Doctoral Thesis

Areas of application of thin-walled precast concrete elements: From integral bridges with short and medium spans to multi-span bridges

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Dissertation

Anwendungsgebiete für dünnwandige Betonhalbfertigteile: Von integralen Brücken mit kurzen und mittleren Spannweiten zu Mehrfeldbrücken

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Kurzfassung

Eine Kombination aus rasant wachsender Infrastruktur und dem Streben nach innovativen Bauverfahren, die besonders wirtschaftlich auf Grund von ihrer Kosteneffizienz, Zeiteinsparung und Dauerhaftigkeit sind, können als Motivation für die Forschung, die in dieser Dissertation präsentiert wird, angesehen werden. Die grundliegende Idee auf der die vorliegende Forschung basiert ist die Kombination von regulären dünnwandigen Betonhalbfertigteilen und Ortbeton für eine schnelle und ökonomische Errichtung von Brücken. Durch die Umfunktionierung von den kostengünstigen Betonhalbfertigteilen zu Fertigteilträgern, die als verlorene Schalung für den nachträglich ein- beziehungsweise aufgebrachten Ortbeton dienen, entsteht ein monolithisches Tragwerk, wobei der Vorfertigungsgrad sich äußerst positiv auf die Bauzeit auswirkt. Zusätzlich gewährt die hohe Qualität, die durch die Werkproduktion gewährleistet wird, eine erhöhte Dauerhaftigkeit der Struktur.

In dieser Arbeit werden die Anwendungsgebiete für dünnwandige Betonhalbfertigteile, basierend auf den Forschungen von Gmainer (2011) und Wimmer (2016), für integrale Brücken mit kurzen und mittleren Spannweiten bis zu Mehrfeldbrücken genauer untersucht. Im Laufe von drei Forschungsjahren wurde eine gründliche Literaturstudie in Verbindung mit einer internationalen Umfrage fokussierend auf die Dauerhaftigkeit und die Nachhaltigkeit von Fertigteilbrücken durchgeführt. Zusätzlich wurden Mörtelproben standardisierten und nicht standardisierten Versuchen unterzogen, die Fugen zwischen den dünnwandigen Halbfertigteilen neugestaltet, Dauerschwingversuche durchgeführt, eine Beurteilung der Eignung von Fertigteilträgern aus Betonhalbfertigteilen für den integralen Brückenbau geprüft, Kastenquerschnitte für Brücken mit größeren Spanweiten entwickelt, produziert und getestet und für einen Designwettbewerb adaptiert.

Verbesserungsmöglichkeiten für den Entwurf, die Berechnung und die Produktion von Fertigteilträgern aus Betonhalbfertigteilen, eine Beurteilung des Anwendungsbereiches für jene Fertigteile im integralen Brückenbau, die Umsetzbarkeit von den entwickelten Kastenquerschnitten sowie alle Erkenntnisse die aus den Groß- und Kleinversuchen gewonnen werden konnten werden in der folgenden Arbeit präsentiert.

Abstract

The rapid growth in the worlds infrastructure in combination with the high expectations in innovative construction solutions, that should not only be cost- but also time-efficient and durable, have been a great motivation for the research presented in this thesis, with the main idea being the implementation of standard thin-walled precast concrete elements in combination with cast in-situ concrete for the construction of bridge girders. By repurposing standard and cost-efficient prefabricated elements, which would serve as permanent formwork for the subsequently added in-situ concrete, creating monolithic structures, considerable reduction in the on-site construction work would be achieved. Additionally the high production quality of the elements would ensure the durability of the structure.

This thesis describes the continuation of the research of Gmainer (2011) and Wimmer (2016) by looking into the areas of application of these thin-walled concrete elements starting from integral bridges with short and medium spans to multi-span bridges. In the course of three years of research a thorough literature study in combination with an international survey on the experiences with prefabrication in bridge construction with a focus on durability and sustainability, standardized and non-standardized tests of mortars needed for the construction of the girders out of thin-walled precast elements, a redesign of the joints between individual precast elements, large-scale fatigue testing, the assessment of the suitability of thin-walled precast bridge girders in regard to integral bridge construction, the design, construction and testing of large box-cross-sectioned prototypes made out of thin-walled precast elements as well as the participation in a design competition had been carried out.

Considerations on possible improvements in design, the affirmation of the feasibility of building integral and multi-span bridges with girders out of thin-walled precast elements as well as all insights gained from the positive and negative outcomes of the conducted large-scale tests are described in the following thesis.

Acknowledgement

As Joseph Joubert said: "Children need models rather than critics."

It seems that this quote has accompanied me throughout life, and I am truly grateful for everyone that has supported me and inspired me to always reach for the stars.

First of all I would like to thank my parents, who, despite all hardships in their lives, have always encouraged my sister and me to follow our dreams and have stood behind us no matter what. Thanks to them I have never seen any obstacle to be to difficult to hurdle.

With over half our lives spent together, a special thank you goes out to my husband, who has never shown any doubt in me. I thank him for every single encouraging smile. In the same breath I would like to thank my sister for always being there for me, always.

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Last but not least I would like to thank Prof. Andreas Kolbitsch and Prof. Jan Biliszczuk for their interest in this thesis.

This thesis is dedicated to my children. I will always try to be model rather than a critic, remind me if I forget.

Contents

1 Introduction

Bridges are, with a great variety of possible construction methods, a core element in modern road construction. As is the case for most construction methods, experts are looking for innovative, economical and rapid methods to optimize construction processes, thus saving costs and time. At the same time huge emphasis is placed on the durability and perfect execution. A possibility to meet all the demands stated is by implementing innovative prefabrication, that can be adjusted to any kind of boundary conditions, in bridge design, no matter if its for integral bridges with short and medium spans or multi-span bridges with spans of 100 m and over.

1.1 Motivation

With the rapid growth in the worlds infrastructure, the construction of roads and bridges needed is expected to happen quickly, economically and with as little impact on the existing infrastructure as possible. If prefabricated elements are used, large parts of the structures can be produced off-site, reducing on-site construction time. Furthermore the elements that are produced are manufactured under regulated and monitored conditions resulting in high quality products with increased accuracy and quality of finish.

The search for the most appropriate cross-section of precast bridge elements is according to the *fib* Bulletin No. 29 (2004) of major importance in prefabrication. With this in mind, designs have been focusing strongly on the idealization of the dimensions of the concrete elements by reducing web thickness as well as the dimensions of the upper and lower flanges or even rethinking the necessity of end blocks.

The Institute of Structural Engineering of the TU Wien has focused on the development of lightweight girders constructed out of thin-walled precast elements that could easily become an attractive alternative option to steel when it comes to the construction of bridges without formwork. The idea was to combine the attributes of precast construction, by using lattice-girder floor slabs and double wall elements, which have been successfully used in building and industrial construction for decades, with all positive aspects of cast in-situ concrete. With this goal, trough-shaped girders were developed, which would serve as permanent formwork for subsequently added in-situ concrete therefore considerably reducing the use of formwork and falsework on the construction site and creating monolithic maintenance-friendly constructions. According to Wimmer (2016) the scope of application includes bridge structures with field-spans of up to 50 m or as additionally described by Gmainer (2011) the implementation in structures erected by the balanced lift method, a bridge construction method also developed at the Institute of Structural Engineering of the TU Wien.

Based on the research presented by Wimmer (2016) and Gmainer (2011) the further development of the area of application for lightweight thin-walled precast concrete elements was pursued. The possibility of creating larger cross-sections than the ones presented and developing detail solutions for existing problems of the current designs were set as goals.

1.2 Approach

In order to investigate further possible applications for thin-walled precast elements in bridge construction, whether for integral or multi-span constructions, various approaches were chosen.

The process of design development would be accompanied by a thorough literature study as well as an international survey on the experiences with prefabrication in bridge construction with a focus on durability and sustainability.

Based on the research presented by Wimmer (2016) and Gmainer (2011), testing the grout material used for the assembly of the girders made out of thin-walled precast elements was seen as necessary. For this purpose several mortars were tested for their material parameters. In addition the joints between the individual thin-walled precast elements were redesigned and tested.

With fatigue being a very prominent issue when it comes to bridge construction, experimental tests of fatigue behavior of thin-walled precast elements were conducted on large-scale specimens.

The assessment of the suitability of thin-walled precast bridge girders regarding the implantation in integral bridges was conducted by redesigning existing motorway overpasses and programming a numerical parameter study.

In order to evaluate the possibility of using thin-walled precast elements for the construction of box-cross-sectioned girders for bridges with large spans a prototype of said girder was manufactured and tested providing insight into its structural stability.

With the feasibility of the commercial production of the lightweight cross-sections proven, the innovative cross-section was used in a design of the engineering firm FCP - Fritsch, Chiari & Partner ZT GmbH entered in a design competition for the construction of two bypass bridges across the Danube river in Linz.

2 State of the art - Prefabrication in bridge construction

This chapter will give a short overview of the history and the developments of prefabrication in bridge construction, presenting the various classification of this building method with all its perks, advantages and disadvantages, before going into detail when it comes to the utilization of thin-walled precast concrete elements in this construction field. The manufacturing process of thin-walled precast concrete bridge girders will be explained in detail followed by a summary of the already carried out tests on large-scale specimens in the past. In addition to the literature study, reports based on the experiences with prefabrication in bridge construction concentrating on the durability aspect in combination with the outcome of a conducted international survey, consisting of eight questions, will be presented.

2.1 Prefabricated elements in bridge construction

Prefabrication in bridge construction implies that structural components of the bridge are built off-site or near-site, reducing not only the on-site construction time relative to conventional building methods but also the traffic impact as well as weather-related time delays. With the elements being produced under regulated and monitored conditions a high quality product with an increased production accuracy and quality of finish can be expected. As summarized by the FHWA (2005) the main aspects promoting prefabrication in bridge construction is the minimization of traffic disruption, the improvement of work zone safety, the minimization of environmental impact, the improvement of constructibility, the increase in quality and the lower life-cycle costs when executed properly.

Despite all the advantages, contractors, as explained in detail in the *fib* Bulletin No. 29 (2004), often shy away from the application of prefabricated structural components in bridge construction due to their low opinion on the durability of the intersections and joints, the perceived opinion that precast bridges are monotonous and ugly or other specific technical issues, such as the shear transfer at the interface between cast in-situ and prefabricated elements. The matter of aesthetics and increased number of joints has been addressed by Kargel (2005), who explains that by combining prefabrication with cast in-situ concrete in a favourable manor, as shown by presenting three existing bridge designs, extraordinary and aesthetically pleasing constructions can be created.

2.1.1 Historical overview and development

The first application of prefabricated concrete elements in bridge construction for bridges with short spans dates back to the 1930s for most developed countries. With the considerable increase in road traffic, the construction of new highways and the urge to build economically, building with precast elements gained popularity and was further developed in the 1950s and 1960s. Over the next decades an augmentation in the number of prefabricated bridges with constantly increasing spans could be observed. The transport and mounting options also improved significantly over time, allowing the production, transport over longer distances and installation of prefabricated beams with full span lengths and weights of several hundred tons.

The idea of using prefabricated elements in bridge construction gained solid ground in some countries like Belgium, Italy, the Netherlands, Spain, Poland, the United Kingdom, the USA as well as Canada, becoming widely used and accepted as a classical solution and reaching a share market of up to 50% of the bridges built in the second half of the 20th century. These countries developed an extensive range of viable technical solutions for bridges with short and long spans.

In other countries, like France and Germany, precast bridges were accepted as a good, but often not good enough variant solutions to cast in-situ bridges, with their construction depending on the economical activity of the countries, with their market share lying between 5 % and 20 %, or were not accepted at all, as was the case in the Scandinavian countries or Austria, where very little precast concrete bridges can be found. The disinterest of the latter can be led back to a late interest in the production process itself, resulting in a lack of knowledge, or just simple prejudice on a both technical and aesthetic level.

In the course of the past seventy years continued progress was made in the development of precast bridge constructions, not only concerning the span lengths and transportation and mounting possibilities. The first precast elements were meant for small bridges using match cast systems or systems composed of small inverted T-beams placed against each other and filled with in-situ concrete. These systems were replaced by girder bridges with precast beams and cast in-situ deck slabs with span lengths of around 35 m. Nowadays, spans of 50 m to 60 m are feasible.

With the rise of prestressing and post-tensioning further developments and larger span lengths were possible. Furthermore the complete precasting of the entire bridge deck was aimed for, resulting in systems like box beam bridges, composite bridges, trough bridges and segmental bridges. Furthermore designs of continuous precast bridges with precast, prestressed concrete girders started to get more popular as shown by Freyermuth (1969).

The reconsiderations of bridge design starting around 1975 focused more on the lifetime performance, the improvement of aesthetics and traffic comfort resulting in curved precast bridges, precast bridges with partial or full continuity and even precast integral bridges. Continuous box beam bridges with spans of 90 m, systems with external post tensioning, bridges prestressed for transportation and made continuous by post-tensioning after erection, precast cables stayed bridges reaching spans of up to 260 m and massive construction projects using prefabricated bridge elements as for example in the United Arab Emirates, and integral constructions using precast elements have enriched the scope of prefabricated bridge construction in the past decades.

Design guidelines and the standardization of bridge systems, specifically precast bridge systems, can be found in most countries since the 1970s. The large success of prefabricated bridge construction in the United States can be led back to earliest published guideline of this sort dating back to 1956. Most of the documents consist of the design procedures, detailed calculation examples, standardized beam sections as well as detail solutions. Some, as for example the Belgium standardization, even state production and erection characteristics, quality control procedures, arrangement and deviation of the prestressing tendons as well as required mild steel reinforcement (*fib* Bulletin No. 29 (2004)).

2.1.2 Classification of precast bridges

The *fib* Bulletin No. 29 (2004) classifies the currently constructed precast bridges into nine types, with multiple examples and detailed descriptions of each type included, as following:

- Solid deck bridges
  (span range between 5 m and 20 m)
- Girder bridges
  (span range between 15 m and 55 m)
- Box-beam bridges
  (span range between 20 m and 50 m)
- Mono-box bridges
  (span range between 20 m and 90 m)
- Curved box beam bridges
- Trough bridges
- Segmental bridges
- Cable stayed bridges
- Special precast bridges
  - arch bridges
  - bridges with variable depth
  - canal bridges
  - pedestrian bridges
  - bridges for special industrial purposes

Hue (2005), on the contrary, classifies precast bridge types by the frequency with which precast elements are used in the individual parts of the bridges, subdividing these in specific cross-sections or load-bearing behaviors, with multiple examples and detailed descriptions of each type, as following:

- Decks built with beams
  - Decks built with I-beams
  - Decks built with U-beams
  - Decks built with mono-beams
    (single U-beams)
  - Decks built with U-beams with a longitudinal joint forming unicellular or multicellular box-girders
  - Decks built with inverted T-beams
- Deck slabs on beams
  - Slabs between beams used as lost formwork
  - Partially precast slabs between beams
  - Fully cast slabs
  - Deck slabs on steel beams
- Special decks
- Decks of segments
  - Segments with full or partial transversal section
  - Segments with full transversal section joined by a deck slab
  - Segments joined by top and bottom slabs forming unicellular or multicellular box-girders
- Complete decks
- Abutments (further subdivision mentioned but not seen as relevant)
- Piers (further subdivision mentioned but not seen as relevant)
- Foundations (further subdivision mentioned but not seen as relevant)
- Auxiliary elements (further subdivision mentioned but not seen as relevant)

The differentiation made by Biliszczuk and Onysyk (2016) divides the construction of precast bridges on the one hand into bridges built using prefabricated beams and bridges built using prefabricated segments, and on the other hand according to their mounting process, distinguishing between the installation using cranes or similar devices, the installation using moving installation carriages and the construction using incremental launching (solely possible using segments). Furthermore Biliszczuk and Onysyk (2016) discuss special precast bridge constructions, similarly to the classifications made in the *fib* Bulletin No. 29 (2004), presenting existing bridge constructions of each described type as well as examples for prefabricated substructures (with a similar subdivision as Hue (2005)).

In addition to the classification of bridges constructed using prefabricated elements by the prefabrication type or installation procedure, a distinction can by made by analyzing their structural systems as shown in the *fib* Bulletin No. 29 (2004). One distinguishes between simply supported bridges, simply supported bridges with continuous slabs, continuous bridges, as well as semi-integral and integral bridges.

With almost one hundred years of history, there are surely more variations for the classification of precast bridges or bridges made with precast elements. One last distinction, described by Vennemann (2005), should be mentioned nonetheless. According to the latter, one must differentiate between precast bridge constructions, which implies that a standardized bridge design can be implemented wherever it is seen as appropriate, and building bridges with precast elements, which are manufactured especially for one bridge design meeting all requirements for a specific project. Precast bridge construction can therefore be compared to a standard prefabricated house whereas the design options for bridges constructed using precast elements are almost unlimited, setting barely any boundaries as would be the case when designing any other extraordinary engineering structure.

2.1.3 International comparison of prefabrication in bridge construction

The development and the popularity of precast bridges and bridges constructed using precast elements vary greatly from continent to continent or even country to country. Gaining solid ground in countries like Belgium, Italy, the Netherlands, Spain, Poland, the United Kingdom, the USA, Canada as well as China, the application of prefabricated elements reached a share market of up to 50% of the bridges built in the second half of the 20th century. These countries developed an extensive range of viable technical solutions for bridges with short and long spans.

As mentioned in the previous section, despite being considered a good variant solution to cast in-situ bridges, in some countries like France and Germany, the market share of precast bridges only lies between 5 % and 20 %. This can be traced back to the economic activity of the country, with precast bridges not being built in low economical periods. In the past couple of years the German attitude towards prefabricated solutions has altered, after realizing, for example, that the loss of production time in the state of Bavaria caused by road construction was estimated to cost 1 billion Euros a year. Nonetheless, the result of the very strict design criteria set by the German government resulted in only 15 % of the modern bridges using prefabricated main beams (FHWA (2005)).

With the precasters themselves responsible for the development of the precast bridge systems, the used bridge profiles vary from region to region. The same principle applies to the favored construction method and design methodologies used in every country when using prefabricated bridge elements. The main purpose of the FHWA (2005), a technical report of a scanning study, was the identification of international uses of prefabricated bridge elements differing from those used in the United States in order to find viable and innovative solutions for their aging highway bridge infrastructure that should be renewed.

In a country like Japan a rapid bridge construction is a high priority due to the high project and labor costs, the scarcity of skilled labor as well as the need for faster erection, improved quality and better work zone safety for contractors and the public. The reduction in skilled labor due to retirements and rising labor costs have forced Japan to reconsider cast in-situ construction, which was very popular up to the turn of the century, and focus on factory-produced elements. Furthermore, the restriction in hauling weight, which is limited to 30 t, has motivated Japanese engineers to develop ways to lower the weight of the so desperately needed prefabricated elements (FHWA (2005)).

2.1.4 Problematics of using prefabricated elements in bridge construction

As is the case with every construction method, the application of prefabricated elements in bridge construction can only be seen as suitable and economically viable if all boundary conditions supporting this kind of construction permit it. Nevertheless, this construction method has accumulated a lot of prejudice over the years, due to sloppy execution and careless design. As mentioned before and explained in detail in the *fib* Bulletin No. 29 (2004), contractors often shy away from the application of prefabricated structural components in bridge construction due to their low opinion on the durability of the intersections and joints, the perceived opinion that precast bridges are monotonous and ugly, or other specific technical issues, such as the shear transfer at the interface between cast in-situ and prefabricated elements.

In addition to the listed preconceptions, limitations in the application of prefabricated elements in bridge construction can be seen as actual disadvantages of this building method. Even though cranes, launching cradles and assembly methods have developed immensely over the years and the increased length and loading capacity of transport vehicles has allowed the prefabrication sector to enhance and refine its products, certain dimensional limitations will always be present.

As mentioned by Hue (2005), the joints between the individual elements or between the elements and the prior built in-situ bridge parts can be very complicated especially in hyperstatic structures. By simplifying the structural system to, for example, simply supported bridges, an uptake of the deformation due to creep and shrinkage and the accommodation of the differential settlement of the deck supports is possible due to the positioning of the elements on individual bearings. This seems to be a logical solution to the complicated joint details, entails however problems when it comes to traffic comfort and durability of the bearings and expansion joints, which often need inspection or replacement caused by the presence of de-icing salt (*fib* Bulletin No. 29 (2004)).

2.1.5 Reports on durability of precast bridges

The durability of a concrete structure is defined by the characteristics of the individual components, which are exposed to the long term effects of the environment during their service life. The minimum cement quantity in the concrete mix, a low water/cement ratio and the compaction and the strength of the hardened concrete all factor into the actual durability of the structure. In order to prevent corrosion of the reinforcement a sufficient concrete cover must be existent. Inadequate durability is reflected, inter alia, by deterioration of the aesthetics, spalling or cracks in the concrete or corrosion of the reinforcement and can sometimes even be the cause of structural failure. The corrosion of the reinforcement, triggered either by substances embedded in the structure or brought into the structure by the environment, can be prevented by a proper concrete cover. Damages due to weather effects can only occur if the vulnerable parts of the structure have somehow already been exposed, either by external forces, too high loads or the exceed of service life (Stark et al. (2013)).

A systematization and typology of damages in concrete bridges was presented by Bień (2010). He differentiates between deflection and deformation of structural compounds, deterioration - both of the chemical as well as the physical properties - of the material, abrasion, damage in the continuity - cracks or fractures in the material, exterior contamination and damages caused by subsidence and secondary moments.

In the case of bridges, which are designed for a minimum lifetime of 100 years, the main exposure consists of climatic conditions as well as the influence of de-icing salts. When it comes to the durability of precast bridges the experiences, according to *fib* Bulletin No. 29 (2004), can be seen as generally positive, due to the high concrete strength, low water/cement ratio and the quality of execution of the elements, which is influenced by the indoor manufacture and high control levels. Even though joints, especially expansion joints, are not only existent in precast bridges, they are still considered as one of the major concerns when it come to this type of bridge construction. The damages to the bridge structure caused by defective or poorly executed joints can accumulate and pose great risks to the stability of the entire structure (Bień (2010)).

Looking back at almost 100 years of precast bridge construction, conclusions on the durability of such construction can easily be assessed. Countries like Norway, Germany, Belgium and the Netherlands have repeatedly stated that the maintenance and repair costs have been much lower than those of in-situ bridges, especially for simply supported bridges. On the other hand, objections and drawback concerns especially when it comes to the cost of inspection have been stated by other countries (*fib* Bulletin No. 29 (2004)). In the following chapters international reports on precast bridge construction and durability problems in precast bridges will be presented.

2.1.5.1 Austria

With Austria being one of the countries where precast concrete bridge construction never really succeeded in taking root, with only about 2% of the countries bridges being precast or built using precast elements, not many reports on the durability nor on the experiences concerning such construction exist.

Destructive large scale experiments on an existing post-tensioned segmental bridge were conducted by the TU Wien in 2005 (Eichinger et al. (2005)). The bridge was erected in 1975 in Vienna, with a span of 44 m and consisted of a total of 18 segments joined together by epoxy resin and post-tensioned. The aim of the investigations was to obtain valuable insight on the bearing capacity and durability of the 30 year old structure. Furthermore the condition and load-bearing capacity of the individual joints as well as the individual materials used (concrete, reinforcing steel, prestressing steel and epoxy resin) were examined.

In the course of conducting the international survey on the experiences with precast bridges, which will be presented in a following section of this chapter, a field report on the damages on a three-span precast bridge from 1983 located in Austria, with a total span of 54 m and a carriageway width of 8.0 m, were received. Thirty years post construction the following damages were detected: expulsion of moisture, sintering, salactite formation on the superstructure, damage to the screws, spalling, damage to the waterproofing and bridge drainage system as well as cracks in the concrete of the superstructure.

2.1.5.2 Germany

As mentioned before, Germany is one of the countries, in which the market share of precast bridges lies somewhere between 5% and 20%, with the decision whether or not to build precast depending on the economic activity and the big contracting companies themselves who dominate the entire market (*fib* Bulletin No. 29 (2004)). In general, the German market considers precast bridge construction to be a good alternative to in-situ construction and there has been a growing interest in developing new ways of incorporating this time saving construction method as shown by implementing prefabricated bridge girders in combination with an in-situ concrete deck in three integral jointless taxiway bridges in Frankfurt am Main Airport (Schmidt et al. (2012) and Steiger et al. (2012)).

Krumbach et al. (1997) describes the investigation of the long-term behavior and corrosion behavior of prefabricated prestressed concrete bridge girders that were used for bridges with spans up to 20 m in the former East Germany (GDR). The girders used for the testing were original beams that had stayed in the manufacturing plants and had been stored unprotected for up to 15 years. The main goal was to investigate the effects of potential damages that had occurred in the beams over the years, by testing both the entire girders as well as the individual components. All in all the results of the experiments showed that all the materials parameters met the specifications valid at the time of manufacture in accordance to GDR standards.

In the compilation of bridge damages in Germany by Ruhrberg et al. (1982) a 552 m long, 28.5 m wide, post-tensioned precast bridge construction, erected between 1961 and 1963 and consisting of 10 consecutive single-span fields, is described. Only 15 years after completion, the bridge structure showed heavy signs of usage, abrasion and damage. Cracks along the tendon elevation were found in most of the webs of the 60 precast beams, with widths up to 0.70 mm in mid-span. Signs of carbonization and corrosion were determined at the anchors of the tendons and cavities and loose concrete shells were found with corroded reinforcement bars underneath. The inspection report stated that the damages were mainly due to the insufficient concrete cover of both the regular steel reinforcement as well as of the stressed tendons and inadequately grouting of tendon ducts (either there being none or it was executed with too much pressure).

The *fib* Bulletin No. 29 (2004) reports of bad experiences with prestressed precast bridge beams in Germany with spalling cracks at the ends probably resulting from the large prestressing forces needed to prevent decompression under full loading.

2.1.5.3 Switzerland

A summary of applied post-tensioning and prestressing as well as anchorage systems and the findings on corrosion in Switzerland have been described by Hunkeler et al. (2005). The report outlines the investigation of structural elements, amongst others of prefabricated structural elements, which were obtained after dismantling existing bridges. In this case most of the bridges were constructed using a combination of prestressing during casting and subsequent post-tensioning on-site. The durability problems, which were the cause for the necessary bridge replacement, were traced back to the low concrete cover of the tendons, the corrosion of both steel reinforcement and tendons, carbonization, low quality of the concrete, damaged tendon anchorages, insufficient protection of the supplementary components of the tendons, as well as the large amount of joints between the elements.

2.1.5.4 Italy

In addition to the described findings on failure of precast elements in Switzerland, Hunkeler et al. (2005) reports of incidents in further countries. On the 23rd of April, 1999, the San Stefano Bridge, a bridge constructed using precast elements not far away from Taormina in Sicily, suddenly collapsed. At the time of failure there were no loads on the carriageway and no previous heavy or special transports were accounted for.

The collapse was explained in the following manor; the sea-side edge girder failed causing the mortar joints between the precast elements to open enabling it for the tendon strands to be pulled out of the insufficiently filled ducts. The protection of the tendons by an appropriate concrete cover as well as a proper grouting of the ducts was deficient. Due to the lack of monitoring, no measures were taken, although there were clear signs of damage (cracks and spalling). The collapse of the San Stefano Bridge led to a rethinking of priorities for repair and monitoring of bridges in Italy. The five bridges that were built similarly and in proximity to the collapsed bridge would be inspected more closely and repaired.

2.1.5.5 United Kingdom

Hunkeler et al. (2005) also reported of serveral collapses of segmental bridge constructions in the second half of the 20th century in the United Kingdom. Following the collapse of the Brickton Meadows pedestrian bridge in Hempshire in 1967, the Ynys-y-Gwas Bridge collapsed in West Glamorgan in 1987. Both bridges were post-tensioned segmental bridges and collapsed due to corrosion of the tendons. In 1992 a temporary ban, which was lifted in 1996, was imposed on the use of post-tensioning in bridge construction.

Despite the sporadic problems with tendon corrosion, most segmental bridge construction in the United Kingdom are in good condition. Most problems and damages are due to poor quality of workmanship and design flaws rather than to the segmental bridge as a construction type.

2.1.5.6 Poland

In the 1950s bridge construction using precast concrete girders started to gain in popularity in Poland. The share of running meters of the newly constructed bridges from 1956 until 1990 that were erected using precast elements is listed in Table 2.1. From 1966 up to 1985 more than 80% of the constructed concrete bridge running meters were made using precast elements.

By the 1990s, when the number of newly constructed precast bridges started to decline, 20 different systems of precast beams had been designed and successfully implemented in various bridge constructions. The designs ranged from regularly reinforced concrete beams to beams with

Tab. 2.1: Bridge construction using prefabricated elements in Poland (public road system) from 1956 to 1990 (Biliszczuk, Machelski, et al. (1994))

Time Period	**Concrete bridges [m]**	**Concrete bridges built using precast elements [m]**	**Percentage of concrete bridges built using precast elements**
1956 - 1960	15 436	1 954	12.66 %
1961 - 1965	21 621	6 420	29.69 %
1966 - 1970	9 978	8 428	84.47 %
1971 - 1975	13 756	10 954	79.63 %
1976 - 1980	20 664	18 742	90.70 %
1981 - 1985	14 176	12 829	90.50 %
1986 - 1990	16 707	12 201	73.03 %

prestressing or post-tensioning. Most of the solutions were meant to be erected in combination with in-situ concrete, with a tendency to minimize the proportion of in-situ concrete in the 1970s. Over time, not only the variety of girder systems increased but also the number of applied static systems. Multi-span systems started of, from 1956 to 1975, as simple span systems with a high number of dilatation joints, to be constructed later on as bridges with partial or full continuity or rigid frame structures.

Unfortunately, despite the strong development of prefabricated construction in Poland, bridge monitoring and inspections have shown that the prefabricated structures tend to have a very low manufacturing quality and therefore insufficient durability. Siwowski (1995) summed up the usual problems observed in precast bridges in Poland as follows:

- Insufficient transverse rigidity of the constructions
- Poor quality and damage to the waterproofing of the superstructure
- Damage resulting from working and dynamic loads
- Poor quality of the expansion joints
- Insufficient filling of the joints between the precast elements resulting in leakage
- Concrete quality lower than specified

In 1995, an estimation was made, that at least 30% of the bridges in Poland constructed using prefabricated elements require immediate rehabilitation or at least reinforcing measures. Furthermore, the problem of the insufficient or poorly executed sealing and drainage systems, which should normally protect the structure from harmful environmental effects, was addressed. Bridges which were designed as continuous structures exhibited a good stability in transverse direction and showed satisfactory durability (Biliszczuk, Machelski, et al. (1994) and Siwowski (1995)).

2.1.5.7 Slovakia

In the past 50 to 60 years, many bridges with short and medium spans have been erected in Slovakia using prefabricated, mostly post-tensioned, concrete girders. The construction companies wanted to reduce the girder weight to a minimum, resulting in very slender elements, with an often too small concrete cover, which led to an unsatisfactory level of serviceability and durability, with problems occurring after barely 30 to 40 years (Nad (2004)).

One of the most frequently used precast beams in Slovakia was the Vlossak beam, with over 700 individual applications in bridge construction. The Vlossak beam was among the first prefabricated girders produced in Slovakia in th 1950s, with the production process being rather inaccurate. The used wooden formwork caused a rough surface of the beams and was often not sealed properly, which led to leakage and drainage of the cement mortar from the corners of the lower flange, resulting in insufficient concrete quality and inadequate protection of the reinforcement (Brodnan et al. (2014)).

Static and dynamic load tests on a 35-year-old bridge as well as five destructive load tests on 21.4 m long precast beams removed from a bridge, were carried out at the Technical University of Košice. The girders were post-tensioned in both longitudinal and transverse direction. Despite the observed corrosion damage to the reinforcement and the longitudinal tendons, the load tests showed that the load-bearing capacity was still satisfactory. This was explained by the very good condition of the transversal post-tensioning (Nad (2004)).

Brodnan et al. (2014) presents a survey on prestressed concrete bridges with small spans in Slovakia conducted by the University of Žilina as well as three case studies. The described 55.90 m long bridge on the I/28 Road was built using Vlossak beams and showed damage and corrosion of the tendons at the outer beams as well as damages to the joints between the girders and the superstructure. The second presented bridge, built in 1957, spans a total of 71.64 m and consists of forty prestressed T-beams and had solely damages of the waterproofing and some joints, resulting in a damage extent of about 7% to 8% of the total area. The last case study refers to a 292.30 m long fifteen-field bridge which was built in 1971 using Vlossak beams (12 beams per span). Even though the bridge inspection had a positive assessment, damages to the superstructure were existent. The main damages where cracks and mortar losses in the joints between the girders and cracks in girders themselves accompanied by spalling and corrosion of the reinforcement and the tendons.

2.1.5.8 North America

Segmental bridge construction has established itself quite well on the North American market. The first bridge built using this method was the John F. Kennedy Memorial Causeway Bridge, connecting Corpus Cristi and Padre Island in Texas, in 1973. Within the next 30 years breathtaking and technologically advanced segmental bridges had been erected in the USA. The success story of this bridge construction method can be explained by the cost-effectiveness when considering the life-cycle-costs and by the very short construction time on site. A very prominent example for the durability of segmental bridges is the Dauphin Island Bridge in Alabama, which had survived five Hurricanes and several severe tropical storms in the time from 1982 until 2004, after being built as the replacement for the in-situ built concrete bridge which was destroyed 1979 by Hurricane Frederick. The Dauphin Island bridge, with its 5 430 m length, was designed and built in 34 months (Figg et al. (2004)).

A study and survey on the durability of prefabricated segmental bridges was carried out in 1998 as part of the National Cooperative Highway Research Program (Poston et al. (1998)). The survey covered 109 segmental bridges erected in the USA. In general a good durability of the structures was observed without significant corrosion problems of the tendons. It must be noted that the findings were obtained solely from reports from visual inspections.

Even though most of the bridges investigated by Poston et al. (1998) showed no signs of problems with durability, case studies of damages in prestressed segmental bridges were presented in the report. One of these presented bridges was the Mid Bay Bridge, connecting an island to the mainland in the Gulf of Mexico in southern Florida. The construction consists of 140 fields spanning a total of 5 870 m. The bridge was constructed using prefabricated box girders, which were subsequently post-tensioned with external tendons. After only seven years post erection, the failure of two tendons was detected during a visual bridge inspection. The repairs of the damages included the replacement of tendons, regrouting of the anchorages and the improvement of the anchor protection.

2.1.5.9 Japan

An interesting case of fatigue damage was described by Schläfli (1999), Maeda et al. (1980), and HEPC (1987) concerning the Hanshin Expressway in Japan. The Hanshin Expressway was built from 1962 to 1964 by the Hanshin Expressway Public Corporation (HEPC) and is one of the main road networks in Japan connecting the cities Osaka, Kobe and Kyoto, with the majority of the rout running on bridges. As is the case in the USA, most Japanese highway and expressway bridges are built as composite constructions, often using steel girders but sometimes also using precast concrete girders.

Not long after the Expressway was finished, fatigue damage was discovered. Cracks criss-crossed large parts of the structure, however, the crack pattern was initially indistinguishable from a crack pattern that would result from a static load. After some time, separation cracks, going through entire deck plates, were found. Spalling and the damage of entire pieces of the plate soon followed.

Numerous causes were determined for the fatigue damage, none of which were traced back to the fact that prefabricated elements were used, since the same problems would have occurred with in-situ cast structures. One of the main problems was the rapidly growing traffic load, which was not anticipated. Furthermore the deck plates were designed too thin, there was too little transverse reinforcement, the position of zero moment was assumed to be at 1/4 of the span length, therefore the mid-span was undersized, and the concrete quality was not optimal, on the one hand due to the used aggregate which caused increased shrinkage on the other hand due to the separation of the concrete before it was pumped on-site.

2.1.6 International survey - Experiences with prefabrication in bridge construction

An international survey was conducted in order to obtain information and the individual opinion of experiences with prefabrication in bridge construction of experts in Austria and worldwide. The questionnaire consisted of eight questions that would allow the evaluator to get an overview of the present situation in the queried institution or country. The questionnaire was sent to the bridge department of Austria's motorway and expressway agency ASFINAG - Autobahnen- und Schnellstraßen-Finanzierungs-Aktiengesellschaft (hereafter referred to as ASFINAG), the bridge department of the Austrian Federal Railways - Österreichische Bundesbahnen (hereafter referred to as ÖBB), the bridge departments of all nine Austrian states as well as bridge construction experts mainly contacted through the World Road Association - PIARC (hereafter referred to as PIARC).

The questionnaire consisted of the following eight questions:

1. Number and area [m^2] of bridges in area of responsibility.
2. Number and area [m^2] of concrete bridges in area of responsibility.
3. Number and area [m^2] of precast concrete bridges in area of responsibility.
4. Which types of precast bridges have been built?
   a) Longitudinal precast girders and a cast in-situ slab
   b) Longitudinal precast girders and a combination of precast elements and cast in-situ in the slab
   c) Precast and prestressed girders and a cast in-situ slab
   d) Precast and prestressed girders and a combination of precast elements and cast in-situ in the slab
   e) Precast elements and cast in-situ concrete in the slab
   f) Segmental bridges
   g) Other construction systems
5. Did you notice an unfavorable influence of precast bridges in comparison to cast in-situ bridges concerning the following bridge components?
   a) Wearing surface
   b) Bearings
   c) Expansion joints
6. Are there more damages in the load-bearing structure of precast bridges in comparison to cast in-situ bridges?
7. Which damages did you observe in precast bridges which did not also occur in cast in-situ bridges?
8. Would you permit the application of precast elements in the construction of new bridges in your area of responsibility?

2.1.6.1 The questioned parties

Of the sent out questionnaires, 19 were returned, of which ten were from Austria and the remaining nine from Canada, Japan, Belgium, Romania, Argentina, China, the USA, Denmark and South Africa. The listing of the participants with a specification of the institution with the abbreviations that will be used hereafter can be found in the following table.

Tab. 2.2: Participating parties in the questionnaire concerning the application of prefabricated elements in bridge construction

Abbreviation	Country	Institution
A-C	Austria	Office of the Carinthian Government - Department 9. for Roads and Bridges
A-V	Austria	Office of the Vorarlberg Government - Department for Road Construction
A-W	Austria	City Administration of Vienna - Municipal Department MA 29 for Bridge Construction and Foundation Engineering
A-OÖ	Austria	Office of the Upper Austrian Government
A-B	Austria	Office of the Burgenland Government - Department for Bridge Construction
A-S	Austria	Office of the Salzburg Government - Department 6/22 for Bridge Construction
A-ÖBB1	Austria	Austrian Federal Railways (ÖBB) Infra SEA OST 1
A-ÖBB3	Austria	Austrian Federal Railways (ÖBB) Infra SEA OST 3
A-A1	Austria	Austria‘s motorway and expressway agency (ASFINAG) ASG (Tyrol)
A-A2	Austria	Austria‘s motorway and expressway agency (ASFINAG) Service GmbH
CDN	Canada	Ministry for Transports of Quebec
JPN	Japan	Honshu Shikoku Bridge Expressway Company Ltd. - Maintenance Planing Division
BEL	Belgium	Public Service of Walonia
ROU	Romania	Romanian National Company of Highways and National Roads, Regional Administration for Roads and Bridges Bucharest
ARG	Argentina	National Highways Directorate
CHN	China	CCC Highway Consultant CO. LTD, Xicheng District
USA	USA	Bureau of Structures and State Bridge Engineering, Wisconsin Department of Transportation
DNK	Denmark	Road Directorate
ZAF	South Africa	South African National Roads Agency

2.1.6.2 Statistics - Questions 1 through 3

An overview of the answers to the first three question concerning the amount and area of bridges, concrete bridges and precast concrete bridges in the area of responsibility of each expert are shown in Table 2.3, with n.s. (not specified) used if no answer was submitted. The results of the survey correspond well with the information that was obtained from literature. A large percentage of the Austrian bridges are made out of concrete but a very low percentage of these were erected using prefabricated elements, the same applies to Denmark and South Africa. A high percentage of concrete bridges with prefabricated elements was confirmed for Canada, Japan, Belgium, Romania and the USA. The percentage of the total amount of bridges that are made out of concrete exceed 80% in the most cases, except in Canada, Japan and the city of Vienna as well as the bridges of the Austrian Federal Railways.

