viaducts in the Netherlands.

Besides, the first and second technical action point have also been addressed in Chapter 3 which, however, mainly involves a discussion based on the literature study. This has resulted in a proposal for the division of the circular concrete viaduct in different layers (see section 3.1), and an extensive list of 28 key DfD principles specific for concrete viaducts (see section 3.2) respectively. With regards to the layers of the circular viaduct, the main assumption involves the adopted full service lifetime of the load-bearing elements and components of the viaduct of 200 years. Two points of attention associated with this assumption are discussed later. With regards to the list of 28 key DfD principles, it has to be noted that only the so-called ‘design principles’ have been taken into account, and no policy-related principles regarding regulations, incentives, etc. are considered. This has been decided because of the focus of this research on (developing) design innovations rather than on policy-related innovations.

Despite the fact that this research focused on design-related instead of policy-related aspects, it is recognised that important decisions have to be made with regards to the required change in user behaviour and ownership. A major policy-related aspect involves the decision who will be the owner of the circular viaduct, and who will be responsible for it. Based on the findings of Anastasiades et al. [1], it is pointed out that one option is that the contractor would be the owner of the circular viaduct. This implies that the contractor would not only be in charge of the construction of the viaduct, but also the maintenance, the deconstruction, and reconstruction, and would therefore also carry the responsibility for a safe and serviceable circular viaduct during the full service lifetime. The (local) government should then take its responsibility and specifically request a circular (demountable) viaduct, as Rijkswaterstaat is currently doing by means of the SBIR challenge, and rent the viaduct from the contractor. The question remains, however, if and how contractors can (and are willing to) be held responsible for a full service lifetime of 200 years. Therefore, it is necessary to investigate these policy-related aspects in more detail.

Finally, a point of attention concerns the extended full service lifetime that has been considered, namely 200 instead of 100 years. This both impacts the structural design and the durability (i.e. technical lifespan) of the standard viaduct. With regards to the former, it has been emphasised in section 4.4 that the current building codes don't suffice in reference periods longer than 100 years with respect to the impact on the loads to be considered. Therefore, it could be the case that the structural design doesn't meet new building codes as soon as these are updated and include specific regulations with respect to reference periods longer than 100 years for the purpose of circular construction. This, however, is practically impossible to predict, and should therefore be further investigated. Besides, with regards to ensuring the durability of the (elements and components of the) circular viaduct, it could be simply reasoned to increase the standard, i.e. corresponding to a service lifetime of 100 years, concrete cover thickness with an additional 10 mm in order to guarantee the intended full service lifetime of 200 years, as was done in the design of the prototype circular viaduct [42]. On the other hand, it could for example be the case that in the future less de-icing salt will be used because of its environmental impact. This then might reduce or eliminate the need to increase the cover thickness. This also is practically impossible to include in the current design, and should therefore be further investigated.

8.1.2. Concept Demountable F2F Dowel Connection

The concept of demountable (and reusable) connections in concrete structures originated from the principle of Design for Deconstruction (DfD) and its potential to assist in achieving a circular construction industry, which was elaborated upon in detail in the literature study (see sections 2.1 and 2.2). Besides, a number of different types of concrete DfD connection methods were discussed in section 2.3, of which in the end the pinned dowel type connection was identified to be most suitable to solve one of the main bottlenecks in current viaduct design, preventing it from being demountable. It was argued in section 4.1 that this bottleneck is believed to be the connection between the (abutment) footing and the foundation, referred to as the F2F connection. Therefore, the main focus of this research has been on developing a demountable solution for this bottleneck, which has resulted in a concept demountable F2F dowel connection, and has been described in detail in sections 5.1, 5.2, and 5.5. The development of this concept demountable connection can be considered as the realisation of the main goal of this research, namely that of developing circular (demountable) solutions for concrete viaducts. The process to verify the dimensions and properties of the demountable connection is shown in Figure 5.3. The final (main) dimensions and properties of the connection that have been adopted are (see Figure 5.5):

$$a = 150 \text{ mm} \qquad b = 500 \text{ mm}$$
$$d_b = 80 \text{ mm} \qquad f_c = f_{ck} = 30 \text{ N/mm}^2$$
$$d_{plate} = 508 \text{ mm} \qquad t_{plate} = 20 \text{ mm}$$

This layout of the connection has been verified by means of both a 2D model in SCIA Engineer and an analytical 1D semi-infinite beam on elastic foundation model (see Figure 5.4). Modelling the dowel to concrete interaction by means of schematising the connection as a semi-infinite beam on a (linear) elastic foundation is an important assumption, since the actual behaviour is highly non-linear. However, the conclusion of X.G. He and A.K.H. Kwan [50] was used, which stated that for relatively small dowel deformation, and provided that none of the materials have yielded, the dowel force-deformation relation is linearly elastic, and can therefore be reasonably be estimated by using the semi-infinite beam on elastic foundation theory. This, however, directly indicates the main limitation of the analytical model. The deviation of the results of the analytical linear model from the results obtained by means of the non-linear SCIA Engineer model in terms of dowel deformation has not been quantified. However, it has been randomly verified that the results largely coincide by comparing the results of both models for certain layouts and applied forces.

The main limitation with regards to the SCIA Engineer model has already been explicitly discussed in subsection 5.2.6. It mainly comes down to the fact that the model consists of 2D members which makes it practically impossible to model the actual interaction between the steel dowel and the surrounding concrete. It would therefore require the use of a different finite element program such as DIANA FEA in order to accurately model this interaction by means of a 3D analysis with structural solid elements and structural interface elements. This also directly implies that the results of the model could potentially deviate from the actual behaviour or from the behaviour predicted by a 3D solid model as described above. Therefore, this should be further investigated.

Another main assumption involves the parameters that represent the foundation modulus of concrete under dowel action (k_c and k_d). These are both used in the 2D SCIA Engineer model and in the 1D analytical model. In literature, only one expression, proposed by Soroushian et al. [49], was found which represents this parameter (see expression (2.1)). Since this expression is the only known empirically derived expression encountered in current literature, it has been decided to adopt this expression, based on the argument of X.G. He and A.K.H. Kwan [50] that it is the best option to be used until more test data are available. Because of both the influence of the parameter throughout the verification process of the demountable connection (see Figure 5.13), and the large uncertainty with regards to this parameter, sensitivity analyses of the replacing rotational spring stiffness of the dowel connection ($k_{r,con}$) and of the maximum dowel deformation (w_{max}) were performed and described extensively in section 5.4. From these results, it could be concluded that the influence of the foundation modulus parameter on the former is negligible, whereas the influence on the latter was concluded to be sensitive, especially for certain combinations of applied shear force and bending moment, which, however, was expected beforehand.

Besides, two general assumptions and simplifications with regards to the development of the demountable F2F dowel connection were emphasised in subsection 5.2.6, which involve the assumed geometry of the end plate ($t_{plate} = 20$ mm and $d_{plate} = Ø_{pile} = 508$ mm), and the simplification to ignore the influence of the normal force on the dowel behaviour in the model.

Apart from the main limitations discussed above, both the developed concept solution and the developed model have some other specific limitations. With regards to the applicability of the developed concept solution, a limitation of the connection is that it can only be applied if in none of the load combinations tensile forces arise in the foundation piles (i.e. the (vertical) normal force at the F2F interface should permanently be a compressive force). During development of the connection, this was verified (see Table 5.7).

Besides, with regards to the considered critical cross-sectional forces that were obtained from the standard viaduct model in SCIA Engineer (see section 5.2) and used for the design and verification of the concept connection (see Figure 5.7), it is remarked that different load combinations might exist which have not been taken into account and which result in larger cross-sectional forces. If it would turn out that this is the case, and that the current configuration of the connection (i.e. geometry and properties) doesn't meet the requirements anymore (i.e. maximum deformation and contact stress limits), a different configuration might be found by means of using the design table, which was presented in in section 5.3, as a first estimate.

This design table for a demountable F2F dowel connection with the same principle, but for different parameter ranges, can be considered as an additional result to this research. The ranges for parameters included the concrete compressive strength (f_{ck}), the length over which the protective layer is being applied (a), the dowel diameter (d_b), and the range of shear forces (F_0) and bending moments (M_0) that the connection should be able to withstand. Besides, the limit value for the maximum deformation of the dowel (w_{max}) was the final variable. For the considered parameter ranges (see Table 5.7), and for a fixed limit value of the maximum dowel deformation, the design table indicates the potential feasible configurations for the connection, based on the calculated maximum dowel deformations for each configuration. The maximum deformation is calculated by means of the 1D analytical (linear) model. Therefore, it should at all times be remembered that the results of this design table can deviate from the actual non-linear behaviour. This implies that the design table should merely be used to provide a first estimation for different geometries and properties of the demountable F2F dowel connection, as was indicated in the paragraph above.

8.1.3. Monitoring Plans

In the transition from traditional (linear) to circular viaduct construction, a key aspect concerns monitoring of the circular (demountable) viaduct in general, as well as the demountable solutions (e.g. the demountable F2F dowel connection) in particular. Therefore, monitoring plans were drafted with regards to both categories. The goal of these monitoring plans was to make clear *where* one would like to monitor *what* in order to already be aware of this in the design phase of the circular viaduct.

The monitoring plans were mainly based on the expected dominating deterioration mechanisms, which were found in literature and were described in section 2.4. Finally, this resulted in the monitoring plans as were described in sections 6.2 and 6.3.

The most critical potential deterioration and/or damage is expected to be found at the demountable F2F dowel connections. In particular local damage resulting in abrasion of the concrete cover and/or crushing of the mortar surrounding the dowels is expected. Besides, fatigue-related damage has also been considered, mainly regarding the steel components, caused by cyclic loading of the dowel, and, for example, resulting in crushing of the mortar surrounding the dowel, in crack initiation in the dowel itself, or in fractures in the welds. Finally, it was considered that deterioration and/or damage could potentially occur during handling (i.e. production, transportation, storage, and (de)construction) of the components and elements of the viaduct. This is mainly expected to occur during the process of releasing the mortar infills from the (abutment) footings, assuming that execution variant 3 is chosen as was argued in subsection 8.1.2. The expected damage in that case concerns, for example, abrasion of the concrete surface of the precast (abutment) footings due to a non-smooth release of the (abutment) footing from the cone-shaped mortar infill surrounding the dowel.

By considering these potential deterioration and/or damage scenarios in the design phase of a circular viaduct already, possible measures in order to be able to monitor and identify this deterioration and/or damage can be integrated in the design. This indicates the relevance of the results of this subpart of the research to the main research question.

However, it is recognised that possibly other damage and/or deterioration mechanisms have not been identified, because of the limited research that has been performed within this research. This is because the main focus has been on developing circular (demountable) solutions, whereas it was merely the goal to emphasise and advise on the importance of the monitoring aspects within the overall concept of circular construction instead of performing a detailed monitoring-related analysis.

8.1.4. Life-Cycle Cost Analysis

The idea of performing a life-cycle cost analysis (LCCA) was motivated by the desire to gain insight in the costs of a circular viaduct (i.e. circular alternative) relative to the costs for the same viaduct, constructed in a traditional way, characterised by a linear life-cycle model (i.e. traditional alternative). This was desired since, on the one hand, it was expected that the initial costs for the circular alternative would be higher than for the traditional alternative. This, in fact, turned out to be case, since the results showed an increase of the initial investment costs (i.e. the costs for the first life-cycle) associated with the circular alternative of more than 40% for the reference scenario and more than 70% for the upper limit estimation (see Table 7.4). On the other hand, it was also believed that over time, as more life-cycles would be realised within a certain fixed full service lifetime, the circular alternative would eventually become cheaper than the traditional alternative, which has also been confirmed. Based on the reference scenario, a reduction in life-cycle costs of 16% would namely be realised after only 2 life-cycles, whereas a reduction of 14% would be achieved after 3 life-cycles according to the upper limit scenario. Besides, the more life-cycles are realised, the higher these predicted reductions become (see Table 7.4). This is mainly caused by the fact that all elements and components of the circular alternative can be reused and thus, in the most optimistic scenario, no material/production costs have to be made for 200 years. Generally, these results indicate the potential of a circular viaduct to be feasible from a financial perspective, which was the goal of this subpart.

However, it is important to realise that these results have to be interpreted with care, since the LCCAs are limited and simplified, and rely on rough estimations and a number of key assumptions. These estimations and assumptions, which relate to the demarcation of the analysis scope and the main starting points (see section 7.1), as well as the assumptions made to estimate the build-up of the life-cycle costs of the traditional (see section 7.2) and circular (see section 7.3) alternatives, have extensively been described. The fact that the LCCAs are limited and simplified also directly highlights the main limitation of the results. One has to be well aware of all these assumptions, and should be careful with the results in order to prevent hasty conclusions. On the other hand, the intention of the LCCAs was not that much aimed at estimating the exact costs of both alternatives, but rather on the comparison between the life-cycle costs of both in order to demonstrate the potential of a circular viaduct compared to a traditional alternative from a financial perspective. Since the life-cycle costs for both alternatives were build up in a similar way, it is believed that this comparison is relatively accurate.

With regards to the circular alternative, a major point of attention concerns the research and development (R&D) costs. These costs, which actually should be considered as an investment, are related to developing, testing, and producing circular solutions which can be applied in the circular viaduct. An example of these costs would be the costs related to further development of the concept demountable footing to foundation (F2F) dowel connection into a final design. The main question that should be answered is who should pay these costs, i.e. make this one-time R&D investment. It is expected that the most effective scenario would be that the government (i.e. Rijkswaterstaat) would provide funds to institutions, engineering firms and contractors who come up with good ideas in order to stimulate and support this development, which is basically the current procedure in the SBIR challenge [9].

Finally, it is referred to what was addressed in subsection 1.1.2, namely that the larger the scale on which circular products and services eventually will be applied, the more cost reduction and effectiveness can be achieved. Also, it is emphasised that these analyses are solely focused on the monetary costs, whereas the benefits of the circular alternative compared to the traditional alternative from an environmental perspective are not taken into account, but are expected to be considerable. This is due to the fact that the circular alternative is based on the circular life-cycle model, which results in a more sustainable model in which the production of waste and pollution, and loss of embodied energy is reduced, while the service life of (the elements and components of) the circular viaduct is extended. Recognising and considering both the economic and environmental costs (and benefits) in a holistic model of sustainable construction is one of the four key themes and principles that significantly impact on the decision making process of designing a building (i.e. viaduct) for future deconstruction according to Crowther [7] (see subsection 2.1.3). Therefore, it might be very interesting to quantify the environmental impact (i.e. costs and benefits) of the circular alternative compared to the traditional alternative. In order to make such a comparison, however, it is first required to develop circularity indicators as was identified in the action plan to achieve circular bridge construction, developed by Anastasiades et al. [1]. Such circularity indicators can then be used to quantify how circular a certain alternative is, and subsequently well-argued decisions can be made on different (circular) design alternatives.

8.2. Conclusions

In this section, first the sub-research questions and the main research question that were drafted in section 1.3 are answered. Finally, the main conclusions that can be drawn from this research are summarised. First of all, however, the main research question is repeated.

Main research question

> What is required in order to transform the traditional (linear) design of a concrete viaduct in the Netherlands into a circular (demountable) viaduct?

The sub-research questions are answered first in the following subsections before a conclusive answer to the main research question is formulated.

8.2.1. Sub-research Question 1

Sub-research question 1 consists of three parts, which are consecutively answered below.

Sub-research question 1a

> What are the key action points for technical solutions that are needed to achieve a circular (demountable) concrete viaduct?

It was discovered during the literature study that a detailed action plan to achieve circular bridge construction was proposed by Anastasiades et al. [1]. This plan distinguishes between user behaviour and ownership aspects, circularity assessment aspects, and aspects regarding technical solutions. In this plan, three key action points regarding technical solutions were addressed, namely (see Table 2.5):

- Redefine Brand's shearing layers of longevity for bridges.
- Adjust the DfD and DfAD principles to the specific needs and requirements of bridges.
- Develop a complimentary standardisation scheme without compromising on architectural freedom.

These three action points were therefore concluded to be the key action points for technical solutions that are needed to achieve a circular (demountable) concrete viaduct. The action points were extensively elaborated upon and discussed in Chapters 3 and 4, and in subsection 8.1.1.

Sub-research question 1b

> What are the main bottlenecks in current viaduct designs which make it unsuitable and/or impossible to be demountable and reusable?

In section 4.1, it was concluded that the main bottlenecks in current concrete viaduct design with regards to demountability issues are found at the locations where different components are being connected. This conclusion was drawn based on the process of collecting and analysing literature, as well as becoming familiar with the current design practice of concrete viaducts in general, and having discussions with professionals (e.g. engineers at Lievense). By means of a 'top-down' analysis of the connections between components of a viaduct, the main bottlenecks in current concrete viaduct designs were identified and shown in Table 4.1. Finally, it was concluded that the main bottleneck is found at the connection between the (abutment) footing and the foundation, referred to as the F2F connection.

Sub-research question 1c

> What is/are possible technical solution(s) for the main bottleneck(s) in current viaduct design?

Since the F2F interface was concluded to be the main bottleneck in current viaduct designs, it was decided to focus on this connection within this research in order to develop a technical (i.e. demountable) solution. This has resulted in the development of a concept demountable F2F dowel connection, based on existing pinned dowel type connections in concrete structures. In Chapter 5, the development of this connection (see Figure 5.5) has extensively been elaborated upon and explained. Finally, out of three variants, execution variant 3 (see Figure 5.2c) was determined to be most feasible.

Possible technical solutions for the other bottlenecks that are listed in Table 4.1 have not been investigated in this research. However, it is expected that similar types of solutions will be suitable for the other bottlenecks. Examples of such types of solutions are other dowel type connections, prestressed connections (e.g. by means of unbonded post-tensioned bars), or, alternatively, moment resisting beam-to-beam connections. In general, these solutions should be developed in compliance with the key DfD principles.

8.2.2. Sub-research Question 2

Sub-research question 2 consists of two parts, which are consecutively answered below.

Sub-research question 2a

> What data regarding a circular (demountable) concrete viaduct and its elements and components is desired to be monitored?

It was found in section 2.4 that both real-time data of the circular viaduct during the use phase, as well as the condition of, and the deterioration and/or damage to, the different elements and components of the circular viaduct at the end of a life-cycle (i.e. during demounting the structure) are desired to be monitored (see Figure 2.15). The former is concerned with guaranteeing the safety and reliability of the structure during its use phase, whereas the latter is performed in order to check and decide whether the elements and components can be reused safely in a new life-cycle. From the perspective of guaranteeing the circularity of the structure, which is the main objective, it was therefore concluded that the latter, which was labelled as a ‘reusability assessment’, is the most important to be monitored.

Sub-research question 2b

> How can the desire for monitoring-related data be incorporated into the design process of a circular concrete viaduct?