Tab. 2.3: Listing of the distribution of bridge types in the areas of responsibility of the questioned experts (Questions 1 through 3)

Expert	**Bridges**		**Concrete Bridges**			**Precast Bridges**		
	Amount	m^2	Amount	m^2	%*	Amount	m^2	%**
A-C	1 735	356 177	1 406	282 361	81	10	4 686	<1
A-V	615	199 665	545	177 018	89	30	25 164	6
A-W	866	605 838	593	420 000	68	10	9 000	2
A-OÖ	3 100	690 000	2 500	600 000	81	40	14 000	2
A-B	580	n.s.	550	n.s.	95	11	n.s.	2
A-S	1400	n.s.	n.s.	n.s.	-	n.s.	n.s.	-
A-ÖBB1	840	227 254	413	148 821	49	24	5 332	6
A-ÖBB3	986	149 371	381	61 999	39	19	3 218	5
A-A1	585	791 000	539	634 000	92	16	51 000	3
A-A2	924	995 520	824	939 398	89	41	166 490	5
CDN	5 357	5 244 000	1 833	2 108 000	34	744	1 007 000	41
JPN	401	1 485 000	207	312 000	52	80	104 000	39
BEL	3 700	ns	2 500	n.s.	68	1 000	n.s.	40
ROU	535	304 927	494	261 340	92	178	150 698	36
ARG	3 446	1 993 091	3 256	1 826 771	95	734	411 750	23
CHN	n.s.	n.s.	n.s.	n.s.	-	n.s.	n.s.	-
USA	5 000	n.s.	4 000	n.s.	80	1 500	n.s.	38
DNK	2 150	1 505 000	1 957	1 459 850	91	105	60 2000	5
ZAF	3 410	2 774 753	3 384	2 766 872	99	52	55 754	2

* Percentage of the total amount of bridges that are made out of concrete

** Percentage of the total amount of concrete bridges with prefabricated elements

2.1.6.3 Types of erected precast bridges - Question 4

The answers to the fourth question of the questionnaire gave an overview of the types of precast bridges that had been built in the areas of responsibility of each expert. In Austria, the occurrence of all mentioned types of bridges was confirmed with the addition of rigid frame constructions out of precast elements, and full depth precast slabs.

In Canada solely precast and prestressed girders with a cast in-situ slab were noted, with the exception of one segmental bridge and six partial depth or full depth slab bridges. All 80 bridges mentioned by the Japanese expert are longitudinal precast girders with a cast in-situ slab. The bridges in Romania are either precast and prestressed girders with a cast in-situ slab or a combination of precast elements and cast in-situ in the slab. The Argentinian expert claimed that all but segmental bridges could be found in their area of responsibility.

In Denmark all of the precast bridges are upturned precast T-girders with a layer of in-situ cast concrete on top. In the area of responsibility of the American expert most of the precast bridges (1400 bridges) are precast and prestressed girders with a cast in-situ slab, whereas the precast bridges of the other types, with the exception of bridges made of precast elements and cast in-situ concrete in the slab, and in addition of 3-sided culverts (precast and prestressed) only exist in small numbers. The precast bridges in South Africa dived into three types, two being precast and prestressed girders with either a cast in-situ slab or a combination of precast elements and cast in-situ in the slab, and one being the erection by incremental launching.

No exact figures where given by the Chinese expert but it was explained that more than 95% of the precast bridges with spans smaller than 20 m were constructed using longitudinal prefabricated girders and a cast in-situ slab, more than 95% of the precast bridges with spans between 20 m and 50 m were built using precast and prestressed girders and a cast in-situ slab, and that almost 100% of the precast bridges with spans larger than 100 m were built using segmental construction. Furthermore the expert confirmed that precast bridges erected using longitudinal precast girders or precast and prestressed girders with a combination of precast elements and cast in-situ concrete in the slab or precast elements and cast in-situ concrete in the slab, also existed in his area of responsibility.

2.1.6.4 Experiences with precast bridges - Questions 5 through 8

When asked about the unfavorable influence of precast bridges in comparison to cast in-situ bridges concerning the wearing surface, the bearings and the expansion joints almost all of the questioned claimed that there were no additional harmful influences due to the application of prefabricated elements. Three of the Austrian experts found there to be increased damages to the wearing surface, with one of the experts also stating there to be increased damages to the bearings and the expansion joints. The Japanese expert explained that damage to the wearing surface resulted in water from the pavement penetrating between the precast girders and the cast in-situ concrete and that water which seeped through the expansion joints had an effect on the ends of the precast girders by penetrating into the ducts through the anchors of the tendons.

Four of the questioned felt that precast bridges or bridges erected with prefabricated elements had a higher risk of damage than cast in-situ bridges. When asked about the damages that could be observed in precast bridges which did not also occur in cast in-situ bridges eight experts were able to name the follow problems:

- A large amount of joints (weakening points of the construction). Water penetrates between the precast girders and the in-situ concrete causing damage to sub- and superstructure.

- Precast girders from the 1970s tend to have small concrete cover resulting in damage to the reinforcement.
- Complications during the repair due to high concrete strength.
- Beams are vulnerable to impact (over-height trucks and buses).

When asked about the approval of the incorporation of precast concrete elements in the construction of new bridges in their area of responsibility (Question 8) all but one answered positively, but not without stating various requirements and conditions. A justification for the application of the prefabricated elements would be needed as well as a guarantee of appropriate elements thicknesses, sufficient concrete cover and proper sealing of the structure.

2.1.6.5 Further remarks

One third of the questioned experts left further remarks or accounts of experiences with precast bridges at the end of the questionnaire. The Austrian experts addressed the problems of the prefabricated bridges built between 1968 and 1985, in which the beams were kept very slender and the concrete cover to a minimum. Furthermore they mentioned the difficulties that occur when restoring precast elements, that often have a very high concrete strength.

The problems with the bridges of the Honshiu-Shikoku Expressway in Japan, that were built before 1998, were discussed by the Japanese expert. The bridges were mainly built using prefabricated T- and I-beams, in-situ cast transverse beams with an in-situ cast concrete deck. Durability problems occurred with water leaking between the T-beams and the cast in-situ concrete and water entering the ducts causing the tendons to corrode. The current improved construction method redesigned the connection between the precast girders and the cast in-situ concrete by including rebars between the individual girders. Furthermore the anchorage of the tendons was moved from the deck surface, as was the case until 1993, to the end of the girders, therefore reducing the risk of water penetration.

2.2 Lightweight thin-walled precast concrete elements in bridge construction

The search for the most appropriate cross-section of precast bridge elements is according to the *fib* Bulletin No. 29 (2004) of major importance in prefabrication, due to the impact on the weight of the units, which can be a big factor in the bridge construction costs. With this in mind, designs have been focusing strongly on the idealization of the dimensions of the concrete elements by reducing web thickness as well as the dimensions of the upper and lower flanges or even rethinking the necessity of end blocks. The different developments are often based on specific research programs of, for example, universities or governmentally financed research facilities and national code stipulations.

The Institute of Structural Engineering of the TU Wien has focused on the development of lightweight girders constructed out of thin-walled precast elements that could easily become an attractive alternative option to steel when it comes to the construction of bridges without formwork. The idea of Kollegger and Wimmer (2014) was to combine the attributes of precast construction, by using lattice-girder floor slabs and double wall elements, which have been successfully used in building and industrial construction for decades, with all positive aspects of cast in-situ concrete. With this goal, trough-shaped girders were developed, which would serve as permanent formwork for subsequently added in-situ concrete therefore considerably reducing the use of formwork and falsework on the construction site and creating monolithic maintenance-friendly constructions. Furthermore the girders would be light enough for transport and erection by conventional means (conventional transport and lifting equipment) and could be used for the construction using conventional and innovative building methods, as for example the balanced lift method, a building method which was developed at the Institute of Structural Engineering of the TU Wien (Kollegger, Foremniak, et al. (2014)).

The development of the balanced lift method with patents granted in Germany (DE 102006039551), the USA (US 7996944), Russia (RU 2436890), Canada (CA 2661311, China (CN 101535571), Australia (AU 20007288151), Japan (JP 022359), India (IN 022359), Norway (NO 338580) as well as the European Patent (EP 2054553), actually initiated the deliberations on lightweight thin-walled precast conrete girders, due to the fact that the weight of the girders used during the lifting and rotating process is of utmost importance. As shown in Figure 2.1 for bridges with tall and short piers, the individual girders of the bridge are assembled in a vertical position and are then rotated into their final horizontal position with the aid of compression struts. This kind of erection could be very favourable depending on the given boundary condition as for example limitations of work space underneath the bridge itself, therefore eliminating the possibility of building with falsework (Gmainer (2011)).

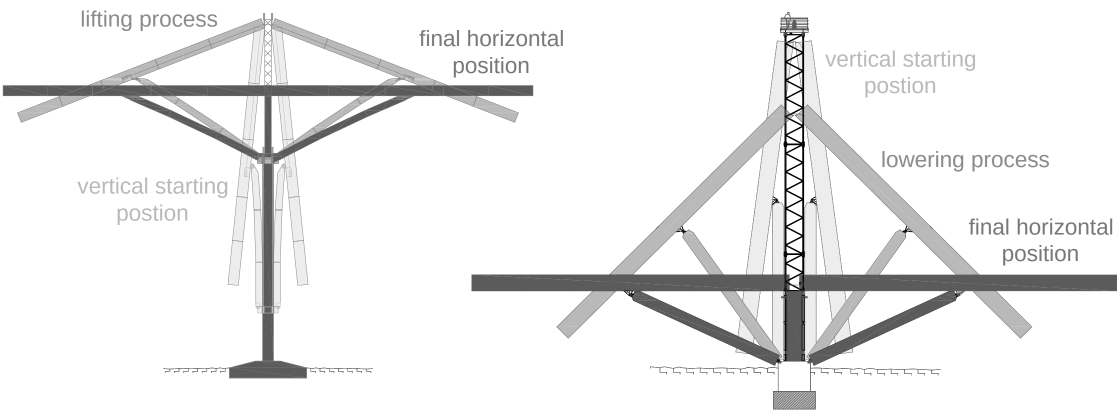


Fig. 2.1: Balanced lift method for bridges with tall and short piers

By combining thin-walled precast girders with cast in-situ concrete a monolithic structure with a perfect connection between the individual elements can be created. This process offers many possible static systems and a wide range of applications. Potential application of these precast elements for bridge and civil engineering structures have been investigated ever since their development in 2010. All designs for the application of thin-walled precast concrete bridge girders in bridge constructions up until the year 2014 will be described in this chapter.

2.2.1 Construction of bridge girders made of thin-walled precast elements

Wimmer (2016) describes two different production processes for the construction of the developed lightweight thin-walled precast trough-shaped bridge girders; one using lattice-girder floor slabs, the other double wall elements. A girder cross-section made using 70 mm thick floor slabs as well as a girder cross-section made using double wall elements are shown in Figure 2.2.

Fig. 2.2: Cross-sections of the designed lightweight thin-walled precast girders with the girder made of lattice-girder floor slabs on the left and the girder made of double wall elements on the right (Wimmer (2016))

The production of both the lattice-girder floor slabs and the double wall elements takes place in a conventional manner in a manufacturing plants. Once the 70 mm thick floor slabs are hardened the elements are placed upright, the additionally needed reinforcement and the ducts for the post-tensioning are installed, before the 100 mm to 200 mm thick base plate is cast, creating a trough-shaped cross section. To stabilize the cross-section for the different loads during the construction process, starting with all assembly and lifting operations, a truss made of reinforcing bars is welded to the reinforcing bars protruding from the precast side wall elements at the top of the cross-section, converting the U-shaped cross-section into a box-section.

If double walls are used instead of precast slabs the production steps are reduced by eliminating the process of placing the individual slabs upright and welding the truss out of reinforcing bars to the top of the cross-section. The double walls can simply be placed upright and the base plate can be cast immediately. Unfortunately, due to production reasons, the maximum width of the double wall girders is limited to 500 mm, leaving a restricted amount space for any necessary tendons or reinforcement and therefore setting boundaries to their field of application to multi-girder bridges with spans of approximately 20 m. All necessary ducts and additional reinforcement have to be incorporated into the reinforcement cages of the double wall elements during their production in the manufacturing plant.

Owing to the automatic manufacturing process not only the widths of the double wall elements are restricted. The maximal height and length of both double walls and prefabricated slabs are limited to 3.40 m and 12.0 m respectively.

In order to create girders with longer lengths than 12.0 m, the individual slab elements or double wall elements are placed next to each other, the joints are filled with grout and the continuous base plate is cast. By subsequently post-tensioning U-shaped cross sections, girders with enough stability for all further construction processes are created. Individual segments can also be simply post-tensioned together without a continuous base plate to obtain longer girders than 12.0 m.

To ensure the load capacity and serviceability of the finished parts during transport and concreting, the semi-finished part girders are post-tensioned. By post-tensioning the transverse joints are suppressed and weak points in the final state can be excluded. At the construction site, the lightweight thin-walled precast girders are mounted to their final position before being filled with in-situ concrete. According to Wimmer (2016) the scope of application includes bridge structures with field-spans of up to 50 m.

2.2.2 Executed tests on thin-walled precast concrete bridge girders

In order to investigate the feasibility and the structural behavior of the developed girders, numerous large scale tests on three 30 m long girders as well as the construction of a 70% scale bridge structure using lightweight thin-walled precast girders erected by the balanced lift method were carried out (Wimmer (2016)).

The first of the three 30 m long girders, consisting of three individual segments, was constructed using 70 mm thick lattice-girder floor slabs assembled together to form a trough-shaped cross-section with a 200 mm thick base plate. The middle segment of the girder consisted of four slabs, two on each side, and had a total length of 18.0 m. Both end segments, with lengths of 5.40 m were additionally equipped with 3.0 m wide transverse beams, which had the purpose of an anchoring block for the post-tensioning and would counteract any torsion loads during the testing. The three segments were joined together by simply filling the joints and subsequently post-tensioning the entire girder. The girder did not have a continuously cast base plate and no additional reinforcement was placed over the joints. This kind of connection was chosen in order to show that an assembly of girders, with larger lengths than allowed for transport, made out of individual sections would be possible on a construction site.

With the girder made out of the lattice-girder slabs completed, the load tests could commence. The goal of the first tests was to show that the designed girders would be able to withstand the loads of being filled with concrete. The girder was placed on two abutments and was filled with cast in-situ concrete in four steps within 24 hours, with the post-tensioning being increased during the procedure. The recorded deflections were used to determine the stresses caused in the precast girder due to the applied load. The casting of the concrete using a pump can be seen in Figure 2.3.

Fig. 2.3: Large-scale experiment on girder out of lattice-girder slabs - Filling of the girder with cast in-situ concrete

The positive results of the first experiments demonstrated that not only the construction of the newly developed lightweight thin-walled precast girders was possible but also time-efficient and uncomplicated. Furthermore, the feasibility of the construction method was proven by the positive outcome of the concreting tests.

Once the girder constructed using floor slabs was filled with in-situ cast concrete, creating a girder with an rectangular cross-section, it was subjected to further large-scale tests. Two consoles were subsequently added in mid-span in order to induce a pure torsion load. At a maximum force of 500 kN/press a torsional moment of 2 150 kNm per side, as shown in Figure 2.4, could be induced. The results confirmed that the load capacity of the girder made out of thin-walled precast elements which were subsequently filled with in-situ concrete did not differ from the load capacity of a girder made purely out of cast in-situ concrete with the same dimensions.

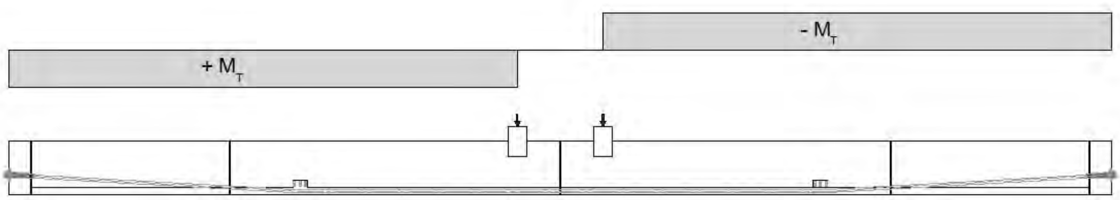


Fig. 2.4: Large-scale experiment on girder out of lattice-girder slabs - Pure torsion loading (Wimmer (2016))

The stress redistribution between the precast elements and the cast in-situ concrete and the creep behavior of the entire girder were monitored over a period of four years. Suza et al. (2015) shows how the initially high stresses in the prefabricated outer shell are gradually reduced with the cast in-situ concrete subsequently subjected to axial compression strength. This redistribution should be considered in all calculations when designing with lightweight thin-walled precast girders.

In addition to proving that neither the construction, nor the filling with concrete or the application of torsion loads caused any problems for the girders made out of thin-walled precast elements, it was decided to investigate if the introduction and distribution of the post-tensioning caused any problems in the joint between the transverse beam and the girder itself. For this experiment one of the end sections of the first tested girder was additionally loaded using six monostrands applying six times 180 kN to the sections. The results showed that the load was easily transfered from the transversal beam to the girder causing no problems in the joints.

After testing the construction, filling with cast in-situ concrete and the behavior of the girders when loaded with a torsion load, Wimmer (2016) ran further tests on additional girders. The following two test specimens were 30 m long girders constructed using double wall elements with the maximal width of 500 mm and a height of 1.5 m. Both girders had the same dimensions and differed solely in the assembly process.

The first girder was assembled exactly in the same manor as the previously described girder out of thin-walled slabs. It consisted of four 7.50 m long U-shaped sections without a continuous base plate and two transverse beams that were all connected together by grout and subsequent post-tensioning. The second girder was produced entirely in the manufacturing plant, with a continuous base plate and continuous reinforcement over the entire length of 30 m. The production and transport of this girder to the test site showed that girders with such lengths could be produced in manufacturing plants and transported to the construction site as a whole without any severe problems.

The girders constructed using double walls were used to gain insight on the load bearing capacity of the girders in an unfilled state when it came to large compressive stresses caused by the post-tensioning. The girders were submitted to centric pressure tests using a large hydraulic jack placed directly at one end of the girder on the transverse beam. The first girder failed not due to concrete failure but due to the failure of the joint between the transverse beam and the girder. The joint was not executed correctly, causing a slip, resulting in the lack of post-tensioning of the entire structure making the four pieces fall apart and fall to the ground. This mishap showed how important a proper joint design and execution is when it comes to the construction with thin-walled elements and that special care must be taken when it comes to distribution of the post-tensioning forces.

The second girder made out of double wall elements and subjected to the centric pressure tests, showed that if all joints are executed correctly a local buckling as well as a stability failure of the light-weight girders can be ruled out. The girder failed under a pressure of -50 MPa, a value that is way above any limit set by current standards. In addition to the individual 30 m long bridge girders that were manufactured and tested, two 25 m long girders and two 12.67 m long compression struts, as shown in Figure 2.5, were designed, produced and implemented in the course of a research project exploring the application of thin-walled concrete elements in the construction of bridges using the balanced lift method explained in detail by Gmainer (2011).

Fig. 2.5: Large-scale testing of the balanced lift method with the application of lightweight thin-walled precast concrete girders (Wimmer (2016))

In comparison to the previously described test girders the bridge girders used for the balanced lift experiment had a varying width, with the larger width of 1.4 m at the connection points with the compression struts. The compression struts themselves were precast hollow reinforced concrete elements with a weights of 115 kN each. The bridge girders weighed 208 kN each. The entire manufacturing process for all individual elements is described in detail by Wimmer (2016), whereas the assembly of the large-scale experiment is explained by Gmainer (2011).

A 100 t mobile crane was used for all mounting operations and after only four days of erection, the bridge was ready for the balance lift phase of the construction process. The lowering process was carried out with the help of two mobile cranes. The maximum lifting force, which had been calculated by Gmainer (2011) to 270 kN, corresponded well with the actual lifting force measured by the cranes.

With this large-scale experiments not only the feasibility of the balanced lift method was proven but also the effective implementation of the lightweight precast girders. The girders and compression struts were entirely prefabricated, therefore no additional assembly was necessary on the test site. Once the bridge girders were rotated from the vertical assembly position into their final horizontal position the geometry was adjusted before the node above the pier was filled with cast in-situ concrete in order to provide a higher resistance against horizontal wind loads.

2.2.3 Application of thin-walled precast concrete bridge girders in bridge construction

Applications for thin-walled precast concrete bridge girders in bridge construction have been described in detail by Kollegger, Foremniak, et al. (2014) and Wimmer (2016) for the case of building with the balanced lift method. In addition to all presented applications Wimmer (2016) assessed, with the means of a parameter study shown in Chapter 5, the scope of application of the girders for single-span bridges with carriageway widths of 8.5 m and 14.5 m. Furthermore Wimmer (2016) shows a design-scheme of a motorway overpass, presented in Figure 2.6, using lightweight thin-walled precast concrete girders in combination with prefabricated slab elements. In Chapter 5 and Chapter 7 further pratical application examples developed based on all research of Wimmer (2016) will be presented.

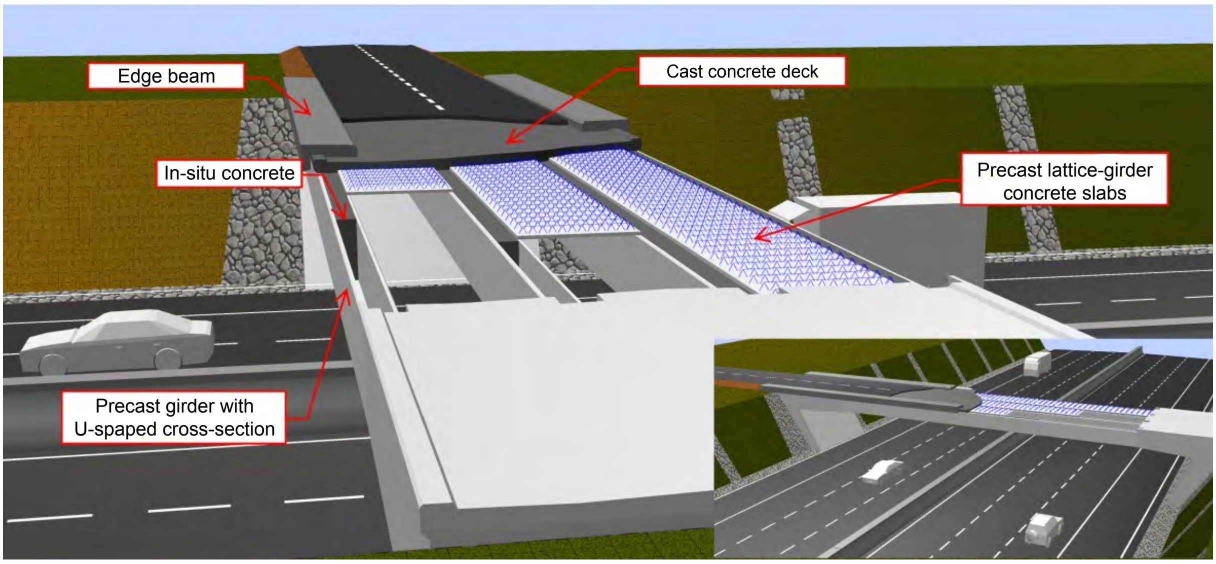


Fig. 2.6: Design-scheme of a motorway overpass erected using lightweight precast concrete girders and prefabricated slab elements (Wimmer (2016))

The first application described by Kollegger, Foremniak, et al. (2014) is a design for a bridge over the River Salzach in Golling. Two piers with heights of 20 m and 22 m would be needed for the bridge with a total length of 78 m and a central span of 33 m. A carriageway width of 5.5 m was designed with a T-beam cross-section using precast girders with a width of 2.0 m and a height of 0.8 m.

The second application described by Kollegger, Foremniak, et al. (2014) as well as by Wimmer (2016) and Gmainer (2011) is a design of two bridges over the Rivers Lafnitz and Lahnbach in the south-eastern part of Austria. The bridge of the River Lafnitz is the original design that was scaled down for the experiments described in the previous section and shown in Figure 2.5. It was possible to convince ASFINAG of both financial and aesthetic benefits of the design, resulting in a commission of the detailed design and a building begin in 2018.

3 Experimental preliminary tests of grout materials

When building with prefabricated elements one of the main concerns and focus points of the contractors is the detailed constructions of the resulting joints. In precast construction, in particular in the construction of bridges using lightweight thin-walled precast girders, a distinction is made between the different kind of joints and their stresses.

Thin-walled girders consist of either double wall elements or thin precast slabs that are connected together by a bottom plate. Owing to the fully automatic manufacturing process, the maximum width of the double wall elements is limited to 0.50 m, and the maximal height and length of both double wall elements and precast slabs are limited to 3.40 m and 12.0 m respectively. If a girders length surpasses these measurements more than one of the double wall elements or precast slabs can be put side by side before the bottom slab is cast, therefore creating bridge girders with various lengths but at the same time with vertical joints.

As described by Kollegger and Wimmer (2014) and by Wimmer and Kleiser (2013) lightweight thin-walled bridge girders with lengths up to 30.0 m are realistically producible in a manufacturing plant and transportable to a construction site. Short single-span bridges with spans up to about 30 m can therefore be constructed using these girders. The only joints that exist in this case are the vertical ones between the individual double wall elements or individual thin precast slabs in addition to the horizontal ones between the precast girders and the subsequently added in-situ concrete for the deck slab of the bridge, as shown in Figure 3.1, as well as the joints between the thin-walled precast elements and the cast bottom plate only visible when looked at the girders from the bottom.

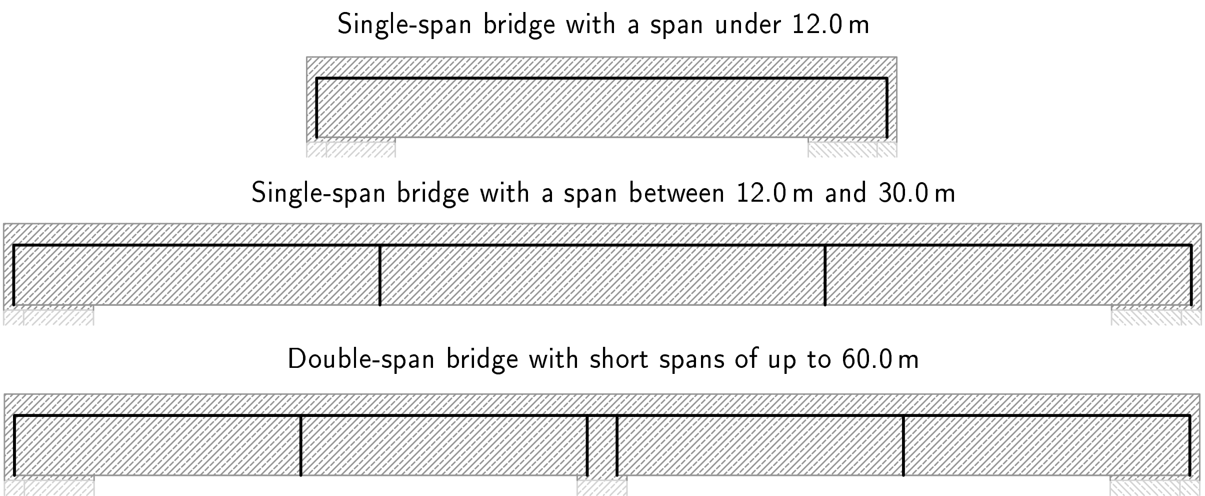


Fig. 3.1: Single-span and double-span bridges using lightweight thin-walled girders with a illustration of the vertical and horizontal joints

For double-span bridges, as elaborated for a bridge with two 25.0 m spans by Wimmer (2013), two girders are used and the connection between the pillar and the girders is filled on site, therefore creating a further joint that has to be considered. Even larger spans can be accomplished by using temporary supports in combination with post-tensioning as described in detail for larger cross-sections out of double wall elements and precast slab elements for large multi-span bridges in Chapter 6 and Chapter 7.

It was decided to take a closer look at problems due to too high deformation in the joints experienced during the testing of the lightweight girders carried out and described in depth by Wimmer (2016). Several grout materials were tested using standardized and non-standardized tests in order to define the most reasonable material to be implemented in the future. The most suitable substances were then further tested in static experiments. A redesigning of the joints, especially for the case of a girder connection without post-tensioning, was also conducted.

3.1 Determination of material parameters

All previously conducted large scale tests (Wimmer, 2016) showed that a joint thickness of 20 mm is optimal for the construction with thin-walled precast elements. When building with thin-walled girders with post-tensioning, the grout in the joints has to withstand high strains a few days after installation. Furthermore the grout should, as do the prefabricated elements, protect the installed reinforcement and post-tensioning cables from carbonation and corrosion. This protection is only possible if the interface between the precast element and the grout stays intact, therefore the grout should not shrink extensively and should be capable to absorb and transmit all strains.

With a great number of different grouting materials meant for all kinds of applications the first step to finding the appropriate grout was by defining the desired properties. The material would be filled into joints with a width of 20 mm to 30 mm, would need to have a high flowability, a low shrinkage, a high compressive strength after a short amount of time and a good bond to the precast elements, making cement based mortar to be the best suitable choice.

The main difference between cement mortar and concrete lies in the grading curve of the aggregates (Grütze, 2007). Concrete consists primarily of water, cement and aggregates with a maximum grain size of approximately 32 mm. Mortar has the same material composition, with the difference that the maximum grain size is set to 4 mm. The fine aggregates increase the compressive and tensile strength as well as the flowability in a favourable manor.

Thirty-one in Austria available mortars were considered, the product data sheets examined and compared. By excluding any grout which would not reach an early compressive strength of 30 MPa after 24 hours, a final compressive strength of 80 MPa after 28 days, exceeded the maximal aggregate size of 4 mm and was not meant for joint widths between 15 mm and 30 mm, five expanding grouts were chosen for further testing. The data describing the flowability of all five mortars was acceptable. Additionally all mortars were described as resistant to frost and de-icing salt making them suitable for the application in bridge construction.

The aim of the determination of the material parameters of the five chosen mortars was the verification of the information obtained from the manufacturers data sheets on the compression strengths, flexural tensile strengths, the shrinkage and expansion, the flowability and the flow spread. The material parameters according to the data sheets are listed in Table 3.1.

Tab. 3.1: Material parameters according to the data sheets from the manufacturers of the five grouts chosen for testing

		Grout 1	Grout 2	Grout 3	Grout 4	Grout 5
Uniaxial compression strength [MPa]	24h	> 70	> 40	34	40	30
	7d	> 90	> 60	66	65	60
	28d	> 100	> 80	85	80	80
Flexural tensile strength [MPa]	24h	-	> 4	6	-	5
	7d	-	> 6	9	-	-
	28d	-	> 8	11	-	10
Shrinkage		SKVM II*	SKVM II*	-	SKVM III**	-
Expansion	24h	$> 0.1\%$	0.5%	0.3%	-	$> 0.1\%$
Flowability		good flow	good flow	high flow	f2	good flow

*$\varepsilon_{s,m,91} \leq 0.12\%$ and $\varepsilon_{s,i,91} \leq 0.14\%$ after 91d

**$\varepsilon_{s,m,91} \leq 0.15\%$ and $\varepsilon_{s,i,91} \leq 0.2\%$ after 91d

3.1.1 Specimens

A total of 15 prismatic specimens and three additional conical specimens were needed per mortar type, resulting in a total of 90 produced and tested specimens. The prismatic specimens were produced in conventional steel molds according to ÖNORM EN 196-1 (2005) resulting in the following specimen dimensions: 40 mm by 40 mm by 160 mm. The conical specimens were produced in special 160 mm high molds which would allow the grout to harden without any constraints therefore allowing an optimal assessment of the expansion and subsequent shrinkage of the material.

The water-dry material ratio was set to 3.0 l/25 kg (12%). This mixing ratio corresponded to the information provided by the data sheets of all five tested types of mortar. During each mixing process 1.80 kg of dry matter was mixed with 216 ml of water producing enough material for three molds. A standard-compliant mixer according to ÖNORM EN 196-1 (2005) was used and the different mixing instructions explained in the data sheets were implemented.

3.1.2 Compression and flexural tensile strength

In the course of the investigation to determine the compression strength and flexural tensile strength 75 specimens were tested. The uniaxial compression strength was determined using three specimens after 24 hours, 7 days and 28 days by testing the specimens in an upright position (with a height of 160 mm and standing on the 40 mm by 40 mm surface). The assessment of the multiaxial compression strength and the flexural tensile strength after 24 hours and 28 days was conducted according to ÖNORM EN 196-1 (2005). First the flexural tensile strength was obtained dividing three specimens into six which were subsequently tested for their multiaxial compressive strength. The results of the conducted tests were averaged for each grout and can be found in Table 3.2 with the six means, that did not meet the characteristics from Table 3.1 stated by the manufacturers, set in bold.

Tab. 3.2: Average compression and flexural tensile strength in [MPa] assessed in the tests

		Grout 1	**Grout 2**	**Grout 3**	**Grout 4**	**Grout 5**
Multiaxial compression strength*** [MPa]	24h	96.6	49.9	61.4	58.7	39.1
	28d	117.8	**71.3***	**80.4***	**68.2***	**75.8***
Uniaxial compression strength** [MPa]	24h	90.3	52.3	49.8	52.7	37.9
	7d	105.9	59.4	66.0	58.5	59.6
	28d	112.9	56.7	64.8	63.7	70.3
Flexural tensile strength** [MPa]	24h	6.5	7.2	6.5	5.9	7.7
	28d	8.3	8.5	**7.1***	9.1	**8.9***

* Bold results do not meet characteristics stated by the manufacturers (see Table 3.1)

** Mean calculated from three test results

*** Mean calculated from six test results

The failure mechanism during the compression tests was very sudden and explosive, as shown with the shard left of the specimen made of grout 1 in Figure 3.2, which can be easily explained by the very high compression strengths of the tested mortars. The failures during the flexural tensile tests were rather subtle, sometimes only resulting in a fine crack that was barely visible as was the case with grout 3 shown in Figure 3.2.

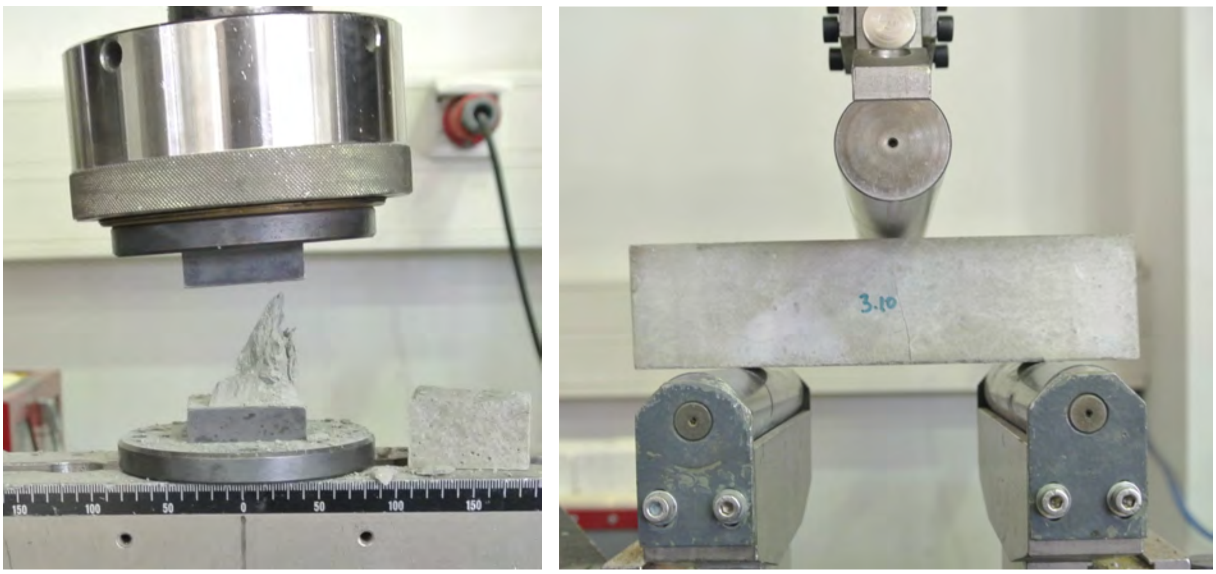


Fig. 3.2: Explosive failure mechanism during the compression strength test of grout 1 and very subtle failure mechanism during the flexural tensile strength test of grout 3.

A specified testing of the grout that is chosen to be implemented for girders out of thin-walled elements in any major building project in the future is recommended. The exact mixing ration should be set according to the desired flow consistency and viscosity before the tests are conducted.

3.1.3 Flow consistency and viscosity

The flow consistency and viscosity of the mortars (Müller et al., 2012), which are of particular importance when it comes to a high-quality execution of the joints between the thin-walled elements, were determined using flow table tests according to ÖNORM EN 1015-3 (2010) and v-funnel tests for mortar according to ÖNORM EN 12350-9 (2010). The consistency and viscosity can be altered if the water-dry material ratio is adjusted, therefore it must be noted that all results only apply for the mixing ration of 12%.

The v-funnel test showed that the flow consistency varied widely, with grout 3 clearing the funnel in 4.16 sec, grout 1, 2 and 5 needing 7.00 sec, 8.50 sec and 26.23 sec respectively and grout 4 not clearing the funnel at all. The flow table test ranked the flow consistency from a good flow to no flow as follows: grout 1, grout 2, grout 3, grout 5, grout 4.

3.1.4 Temperature behavior, expansion and shrinkage

Using the 160 mm high conical shaped specimens it was possible to asses the expansion and temperature development during the hardening process as well as the shrinkage over a longer period of time. After the conical specimens were filled temperature sensors were placed inside. The first 17 days the expansion and subsequent shrinkage of the grouts were measured using lasers in a environment with uniform temperature and humidity.

The first ten days the specimens stayed in their molds, which due to their conical shape did not restrict any expansion or shrinkage. After the ten days the specimens were removed from their molds and placed under the lasers for an additional week. A manual measurement of the shrinkage based on ÖNORM EN 12617-4 (2002) and ONR 23303 (2010) continued in regular intervals over the following 160 days.

The expansions of all grouts, apart from grout 4, were not able to compensate the later shrinkage of the specimens. The obtained data explains the large measured compressions in the joints during the large scale tests conducted by Wimmer (2016). If post-tensioning is applied 500 hours post joint-filling most of the shrinkage has occurred, if post-tensioning is scheduled before, further shrinkage in the joints must be taken in account for any calculations of the stressing strength.

3.1.5 Results

The results of all grout tests showed that most of the properties and characteristics stated by the manufacturers could not be met using the chosen water-dry material ratio of 12%. Only one of the tested grouts was able to exceed the promised compression strength and two of the grouts the promised flexural tensile strengths. Due to the high compression strength of the tested materials most of the failures were very sudden and explosive. All grouts were stated as shrinkage compensating expanding grouts. The expanding characteristics were confirmed, but the expansion were not always able to compensate the shrinkage. The entire experimental data and graphical visualizations of all tests are compiled in Appendix A - Detailed results of experimental preliminary tests of grout materials.

For the case of the implementation of the tested materials in girders out of thin-walled precast concrete elements the applied grout material should be exactly specified and tested again prior to application. Additionally the exact mixing ration should be set according to the desired flow consistency and viscosity. The flow consistency and viscosity should be checked regularly during the construction process to ensure a high-quality execution of all joints.

3.2 Redesign of joints

The joint design used in all large scale experiments by Wimmer (2016) is shown in Figure 3.3. The shuttering formwork was connected to both sides of the thin-walled elements before the joints were filled with an expanding grouting material. The joints including the chamfers, that exist on all sides of the precast elements ensuring a clean finish of the edges, were filled resulting in a ragged and rough looking execution of the outer surface of the bridge girders.

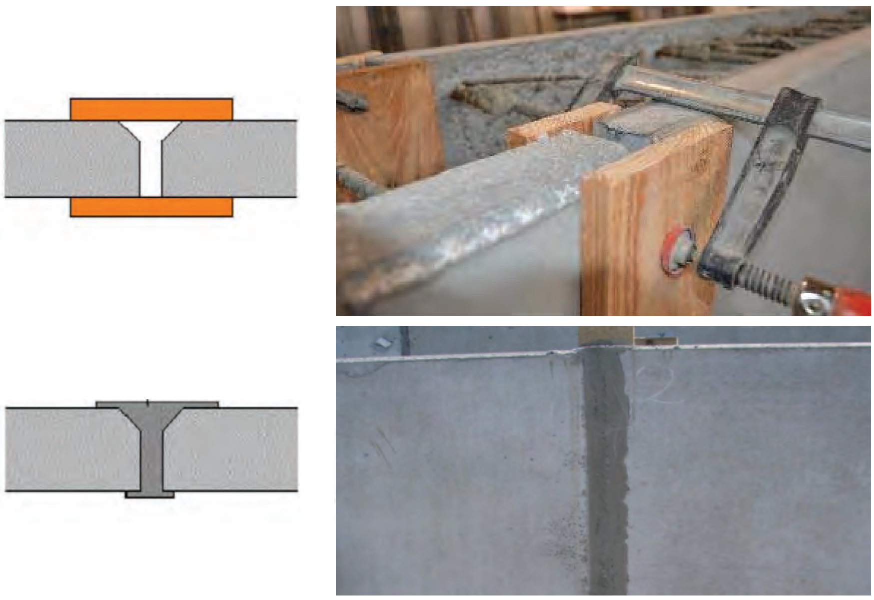


Fig. 3.3: Joints between thin-walled precast concrete elements according to Wimmer (2016)

In the course of further studies and deliberations it was decided to revise the existing joint designs. Additional to the joints capable of transmitting strains for thin-walled connections using post-tensioning, designs for unstressed reinforced joint connections were considered.

3.2.1 Joints with post-tensioning

Wimmer (2016) showed that the joints between the thin-walled elements were capable of transmitting all applied strains during the post-tensioning, solemnly a higher deformation in the joints had to be accounted for due to shrinkage of the implemented filling material. With the effectiveness and functionality of the joints already accomplished the set goal of the redesign was the improvement of the visual appearance. During the deliberations, the question was raised whether the grout filling between the chamfers (10 mm chamfers with a 45° angle) contributed in any way to the structural behavior of the joints.

It was decided to investigate if it would be possible to omit the filling of the aforementioned area without weakening the cross-section of the joints. For this purpose, large-scale tests were conducted in which prefabricated double wall element specimens were connected by joints that were cast using joint inlays, as shown in Figure 3.4, and subsequently tested for compression failure. The joint inlay not only created a clean finished joint appearance but also simplified the inspection of the joint width of 20 mm during filling.