By creating two draft versions of monitoring plans, the first with respect to the (standard) circular concrete viaduct in general (see Table 6.1), and the second specific to the developed concept demountable F2F dowel connection (see Table 6.2), an advise has been given on *where* to monitor *what* in order to to be able to incorporate provisions for these monitoring aspects in the design of the circular viaduct. Therefore, it is concluded that for future development of the circular viaduct a monitoring plan should be drafted in the design stage in a similar way as has been done in this research. This monitoring plan should include all physical parameters that are desired to be monitored, and if applicable, where to monitor these parameters in order to be able to incorporate provisions for this purpose in the design of the viaduct. Ideally, these parameters should be coupled to the expected deterioration mechanism(s) and/or damage(s) at these specific locations in order to clarify why these parameters should be monitored.

8.2.3. Sub-research Question 3

Sub-research question 3 consists of two parts, which are consecutively answered below.

Sub-research question 3a

> What are the life-cycle costs of both a circular concrete viaduct and of the same viaduct, constructed in a traditional way, characterised by a linear life-cycle model?

In Chapter 7, the life-cycle costs of both a circular concrete viaduct and of the same viaduct, constructed in a traditional way, characterised by a linear life-cycle model were calculated, based on a number of important starting points and assumptions regarding the analysis scope and the estimation of the build-up of the life-cycle costs of both a traditional and circular alternative. Finally, this resulted in an estimation of the net life-cycle costs for the traditional alternative of €1.290.319,- per life-cycle (see subsection 7.2.5), whereas the net life-cycle costs for the circular alternative differ per life-cycle. Specifically, the life-cycle costs for the first life-cycle of the circular alternative were estimated at €1.848.268,- whereas the estimation of the life-cycle costs for every next life-cycle resulted in €325.410,- (see subsection 7.3.5). This implies an increase of the initial costs for the circular alternative (i.e. costs for the first life-cycle) of 43% compared to the traditional alternative. However, the costs for every next life-cycle of the circular alternative are reduced with 75% compared to the traditional alternative.

Sub-research question 3b

> Under what conditions is the concept of a circular concrete viaduct feasible from a financial perspective in comparison to the same viaduct, constructed in a traditional way, characterised by a linear life-cycle model?

The conditions under which the circular concrete viaduct is feasible from a financial perspective relate to the demarcation of the analysis scope and the main starting points (see section 7.1), as well as the assumptions made to estimate the build-up of the life-cycle costs of the traditional (see section 7.2) and circular (see section 7.3) alternative. Because of the size of this list of starting points and assumptions, which is basically all of sections 7.1–7.3, these are not repeated here. However, the most important general starting points and assumptions (i.e. conditions) are:

- Only the costs related to production and execution (construction costs), maintenance, and removal (deconstruction/demolition costs) of the load-bearing elements and components are taken into account.
- The layout and design of the standard viaduct is used.
- A full service lifetime of 200 years with a maximum of 5 life-cycles is adopted.
- Both alternatives are compared to each other based on the same (assumed) service lifetime per life-cycle.

8.2.4. Main Research Question

After answering the sub-research questions, a final conclusive answer to the main research question can be formulated. The main research question is repeated one more time.

Main research question

> What is required in order to transform the traditional (linear) design of a concrete viaduct in the Netherlands into a circular (demountable) viaduct?

This research aimed to identify what is required in order to transform the traditional (linear) design of a concrete viaduct in the Netherlands into a circular (demountable) viaduct. Based on the answers to subquestion 1 it can be concluded that first of all it is required to address the three main aspects, which are summarised in Table 2.5. These are the development of technical solutions, a change in user behaviour and ownership, and the development of circularity indicators. Considering the second and third aspect, these have not been elaborated upon within this research.

However, regarding the first of these three aspects, this implies that it is required to develop demountable solutions for all bottlenecks which are listed in Table 4.1. With regards to the concept demountable F2F dowel connection that has been developed in this research, it is concluded that the connection provides a potential solution for this specific bottleneck, although it needs to be further developed before it can be applied in a real viaduct. Besides, it is required to develop a final version of the standardisation scheme for the design and layout of a standard circular viaduct, for which a proposal has been made in this research. As soon as such a final standardisation scheme is established, demountable solutions can be developed and applied on a large scale. Based on these results, it can be concluded that a contribution to a future of large scale circular (demountable) viaduct construction from a technical perspective has been made.

Finally, based on the answers to subquestion 2, it can be concluded that it is required to draft a detailed monitoring plan during the design phase of the circular viaduct in order to ensure high-quality reuse of the elements and components, and thereby guaranteeing the circularity of the viaduct during its entire intended full service lifetime of 200 years. Furthermore, based on the answers to subquestion 3, it can be concluded that a more detailed LCCA should be performed to confirm the provisional conclusion that a circular viaduct indeed is feasible from a financial perspective (i.e. is cheaper) compared to a traditional alternative after 2 (or 3 according to the upper limit scenario) life-cycles. These results indicate that there is a large potential for the concept of circular viaduct construction to be achieved in the Netherlands, not only from a technical, but also from a durability (i.e. monitoring-related) and a financial perspective.

8.2.5. Main Conclusions

Finally, the main conclusions that can be drawn based on the results of this research are summarised.

- A final version of a standardisation scheme for the design and layout of a standard circular viaduct, for which a proposal has been made in this research, is required and will contribute to the transformation of the traditional (linear) design of a concrete viaduct in the Netherlands into a circular (demountable) viaduct which can be applied on a large scale.
- Provided that the critical cross-sectional forces at the F2F interfaces don't change significantly as a consequence of making a final version of the standard viaduct (specifically no tensile forces in the foundation piles should occur), it can be concluded that execution variant 3 of the developed concept demountable F2F dowel connection has proved to be a potential solution for the connections between the (abutment) footings and foundation.
- Provided that the loads that have been considered are representative for a reference period of 200 years, it can be concluded that the intended full service lifetime of 200 years of the (elements and components of the) circular concrete viaduct is realisable by means of drafting a detailed monitoring plan during the design stage of this circular viaduct, addressing both real-time monitoring aspects and monitoring aspects regarding the reusability assessment..
- Based on the current available information, the assumptions, and the resulting outcomes, the (preliminary) conclusion can be drawn that a circular viaduct is feasible from a financial perspective, assuming that it is reasonable for the circular viaduct to be reused at least once (i.e. at least perform 2 full life-cycles) during a reference period of 200 years.

8.3. Recommendations

The following recommendations are made for future research and development of both the circular concrete viaduct in general and the demountable footing to foundation (F2F) dowel connection specifically.

1. **Standard circular viaduct**
   It is recommended to develop a final version of the standardisation scheme for the design and layout of a standard circular viaduct. This implies that not only the load-bearing elements and components are considered, but also the finishing and safety provisions, such as an asphalt layer, safety barriers and parapets (i.e. Skin and Services layers, see Table 3.1) are incorporated in order to realise a finished standard circular viaduct. The main point of attention in this respect concerns the separation of layers, i.e. how to ensure that the components in different layers (e.g. box beams and asphalt layer) can be removed safely and in a circular way. Besides, different 'versions' of a standard circular viaduct could be developed in order to accommodate different spans, deck widths, or crossing angles. Finally, it is recommended to investigate what the optimal scenario is regarding the ownership of, and responsibility for the circular viaduct during its lifetime. One option has been suggested in section 8.1.1, but a more preferred scenario might exist.

2. **Demountable F2F dowel connection**
   Based on the results of this research, it is recommended that the development of the concept demountable F2F dowel connection is continued in order to realise a final design of the connection, appropriate for execution. For this purpose, it is explicitly recommended to develop a 3D finite element model of the connection, and to eventually design and execute lab tests on the connection to verify both the modelled behaviour and to gain more insight in the foundation modulus of concrete under dowel action (i.e. the k_c and k_d parameters). Besides, to overcome one of the main limitations of the current concept connection, it is suggested to investigate an alternative solution in which the dowel (or other, e.g. a prestressed bar) penetrates the entire (abutment) footing and is externally fixated. This way, the connection is able to transfer tensile (vertical) normal forces between the foundation piles and the (abutment) footing. An impression of such an alternative solution is shown in Figure 8.1. Finally, with regards to the most feasible execution variant, namely variant 3, it is recommended to test a prototype connection in order to verify whether smooth deconstruction is realised by means of applying demoulding oil to the surface of the cone-shaped holes, and whether this still holds also after, for example, a life-cycle of 80 years. Besides, during the same experiment, it is recommended to verify whether filling the voids with mortar works properly and, if so, what is an appropriate execution protocol.

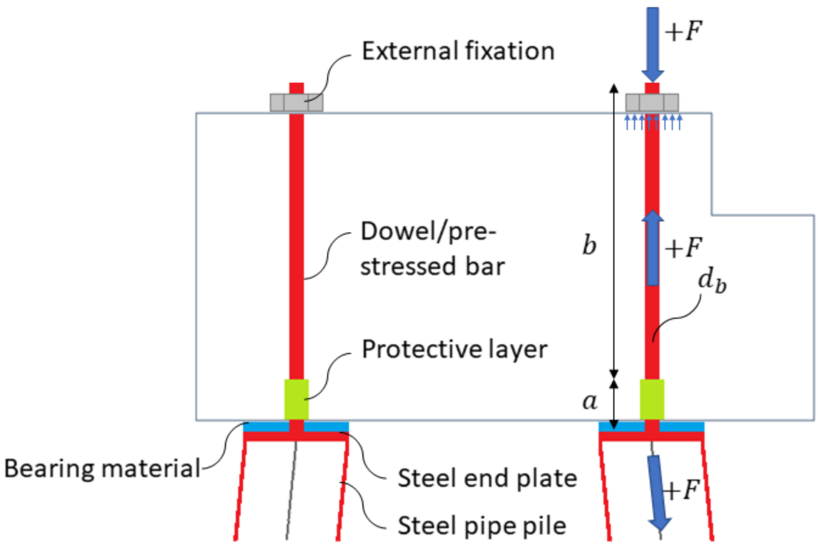


Figure 8.1: Impression of alternative demountable connection which is able to transfer tensile normal forces

3. **Different types of demountable solutions**
As soon as a final version of the standardisation scheme for the design and layout of a standard circular viaduct has been determined, it is recommended, in fact it is required, to develop different types of demountable solutions in order to solve all the bottlenecks in current concrete viaduct design which prevent it from being demountable. It is strongly recommended to develop these solutions with the principal of DfD kept in mind.

4. **Building codes**
It is recommended that building codes begin to include specific rules and regulations as well as to address methodologies on how to design (circular solutions for) circular structures. Based on what was found in this research, the building codes should address how to account for reference periods longer than 100 years, both regarding structural design (i.e. impact on loads to be considered) and durability aspects (e.g. concrete cover thickness to be applied to guarantee the technical lifespan). Besides, it is recommended to draft a guideline (e.g. step-wise procedure and/or checklist) explaining how to design a circular structure. This will assist and motivate designers and engineers to actually develop a circular viaduct, or circular structures in general, and will therefore accelerate the transition from a linear to a circular construction industry.

5. **Monitoring**
First of all, it is recommended to perform a more detailed analysis with regards to the desired monitoring aspects in order to both verify the currently expected damage and/or deterioration mechanisms and to identify possible other mechanisms. Besides, it should be investigated how provisions, for example for monitoring equipment, should be physically incorporated into the design of the circular viaduct. Furthermore, a detailed deconstruction plan including an extensive reusability assessment (i.e. step-wise procedure/checklist) should be drafted. Finally, an overarching data system should be developed in order to, among others, provide the relevant information to enable the reuse of elements and components, to keep track of the stock status, and to match supply and demand.

6. **LCCA**
Considering the vast amount of assumptions that have been made in the current LCCAs, it is recommended that a more detailed analysis and comparison is done in order to verify the business case (i.e. to check whether a circular alternative is really financially feasible). Besides, in order to account for the environmental impact of a circular concrete viaduct, for example in comparison to the traditional alternative, it is recommended to perform a life-cycle assessment (LCA), or a multi-cycle assessment (MCA), as was opted by Anastasiades et al. [1]. Additionally, it might be interesting to investigate how the standard circular viaduct in general and the concept demountable F2F dowel connection specifically score on other indicators such as TRL (Technology Readiness Levels), which are used by Rijkswaterstaat to assess circular innovations [41].

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Normative References

The exact versions of the referenced building codes are listed here. The codes are obtained from https://connect.nen.nl/home/detail by means of the TU Delft license, unless stated differently.

[I] **NEN-EN 1990+A1+A1/C2:2019** Eurocode: Grondslagen voor het constructief ontwerp

a. **NEN-EN 1990+A1+A1/C2/NB:2019** Nationale bijlage bij NEN-EN 1990+A1:2006+A1:2006/C2:2019

[II] **NEN-EN 1991-2+C1:2015** Eurocode 1: Belastingen op constructies - Deel 2: Verkeersbelasting op bruggen

a. **NEN-EN 1991-2+C1:2015/NB:2019** Nationale bijlage bij NEN-EN 1991-2+C1

[III] **NEN-EN 1992-1-1+C2:2011** Eurocode 2: Ontwerp en berekening van betonconstructies - Deel 1-1: Algemene regels en regels voor gebouwen

a. **NEN-EN 1992-1-1+C2/NB+A1:2020** Nationale bijlage bij NEN-EN 1992-1-1+C2

[IV] **NEN-EN 1992-2+C1:2011** Eurocode 2: Ontwerp en berekening van betonconstructies - Betonnen bruggen - Regels voor ontwerp, berekening en detaillering

a. **NEN-EN 1992-2+C1:2011/NB:2016** Nationale bijlage bij NEN-EN 1992-2+C1

[V] **NEN-EN 1090-2:2018** Het vervaardigen van staal- en aluminiumconstructies - Deel 2: Technische eisen voor staalconstructies

[VI] **NEN-EN 1337** Opleggingen voor bouwkundige en civieltechnische toepassingen

[VII] **NEN-EN 10060:2003** Hot rolled round steel bars for general purposes - Dimensions and tolerances on shape and dimensions

[VIII] **NEN-EN 12699:2015** Execution of special geotechnical works - Displacement piles

[IX] **NEN-EN 13670:2009** Het vervaardigen van betonconstructies

[X] **NEN-EN 15050:2007+A1:2012** Vooraf vervaardigde betonproducten - Brugelementen

[XI] **Richtlijnen Ontwerp Kunstwerken (ROK) 1.4** Rijkswaterstaat GPO, April 2017. URL http://publicaties.minienm.nl/documenten/richtlijnen-ontwerp-kunstwerken-rok-1-4

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b. **Richtlijnen Ontwerp Kunstwerken Bijlagendocument deel B 1.4** Rijkswaterstaat GPO, April 2017. URL http://publicaties.minienm.nl/documenten/richtlijnen-ontwerp-kunstwerken-rok-1-4

[XII] **fib Model Code for Concrete Structures 2010** Wilhelm Ernst & Sohn Verlag für Architektur und Technische Wissenschaften GmbH & Co. KG, 2013. URL https://ebookcentral-proquest-com.tudelft.idm.oclc.org/lib/delft/detail.action?docID=1443878#

A

Explanation of 27 Key Principles

On the next pages, a copy of the pages from the paper by Crowther [7], explaining the 27 key principles for DfD, is added.

resource input than the recycling of base materials. In a society where all energy has some environmental cost, and indeed where most is produced through major environmentally damaging processes such as the burning of fossil fuels, any strategy that reduces energy and resource use has environmental advantages. Buildings might, for example, be better designed for the reuse of components rather than simply the recycling of materials. In reality it will be advantageous for buildings to be designed for all of these levels of 'recycling' since the future reuse possibilities of a building cannot be accurately predicted decades before eventual disassembly.

An understanding of the hierarchy of recycling offers guidance on *what* to disassemble for any given end-of-life scenario. It must be noted that it may not always be preferable to design for disassembly at building or component level. It is quite possible that for a particular project there are other environmental concerns such as autonomous energy generation, or the avoidance of all toxic content, that may outweigh the benefits of a design for disassembly strategy. This is why the holistic picture of a sustainable construction industry is needed to guide this decision making process.

5.0 PRINCIPLES OF DESIGN FOR DISASSEMBLY

These three broad themes of a model for environmentally sustainable construction, time related building layers, and a recycling hierarchy, are important in assisting to manage the process of design for disassembly. They do not, however, answer the question of *how* to design for disassembly. For that, a number of design principles, or guidelines, are required.

While the design for disassembly of buildings is not common practice, there are a number of important historic examples of buildings that have been disassembled, either by design or otherwise, that can offer significant information about the technical aspects of such disassembly, these include: traditional and vernacular timber buildings, temporary buildings for military use such as the Nissen hut, the Dymaxion projects of Buckminster Fuller, the Fun Palace of Cedric Price, the Centre George Pompidou, Lloyds of London, and several of the projects of Nicholas Grimshaw. Review of these buildings and many others, some realised projects and some conceptual investigations, reveals a pattern of common solutions or approaches to the difficulties of designing for disassembly. These common approaches offer recurring principles as design guidance for architects and building designers.

Design for disassembly principles

1. **Use recycled and recyclable materials** – to allow for all levels of the recycling hierarchy, increased use of recycled materials will also encourage industry and government to develop new technologies for recycling, and to create larger support networks and markets for future recycling.
2. **Minimise the number of different types of materials** – this will simplify the process of sorting during disassembly, and reduce transport to different recycling locations, and result in greater quantities of each material.
3. **Avoid toxic and hazardous materials** – this will reduce the potential for contaminating materials that are being sorted for recycling, and will reduce the potential for health risks that might otherwise discourage disassembly.
4. **Avoid composite materials and make inseparable subassemblies from the same material** – in this way large amounts of one material will not be contaminated by a small amount of a foreign material that cannot be easily separated.
5. **Avoid secondary finishes** – such coatings may contaminate the base material and make recycling difficult. Where possible use materials that provide their own suitable finish or use mechanically separable finishes (Note: some protective finishes such as galvanising may still on balance be desirable since they extend the service life of the component despite disassembly or recycling problems).
6. **Provide standard and permanent identification of material types** – many materials such as plastics are not easily identifiable and should be provided with a non-removable and non-contaminating identification mark to allow for future sorting, such a mark could provide information on material type, place and time or origin, structural capacity, toxic content, etc.
7. **Minimise the number of different types of components** – this will simplify the process of sorting and reduce the number of different disassembly procedures to be undertaken, it will also make component reuse more attractive due to greater numbers of fewer components.
8. **Use mechanical connections rather than chemical ones** – this will allow the easy separation of components and materials without force, reduce contamination of materials, and reduce damage to components.
9. **Use an open building system where parts of the building are more freely interchangeable and less unique to one application** – this will allow alterations in the building layout through relocation of components without significant modification.
10. **Use modular design** – use components and materials that are compatible with other systems both dimensionally and functionally. This type of modular co-ordination, not only has assembly advantages, but clearly also has disassembly advantages, such as standardisation of disassembly procedure and a broader market for reused components.
11. **Use construction technologies that are compatible with standard, simple, and 'low-tech' building practice and common tools** – specialist technologies will make disassembly difficult to perform and a less attractive option,

particularly for the user. Specialist technologies, materials, and systems that have limited application today may not be readily available in the future when a building is to be disassembled.

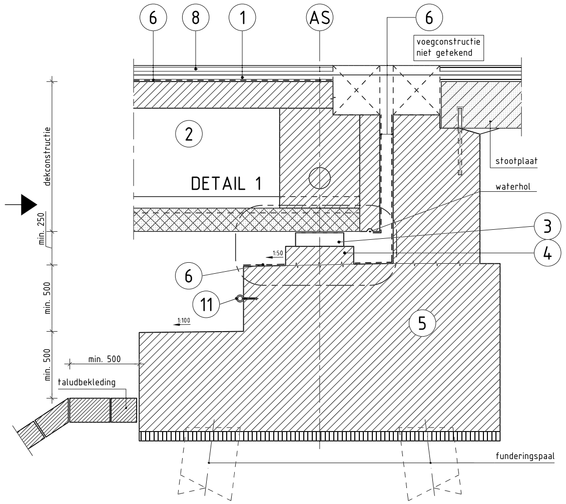
12. **Separate the structure from the cladding, internal walls, and services** – to allow for parallel disassembly such that some parts or systems of the building may be removed without affecting other parts. Most construction methods can be considered as being either a system of load-bearing walls, or a system of separate structural frame and in-fill. The system of separate frame and in-fill is by far the more compatible of the two, with a range of disassembly requirements.
13. **Provide access to all parts of the building and to all components** – ease of access will allow ease of disassembly, allow access for disassembly from within the building if possible.
14. **Make components and materials of a size that suits the intended means of handling** – allow for various handling operations during assembly, disassembly, transport, reprocessing, and re-assembly. The handling of building materials and components is an important consideration in any building, more so if the building is to be disassembled and components later re-assembled.
15. **Provide a means of handling and locating components during the assembly and disassembly procedure** – handling may require points of attachment for lifting equipment as well as temporary supporting and locating devices. The provision of a means of handling components is not often considered in building design because the current approach within the building industry is that a component will only be handled once during the initial assembly.
16. **Provide realistic tolerances to allow for manoeuvring during disassembly** – the repeated assembly and disassembly process may require greater tolerance than for the manufacture process or for a one-off assembly process.
17. **Use a minimum number of fasteners or connectors** – to allow for easy and quick disassembly and so that the disassembly procedure is not complex or difficult to understand. Such a principle will assist in the repair of the component or in the rebuilding of it, though it is not so relevant for the reclaiming (for recycling) of the material, which might be recovered by simply breaking the component.
18. **Use a minimum number of different types of fasteners or connectors** – to allow for a more standardised process of assembly and disassembly without the need for numerous different tools and operations.
19. **Design joints and connectors to withstand repeated use** – to minimise irreparable damage or distortion of components and materials during repeated assembly and disassembly procedures, to allow for the rigors of repeated assembly and disassembly.
20. **Allow for parallel disassembly rather than sequential disassembly** – so that components or materials can be removed without disrupting other components or materials. Where this is not possible make the most reusable or 'valuable' parts of the building most accessible, to allow for maximum recovery of those components and materials that are most likely to be reused.
21. **Provide permanent identification of component type** – in a coordinated way with material information and total building system information, ideally electronically readable to international standards.
22. **Use a structural grid** – the grid dimension and orientation should be related to the materials used such that structural spans are designed to make the most efficient use of material type and allow coordinated relocating of components such as cladding. This will also result in more components of same/standard size, and the grid responds to issues of material efficiency.
23. **Use prefabricated subassemblies and a system of mass production** – to reduce site work and allow greater control over component quality and conformity. The prefabrication of these components reduces the amount of on-site work required and thereby eases the process of assembly, and later disassembly, of the building.
24. **Use lightweight materials and components** – this will make handling easier and quicker, making disassembly and reuse a more attractive option. This will also allow disassembly for regular maintenance and replacement of parts.
25. **Permanently identify points of disassembly** – so as not to be confused with other design features and to sustain knowledge on the component systems of the building. As well as indicating points of disassembly, it may be necessary to indicate disassembly procedures as instructions.
26. **Provide spare parts and on-site storage for them** – particularly for custom designed parts, both to replace broken or damaged components and when required for minor alterations to the building design. Storage for spare components is an integral part of the building design.
27. **Retain all information on the building construction systems and assembly and disassembly procedures** – efforts should be made to retain and update information such as 'as built' drawings, including all reuse and recycling potentials as an assets register. The retention of such complete information about the whole building enhances its potential value for relocation, reuse, or recycling.

It is apparent from this list of design for disassembly principles that there will be many occasions when there will be a conflict between some of them. For example, the need to 'minimise the number of different material types' will not always be compatible with the need to 'use light weight materials'. In such a case the potential

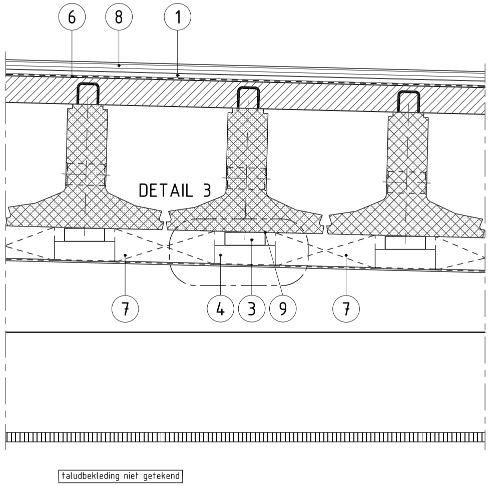
B

RTD 1010 Standard Details Girder Bridge

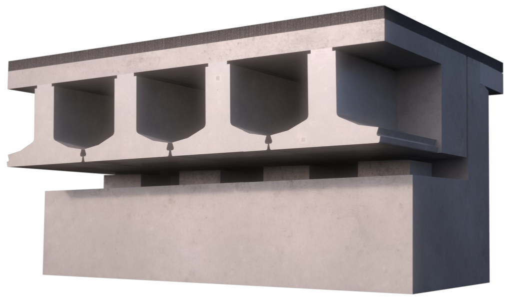
On the next pages, standard details from Rijkswaterstaat (document RTD 1010) of girder decks (inverted T-profiles) are added [XIb].



DOORSNEDE OPLEGGING T-LIGGER T.P.V. EINDSTEUNPUNT
schaal 1:20

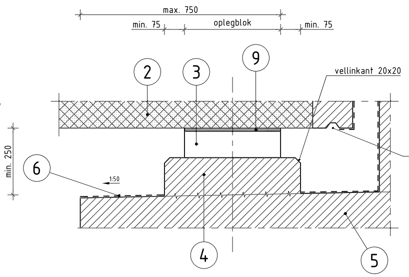


AANZICHT A (T-LIGGERS)
schaal 1:20

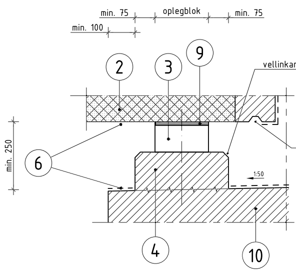


3D VIEW

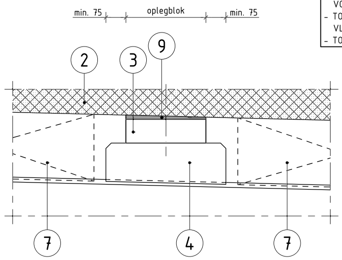
- OPLEGBLOK VOOR PLAATSEN ONTVETTEN
- GEEN CURING MIDDEL OF HYDROFOBEERMIDDEL TOEPASSEN OP BOVENZIJDE OPSTORT, CEMENTHUID VERWIJDEREN VAN OPLEGVLAK VOOR PLAATSEN OPLEGBLOK.
- TOLERANTIE BOVENKANT OPSTORT = 0,003 RAD TOV HORIZONTALE VLAK IN ALLE RICHTINGEN.
- TOLERANTIE PLAATSING OPLEGBLOK IN X,Y,Z RICHTING = +/- 3MM.



DETAIL 1
schaal 1:10



DETAIL 2
schaal 1:10

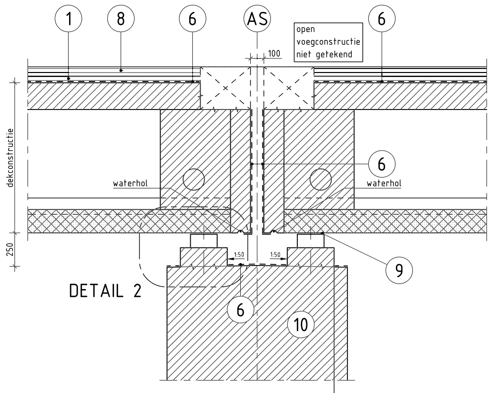


DETAIL 3
schaal 1:10

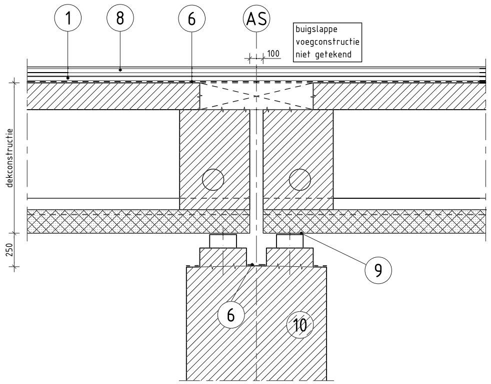
VERKLARING:

- gewapend beton
- gewapend geprefabriceerd beton
- voorgespannen geprefabriceerd beton
- epoxy gebonden mortel
- doorsnede asfalt
- werkvloer

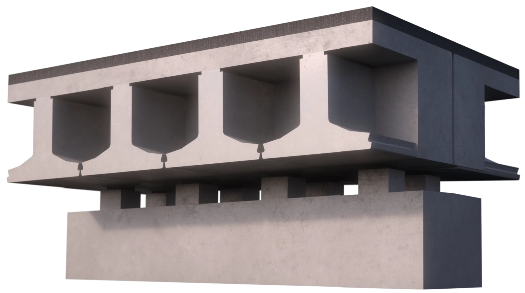
TOELICHTING			
POS.	ONDERDEEL	MATERIAAL AANDUIDING	AFMETING IN MM / SPECIFICATIES
1	VERHARDING	DICHTE ASFALT VERH.	MIN.: D=50
2	DEKCONSTRUCTIE	PREFAB VOORGESPANNEN BETON	T-LIGGER STANDAARD LEVERANCIER
3	OPLEGBLOK	RUBBER / STAAL	
4	OPSTORT	GEWAPEND BETON	
5	LANDHOOFD	GEWAPEND BETON	
6	OPPERVLAKTEBESCHERMINGSSYSTEEM	HYDROFOBEERLAAG	RTD 1002
7	RUIMTE VOOR PLAATSEN VIJZEL		
8	WEGVERHARDING	OPEN DEKLAAG	D=70
9	KEG	EPOXY GEBONDEN MORTEL	
10	TUSSENSTEUNPUNT	GEWAPEND BETON	
11	VEILIGHEIDSLIJN ANKER	RVS BETONANKER MET OOG EN RVS KABEL	



DOORSNEDE OPLEGGING T.P.V. TUSSENSTEUNPUNT
schaal 1:20
dilatatievoeg



DOORSNEDE OPLEGGING T.P.V. TUSSENSTEUNPUNT
schaal 1:20
met buigslappe voegconstructie



3D VIEW

OPMERKINGEN :
- ALLE MATEN IN MM
- GEEN VERLOREN BEKISTING TOEPASSEN.
- KEG AANBRENGEN OP OPLEGBLOK.

Rijkswaterstaat
Ministerie van Infrastructuur en Milieu

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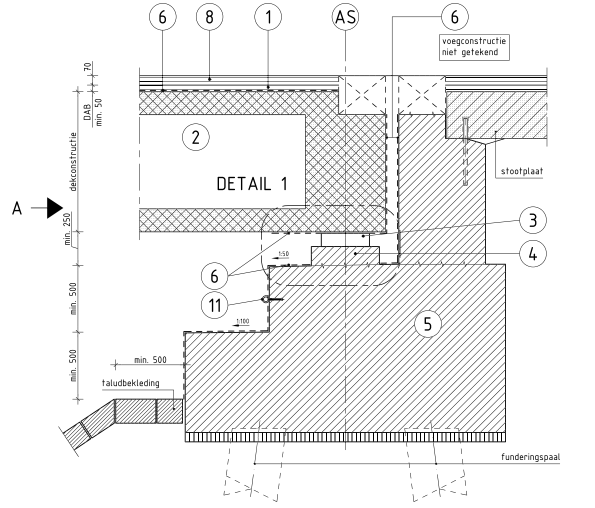
RTD 1010 - STANDAARDDETAILS BETONNEN BRUGGEN
OPLEGGINGEN
DEKCONSTRUCTIE T-LIGGERS

getekend	WAGEMAKER	03-12-2018	behoort bij	formaat A1 schaal div.
gecontroleerd	WAGEMAKER	03-12-2018	blad in bladen	regnr
vrijgegeven	RWS-GPO	03-12-2018	status DEFINITIEF versie 3.0	teknr RWS-OPLEG-02

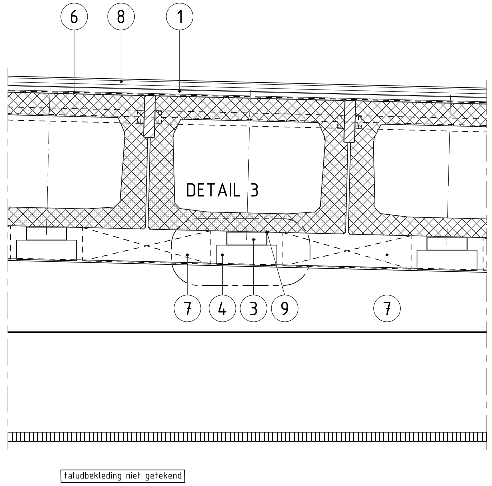
C
RTD 1010 Standard Details Box Beam Bridge

On the next pages, standard details from Rijkswaterstaat (document RTD 1010) of box beam decks are added [XIb].



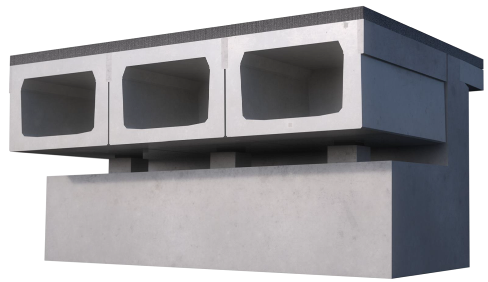
DOORSNEDE OPLEGGING KOKERLIGGER T.P.V. EINDSTEUNPUNT

schaal 1:20



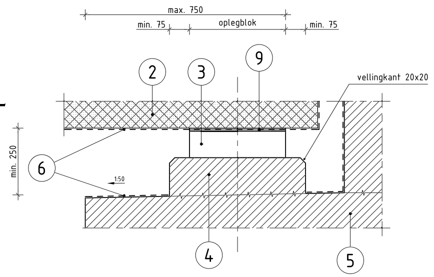
AANZICHT A (KOKERLIGGERS)

schaal 1:20



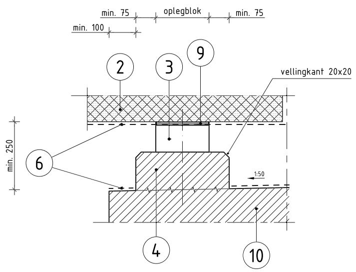
3D VIEW

- OPLEGBLOK VOOR PLAATSEN ONTVETTEN
- GEEN CURING MIDDEL OF HYDROFOBEERMIDDEL TOEPASSEN OP BOVENZIJDE OPSTORT, CEMENTHUID VERWIJDEREN VAN OPLEGVLAK VOOR PLAATSEN OPLEGBLOK.
- TOLERANTIE BOVENKANT OPSTORT = 0,003 RAD TOV HORIZONTALE VLAK IN ALLE RICHTINGEN.
- TOLERANTIE PLAATSING OPLEGBLOK IN X,Y,Z RICHTING = +/- 3MM.



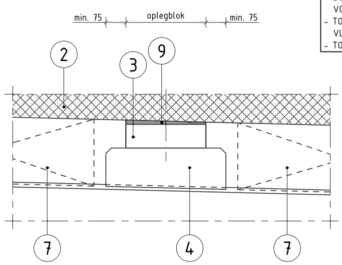
DETAIL 1

schaal 1:10



DETAIL 2

schaal 1:10



DETAIL 3

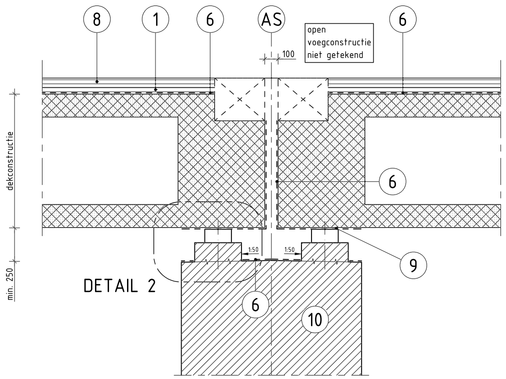
schaal 1:10

VERKLARING:

- gewapend beton
- gewapend geprefabriceerd beton
- voorgespannen geprefabriceerd beton
- epoxy gebonden mortel
- doorsnede asfalt
- werkvloer

TOELICHTING

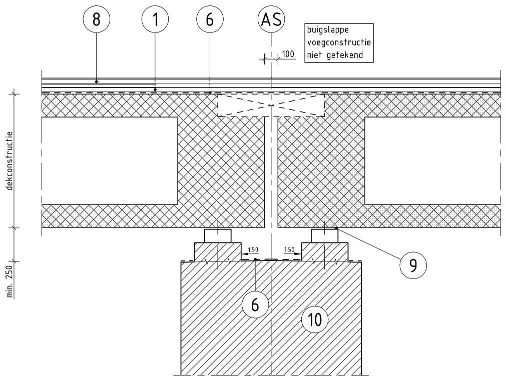
POS.	ONDERDEEL	MATERIAAL AANDUIDING	AFMETING IN MM / SPECIFICATIES
1	VERHARDING	DICHTE ASFALT VERH.	MIN.: D=50
2	DEKCONSTRUCTIE	PREFAB VOORGESPANNEN BETON	KOKERLIGGER STANDAARD LEVERANCIER
3	OPLEGBLOK	RUBBER / STAAL	
4	OPSTORT	GEWAPEND BETON	
5	LANDHOOFD	GEWAPEND BETON	
6	OPPERVLAKTEBESCHERMINGSSYSTEEM	HYDROFOBEERLAAG	RTD 1002
7	RUIMTE VOOR PLAATSEN VIJZEL		
8	WEGVERHARDING	OPEN DEKLAAG	D=70
9	KEG	EPOXY GEBONDEN MORTEL	
10	TUSSENSTEUNPUNT	GEWAPEND BETON	
11	VEILIGHEIDSLIJN ANKER	RVS BETONANKER MET OOG EN RVS KABEL	



DOORSNEDE OPLEGGING T.P.V. TUSSENSTEUNPUNT

schaal 1:20

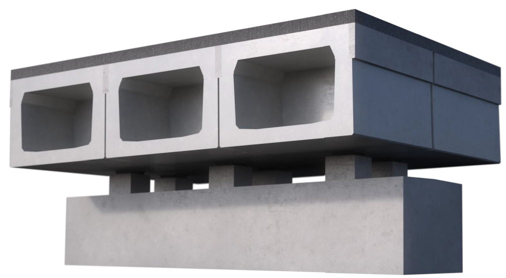
dilatatievoeg



DOORSNEDE OPLEGGING T.P.V. TUSSENSTEUNPUNT

schaal 1:20

met buigslappe voegconstructie



3D VIEW

OPMERKINGEN :
- ALLE MATEN IN MM
- GEEN VERLOREN BEKISTING TOEPASSEN.
- KEG AANBRENGEN OP OPLEGBLOK.