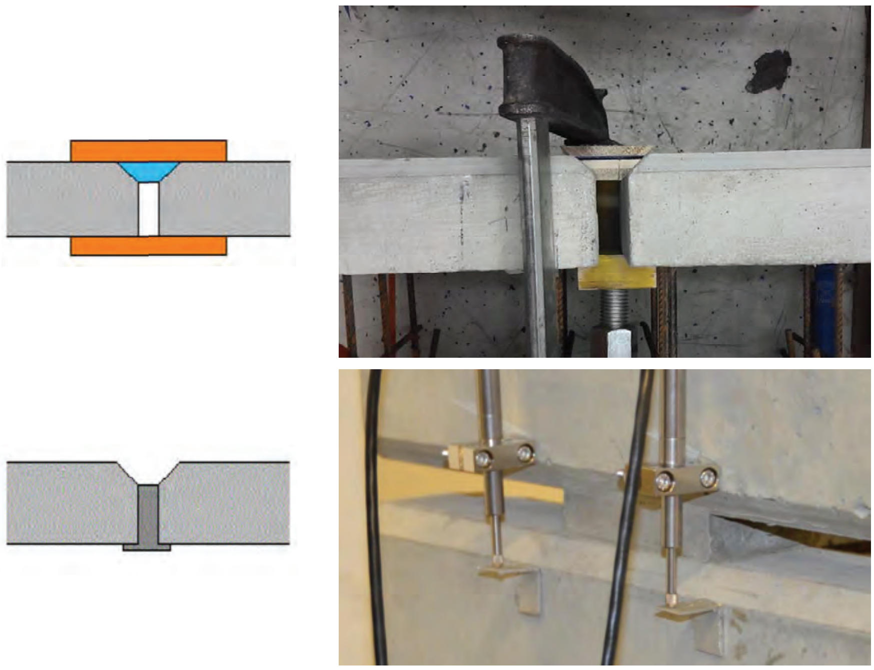


Fig. 3.4: Joint design with joint inlay placed between the chamfers

In order to make more precise statements regarding the structural behavior of the now narrower (not 70 mm but 60 mm) joint, three specimens where produced and tested. The test setup is shown in Figure 3.5 and Figure 3.6. The test specimens consisted of two double wall elements with a length of 400 mm, a height of 200 mm and a width of 300 mm which were connected together by the newly developed joints. The walls of the double wall elements had a standard width of 70 mm with a chamfer of 10 mm on each side.

The joints were filled over a length of 100 mm with a thickness of 20 mm using grout 5 (see Table 3.1 for full details of the material parameter). Due to the high compression strengths of the grouts the experiment would not have been conductible if the entire length of the specimens would have been filled. The joints were tested 24 hours post filling to ensure realistic test parameters. An uniaxial load application was guaranteed by placing load distribution blocks and elastomer plates between the testing machine and the specimens. Four linear variable differential transformers were used to measure the compression of the filling material.

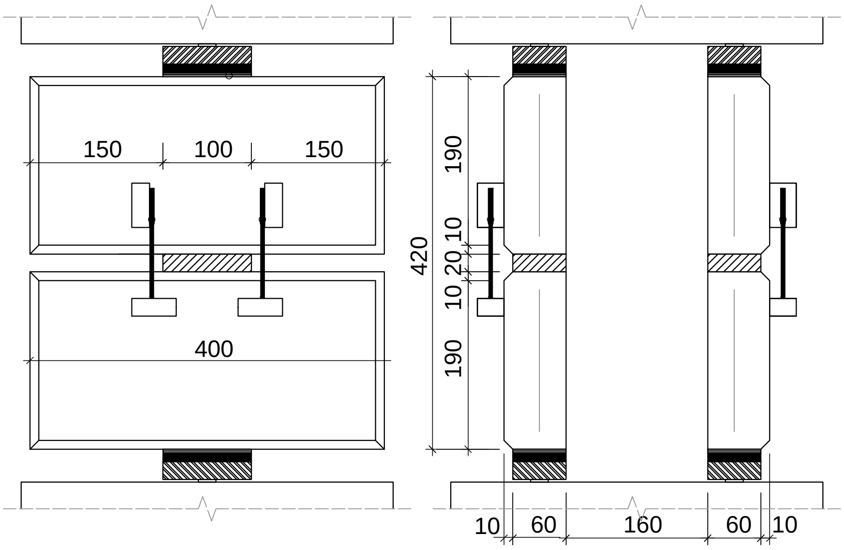


Fig. 3.5: Test setup for the joint compression testing in front view and side view, all measurements in [mm]

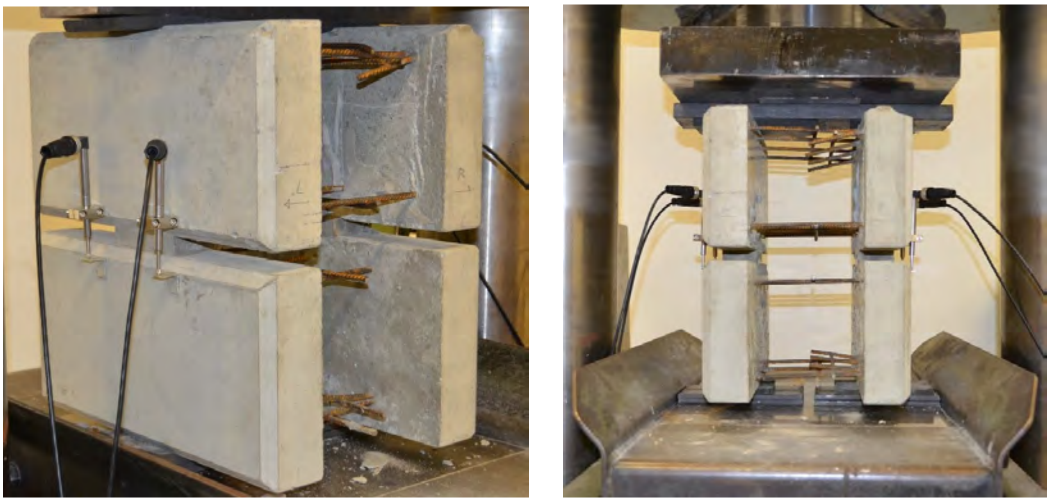


Fig. 3.6: Double wall element specimens connected by a grout joint placed in the testing machine and equipped with four linear variable differential transformers

The load was applied uniformly during testing with a load increase of 2400 ± 200 N/s according to ÖNORM EN 196-1 (2005). The compression strength of the filling material (grout 5) was simultaneously tested on standardized specimens according to ÖNORM EN 196-1 (2005) in order to compare the results to the data from the production data sheets as well as the previously conducted testing described in Chapter 3.1.2. The obtained values corresponded with the data from Table 3.2. No failure of the joint filling material was determined despite high strains of up to 40 MPa. The cracks in the double wall elements, that can be seen in Figure 3.7, indicate a failure of the double wall elements themselves instead of the grout, which can be explained by the fact that the precast double wall elements were cast using C25/30 concrete and therefore not capable of withstanding the high loads.

Fig. 3.7: Double wall specimens after joint compression tests using the newly designed joints

The tests on the double wall element specimens with the newly designed joints show that due to the high compression strength of the joint filling material, even after only 24 hours, the narrowing of the joint cross-section does not affect its structural behavior. It was also shown that a clean execution of the joints can be obtained when using joint inlays in the area of the chamfers.

Of all the tested grouts from Chapter 3.1 the one with the lowest compressive strength after 24 hours was chosen for the experiments. A considerable load increase can be achieved using a material with better parameters for the case of the application of double wall elements with a higher concrete strength class.

3.2.2 Joints without post-tensioning

If girders out of thin-walled elements without post-tensioning are requested by a contractor it must be taken in account that the implementation comes with certain restrictions. The maximal producible girder-spans are limited to the maximal production lengths of the double wall elements or precast slab elements. If larger spans are required individual girders must be placed on temporary supports on the construction site before the girders can be joined together and subsequently filled with in-situ concrete.

A joint design for the case of the construction without subsequent post-tensioning, based on the standard joint design described in Furche et al. (2009), can be seen in Figure 3.8. With the joint being the weak spot when it comes to the exposure of reinforcement to carbonation and corrosion the concrete cover for the reinforcement placed over the joint should be upheld. The joint, in contrary to the joints with post-tensioning, are not filled with grout but can be filled directly with the in-situ concrete during the filling of the lightweight thin-walled girders.

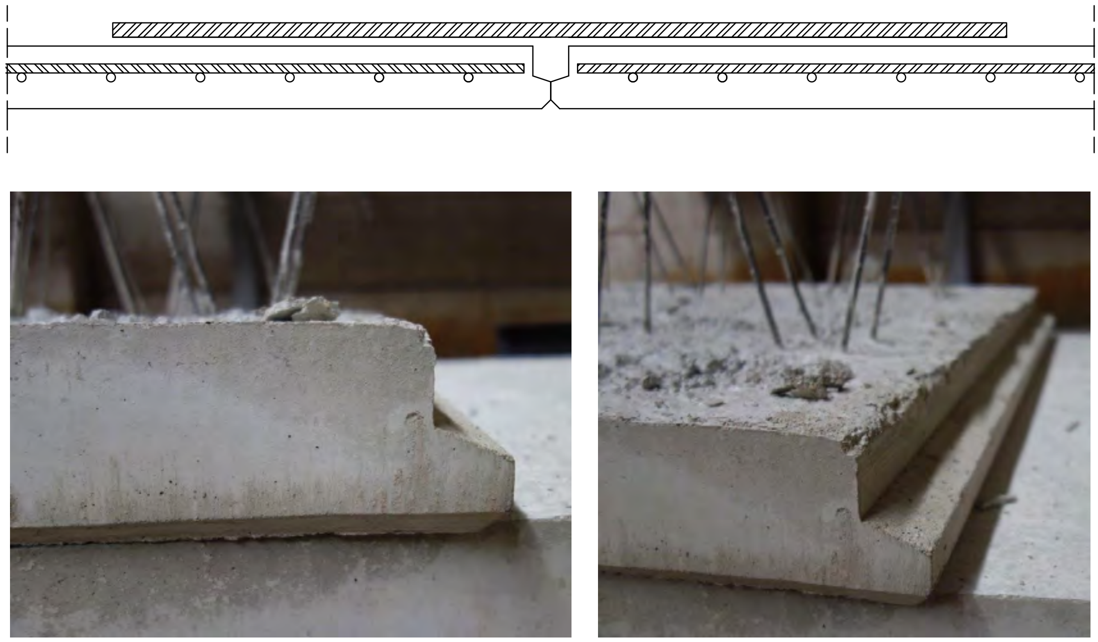


Fig. 3.8: Redesigned joint without post-tensioning and produced prefabricated elements with the new joint design

In order to test the feasibility of the designed joint without post tensioning, several test specimens, as shown in Figure 3.8, were produced. The specific form of the precast element edges was easily achieved by using regular wooden form boards. Post hardening and removal from the formwork two elements were placed next to each other, the required additional reinforcement was positioned over the joint and the elements were connected together creating a test specimen which would later be used for the assessment of fatigue behavior conducted by K. Fuchs and Kollegger (2016). A version of the joint design without post-tensioning is described by K. Fuchs, Gaßner, et al. (2017b) with two separate width reductions expanding over a longer part of the slightly thicker precast elements (120 mm) as shown in Figure 3.9.

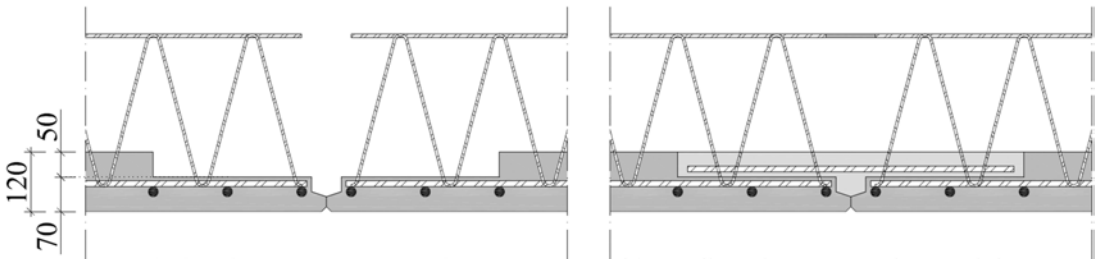


Fig. 3.9: Joint design according to K. Fuchs, Gaßner, et al. (2017b)

3.3 Considerations

The experimental preliminary tests of grout materials included standardized and non-standardized testing determining the material parameters and comparing these to the ones stated by the material manufacturers. The results of the experiments show that not only the selection of the right grouting material is of utmost importance but that by varying the water-dry material ratio a filling material consistency perfectly suitable for the filling of the joints between thin-walled precast elements can be obtained. Before implementing the grout in any major building projects further preliminary tests should be conducted in order to set the exact water-dry material ratio for the chosen grout.

The results of the shrinkage assessment showed that even though the grouts were classified as expanding grouts the expansion was not able to compensate the subsequent shrinkage. This would have to be taken into account for any calculations of the exerted post-tensioning force during construction.

In addition to the grout testing new joint designs for joints with and without post-tensioning were developed, produced and tested. The visual appearance of the joint design with post-tensioning according to Wimmer (2016) was improved without reducing any of its structural strength, therefore creating a cleaner finished version of the already accomplished joint execution. This joint design was used for a bridge designed using lightweight thin-walled bridge girders for a tender offer described in Chapter 7. In the case of the joint design without post-tensioning the feasibility was demonstrated. An example of application for this kind of joint structure will be shown in Chapter 5.

4 Experimental tests of fatigue behavior of thin-walled precast elements

When designing bridge constructions material fatigue, or in other words the weakening, damage or failure of structural elements as a result of time-varying, repeatedly applied loads, has to be considered. The nominal maximum stress values causing such damage may be much less than the ultimate tensile stress limit of the materials. In order for thin-walled precast elements with plant manufactured tack welded mesh reinforcement to even be considered for bridge construction the fatigue behavior of these elements had to be investigated.

A total of eight fatigue tests on precast and then filled double wall elements were carried out. Six girders with two different reinforcement configurations were tested with maximum stresses σ_{max} of 434 MPa and 300 MPa and stress ranges $\Delta\sigma$ between 100 MPa and 280 MPa. The goal of the fatigue tests was to prove that tack welded reinforcement, as is standardly used in precast elements, can withstand $2 \cdot 10^6$ load cycles and therefore pass the requirements for the implementation in bridge construction stated by valid standards as for example the Eurocode EN 1992-2 (2005) and ÖNORM B 1992-1-1 (2018). After the fatigue tests, two of the previously examined girders were subjected to static tests in order to asses their load-bearing-capacity after fatigue loading.

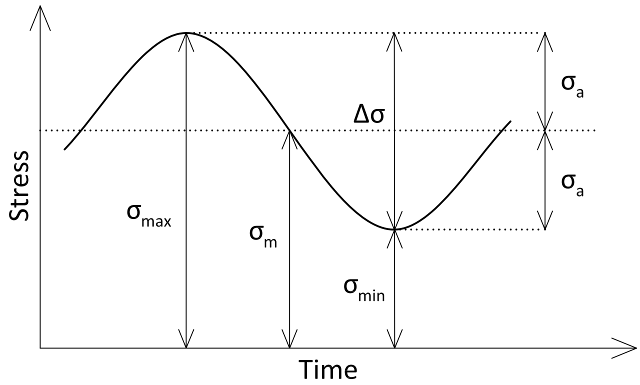
4.1 Fatigue behavior and failure

Fatigue failure begins with the forming of microscopic cracks which, after reaching a critical size, suddenly propagate resulting in the failure of the entire structure. Fatigue damage is caused by a simultaneous occurrence of cyclic stress which initiates the crack, tensile stress which promotes the crack growth, and plastic strain. If any of those three is not present, a fatigue crack will not initiate and propagate (ASM (1990)).

The fatigue life of a structure subjected to repeated cyclic loadings is defined as the number of stress cycles N it can stand before failure. The fatigue strength can be influenced by four main parameters: the stress range $\Delta\sigma$ (described in Figure 4.1), the structural detail geometry, the material characteristics and the environment (Geißler (2014) and Radaj et al. (2007)).

The first fatigue strength tests were conducted by August Wöhler, who experimentally investigated fatigue fractures of railway axles. The Wöhler curves, also known as the S-N curves, characterize the fatigue behavior of materials by evaluating a large number of sinusoidal stress tests resulting in a graph of magnitude of cyclic stress S against the logarithmic scale of cycles to failure N.

The progressions of the Wöhler curves during which the cyclic stress S decreases with the number of cycles N is divided, as shown in Figure 4.2, into three different areas. The low-cycle fatigue, characterized by a small number of cycles ($N < 10^5$) and high loads, causes plastic and elastic deformation. The high-cycle or finite life fatigue, characterized by a large number of cycles ($N > 10^5$) and low loads, causes elastic deformation. The stress amplitude below which the material



σ_{max}	maximum stress
σ_{min}	minimum stress
$\Delta\sigma$	stress range (σ_{max} - σ_{min})
σ_m	mean stress $((\sigma_{max} + \sigma_{min})/2)$
σ_a	stress amplitude $(\Delta\sigma/2)$
R	stress ration $(\sigma_{min}/\sigma_{max})$
A	amplitude ratio (σ_a/σ_m)

Fig. 4.1: Stress cycles (sin wave) during fatigue loading and terms of fatigue (Yapparina (2013))

never fails, no matter how large the number of cycles, is called the endurance limit and lies between N = 10^6 and N = 10^7. Since the endurance limit is only theoretically, valid standards speak of very-high-cycle or infinite life fatigue and a limit of $2 \cdot 10^6$ cycles (Radaj et al. (2007)). An infinite fatigue life on an asymptotic S-N curve is presumed beyond 10^7 cycles.

The varying load distribution over time caused by traffic, waves and wind complicates the calculation of fatigue resistance of engineering structures. Damage accumulation hypotheses are used to adjust test results of fatigue testing with a constant amplitude to realistic conditions in order to enable a viable portrayal of structural behavior and determine service life of built structures.

The Palmgren-Miner linear damage hypothesis is one of the simplest and most widely used cumulative damage models. It states that if there are k different stress levels and the average number of cycles to failure at the i^{th} stress (S_i) is N_i, then the damage fraction D, with failure occurring at D = 1, can be calculated as the sum of all the cycles accumulated at the various stresses divided by the average number of cycles to failure ($D = \sum n_i/N_i$) (Radaj et al. (2007)).

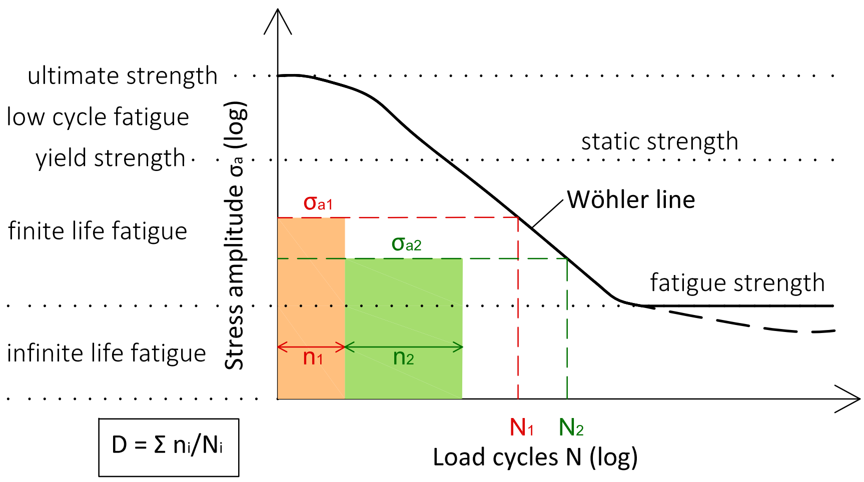


Fig. 4.2: Wöhler curve also known as the S-N curve and visualisation of the Palmgren-Miner linear damage hypothesis (CAE Simulation Solutions (2015))

4.1.1 Fatigue behavior of concrete

Repeated, cyclic loading can lead to fatigue damage and fatigue failure in reinforced concrete structures. The failure of the reinforced concrete structure can either be caused by concrete or steel failure. A failure of the compressive concrete zone, in contrast to steel failure that comes without any advanced notice, is indicated by an increase in deformation and crack propagation.

The fatigue behavior of concrete depends on the type of loading, maximum and minimum stress levels (σ_{min} and σ_{max}), stress amplitude σ_a, speed of load application, sequence in which high and low stresses are applied, recovery phases, initial loading, load eccentricity, moisture level, density and if the concrete class f_{ck} is higher than 60 MPa (Fehlmann (2012)).

Fatigue behavior of concrete can be divided, as shown in Figure 4.3 and described in detail by Schläfli (1999), into three stages. The first stage represents about 10% of a structures service life and is characterized by a strong increase in irreversible deformations with a proportional decrease of the modulus of elasticity. The deformation increases steadily during the second stage until about 80% to 90% of the service life has been reached. The last stage is characterized by an unstable increase in deformation and subsequent failure.

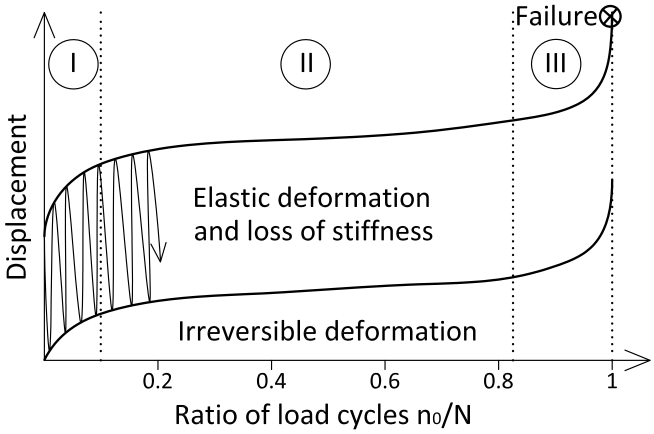


Fig. 4.3: Three stages of fatigue behavior of concrete (Schläfli (1999))

4.1.2 Fatigue behavior of reinforcement bars

The fatigue behavior of steel is very different to that of concrete. Microscopic cracks in the reinforcement bars can already be present at low levels of stress where the strain appears to be elastic. The cracks propagate with a stable crack growth until the remaining un-cracked cross section becomes too weak to carry the load and a sudden fracture occurs. A schematic representation and a photograph of a fatigue failure surface showing the initiation region, the propagation of the fatigue crack and the rapid fracture area of the final failure are shown in Figure 4.4. The failure happens without an advanced notice since, in contrast to fatigue failure of concrete, no increased deformations nor large cracks can be observed (ASM (1990) and Zilch et al. (2008)).

In order to determine the fatigue resistance of reinforcement bars, fatigue tests can either be performed on small rebar specimens or entire construction elements where the rebars are cast in concrete. If only the rebar is tested the failure occurs at the weakest point of the rod (various

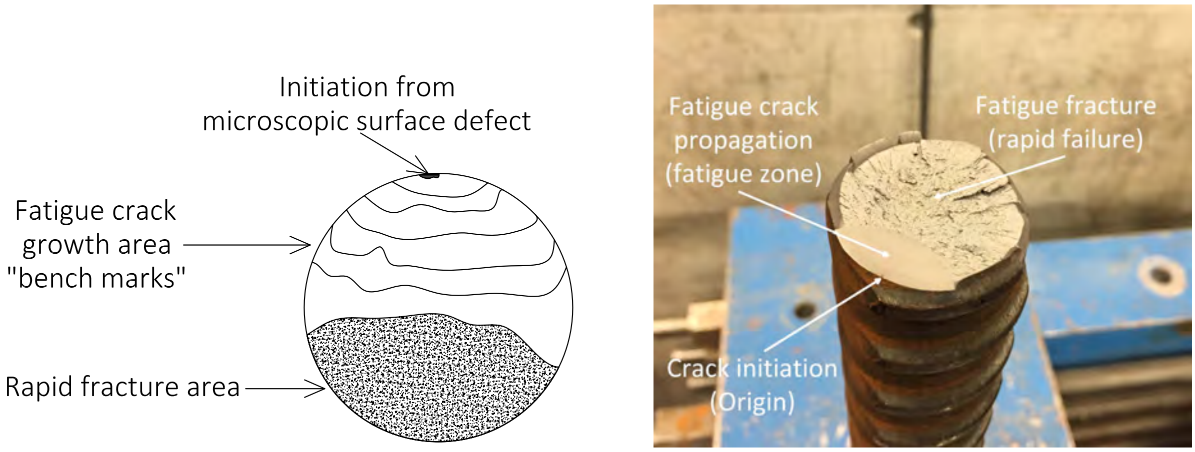


Fig. 4.4: Schematic representation and photograph of a fatigue fracture surface showing the initiation region, propagation of the fatigue crack and rapid fracture area of the final failure

surface conditions, welding points, etcetera). If the rebars are tested embedded in concrete a higher fatigue resistance can be expected even though the reduction of fatigue stength due to friction stress causesd by the slipp between rebar and concrete must be taken in account. This higher fatigue resistance can be explained by the low probability of a crack propagating exactly at the weakest point of the reinforcement bar. According to Zilch et al. (2008) the following parameters influence the fatigue behavior of reinforcing steel: loading cycle, rebar diameter, rebar curvature, corrosion, welding points and mechanical connections.

Experiments conducted by Bathias (1999) show that fatigue failure can still occur in a large number of metallic alloys beyond 10^7 cylces, meaning that a infinite fatigue life on an asymptotic S-N curve, as shown in Figure 4.5, and stated by the standards is theoretically not realistic.

4.1.3 Basic design criteria of the fatigue testing of thin-walled precast elements

Reinforced concrete is a composite material in which the high compression strength of concrete is combined with the ductility and tensile strength of reinforcement. Depending on the extent of tensile stresses in the reinforced concrete element standards as the Eurocode EN 1992-1-1 (2004) differentiate between the un-cracked and cracked state for any kind of calculations. In an un-cracked state the concrete is able to take up all tensile stresses, whereas in the cracked state the entire tensile stresses are borne up by the steel. For the fatigue testing of the precast double wall elements it was important for the specimens to be in a cracked state in order to obtain the fatigue resistance of the embedded tack welded mesh reinforcement. If the girders would have stayed in an un-cracked state no actual fatigue testing of the reinforcement would have been possible.

The setup of the conducted fatigue tests, as explained extensively by Huber (2011), can be simply described as a single mass oscillator with one degree of freedom. In order to determine the eigenfrequency and dynamic deformation of the tested single span beam the Ritz method using an equivalent mass and stiffness must be implemented. Additionally, the dynamic load caused by the unbalanced exciter must be calculated as a sinusoidal distributed knife-edge load for the evaluation of the data to be realistic.

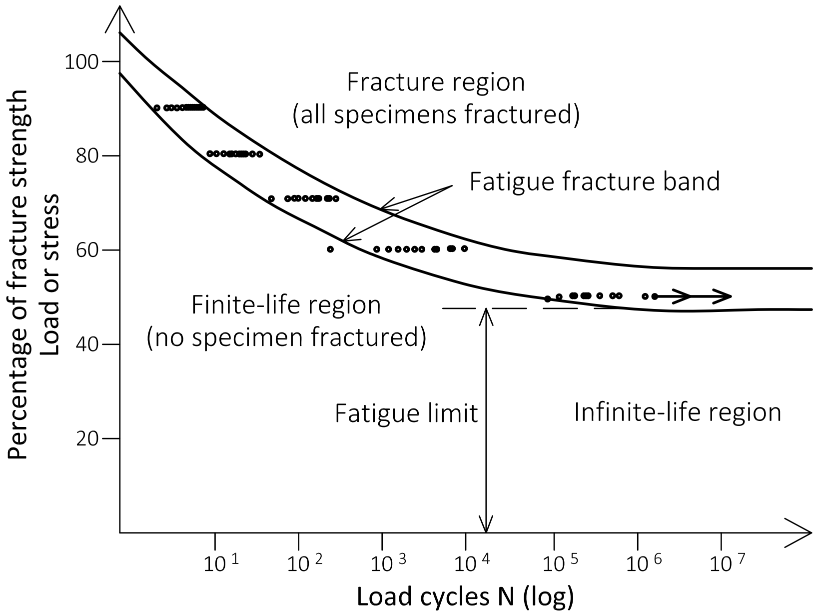


Fig. 4.5: Wöhler curves also known as the S-N curves that typify fatigue test results when testing medium-strength steels (ASM (1985))

4.1.4 Construction rules according to Eurocode 2

The general reinforcement and detailing rules for concrete bridge structures are stated by the Eurocode EN 1992-2 (2005). When using welded mesh reinforcement made of ribbed steel, the minimum diameter is set to 8 mm. In addition, the application is limited to areas with a maximum stress, due to unfavorable characteristic traffic loads, of 80 MPa. The installation of welded reinforcement is forbidden in bridges with traffic categories 1 and 2 as well as railway bridges. Tack welds, except in the case of installation reinforcement that is not used for any load distribution, are inadmissible if there is any dynamic load.

The implementation and the assessment of the fatigue behavior of welded and tack welded mesh reinforcement is stated in ÖNORM B 1992-1-1 (2018) and ÖNORM B 4707 (2014). The welded connection, due to the geometric and metallurgical changes in the material caused by the welding process, corresponds to a geometric notch, which reduces the fatigue strength of the two connected reinforcement bars extensively. ÖNORM B 1992-1-1 (2018) separates the existing welding processes in processes that are allowed to be used for predominantly static and non-static loads. According to this standard the application of crossed tack weld connections (Kreuzungsstoß Heftverbindung (HS-KV)), which are used in manufacturing plants for the production of mesh reinforcement, is permitted for elements subjected to dynamic loading.

The maximum stress σ_{max} for the fatigue testing of reinforcement bars is set to 300 MPa according to ÖNORM B 4707 (2014). This standard also states that the stress ranges $\Delta\sigma$ for $2 \cdot 10^6$ load cycles is dependent on the diameter of the tested rebars and lies between 100 MPa and 150 MPa and can be reduced to 100 MPa for welded reinforcement. The Wöhler line for welded rebars and mesh reinforcement, with a set stress range $\Delta\sigma_{Rsk}$ of 85 MPa at 10^6 load cycles, can be found in ÖNORM B 1992-1-1 (2018).

4.2 Test specimens for conducted fatigue tests

Six girders with two different reinforcement configurations, as shown in Figure 4.6, were produced in an Austrian prefabrication plant specialized in the manufacturing of double wall elements. All six specimens had a length of 5.0 m, a width of 0.80 m and a height of 0.40 m. The two panels of the double wall elements, which were reinforced using tack welded mesh reinforcement, had a thickness of 70 mm leaving an intermediate space of 260 mm for the filling concrete. KAP-steel-waves (KAPPEMA (2011)) were intentionally chosen as separation reinforcement for the double wall elements in order to rule out any possible influence on the load capacity of the mesh reinforcement as would be the case if lattice-girders were installed.

The anticipated results and the main goal of the fatigue tests was the demonstration that tack welded mesh reinforcement could resist fatigue loading and therefore be used in bridge construction. If this could be proven, girders out of thin-walled precast elements could be produced in the standard automated production process without any kind of adjustments, therefore upholding the high efficiency and profitability of the product. The replacement of the tack welded with manually constructed mesh reinforcement, as was the case for all bridge girders tested by Wimmer (2016), comes with a noticeable increase in production costs.

4.2.1 Material parameters

The double wall panels of the test specimens were made, according to data received from the manufacturing plant, using a concrete with the concrete class C50/60. The filling concrete was produced with a concrete class of C30/37. The actual compressive cube strength of both concrete types was determined briefly before the fatigue tests commenced using twelve standardized concrete cubes (150 mm by 150 mm by 150 mm). Six cubes were stored dry and six in water. The tests were conducted according to ONR 23303 (2010) with a load application speed of 1.50 MPa/s. The measured compressive cube strength exceeded the concrete classes stated by the manufacturer, with the required strength being 38.0 MPa for C30/37 and 58.0 MPa for C50/60 and the results lying between 57.0 MPa and 70.60 MPa.

The mesh reinforcement, which was produced in the manufacturing plant using ring wire with a diameter of 10 mm, had a specified steel grade of 550 MPa, a minimum tensile strength of 620 MPa and a modulus of elasticity of 210 000 MPa.

4.2.2 Mesh reinforcement configuration

The fatigue tests consisted of two testing series which differed in the amount of installed longitudinal reinforcement in the double wall element panels. It was decided to use the mesh reinforcement that was produced in-house of the manufacturing plant in order to test standard double wall elements with the typically used reinforcement. Any changes to the production process would result in the increase of production costs therefore reducing the cost efficiency of the lightweight thin-walled precast concrete girders.

The production of the mesh reinforcement in the manufacturing plant is a fully automatic process which begins with the feeding of the measurement data of the elements into the production systems network. The rods for the mesh reinforcement are unwound from the coils, straightened and cut into the lengths needed for the vertical and horizontal application. Subsequently a

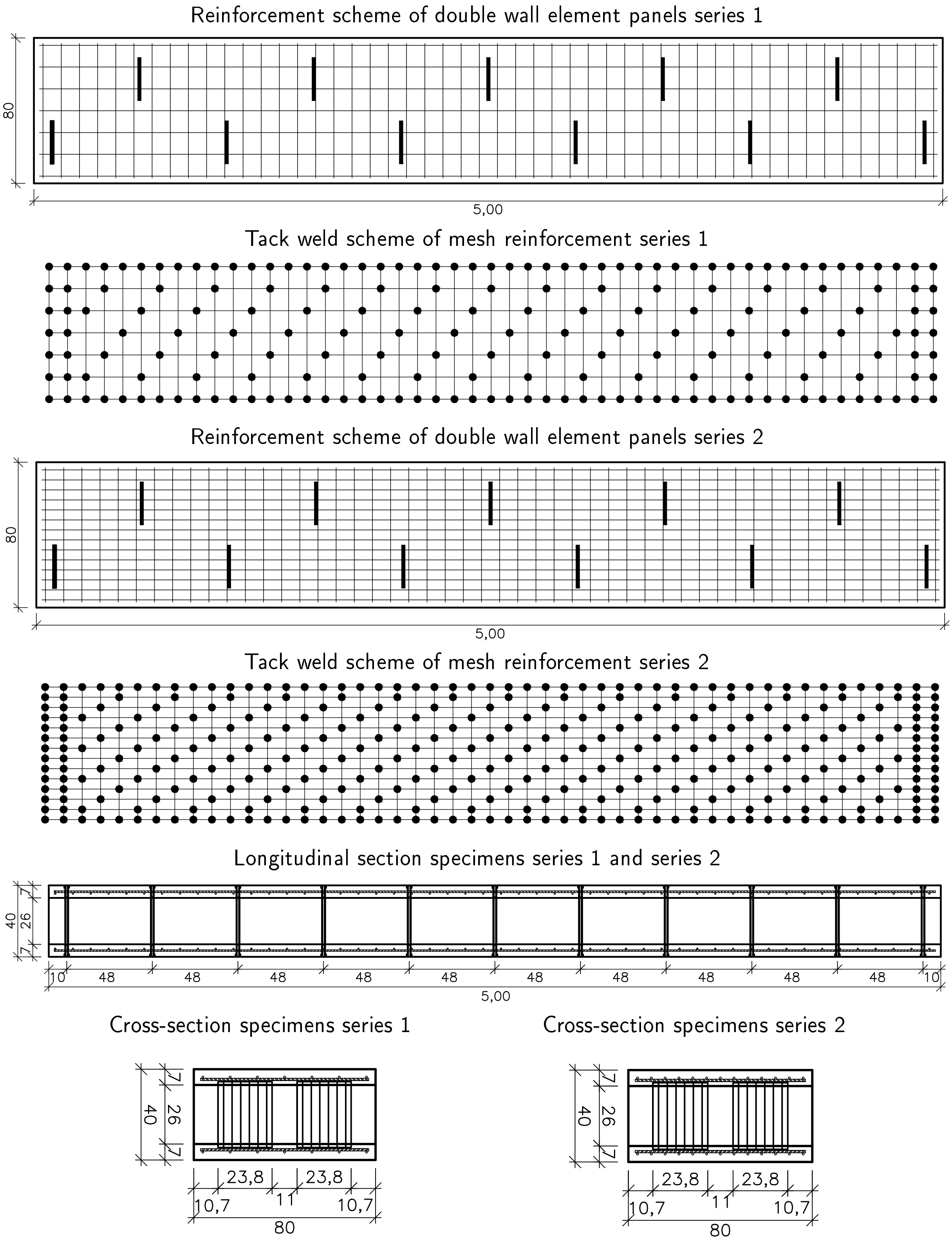


Fig. 4.6: Reinforcement and tack weld schemes for specimens of series 1 and series 2. The tack weld scheme were generated automatically. Eleven KAP-steel-waves were used as connecting elements between the double wall panels in each test girder.

series of lengths of horizontal and vertical reinforcement bars are tack welded together in a grid formation. The required tack welding points are usually generated automatically by the machine, but in special cases a manual input is possible. All mesh reinforcements of the six test specimens were manufactured using the automatic generation. The tack weld is only a temporary, non-load-bearing connection, used for securing the position of the reinforcement bars in the mesh during the manufacturing process.

The mesh reinforcement installed in the four girders of series 1 consisted of Ø10/120 mm as longitudinal reinforcement and Ø10/100 mm as transverse reinforcement. The amount of longitudinal reinforcement (7 rods of Ø10) of series 1 was doubled for the two girders of series 2 resulting in Ø10/55 mm. The amount of transverse reinforcement remained the same for both series. The reinforcement and tack welding schemes of both test series as well as the placement of the separation reinforcement (KAP-steel-waves) is shown in Figure 4.6.

4.2.3 Test specimen production

The double wall elements used for the fatigue testing were produced according to the usual production protocol in a fully automatic manufacturing process in a plant in Lower Austria. After the measurements of the desired elements were fed into the manufacturing system the production commenced. Formwork panels were placed automatically by shuttering robots, the exact placement was monitored using lasers before the panels were magnetically fixed to the mobile casting tables.

With the formwork in place the casting tables were moved below the mesh reinforcement production machine. Workers placed plastic separators on the casting tables, ensuring adequate concrete cover, before the mesh reinforcement dropped onto the casting table. Once the separating reinforcement (KAP-steel-waves), fixating the distance between two element panels, was installed as shown in Figure 4.7, the casting tables were moved to the concrete spreader. Finally, the concrete spreader filled the casting tables with concrete before it was compacted and stored for hardening.

Fig. 4.7: Mesh reinforcement and KAP-steel-waves in place on the formwork for the first panel of the double wall element and the merger of the hardened first double wall panel with the fresh concrete of the second panel.

After about eight hours the second panel of the double wall elements was produced. The production process was almost exactly the same as for the first panel with the only difference being that no separation reinforcement was placed on the casting tables. Once the concrete was poured the first panel element with the separation reinforcement was turned up side down in order to be placed in the fresh concrete of the second panel. With the two walls merged together, as shown in Figure 4.7, the concrete of the second panel was compacted, the distance between the two walls checked and thereafter the entire element stored for hardening. Last of all the double wall elements were filled with in-situ concrete before being transported to the testing laboratory.

4.3 Testing of fatigue behavior

A total of eight fatigue tests, on six precast and then filled double wall elements with two different reinforcement configurations, were carried out. The entire experimental procedure was divided into the following steps:

1. Delivery of test specimen to the testing site.
2. Reset of all load cells to zero.
3. Placement of the test girder on the test facility and installation of the test setup.
4. Connection of all measuring instruments to the computer and the specimen. Testing of the installed measuring instruments and setting all values to zero. The deflection due to dead load and load caused by the test setup was not taken into account during the tests. The deflection was calculated separately and accounted to 0.513 mm.
5. Choice of maximum stress σ_{max} and stress range $\Delta\sigma$ for the test. Calculation of mean stress and resulting preload.
6. Application of the preload (maximum stress σ_{max}) to initiate crack formation in the concrete ($M_{Rk} > M_{crack}$) resulting in a cracked state of the concrete girder. This step was not executed for the first fatigue test.
7. Application of the mean load F_m to prevent the lift-off of the girder during dynamic loading.
8. Commencement of the oscillation process. Cautious approach up to the desired frequency (test frequency equals the eigenfrequency of the test girder).
9. With the desired frequency fixated and the right amount of dynamic load applied the testing phase begins. Commencement of all measurements.
10. Readjustment of the equipment of the counter-rotating fly-wheels of the unbalanced exciter or re-tightening of the threaded rods in the case of a sudden drop in the eigenfrequency of the girder in combination with an increase in deflection and a decrease of the applicable force by the unbalanced exciter.
11. Fatigue testing up to $2 \cdot 10^6$ cycles or failure. A premature test termination was instigated once the eigenfrequency dropped beneath a certain value or the deflection suddenly increased in a non-proportional manor or once it was no longer possible to apply the desired dynamic load without continuous readjusting the equipment of the counter-rotating fly-wheels of the unbalanced exciter.
12. Dismantling of test setup. Selective inspection of reinforcement. Selective retesting of reverse side of girders in further fatigue testing. Selective testing of structural behavior.