Rijkswaterstaat
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RTD 1010 - STANDAARDDETAILS BETONNEN BRUGGEN

OPLEGGINGEN

DEKCONSTRUCTIE KOKERLIGGERS

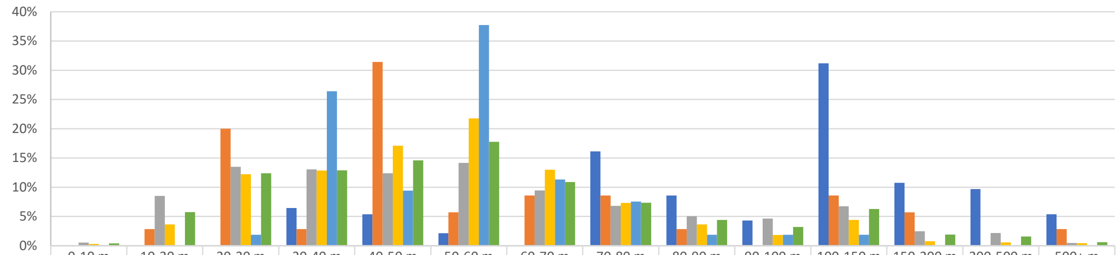
getekend	WAGEMAKER	03-12-2018	behoort bij		formaat A1 schaal div.
gecontroleerd	WAGEMAKER	03-12-2018	blad in bladen		regnr
vrijgegeven	RWS-GPO	03-12-2018	status DEFINITIEF	versie 3.0	teknr RWS-OPLEG-01

D

Frequency Distribution Diagrams of Viaduct Characteristics

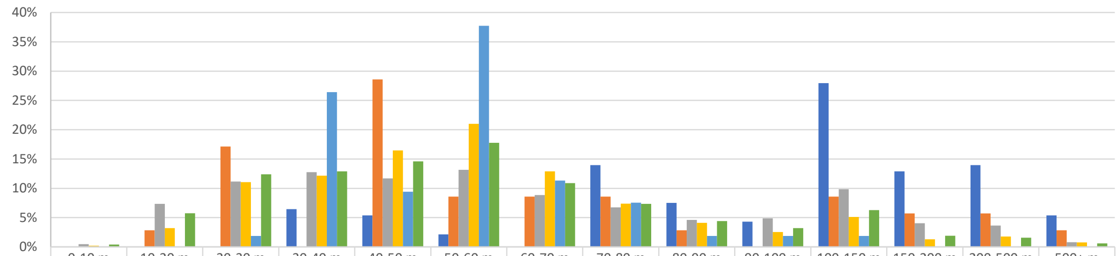
On the next pages, frequency distribution diagrams of 5 dimensional characteristics and of the location of viaducts in the road layout are shown resulting from an analysis of 3384 existing viaducts in the Netherlands.

Frequency distribution object type length



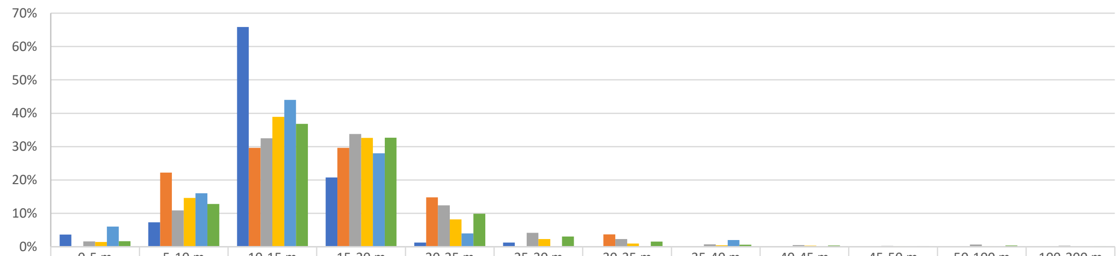
	0-10 m	10-20 m	20-30 m	30-40 m	40-50 m	50-60 m	60-70 m	70-80 m	80-90 m	90-100 m	100-150 m	150-200 m	200-500 m	500+ m
Box beam	0%	0%	0%	6%	5%	2%	0%	16%	9%	4%	31%	11%	10%	5%
Voided slab	0%	3%	20%	3%	31%	6%	9%	9%	3%	0%	9%	6%	0%	3%
Girder	1%	9%	14%	13%	12%	14%	9%	7%	5%	5%	7%	2%	2%	0%
Slab	0%	4%	12%	13%	17%	22%	13%	7%	4%	2%	4%	1%	1%	0%
"Springwerk"	0%	0%	2%	26%	9%	38%	11%	8%	2%	2%	2%	0%	0%	0%
Combined	0%	6%	12%	13%	15%	18%	11%	7%	4%	3%	6%	2%	2%	1%

Frequency distribution object length



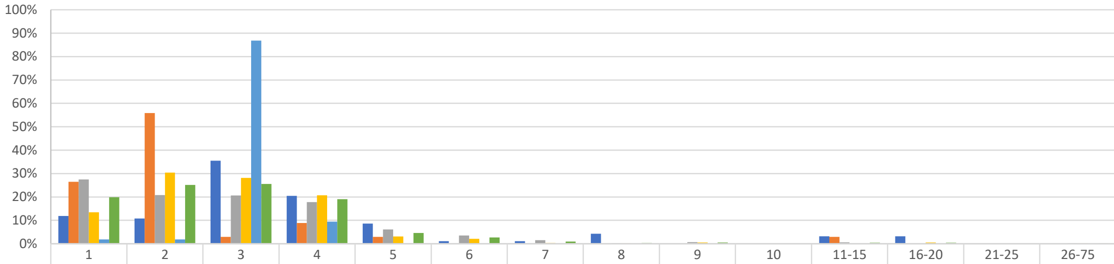
	0-10 m	10-20 m	20-30 m	30-40 m	40-50 m	50-60 m	60-70 m	70-80 m	80-90 m	90-100 m	100-150 m	150-200 m	200-500 m	500+ m
Box beam	0%	0%	0%	6%	5%	2%	0%	14%	8%	4%	28%	13%	14%	5%
Voided slab	0%	3%	17%	0%	29%	9%	9%	9%	3%	0%	9%	6%	6%	3%
Girder	0%	7%	11%	13%	12%	13%	9%	7%	5%	5%	10%	4%	4%	1%
Slab	0%	3%	11%	12%	16%	21%	13%	7%	4%	3%	5%	1%	2%	1%
"Springwerk"	0%	0%	2%	26%	9%	38%	11%	8%	2%	2%	2%	0%	0%	0%
Combined	0%	6%	12%	13%	15%	18%	11%	7%	4%	3%	6%	2%	2%	1%

Frequency distribution object width



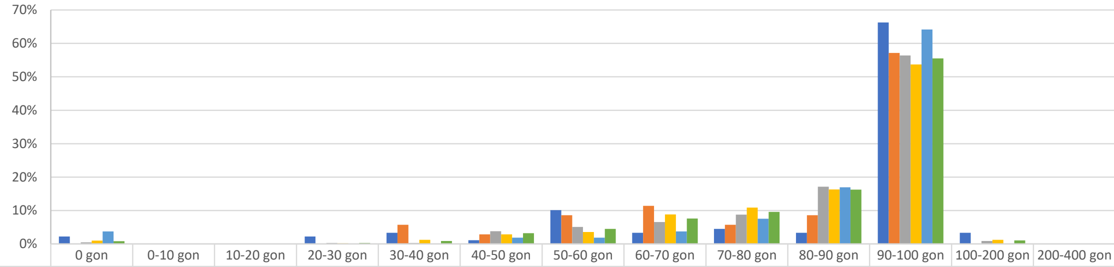
	0-5 m	5-10 m	10-15 m	15-20 m	20-25 m	25-30 m	30-35 m	35-40 m	40-45 m	45-50 m	50-100 m	100-200 m
Box beam	4%	7%	66%	21%	1%	1%	0%	0%	0%	0%	0%	0%
Voided slab	0%	22%	30%	30%	15%	0%	4%	0%	0%	0%	0%	0%
Girder	2%	11%	32%	34%	12%	4%	2%	1%	0%	0%	1%	0%
Slab	1%	15%	39%	33%	8%	2%	1%	0%	0%	0%	0%	0%
"Springwerk"	6%	16%	44%	28%	4%	0%	0%	2%	0%	0%	0%	0%
Combined	2%	13%	37%	33%	10%	3%	2%	1%	0%	0%	0%	0%

Frequency distribution number of spans



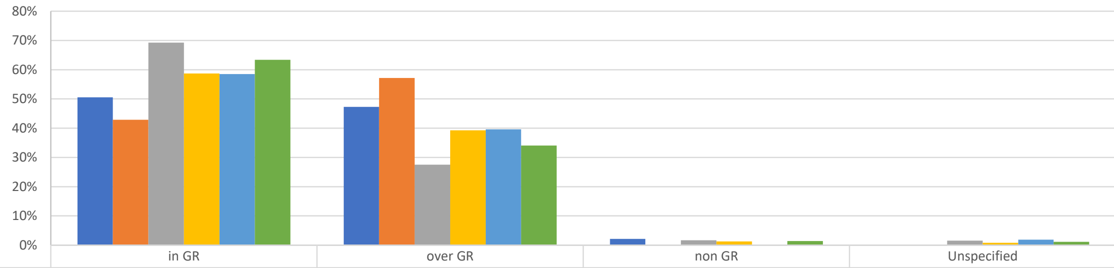
	1	2	3	4	5	6	7	8	9	10	11-15	16-20	21-25	26-75
Box beam	12%	11%	35%	20%	9%	1%	1%	4%	0%	0%	3%	3%	0%	0%
Voided slab	26%	56%	3%	9%	3%	0%	0%	0%	0%	0%	3%	0%	0%	0%
Girder	27%	21%	21%	18%	6%	4%	2%	0%	1%	0%	1%	0%	0%	0%
Slab	13%	30%	28%	21%	3%	2%	0%	0%	1%	0%	0%	1%	0%	0%
"Springwerk"	2%	2%	87%	9%	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%
Combined	20%	25%	26%	19%	5%	3%	1%	0%	1%	0%	0%	0%	0%	0%

Frequency distribution crossing angle



	0 gon	0-10 gon	10-20 gon	20-30 gon	30-40 gon	40-50 gon	50-60 gon	60-70 gon	70-80 gon	80-90 gon	90-100 gon	100-200 gon	200-400 gon
Box beam	2%	0%	0%	2%	3%	1%	10%	3%	4%	3%	66%	3%	0%
Voided slab	0%	0%	0%	0%	6%	3%	9%	11%	6%	9%	57%	0%	0%
Girder	0%	0%	0%	0%	0%	4%	5%	7%	9%	17%	56%	1%	0%
Slab	1%	0%	0%	0%	1%	3%	4%	9%	11%	16%	54%	1%	0%
"Springwerk"	4%	0%	0%	0%	0%	2%	2%	4%	8%	17%	64%	0%	0%
Combined	1%	0%	0%	0%	1%	3%	5%	8%	10%	16%	55%	1%	0%

Frequency distribution object location



	in GR	over GR	non GR	Unspecified
Box beam	51%	47%	2%	0%
Voided slab	43%	57%	0%	0%
Girder	69%	28%	2%	2%
Slab	59%	39%	1%	1%
"Springwerk"	58%	40%	0%	2%
Combined	63%	34%	1%	1%

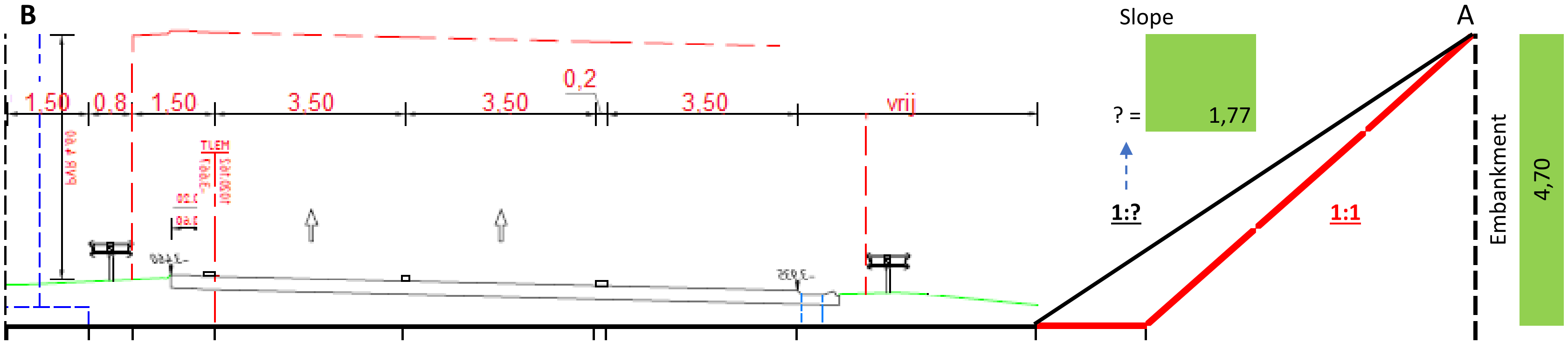
E

Calculation of Standardised Beam Length

On the next page, a printout of the spreadsheet that was created to calculate the required (standardised) box beam length is added. The spreadsheet uses the assumption that the 2-span viaduct has a vertical symmetry plane at the intermediate support.

Parameters
Variables

27250 mm



2x 2 lanes + emergency lane

	Int. sup.	Rail	Int. dist.	Lane 1	Lane 2	Emerg. lane	Free (1)		Embankment
W = 37,90 m	1,50	0,8	1,50	3,50	3,50	3,50	4,45		8,30

2x 3 lanes + emergency lane

	Int. sup.	Rail	Int. dist.	Lane 1	Lane 2	Lane 3	Emerg. lane	Free (2)	Embankment
W_{future} = 42,60 m	1,50	0,8	1,50	3,50	3,50	3,50	3,50	3,30	4,70

<u>Widths:</u>

Int. support	3,00	m
Rail	0,80	m
Int. distance	1,50	m
Lane	3,50	m
Line	0,20	m
Emerg. lane	3,50	m

<u>Int. sup.:</u>

Spacing	100	mm

(minimum = 100 mm)

<u>Heights:</u>

Clearance	4,60	m
Toleran.	0,10	m

<u>Abutment:</u>

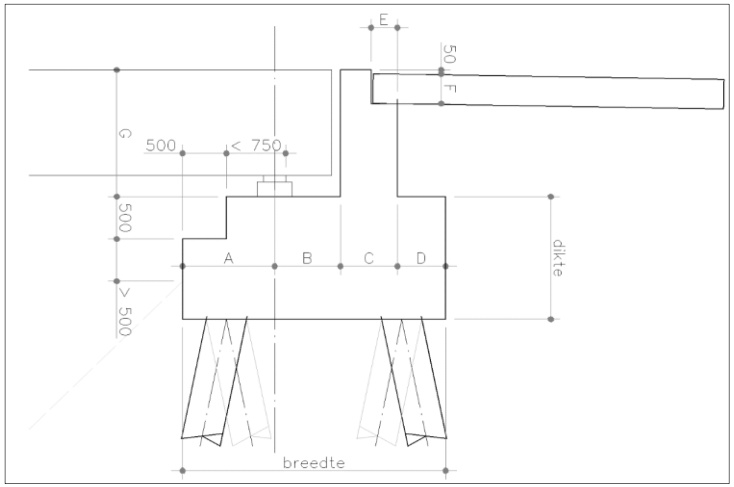
A*	1250	mm
"B"**	500	mm

* max (500 + 750)

** distance centre of bearing to end of beam

Results:

Grid size	A-B	27250	mm
Grid size	B-C	27250	mm
Beam length		27700	mm



F

Orthotropy Box Beam Deck for SCIA Input

On the next pages, a printout of the spreadsheet (in Dutch) that was used to calculate the orthotropic properties of the box beam deck of the standard viaduct is added. The properties are calculated based on the dimensions of the SKK 900 profile.