All fatigue tests were carried out in the same manor, with the only difference lying in the choice of the applied stress range $\Delta\sigma$ and maximum stress σ_{max}. The maximum stress σ_{max} of the first test series was set to 434 MPa, corresponding to 70% of the maximum tensile strength of the reinforcing steel, and was later adjusted to 300 MPa, corresponding to the requirements stated in ÖNORM B 4707 (2014), for the second test series. The applied stress ranges $\Delta\sigma$ lied between 100 MPa and 280 MPa. The chosen stress ranges $\Delta\sigma$ and corresponding maximum stresses σ_{max} of the tests are listed in Table 4.1. Furthermore the data from Table 4.1 shows which specimens belonged to which test series and which were tested for their fatigue behavior twice.

Tab. 4.1: Stress ranges and maximum stresses selected for the fatigue testing of double wall element girders

Test 1-8	**Specimen TS 1-6**	**Series 1 or 2**	**max. stress σ_{max} [MPa]**	**Stress range $\Delta\sigma$ [MPa]**
Test 1	TS 1			280
Test 2	TS 2			180
Test 3	TS 3	Series 1	434	180
Test 4	TS 4			140
Test 5				100
Test 6	TS 5			180
Test 7		Series 2	300	140
Test 8	TS 6			100

The mesh reinforcement of two girders was inspected after the fatigue testing in order to check for any fatigue failure. Two girders, test specimen TS4 and test specimen TS5, were subjected to a second round of fatigue testing after being turned up side down. The testing of the static load-bearing capacity and deflection of the interface between the prefabricated panels and the cast in-situ concrete was conducted on the two girders of the test series 2, one of them being a double fatigue test specimen.

4.3.1 Fatigue test setup

All tested beams were subjected to 3-point bending using two threaded rods and a unbalance exciter as shown in Figure 4.8. The tests were conducted on a spring-loaded concrete trough which served as a clamping field and damper for the generated vibrations.

The test girders were simply supported on three self-centering pendulum supports, which rested on steel I-girders positioned on the concrete trough with a support spacing of 4.80 m. Two threaded rods were used to connect a steel beam, which was placed at mid-span and served as a support for the unbalance exciter, with the concrete trough. Additionally the rods were used to apply supplementary static loading for the precracking of the test beams and the loading to achieve the desired mean stress during the experiments.

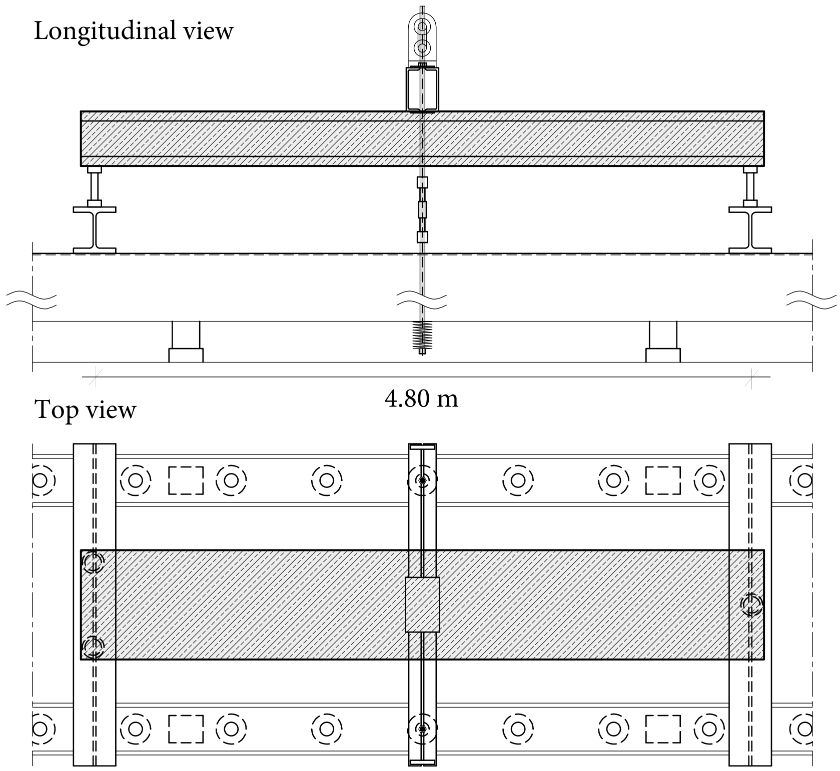


Fig. 4.8: Test setup in longitudinal and top view

4.3.2 Instrumentation and measuring concept

The excitation frequency produced by the unbalanced exciter, which was placed mid-span on a steel beam, was aligned with the eigenfrequency of the tested beams achieving a resonance effect. Due to this effect, described in detail by Köberl et al. (2008), the working stroke was achieved with relatively little expenditure of energy and the duration of the experiment was reduced considerably.

By varying the equipment of the two counter-rotating fly-wheels of the unbalanced exciter using iron and aluminum weights the imbalance mass was arbitrarily changed. According to Köberl (2008) a load of up to 4 000 N can be applied by the unbalanced exciter in radial direction and a centrifugal force can be set between 10 Hz and 75 Hz. The main control unit that was utilized for handling the unbalanced exciter was a MicroMaster. A manual readjustment of the frequency was occasionally needed. A supervisor was always present during the entire test phase in case of sudden changes in the eigenfrequency or abrupt failure of the girders.

The applied measuring concept, shown in Figure 4.9, allowed the assessment of the vertical displacement in various points as well as the deflections in mid-span, the applied forces, as well as the concrete deformations and strains. The three load cells of the type RTN100, named K1 through K3 (marked red in Figure 4.9) were part of the self-centering pendulum supports. The measurements made by the load cells were verified by the outputs from the strain gauges which were placed between the connection of the threaded rods and are symbolized Z1 and Z2 and marked green in Figure 4.9 with only Z1 showing due to the restricted front view of the image and Z2 being positioned behind the test beam.

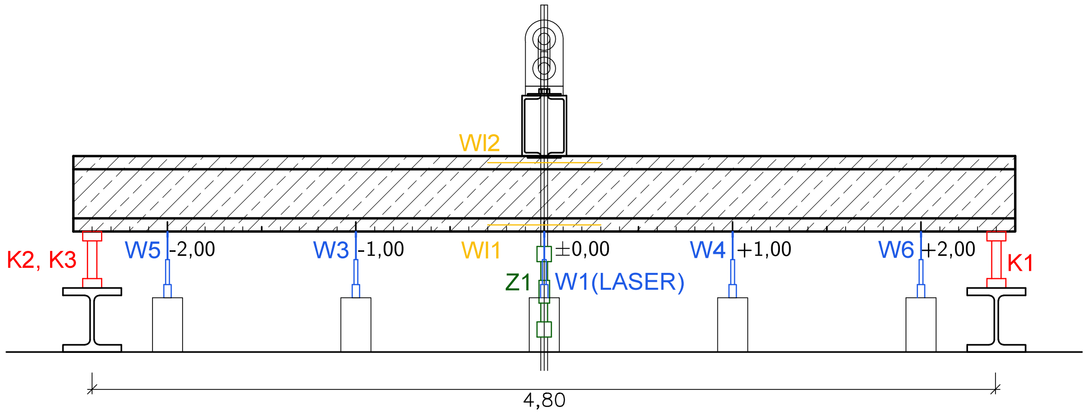


Fig. 4.9: Front view of the measuring concept during the fatigue tests

The measurements of the vertical displacements were conducted 1.0 m and 2.0 m from mid-span using a set of linear variable differential transformers of the type Solatron BS25, featured in Figure 4.9 in blue and named W3 through W6. Two lasers placed in mid-span, named W1 and W2 and marked blue in Figure 4.9 with only W1 showing due to the restricted front view of the image and W2 being positioned behind W1, assessed the deflection of the beam in mid-span. Finally, four additional linear variable differential transformers, named WI1 through WI4 and marked yellow in Figure 4.9, with only WI1 and WI2 showing due to the restricted view of the image and WI3 and WI4 being positioned on the back side of the test beam, were used to investigate the strains and deformations of the concrete in mid-span.

In addition to the continuous electronic measurements manual observations were carried out. The crack formation and crack width were documented after the termination of the fatigue tests before the mean load and test setup was removed from the girders.

4.3.3 Analytical analysis

In order to determine the eigenfrequency of the girders and the magnitude of the adequate load needed for the various stress ranges $\Delta\sigma$ of the fatigue tests, a precise analytical analysis of the experiments and the individual girders had to be conducted. With the specimen being in a cracked state during the fatigue tests the corresponding section properties for both an un-cracked and a cracked state had to be calculated.

The moment load capacities, the shear resistances, the moments of intertia and the eigenfrequencies of both test series can be found in Table 4.2. With the eigenfrequency of the actual test specimens falling somewhere in between the calculated eigenfrequencies (un-cracked and cracked state) a test duration of approximately 30 hours for $2 \cdot 10^6$ cycles was calculated.

The difference in the concrete compression strength of the double wall elements and the cast in-situ concrete were factored into all the considerations. The partial safety coefficient for all calculations was set to 1.

The girder dead load of 8.0 kN/m and the test setup load of approximately 65 kN had to be factored into the calculations of the mean loads (additional load F_m in order to obtain the desired mean stress σ_m) as well as the working strokes of the dynamic loading for all eight experiments.

Tab. 4.2: Section properties of the test girders of both test series in a cracked and un-cracked state

			Series 1	Series 2
Moment load capacity	M_R	[kNm]	107.70	213.10
Shear resistance	V_R	[kN]	191.95	241.84
Moment of inertia un-cracked state	I_I	[cm^4]	429 482.5	440 113.0
Moment of inertia cracked state	I_{II}	[cm^4]	35 351.30	65 189.6
Eigenfrequency un-cracked state	$f_{0(I)}$	[Hz]	24.35	24.35
Eigenfrequency cracked state	$f_{0(II)}$	[Hz]	7.01	9.52

With a varying tested stress range $\Delta\sigma$ and two different maximum stresses σ_{max} the mean stress σ_m and all subordinated loads had to be calculated separately for each experiment. The additional load F_m needed to obtain the mean stress σ_m as well as the applied dynamic load ΔF are listed for each fatigue test in Table 4.3.

Tab. 4.3: Additional load F_m needed to obtain mean stress σ_m and applied dynamic load ΔF for fatigue tests 1 through 8

Test 1-8	**Mean stress σ_m** $\sigma_{max} - (\Delta\sigma)/2$ **[MPa]**	**Additional load F_m needed for σ_m [kN]**	**Applied dynamic load ΔF [kN]**
Test 1	434 - 280/2 = 294	20.64	61.73
Test 2	434 - 180/2 = 344	28.49	39.69
Test 3	434 - 180/2 = 344	28.49	39.69
Test 4	434 - 140/2 = 364	31.64	30.87
Test 5	434 - 100/2 = 384	34.78	22.05
Test 6	300 - 180/2 = 210	39.23	77.92
Test 7	300 - 140/2 = 230	45.41	60.60
Test 8	300 - 100/2 = 200	51.58	43.29

4.3.4 Fatigue test results

A total of eight fatigue tests on precast and subsequently filled double wall elements were carried out. Six girders with two different reinforcement configurations were tested with maximum stresses σ_{max} of 434 MPa and 300 MPa and stress ranges $\Delta\sigma$ between 100 MPa and 280 MPa. The goal of the fatigue tests was to prove that tack welded reinforcement, as is normally used in precast elements, can withstand $2 \cdot 10^6$ load cycles and therefore pass any requirements stated by valid standards.

The results of all eight experiments are plotted into the Wöhler curve according to ÖNORM B 1992-1-1 (2018) in Figure 4.10. The comparison of the results to the Wöhler curve for welded reinforcement bars and mesh reinforcement show that the the achieved amount of load cycles N for the tested stress ranges $\Delta\sigma$ lie significantly above the stated standards. A shift of the curve from the set stress range $\Delta\sigma_{Rsk}$ of 85 MPa at 10^6 load cycles according to ÖNORM B 1992-1-1 (2018) to the the stress range $\Delta\sigma$ of 100 MPa for $2 \cdot 10^6$ load cycles according to ÖNORM B 4707 (2014) is easily possible.

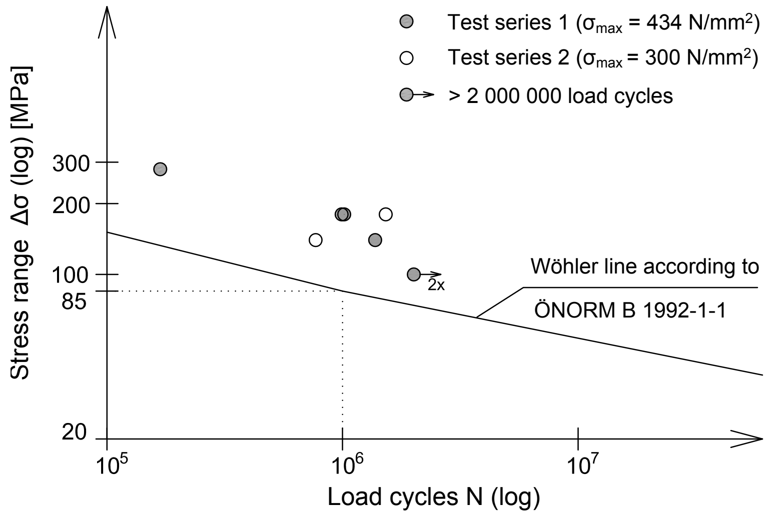


Fig. 4.10: Fatigue test results plotted into the Wöhler curve for welded reinforcement bars and mesh reinforcement according to ÖNORM B 1992-1-1 (2018)

In the course of the experiments, several problems arose concerning the testing procedure and the girder design. Various enhancements were implemented as soon as the cause for the existing problems was determined and a solution found.

The difficulties in the fixation of the desired amplitude σ_a and the applied amount of dynamic loading ΔF during the first fatigue test were resolved by preloading all the subsequent test girders to the point of the chosen maximum stress σ_{max}. By doing so a cracked state of the test specimens was achieved resulting in a stable and harmonic oscillation of the entire experimental setup and girder during the fatigue tests. Once the desired preload was reached the load was decreased back to the additional load F_m, which was needed to obtain the calculated mean stress σ_m in the mesh reinforcement during the experiments.

Figure 4.11 shows the amount of applied dynamic loading ΔF in the course of all eight experiments. The difficulties in the fixation of the load during fatigue test 1 are clearly visible. The bigger fluctuations that can be observed in the curves of fatigue test 7 and fatigue test 2 are due to crack propagation in the concrete of the girders and the gradual failure of the reinforcement.

The increase of the reinforcement ratio and decrease in the chosen maximum stress σ_{max} for the second test series can not only be traced back to the expansion of the variety of testing parameters but also to the low load-bearing-capacity due to the choice of too little reinforcement in the girders of the first test series. With the dead load and the load from the test setup lowering the load-bearing-capacity extensively it was difficult to be certain if the reinforcement did not

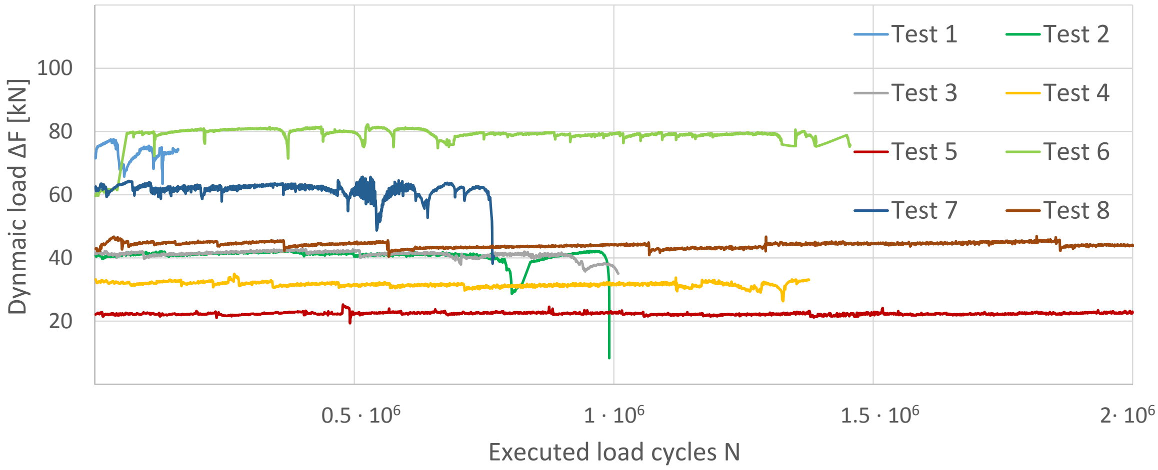


Fig. 4.11: Applied dynamic load ΔF calculated in Table 4.3 as a function of the executed load cycles for all fatigue tests

reach its yield point during the testing. After the first test series it was decided to conduct further experiments on a second test series with a higher reinforcement ratio resulting in the production of two additional specimen with the double amount of longitudinal reinforcement.

The severe problematic of bond failure between the precast elements and the in-situ concrete became prominent during the second test series. A clearly visible crack along the interface between the precast panels of the double wall elements and the cast in-situ concrete as well as the phenomenon of cracks propagating well spread in the double wall panels, stopping at the interface and restarting cumulated into one to three large cracks at a different position in the in-situ concrete indicated existing bond issues in the interface. During some experiments only a single wide crack propagated in the in-situ concrete while multiple cracks propagated simultaneously in the precast panels. The differing crack distribution in the double wall element panels and the in-situ concrete is shown in Figure 4.12.

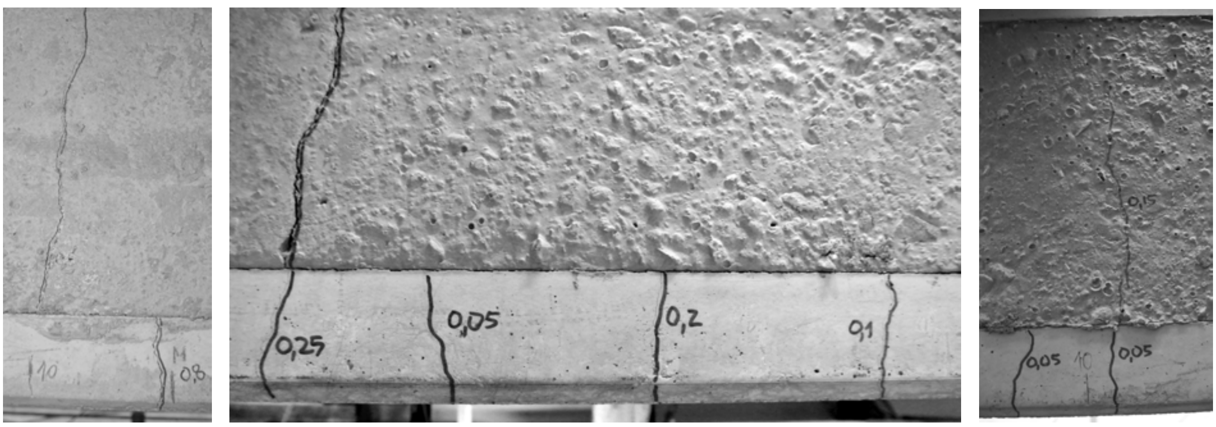


Fig. 4.12: Differing crack distribution in double wall element panels and the in-situ concrete

Due to the noticed bond failure and in order to determine the remaining load-bearing capacity of the tested girders of the second series, two static bending tests on test specimens TS5 and TS6 were carried out once the fatigue tests on both girders were terminated. Detailed data on the test setup and measuring concept, which were both based on the fatigue tests, as well as the results are outlined in the following subsection.

The results of the fatigue tests consisting of the executed load cycles, test frequency and deflection are shown in Figure 4.13 and supplemented by the crack widths in the panels, the in-situ concrete and the interface in Table 4.4.

Tab. 4.4: Fatigue behavior of each test specimen throughout the fatigue testing

<table>
<tr><th rowspan="2">Fatigue Test 1-8 (Stress range $\Delta\sigma$ [MPa])</th><th rowspan="2">Test Specimen TS 1-6</th><th rowspan="2">Executed load cycles (start-middle-end)</th><th colspan="2">Frequency</th><th rowspan="2">Deflection mid-span [mm]</th><th colspan="3">Crack width [mm]</th></tr>
<tr><th>[Hz]</th><th>Drops</th><th>Panels</th><th>In-situ</th><th>Interface</th></tr>
<tr><td>Test 1 (280)</td><td>TS 1</td><td>4 654
100 000
168 296</td><td>15.5
14.5
13.8</td><td>3</td><td>5.2
6.0
6.4</td><td>0.6</td><td>1.5</td><td>top</td></tr>
<tr><td>Test 2 (180)</td><td>TS 2</td><td>1 198
500 000
991 292</td><td>20.0
19.5
15.5</td><td>1</td><td>2.2
2.5
9.3</td><td>3.0</td><td>3.0</td><td>top</td></tr>
<tr><td>Test 3 (180)</td><td>TS 3</td><td>1 312
500 000
1 016 892</td><td>20.9
20.5
15.5</td><td>2</td><td>2.7
2.9
5.1</td><td>1.0</td><td>1.0</td><td>both</td></tr>
<tr><td>Test 4 (140)</td><td rowspan="2">TS 4</td><td>1 256
500 000
1 376 131</td><td>21.0
20.5
18.4</td><td>1</td><td>1.6
2.1
4.1</td><td>0.8</td><td>0.8</td><td>both</td></tr>
<tr><td>Test 5 (100)</td><td>1 256
1 000 000
2 000 000</td><td>20.7
20.5
20.4</td><td>0</td><td>2.3
2.7
2.8</td><td>0.5</td><td>0.5</td><td>both</td></tr>
<tr><td>Test 6 (180)</td><td rowspan="2">TS 5</td><td>1 123
750 000
1 525 704</td><td>17.8
16.8
16.1</td><td>3</td><td>2.8
3.8
5.5</td><td>0.7</td><td>0.7</td><td>bottom</td></tr>
<tr><td>Test 7 (140)</td><td>986
500 000
789 931</td><td>16.4
15.5
14.4</td><td>3</td><td>6.8
8.4
10.5</td><td>0.2</td><td>0.35</td><td>bottom</td></tr>
<tr><td>Test 8 (100)</td><td>TS 6</td><td>1 287
1 000 000
2 000 000</td><td>21.4
21.0
20.9</td><td>0</td><td>2.2
2.6
2.6</td><td>0.15</td><td>0.15</td><td>bottom</td></tr>
</table>

Table 4.4 lists the testing frequency and deflection of the girders in mid-span for the beginning, approximately the middle and the end of the eight experiments. The numbers in bold represent the final load cycle count, final frequency and final deflection of each fatigue test specimen. Despite the problems due to the high chosen maximum stress σ_{max} and additionally due to bond failure in the interface two of the six specimens reached the exposure of 2 $\cdot 10^6$ load cycles.

With the termination of each fatigue test the crack patterns and crack widths, which had resulted from the experiment, were recorded. The largest crack widths in the precast panels of the double wall elements and the cast in-situ concrete are listed in the last column of Table 4.4. Furthermore the occurrence of a crack in the interface between the precast panels and the cast in-situ concrete, differentiating between the two interfaces (top and bottom panel-in-situ-interface), is stated.

The number of drastic drops in the eigenfrequencies for each fatigue test are additionally listed in Table 4.4. The cause of the drops was traced back to fatigue breaks of the individual reinforcement bars. It is not clear to say if with every drop only one fatigue fracture in the mesh reinforcement occurred or if, as it could be expected during the failure of test specimen TS2, the frequency drop and specimen failure was the result of a break of multiple bars.

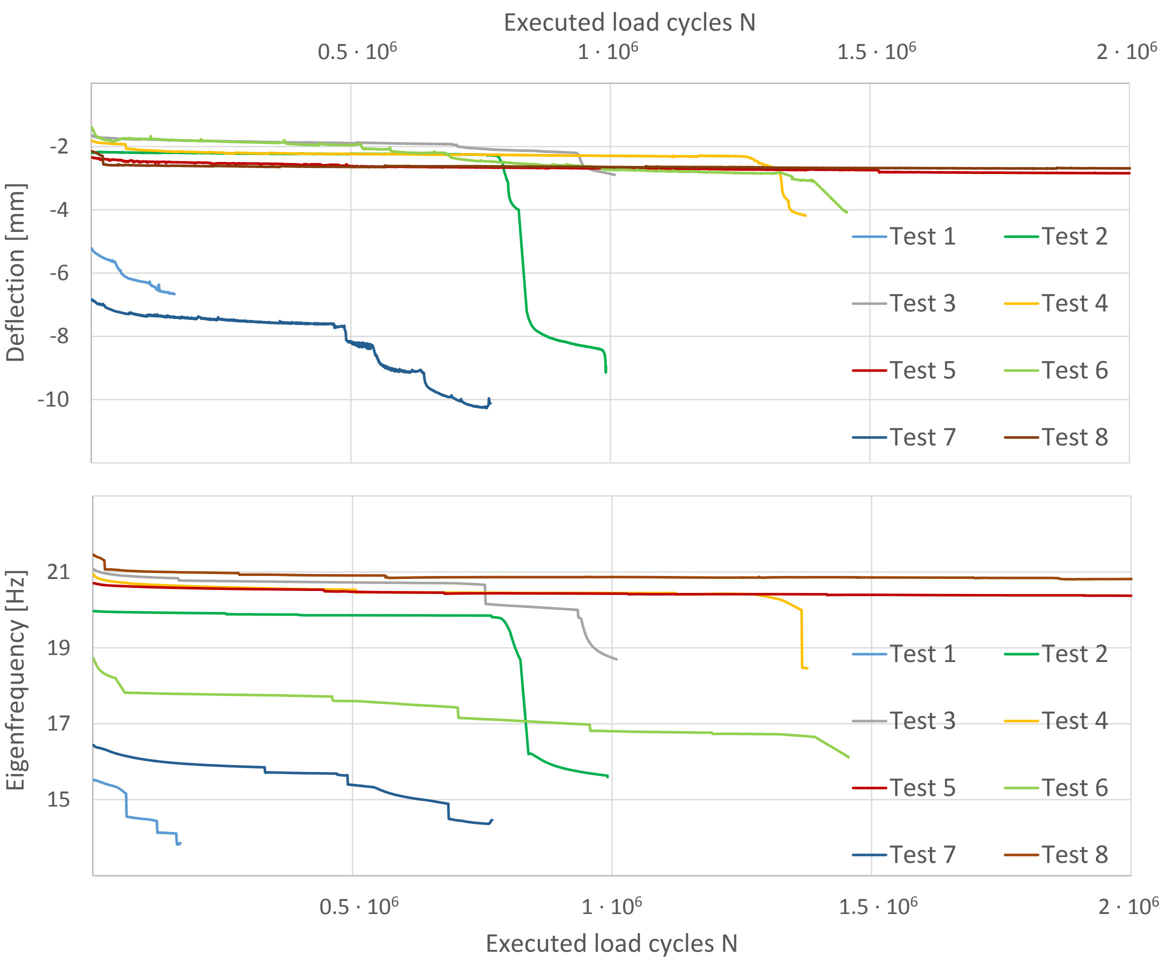


Fig. 4.13: Progression of deflection and progression of the eigenfrequencies as functions of the executed load cycles for all fatigue tests

The progression of the eigenfrequency as a function of the executed load cycles and the deflection as a function of the executed load cycles are shown in Figure 4.13. The drastic drops in the eigenfrequencies that are clearly visible in the diagram were traced back to the fatigue failure of individual reinforcement bars at the tack welded intersections of the mesh reinforcement. With one or more reinforcement bars breaking the eigenfrequency of the entire test girder dropped up to 4.50 Hz as was the case during the second fatigue test. When comparing the eigenfrequency-load cycle-diagram with the deflection-load cycle-diagram it can be observed that every drop in the eigenfrequency came hand in hand with a sudden increase in deflection.

The deflection progression in mid-span of the girders during six of the eight experiments show good correlation with beginning deflections between 1.60 mm and 2.80 mm. The concordance of values was also noticeable for the eigenfrequencies of most of the test girders. The large deflections of test specimen TS1 of fatigue test 1 were caused by the difficulties in setting the desired frequency in the beginning of the experiment. The very irregular and partially very rapid dynamic load application caused irreversible damage to the test girder resulting in fatigue failure after barely 168 296 load cycles. The very low eigenfrequency of the test specimen confirms this theory. The large deflection of test specimen TS5 which was already present in the beginning of fatigue test 7 can be explained by the previous exposure of the girder to over $1.5 \cdot 10^6$ load cycles with a stress range $\Delta\sigma$ of 180 MPa in fatigue test 6. The eigenfrequency of test specimen TS5 during fatigue test 6 was noticeably lower than the frequencies of the other girders and decreased even more during fatigue test 7.

By comparing the results of the fatigue tests of the six girders the impact of the various testing parameters, which included the two maximum stresses σ_{max}, the two reinforcement ratios, the four different stress ranges $\Delta\sigma$ and the fact that two of the six girders were tested on both sides, can be assessed. The general condition of the test girders before and during the fatigue tests could be well determined by observing the eigenfrequencies. If the frequency fell below 18 Hz it could be assumed that irreversible damage to the mesh reinforcement had occurred. For this reason the results of fatigue test 1 and fatigue test 7 as well as the results of fatigue test 6 must be considered with caution even though they passed the requirements stated by ÖNORM B 1992-1-1 (2018).

The results of the measurements of the the strain and deformations of the concrete in mid-span showed no irregularities. It can be said with certainty that the amount of longitudinal reinforcement had no impact on the fatigue behavior of the test elements.

After the fatigue tests the state of the reinforcement of two specimens was investigated. Fatigue breaks of the reinforcement next to the welded intersections were found. A fatigue fracture in the mesh reinforcement at a tack weld is shown for the case of fatigue test 3 (test specimen TS3) in Figure 4.14. The reinforcement was exposed at the largest crack in the double wall element panels. Three positions were chosen for the analysis of the reinforcement state. Of the twelve intersections positioned in the cracks path five were tack welded. Of these five a fatigue break was detected in three intersection right at the tack weld.

Additional experimental data and graphical visualizations of the individual fatigue tests are compiled in Appendix B - Detailed results of experimental tests of fatigue behavior of thin-walled precast elements.

Fig. 4.14: Mesh reinforcement inspection after finished fatigue tests 3 (TS3). Fatigue fracture in mesh reinforcement at tack weld.

4.4 Static testing of structural behavior of girders weakened by fatigue testing

Two static bending tests were carried out on two of the six previously fatigue tested girders. The aim of the experiments was to obtain the remaining load-bearing capacity in order to draw conclusions on the structural behavior of the interface between the precast elements and the in-situ concrete. The test setup of the static tests was based on the test setup of the fatigue tests differing only in the manner of load application. The unbalanced exciter was removed from the steel girder in mid-span and replaced by two hydraulic jacks.

The instrumentation and measuring concept from the fatigue tests was adjusted to be more suitable for the investigation of the interface and is shown for both tests in Figure 4.15. It was decided to only measure the deflection of the girders using linear variable differential transformers in mid-span (W2 and W3) and in close proximity of the supports (W1 and W4). The remaining linear variable differential transformers used during the fatigue tests were moved and rededicated during the first static tests for the measurement of the displacement in the interface (WI1 through WI8). For the second static test the linear variable differential transformers were omitted and replaced by a 3D optical deformation analysis using an ARAMIS system.

The girders that were submitted to static testing after previous fatigue testing were test specimens TS5 and TS6. In the case of test specimen TS5 the static test was the third experiment conducted after fatigue test 6 and fatigue test 7 with the side of test 7 being submitted to the further loading. Test specimen TS6 on the other hand had only been tested once with a very low stress range during fatigue test 8 and had endured $2 \cdot 10^6$ load cycles without any major noticeable defects. With the reverse side to the fatigue test being tested during the static tests it could be assumed that the previous dynamic loading would not influence the load-bearing behavior at all.

The analytical analysis of the load-bearing capacity of the girders showed that from a purely mathematical point of view, with the filling concrete fully bonded to the double wall elements and the partial safety coefficient set to 1, a failure would occur at a load of 156.82 kN with a moment load capacity M_{Rk} of 213.10 kNm as stated in Table 4.2.

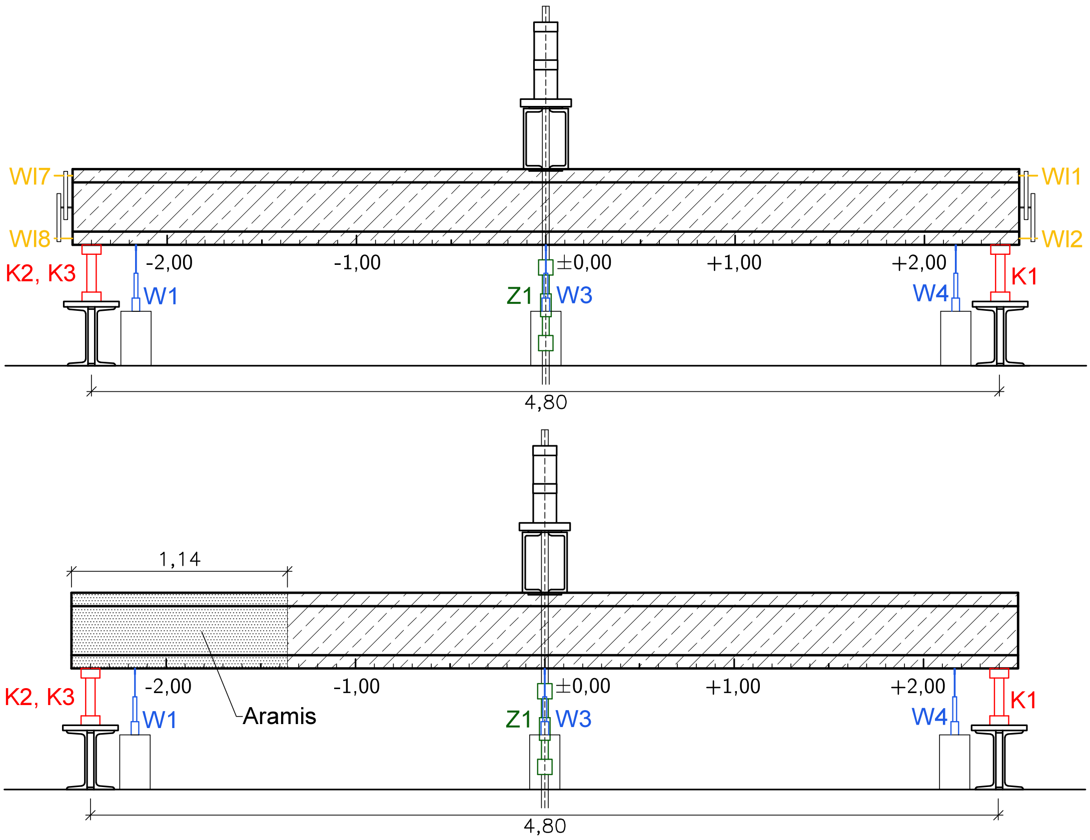


Fig. 4.15: Front view of the measuring concepts during the two static tests

The loading intervals, applied force and resulting deflection of both static tests are shown in Figure 4.16. In order to document the crack propagation throughout the experiments the raise in the applied force was held from a certain loading every 20 kN. The static tests were terminated once an increase in force was no longer possible. Due to the multiple fatigue testing test specimen TS5 was very run down reaching its load-bearing capacity at 96.70 kN with a corresponding deflection of 18.21 mm. After 2 $\cdot 10^6$ cycles with a stress range $\Delta\sigma$ of 100 MPa test specimen TS6 showed a load-bearing capacity of 170.60 kN with a corresponding deflection of 35.92 mm.

The crack propagation was logged after each loading step during the experiments. The final crack distribution with corresponding crack widths for both test specimens is shown in Figure 4.17, with the black lines being the cracks that only resulted from the fatigue tests and the red lines being the cracks which propagated or initiated during the static loading.

The crack patterns of test specimens TS5 and TS6 differ greatly from each other, especially when comparing the back view of TS5 and the front view from TS6. As can be seen distinctly, the cracks in the double wall elements of TS6 spread evenly throughout the length of the girder with a maximum crack width of 1.0 mm. The pronounced and well-advanced cracks after the two fatigue tests of TS 5 hindered the formation of new cracks in the double wall elements causing a maximum crack width of 7.0 mm on the back side of the girder. The cracks in the filling concrete cumulated into two or three larger cracks with crack widths of up to 10.0 mm in TS6 and 5.0 mm in TS5.

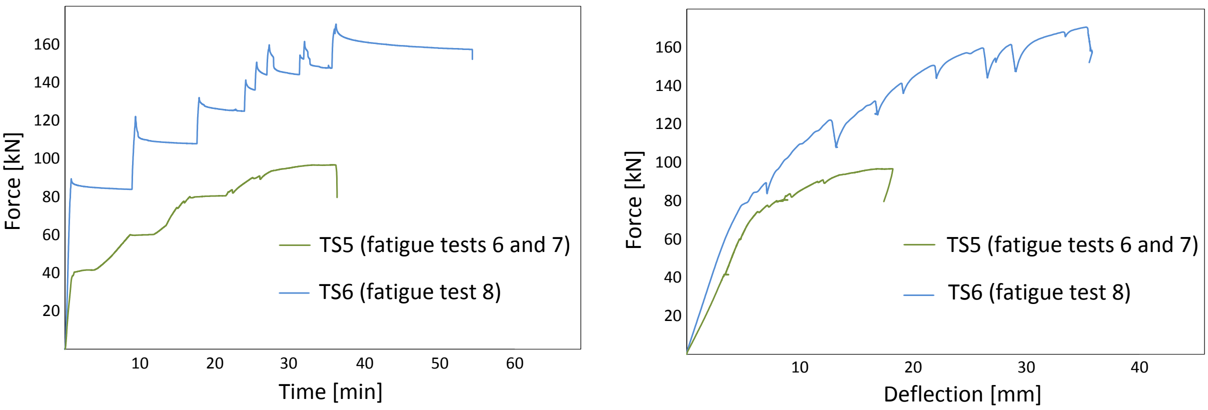


Fig. 4.16: Load-time-diagram and load-deflection-diagram of the test specimens TS5 and TS6 during the static tests

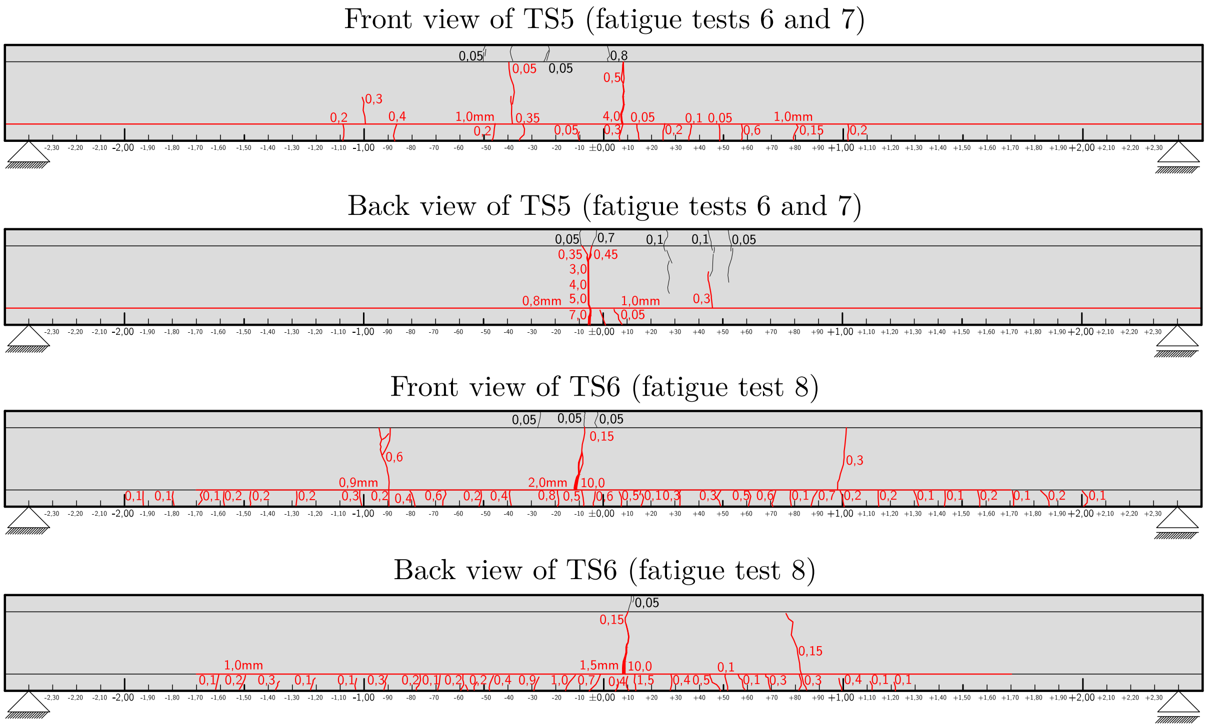


Fig. 4.17: Crack patterns of TS5 and TS6 after static testing

The irregularity in crack distribution between the pre-cast element panels and the in-situ concrete can be traced back to the detachment of the two concrete elements in the interface. The interface opened up in mid-span to a gap of 2.0 mm in TS6 and 1.0 mm in TS5. Despite the detachment of the pre-cast elements from the in-situ concrete no displacement of the interface on the edge of the test girders was measured by the linear variable differential transformers nor by the 3D optical deformation analysis.

4.5 Considerations

With the results of the eight fatigue tests and the two load-bearing tests it is thus possible to make certain statements on the overall issue of fatigue behavior of thin-walled precast elements using tack welded mesh reinforcement. The somewhat unexpected reactions of the test specimens to the cyclic loading highlighted design and production mistakes that had occurred and must be considered for any designs of girders out of thin precast elements and for any further fatigue tests on specimens made of precast elements with additional in-situ concrete.

Despite the problems the fatigue behavior of the tested specimens was better than any valid standards state (see Figure 4.10). This and the successive tests conducted by K. Fuchs and Kollegger (2016) and described in detail by Berger (2017) show that the application of tack welded reinforcement in fatigue loaded elements should be considered.

A further fact speaking for the implementation of light weight precast elements with plant manufactured tack welded mesh reinforcement, in addition to the cost efficiency and the passing of the requirements stated by the valid standards, is the aspect that if thin-walled precast girders are used in bridge construction they would normally be post-tensioned, therefore a cracked state of the girders, as was tested during the fatigue experiments, would never be the case. If the girders stay in an un-cracked state a fatigue failure of the mesh reinforcement, which is not activated, is unrealistic.

If tack welded mesh reinforcement were to be used in bridge construction it should be considered to reduce the tack weld points in the meshes to a minimum. The tack welding is solemnly needed for the reinforcement bars of the mesh to stay in place during assembly and subsequent casting of the concrete. The welding of every third connection point as was the case for the test girders and can be seen in the tack weld schemes shown in Figure 4.6 is not necessary and should be decreased by at least a half.

During the fatigue testing bond failure occurred in the interface between the prefabricated panel and the cast in-situ concrete. The crack patterns of all fatigue tests showed that the interface was not able to transfer all strains directly from the double wall panels to the in-situ concrete and a clear gap between the double wall panels and the in-situ concrete became visible during the following static tests.

The failure in the interface was traced back to two causes which, when occurring simultaneously, result in bond failure. The first failure trigger was the choice in connection elements (steel-waves and not lattice girders) which were used as connectors between the two panels of the double walls. The wave panels not only divided the in-situ concrete into segments which weekend the cross-section immensely but were also not able to create a proper connection between the three elements (two panels and in-situ concrete). A possible solution to this problem would be the installation of lattice girders instead of steel-waves in construction elements which are submitted to cyclic loading as shown in subsequent investigations of fatigue behavior conducted by K. Fuchs, Gaßner, et al. (2017a).

The second failure trigger was the very smooth and even surface structure of the precast double wall panels. The sleek surface of the interface was not able to transfer any forces due to the lack of interlock reducing the bond behavior to a minimum. This phenomenon is described by Peyerl

(2012) and Peyerl and Krispel (2013) for different cement bonded interfaces.

The varying surface structure of the precast elements was observed at a later time during the manufacturing process of double wall elements and is clearly visible in Figure 4.18. If the surface is raked after compacting a very coarse and rough surface structure can be achieve, if it is not raked and is compacted for too long the concrete slag rises to the top creating a very sleek and smooth surface. The clear solution for this problem is the declaration of a minimum roughness of the precast elements, which could be set to a peak-to-valley height of 1.50 mm (Peyerl (2016)).

Fig. 4.18: Varying surface structure of the precast elements resulting in very different bond behavior with subsequently added in-situ concrete

Despite all the occurred problems the results of the fatigue tests show that an application of tack welded mesh reinforcement should not be completely prohibited in elements that are subjected to dynamic loading. If reasonable measures are taken, as for example post-tensioning of the elements to avoid a cracked state, a risk of fatigue failure of the tack welded mesh reinforcement is minimized to a sheer minimum.

5 Theoretical and structural considerations for thin-walled precast bridge girders regarding integral bridges

Based on the research of Wimmer (2013) and Kollegger and Blail (2008) the application of thin-walled precast bridge girders in the construction of integral bridges with short and medium spans was investigated. In order to be able to evaluate the adequacy of the suitable single-span and double-span structural systems shown in Figure 5.1 a parameter study based on an existing bridge and detailed redesigns of integral bridges with short and medium spans were carried out.

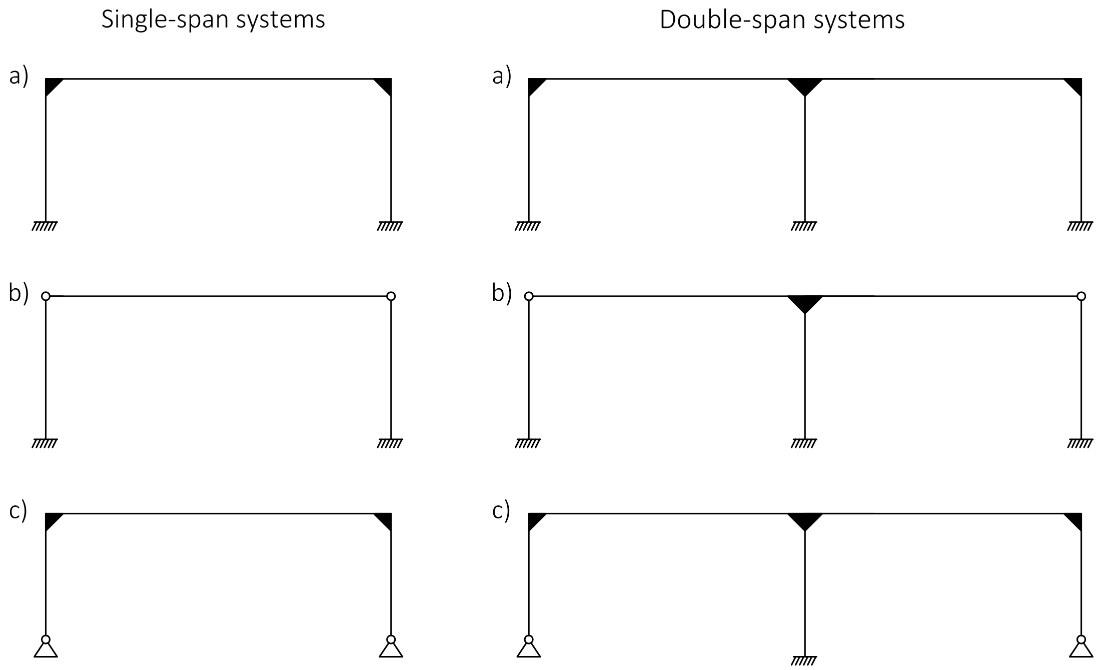


Fig. 5.1: Suitable static systems for integral bridges with short and medium spans

By designing integral concrete bridges using thin-walled precast bridge girders as alternatives to two executed steel-concrete composite bridges and a conventionally cast concrete bridge, it was possible to draw conclusions on the efficiency of the developed building method. In contrast to the parameter study, which focused solely on the application of post-tensioning with additional reinforcement, the redesigns also looked into the construction without post-tensioning and focused on more than one of the structural systems shown in Figure 5.1.

One of the main tasks during the programing of the parameter study as well as the calculation of the redesigns was the consideration and analysis of all construction phases during the erection using thin-walled precast girders. Each static system of each construction phase had to be considered and separately calculated with the effects of the applied loads on the structure members carried from a previous construction stage to the next.

The extensive numerical parameter study and the redesigning process of the existing bridges confirmed that the application of thin-walled precast bridge girders should be considered for the construction of integral bridges with small and medium field spans of up to 35 m. Solutions for the detail construction for integral bridges using thin-walled precast bridge girders with and without post-tensioning are outlined towards the end of the chapter.

5.1 Integral abutment bridges

This section will give a short overview of the term integral abutment bridge according to RVS 15.02.2.12 (2018), Mehlhorn et al. (2015), Kaufmann (2008), and Engelsmann et al. (1999) as well as the examples for the application of prefabricated elements in combination with prestressing or post-tensioning in these types of bridge structures. Furthermore three existing integral abutment bridges in Austria, which were considered to be examined more closely in order to investigate the redesign of the structures using thin-walled precast concrete bridge girders, will be presented.

5.1.1 Definition and brief summary of advantages and disadvantages of integral abutment bridges

Integral abutment bridges, which are bridges executed without bearings and without costly expansion joints in the superstructure nor any joints or roller supports between the superstructure and the abutments, are the perfect answer when it comes to the construction of bridges with short and medium spans. By completely eliminating the particularly high-maintenance construction elements a more durable bridge construction is created, in which the superstructure and substructure are monolithically connected.

With cost-efficiency being one of the decisive criteria when it comes to building projects, it can be expected that integral bridges will play an increasingly important role in future bridge construction. Unfortunately, the design and construction of integral bridges does not only have its virtues, among other things a more complicated calculation process and the complex soil-structure interaction must be taken into account.

The advantages of integral bridges can be divided into three categories: maintenance advantages, user-friendly advantages and constructive advantages. The maintenance advantages consist not only of the already mentioned minimization of costs due to the elimination of high-maintenance construction elements, but also of the increase in durability since de-icing agents and other debris cannot seep through any existing joints causing deterioration and therefore resulting in longer inspection intervals. The increase in user-friendliness can be seen in the decrease in noise pollution due to the lack of joints, which also increases the riding quality and safety.

When it comes to the constructive and design advantages of integral bridges the following points come to mind: horizontal loads that transfer directly into the surrounding, the rigid frame structure that increases the ductile reserves of the system, faster and easier construction due to elimination of bearings and joints, abutments that do not have to be accessible, and structural continuity that increases the resistance in case of seismic events and overloads.

The disadvantages of integral bridges are mainly caused by the uncertainties persisting from the design process, caused by the the large scattering of the input data and the complexity of the entire structural system. The superstructure is also often slightly more expensive than one of regular bridges due to the requirement of additional longitudinal reinforcement in the bridge deck slab. Furthermore the complex soil-structure interaction as well as the response of the entire structure to temperature changes, post-tensioning, shrinkage and creep must be meticulously evaluated and modeled for all calculations.

As can be gathered from the listed disadvantages, the design and calculations of integral bridges is more complicated than that of normal bridges, which can cause an increase in design costs. Special attention must be paid to the bridge endings, the entire construction process and the concrete casting process. The transition from the bridge structure to the open road should be well planed and executed in order to assure a smooth overpass. To reduce strains in the entire system due to temperature changes the casting of the concrete should be timed so that it takes place at the lowest possible temperatures.

When it comes to the implementation of post-tensioning in integral bridges, the application for larger spans is seen as plausible and reasonable according to W. Fuchs et al. (2007), as long as the boundary conditions allow it. The application of prestressing or post-tensioning in combination with prefabricated elements in integral abutment bridges will be described in detail in the following section.

5.1.2 Application of prefabricated elements in combination with prestressing or post-tensioning in integral abutment bridges

The application of prefabricated elements in combination with prestressing or post-tensioning in integral abutment bridges is a rather sensitive undertaking. As described by Dicleli in "A rational design approach for prestressed-concrete-girder integral bridges", integral bridges are almost always only analyzed for the final stage assuming a complete structure, therefore failing to reflect the actual behavior and effects of several loading conditions which often play a decisive role especially when it comes to the application of prefabricated elements with or without prestressing or post-tensioning. Dicleli (2000) recommends the analysis of integral bridges for each construction stage, with the effects of the applied loads on the structure members carried from a previous construction stage to the next. This calculation method has been seen as vital for the design of thin-walled precast girders intended to be used in bridge construction (Wimmer (2016) and Kollegger, Foremniak, et al. (2014)).

Solution for the detail construction of the insertion of the precast concrete girders in combination with prestressing or post-tensioning into the integral structure can be found in Alberta Transportation Bridge Structures Design Criteria (2012), Kunin et al. (2000), Dicleli (2000), Steiger et al. (2012), and Schmidt et al. (2012). The piles are cast into the abutment and the beams rest on bearing pads, which are usually elastomeric or grout, on the abutment stem. The connections between the precast girders and the substructure can be ensured by steel pins, as shown in Alberta Transportation Bridge Structures Design Criteria (2012), dowels as shown in Dicleli (2000), or even extended bottom deck beam strands or bonded prestressed strands in combination with grouted pins as shown in Kunin et al. (2000). The beams are cast integrally into the abutments, normally during the casting of the in-situ concrete deck slab creating an ultimate rigid connection between the substructure, the precast girder and the superstructure.

The survey conducted by Kunin et al. (2000) shows that the implementation of precast and prestressed girders in integral abutment bridges has had an all in all good if not even excellent response from agencies all over the United States and Canada, with a majority finding no differences to the application of steel girders. Nonetheless a shortening of the structure, which should be considered when designing an integral bridge using precast prestressed or post-tensioned girders, has been observed in comparison to bridges constructed using steel girders.

Very impressive examples for the application of prefabricated bridge girders in combination with prestressing and post-tensioning in integral abutment bridges are the taxiway bridges of the landing runway at the Frankfurt am Main Airport. The integral jointless construction in connection with pre-stressed prefabricated girders completed with a cast in-situ concrete deck was chosen because of its low-maintenance properties but also to ensure a short building time with minimal limitations for public transport. The taxiway bridges of the Frankfurt am Main Airport show that an integral construction, even on this massive scale, can be economically viable (Schmidt et al. (2012) and Steiger et al. (2012)).

5.1.3 Selected examples of integral bridges with short and medium spans built in Austria

This section will give a short overview of three existing integral bridges in Austria, which were considered to be examined more closely in order to investigate if a redesign of the structures using thin-walled precast concrete bridge girders would be possible and plausible. Two of the bridges were constructed as steel-concrete composite bridges, one of them being a 15.50 m wide double-span bridge using four 1.20 m high I-beams and one being a 8.50 m wide single-span bridge using four rectangular steel beams with a varying height between 0.60 m and 0.95 m. The third evaluated bridge, was a conventionally cast double-span concrete bridge with a carriageway width of 12.0 m. All three bridges are overpasses over different motorways in Austria and were constructed as integral abutment bridges.

5.1.3.1 L21 Overpass - A1 Motorway

The conventionally cast double-span concrete bridge, Overpass L21 crossing the A1 Motorway shown in Figure 5.2, was used, in addition to being redesigned using thin-walled precast girders, as a model basis for the conducted numerical parameter study which will be presented in the following section. With a carriageway width of 11.50 m without the edge beams and field spans of 30 m, resulting in a total bridge span of 60 m, this bridge was a perfect suit to serve as a model for any further numerical investigations. The decision for this bridge to be built as an integral abutment bridge was vindicated by the optimal boundary conditions, consisting of the almost symmetrical longitudinal section, field spans not greater than 30 m and the high adjoining dam body.

The foundation of the L21 Overpass was built using in-situ piles with a diameter of 0.90 m. The piles were rigidly connected to the substructure, which was, in order to obtain a rigid frame structure, rigidly connected to the superstructure of the bridge. Wing walls ensured the lateral stability of the embankment. Transition plates were used to avoid settlement troughs behind the

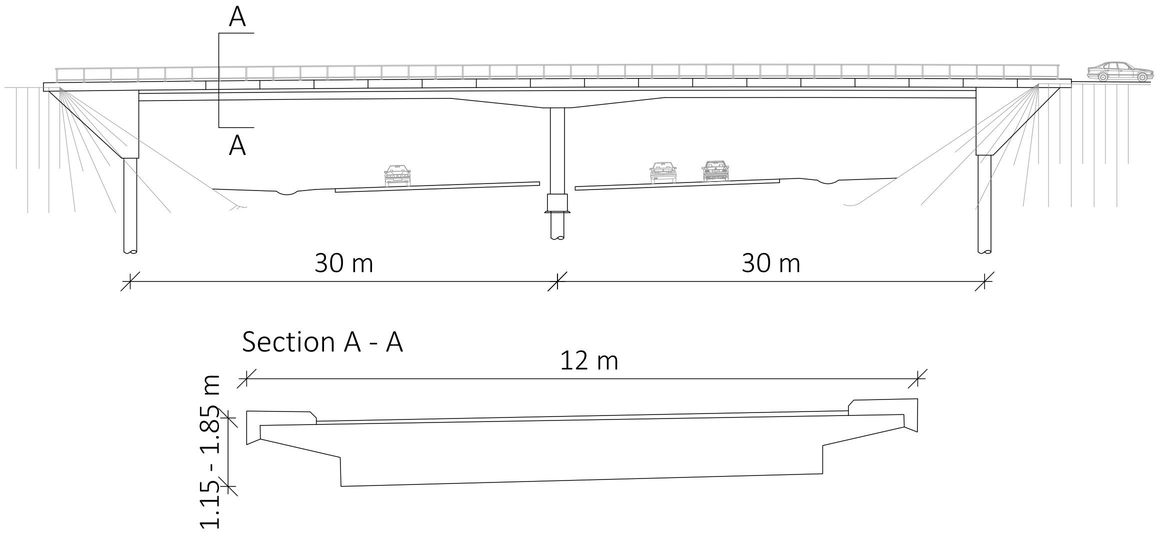


Fig. 5.2: Overpass L21 crossing the A1 Motorway in Austria

bridge structure. In order to reduce the constraints due to temperature in the abutment area, a modified backfill, with an elastic separation layer and geotextile reinforcement, was chosen. As can be seen in the side view of the bridge pictured in Figure 5.2, the height of the cross-section enlarged over the pier.

The erection of the L21 Overpass was divided into four construction phases. In the first phase two of the eight traffic lanes of the A1 Motorway, which were adjacent to the central piers of the overpass, were closed to traffic. Thereafter, in the following construction phase, one entire carriageway consisting of four traffic lanes was closed. During this closure half of the new overpass was cast on falsework. With the first half of the overpass finished, the carriageway was again opened for traffic and the procedure was repeated for the second half of the bridge (third construction phase). Once the entire overpass was erected and opened for traffic the old overpass had to be demolished. For this purpose the second carriageway of the motorway was left closed until half of the outdated structure was torn down. The last construction phase consisted in the switch of carriageway closures on the A1 Motorway and the demolition of the remaining old overpass.

5.1.3.2 S103 Overpass - A1 Motorway

The S103 Overpass, also being an overpass over the A1 Motorway in Austria, was designed and constructed as a single-span integral steel-concrete composite bridge as shown in Figure 5.3. The abutments including part of the superstructure were cast on falsework creating the corners of the rigid frame structure. The 28.80 m middle section of the rigid frame consists of four steel girders acting as webs. Precast concrete slabs were placed between the steel girders before the rest of the concrete deck was cast. The cross-section of the middle part of the bridge is illustrated in Figure 5.3, with the steel girders highlighted in blue. Bored piles with a diameter of 1.18 m and lengths of 15 m and 18 m were used for the foundation of the entire structure. The high adjoining dam body as well as the good soil conditions created optimal boundary conditions for the implementation of the integral abutment construction method.

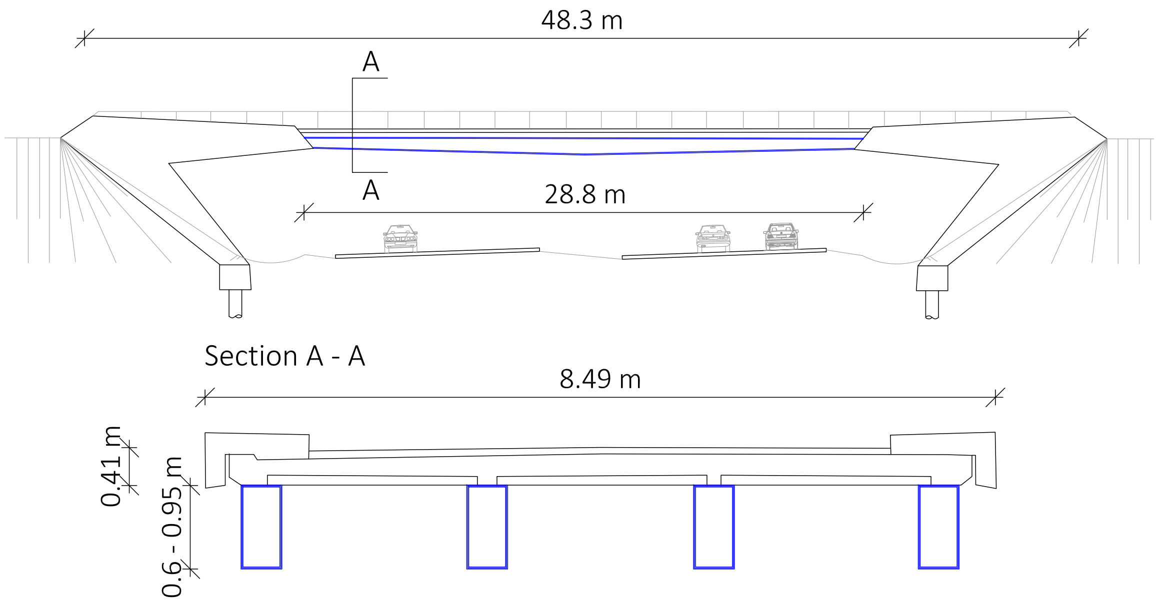


Fig. 5.3: Overpass S103 crossing the A1 Motorway in Austria

5.1.3.3 A2.Ü22a Overpass - A2 Motorway

The A2.Ü22a Overpass, a double-span integral steel-concrete composite bridge, spans over the A2 Motorway in Austria and is illustrated in Figure 5.4. The cross-section is made up of four I-beam steel girders, highlighted in blue in Figure 5.4, which are connected by dowel bolts to precast concrete elements completed with a cast in-situ concrete deck. As was the case with the other two described bridges the high adjoining dam body as well as the optimal span lengths enabled the execution of the bridge as an integral structure.

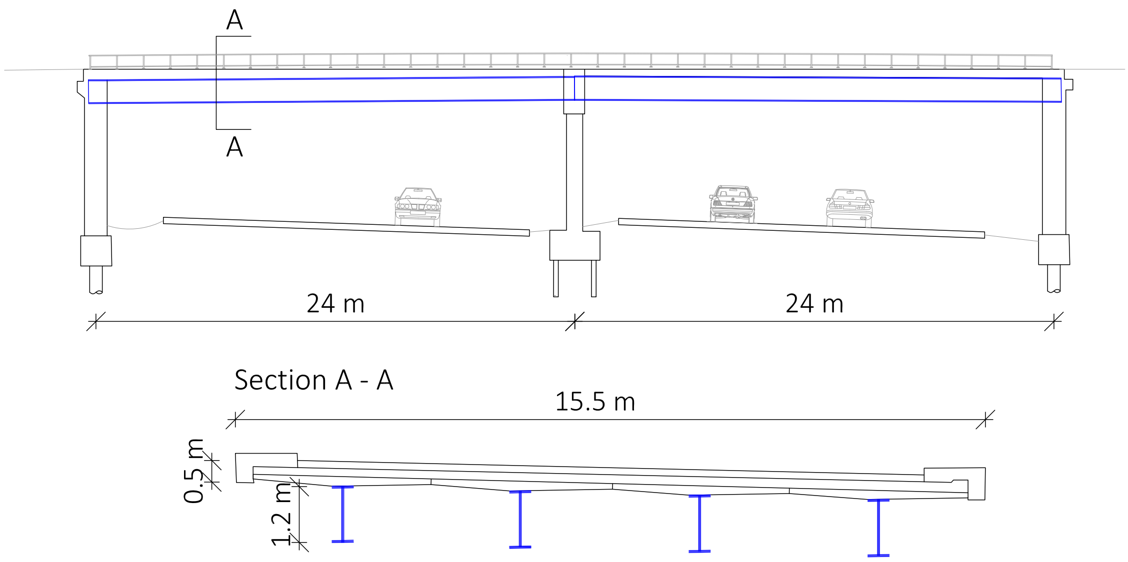


Fig. 5.4: Overpass A2.Ü22a crossing the A2 Motorway in Austria

In order to compensate for the longitudinal strains a 100 mm EPS plate and a 300 mm SSM concrete layer in combination with geotextile reinforcement were used as backfill for the abutments. Four bored piled with diameters of 0.9 m and lengths of 28 m were used for the foundation of each abutment. For the foundation of the central pier ductile piles with a diameter of 170 mm and lengths of 16 m were instituted.

5.2 A numerical parameter study - thin-walled precast bridge girders in integral bridges

In the course of investigating the application of thin-walled precast concrete bridge girders in the construction of integral bridges with small and medium spans a numerical parameter study on a double-webbed T-beam cross-sectioned double-span bridge with two lanes was conducted. The chosen static system for the calculation was a double-span rigid frame as shown in Figure 5.1 - Double spans systems - Point a), with field spans between 15 m and 35 m. The examined parameters comprised of the bridge span, the concrete strength of the precast elements and cast in-situ concrete as well as the dimensions of the precast bridge girders.

Due to the great customizability of the entire construction process of the thin-walled precast concrete bridge girders, certain parameters of the study were set before hand. As mentioned before the static system was chosen to be a double-span rigid frame with field spans between 15 m and 35 m. Furthermore, it was decided to set the bridge cross-section to a width of 12 m with a 0.40 m thick deck plate. The foundation modeling remained unvaried for all calculated models to obtain comparable results. The modeling of the substructure, the layering of the road structure, the input data for the thermal actions as well as the soil pressure were based on the design of an existing overpass (L21 Overpass) crossing the A1 Motorway described in the previous section and shown in Figure 5.2.

The goal of the parameter study was to demonstrate the feasibility of the construction method when it comes to integral abutment bridges. This was achieved by applying the decompression limit in the final state (cumulation of all construction phases) with the stress for a characteristic loading limited to $0.6 \cdot f_{ck}$.

5.2.1 Construction phases

For the parameter study four construction phases or loading cases, as shown in Figure 5.5, were considered for the calculations. The applied post-tensioning, being a very important factor in the construction using thin-walled precast concrete bridge girders, had to be considered separately for each construction phase. Furthermore the changing of the cross-sections caused by the filling with in-situ concrete had to be factored in. As recommended by Wimmer (2016) and Dicleli (2000) each construction stage was analyzed separately, with the effects of the applied loads on the structure members carried from a previous construction stage to the next.

In the first construction phase the assembled and post-tensioned thin-walled bridge girders were transported to the construction site, mounted to their final position and loaded with a working load of 1 kN/m. For the static calculations of construction phase 1 the individual girders were regarded as single-span beams with a trough cross-section. All constructive measures to ensure

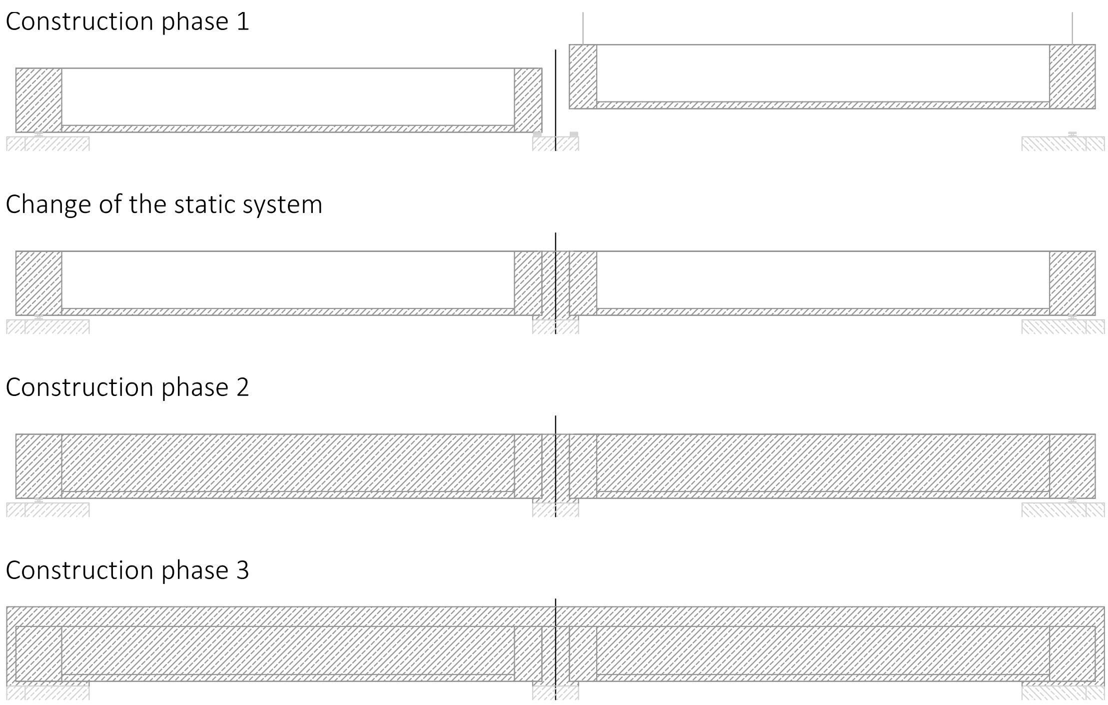


Fig. 5.5: Construction phases considered in the parameter study

the anchorage and redirecting of the tendons as well as all additional reinforcement and ducts needed for the subsequent construction phases were placed in the girders during the assembly in the manufacturing plant.

By filling the joint between the two single-span beams a double-span beam with a movable bearing at each end was created. The ends of the beams were not connected to the abutment until construction phase 3 in order to reduce the constraining forces which resulted from the additional post-tensioning during construction.

The evolved static system (double-span beam) was used for all further calculations of the second construction phase, in which the trough cross-sections of the thin-walled girders were filled with in-situ concrete. The concrete was filled in two stages (construction phase 2a and 2b), with 24 hours of hardening time lying between the two stages. The insertion should occur symmetrically and in layers and the post-tensioning should be applied during the concreting process. For the calculation the load-bearing capacity of the hardened in-situ concrete in construction phase 2b was factored in, resulting in two different cross-sections to be considered.

During the third construction phase the deck slab of the bridge as well as the connection between the superstructure (double-webbed T-beam) and the substructure was cast. The cross-section used for all calculations of this construction phase was the filled rectangular cross-section of the precast girders. With the corners of the rigid frames fixed the static system used to calculate the last construction phase and final state, comprising of all bridges loads, such as the road construction, the edge beams and all traffic loads as well as temperature and soil pressure, was the double-span rigid frame as shown in Figure 5.1 - Double spans systems - Point a).

5.2.2 Modeling and FEM calculations

The model used for the FEM calculations of the bridge in its final state is shown in Figure 5.6. The system was divided in two, leaving a single T-beam girder with a 6.0 m wide deck slab, for evaluation of all variants of the parameter study.

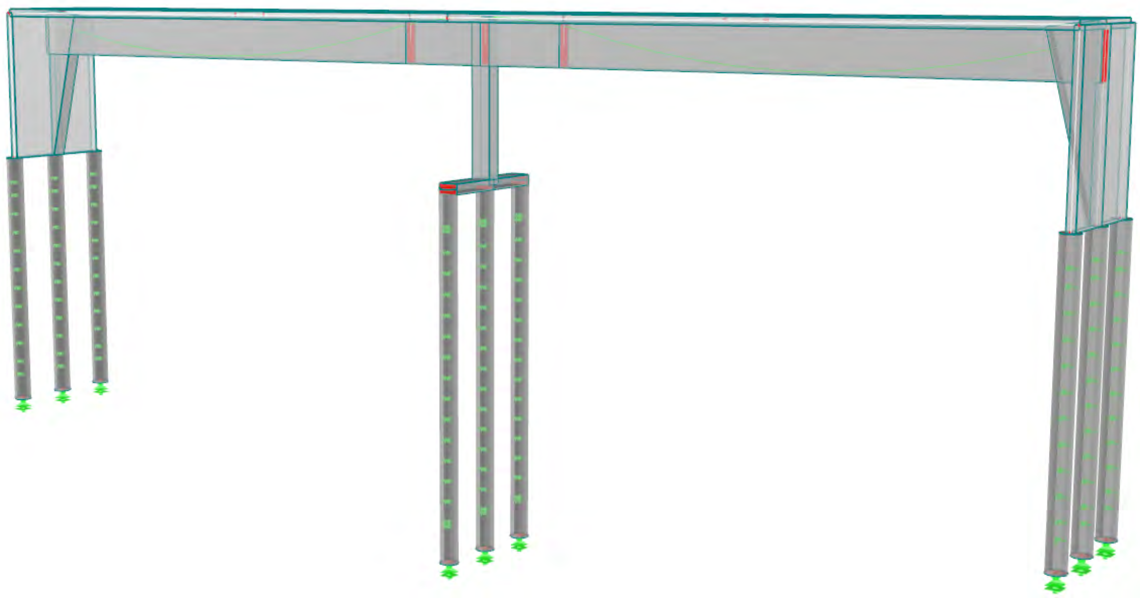


Fig. 5.6: FEM Model used for the parameter study

The bridge girders, modeled as ribs, were submerged halfway into the 400 mm thick deck slab and connected to the abutment walls, which were modeled as a 2D panel element. The height of the superstructure was calculated from the height of the modeled precast beam in addition of half the deck slab (200 mm).

The abutment wall was defined to be of the same thickness as the deck slab. Based on the research conducted by Kleiser (2016), it was decided to adapt the cross-sectional shape of the superstructure into the abutment wall, easing in the static compatibility of the two elements. The rib (precast beam) of the superstructure was thus pulled around the rigid frame corner to be then tapered down towards the pile head plate (clearly visible in Figure 5.6)

The dimensions of the piers were chosen to fit the width of the precast beam, resulting in a cross-section of 0.70 m by 1.0 m. The subdivision of the beam that is marked red in Figure 5.6 was implemented to factor in a widening of the beam near the piers if necessary.

The first three construction phases were programmed in Excel due to the simplicity of the static systems (single-span and double-span beams) and equal load distributions. The calculation of the internal forces were based on a linear-elastic behavior of the structure.

5.2.2.1 Examined parameters

In the presented parameter study a double-field rigid frame with field spans between 15 m and 35 m was examined. Additionally to the varying span lengths, two different concrete classes

(C50/60 and C30/37) were considered. Bases on the field length the optimum height and number of required tension strands for each case were calculated. The chosen static system as well as the selected cross-section are shown in Figure 5.7. A widening of the thin-walled precast girder to double the width near the piers was considered for all calculations.

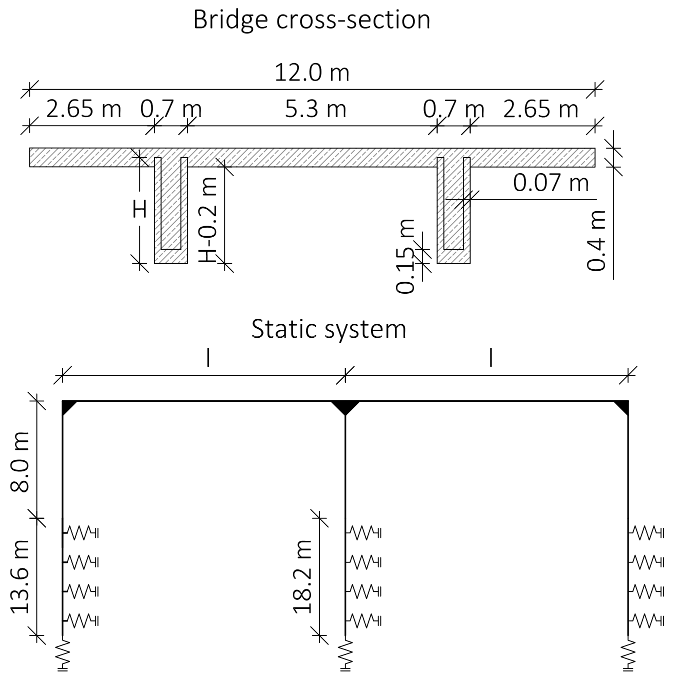


Fig. 5.7: Selected cross-section and chosen static system used for the parameter study with all examined parameters

Due to the great customizability of the entire construction process with thin-walled precast concrete bridge girders, certain parameters of the study were fixated before hand. The walls of the thin-walled precast bridge girders were set to the usual 70 mm thickness with a 150 mm thick bottom slab connecting them together to create a 0.70 m wide trough shaped cross-section with a variable height H. The entire bridge cross-section was set to a width of 12.0 m with a 400 mm thick deck plate. The foundation modeling remained unvaried for all calculated models in order to obtain comparable results. The field span length l were examined in steps of 5.0 m with a chosen maximal length of 35 m, considering the technical feasibility of transporting the precast girders to the construction site.

5.2.2.2 Modeling of the individual construction phases

When building bridges using thin-walled precast concrete girders, not only the static systems but also the cross-sections change with almost every construction phase. The simplified static systems and corresponding cross-sections used for all calculations are shown in Figures 5.8 through 5.11. In addition, the implemented post-tensioning in each construction phase is illustrated in dark blue in all Figures. The highlighted orange elements in the Figures represent the cast in-situ concrete, that had to be considered as a new load in the respective construction phase.

The first construction phase, in which the thin-walled precast concrete bridge girders are transported and mounted on the construction site, was calculated as two single-span beams, as shown in Figure 5.8, whereby only the cross-sections of the thin-walled precast concrete girders were load-bearing. The placement of the post-tensioning tendons, which were already installed and stressed in the manufacturing plant, was designed with the objective to reduce the bending moment of the girders resulting from dead load. With the girders being hollow the course of the tendons was not perfectly counteractive to the resulting moments and had to be installed using small concrete blocks which diverted the straight tendons. Once the girders were positioned on the abutments and the pier, the joint between the two girders was filled.

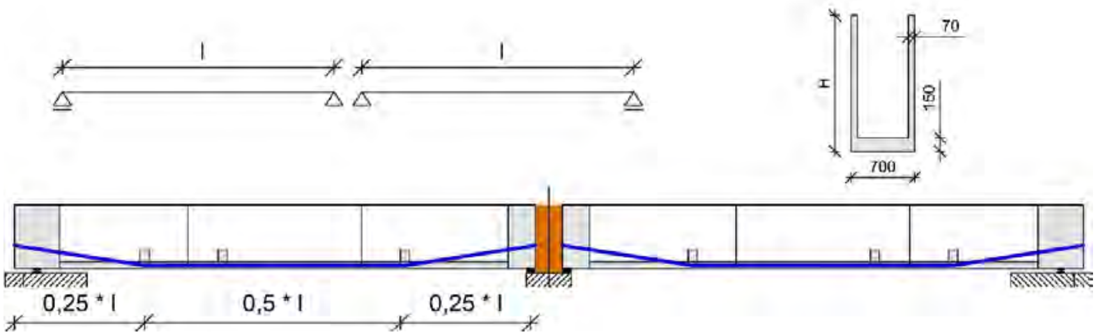


Fig. 5.8: Static system, cross-section and post-tensioning of construction phase 1

With the concrete in the joint between the two bridge girders hardened, the static system for the calculations changed, as can be seen in Figure 5.9, from two single-span beams to a double-span beam. The cross-section from construction phase 1 was used for the calculations of the first part of construction phase 2 (2a) in which the thin-walled girders were filled halfway with cast in-situ concrete (left cross-section of the two shown cross-sections in Figure 5.9 with the amount of cast in-situ concrete visualized in orange).

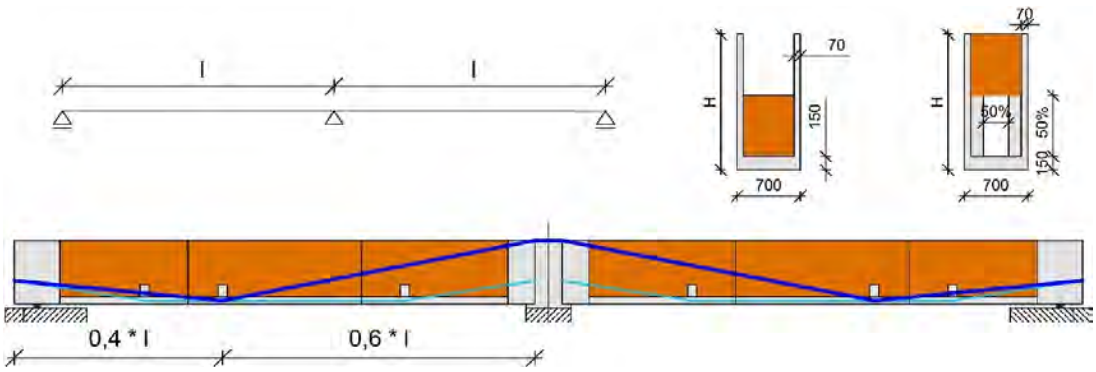


Fig. 5.9: Static system, cross-sections and post-tensioning of construction phase 2 (2a and 2b)

The second half of the girders was filled (construction phase 2b) after the first half had already hardened, making it possible to factor in a part of the cast concrete from construction phase 2a into the load bearing cross-section. It was assumed that half of the concrete strength could be applied, resulting in a new cross-section as shown in Figure 5.9 (right cross-section of the two). The tendons for the post-tensioning for this construction phase were placed with the intent to reduce the moments in mid-span as well as at the support. As was the case in construction phase

1, the straight tendon course was diverted by concrete blocks in the thin-walled precast girders.

During the third construction phase (Figure 5.10) the deck slab of the bridge as well as the connections between the bridge superstructure and substructure (frame corner) were cast. The cross-section used for all calculations of this construction phase was a rectangular section. As was the case in the other construction phases, the tendons for the post-tensioning for this construction phase were placed with the intent to reduce the moments or in other words the stresses caused by the newly applied loads. The difference to the previous phases was that, with the thin-walled bridge girders being filled with concrete, it was possible to achieve a parabolic tendon elevation.