LIEVENSE adviseurs ingenieurs

Variables
SCIA input parameters

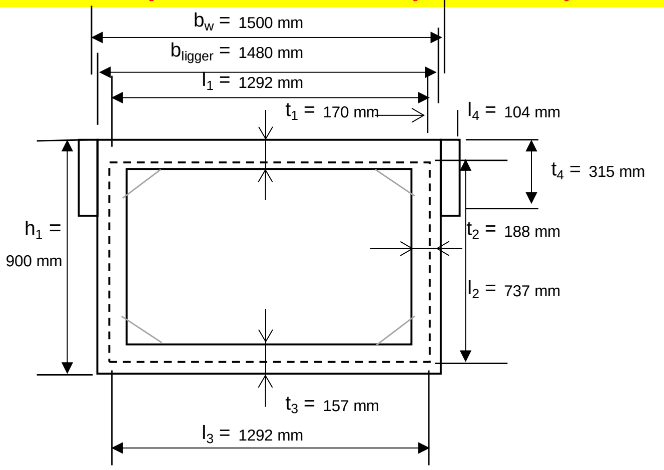
Titel: Orthotropie dek met kokerliggers tbv Scia invoer *Versie Spreadsheet : 1.(2019)*

Invoergegevens dek met kokerliggers

Betonkwaliteit prefab liggers		C60/75	
Betonkwaliteit voegen		C30/37	
Elasticiteitsmodulus prefab liggers	$E_{cm;1}$ =	39100	[N/mm²]
Elasticiteitsmodulus voegen	$E_{cm;2}$ =	32900	[N/mm²]
Dwarscontractiecoëfficient	ν =	0,2	[-]
Uitwendige hoogte ligger	h_1 =	900	[mm]
Breedte ligger	b_{ligger} =	1480	[mm]
H.o.h.-afstand liggers	b_w =	1500	[mm]
Dikte bovenflens	t_1 =	170	[mm]
Lijfdikte	t_2 =	188	[mm]
Dikte onderflens	t_3 =	157	[mm]
Hoogte druklaag/voeg	t_4 =	315	[mm]
Afschuining binnenzijde (niet constr.)			[mm]
A_{koker} incl voeg		N.v.t.	[mm²]
$A_{massief}$ incl voeg		1344600	[mm²]

γ_{beton} = 25 [kN/m³]

Als dwarsvsp: $E_{ongescheurd}$; als geen dwarsvsp: $E_{gescheurd}$ (= 50% $E_{ongescheurd}$)



Buigstijfheid langsrichting; samengesteld profiel:

	A [mm²]	z [mm] (tov onderzijde)	I [mm⁴]
Kokerprofiel	6,81E+05	448	7,100E+10
Voeg	6,30E+03	743	5,418E+08
Samengesteld profiel	6,87E+05	457	7,300E+10

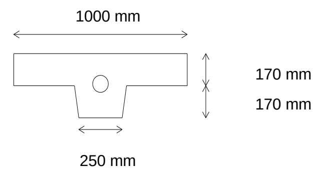
D_{11} =	$(E_1 * I_y) / b_w$ =	1903	[MNm]

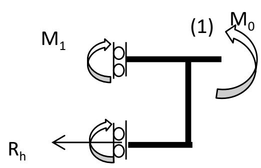
Buigstijfheid dwarsrichting

<u>Gegevens bovenflens</u>

Breedte	b_{flens} =	1000	[mm]
Breedte rib, onderzijde	b_{rib} =	250	[mm]
Hoogte rib	t_{rib} =	170	[mm]
Afschuining rib		25	[mm]
	z_{bf} =	219	[mm]
	EI_1 =	6,059E+10	[Nmm²]
	EI_2 =	1,083E+10	[Nmm²]
	EI_3 =	6,305E+09	[Nmm²]
	EI_4 =	8,569E+10	[Nmm²]

Dwarsdoorsnede bovenflens





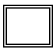
Eenheidsmoment	M_0 =	1000	[kNm/m]
α =	$(2EI_1 * l_2)$ $(/6EI_2 * l_1)$ =	1,06	[-]
β = 2 +	$(3l_1 * EI_2) / (l_2 * EI_3)$ =	11,04	[-]
M_1 =	$M_0 - M_2$ =	670	[kNm/m]
M_2 =	$M_0/(2α-α/β+1)$ =	330	[kNm/m]
M_3 =	$M_2/β$ =	30	[kNm/m]
	$φ_0$ =	8,36E-06	[rad]
	$φ_1$ =	7,14E-06	[rad]
	$φ_2$ =	3,06E-06	[rad]

D_{22} =	$(M_0 \times b_w) / (2 \times φ_0)$ =	90	[MNm]
D_{12} =	$ν * D_{22}$ =	18	[MNm]

Torsiestijfheid samengesteld profiel

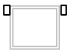
C_{red} = 0,60 Gerekend wordt met 40% reductie in de torsiestijfheid tgv scheurvorming

Dunwandig kokerprofiel:



$4{*}A_k^2 / (l_1/t_1 + 2 \times (l_2/t_2) + l_3/t_3)$ =	1,53E+11	[mm^4]
A_k =	9,52E+05	[mm^2]

Voegen:



$1/3 * l_4 * t_4^3$ =	2,17E+09	[mm^4]
$I_{T;totaal}$ =	1,55E+11	[mm^4]

K_{xy} =	$G_1 \times I_t / b_w$ =	1,69E+12 [Nmm]	**1686**	**[MNm]**
K_{yx} =	$G_2 \times h_2^3 / 6$ =	7,14E+10 [Nmm]	**71**	**[MNm]**

$\mathbf{D_{33}}$ =	$\mathbf{C_{red}} \times (K_{xy} + K_{yx}) / 4$ =	**264**	**[MNm]**

Afschuif buigstijfheid

G_1 =	$E_1 / (2 (1 + v))$=	16292	[MPa]
G_2 =	$E_2 / (2 (1 + v))$=	13708	[MPa]

$\mathbf{D_{44}}$ =	**5/6 x G_1** x b_{lijf} x h_1 / b_w =	**3063**	**[MN/m]**
$\mathbf{D_{55}}$ =	**5/6 x G_2** x h_2 =	**3598**	**[MN/m]**

Membraan parameters

d_{11} = **18662** [MN/m] $= (E_1 * h_{fict}) / (1 - v^2)$

h_{fict} = 458,2 [mm] $= A_{tot} / b_w$

d_{22} = **10795** [MN/m] $= (E_2 * h_2) / (1 - v^2)$

d_{12} = **2159** [MN/m] $= v * d_{22}$

d_{33} = **5777** [MN/m] $= G_{12} * h_{gem}$

G_{12} = 14944 [MPa] $= \sqrt{(E_1 * E_2)} / (2 (1 + v))$

h_{gem} = 387 [mm] $= (h_{fict} + h_2) / 2$

G

Horizontal Moduli of Subgrade Reaction according to Ménard for SCIA Input

On the next pages, a printout of the spreadsheet (in Dutch) that was used to calculate the horizontal moduli of subgrade reactions according to énard is added.

LIEVENSE adviseurs ingenieurs

Horizontale beddingen conform methode Ménard	**Versie:**	**0.4**
Variables	**Projectnummer:**	
SCIA input parameters	**Datum:**	29-08-20

Gegevens

Geometrie paal		rond		Equivalente diameter	D_{eq}	0,508	m
				Equivalente straal	R	0,254	m
Diameter paal	D	0,508	m	Referentiestraal	R_0	0,300	m

Grondlagen voor berekening horizontale beddingsconstanten

	Grond-soort	γ_{droog} [kN/m^3]	γ_{nat} [kN/m^3]	φ [°]	schelpfct. [-]	q_c [N/mm^2]	$q_{c,red}$ [N/mm^2]	Bk laag [m NAP]	Ok laag [m NAP]
Laag 1	klei	17	17			1,0	1,0	0,0	-16,0
Laag 2	zand	18	20			10,0	10,0	-16,0	-20,0
Laag 3								-20,0	
Laag 4									
Laag 5									
Laag 6									
Laag 7									
Laag 8									
Laag 9									

Reductiefactor conusweerstand t.g.v. ontgraving

Grondwaterstand	0,00	m NAP
Maaiveld initieel	0,00	m NAP
Maaiveld ontgraving	0,00	m NAP

Situatie 1: palen niet-trillingsarm geïnstalleerd na ontgraven

Situatie 1:

$$q_{c;z;ontgr} = q_{c;z} \times \frac{\sigma'_{v;z;ontgr}}{\sigma'_{v;z;0}}$$

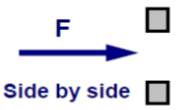
Situatie 2:

$$q_{c;z;ontgr} = q_{c;z} \times \sqrt{\frac{\sigma'_{v;z;ontgr}}{\sigma'_{v;z;0}}}$$

	Grond-soort	Niveau bk [m NAP]	[kN/m^2]	[kN/m^2]	Niveau mid [m NAP]	$\sigma'_{v,mid,init}$ [kN/m^2]	$\sigma'_{v,mid}$ [kN/m^2]	red. q_c [-]
Laag 1	klei	0,0	112,0	112,0	-8,0	56,0	56,0	1,00
Laag 2	zand	-16,0	152,0	152,0	-18,0	132,0	132,0	1,00
Laag 3	PPN	-20,0						

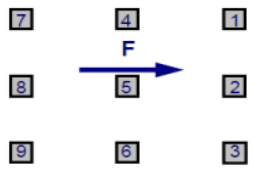
Reductiefactor t.g.v. schaduwwerking palen

H.o.h. afstand dwarsrichting	m		
Reductiefactor naast elkaar staande palen		e_1	1,00 -



$$e_1 = 0.64\left(\frac{s}{D}\right)^{0.34} \quad voor\ 1 \le \frac{s}{D} \le 3.75\ en\ e = 1.0\ voor\ \frac{s}{D} \ge 3.75$$

H.o.h. afstand lengterichting	m		
Reductiefactor voorste rij achter elkaar staande palen (1-3)		e_2	1,00 -
Reductiefactor andere rijen achter elkaar staande palen (4-9)		e_3	1,00 -



$$e_2 = 0.7\left(\frac{s}{D}\right)^{0.26} \quad voor\ 1 \le \frac{s}{D} \le 4.0\ en\ e = 1.0\ voor\ \frac{s}{D} \ge 4.0$$

$$e_3 = 0.48\left(\frac{s}{D}\right)^{0.38} \quad voor\ 1 \le \frac{s}{D} \le 7.0\ en\ e = 1.0\ voor\ \frac{s}{D} \ge 7.0$$

Totale gemiddelde reductiefactor schaduwwerking	red. schad	1,00 -

LIEVENSE adviseurs ingenieurs

Algemene formules

	α			β		
	over consolid.	**normally consolid.**	decomp. weathered	β_{laag}	β_{gem}	β_{hoog}
Veen	-	**1,00**	-	3,00	3,50	4,00
Klei	1,00	**0,67**	0,50	2,00	2,50	3,00
Leem	0,67	**0,50**	0,50	1,00	1,50	2,00
Zand	0,50	**0,33**	0,33	0,70	0,85	1,00
Gravel	0,33	**0,25**	0,25	0,50	0,60	0,70

Als $R < R_0$: *van toepassing*

$$\frac{1}{k_h} = \frac{2 \times R}{E_p} \times \frac{4 \times (2{,}65)^{\alpha} + 3 \times \alpha}{18}$$

Als $R \geq R_0$: *niet van toepassing*

$$\frac{1}{k_h} = \frac{1}{3E_p} \times \left(1{,}3 \times R_0 \times \left(\frac{2{,}65 \times R}{R_0} \right)^{\alpha} + \alpha \times R \right)$$

$E_p = \beta \times q_c$

E_p = elasticiteitsmodulus van Ménard

rekening houdend met γ_k = 1,0

Berekeningsblad als: R = 0,25 m < R_0 = 0,30 m

Gemiddelde bedding

	Grond-soort	Niveau bk [m NAP]	$q_{c;red}$ [N/mm^2]	α [-]	β [-]	E_p [kN/m^2]	red. schad [-]	k_h [kN/m^3]	k_h [MN/m^2]
Laag 1	klei	0,0	1,0	0,67	2,50	2.500	1,00	9.100	4,65
Laag 2	zand	-16,0	10,0	0,33	0,85	8.500	1,00	46.000	23,41
		-20,0							

H

Bearing Spring Stiffness for SCIA Input

On the next page, a printout of the spreadsheet (in Dutch) that was used to calculate the vertical and horizontal spring stiffnesses of a single bearing is added.

Berekening van de eigenschappen van een oplegblok

Rechthoekig gelamineerd oplegblok met tenminste 2 staalplaten

Variables

SCIA input parameters

Controle volgens NEN-EN 1337-3:2005 (en)

Gegevens oplegblok

a =	300	mm	breedte blok (// aan overspanning)	T_q =	73	mm	totale rubber dikte
b =	400	mm	lengte blok (⊥ overspannning)	a' =	290	mm	breedte staalplaat
t_b =	100	mm	totale dikte van het oplegblok	b' =	390	mm	lengte staalplaat
t_i =	8	mm	dikte rubberlaag	G =	0,9	N/mm^2	glijmodulus rubber
n =	8	-	aantal rubberlagen	f_y =	235	N/mm^2	vloeigrens staal
t_s =	3	mm	dikte staalplaat	A_1 =	1E+05	mm^2	opp. staalplaat
n_s =	9	-	aantal stalenplaten	l_p =	1360	mm	omtrek staalplaat
$t_{o/b}$ =	4,5	mm	laagdikte omhulling onder en boven	E_b =	2000	N/mm^2	compressiemodulus
$t_{l/r}$ =	5	mm	laagdikte omhulling zijkanten	A =	1E+05	mm^2	opp. blok
$k_{v,blok}$ =	691	MN/m	verticale veerstijfheid oplegblok	$k_v = 1 / (t_i * n / A_1 * 10^3 * (1 / (5 * G * S_1^2) + 1 / E_b))$			
$k_{h,blok}$ =	1,48	MN/m	horizontale veerstijfheid oplegblok	$k_h = 1 / (T_e * 10^3 / (A * G))$			

Calculation of Loads

On the next pages, a printout of the spreadsheet that was used for the calculation of the loads on the standard viaduct is added.

Calculation of loads for standard viaduct

Permanent loads

LC1 Self weight - [...]

a	Deck	Q_{Ek} =	19,0	kN/m	/	1,48	m	=	12,84	kN/m^2	based on SKK 900 profile by Spanbeton
b	Other	*Automatically generated by SCIA Engineer*									

LC2 Dead load

Kerbs	Q_{Ek} =	0,25	m	*	25,0	kN/m^3	=	6,25	kN/m^2	over a width of 1,40 m from both edges
Asphalt	Q_{Ek} =	0,14	m	*	23,0	kN/m^3	=	3,22	kN/m^2	in between boundaries of kerbs
Safety barriers	q_{Ek} =							0,60	kN/m	on 1,40 m from both edges
Parapets	q_{Ek} =							1,00	kN/m	on both deck edges
Soil	Q_{Ek} =	0,5	m	*	20,0	kN/m^3	=	10,00	kN/m^2	
	q_{Ek} =	3,5	m	*	10,0	kN/m^2	=	35,00	kN/m	on footing of intermediate support

LC3 Shrinkage/creep

The shortening of the deck as a consequence of combined shrinkage and creep is inputted as a temperature load

Shortening	ε_{s+c} =	0,3 ‰			
Temperature load	ΔT =		-30,00	K	over entire deck

Traffic loads (gr1a) (EN1991-2, art. 4.3.2 + 4.4.1)

LC11 LM1 - deck- edge - UDL -[...]

a	Lane 1	Q_{Ek} =	$\alpha_{q1} * q_{1k}$ =	1,15	*	9,0	kN/m^2	=	10,35	kN/m^2
b	Lane 2	Q_{Ek} =	$\alpha_{q>1} * q_{2k}$ =	1,40	*	2,5	kN/m^2	=	3,50	kN/m^2
c	Lane 3	Q_{Ek} =	$\alpha_{q>1} * q_{3k}$ =	1,40	*	2,5	kN/m^2	=	3,50	kN/m^2
d	Remaining	Q_{Ek} =	$\alpha_{qr} * q_{rk}$ =	1,00	*	2,5	kN/m^2	=	2,50	kN/m^2

LC12 LM1 - deck - center - UDL - [...]

a-d Lanes 1-3 + rem. *See LC11 for loads in LC12a-d*

LC13 LM1 - T.slab - edge - UDL - [...]

a-d Lanes 1-3 + rem. *See LC11 for loads in LC12a-d*

LC14 LM1 - T.slab - center - UDL - [...]

a-d Lanes 1-3 + rem. *See LC11 for loads in LC12a-d*

LC15		<u>LM1 - edge - TS - Lane 1 - [...]</u>						
	a	Pos. 1 (T.slab)	Q_{Ek} =	α_{Q1} * $(1/2*Q_{1k})$ / (0,4*0,4) =	1,00 * 937,5 kN/m^2	=	937,50 kN/m^2	wheels on edge of transition to deck
	b	Pos. 2 (deck)	Q_{Ek} =	α_{Q1} * $(1/2*Q_{1k})$ / (0,4*0,4) =	1,00 * 937,5 kN/m^2	=	937,50 kN/m^2	center of 4 wheels at half-way of transition joint
	c	Pos. 3 (deck)	Q_{Ek} =	α_{Q1} * $(1/2*Q_{1k})$ / (0,4*0,4) =	1,00 * 937,5 kN/m^2	=	937,50 kN/m^2	wheels on edge of transition to abutment
	d	Pos. 4 (deck)	Q_{Ek} =	α_{Q1} * $(1/2*Q_{1k})$ / (0,4*0,4) =	1,00 * 937,5 kN/m^2	=	937,50 kN/m^2	center of 4 wheels at half-way span
	e	Pos. 5 (deck)	Q_{Ek} =	α_{Q1} * $(1/2*Q_{1k})$ / (0,4*0,4) =	1,00 * 937,5 kN/m^2	=	937,50 kN/m^2	wheels on edge of transition joint
	f	Pos. 6 (deck)	Q_{Ek} =	α_{Q1} * $(1/2*Q_{1k})$ / (0,4*0,4) =	1,00 * 937,5 kN/m^2	=	937,50 kN/m^2	center of 4 wheels at half-way of transition joint
LC16		<u>LM1 - edge - TS - Lane 2 - [...]</u>						
	a-f	Pos. 1-6	Q_{Ek} =	α_{Q2} * $(1/2*Q2_k)$ / (0,4*0,4) =	1,00 * 625,0 kN/m^2	=	625,00 kN/m^2	*see LC12 for locations a-f*
LC17		<u>LM1 - edge - TS - Lane 3 - [...]</u>						
	a-f	Pos. 1-6	Q_{Ek} =	α_{Q3} * $(1/2*Q3_k)$ / (0,4*0,4) =	1,00 * 312,5 kN/m^2	=	312,50 kN/m^2	*see LC12 for locations a-f*
LC18		<u>LM1 - center - TS - Lane 1 - [...]</u>						
	a-f	Pos. 1-6	Q_{Ek} =	α_{Q1} * $(1/2*Q_{1k})$ / (0,4*0,4) =	1,00 * 937,5 kN/m^2	=	937,50 kN/m^2	*see LC12 for locations a-f*
LC19		<u>LM1 - center - TS - Lane 2 - [...]</u>						
	a-f	Pos. 1-6	Q_{Ek} =	α_{Q2} * $(1/2*Q2_k)$ / (0,4*0,4) =	1,00 * 625,0 kN/m^2	=	625,00 kN/m^2	*see LC12 for locations a-f*
LC20		<u>LM1 - center - TS - Lane 3 - [...]</u>						
	a-f	Pos. 1-6	Q_{Ek} =	α_{Q3} * $(1/2*Q3_k)$ / (0,4*0,4) =	1,00 * 312,5 kN/m^2	=	312,50 kN/m^2	*see LC12 for locations a-f*
LC21		<u>Braking force - edge - UDL+TS - [...]</u>						
		UDL	Q_{Ek} =	0,10 * α_{q1} * q_{1k} =	0,10 * 10,4 kN/m^2	=	1,04 kN/m^2	
	c-f	Pos. 3-6	Q_{Ek} =	0,6 * α_{Q1} * $(2*Q_{1k})$ / (4*(0,4*0,4)) =	0,60 * 937,5 kN/m^2	=	562,50 kN/m^2	*see LC12 for locations c-f*
LC22		<u>Acceleration force - edge - UDL+TS - [...]</u>						
		UDL	Q_{Ek} =	0,10 * α_{q1} * q_{1k} =	0,10 * 10,4 kN/m^2	=	1,04 kN/m^2	
	c-f	Pos. 3-6	Q_{Ek} =	0,6 * α_{Q1} * $(2*Q_{1k})$ / (4*(0,4*0,4)) =	0,60 * 937,5 kN/m^2	=	562,50 kN/m^2	*see LC12 for locations c-f*
LC23		<u>Braking force - center - UDL+TS - [...]</u>						
		UDL	Q_{Ek} =	0,10 * α_{q1} * q_{1k} =	0,10 * 10,4 kN/m^2	=	1,04 kN/m^2	
	c-f	Pos. 3-6	Q_{Ek} =	0,6 * α_{Q1} * $(2*Q_{1k})$ / (4*(0,4*0,4)) =	0,60 * 937,5 kN/m^2	=	562,50 kN/m^2	*see LC12 for locations c-f*
LC24		<u>Acceleration force - center - UDL+TS - [...]</u>						
		UDL	Q_{Ek} =	0,10 * α_{q1} * q_{1k} =	0,10 * 10,4 kN/m^2	=	1,04 kN/m^2	
	c-f	Pos. 3-6	Q_{Ek} =	0,6 * α_{Q1} * $(2*Q_{1k})$ / (4*(0,4*0,4)) =	0,60 * 937,5 kN/m^2	=	562,50 kN/m^2	*see LC12 for locations c-f*

Pedestrian + cycle track **(gr1a)** **(EN1991-2, art. 5.3.2)**

LC31	Pedestrians and/or cyclists									
	UDL	Q_{Ek} =	q_{fk} =					5,00	kN/m^2	over entire width of kerbs

Other live loads **(EN1991-1-4 + EC1991-1-5 + ROK1.4)**

LC41	Wind load [...]									
	a +x (dominant)	$q_{Ek,\perp}$ =	$F_{w;x}$ =					7,60	kN/m	perpendicular wind load + (dominant)
	b -x (dominant)	$q_{Ek,\perp}$ =	$F_{w;x}$ =					7,60	kN/m	perpendicular wind load - (dominant)
	c +x (c.w.t.*)	$q_{Ek,\perp}$ =	MAX($F_{w;x}$ * Ψ_0; $F^*_{w;x}$ * Ψ_0) =					5,50	kN/m	perpendicular wind load + (combined with traffic)
	d -x (c.w.t.*)	$q_{Ek,\perp}$ =	MAX($F_{w;x}$ * Ψ_0; $F^*_{w;x}$ * Ψ_0) =					5,50	kN/m	perpendicular wind load - (combined with traffic)
	e +y (dominant)	$q_{Ek,\perp}$ =	0,40 * $F_{w;x}$ =	0,40 *	7,6	kN/m	=	3,04	kN/m	perpendicular wind load + (dominant)
		$q_{Ek,//}$ =	0,40 * $F_{w;x}$ * (L/w) =	0,40 *	35,15	kN/m	=	14,06	kN/m	parallel wind load + (dominant)
	f -y (dominant)	$q_{Ek,\perp}$ =	0,40 * $F_{w;x}$ =	0,40 *	7,6	kN/m	=	3,04	kN/m	perpendicular wind load - (dominant)
		$q_{Ek,//}$ =	0,40 * $F_{w;x}$ * (L/w) =	0,40 *	35,15	kN/m	=	14,06	kN/m	parallel wind load - (dominant)
	g +y (c.w.t.*)	$q_{Ek,\perp}$ =	0,40 * MAX($F_{w;x}$ * Ψ_0; $F^*_{w;x}$ * Ψ_0) =	0,40 *	5,5	kN/m	=	2,20	kN/m	perpendicular wind load + (combined with traffic)
		$q_{Ek,//}$ =	0,40 * MAX($F_{w;x}$ * Ψ_0; $F^*_{w;x}$ * Ψ_0) * (L/w) =	0,40 *	25,44	kN/m	=	10,18	kN/m	parallel wind load + (combined with traffic)
	h -y (c.w.t.*)	$q_{Ek,\perp}$ =	0,40 * MAX($F_{w;x}$ * Ψ_0; $F^*_{w;x}$ * Ψ_0) =	0,40 *	5,5	kN/m	=	2,20	kN/m	perpendicular wind load - (combined with traffic)
		$q_{Ek,//}$ =	0,40 * MAX($F_{w;x}$ * Ψ_0; $F^*_{w;x}$ * Ψ_0) * (L/w) =	0,40 *	25,44	kN/m	=	10,18	kN/m	parallel wind load - (combined with traffic)

LC42	Thermal load - [...]			
	a Yearly +	$\Delta T_{N,exp}$ =	+23,20	K
	b Yearly -	$\Delta T_{N,con}$ =	-29,50	K
	c Daily +	$T_{M,heat,bk}$ =	+10,10	K
		$T_{M,heat,ok}$ =	-2,70	K
	d Daily -	$T_{M,cool,bk}$ =	-4,50	K
		$T_{M,cool,ok}$ =	+1,20	K

Accidental loads **(EN1991-1-7 + ROK1.4)**

LC51		Collision under bridge - [...]				
	a	Pier 1 - par.	F_{dx} =	1000,00	kN	at 1,2 m above road level
	b	Pier 1 - perp.	F_{dy} =	500,00	kN	at 1,2 m above road level
	c	Pier 2 - par.	F_{dx} =	1000,00	kN	at 1,2 m above road level
	d	Pier 2 -perp.	F_{dy} =	500,00	kN	at 1,2 m above road level
	e	Pier 3 - par.	F_{dx} =	1000,00	kN	at 1,2 m above road level
	f	Pier 3 - perp.	F_{dy} =	500,00	kN	at 1,2 m above road level
	g	Pier 4 - par.	F_{dx} =	1000,00	kN	at 1,2 m above road level
	h	Pier 4 -perp.	F_{dy} =	500,00	kN	at 1,2 m above road level
LC52		Collision with edge of deck - [...]				
	a	Pos. 11	F_{dx} =	1000,00	kN	right above abutment support bearing
	b	Pos. 22	F_{dx} =	1000,00	kN	at half-way span
	c	Pos. 33	F_{dx} =	1000,00	kN	right above intermediate support bearing
LC53		Accident on bridge - [...]				
	c-f	Pos. 3-6	Q_{Ek} = α_{Q1} * (1/2*Q_{1k}) / (0,4*0,4) = 1,00 * 937,5 kN/m^2 =	937,50	kN/m^2	wheels on edge of bridge deck; *see LC12 for locations c-f*

J

Load Combinations

On the next pages, a printout of the spreadsheet that was used for drawing up the load combinations, both ULS and SLS, is added.

Envelope ULS STR/GEO + CAL combinations															
Load group	**Relation**						**ULS 1**	**ULS 2**	**ULS 3**	**ULS 4**	**ULS 5**	**ULS 6**	**ULS 7**	**ULS 8**	**CAL 1**	**CAL 2**
							6.10a - gr1a		6.10b - gr1a		6.10b - gr2		6.10b - W (gr2)		6.11b - A1 (gr1a)	6.11b - A1 (gr2)
							Permanent loads									
Permanent	-	LC1	a-b	Self weight - […]			**1,4**	**1,4**	1,25	1,25	1,25	0,9	1,25	0,9	1,0	1,0
		LC2		Dead load			**1,4**	**1,4**	1,25	1,25	1,25	0,9	1,25	0,9	1,0	1,0
		LC3		Shrinkage/creep			**1,4**	**1,4**	1,25	1,25	1,25	1,25	1,25	1,25	1,0	1,0
Load group	**Relation**						***Traffic loads*** $\gamma_{Q,i,traffic}$ = *1,5*								$\gamma_{Q,i,A1,traffic}$ = *1,0*	
				Combination factor -->	*gr1a*	*gr2*										
LM1 - deck - gr1a - UDL	Standard	LC11	a-d	LM1 - deck- edge - UDL -[…]	1,0	0,8	1,2	1,2	**1,5**	1,2	1,2	1,2	0,96	0,96	0,8	0,64
		LC12	a-d	LM1 - deck - center - UDL - […]	1,0	0,8	1,2	1,2	**1,5**	1,2	1,2	1,2	0,96	0,96		0,64
LM1 - T.slab - gr1a - UDL	Standard	LC13	a-d	LM1 - T.slab - edge - UDL - […]	1,0	0,8		1,2		**1,5**						
		LC14	a-d	LM1 - T.slab - center - UDL - […]	1,0	0,8		1,2		**1,5**						
LM1 - gr1a - TS - Lane 1	Exclusive	LC15	a	LM1 - edge - TS - Lane 1 - Pos. 1 (T.slab)	1,0	0,8		1,2		**1,5**						
		LC15	b-f	LM1 - edge - TS - Lane 1 - […]	1,0	0,8	1,2	1,2	**1,5**	1,2	1,2	1,2	0,96	0,96		0,64
LM1 - gr1a - TS - Lane 2	Exclusive	LC16	a	LM1 - edge - TS - Lane 2 - Pos. 1 (T.slab)	1,0	0,8		1,2		**1,5**						
		LC16	b-f	LM1 - edge - TS - Lane 2 - […]	1,0	0,8	1,2	1,2	**1,5**	1,2	1,2	1,2	0,96	0,96	0,8	0,64
LM1 - gr1a - TS - Lane 3	Exclusive	LC17	a	LM1 - edge - TS - Lane 3 - Pos. 1 (T.slab)	1,0	0,8		1,2		**1,5**						
		LC17	b-f	LM1 - edge - TS - Lane 3 - […]	1,0	0,8	1,2	1,2	**1,5**	1,2	1,2	1,2	0,96	0,96	0,8	0,64
LM1 - gr1a - TS - Lane 1	Exclusive	LC18	a	LM1 - center - TS - Lane 1 - Pos. 1 (T.slab)	1,0	0,8		1,2		**1,5**						
		LC18	b-f	LM1 - center - TS - Lane 1 - […]	1,0	0,8	1,2	1,2	**1,5**	1,2	1,2	1,2	0,96	0,96		0,64
LM1 - gr1a - TS - Lane 2	Exclusive	LC19	a	LM1 - center - TS - Lane 2 - Pos. 1 (T.slab)	1,0	0,8		1,2		**1,5**						
		LC19	b-f	LM1 - center - TS - Lane 2 - […]	1,0	0,8	1,2	1,2	**1,5**	1,2	1,2	1,2	0,96	0,96		0,64
LM1 - gr1a - TS - Lane 3	Exclusive	LC20	a	LM1 - center - TS - Lane 3 - Pos. 1 (T.slab)	1,0	0,8		1,2		**1,5**						
		LC20	b-f	LM1 - center - TS - Lane 3 - […]	1,0	0,8	1,2	1,2	**1,5**	1,2	1,2	1,2	0,96	0,96		0,64
Horizontal forces (gr2)	Exclusive	LC21	c-f	Braking force - edge - UDL+TS - […]	0,8	1,0	0,96	0,96	1,2	0,96	**1,5**	**1,5**	1,2	1,2	0,64	0,8
		LC22	c-f	Accelerating force - edge - UDL+TS - […]	0,8	1,0	0,96	0,96	1,2	0,96	**1,5**	**1,5**	1,2	1,2	0,64	0,8
		LC23	c-f	Braking force - center - UDL+TS - […]	0,8	1,0	0,96	0,96	1,2	0,96	**1,5**	**1,5**	1,2	1,2		0,8
		LC24	c-f	Accelerating force - center - UDL+TS - […]	0,8	1,0	0,96	0,96	1,2	0,96	**1,5**	**1,5**	1,2	1,2		0,8
Pedestr. + cycle track (gr1a - UDL)	Standard	LC31		Pedestrians and/or cyclists	0,4	0,4	0,48	0,48	0,6	0,6	0,6	0,6	0,48	0,48	0,32	0,32

Load group	Relation					*Other live loads* $\gamma_{Q,i,other}$ = *1,65*									
Wind - FWk	Exclusive	LC41	a/b/e/f	Wind load [...]	($F_{w;x}$)							**1,65**	**1,65**		
Wind - F*W	Exclusive	LC41	c/d/g/h	Wind load [...] (to combine with traffic loads)	($F^*_{w;x}$)	1,65	1,65	1,65	1,65	1,65	1,65				
Thermal loads (yearly)	Exclusive	LC42	a-b	Thermal load - yearly [...]											
Thermal loads (daily)	Exclusive	LC42	c-d	Thermal load - daily [...]											
Load group	**Relation**					***Accidental loads***								$\gamma_{Q,i,A1}$ = *1,0*	
Accidental loads	Exclusive	LC51	a-h	Collision under bridge - [...]											**1,0**
		LC52	a-c	Collision with edge of deck - [...]											**1,0**
		LC53	c-f	Accident on bridge - [...]										**1,0**	

Envelope SLS - frequent combinations

Load group	Relation						SLS 1	SLS 2	SLS 3	SLS 4	SLS 5	SLS 6				
							Perm.	LM1 - gr1a/2		W	T (yearly)	T (daily)				
							Permanent loads									
Permanent	-	LC1	a-b	Self weight - [...]			**1,0**	1,0	1,0	1,0	1,0	1,0				
		LC2		Dead load			**1,0**	1,0	1,0	1,0	1,0	1,0				
		LC3		Shrinkage/creep			**1,0**	1,0	1,0	1,0	1,0	1,0				
Load group	**Relation**						***Traffic loads***									
							$\gamma_{Q,i,traffic}$ = *1,0*									
				Combination factor -->	*gr1a*	*gr2*										
LM1 - deck - gr1a - UDL	Standard	LC11	a-d	LM1 - deck- edge - UDL -[...]	1,0	1,0		**0,8**	0,8	0,4	0,4	0,4				
		LC12	a-d	LM1 - deck - center - UDL - [...]	1,0	1,0		**0,8**	0,8	0,4	0,4	0,4				
LM1 - T.slab - gr1a - UDL	Standard	LC13	a-d	LM1 - T.slab - edge - UDL - [...]	1,0	1,0			**0,8**	0,4	0,4	0,4				
		LC14	a-d	LM1 - T.slab - center - UDL - [...]	1,0	1,0			**0,8**	0,4	0,4	0,4				
LM1 - gr1a - TS - Lane 1	Exclusive	LC15	a	LM1 - edge - TS - Lane 1 - Pos. 1 (T.slab)	1,0	1,0			**0,8**	0,4	0,4	0,4				
		LC15	b-f	LM1 - edge - TS - Lane 1 - [...]	1,0	1,0		**0,8**	0,8	0,4	0,4	0,4				
LM1 - gr1a - TS - Lane 2	Exclusive	LC16	a	LM1 - edge - TS - Lane 2 - Pos. 1 (T.slab)	1,0	1,0			**0,8**	0,4	0,4	0,4				
		LC16	b-f	LM1 - edge - TS - Lane 2 - [...]	1,0	1,0		**0,8**	0,8	0,4	0,4	0,4				
LM1 - gr1a - TS - Lane 3	Exclusive	LC17	a	LM1 - edge - TS - Lane 3 - Pos. 1 (T.slab)	1,0	1,0			**0,8**	0,4	0,4	0,4				
		LC17	b-f	LM1 - edge - TS - Lane 3 - [...]	1,0	1,0		**0,8**	0,8	0,4	0,4	0,4				
LM1 - gr1a - TS - Lane 1	Exclusive	LC18	a	LM1 - center - TS - Lane 1 - Pos. 1 (T.slab)	1,0	1,0			**0,8**	0,4	0,4	0,4				
		LC18	b-f	LM1 - center - TS - Lane 1 - [...]	1,0	1,0		**0,8**	0,8	0,4	0,4	0,4				
LM1 - gr1a - TS - Lane 2	Exclusive	LC19	a	LM1 - center - TS - Lane 2 - Pos. 1 (T.slab)	1,0	1,0			**0,8**	0,4	0,4	0,4				
		LC19	b-f	LM1 - center - TS - Lane 2 - [...]	1,0	1,0		**0,8**	0,8	0,4	0,4	0,4				
LM1 - gr1a - TS - Lane 3	Exclusive	LC20	a	LM1 - center - TS - Lane 3 - Pos. 1 (T.slab)	1,0	1,0			**0,8**	0,4	0,4	0,4				
		LC20	b-f	LM1 - center - TS - Lane 3 - [...]	1,0	1,0		**0,8**	0,8	0,4	0,4	0,4				
Horizontal forces (gr2)	Exclusive	LC21	c-f	Braking force - edge - UDL+TS - [...]	1,0	1,0		0,8	0,8	0	0	0				
		LC22	c-f	Accelerating force - edge - UDL+TS - [...]	1,0	1,0		0,8	0,8	0	0	0				
		LC23	c-f	Braking force - center - UDL+TS - [...]	1,0	1,0		0,8	0,8	0	0	0				
		LC24	c-f	Accelerating force - center - UDL+TS - [...]	1,0	1,0		0,8	0,8	0	0	0				
Pedestr. + cycle track (gr1a - UDL)	Standard	LC31		Pedestrians and/or cyclists	1,0	1,0		0,8	0,8	0,4	0,4	0,4				

Load group	Relation	***Other live loads*** $\gamma_{Q,i,other}$ = *1,0*													
Wind - FWk	Exclusive	LC41	a/b/e/f	Wind load […]	($F_{w;x}$)				**0,6**						
Wind - F*W	Exclusive	LC41	c/d/g/h	Wind load […] (to combine with traffic loads)	($F^*_{w;x}$)										
Thermal loads (yearly)	Exclusive	LC42	a-b	Thermal load - yearly […]			0,3	0,3	0,3	**0,8**					
Thermal loads (daily)	Exclusive	LC42	c-d	Thermal load - daily […]							**0,8**				
Load	**Relation**	***Accidental loads*** $\gamma_{Q,i,A1}$ = *1,0*													
Accidental loads	Exclusive	LC51	a-h	Collision under bridge - […]											
		LC52	a-c	Collision with edge of deck - […]											
		LC53	c-f	Accident on bridge - […]											

K

Standard Viaduct Model Verification

On the next pages, a printout of the spreadsheet that was used to verify the model by means of comparing the sum of loads ($\sum F_x$, $\sum F_y$, $\sum F_z$) with the calculated loads is added.