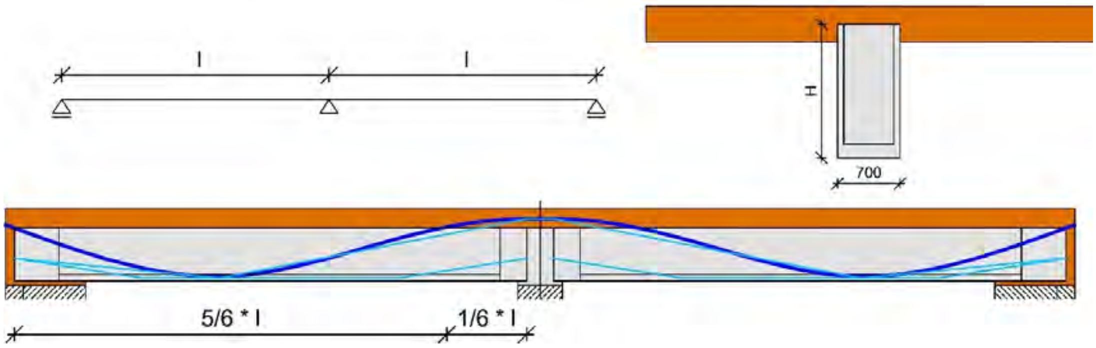


Fig. 5.10: Static system, cross-section and post-tensioning of construction phase 3

With the corners of the frame cast, all calculations for construction phase 4 were executed on a double-span frame with the final cross-section as shown in Figure 5.11. The effective width of the T-beam was, needless to say, taken into account for all calculations. The tendons, which were introduced in construction phase 3, were re-stressed in order to apply the needed post-tensioning for this construction phase.

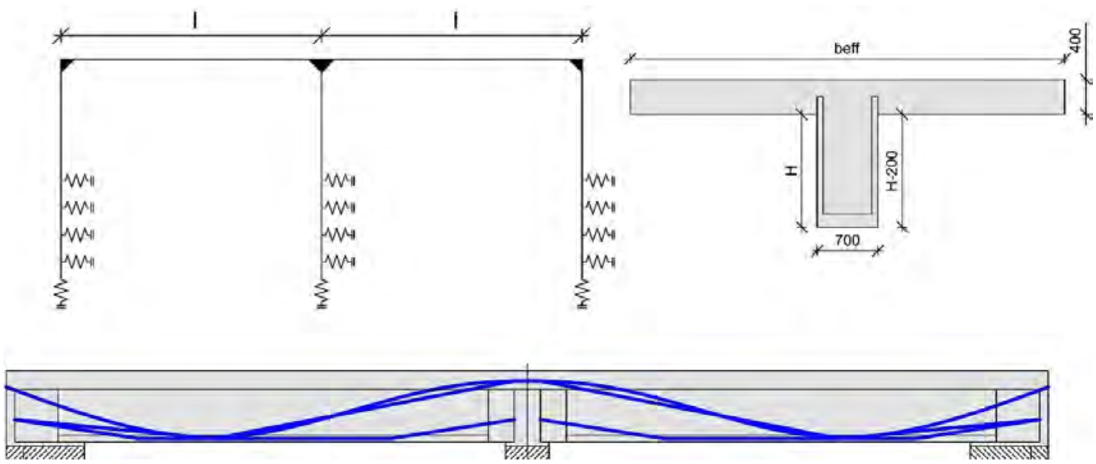


Fig. 5.11: Static system, cross-section and post-tensioning of construction phase 4

5.2.2.3 Modeling of the piers, abutments and foundation

The modeling of the foundation was based on the designs of the L21 Overpass, which is shown in Figure 5.2 and described in detail in section 5.1.3.1 and was subsequently redesigned using thin-walled precast bridge girders.

The foundation of the entire structure was modeled as rigidly connected in-situ piles with a diameter of 0.90 m. The piles had a length of 13.60 m at the abutments and 18.20 m under the middle pier. The bedding of the piles was chosen with 54 000 kN/m^2, as was the case for the original bridge. The cross-section of the piers was chosen to match the width of the precast bridge girders, resulting in a cross-section of 0.70 m by 1.0 m. The piers were rigidly connected to the superstructure. For the first three construction phases the modeling of the substructure was insignificant since the ends of the calculated double-span beam were not connected to the abutments.

5.2.2.4 Composition of the results

For the evaluation of the results only one web of the double-webbed T-beam cross-section was investigated. In order to determine if the decompression limit was fulfilled in the final state (cumulation of all construction phases) the stresses at the upper and the lower edge of the precast girder were looked into. The development of the stress resultants along the entire length of the precast girder resulted out of the linear-elastic calculations. By adding the stresses caused by the moments and the axial forces together the stress distribution over the height along the entire length of the girders was evaluated.

5.2.2.5 Tendon elevations and determination of the required post-tensioning load

The importance of post-tensioning when building with thin-walled precast elements is clearly explained by Wimmer (2016). If girders out of thin-walled precast elements with lengths between 12 m and 35 m are to be installed support-free in any bridge construction post-tensioning must be applied. For the first construction stages the tendons are installed in the hollow girders using anchoring blocks and tendon deviation saddles as would be the case when using external post-tensioning.

An overview of the three different elevations of the post-tensioning tendons applied in the parameter study, with only the last being parabolic, are shown in Figure 5.12.

For the first construction phase the tendons were deflected using two deviation saddles positioned in the quarter points of the thin-walled precast girders at $x = l/4$ and $x = 3 \cdot l/4$. An additional deviation saddle was cast for the tendons needed for the second construction phase at $x = 0.4 \cdot l$. The anchors for the tendons needed for the first construction stage were situated on both sides in the centroidal axis of the trough cross-section. After the static system of the structure changed the placement of the anchors for the tendons needed for the second construction stage were adjusted. At the abutments the anchors stayed in the centroidal axis of the trough cross-section. Above the piers the anchors were moved to the top of the girder in order to counteract the negative moment at the support.

A parabolic tendon elevation was technical feasible once the thin-walled precast girders were filled with cast in-situ concrete. The tendon elevation for the third and for the final construction phases started of at the abutments in the centroidal axis of the T-beam, had its vertex at $x = 5 \cdot l/12$, the point of inflection at $x = 5 \cdot l/6$ and was placed, exactly like the tendon of construction

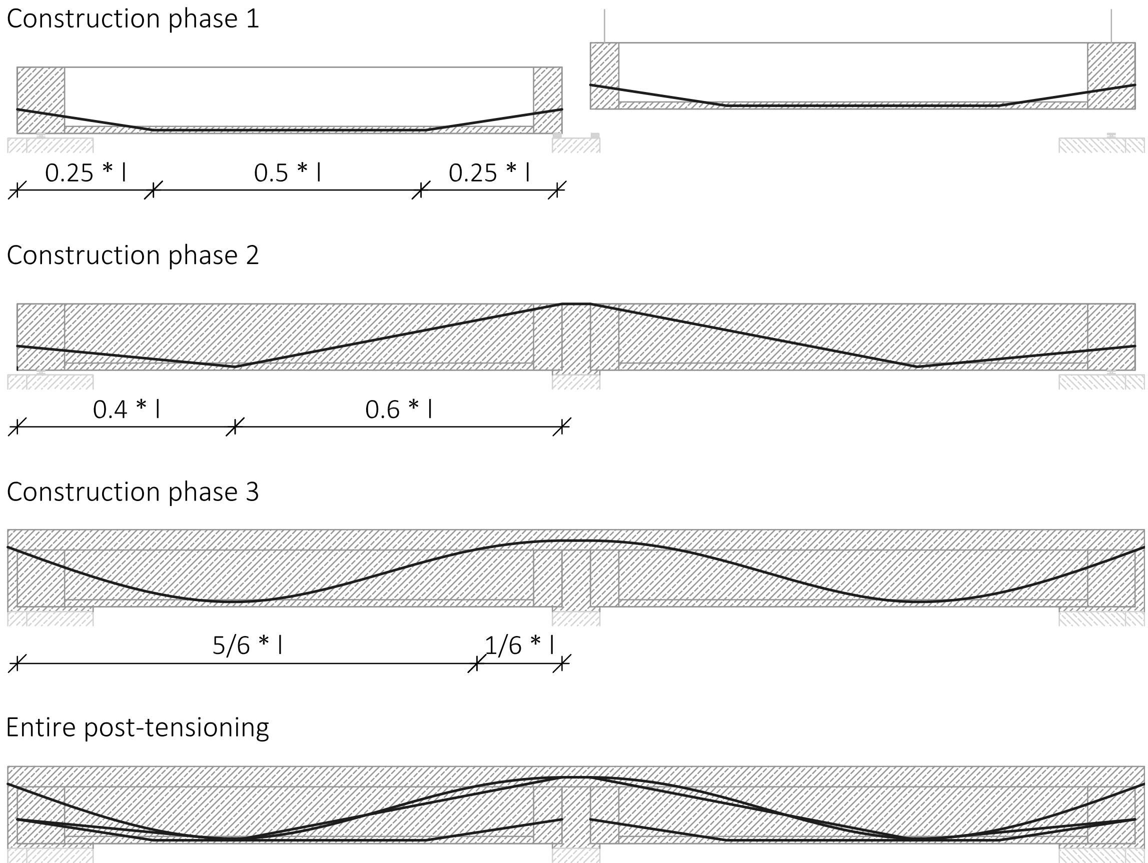


Fig. 5.12: Tendon elevation in each construction phase

phase 2, on the top edge of the precast girder above the pier.

The determination of the required stress for each individual construction phase was iterated with the aim of fulfilling the decompression limit with a characteristic load placement.

5.2.3 Applied loads

For the compilation of the applied loads, consisting of the constant dead loads, the additional loads, the traffic loads, the temperature impact, the impact of shrinkage, the soil pressure as well as the post-tensioning of the superstructure, for all the various static systems in the four construction phases special care had to be taken in order to avoid repeated loading of the bridge girders.

The dead and additional loads, according to Eurocode EN 1991-1-1 (2003) and in particular to ÖNORM B 1991-1-1 (2017), for all construction phases are listed in Table 5.1. By totaling all loads from construction phases 1 through 3 the entire self-weight of the superstructure was compiled. All the additional loads were applied in construction phase 4. The layering of the road structure was based on the designs of the existing L21 Overpass (section 5.1.3.1), which already served as a model for the modeling of the piers, abutments and foundation.

Tab. 5.1: Compilation of the dead and additional loads applied in the parameter study in the four construction phases (thin-walled precast bridge girder height H = 1.70 m)

Constr. Phase	**Load**	**Applied load [kN/m]**	**Section modulus top/bottom [m^3]**
1	Trough cross-section	8.00	-0.09 / 0.15
2	In-situ concrete 1^{st} half	10.85	
	In-situ concrete 2^{nd} half	10.85	-0.10 / 0.22
3	Cast deck slab	56.50	-0.34 / 0.34
4	Pavement AC11	3.60	-1.82 / 0.63
	Leveling layer AC22	11.52	
	Protective layer AC8	4.32	
	Insulation	0.72	
	Edge beams	10.00	

The effects of the loads due to traffic, consisting of cars, lorries and special vehicles were also taken into account in the parameter study. For the calculations of the decompression limit ψ_2 was set, according to ÖNORM B 1992-2 (2008), to 0,3 for traffic loads of Traffic Load Model LM1 according to Eurocode EN 1991-2 (2003) and in particular to ÖNORM B 1991-2 (2011). The critical load cases were identified and for each critical load case, the design values of the effects of action in combination were determined according to the standards. In addition to the vertical forces, the horizontal forces, consisting of soil pressure resulting from the traffic as well as braking and acceleration forces, were considered. With only half the system being calculated (carriageway width of 6.0 m) two notional lanes were defined, producing a knife-edge load $q_k = 34.5\,\text{kN/m}$ and two single loads (axle loads) $Q_k = 500\,\text{kN}$.

In order to factor in the thermal actions the uniform bridge temperature component ΔT_N, according to Eurocode EN 1991-1-5 (2003) and in particular to ÖNORM B 1991-1-5 (2004), was applied in the parameter study. The input data of the existing L21 Overpass (section 5.1.3.1), that had already been used as a calculation basis for the substructure and the layering of the pavement structure, was used to calculate the needed temperature components and fluctuations. The overall range of the uniform temperature component ΔT_N therefore resulted in 62°C that had to be considered. The effects on the structure caused by shrinkage were taken into account by applying an equivalent cooling temperature, according to Eurocode EN 1991-1-1 (2003) and in particular to ÖNORM B 1991-1-1 (2017), to the bridge with an temperature difference of $\Delta T = 26\,\text{K}$.

As was the case for the modeling of the substructure, the layering of the pavement structure and the input data for the thermalactions, the soil pressure on repose was chosen according to the data of the existing L21 Overpass (section 5.1.3.1). The soil pressure was divided, as was the case for the traffic loads, into notional lanes and was set as an approximation to a constant level. Half of the load was applied as a permanent fixed action the second half as a variable free action.

As already mentioned, the post-tensioning had to be optimized and calculated separately for each construction phase. The tendon elevations were practically customized to each construction phase, resulting in three different tendon types, in order to counteract the inflicted loads. The number of strands and the amount of force applied to the tendons was calculated by limiting the stress in the concrete girders for a characteristic loading to $0.6 \cdot f_{ck}$ but simultaneously fulfilling the decompression limit (including $0.3 \cdot$ LM1).

5.2.4 Results of the numerical parameter study

The results of the investigation of the application of thin-walled precast concrete bridge girders in the construction of integral bridges with small and medium spans are shown in Figure 5.13. The examined parameter comprised of the bridge span, the concrete strength of the precast elements and cast in-situ concrete as well as the dimensions of the precast bridge girders, in particular the girder height H.

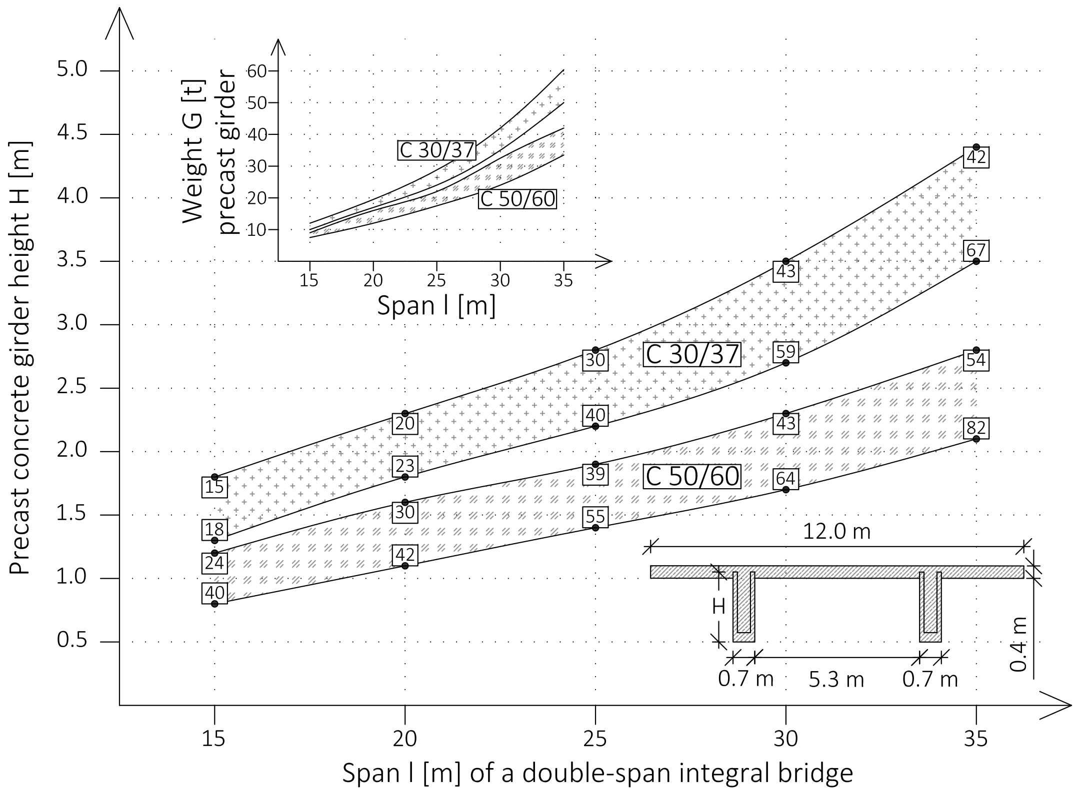


Fig. 5.13: Results of the parameter study of a double-span bridge constructed using thin-walled precast bridge girders: height-span-ratio of the beams depending on their concrete strength

The required precast girder height H in combination with the required number of strands (the numbers in the square boxes) are illustrated in Figure 5.13 as functions of the bridge field spans and the compressive concrete strength. The results are shown for concrete strengths of C30/37 and C50/60. Furthermore an embedded graph shows the lifting weight of the precast girders depending on the girder spans.

The area of application of thin-walled precast concrete girders for the construction of double-span integral bridges can be verified for field spans of 15 m to 35 m. The displayed height calculation as well as amount of required strands is to be seen as a recommendation or guideline. By slight adjustments of the tendon elevations or exact detailed calculation an optimization of the height and strand amount should be possible. The calculations are to be made on a case-by-case basis, factoring in all possible parameters and the exact cross-section dimensions.

In order to stay in a realistic height range girders heights should be held to a maximum of 1.50 m, therefore a span of 25 m with a concrete strength of C50/60, according to Figure 5.13, is attainable. By increasing the concrete strength or changing the bridge cross-section, for example from a double-webbed to a four-webbed T-beam, the necessary girder height H could be lowered noticeably. A change in the precast girder width also reduces the necessary height H accordingly.

Wimmer (2016) investigated the application of thin-walled precast concrete girders made out of double wall elements or precast slabs for the construction of single-span bridges with carriageway widths of 8.50 m and 14.50 m. The results of his parameter studies are shown in Figure 5.14 and Figure 5.15. As in Figure 5.13 the required precast girder height H in combination with the required number of strands (the numbers in the square boxes) are illustrated as functions of the bridge field spans and the compressive concrete strength. Furthermore an embedded graph shows the lifting weight of the precast girders depending on the girder spans.

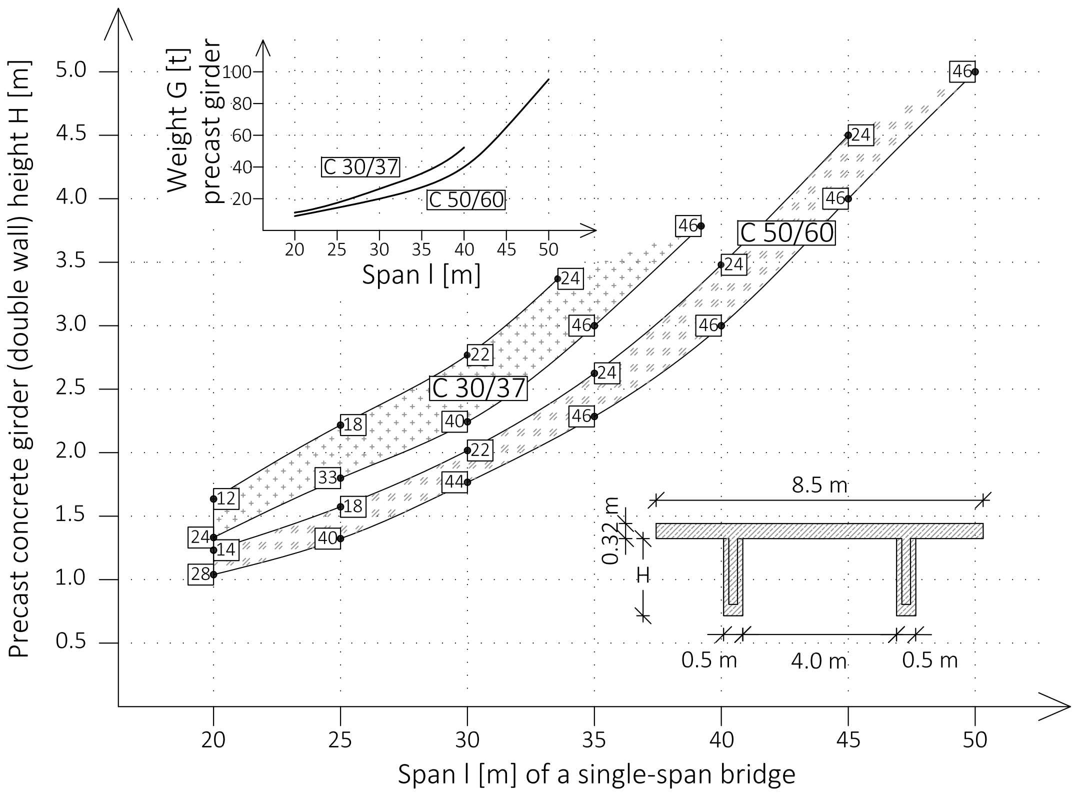


Fig. 5.14: Parameter study of a single-span bridge with carriageway widths of 8.50 m constructed using thin-walled precast bridge girders made out of double wall elements: height-span-ratio of the beams depending on their concrete strength according to Wimmer (2016)

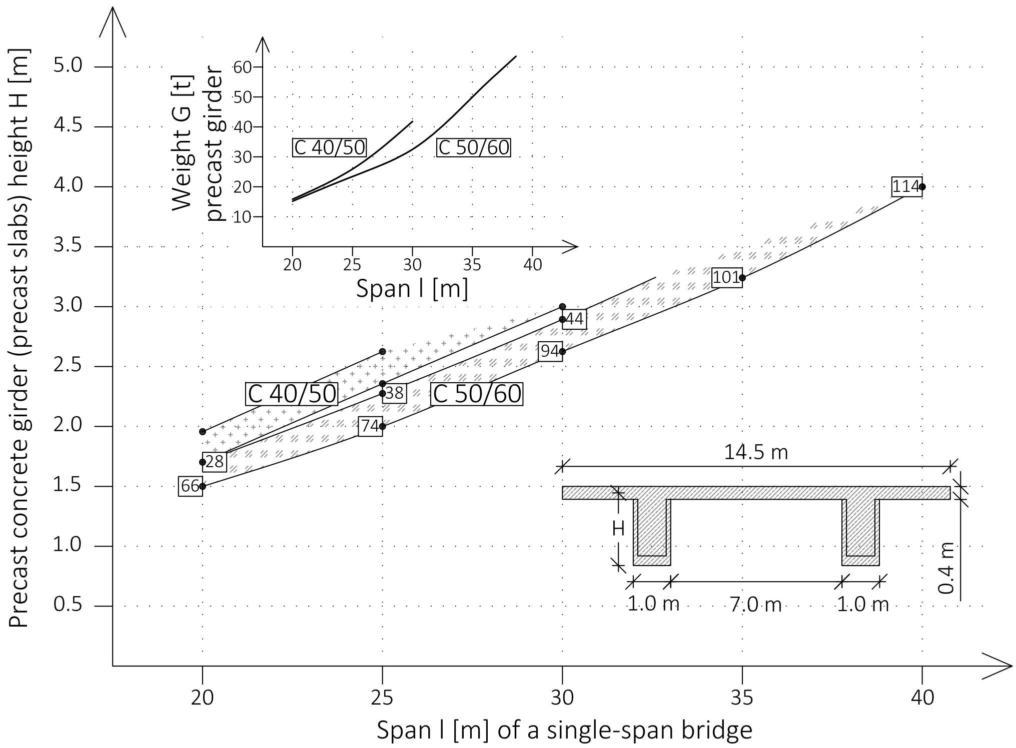


Fig. 5.15: Parameter study of a single-span integral bridge with a carriageway width of 14.50 m constructed using thin-walled precast bridge girders: height-span-ratio of the beams depending on their concrete strength according to Wimmer (2016)

The investigated construction phases and assumptions made by Wimmer (2016) can easily be compared to those of the presented parameter study. According to Wimmer (2016) lower precast girder heights H can be achieved if the bridges are constructed in an integral manner. By comparing Wimmers results to those of the presented research, this statement can be confirmed. Due to the monolithic connection between the abutment and the bridge superstructure in the later construction phases, the stresses occurring in the cross-sections can be evidently reduced.

5.3 Redesign of existing integral bridges

As mentioned in a previous section of this chapter, three existing integral bridges built in Austria, all being overpasses crossing major motorways, were examined closely in order to investigate if a redesign of the structures using thin-walled precast concrete bridge girders would be possible and economically plausible. The static systems used for the redesign were taken from Figure 5.1, whereas sometimes one bridge was revised using more than one static system. Furthermore not only post-tensioned variants were looked into but also the construction using girders with merely steel reinforcement. The roadway structure as well as the modeling of the foundations for the redesigns were always chosen and calculated in the same way as was the case for the original designs. All spans and cross-sectionals widths remained unchanged.

5.3.1 L21 Overpass - A1 Motorway - Redesign

For the alternative designs of the L21 Overpass, which is shown in Figure 5.2 and described in detail in section 5.1.3.1, three different design alternatives for the production of an integral concrete bridge using thin-walled precast concrete bridge girders, based on two static systems (Figure 5.1 Double-span system a) and b)), were investigated. An integral construction using concrete hinges at the connection between the superstructure and the substructure (Figure 5.1 Double-span system b)) was considered to be an intriguing modification of the entire system and was therefore evaluated for both post-tensioned girders and girders with merely steel reinforcement. Additionally the double-span system a) from Figure 5.1, a standard double-span rigid frame, was reviewed for a design using girders with merely steel reinforcement. The static systems used to calculate the alternatives are shown in Figure 5.16.

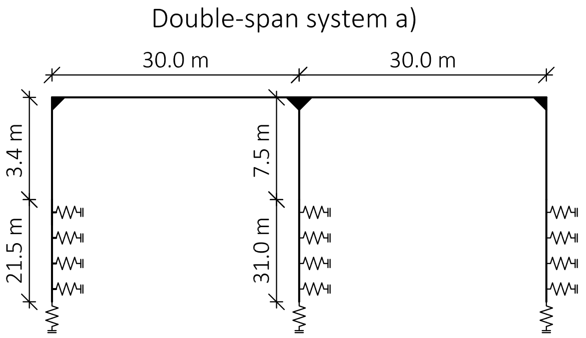


Fig. 5.16: Chosen static systems used for the redesign of the existing L21 Overpass

In order to compare the results as well as the construction phases of all three alternative designs it was decided to chose one carriageway cross-section, in particular a double-webbed T-beam cross-section, which would be used in all three designs. Instead of a haunch at the pier a widening of the beams to their double width near the pier was conducted. The regular cross-section and the the cross-section with the widened webs are shown in Figure 5.17. The 1.40 m high thin-walled precast girders were made up of 70 mm thick precast slabs which were connected together by a 150 mm thick cast bottom slab creating U-shaped elements. The entire abutment area was, with the exception of the intersection between superstructure and the substructure, left unchanged. The central pier was altered from a continuous pier wall to two piers matching the dimensions of the precast bridge girders, resulting in a cross-section of 1.40 m by 1.0 m for each pier.

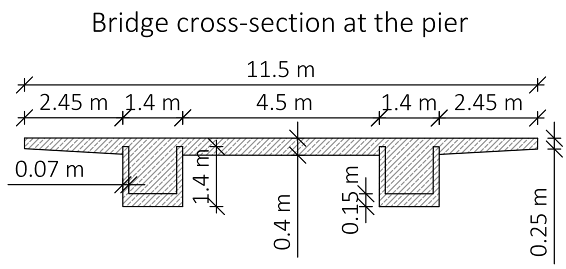


Fig. 5.17: Selected cross-sections used for the redesign of the existing L21 Overpass

The calculations of the individual construction phases were handled in the same way as during the numerical parameter study. The effects of the applied loads in each construction phase on the structural members were carried from a previous construction stage to the next. The only difference to the parameter study was the need of auxiliary supports for the two alternatives without post-tensioning. The compilation of the applied loads, consisting of the constant dead loads, the additional loads, the traffic loads, the temperature impact, the impact of shrinkage, the impact of creep, the soil pressure as well as the post-tensioning of the superstructure (if used) were taken from the original calculations or calculated according to current standards.

As was the case in the parameter study, three different types of tendon elevations were implemented for the alternative design using post-tensioning, one meant for transport and installation, one for the loads caused by the filling of the girders with in-situ concrete and one meant for all of the remaining loads including the meeting of the decompression limit in the final state. All three tendon elevations are shown in Figure 5.18. A total of 53 strands was needed to uphold all requirements according to the recent standards, with some tendons being re-stressed during the construction process. When comparing the girder height and the amount of strands needed to the results of the parameter study, shown in Figure 5.13, an optimization of both parameters is noticeable. This phenomenon can be traced back to the adjustment of the tendon elevations as well as the slightly different boundary conditions. The tendons with a parabolic elevation, for example, were divided into two, with each half stressed on one abutment and ending 6.0 m short of the abutment on the other side.

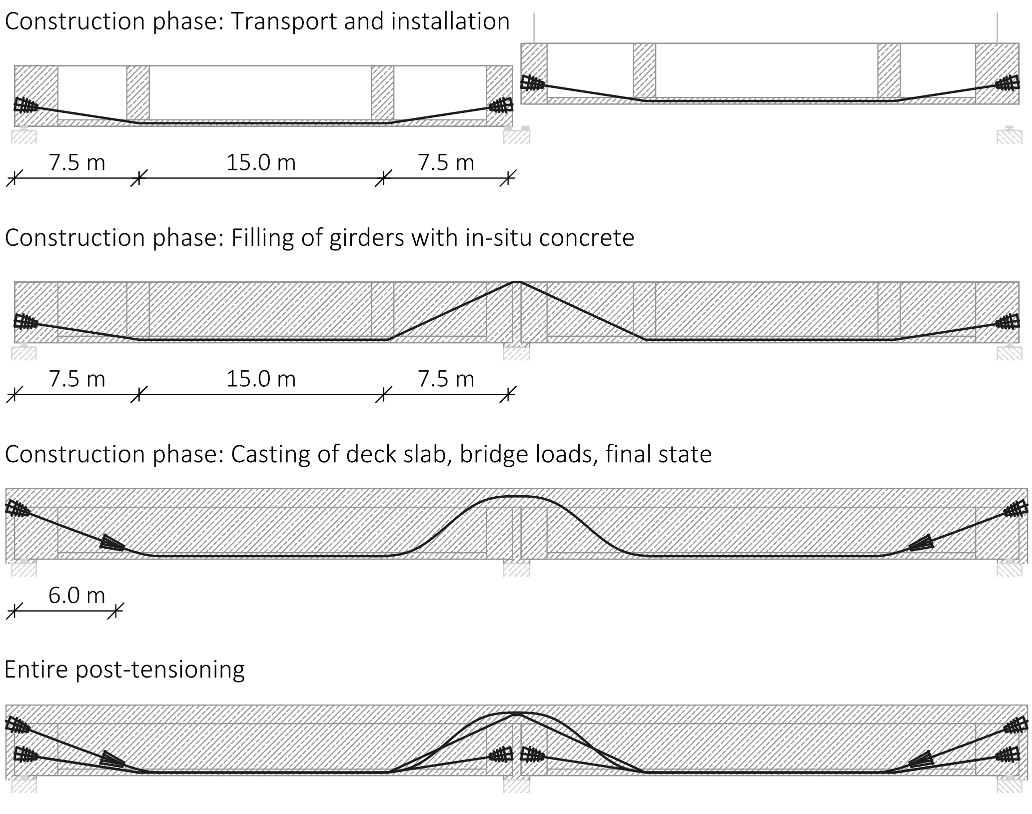


Fig. 5.18: Tendon elevation in each construction phase of the redesigned L21 Overpass

The two alternative designs only using reinforcement and no post-tensioning merely differ in the intersection between superstructure and substructure, with one being a standard rigid connection as shown in Alberta Transportation Bridge Structures Design Criteria (2012), Kunin et al. (2000), Dicleli (2000), Steiger et al. (2012), and Schmidt et al. (2012) and the other a simple concrete hinge as described in Marx et al. (2010), Schlappal et al. (2017), and Schacht et al. (2010). Since no post-tensioning is integrated into the girders a girder length of 30 m cannot be achieved without using auxiliary piers for support, as shown in Figure 5.19. The joints between the two separate girders can either be chosen to be wide (from 100 mm up to 200 mm) and be filled simultaneously with the first layer of cast in-situ concrete when the girders are filled or can be executed as described in section 3.2 Redesign of joints.

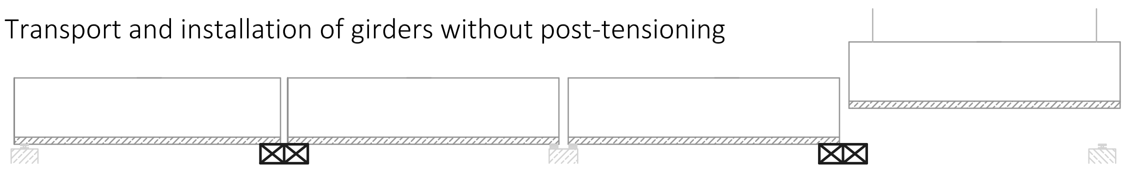


Fig. 5.19: Transport and installation of girders without post-tensioning using auxiliary piers for the redesign of the L21 Overpass

The removal of the auxiliary supports was simulated as a single applied load in the final state. The load consisted of the cumulated reactions over the the entire building process. In contrast to the post-tensioned calculations, in which the meeting of the decompression limit was decisive for the entire designing procedure, the calculations of the design without post-tensioning were all about the compliance with the ultimate limit state. When comparing the two designs calculated without post-tensioning, one with a standard rigid intersection between superstructure and substructure and one with a simple concrete hinge, a reduction of longitudinal and stirrup reinforcement was observed at mid-span if a rigid intersection was chosen. On the other side the implementation of a concrete hinge reduced the needed amount of reinforcement in the support area.

5.3.2 A2.Ü22a Overpass - A2 Motorway - Redesign

For the alternative designs of the A2.Ü22a Overpass, which is shown in Figure 5.4 and described in detail in section 5.1.3.3, two different design alternatives, differing in the amount of applied post-tensioning, for the production of an integral concrete bridge using thin-walled precast concrete bridge girders were investigated. For all calculations a rigid frame static system according to Figure 5.1 (Double-span system a)), as shown in Figure 5.20, was chosen.

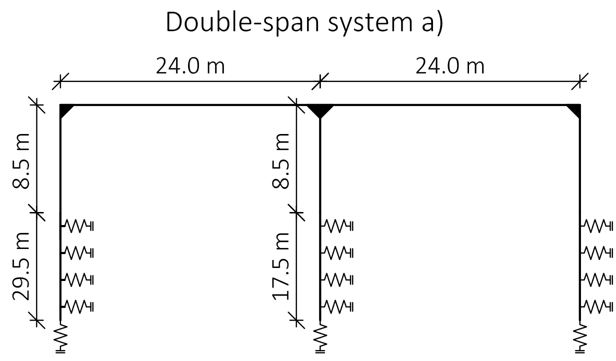


Fig. 5.20: Chosen static systems used for the redesign of the existing A2.Ü22a Overpass

For both evaluated designs the double-webbed T-beam cross-section shown in Figure 5.21 without any widening of the webs at the piers was seen as adequate. In comparison to the cross-sections of the parameter study and the L21 Overpass a wider carriageway was required. The 1.35 m high thin-walled precast girders were made up of 70 mm thick precast slabs which were connected together by a 150 mm thick cast bottom slab to create U-shaped elements. The tendon deviation saddles as well as the anchoring blocks were kept to the ultimate minimum for both alternative designs.

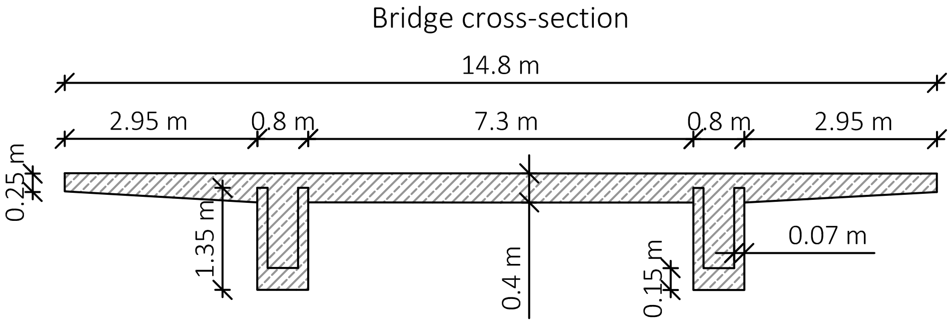


Fig. 5.21: Selected cross-sections used for the redesign of the existing A2.Ü22a Overpass

The abutments were altered in the same way as in the parameter study, with the walls being defined to be the same thickness as the deck slab and with an adaptation of the superstructures cross-section as suggested by Kleiser (2016). The ribs of the superstructure were pulled around the rigid frame corners to be then tapered down towards the pile head plate. The dimensions of the piers were chosen to fit the width of the precast beams, resulting in a rectangular cross-section of 0.80 m by 0.80 m.

The calculations of the individual construction phases was handled in the same way as during the numerical parameter study and the calculations of the redesigned L21 Overpass. The effects of the applied loads in each construction phase on the structural members were carried from a previous construction stage to next up to the final state. The compilation of the applied loads, consisting of the constant dead loads, the additional loads, the soil pressure, the traffic loads, the temperature impact, the impact of shrinkage and creep as well as the post-tensioning of the superstructure were based on the original calculations or calculated according to current standards.

As mentioned before, two different designs for the overpass were investigated. Based on the results of the redesign without any post-tensioning of the L21 Overpass, the idea of zero post-tensioning was revised and it was decided to examine a design with minimal post-tensioning, in particular solely post-tensioning needed for transport and installation in order to avoid the necessity of auxiliary piers. For this case a modification of the construction phases had to be carried out. Additionally to the design with minimal post-tensioning a design, similar to the post-tensioned designs of the L21 Overpass and the parameter study, set to meet the decompression limit, was developed.

The construction phases which were analyzed for the alternative with minimal post-tensioning are shown in Figure 5.22. With no further post-tensioning being added on the construction site the rigid connection between the superstructure and substructure could be obtained in a relative early stage, to be precise immediately after mounting. This step would change the static system from two single span beams to a double-span rigid frame with enhanced load bearing behavior.

In the following construction phases the girders would be filled with concrete in two separate steps, followed by the casting of the deck slab. Once the bridge was finished the remaining loads could be applied to the structure. As was the case for the design without post-tensioning of the L21 Overpass, the compliance with the ultimate limit state was decisive for all calculations.

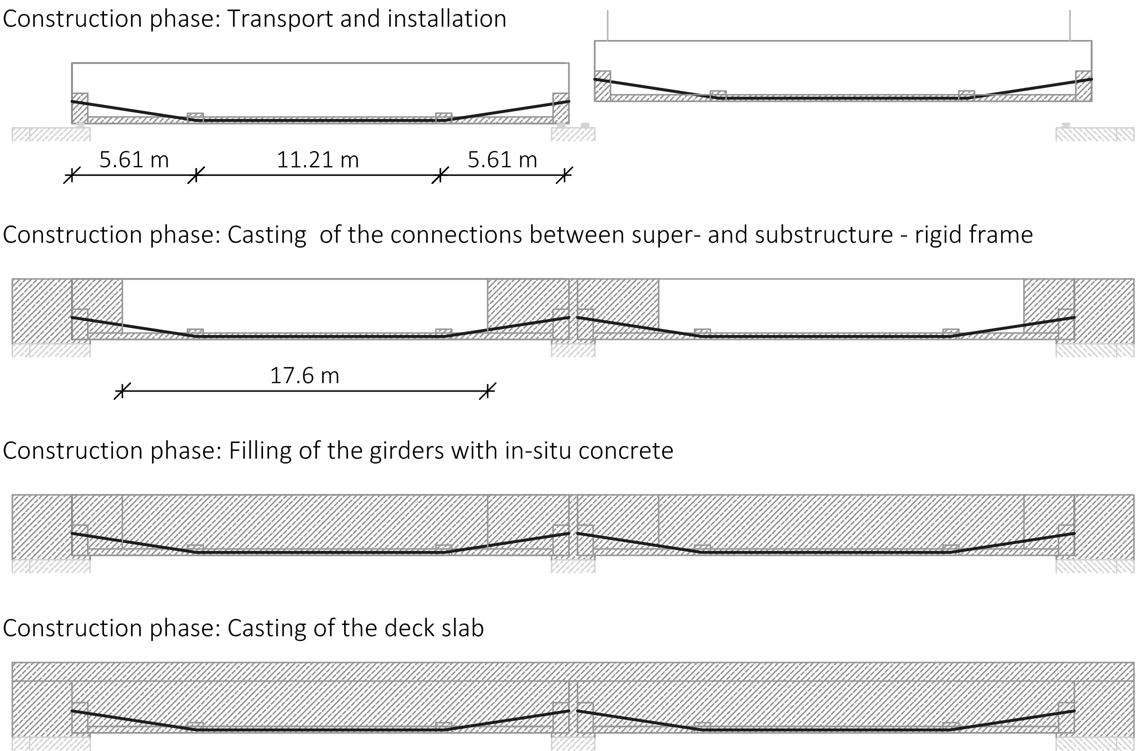


Fig. 5.22: Construction phases of the redesigned A2.Ü22a Overpass alternative with minimal post-tensioning

For the second design alternative an approach similar to the one chosen for the parameter study was looked into. The construction phases were very similar to those of the redesign of the L21 Overpass and the parameter study. The only difference lied in the additional tendon that was installed for the bridge loads and the final state. The tendon elevation for each construction phase are shown in Figure 5.23. A total of 51 strands were needed to uphold all requirements according to recent standards. When comparing the girder height and the amount of strands to the results of the parameter study shown in Figure 5.13, keeping in mind the 100 mm wider girder and 2.50 m wider carriageway, a good correlation can be noticed.