Verification of inputted loads into SCIA Engineer

Deck length (1 span):	27,7 m			
Deck length (total):	55,5 m			
Transition slab length:	8,0 m			
Deck width:	12,0 m			
Lane width:	3,0 m			
Axle load area:	0,4 m *	0,4 m	Max. allowable difference:	5,00%

Input							SCIA	Difference
Load case	Number	p/q/F-load	Length	Width	Total	Direction	Total	Check
LC1a	2 *	-12,84 *	27,7 *	12,0 =	-8536 kN	Z	-8536 kN	0,00% → OK
LC1bb	*Loads automatically generated by SCIA Engineer*					Z	-9640 kN	N / A
LC2	2 *	-6,25 *	55,5 *	1,4 =	-971 kN	Z		
	1 *	-3,22 *	55,5 *	9,2 =	-1644 kN	Z		
	2 *	-3,22 *	8,0 *	9,2 =	-474 kN	Z		
	2 *	-0,6 *	55,5	=	-67 kN	Z		
	2 *	-1,0 *	55,5	=	-111 kN	Z		
	1 *	-35,0 *	12,0	=	-420 kN +	Z		
					-3687 kN	Z	-3691 kN	-0,10% → OK
LC3					0 kN	X / Y / Z	0 kN	N / A
LC11a / LC12a	1 *	-10,35 *	55,5 *	3,0 =	-1723 kN	Z	-1723 kN	0,00% → OK
LC11b / LC12b	1 *	-3,5 *	55,5 *	3,0 =	-583 kN	Z	-583 kN	0,00% → OK
LC11c / LC12c	1 *	-3,5 *	55,5 *	3,0 =	-583 kN	Z	-583 kN	0,00% → OK
LC11d / LC12d	1 *	-2,5 *	55,5 *	0,2 =	-28 kN	Z	-28 kN	0,00% → OK
LC13a / LC14a	1 *	-10,35 *	8,0 *	3,0 =	-248 kN	Z	-250 kN	-0,78% → OK
LC13b / LC14b	1 *	-3,5 *	8,0 *	3,0 =	-84 kN	Z	-85 kN	-0,77% → OK
LC13c / LC14c	1 *	-3,5 *	8,0 *	3,0 =	-84 kN	Z	-85 kN	-0,77% → OK
LC13d / LC14d	1 *	-2,5 *	8,0 *	0,2 =	-4 kN	Z	-4 kN	-0,75% → OK
LC15a / LC18a	4 *	-937,5 *	0,4 *	0,4 =	-600 kN	Z	-605 kN	-0,78% → OK
LC15b / LC18b	4 *	-937,5 *	0,4 *	0,4 =	-600 kN	Z	-602 kN	-0,39% → OK
LC15c / LC18c	4 *	-937,5 *	0,4 *	0,4 =	-600 kN	Z	-600 kN	0,00% → OK
LC15d / LC18d	4 *	-937,5 *	0,4 *	0,4 =	-600 kN	Z	-600 kN	0,00% → OK
LC15e / LC18e	4 *	-937,5 *	0,4 *	0,4 =	-600 kN	Z	-600 kN	0,00% → OK

Load case	Number	p/q/F-load	Length	Width	Total	Direction	Total	Check
LC15f / LC18f	4 *	-937,5 *	0,4 *	0,4 =	-600 kN	Z	-600 kN	0,00% → OK
LC16a / LC19a	4 *	-625,0 *	0,4 *	0,4 =	-400 kN	Z	-403 kN	-0,78% → OK
LC16b / LC19b	4 *	-625,0 *	0,4 *	0,4 =	-400 kN	Z	-402 kN	-0,39% → OK
LC16c / LC19c	4 *	-625,0 *	0,4 *	0,4 =	-400 kN	Z	-400 kN	0,00% → OK
LC16d / LC19d	4 *	-625,0 *	0,4 *	0,4 =	-400 kN	Z	-400 kN	0,00% → OK
LC16e / LC19e	4 *	-625,0 *	0,4 *	0,4 =	-400 kN	Z	-400 kN	0,00% → OK
LC16f / LC19f	4 *	-625,0 *	0,4 *	0,4 =	-400 kN	Z	-400 kN	0,00% → OK
LC17a / LC20a	4 *	-312,5 *	0,4 *	0,4 =	-200 kN	Z	-202 kN	-0,78% → OK
LC17b / LC20b	4 *	-312,5 *	0,4 *	0,4 =	-200 kN	Z	-201 kN	-0,39% → OK
LC17c / LC20c	4 *	-312,5 *	0,4 *	0,4 =	-200 kN	Z	-200 kN	0,00% → OK
LC17d / LC20d	4 *	-312,5 *	0,4 *	0,4 =	-200 kN	Z	-200 kN	0,00% → OK
LC17e / LC20e	4 *	-312,5 *	0,4 *	0,4 =	-200 kN	Z	-200 kN	0,00% → OK
LC17f / LC20f	4 *	-312,5 *	0,4 *	0,4 =	-200 kN	Z	-200 kN	0,00% → OK
LC21c / LC23c	1 *	1,04 *	27,7 *	3,0 =	86 kN	X		
	4 *	562,5 *	0,4 *	0,4 =	360 kN +	X		
					446 kN	X	446 kN	0,00% → OK
LC21d / LC23d	1 *	1,04 *	27,7 *	3,0 =	86 kN	X		
	4 *	562,5 *	0,4 *	0,4 =	360 kN +	X		
					446 kN	X	446 kN	0,00% → OK
LC21e / LC23e	1 *	1,04 *	27,7 *	3,0 =	86 kN	X		
	4 *	562,5 *	0,4 *	0,4 =	360 kN +	X		
					446 kN	X	446 kN	0,00% → OK
LC21f / LC23f	1 *	1,04 *	27,7 *	3,0 =	86 kN	X		
	4 *	562,5 *	0,4 *	0,4 =	360 kN +	X		
					446 kN	X	446 kN	0,00% → OK
LC22c / LC24c	1 *	-1,04 *	27,7 *	3,0 =	-86 kN	X		
	4 *	-562,5 *	0,4 *	0,4 =	-360 kN +	X		
					-446 kN	X	-446 kN	0,00% → OK
LC22d / LC24d	1 *	-1,04 *	27,7 *	3,0 =	-86 kN	X		
	4 *	-562,5 *	0,4 *	0,4 =	-360 kN +	X		
					-446 kN	X	-446 kN	0,00% → OK

Load case	Number	p/q/F-load	Length	Width	Total	Direction	Total	Check
LC22e / LC24e	1 *	-1,04 *	27,7 *	3,0 =	-86 kN	X		
	4 *	-562,5 *	0,4 *	0,4 =	-360 kN +	X		
					-446 kN	X	-446 kN	0,00% → OK
LC22f / LC24f	1 *	-1,04 *	27,7 *	3,0 =	-86 kN	X		
	4 *	-562,5 *	0,4 *	0,4 =	-360 kN +	X		
					-446 kN	X	-446 kN	0,00% → OK
LC31	2 *	-5,0 *	55,5 *	1,4 =	-777 kN	Z	-777 kN	0,00% → OK
LC41a	1 *	7,6 *	55,5	=	422 kN	Y	422 kN	0,00% → OK
LC41b	1 *	-7,6 *	55,5	=	-422 kN	Y	-422 kN	0,00% → OK
LC41c	1 *	5,5 *	55,5	=	305 kN	Y	305 kN	0,00% → OK
LC41d	1 *	-5,5 *	55,5	=	-305 kN	Y	-305 kN	0,00% → OK
LC41e	1 *	14,06 *	12,0	=	169 kN	X	169 kN	-0,02% → OK
	1 *	3,04 *	55,5	=	169 kN	Y	169 kN	0,00% → OK
LC41f	1 *	-14,06 *	12,0	=	-169 kN	X	-169 kN	0,00% → OK
	1 *	-3,04 *	55,5	=	-169 kN	Y	-169 kN	0,00% → OK
LC41g	1 *	10,18 *	12,0	=	122 kN	X	122 kN	0,00% → OK
	1 *	2,2 *	55,5	=	122 kN	Y	122 kN	0,00% → OK
LC41h	1 *	-10,18 *	12,0	=	-122 kN	X	-122 kN	0,00% → OK
	1 *	-2,2 *	55,5	=	-122 kN	Y	-122 kN	0,00% → OK
LC42a-d					0 kN	X / Y / Z	0 kN	N / A
LC51a/c/e/g	1 *	1000,0		=	1000 kN	Y	1000 kN	0,00% → OK
LC51b/d/f/h	1 *	-500,0		=	-500 kN	X	-500 kN	0,00% → OK
LC52a-c	1 *	1000,0		=	1000 kN	Y	1000 kN	0,00% → OK
LC53c-f	4 *	-937,5 *	0,4 *	0,4 =	-600 kN	Z	-600 kN	0,00% → OK

L

Calculation of Replacing Rotational Spring Stiffness of Steel End Plate

On the next page, a printout of the MAPLE worksheet that was used to calculate the replacing rotational spring stiffnesses of the steel end plate is added.

> restart;

Plate properties

> $E := 210000 :$

> $d\_pile := \frac{508}{1000} : t\_plate := \frac{20}{1000} : t\_pile := \frac{12.5}{1000} : L\_pile := 20 :$

> $d\_plate := d\_pile : r\_plate := evalf\left(\frac{d\_plate}{2}\right) : w\_plate := 2 \cdot \mathrm{sqrt}\left((1.000000001 \cdot r\_plate)^2 - x^2 \right) :$

> $Iz\_plate := \frac{1}{12} \cdot w\_plate \cdot t\_plate^3 : EI\_plate := E \cdot Iz\_plate \cdot 10^3 :$

> $k\_pile := \frac{(E \cdot 1000) \cdot t\_pile}{L\_pile} \cdot \mathrm{Pi} \cdot r\_plate :$

Solving beam ODEs for BCs

> $ODE1 := EI\_plate \cdot diff(u1(x), x\$4) = 0 : ODE2 := EI\_plate \cdot diff(u2(x), x\$4) = 0 :$

> $sol := dsolve(\{ODE1, ODE2\}, \{u1(x), u2(x)\}) : assign(sol) :$

> $u1 := u1(x) : u2 := u2(x) :$

> $phi1 := -diff(u1, x) : kappa1 := diff(phi1, x) : M1 := EI\_plate \cdot kappa1 : V1 := diff(M1, x) :$

> $phi2 := -diff(u2, x) : kappa2 := diff(phi2, x) : M2 := EI\_plate \cdot kappa2 : V2 := diff(M2, x) :$

> $x := -r\_plate : eq1 := V1 = k\_pile \cdot u1 : eq2 := M1 = 0 :$

> $x := 0 : eq3 := u1 = u2 : eq4 := phi1 = phi2 : eq5 := V1 = V2 : eq6 := M1 = M2 + M0 :$

> $x := r\_plate : eq7 := V2 = -k\_pile \cdot u2 : eq8 := M2 = 0 :$

> $sol2 := solve(\{eq1, eq2, eq3, eq4, eq5, eq6, eq7, eq8\}, \{\_C1, \_C2, \_C3, \_C4, \_C5, \_C6, \_C7, \_C8\}) : assign(sol2) : x := 'x':$

Calculation of replacing rotational spring stiffness of plate

> $x := 0; M0 := M\_unit;$

$$x := 0$$

$$M0 := M\_unit \tag{1}$$

> $phi\_M\_unit := phi1;$

$$phi\_M\_unit := 0.0005952410186\, M\_unit \tag{2}$$

> $kr\_plate := \frac{M\_unit}{phi\_M\_unit};$

$$kr\_plate := 1679.991749 \tag{3}$$

>

M

Calculation of Replacing Rotational Spring Stiffness of Demountable Connection

On the next pages, a printout of the MAPLE worksheet that was used to calculate the replacing rotational spring stiffnesses of the demountable connection is added.

> restart;

Solving beam (on elastic foundation) ODEs for BCs

> ODE1 := EI·diff(w1(x), x$4) = 0; ODE2 := EI·diff(w2(x), x$4) + k_d·w2(x) = 0;

$$ODE1 := EI\left(\frac{d^4}{dx^4}\,w1(x)\right) = 0$$

$$ODE2 := EI\left(\frac{d^4}{dx^4}\,w2(x)\right) + k\_d\,w2(x) = 0 \quad (1)$$

> sol1 := dsolve(ODE1, w1(x)) : assign(sol1) : sol2 := dsolve(ODE2, w2(x)) : assign(sol2) :

> w1 := w1(x); w2 := w2(x);

$$w1 := \frac{1}{6}\,\_C1\,x^3 + \frac{1}{2}\,\_C2\,x^2 + \_C3\,x + \_C4$$

$$w2 := \_C1\,e^{\frac{-I(-k\_d\,EI^3)^{1/4}x}{EI}} + \_C2\,e^{\frac{I(-k\_d\,EI^3)^{1/4}x}{EI}} + \_C3\,e^{-\frac{(-k\_d\,EI^3)^{1/4}x}{EI}} + \_C4\,e^{\frac{(-k\_d\,EI^3)^{1/4}x}{EI}} \quad (2)$$

> w2 := e^(−lambda·x)·(_C5·cos(lambda·x) + _C6·sin(lambda·x));

$$w2 := e^{-\lambda x}\left(\_C5\cos(\lambda x) + \_C6\sin(\lambda x)\right) \quad (3)$$

> phi1 := −diff(w1, x) : kappa1 := diff(phi1, x) : M1 := EI·kappa1 : V1 := diff(M1, x) :

> phi2 := −diff(w2, x) : kappa2 := diff(phi2, x) : M2 := EI·kappa2 : V2 := diff(M2, x) :

> x := −a : eq1 := V1 =−F0 : eq2 := M1 = M0 + k_r_plate·phi1 :

> x := 0 : eq3 := w1 = w2 : eq4 := phi1 = phi2 : eq5 := V1 = V2 : eq6 := M1 = M2 :

> sol := solve({eq1, eq2, eq3, eq4, eq5, eq6}, {_C1, _C2, _C3, _C4, _C5, _C6}) : assign(sol) :
x :='x':

> w1 := w1; w2 := w2;

$$w1 := \frac{F0\,x^3}{6\,EI} + \frac{\left(F0\,a^2\lambda^2 k\_r\_plate + 2\,EI\,F0\,a\,\lambda^2 - 2\,EI\,M0\,\lambda^2 - F0\,k\_r\_plate\right)x^2}{4\,\lambda\,EI\,(a\,\lambda\,k\_r\_plate + EI\,\lambda + k\_r\_plate)} - \frac{\left(F0\,a^2\lambda\,k\_r\_plate + 2\,EI\,F0\,a\,\lambda - 2\,EI\,M0\,\lambda + F0\,a\,k\_r\_plate + EI\,F0\right)x}{2\,\lambda\,EI\,(a\,\lambda\,k\_r\_plate + EI\,\lambda + k\_r\_plate)} + \frac{1}{4\,EI\,\lambda^3\,(a\,\lambda\,k\_r\_plate + EI\,\lambda + k\_r\_plate)}\left(F0\,a^2\lambda^2 k\_r\_plate + 2\,EI\,F0\,a\,\lambda^2 - 2\,EI\,M0\,\lambda^2 + 2\,F0\,a\,\lambda\,k\_r\_plate + 2\,EI\,F0\,\lambda + F0\,k\_r\_plate\right)$$

$$w2 := e^{-\lambda x}\left(\frac{1}{4\,EI\,\lambda^3\,(a\,\lambda\,k\_r\_plate + EI\,\lambda + k\_r\_plate)}\left(\left(F0\,a^2\lambda^2 k\_r\_plate + 2\,EI\,F0\,a\,\lambda^2 - 2\,EI\,M0\,\lambda^2 + 2\,F0\,a\,\lambda\,k\_r\_plate + 2\,EI\,F0\,\lambda + F0\,k\_r\_plate\right)\cos(\lambda x)\right) - \frac{\left(F0\,a^2\lambda^2 k\_r\_plate + 2\,EI\,F0\,a\,\lambda^2 - 2\,EI\,M0\,\lambda^2 - F0\,k\_r\_plate\right)\sin(\lambda x)}{4\,EI\,\lambda^3\,(a\,\lambda\,k\_r\_plate + EI\,\lambda + k\_r\_plate)}\right) \quad (4)$$

Calculation of replacing rotational spring stiffness of dowel

> $x := -a;\ F0 := 0;\ M0 := M\_unit;$

$$x := -a$$
$$F0 := 0$$
$$M0 := M\_unit \tag{5}$$

> $phi1\_M\_unit := phi1;$

$$phi1\_M\_unit := -\frac{\lambda\, M\_unit\, a}{a\,\lambda\, k\_r\_plate + EI\,\lambda + k\_r\_plate} - \frac{M\_unit}{a\,\lambda\, k\_r\_plate + EI\,\lambda + k\_r\_plate} \tag{6}$$

> $k\_r := simplify\left(\mathrm{abs}\left(\frac{M\_unit}{phi1\_M\_unit}\right)\right);$

$$k\_r := \left|\frac{(a\, k\_r\_plate + EI)\,\lambda + k\_r\_plate}{a\,\lambda + 1}\right| \tag{7}$$

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Calculation of Maximum Dowel Deformation

On the next pages, a printout of a simplified version of the MAPLE worksheet that was used to calculate the maximum deformation of one specific combination of parameters of the demountable connection is added.