The necessity of adding an additional tendon elevation for the bridge loads and all loads applied to the final state can be called into question. The reason behind this adjustment was the broadened corner detail. The thin-walled precast girders were set very close to the edge of the abutment wall in order to facilitate the installation of the required connection reinforcement. A re-stressing of the tendons from the deck slab casting construction phase would have therefore been either very complicated or mere impossible.

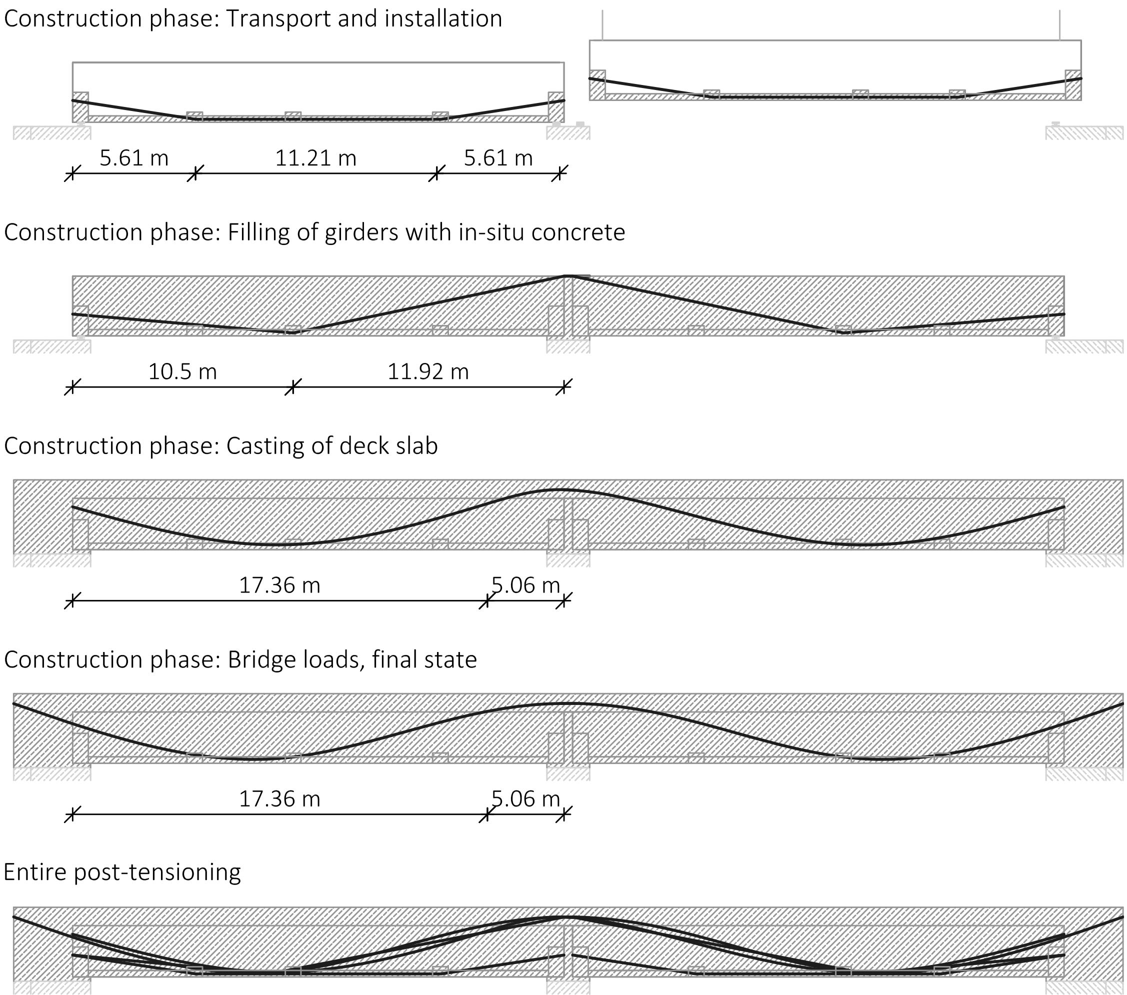


Fig. 5.23: Tendon elevation in each construction phase of the redesigned A2.Ü22a Overpass

5.3.3 S103 Overpass- A1 Motorway - Redesign

For the S103 Overpass, which is shown in Figure 5.3 and described in detail in section 5.1.3.2, a design alternative for the production of an integral concrete bridge using thin-walled precast concrete bridge girders, based on a rigid frame system as shown in Figure 5.1 Single-span system a), was investigated. The works on the redesign were very simple, keeping the entire considerations to a mere theoretical basis. The original design of the bridge is rather unusual for a simple motorway overpass and not fitting the profile of the other reviewed bridges nor the parameter study. Since the static system as well as the abutment area differed so greatly from the other overhauled bridges it was decided to stay true to the existing bridge in the cross-section redesign, resulting in a four webbed T-beam carriageway as shown in Figure 5.24. The geometry of the existing bridge abutments was adopted without change into the redesign, leaving 28.80 m to be spanned by the thin-walled precast girders. Four thin-walled precast concrete girders would substitute the four steel girders. As was the case during the construction of the existing bridge deck, precast concrete slabs would be placed on the concrete girders before the rest of the concrete deck would be cast.

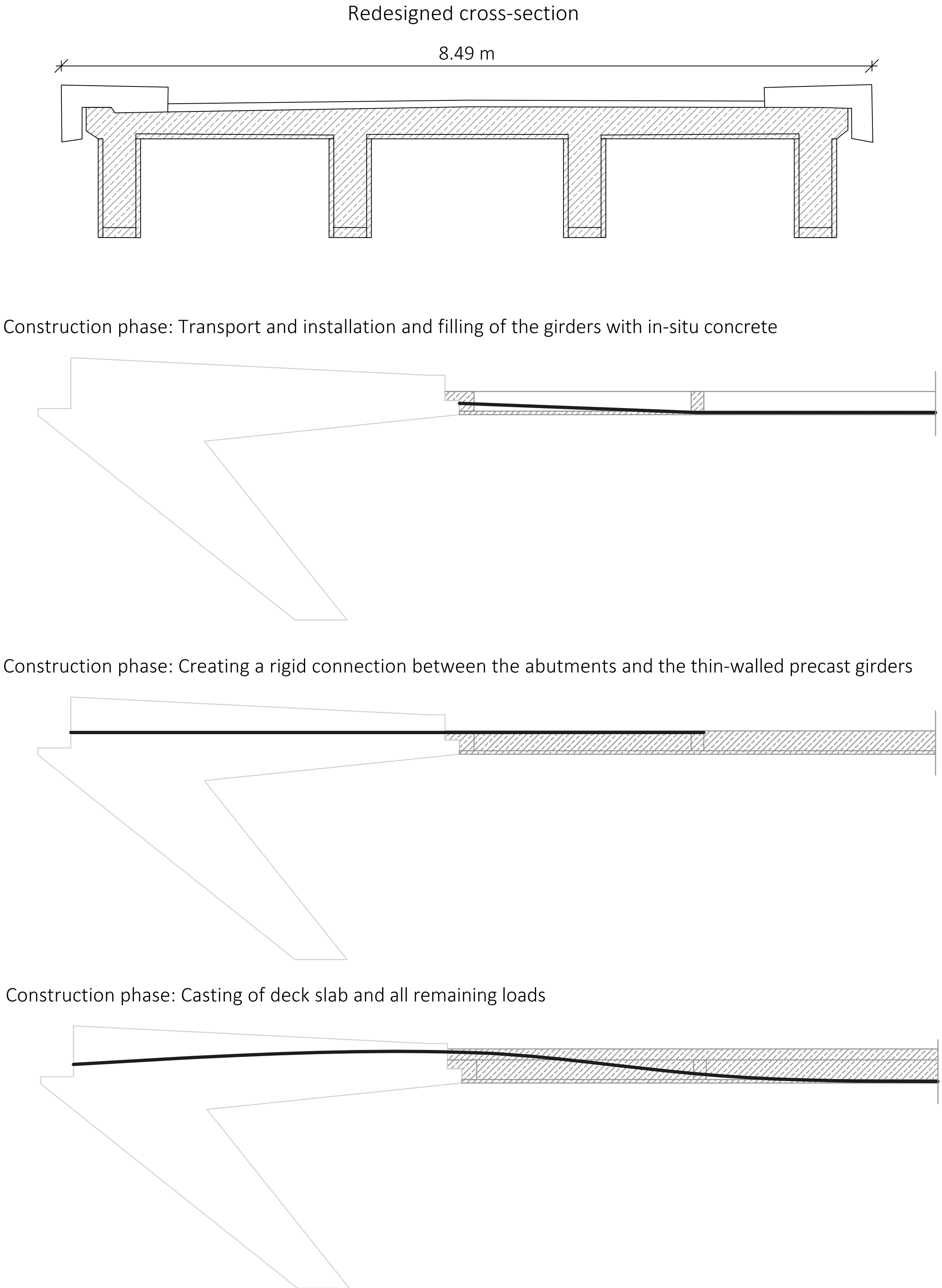


Fig. 5.24: Proposed cross-sections and tendon elevation in each construction phase for the redesign of the existing S103 Overpass

Post-tensioning would be used to ensure that the thin-walled precast girders could resist their dead load as well as the additional weight caused by the filling with in-situ concrete. The girders would be transported with enough post-tensioning to withstand transport and mounting as was the case for all the previously described redesigns and the parameter study. The first post-tensioning would also be used to increase the load bearing capacity of the girders in order to withstand the filling with cast in-situ concrete. Additional post-tensioning would be added to connect the precast beams to the abutment creating a rigid connection of the girders to the abutments, resulting in an integral bridge. Last but not least a continuous tendon from one abutment end to the other would be stressed before the precast slabs would be placed on the girders and the deck slab would be cast. The construction phases with the matching tendon elevations are shown in Figure 5.24.

5.3.4 Connection details

In order to obtain an integral structure a rigid connection of the thin-walled precast bridge girders with the rest of the bridge structure has to be ensured. The mean to success in the case of the construction using girders out of thin-walled precast elements is the creation of a monolithic structure by casting a continuous deck slab and additionally, in cases where of post-tensioned bridges, by post-tensioning the entire finished structure.

The connection details of the thin-walled concrete precast girders with the abutments can easily be executed as would be the case for regular precast and prestressed or post-tensioned girders. Various detail solutions can be found in Alberta Transportation Bridge Structures Design Criteria (2012), Kunin et al. (2000), Dicleli (2000), Steiger et al. (2012), and Schmidt et al. (2012) with the beams resting on bearing pads, either elastomeric or grout, on the abutment stems. The connections between the precast girders and the substructure can be ensured by steel pins, as shown in Alberta Transportation Bridge Structures Design Criteria (2012), dowels as shown in Dicleli (2000), or even extended bottom deck beam strands or bonded prestressed strands in combination with grouted pins as shown in Kunin et al. (2000). As mentioned before the most important connection is the integral casting of the beams into the abutments, executed by casting the in-situ concrete deck slab and creating the ultimate rigid connection between the substructure, the precast girder and the superstructure.

The possibility of installing additional post-tensioning connecting the abutment area with the girder itself during the construction of the bridge allows an even better incorporation of the precast girders into the integral structure. Most of the needed ducts for the additional post-tensioning would be installed in the girders beforehand during their manufacturing. Any protruding ducts could easily be attached on the construction site to the existing elements before the in-situ concrete would be cast. This allows a very adaptable tendon elevation throughout the entire bridge structure.

5.4 Results and Considerations

By evaluating the results of the parameter study investigating the application of thin-walled precast concrete bridge girders in the construction of integral bridges with small and medium spans as well as the outcomes of the three redesigns of existing integral bridges a rather positive outcome can be observed. The implementation of the girders is rather uncomplicated thanks to the flexibility of the entire system using thin-walled precast elements and can therefore be recommended for the construction of integral bridges with field-spans of up to 35 m.

An important aspect that is to be considered is the fact that all proposals that were made for the cross-sections of the bridges, specifically the cross-sections of the girders, are to be seen as simple recommendations. The high number of variable parameters in the design of integral bridges constructed using thin-walled precast girders can on the one side be seen as a virtue on the other as a great challange for planners and structural engineers. A universal static calculation model is not attainable due to the fact that the design and the calculation varies immensely depending on the girder height, girder width, carriageway width, chosen bridge cross-section, bridge span, soil-structure interaction, concrete strength, amount of applied stress by post-tensioning, tendon elevation, the chosen static system, the fragmentation of the construction phases and many other parameters.

The results of the parameter study show the height-span-ratio of thin-walled bridge girders for a 12 m wide carriageway with a double-webbed T-beam cross-section. The area of application of the girders for the construction of double-span integral bridges was verified for field spans of 15 m to 35 m. When staying in a realistic height range of 1.50 m for the girders heights a span of 25 m would be attainable.

With the parameter study only concentrating on one cross-section, one static system and a highly post-tensioned design, further design parameters were investigated in the redesigns of the three existing bridges. The following possibilities were looked into: no post-tensioning, minimal post-tensioning, widening of the girder cross-section at the pier, concrete hinges at the intersection between superstructure and substructure resulting in a different static system, four-webbed T-beam cross-sections as well as altered tendon elevations.

The redesigns of existing bridges, in the post-tensionless redesign of the L21 Overpass, showed that the construction of integral bridges with thin-walled precast girders without post-tensioning is possible but not recommendable for spans as large as 30 m. The necessity of an auxiliary pier contradicts the actual reason behind the construction using precast elements, which are used in order to maintain the traffic flow underneath without obstructing it by say supports or any scaffolding. The design alternative for the A2.Ü22a Overpass using minimal post-tensioning on the other hand seems to be a rather intriguing modification of the design approach. In this case the traffic flow would not be obstructed and no further post-tensioning would be added on the construction site, simplifying the construction process.

The calculations of the L21 Overpass redesigns showed that the incorporation of concrete hinges into the designs and the widening of the girder cross-section at the pier could be used to obtain slightly lower reinforcement amounts by lowering the resulting moments. It remains doubtful, however, if the additional effort needed to construct the concrete hinges as well as the girder widening would be justified by the saved costs on additionally needed reinforcement.

The four-webbed T-beam cross-section which was chosen for the redesign of the S103 Overpass could also be a very reasonable cross-section that should be investigated in further parameter studies. This would simplify the casting of the bridge deck slabs since, as was the case for the existing S103 Overpass and its redesign, precast concrete slabs could be used between the girders and no cantilever suspension system would be necessary to cast the bridge deck.

An additional optimization of the construction process presented in this chapter that be should looked into is the consideration of reducing the filling of the thin-walled precast girders with in-situ concrete to a one step process. Up to this point the girders would always be filled in two steps based on the low load-bearing capacity of the thin-walled U-shaped cross-sections. The possibility of enhancing the bottom slab of the U-shaped elements or using lightweight concrete as filling concrete should be considered. Through theses measures, a construction step on the construction site could be omitted, thus saving construction time and further costs.

In conclusion, it should be emphasized that building with thin-walled precast concrete girders can be considered as very flexible and suitable for the production of integral bridges with short and medium spans. The high degree of prefabrication of the precast girders, considering that most of the reinforcement and needed post-tensioning is installed in the manufacturing plants, shortens the construction time on site. With the girders coming straight from the manufacturing plant, where the construction is weather independent and of highest quality, a high durability of the elements can be expected. By replacing more expensive steel girders or concrete bridges with more massive cross-sections a substantial cost-efficiency of the thin-walled girders is indisputable. Nevertheless, the increase in design and calculation costs should always be factored in when considering the construction with light-weight girders.

6 Experimental tests resulting in a prototype of a bridge girder out of thin-walled precast elements for large bridges

In order to erect bridges with large spans using lightweight thin-walled precast concrete girders, sturdy box-cross-sections were developed at the TU Wien. The objective, as it had been for the development of the cross-sections for bridges with smaller spans, was the creation of a girder made of precast elements light enough for transport and erection by conventional transport and lifting equipment. As has been the case for the smaller cross-sections, the precast elements would serve as permanent formwork for the subsequently added in-situ concrete, therefore considerably reducing the necessary formwork and falsework on the construction site. Similarly to the building procedure with the small cross-sections, once the lightweight girders were mounted in their final position on the construction site the missing or hollow parts of the box-sections would be cast with in-situ concrete creating a massive monolithic structure.

A cross-section with plausible dimension was taken as a template for the development of the new lightweight cross-section. The cross-section was dismantled into its various components which were then analyzed. The load-bearing capacity of each element was evaluated with the objective of reducing the elements to their minimum weight while maintaining enough stability. It was decided to use customary double wall elements for the webs and thin walled lattice-girder floor slabs for the bottom and deck slabs of the box-sectioned prototype. By repurposing these standard prefabricated elements it was possible to not only reduce the weight of the girders but also create a very cost efficient construction method for bridge girders. The developed girder cross-section is shown in Figure 6.1 with the lightweight box-cross-section featured in dark grey.

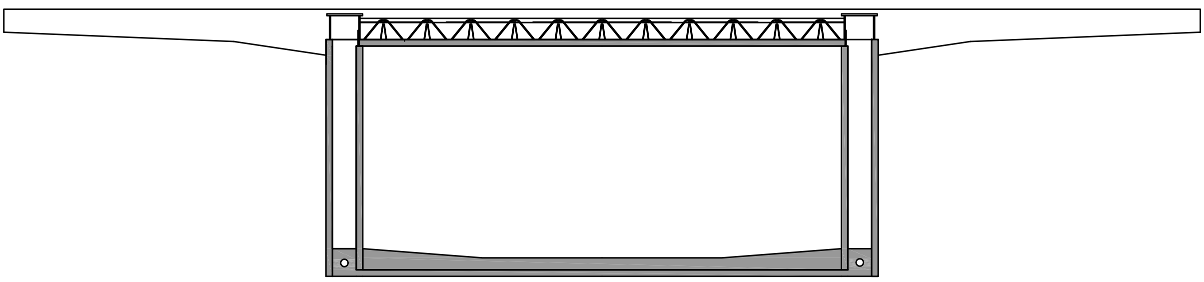


Fig. 6.1: Cross-section of the developed bridge girder using double wall elements and thin-walled precast elements for large multi span bridges with the lightweight box-cross-section shown in dark gray

The webs and deck slab would be prefabricated in a production plant, afterwards transported to the construction site where all pieces would be assembled and the bottom slab would be cast creating the lightweight box girders. Since the individual parts of the cross sections have regular measurements, conventional trucks could be used for transportation. On the construction site the individual elements would be pieced together creating a cross-section with a desired length that could then be mounted to its final location.

The production of two full-scale prototypes was commissioned with the goal of testing the feasibility of the construction method. During the production and the assembly process valuable insights into various possible improvements were made. Once the feasibility was proven, the cross-sections were tested for their stability.

6.1 Fabrication of the prototype

The main goal for the developed bridge girders was the creation of an alternative cross-section that would be able to compete with the span-weight ration of steel girders. By redesignating standard prefabricated elements a box-sectioned bridge girder, as shown in Figure 6.2, was developed with a height of 2.85 m and a width of 6.0 m. Since, in order to stay economically justifiable, it was decided to only use standard prefabricated elements, certain boundary conditions for the element measurements had to be met. The fabrication process of the two 2.0 m long prototypes will be outlined based on the underlying considerations for the design of each structural element.

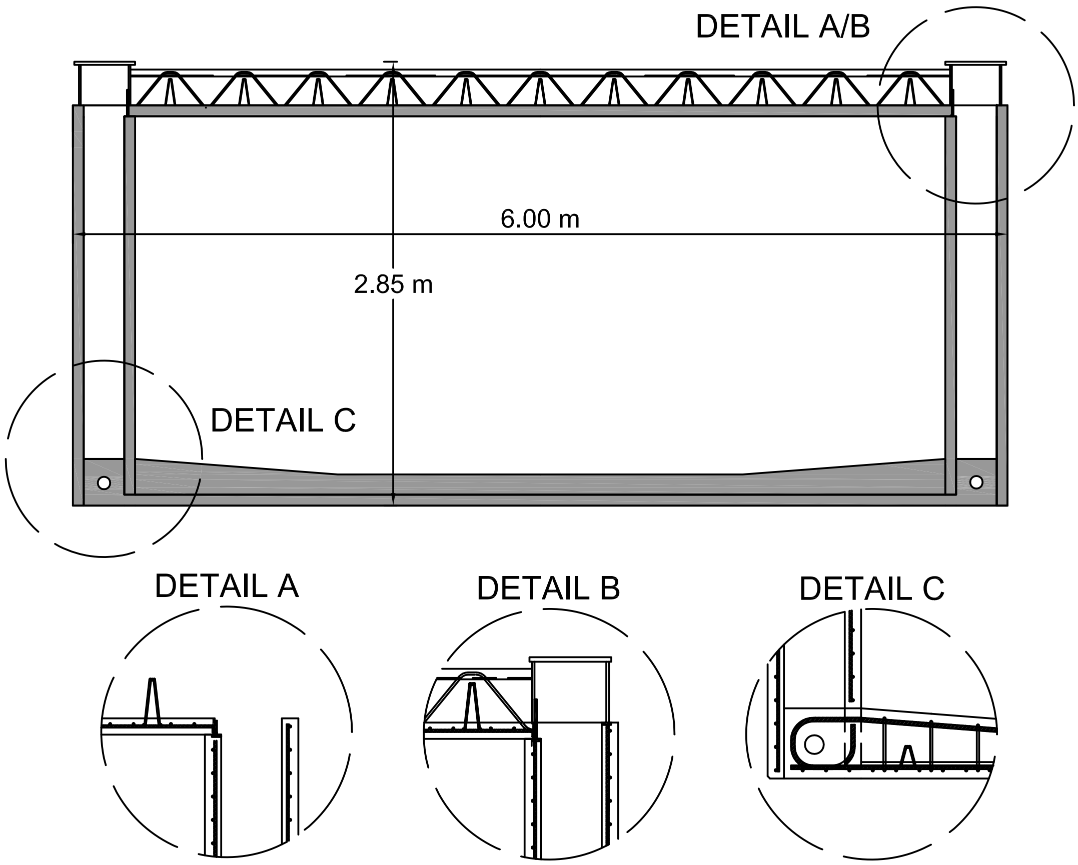


Fig. 6.2: Prototype cross-section with Details A through C showing the connections between webs and slabs of the cross-section

In order to test the feasibility of the designed cross-sections the two full-scale prototypes where produced in the exact manor as would be the case for an authentic bridge segment out of thin-walled precast elements. The double wall elements that would make up the cross-sections webs were produced in a production plant that specializes in the automatic manufacturing of double wall elements. Simultaneously the deck slab was cast in a second plant that specializes in the production of slabs and prefabricated staircases. The assembly of the cross-sections took place in a secluded area of the second manufacturing plant.

As mentioned before certain restrictions for the element measurements had to be considered. Owing to the fully automatic manufacturing process, the maximum width of the double wall elements is limited to 0.50 m, and the maximal height and length are limited to 3.40 m and 12.0 m respectively. In addition to the mentioned restrictions, it must be noted that all reinforcement and required ducts for subsequent post-tensioning must be attached to the reinforcement cages before the double wall elements are cast, due to the limited space between the two slabs of the wall elements. As can be gathered from the featured cross-section and all three details in Figure 6.2, the inner walls of the double wall elements were designed and manufactured slightly shorter than the outer walls. By this measure a rigid connection between the bottom slab and the webs could be achieved. The reinforcement of the concrete stabilization beam is also placed through the inner wall of the double wall elements, as shown in Detail C of Figure 6.2, to strengthen the connection of the u-shaped section.

Once the four webs for the two prototypes where manufactured they were transported to the assembly site. The reinforcement cages for the bottom slabs where placed on the casting tables and the webs were, as shown in Figure 6.3, positioned upright on each end. The two designed prototype-sections differed only in the kind of separation-reinforcement in the double wall elements (see Figure 6.3). One set of double wall element was manufactured using conventional lattice-girders, the second set used KAP-steel-waves (KAPPEMA, 2011).

Fig. 6.3: Vertically placed webs of the box-girder section with installed reinforcement cage for the bottom slab. The difference between the reinforcement cages of KAP-steel-wave webs and lattice-girder webs.

With the upright placed wall elements the 70 mm thick bottom slab, including the concrete stabilizing beam, was cast creating a u-shaped cross-section (see Figure 6.4). After the concrete hardened the supports of the webs were removed. In this context it is important to say that the length of the bridge girder segments is not limited by the measurement restrictions of the double wall segments. The double wall elements can be put side by side before the bottom slab is cast, therefore creating a bridge girder segment with any desired length.

Fig. 6.4: Finished u-shaped cross-section and deck slab with stabilizing beam on casting table

The finished 70 mm thick deck slab, with a modified lattice girder using a steel stabilizing beam is shown in Figure 6.4. When looking at the cross sections of both the deck (Figure 6.5) and the bottom slabs (Figure 6.6), the difference of the used stabilizing beams is visible, one being a lattice-girder with a u-shaped steel upper flange, the other a concrete beam. It must be noted that in contrast to the usual placement of the lattice-girders in standard precast slabs, the standard lattice-girders are placed in the transverse direction of the finished slabs, thus increasing the load bearing capacity of the slabs. The overall higher loads in bridge construction in comparison to regular high-rise construction justify the increased installation (every 475 mm) of lattice-girders in the slabs.

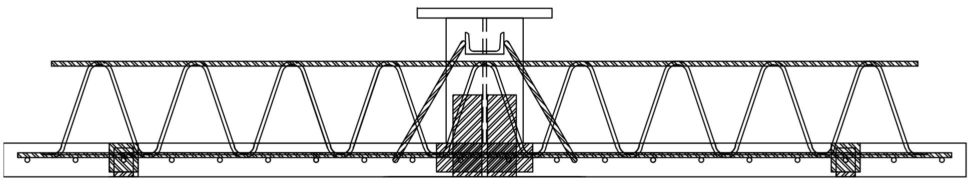


Fig. 6.5: Cross-section of the deck slab with a steel stabilizing beam and the embedded steel plates intended for the connection with the girder webs

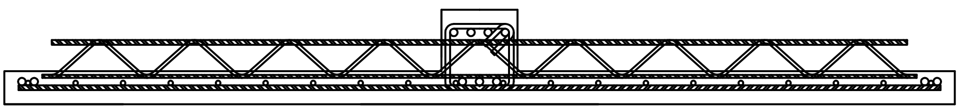


Fig. 6.6: Cross-section of the bottom slab with a concrete stabilizing beam

The steel profile used to achieve the rigid connection between the deck slab and the webs of the cross-section, as shown in Detail B of Figure 6.2, is clearly portrayed in Figure 6.4. The profile connects the steel stabilization beam with both walls of the double wall elements enabling a good distribution of all stresses.

The assembly of the two lightweight thin-walled box-sectioned bridge girder prototypes was completed when the deck slabs were connected with the u-shaped sections. After the deck slabs were placed on top of the inner walls of the webs the embedded steel plates in the webs and slabs as well as the steel profile were welded together (see Figure 6.7). The finished prototype with the KAP-steel-wave webs and a clearly visible concrete stabilizing beam of the bottom slab is shown in Figure 6.8.

Fig. 6.7: Welded rigid connection between deck slab and webs

Fig. 6.8: Finished box-sectioned bridge girder out of double wall elements and thin-walled precast elements

6.2 Load testing

After the feasibility of the commercial production of the lightweight cross-sections was proven, full-scale tests were used to provide valuable insight into the structural stability of the girders, with special attention payed to all connection elements and detail design. Additionally, the weight of the produced prototypes was compared to the calculated weight of the designed cross-section.

The experimental setup of the tests is shown in Figure 6.9 and was based on the load distribution during all transport and lifting operations.

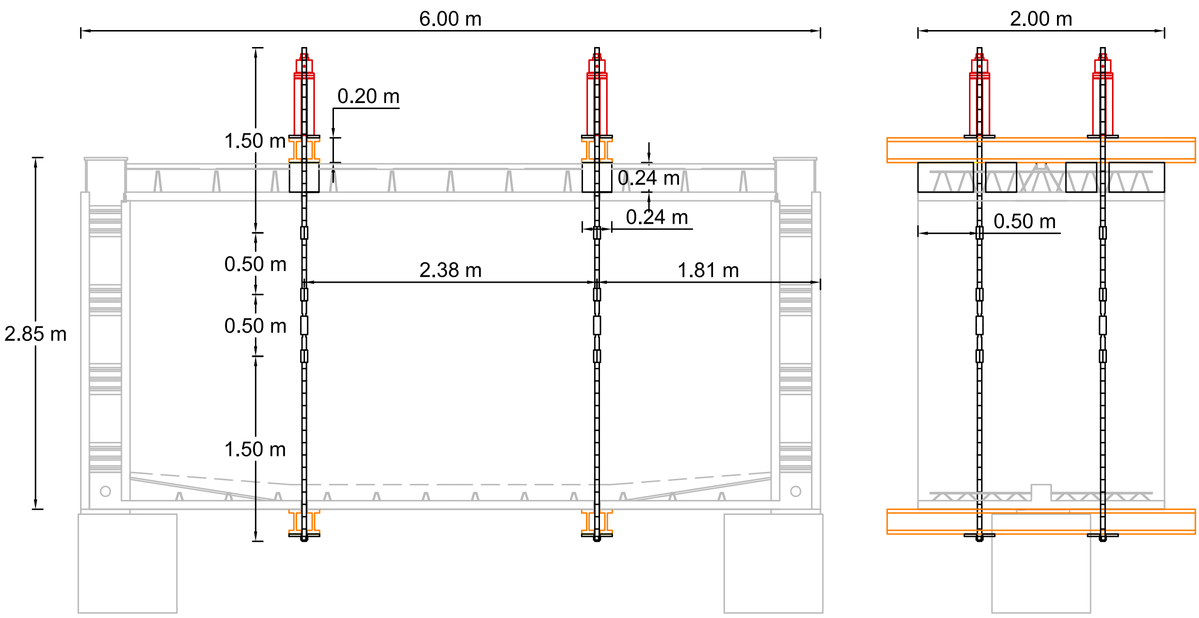


Fig. 6.9: Test setup for the structural stability tests in front view and section A-A

The bridge girder sections were placed on three bearings and the load was applied by four hydraulic jacks. A linear load distribution was achieved by placing the jacks on wooden beams. The deflection of the deck and bottom slab were measured using a set of linear variable differential transformers of the type Solatron BS25 (hereafter referred to as LVDT's). Each prototype was instrumented with 18 LVDT's during the testing, of which nine LVDT's, named W1 through W9, measured the deflection of the deck slabs and further nine, named W10 through W18, measured the deflection of the bottom slabs. The positioning of the LVDT's W1 through W18 is illustrated in Figure 6.10.

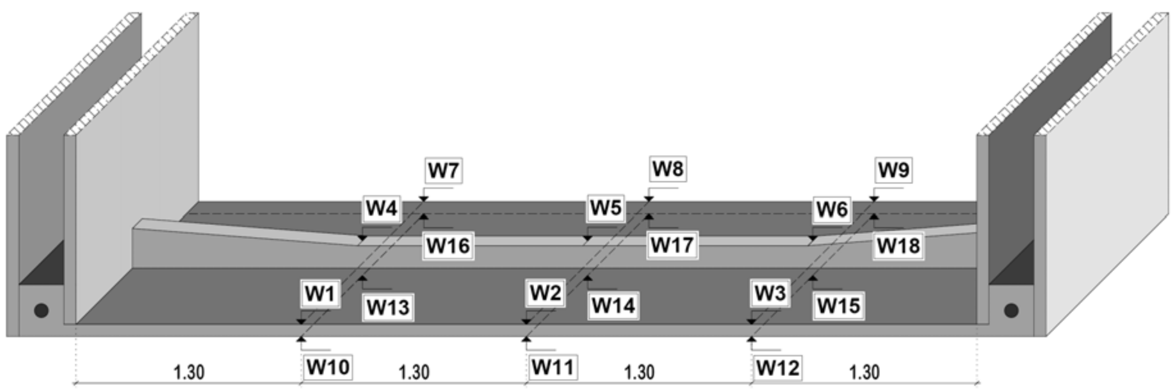


Fig. 6.10: Placement of the linear variable differential transformers in the structural stability tests (Hausleitner, 2014)

6.3 Results and considerations

The large-scale tests showed the feasibility of the commercial production of box-cross-sectioned lightweight bridge girders out of thin-walled precast elements and double wall elements. A weight deviation between 5.84% and 6.30% was calculated between the designed cross-section and the manufactured prototypes. The envisaged 4.35 t/m were surpassed by 0.25 t/m and 0.27 t/m by the lattice-girder prototype and Kap-steel-wave prototype respectively. This discrepancy is due to the fact that the bottom slab was not cast with the exact thickness of 70 mm. The weight fluctuations must be considered when choosing transportation and lifting equipment for future projects especially when thinking of working with longer thin-walled sections.

As was the case for the very similar weight outcome, the two prototype cross-sections demonstrated a very uniform load bearing behavior. With the goal of the loading tests being the evaluation of the stability of the girder sections for the lifting and transportation process the experiment was stopped at a load of 45 kN.

The mean of the deflections of the deck slab in correlation with the load is shown in Figure 6.11. The force-deflection graph shows the mean of the measurements of the LVDT's W1, W4 and W7 as "Left", the mean of the measurements of the LVDT's W2,W5 and W8 as "Middle" and the mean of the measurements of the LVDT's W3, W6 and W9 as "Right". At a load of 45 kN the maximal measured deflection of the deck slab was approximately 12.0 mm.

Based on these results a stiffness of the lightweight box-sectioned thin-walled bridge girder can be calculated to $EI = 7{,}9\,MNm^2$. The results confirm that the elements have enough stability to be transported and installed. Even though the subsequent construction steps after the mounting of the cross-section on the construction site (filling of the webs with concrete and casting the rest of the bottom slab) strengthen the cross-section immensely it can be said with certainty that extra support of the deck slab will be needed while the deck slab is being cast.

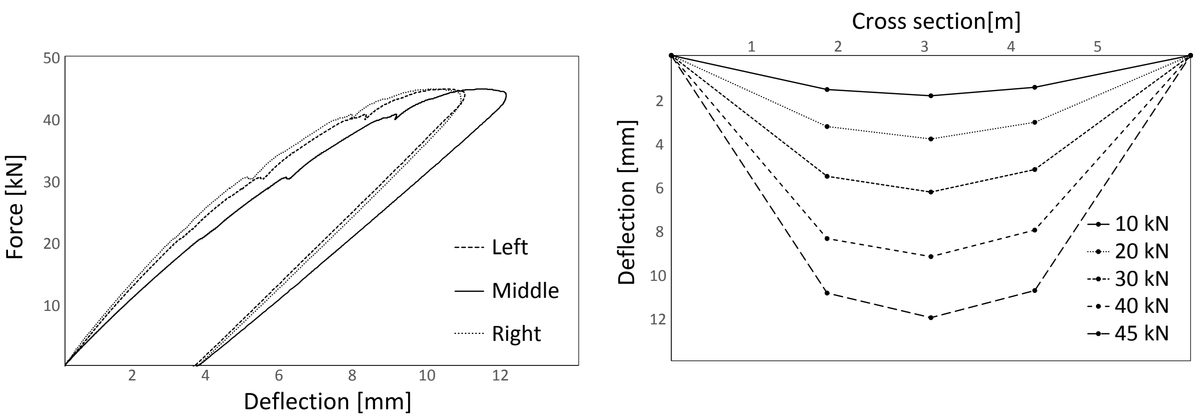


Fig. 6.11: Deflection of the deck slab during the structural stability testing

With the results of the two large-scale tests a static calculation model, as shown in Figure 6.12 and described in detail by Hausleitner (2014), was calibrated to allow reliable predictions for further examinations of cross-sections with variable dimensions. Furthermore the calculation model was used to single out deficiencies of the individual components of the cross-sections.

Attention was paid to a realistic modeling of the separate components of the structural elements. The deck-slab was divided into four main elements: the upper flange of the stabilizing beam, the transverse truss elements, the bottom flange being the precast concrete slab with the lattice-girders as well as the I-profile serving as the connection of the deck slab to the webs. The webs were modeled as two concrete plates connected by transverse truss elements. The bottom plate with the concrete stabilizing beam was modeled as a T-beam with the consideration of the effective width of the slab.

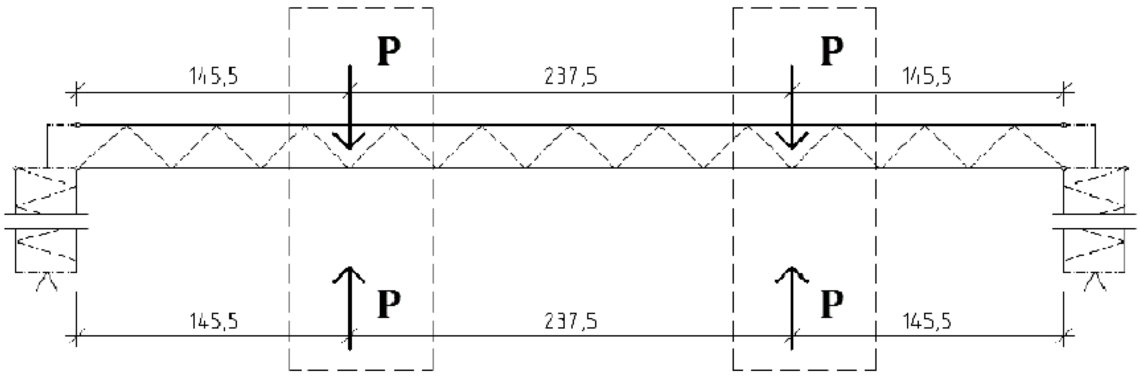


Fig. 6.12: Static calculation model created based on the results of the two large-scale structural stability tests(Hausleitner, 2014)

The calculation showed that up to a load of approximately 30 kN the cross-section behaved linear-elastic before the system slightly softened due to the transition into a cracked state. Starting at 40 kN the transverse elements of the deck slab start buckling increasing the deflection rapidly.

Valuable insight was taken from both the large-scale testing as well as the static calculation model. These new findings would have to be considered for the future design of the cross-sections. The transition of the deck slab into a cracked state should be prevented if possible even though it only softened the system marginally. The more important aspect is the aversion of the buckling of the transverse elements of the deck slab. The complete system failure could easily be obviated by using thicker rebars as transverse beams. Furthermore the calculations showed that the steel u-profile used as the stabilizing beam of the deck slab were only stressed by normal forces, therefore a replacement with a flat steel beam with a similar cross-sectional area should be considered.

7 Application of the developed lightweight cross-section in design competition

A design competition for the construction of two bypass bridges for the Voest bridge across the Danube river in Linz, Austria was issued in late 2013. The engineering firm FCP - Fritsch, Chiari & Partner ZG GmbH (hereafter referred to as FCP) decided to engage in this competition, supported by the architecture firm Treusch architecture ZT GmbH and the Institute of Structural Engineering, in specific the Department of Structural Concrete, of the TU Wien. The favourable results of the large scale tests on the prototype and the novelty of this construction method for large bridge girders intrigued the engineering office FCP, which was looking for an innovative new approach for their competition entry.

7.1 Competition tender for two bypass bridges for the Voest bridge in Linz, Austria

As described by Burgholzer et al. (1973) the original 34.86 m wide Voest bridge was completed, after three years of constructions, in 1972 and offers a total of six lanes, with three lanes in each direction. The main focus-point is the 65.0 m high pylon with three parallel harp-shaped suspensions on both sides allowing, as shown in Figure 7.1 and Figure 7.2, the bridge to freely span 215 m over the Danube river. The rest of the bridge is divided into three spans with significantly smaller lengths of two times 60.0 m and one time 72.0 m, resulting in a final bridge length of 407 m. On the one side of the pylon the suspensions are anchored in the cross-section, on the other side in the pillars and the abutment.