> restart;

Dowel parameters

> $E := 210000:$

> $c := 1:$

> $f\_ck := 30;$

$$f\_ck := 30 \tag{1}$$

> $a := 0.15;$

$$a := 0.15 \tag{2}$$

> $d := 80;$

$$d := 80 \tag{3}$$

Cross-sectional forces

> $F0 := 100;$

$$F0 := 100 \tag{4}$$

> $M0 := 10;$

$$M0 := 10 \tag{5}$$

Dowel variables

> $Iz := evalf\left(\frac{\mathrm{Pi}}{64} \cdot d^4\right); EI := E \cdot Iz \cdot 10^{-9};$

$$Iz := 2.010619299\ 10^6$$
$$EI := 422.2300528 \tag{6}$$

> $k\_d := evalf\left(\frac{127 \cdot c \cdot \mathrm{sqrt}(f\_ck)}{d^{\frac{2}{3}}} \cdot d \cdot 10^3\right);$

$$k\_d := 2.997282495\ 10^6 \tag{7}$$

> $\mathrm{lambda} := subs\left(kd = k\_d, \sqrt[4]{\frac{kd}{4 \cdot EI}}\right);$

$$\lambda := 6.490522144 \tag{8}$$

> $l\_b := subs\left(kd = k\_d, \frac{\pi}{\mathrm{lambda}}\right);$

$$l\_b := 0.4840277230 \tag{9}$$

Retrieving rotational spring stiffness of plate from other file

> $matching\_parameter1 := 1 : k\_r\_plate :=$
$DocumentTools[RunWorksheet]($"Replacing rotational spring stiffness - plate.mw",
$[matching\_parameter2 = matching\_parameter1]);$

$$k\_r\_plate := 1679.991029 \tag{10}$$

Solving beam (on elastic foundation) equations for BCs

> $ODE1 := EI \cdot diff(w1(x), x\$4) = 0; ODE2 := EI \cdot diff(w2(x), x\$4) + kd \cdot w2(x) = 0;$

$$ODE1 := 422.2300528 \frac{\mathrm{d}^4}{\mathrm{d}x^4} w1(x) = 0$$

$$ODE2 := 422.2300528 \frac{d^4}{dx^4} w2(x) + kd\, w2(x) = 0 \tag{11}$$

> $sol1 := dsolve(ODE1, w1(x)) : assign(sol1) : sol2 := dsolve(ODE2, w2(x)) : assign(sol2) :$

> $w1 := w1(x); w2 := w2(x);$

$$w1 := \frac{1}{6} \_C1\, x^3 + \frac{1}{2} \_C2\, x^2 + \_C3\, x + \_C4$$

$$w2 := \_C1\, e^{-\frac{5 I}{263893783} 10^{3/4} 263893783^{3/4} (-kd)^{1/4} x} + \_C2\, e^{\frac{5 I}{263893783} 10^{3/4} 263893783^{3/4} (-kd)^{1/4} x} + \_C3\, e^{-\frac{5\, 10^{3/4} 263893783^{3/4} (-kd)^{1/4} x}{263893783}} + \_C4\, e^{\frac{5\, 10^{3/4} 263893783^{3/4} (-kd)^{1/4} x}{263893783}} \tag{12}$$

> $w2 := e^{-\text{lambda}\cdot x} \cdot (\_C5 \cdot \cos(\text{lambda}\cdot x) + \_C6 \cdot \sin(\text{lambda} \cdot x));$

$$w2 := e^{-6.490522144\, x} (\_C5 \cos(6.490522144\, x) + \_C6 \sin(6.490522144\, x)) \tag{13}$$

> $phi1 := -diff(w1, x) : kappa1 := diff(phi1, x) : M1 := EI \cdot kappa1 : V1 := diff(M1, x) :$

> $phi2 := -diff(w2, x) : kappa2 := diff(phi2, x) : M2 := EI \cdot kappa2 : V2 := diff(M2, x) :$

> $x := -a : eq1 := V1 = -F0 : eq2 := M1 = M0 + k\_r\_plate \cdot phi1 :$

> $x := 0 : eq3 := w1 = w2 : eq4 := phi1 = phi2 : eq5 := V1 = V2 : eq6 := M1 = M2 :$

> $sol := solve(\{eq1, eq2, eq3, eq4, eq5, eq6\}, \{\_C1, \_C2, \_C3, \_C4, \_C5, \_C6\}) : assign(sol) :$
$x :='x':$

> $w1 := w1 : w2 := w2 :$

Calculation of replacing rotational spring stiffness of dowel

> $w1\_max := \max(\text{abs}(minimize(w1 \cdot 1000, x = -a..0)), \text{abs}(maximize(w1 \cdot 1000, x = -a..0)));$

$$w1\_max := 0.9570515127 \tag{14}$$

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Life-Cycle Costs of Traditional Alternative

On the next pages, a printout of the spreadsheet that was used to calculate the life-cycle costs of the traditional alternative is added. In the calculation, 5 life-cycles of 40 years each are assumed.

COSTS			
CONSTRUCTION			
Description	**Quantity**	**Unit price**	**Total**
Foundation			
Material/production and installation of steel pipe piles	48 pcs	€ 3.000,00	€ 144.000
Concrete works (in-situ)			
Abutments			
Formwork	278,4 m^2	€ 85,00	€ 23.664
Reinforcement (300 kg/m^3)	36540 kg	€ 1,35	€ 49.329
Concrete pouring	121,8 m^3	€ 120,00	€ 14.616
Finishing concrete pouring surface	10,8 m^2	€ 5,50	€ 59
Finishing formwork surface of concrete	278,4 m^2	€ 9,00	€ 2.506
After-treatment total concrete surface	289,2 m^2	€ 0,90	€ 260
Wing walls			
Formwork	57,6 m^2	€ 85,00	€ 4.896
Reinforcement (300 kg/m^3)	3120 kg	€ 1,35	€ 4.212
Concrete pouring	10,4 m^3	€ 160,00	€ 1.664
Finishing concrete pouring surface	8 m^2	€ 5,50	€ 44
Finishing formwork surface of concrete	57,6 m^2	€ 9,00	€ 518
After-treatment total concrete surface	65,6 m^2	€ 0,90	€ 59
Intermediate support			
Formwork	242,8 m^2	€ 85,00	€ 20.637
Reinforcement (300 kg/m^3)	30840 kg	€ 1,35	€ 41.634
Concrete pouring	102,8 m^3	€ 150,00	€ 15.420
Finishing concrete pouring surface	64,1 m^2	€ 5,50	€ 353
Finishing formwork surface of concrete	242,8 m^2	€ 9,00	€ 2.185
After-treatment total concrete surface	306,9 m^2	€ 0,90	€ 276
Concrete works (prefab)			
Transition slabs			
Material/production and installation of transition slabs	22 pcs	€ 1.500,00	€ 33.000
Deck			
Material/production and installation of box beam deck	2 pcs	€ 300.000,00	€ 600.000
Transition joints			
Material/production and installation of transition joints	36 m	€ 1.000,00	€ 36.000 +
CONSTRUCTION COSTS			**€ 995.332**
EXECUTION COSTS (20% OF CONSTRUCTION COSTS)	**20 %**	**€ 995.332**	**€ 199.066 +**
TOTAL CONSTRUCTION COSTS			**€ 1.194.399**

MAINTENANCE			
Description	**Quantity**	**Unit price**	**Total**
Small maintenance			
Every 2 years (estimated)	20 x	€ 1.000,00	€ 20.000
Large maintenance			
Every 25 years (estimated)	2 x	€ 25.000,00	€ 40.000 +
TOTAL MAINTENANCE COSTS			**€ 60.000**

DECONSTRUCTION			
Description	**Quantity**	**Unit price**	**Total**
Demolition of viaduct			
Crushing/sawing and transportation	1 x	€ 25.000,00	€ 25.000
Processing/landfilling			
Processing/landfilling by certified company	1 x	€ 25.000,00	€ 25.000 +
TOTAL DECONSTRUCTION COSTS			**€ 50.000**

TOTAL COSTS	**€ 1.304.399**

RESIDUAL VALUE			
MATERIALS RECOVERY			
Description	**Quantity**	**Unit price**	**Total**
Crushed concrete for road foundation			
Substructure			
Abutments	109,6 m^3	€ 12,50	€ 1.370
Wing walls	9,4 m^3	€ 12,50	€ 117
Intermediate support	92,5 m^3	€ 12,50	€ 1.157
Superstructure			
Transition slabs	79,2 m^3	€ 12,50	€ 990
Deck	271,6 m^3	€ 12,50	€ 3.395
Steel recovery			
Recycling of reinforcement	70500 kg	€ 0,10	€ 7.050 +
TOTAL MATERIALS RECOVERY VALUE			**€ 14.079**

TOTAL RESIDUAL VALUE	**€ 14.079**

NET COSTS TRADITIONAL ALTERNATIVE	**€ 1.290.319**

TOTAL COSTS PER LIFE-CYCLE						
CONSTRUCTION						
Life-cycle 1	Life-cycle 2	Life-cycle 3	Life-cycle 4	Life-cycle 5	Total	
€ 144.000	€ 144.000	€ 144.000	€ 144.000	€ 144.000	€ 720.000	
€ 23.664	€ 23.664	€ 23.664	€ 23.664	€ 23.664	€ 118.320	
€ 49.329	€ 49.329	€ 49.329	€ 49.329	€ 49.329	€ 246.645	
€ 14.616	€ 14.616	€ 14.616	€ 14.616	€ 14.616	€ 73.080	
€ 59	€ 59	€ 59	€ 59	€ 59	€ 297	
€ 2.506	€ 2.506	€ 2.506	€ 2.506	€ 2.506	€ 12.528	
€ 260	€ 260	€ 260	€ 260	€ 260	€ 1.301	
€ 4.896	€ 4.896	€ 4.896	€ 4.896	€ 4.896	€ 24.480	
€ 4.212	€ 4.212	€ 4.212	€ 4.212	€ 4.212	€ 21.060	
€ 1.664	€ 1.664	€ 1.664	€ 1.664	€ 1.664	€ 8.320	
€ 44	€ 44	€ 44	€ 44	€ 44	€ 220	
€ 518	€ 518	€ 518	€ 518	€ 518	€ 2.592	
€ 59	€ 59	€ 59	€ 59	€ 59	€ 295	
€ 20.637	€ 20.637	€ 20.637	€ 20.637	€ 20.637	€ 103.183	
€ 41.634	€ 41.634	€ 41.634	€ 41.634	€ 41.634	€ 208.170	
€ 15.420	€ 15.420	€ 15.420	€ 15.420	€ 15.420	€ 77.100	
€ 353	€ 353	€ 353	€ 353	€ 353	€ 1.763	
€ 2.185	€ 2.185	€ 2.185	€ 2.185	€ 2.185	€ 10.925	
€ 276	€ 276	€ 276	€ 276	€ 276	€ 1.381	
€ 33.000	€ 33.000	€ 33.000	€ 33.000	€ 33.000	€ 165.000	
€ 600.000	€ 600.000	€ 600.000	€ 600.000	€ 600.000	€ 3.000.000	
€ 36.000	€ 36.000	€ 36.000	€ 36.000	€ 36.000	€ 180.000	+
€ 995.332	€ 995.332	€ 995.332	€ 995.332	€ 995.332	**€ 4.976.661**	
€ 199.066	€ 199.066	€ 199.066	€ 199.066	€ 199.066	**€ 995.332**	**+**
€ 1.194.399	€ 1.194.399	€ 1.194.399	€ 1.194.399	€ 1.194.399	**€ 5.971.993**	

MAINTENANCE						
Life-cycle 1	Life-cycle 2	Life-cycle 3	Life-cycle 4	Life-cycle 5	Total	
€ 20.000	€ 20.000	€ 20.000	€ 20.000	€ 20.000	€ 100.000	
€ 40.000	€ 40.000	€ 40.000	€ 40.000	€ 40.000	€ 200.000	+
€ 60.000	€ 60.000	€ 60.000	€ 60.000	€ 60.000	**€ 300.000**	

DECONSTRUCTION						
Life-cycle 1	Life-cycle 2	Life-cycle 3	Life-cycle 4	Life-cycle 5	Total	
€ 25.000	€ 25.000	€ 25.000	€ 25.000	€ 25.000	€ 125.000	
€ 25.000	€ 25.000	€ 25.000	€ 25.000	€ 25.000	€ 125.000	+
€ 50.000	€ 50.000	€ 50.000	€ 50.000	€ 50.000	**€ 250.000**	

€ 1.304.399	€ 1.304.399	€ 1.304.399	€ 1.304.399	€ 1.304.399	**€ 6.521.993**

TOTAL RESIDUAL VALUE PER LIFE-CYCLE						
MATERIALS RECOVERY						
Life-cycle 1	Life-cycle 2	Life-cycle 3	Life-cycle 4	Life-cycle 5	Total	
€ 1.370	€ 1.370	€ 1.370	€ 1.370	€ 1.370	€ 6.851	
€ 117	€ 117	€ 117	€ 117	€ 117	€ 585	
€ 1.157	€ 1.157	€ 1.157	€ 1.157	€ 1.157	€ 5.783	
€ 990	€ 990	€ 990	€ 990	€ 990	€ 4.950	
€ 3.395	€ 3.395	€ 3.395	€ 3.395	€ 3.395	€ 16.977	
€ 7.050	€ 7.050	€ 7.050	€ 7.050	€ 7.050	€ 35.250	+
€ 14.079	€ 14.079	€ 14.079	€ 14.079	€ 14.079	**€ 70.396**	

€ 14.079	€ 14.079	€ 14.079	€ 14.079	€ 14.079	**€ 70.396**

€ 1.290.319	€ 1.290.319	€ 1.290.319	€ 1.290.319	€ 1.290.319	**€ 6.451.597**

P

Life-Cycle Costs of Circular Alternative

On the next pages, a printout of the spreadsheet that was used to calculate the life-cycle costs of the circular alternative is added. In the calculation, 5 life-cycles of 40 years each are assumed.

COSTS			
CONSTRUCTION			
Description	**Quantity**	**Unit price**	**Total**
Foundation			
Material/production of steel pipe piles	48 pcs	€ 3.825,00	€ 183.600
Installation of steel pipe piles	48 pcs	€ 450,00	€ 21.600
Concrete works (prefab)			
Abutments			
Material/production of abutments	2 pcs	€ 57.651,85	€ 115.304
Installation of abutments	2 pcs	€ 6.782,57	€ 13.565
Wing walls			
Material/production of wing walls	4 pcs	€ 3.631,66	€ 14.527
Installation of wing walls	4 pcs	€ 427,25	€ 1.709
Intermediate support			
Material/production of intermediate support	1 pcs	€ 102.643,24	€ 102.643
Installation of intermediate support	1 pcs	€ 12.075,68	€ 12.076
Transition slabs			
Material/production of transition slabs	22 pcs	€ 1.912,50	€ 42.075
Installation of transition slabs	22 pcs	€ 225,00	€ 4.950
Deck			
Material/production of box beam deck	2 pcs	€ 382.500,00	€ 765.000
Installation of box beam deck	2 pcs	€ 45.000,00	€ 90.000
Transition joints			
Material/production of transition joints	36 m	€ 1.275,00	€ 45.900
Installation of transition joints	36 m	€ 150,00	€ 5.400
CONSTRUCTION COSTS			**€ 1.418.348**
EXECUTION COSTS (20% OF CONSTRUCTION COSTS)	**20 %**	**€ 1.418.348**	**€ 283.670 +**
TOTAL CONSTRUCTION COSTS			**€ 1.702.018**

MAINTENANCE			
Description	**Quantity**	**Unit price**	**Total**
Small maintenance			
Every 2 years (estimated)	20 x	€ 1.500,00	€ 30.000
Large maintenance			
Every 25 years (estimated)	2 x	€ 37.500,00	€ 60.000
End of life-cylce mainenance			
Reusability assessment + maintenance/repair	1 x	€ 18.750,00	€ 18.750 +
TOTAL MAINTENANCE COSTS			**€ 108.750**

DECONSTRUCTION			
Description	**Quantity**	**Unit price**	**Total**
Deconstruction of viaduct			
Disassembly and transport of elements and components	1 x	€ 37.500,00	€ 37.500 +
TOTAL DECONSTRUCTION COSTS			**€ 37.500**

TOTAL COSTS	**€ 1.848.268**

RESIDUAL VALUE			
MATERIALS SAVING			
Description	Quantity	Unit price	Total
N / A			€ - +
TOTAL MATERIALS SAVING VALUE			€ -

TOTAL RESIDUAL VALUE	€ -

NET COSTS CIRCULAR ALTERNATIVE	€ 1.848.268

TOTAL COSTS PER LIFE-CYCLE

CONSTRUCTION

Life-cycle 1	Life-cycle 2	Life-cycle 3	Life-cycle 4	Life-cycle 5	Total
€ 183.600	N / A	N / A	N / A	N / A	€ 183.600
€ 21.600	€ 21.600	€ 21.600	€ 21.600	€ 21.600	€ 108.000
€ 115.304	N / A	N / A	N / A	N / A	€ 115.304
€ 13.565	€ 13.565	€ 13.565	€ 13.565	€ 13.565	€ 67.826
€ 14.527	N / A	N / A	N / A	N / A	€ 14.527
€ 1.709	€ 1.709	€ 1.709	€ 1.709	€ 1.709	€ 8.545
€ 102.643	N / A	N / A	N / A	N / A	€ 102.643
€ 12.076	€ 12.076	€ 12.076	€ 12.076	€ 12.076	€ 60.378
€ 42.075	N / A	N / A	N / A	N / A	€ 42.075
€ 4.950	€ 4.950	€ 4.950	€ 4.950	€ 4.950	€ 24.750
€ 765.000	N / A	N / A	N / A	N / A	€ 765.000
€ 90.000	€ 90.000	€ 90.000	€ 90.000	€ 90.000	€ 450.000
€ 45.900	N / A	N / A	N / A	N / A	€ 45.900
€ 5.400	€ 5.400	€ 5.400	€ 5.400	€ 5.400	€ 27.000
€ 1.418.348	€ 149.300	€ 149.300	€ 149.300	€ 149.300	**€ 2.015.548**
€ 283.670	€ 29.860	€ 29.860	€ 29.860	€ 29.860	**€ 403.110** +
€ 1.702.018	€ 179.160	€ 179.160	€ 179.160	€ 179.160	**€ 2.418.657**

MAINTENANCE

Life-cycle 1	Life-cycle 2	Life-cycle 3	Life-cycle 4	Life-cycle 5	Total
€ 30.000	€ 30.000	€ 30.000	€ 30.000	€ 30.000	€ 150.000
€ 60.000	€ 60.000	€ 60.000	€ 60.000	€ 60.000	€ 300.000
€ 18.750	€ 18.750	€ 18.750	€ 18.750	€ 18.750	€ 93.750 +
€ 108.750	€ 108.750	€ 108.750	€ 108.750	€ 108.750	**€ 543.750**

DECONSTRUCTION

Life-cycle 1	Life-cycle 2	Life-cycle 3	Life-cycle 4	Life-cycle 5	Total
€ 37.500	€ 37.500	€ 37.500	€ 37.500	€ 37.500	€ 187.500 +
€ 37.500	€ 37.500	€ 37.500	€ 37.500	€ 37.500	**€ 187.500**

€ 1.848.268	€ 325.410	€ 325.410	€ 325.410	€ 325.410	**€ 3.149.907**

TOTAL RESIDUAL VALUE PER LIFE-CYCLE					
MATERIALS SAVING					
Life-cycle 1	Life-cycle 2	Life-cycle 3	Life-cycle 4	Life-cycle 5	Total
€ -	€ -	€ -	€ -	€ -	€ - +
€ -	€ -	€ -	€ -	€ -	**€ -**
€ -	€ -	€ -	€ -	€ -	**€ -**
€ 1.848.268	€ 325.410	€ 325.410	€ 325.410	€ 325.410	**€ 3.149.907**