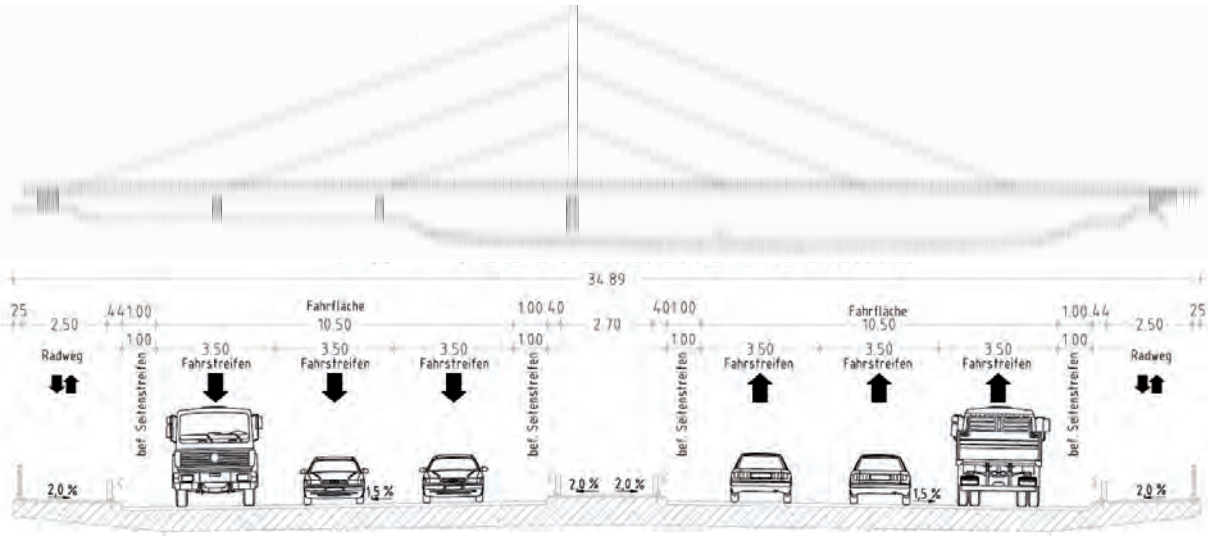


Fig. 7.1: Side view (top) and cross-section of the deck slab (bottom) of the original Voest bridge from 1973 (ASFINAG, 2013)

For the steel cross-section, with a varying height between 3.76 m and 5.49 m and an orthotropic deck, 5,800 tons of steel had to be transported to the assembly site. The bridge was built using the cantilever method with a section length of 12.0 m. The construction started with the three short spans in the foreland area. When the third pillar war reached and the pylon built the 215 m span was built with the help of the suspensions.

After 40 years of continuously rising traffic volume, which had reached its peak of over 100,000 vehicles per day, the Voest bridge had exceeded its limits in 2013. Austria's motorway and expressway agency ASFINAG - Autobahnen- und Schnellstraßen-Finanzierungs-Aktiengesellschaft (hereafter referred to as ASFINAG) decided to fully rehabilitate the bridge, therefore needing it to be closed for traffic. The plan was set for the construction of two bypass bridges, which would maintain traffic flow during construction and afterwards distribute it evenly to allow a traffic reduction on the Voest bridge. After the renovation works the Voest bridge would only carry two lanes plus a service lane.

Fig. 7.2: The original Voest bridge from 1973 (ASFINAG, 2013)

ASFINAG put the bypass project out to tender in November 2013. The boundary conditions for the participation in the design competition were set in the competition tender documents (ASFINAG, 2013). The deck slab of each bridge had to incorporate two lanes for traffic, a bike path and a sidewalk, resulting in a width of 16.0 m. During the renovation of the Voest bridge each bypass bridge had to carry three traffic lanes.

The cross-sections of the two bypass bridges and their placement next to the original bridge as shown in Figure 7.3 were also specified in the competition tender documents. ASFINAG insisted on a minimum distance of 6.0 m between the original Voest bridge and the new bypass bridges, ensuring enough space for any type of renovation works. Furthermore the river navigability had to be upheld during the entire construction process; therefore, the area of 8.0 m above high water level over a river-width of 100 m had to be kept unobstructed. A noise abatement concept was not part of the requirements of the design competition but it was stated that a possibility for the integration of such measures had to be incorporated into the finished proposal.

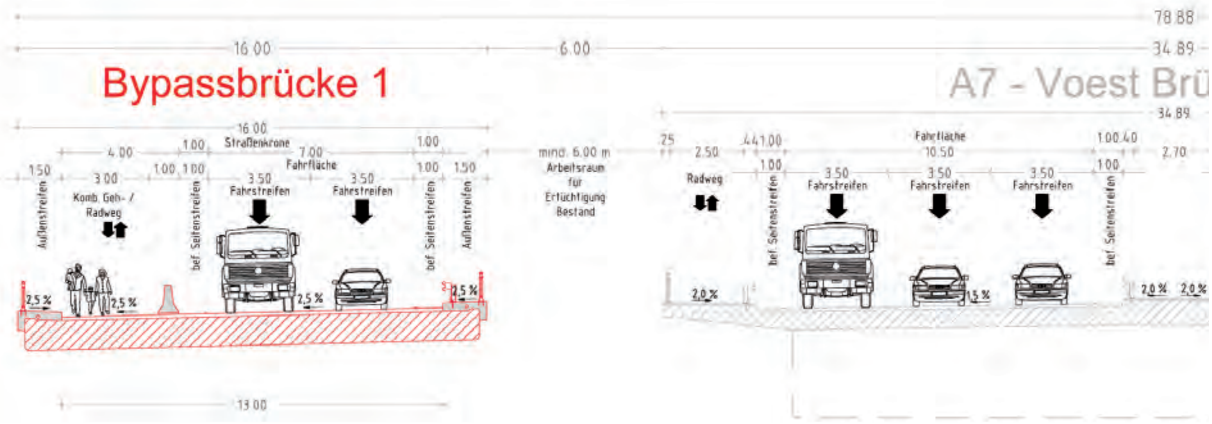


Fig. 7.3: Cross-sections of the deck slabs of the original and the bypass bridges according to competition tender documents (ASFINAG, 2013)

The construction of the bypass bridges was set between 2017 and 2019. Subsequently, the rehabilitation of the Voest bridge would be completed by 2020, with the goal of unrestricted traffic flow over all three bridges by 2021.

The submitted competition entry documents had to include a site plan of the bridge, section of all major elements, schematics of the designed bridges, an estimation of the construction costs and a technical report describing the creative concept, the bridge equipment, production method, mass and all pre-structural calculations.

7.2 Redesigning the prototype cross-section for application in the bypass bridges

When building with girders out of thin-walled precast elements the construction phases strongly influence both the designing, planning as well as the calculation process. In order to obtain a suitable cross-section for the designing competition the original prototype cross-section had to be adapted since a deck slab width of 16.0 m would not have been achievable. In order to obtain the required width all dimensions had to be altered. The standard cross-section, with a height of 3.40 m and a bottom width of 7.50 m, leaving 4.0 m cantilevers on both sides, is shown in Figure 7.4. The height of the applied cross-section would be enlarged to 5.65 m over the pier. In comparison to the prototype the standard height was increased by 0.83 m, the width by 1.50 m and each cantilever by 0.50 m.

Additional to the changes of the dimension of the cross-section the stabilizing structure of the deck and bottom slabs was optimized based on the results of the large-scale testing and the static calculation model. The placement and installation of the connection reinforcement between the webs and deck and bottom slabs was revised and enhanced resulting in the decision to use steel beams, as was the case for the deck slabs in the prototype, instead of the concrete beams, for the bottom slabs.

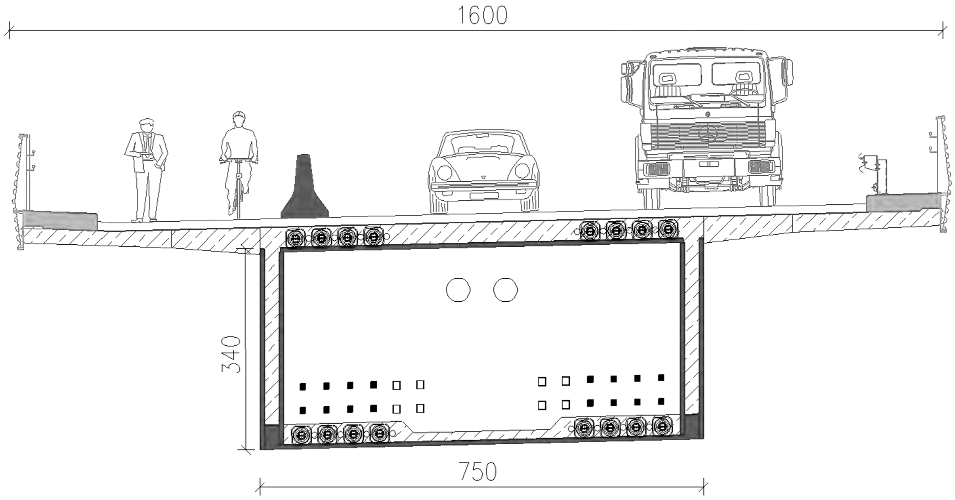


Fig. 7.4: Standard cross-section for the bypass bridges over the Danube river in Linz. Lightweight box-sectioned bridge girder out of double wall elements and thin-walled precast elements shown in dark gray (FCP, 2014)

7.3 Bridge design used in the design competition entry

In the beginning of the design process two versions for the bridge structure where looked into. With the Voest bridge seen as an inspiration, a cable-stayed bridge option was considered, but quickly discarded in favor of a bridge design without any design elements above bridge level solemnly with a haunch at the main pier. This was justified by the fact that in this version, the design of the original Voest bridge, with its single pier and three cables on each side, would not be interfered with, Figure 7.5 shows that the haunches of the new bypass bridges would even emphasize the pier as a focus point.

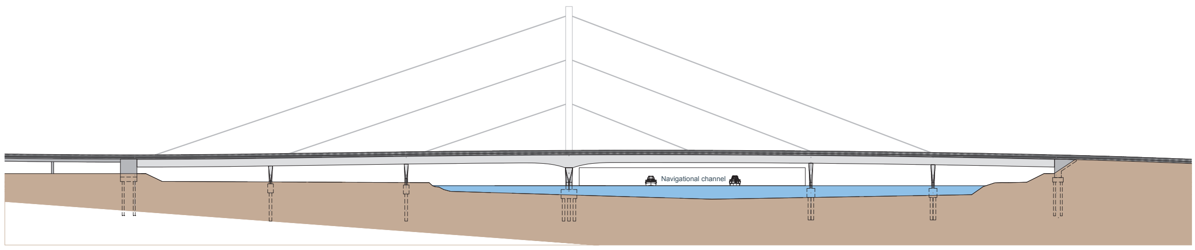


Fig. 7.5: Side view of the Voest bridge including the designed bypass bridges with the haunch placed directly below the main pier and the two additional pillars (FCP, 2014)

Even though the large span of 215 m is one of the main achievements of the engineers that designed the Voest bridge, it was not economically justifiable from today's point of view for the bypass bridges to maintain this large span. The total span of 407 m was divided into six spans with a main field length of 107 m, which would uphold the required 100 m clearance for navigability of the Danube river. The goal was to set subtle pillars in a way that would not affect the overall picture of the Voest bridge. The solution, as can be seen in Figure 7.5, was clear and simple, the additional two pillars would be set at the anchor-points of the suspensions of the Voest bridge. The connection of the bridge with the pillars was to be carried out integrally reducing the maintenance and repair costs by the bearing-free production.

When designing bridges with the intention of using bridge girders out of thin-walled precast elements the construction phases are one of the main focus points during the whole design process. They strongly influence both the designing, planing and the calculation process and must be well thought through. All previously described alterations of the cross-section went hand in hand with the precise specification of each construction phase. The most rational solution was dividing the construction process into ten phases. The construction progress of the first nine construction phases is shown in Figure 7.6. The tenth construction phase consists of the concreting of the bridge deck cantilevers, the installation of the transverse post-tensioning of the bridge deck, the stressing of the external post-tensioning and the installation of all bridge equipment.

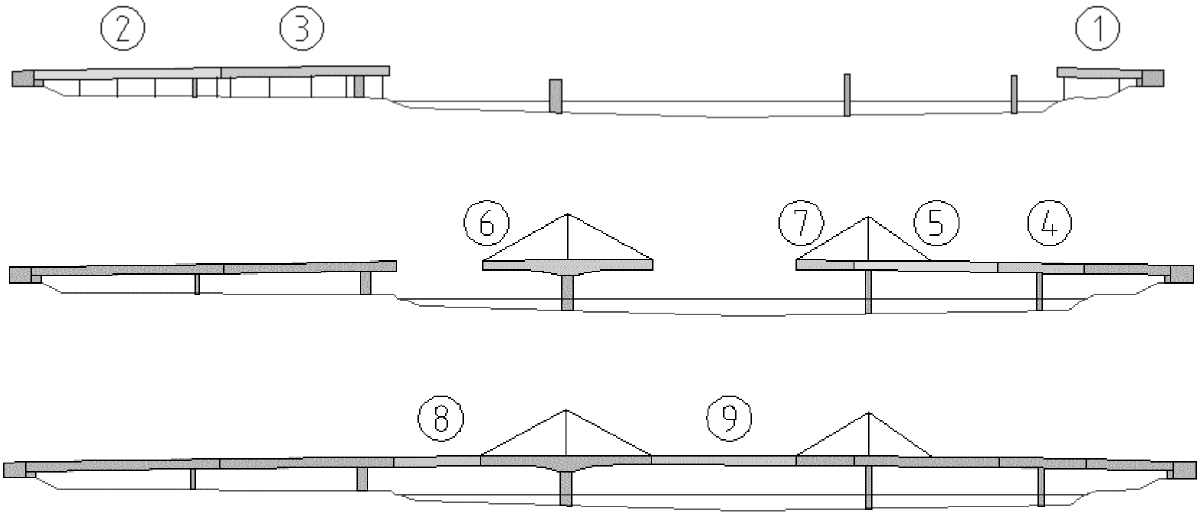


Fig. 7.6: The nine construction phases concerning the erection of the cross-sections for the bypass bridges (Hausleitner, 2014)

The first three phases would be erected using formwork. This conventional construction method seemed most reasonable due to the complexity of the cross-sections resulting from the connections with all the ramps and the easy assembly of the formwork on the foreland of the river. The supports required for the construction would only be temporary, therefore would abide to the rules set in the competition tender documents keeping the flood discharge cross-section clear. The entire bridge girders, including the cantilevers, would be cast in one piece. After casting the short section on the south bank the two sections on the north shore would follow.

For the sections of the bridges that would span the Danube river, the lightweight bridge girder out of thin-walled precast elements would be utilized. The girder sections, with lengths varying from 20.50 m to 60.40 m and therefore weigths from 140 t to 440 t, would be assembled near the construction site before being transported to the site by ships and lifted into position using cranes. Herefore post-tensioning cables would be stressed in the webs at the assembly site to ensure the stability of the lightweight sections. The redesigned joint solution as described in Chapter 3.2 would be utilized to ensure joints with a clean finish between the double wall elements.

After a lightweight box-section bridge element would be mounted, the webs of the mounted section (double wall elements) would be filled with in-situ concrete, strengthening the cross-section immensely. Once the core of the webs hardened the works on the bottom slab could begin. The post-tensioning for the bottom slab, as shown in Figure 7.7 would be installed before the concrete would be cast in stages. The same procedure would apply for the casting of the top layer of the bridge deck with the difference that additional to the ducts for the post-tensioning of the deck slab, ducts for the transversal post-tensioning would be installed and the in-situ concrete would not be cast in stages but all at once resulting in the necessity for extra supports which would be set up between the bottom and the deck slab. The post-tensioning of the deck and bottom slab would be stressed once the box-sectioned girder had been finished. Before the following construction phase could begin all these steps had to be completed resulting in a robust box-sectioned girder that could provide enough stability for the attachment of the subsequent girder section.

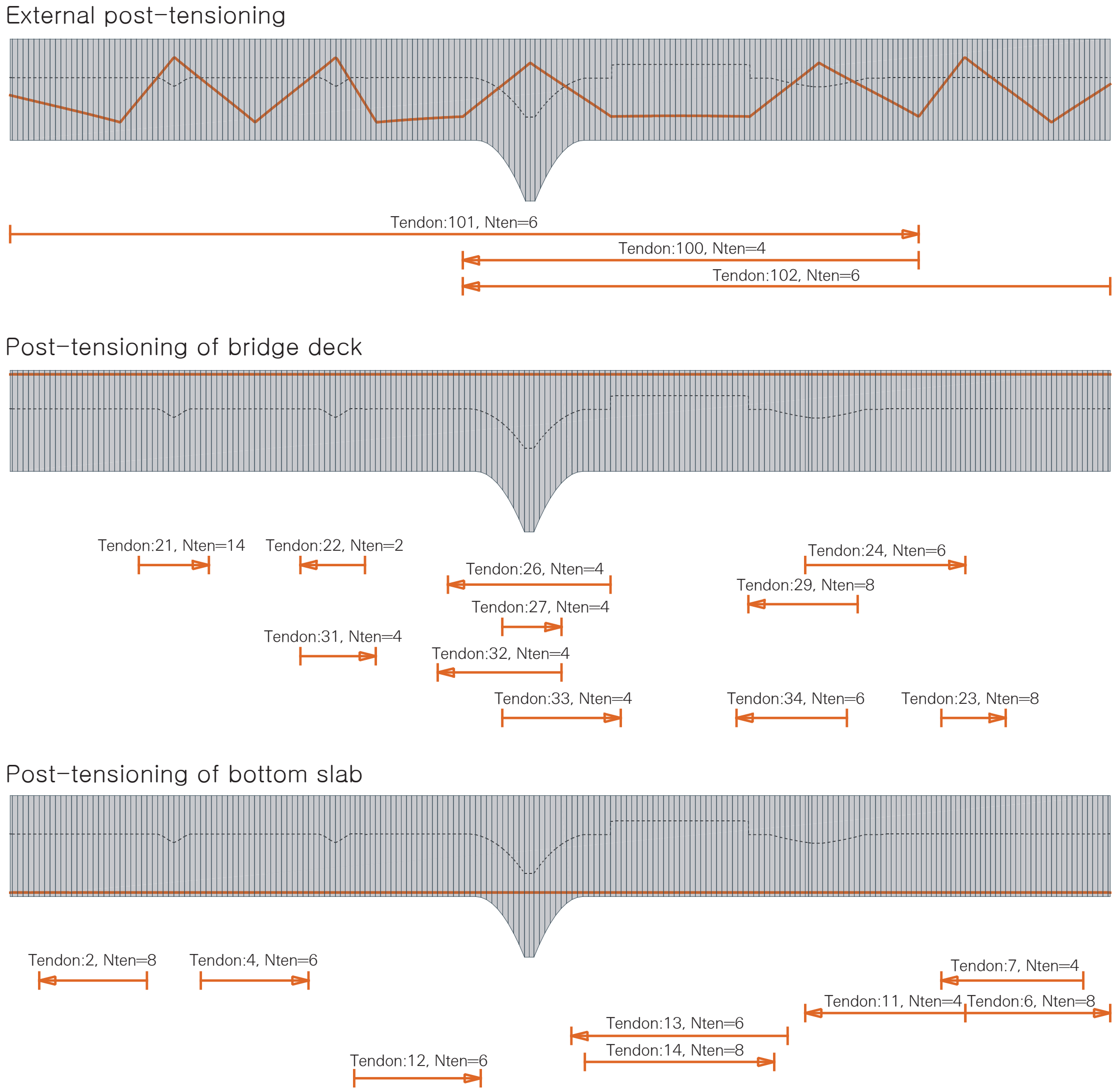


Fig. 7.7: External and internal post-tensioning of the newly designed bypass bridges (FCP, 2014)

For the construction phases six through nine the additional auxiliary piers with stay cables would be installed. As explained above the tenth construction phase would consist of the concreting of the bridge deck cantilevers, the installation of the transverse post-tensioning of the bridge deck, the stressing of the external post-tensioning and the installation of all bridge equipment.

As already described, both internal and external post-tensioning would have been applied for the construction of the two bridges. The internal post-tensioning of both the bottom as well as the deck slab would have had the sole purpose to ensure structural stability for the state of construction. The external post-tensioning would be used for traffic loads and the loads caused by the bridge equipment.

The visualizations of the bypass bridges which were part of the competition entry documents, are shown below in Figure 7.8. Unfortunately, this design did not win the design competition and will not be built. The winning design was a cable stayed bridge.

Fig. 7.8: Visualizations of the Voest bridge and the two bypass bridges (FCP, 2014)

8 Summary

The main goal of this thesis was the account of the developments in the applications of thin-walled precast concrete elements in bridge construction. The research aimed at investigating the possibility of implementing these precast elements for integral bridges with short and medium spans as well as for multi-span bridges.

The process of design development was accompanied by a thorough literature study as well as an international survey on the experiences with prefabrication in bridge construction with a focus on durability and sustainability. It was shown that the development and the popularity of precast bridges and bridges constructed using precast elements vary greatly from continent to continent or even country to country, gaining solid ground in some with market shares of up to 50% of the bridges built to market shares lower than 2% in others. Furthermore the literature study as well as the international survey showed that despite all of the advantages of precast bridge construction, as for example the reduction in construction time, minimization in traffic impact and weather-related delays as well as the increase in quality, many contractors still shy away from the application of prefabricated structural components due to their low opinion on the durability of the intersections and joints, the perceived opinion that precast bridges are monotonous and ugly or other specific technical issues (*fib* Bulletin No. 29 (2004)).

Based on the research presented by Wimmer (2016) and Gmainer (2011) the necessity of testing the grout material used for the assembly of the girders made out of thin-walled precast elements was seen as necessary. The experimental preliminary tests of grout materials included standardized and non-standardized testing determining the material parameters and comparing these to the ones stated by the material manufacturers. The results of the experiments show that not only the selection of the right grouting material is of utmost importance but that by varying the water-dry material ratio a filling material consistency perfectly suitable for the filling of the joints between thin-walled precast elements can be obtained. In addition to the grout testing new joint designs for joints with and without post-tensioning were developed, produced and tested. The visual appearance of the joint design with post-tensioning according to Wimmer (2016) was improved without reducing any of its structural strength, therefore creating a cleaner finished version of the already accomplished joint execution.

With fatigue being a very prominent issue when it comes to bridge construction, experimental tests of fatigue behavior of thin-walled precast elements were conducted on large-scale specimens. With the results of eight fatigue tests and two load-bearing tests it is thus possible to make certain statements on the overall issue of the fatigue behavior of thin-walled precast elements using tack welded mesh reinforcement. Despite certain problems, which occurred during the tests, the fatigue behavior of the tested specimens was better than any valid standards state. This and the successive tests conducted by K. Fuchs and Kollegger (2016) and described in detail by Berger (2017) show that the application of tack welded reinforcement in fatigue loaded elements should be considered. If reasonable measures are taken, as for example post-tensioning of the elements to avoid a cracked state, a risk of fatigue failure of the tack welded mesh reinforcement is minimized to a sheer minimum.

By evaluating the results of the parameter study investigating the application of thin-walled precast concrete bridge girders in the construction of integral bridges with small and medium spans as well as the outcomes of the three redesigns of existing integral bridges a rather positive outcome can be observed. The results of the parameter study showed the height-span-ratio of thin-walled bridge girders for a 12 m wide carriageway with a double-webbed T-beam cross-section. The implementation of the girders is rather uncomplicated thanks to the flexibility of the entire system using the thin-walled precast elements and can therefore be recommended for the construction of integral bridges with field-spans of up to 35 m.

The high degree of prefabrication of the precast girders, considering that most of the reinforcement and needed post-tensioning is installed in the manufacturing plants, shortens the construction time on site. With the girders coming straight from the manufacturing plant, where the construction is weather independent and of highest quality, a high durability of the elements can be expected. By replacing more expensive steel girders or concrete bridges with more massive cross-sections a substantial cost-efficiency of the thin-walled girders is indisputable. Nevertheless, the increase in design and calculation costs should always be factored in when considering building with the light-weight girders.

In order to evaluate the possibility of using thin-walled precast elements for the construction of box-cross-sectioned girders for bridges with large spans prototypes of said girders were manufactured and tested providing insight into its structural stability. By redesignating standard prefabricated elements a box-sectioned bridge girder was developed with a height of 2.85 m and a width of 6.0 m. As has been the case for the smaller cross-sections, the precast elements would serve as permanent formwork for the subsequently added in-situ concrete, therefore considerably reducing the necessary formwork and falsework on the construction site. Similarly to the building procedure with the small cross-sections, once the lightweight girders were mounted in their final position on the construction site the missing or hollow parts of the box-sections would be cast with in-situ concrete creating a massive monolithic structure.

It was decided to use customary double wall elements for the webs and thin walled lattice-girder floor slabs for the bottom and deck slabs of the box-sectioned prototype. By repurposing these standard prefabricated elements it was possible to not only reduce the weight of the girders but also create a very cost efficient construction method for large bridge girders. The feasibility of commercial production was proven and the results of the large-scale tests confirmed that the elements would have enough stability to be transported and installed on-site.

The developed large cross-sections were chosen by the engineering firm FCP - Fritsch, Chiari & Partner ZT GmbH to be used in their design of two bypass bridges across the Danube river in Linz, which were part of a design competition tendered by Austria´s motorway and expressway agency ASFINAG.

8.1 Recommendations

A small list of recommendations has resulted from the research presented in this thesis regarding the applied grouting material for the filling of the joints between the thin-walled elements, the implementation of tack welded mesh reinforcement in the elements themselves, the surface structure of the thin-walled elements, the construction phases when building bridges with small and medium spans as well as optimizations of the design of the large box-cross-sectioned girder for multi-span bridges.

For the case of the implementation of girders out of thin-walled precast concrete elements, the applied grout material should be exactly specified for the contractor and tested prior to application. Additionally the exact mixing ration should be set according to the desired flow consistency and viscosity. The flow consistency and viscosity should be checked regularly during the construction process to ensure a high-quality execution of all joints.

If tack welded mesh reinforcement were to be used in bridge construction a reduction of the tack weld points in the meshes to a minimum should be considered. The tack welding is solemnly needed for the reinforcement bars of the mesh to stay in place during assembly and subsequent casting of the concrete. The welding of every third connection point as was the case for the test girders is not necessary and should be decreased by at least a half.

The varying surface structure of precast elements was observed during the manufacturing process of the numerous specimens. If the surface is raked after compacting a very coarse and rough surface structure can be achieve, if it is not raked and compacted for too long the concrete slag rises to the top creating a very sleek and smooth surface. A sleek and smooth interface surface between the precast elements and subsequently added cast in-situ concrete allows no transfer of force due to lack of interlock reducing the bond behavior to a minimum which can lead to a separation of the two layers. The clear solution for this problem is the declaration of a minimum roughness of the precast elements, which could be set to a peak-to-valley height of 1.50 mm as suggested by Peyerl (2016).

An optimization of the construction process presented in this thesis should be the consideration of reducing the filling of the U-shaped thin-walled precast girders with in-situ concrete to a one step process. Up to this point the girders would always be filled in two steps or more based on the low load-bearing capacity of the thin-walled U-shaped cross-sections. The possibility of enhancing the bottom slab of the U-shaped elements or using lightweight concrete as filling concrete should be considered. Through these measures, a construction step on the construction site could be omitted, thus saving construction time and further costs.

When it comes to the development of large box-cross-sectioned girders out of thin-walled precast elements valuable insight was taken from both the large-scale testing as well as the static calculation model. Most of the obtained insight was already implemented in the design competition for the two bypass bridges over the Danube river in Linz. The stabilizing structure of both the deck and bottom slabs were optimized. Furthermore the placement and installation of the connection reinforcement between the webs and deck and bottom slabs were revised and enhanced.

8.2 Outlook

With research on the application of thin-walled precast elements in bridge and engineering structures continuing at the TU Wien as shown by K. Fuchs, Gaßner, et al. (2017b), Preinstorfer et al. (2017), and Fischer et al. (2017) as well as various still pending patents, there are many new innovations that can be looked forward to.

In addition to the continued research dedicated to the application of thin-walled precast elements in bridge and other engineering structures, four bridges in the south-eastern part of Austria will be erected using girders out of thin-walled precast elements in the combination with the balanced lift method, with the start of construction set for autumn 2018 (Kollegger, Foremniak, et al. (2014), Wimmer (2016), and Gmainer (2011)).

Bibliography

Alberta Transportation Bridge Structures Design Criteria (2012). *Appendix A - Integral Abutment Design Guidelines*. Alberta Government. [Standard].

ASFINAG (2013). *A7 "Bypass"-Brücken zur Voestbrücke über die Donau im Zuge der A7 Müllkreisautobahn*. [Competition tender].

ASM (1985). *Atlas of fatigue curves*. Ed. by Howard E Boyer et al. American Society of Metals International.

ASM (1990). *Handbook, Volume 1: Properties and Selection: Irons, Steels, and High-Performance Alloys*. American Society of Metals International.

Bathias, C. (1999). "There is no infinite fatigue life in metallic materials". In: *Fatigue & Fracture of Engineering Materials & Structures* Volume 22.Number 7, pp. 559–565.

Berger, S. (2017). "Experimentelle Bestimmung der Dauerschwingfestigkeit von Fahrbahnplatten mit untenliegenden Fertigteilplatten und einer darüber angeordneten Aufbetonschicht". MA thesis. TU Wien.

Bień, J. (2010). *Uszkodzenia i diagnostyka obiektów mostowych*. Wydawnictwa Komunikacji i Łączności.

Biliszczuk, J., C. Machelski, P. Maliszkiewicz, and M. Mistewicz (1994). "Typowe uszkodzenia drogowych mostów prefabrykowanych". In: *Drogownictwo* 8, pp. 186–194.

Biliszczuk, J. and J. Onysyk (2016). "Prefabrykacja w mostownictwie". In: *Nowoczesne Budownictwo Inżynieryjne*, pp. 66–75.

Brodnan, M., P. Kotes, and P. Kotula (2014). *Analysis od short span Bridges from prestressed concrete*. 4th fib Congress - Symposium in Mumbai. [Proceedings].

Burgholzer, L., E. Garn, and O. Schimetta (1973). "Die 2. Donaubrücke Linz. 1. Teil: Generelle Planung, Ausschreibung, Wahlentwürfe. 2. Teil: Ausführungsentwürfe. 3. Teil: Fertigung und Montage". In: *Der Stahlbau* 42.Heft 11, pp. 321–332.

CAE Simulation Solutions (2015). [Date: 01.03.2018]. URL: `http://www.cae-sim-sol.com/en/limit-stress-evaluation/finite-life-fatigue-strength`.

Dicleli, M. (2000). "A rational design approach for prestressed-concrete-girder integral bridges". In: *Engineering Structures* 22.3, pp. 230–245.

Eichinger, E., J. Kollegger, Bernd B. Köberl, and M. Vill (2005). "Zerstörende Belastungsversuche an einer vorgespannten Segmentbrücke". In: *Beton- und Stahlbetonbau* 100. Jahrgang.Heft 12, pp. 977–984.

Engelsmann, S., J. Schlaich, and K. Schäfer (1999). "Entwerfen und Bemessen von Betonbrücken ohne Fugen und Lager". In: *Deutscher Ausschuß für Stahlbeton* 496.

Eurocode EN 1991-1-1 (2003). *Actions on structures - Part 1-1: General Actions - Densities, self-weight, imposed loads for buildings*. European Committee for Standardization. [Standard].

Eurocode EN 1991-1-5 (2003). *Actions on structures - Part 1-5: General actions - Thermalactions.* European Committee for Standardization. [Standard].

Eurocode EN 1991-2 (2003). *Actions on structures - Part 2: Traffic loads on bridges.* European Committee for Standardization. [Standard].

Eurocode EN 1992-1-1 (2004). *Design of concrete structures - Part 1-1: General rules and rules for buildings.* European Committee for Standardization. [Standard].

Eurocode EN 1992-2 (2005). *Design of concrete structures - Part 2: Concrete bridges - Design and detailing rules.* European Committee for Standardization. [Standard].

FCP (2014). *A7 "Bypass"-Brücken zur Voestbrücke über die Donau - Technischer Bericht.* [Competition Entry].

Fehlmann, P. (2012). "Zur Ermüdung von Stahlbetonbrücken". PhD thesis. ETH Zürich.

FHWA (2005). *Prefabricated Bridge Elements and Systems in Japan and Europe.* Tech. rep. U.S. Department of Transportation - Federal Highway Admnistration - FHWA.

***fib* Bulletin No. 29 (2004)**. *Precast concrete bridges.* Fédération internationale du béton (*fib*). [State-of-the-art report].

Figg, L. and W. Denney Pate (2004). "Precast Concrete Segmental Bridges - America's Beautiful and Affordable Icons". In: *PCI journal* 49.5.

Fischer, I. and M.C. Schönweger (2017). "Tower out of double walls". PhD thesis. TU Wien.

Freyermuth, C.L. (1969). "Design of continuous highway bridges with precast, prestressed concrete girders". In:

Fuchs, K., G. Gaßner, and J. Kollegger (2017a). *Method for placing Prefabricated Slab Elements on Bridge Decks.* 39th IABSE Symposium. [Proceedings].

Fuchs, K., G. Gaßner, and J. Kollegger (2017b). *Steel-Concrete-Composite Bridge with Innovative Prefabricated Slab Elements.* 2017 fib Symposium - Maastricht. [Proceedings].

Fuchs, K. and J. Kollegger (2016). *Ein neues Verfahren zur Herstellung der Fahrbahnplatte von Stahl-Beton-Verbundbrücken.* VÖZ Kolloquium 2016. [Proceedings].

Fuchs, W., M. Fuchs, V. Danzl, and T. Dannhorn (2007). "Vorgespannte integrale Brücken–Erfahrungen aus der Praxis". In: *Tagungsband Münchner Massivbau Seminar.*

Furche, J. and U. Baumeister (2009). "Kapitel III. Elementbauweise mit Gitterträgern". In: *Beton-Kalender 2009: Schwerpunkte - Konstruktiver Hochbau, Aktuelle Massivbaunormen.* Ernst & Sohn Verlag, pp. 337–498.

Geißler, K. (2014). *Handbuch Brückenbau - Entwurf, Konstruktion, Berechnung, Bewertung und Ertüchtigung.* John Wiley & Sons.

Gmainer, S. (2011). "Brückenklappverfahren - Untersuchungen zur Entwicklung eines praxistauglichen Bauverfahrens". PhD thesis. TU Wien.

Grütze, D. (2007). *Bau-Lexikon.* Carl Hanser Verlag.

Hausleitner, K. (2014). "Bypass-Brücken zur Voestbrücke über die Donau - Entwurf einer Betonbrücke unter Verwendung von dünnwandigen Betonfertigteilen". MA thesis. TU Wien.

HEPC (1987). *Deterioration and repair of bridge structures in Hanshin Expressway.* Tech. rep. Hanshin Expressway Public Corporation.

Huber, P. (2011). "Verstärkung von Brücken durch eine statisch mitwirkende Aufbetonschicht". MA thesis. TU Wien.

Hue, F. (2005). "Use of Precast Concrete Elements in Bridges in Europe (Spain)". In: *Expertenforum Beton 2005 - Betonfertigteile im Brückenbau: sicher, schnell und innovativ.*

Hunkeler, F., P. Matt, U. von Matt, and R. Werner (2005). "Spannglieder, Schrägseile und Anker - Beschreibung der Systeme und Erkenntnisse aus Korrosionsschäden (Prestressing tendons, stay cables and ground anchors - Description of the systems and lessons learnt from corrosion damages)". In: *Eidgenössisches Departement für Umwelt, Verkehr, Energie und Kommunikation/Bundesamt für Strassen, Berne, Switzerland.*

KAPPEMA (2011). *Halbfertigwand mit KAPPEMA-Bewehrungselementen aus Stahl als Verbindungselement.* [Allgemeine bauaufsichtliche Zulassung - DIBt Deutsches Institut für Bautechnik].

Kargel, E. (2005). "Verbindung der Vorteile industrieller Fertigung und monolithischer Bauweise im Brückenbau". In: *Expertenforum Beton 2005 - Betonfertigteile im Brückenbau: sicher, schnell und innovativ.*

Kaufmann, W. (2008). *Integrale Brücken - Sachstandbericht.* Tech. rep. Eidgenösissches Department für Umwelt, Verkehr, Energie und Kommunikation - Budesamt für Strassen.

Kleiser, M. (2016). "Formlogik an Brücken - Teil 1". In: *Betontechnik* Ausgabe 93 (Heft 2), pp. 49–58.

Köberl, B. (2008). "Entwicklung einer hochfrequenten Prüfmethode für Dauerschwingversuche". PhD thesis. TU Wien.

Köberl, B. et al. (2008). "Möglichkeit von hochfrequenten Dauerschwingversuchen an der TU - Wien". In: *Bauingenieur* Heft 83, pp. 513–522.

Kollegger, J. and S. Blail (2008). "Balanced Lift Method for Bridge Construction". In: *Structural Engineering International* Volume 18, pp. 283–289.

Kollegger, J., S. Foremniak, D. Suza, D. Wimmer, and S. Gmainer (2014). "Building bridge using the balanced lift method". In: *Structural Concrete* Volume 15.3, pp. 281–291.

Kollegger, J. and D. Wimmer (2014). *Bridge girders made of double walls erected by balanced lift method - From building component to engineering component.* Betontage - Neu Ulm. [Proceedings].

Krumbach, R., H. Meichsner, and L. Schubert (1997). "Untersuchungen zur Dauerbeständigkeit von vorgefertigten Spannbeton-Brückenträgern". In: *Beton- und Stahlbetonbau* 92. Jahrgang.Heft 12, pp. 325–331.

Kunin, J. and S. Alampalli (2000). "Integral abutment bridges: current practice in United States and Canada". In: *Journal of performance of constructed facilities* Volume 14.3, pp. 104–111.

Maeda, Y., S. Matsui, I. Shimada, and H. Kato (1980). "Deterioration and Repairing of Reinforced Concrete Slabs of Highway Bridges in Japan". In: *Technology Reports (Osaka University)* 30.1599, pp. 135–144.

Marx, S. and G. Schacht (2010). "Gelenke im Massivbau". In: *Beton- und Stahlbetonbau* 105. Jahrgang.Heft 1, pp. 27–35.

Mehlhorn, G. and M. Curbach (2015). *Handbuch Brücken: Entwerfen, konstruieren, berechnen, bauen und erhalten.* Springer-Verlag.

Müller, H.S., H. Reinhardt, and U. Weis (2012). "Kapitel VI. Beton". In: *Beton-Kalender 2012: Schwerpunkte - Infrastrukturbau, Befestigungstechnik.* Ernst & Sohn Verlag, pp. 303–445.

Nad, L. (2004). *Assessment of concrete bridge made of precast beam.* International Symposium - Durability and Maintenance of Concrete Structures - Dubrovnik. [Proceedings].

ÖNORM B 1991-1-1 (2017). *Einwirkungen auf Tragwerke - Teil 1-1: Allgemeine Einwirkungen - Wichten, Eigengewicht, Nutzlasten im Hochbau - Nationale Festlegung zu ÖNORM EN 1991-1-5 und nationale Ergänzungen.* Austrian Standards Institute - Österreichisches Normungsinstitut (ON). [Standard].

ÖNORM B 1991-1-5 (2004). *Einwirkungen auf Tragwerke - Teil 1-5: Allgemeine Einwirkungen - Temperatureinwirkungen - Nationale Festlegung zu ÖNORM EN 1991-1-5 und nationale Ergänzungen.* Austrian Standards Institute - Österreichisches Normungsinstitut (ON). [Standard].

ÖNORM B 1991-2 (2011). *Einwirkungen auf Tragwerke - Teil 2: Verkehrslasten auf Brücken - Nationale Festlegung zu ÖNORM EN 1991-2 und nationale Ergänzungen.* Austrian Standards Institute - Österreichisches Normungsinstitut (ON). [Standard].

ÖNORM B 1992-1-1 (2018). *Bemessung und Kostruktion von Stahlbeton- und Spannbetontragweken - Teil 1-1: Allgemeine Bemmessungsregeln und Regeln für den Hochbau - Nationale Festlegung zu ÖNORM EN 1992-1-1, nationale Erläuterung und nationale Ergänzungen.* Austrian Standards Institute - Österreichisches Normungsinstitut (ON). [Standard].

ÖNORM B 1992-2 (2008). *Bemessung und Kostruktion von Stahlbeton- und Spannbetontragweken - Teil 2: Betonbrücken - Bemessung und Konstruktionsregeln - Nationale Festlegung zu ÖNORM EN 1992-2, nationale Erläuterung und nationale Ergänzungen.* Austrian Standards Institute - Österreichisches Normungsinstitut (ON). [Standard].

ÖNORM B 4707 (2014). *Bewehrungsstahl - Anforderungen, Klassifizierung und Konformitätsnachweis.* Austrian Standards Institute - Österreichisches Normungsinstitut (ON). [Standard].

ÖNORM EN 1015-3 (2010). *Prüfverfahren für Mörtel für Mauerwerk, Teil 3: Bestimmung der Konsistenz von Frischmörtel.* Austrian Standards Institute - Österreichisches Normungsinstitut (ON). [Standard].

ÖNORM EN 12350-9 (2010). *Prüfverfahren von Frischbeton, Teil 9: Selbstverdichtender Beton - Auslauftrichterversuch.* Austrian Standards Institute - Österreichisches Normungsinstitut (ON). [Standard].

ÖNORM EN 12617-4 (2002). *Produkt für den Schutz und die Instandsetzung von Betontragwerken - Prüfverfahren - Bestimmung des Schwindens und des Quellens.* Austrian Standards Institute - Österreichisches Normungsinstitut (ON). [Standard].

ÖNORM EN 196-1 (2005). *Prüfverfahren für Zement, Teil 1: Bestimmung der Festigkeit.* Austrian Standards Institute - Österreichisches Normungsinstitut (ON). [Standard].

ONR 23303 (2010). *Prüfverfahren Beton (PVB - Nationale Anwendung der Prüfnormen für Beton und seine Ausgangsstoffe.* Austrian Standards Institute - Österreichisches Normungsinstitut (ON). [Standard].

Peyerl, M. (2012). "Fracture mechanical and stereoscopic characterization of cement bonded interfaces". PhD thesis. TU Wien.

Peyerl, M. (2016). *Hochfester Beton als nachhaltige Lösung für Aufbetone und Deckschichten.* VÖZ Kolloquium 2016. [Proceedings].

Peyerl, M. and S. Krispel (2013). "Oberflächenstruktur - ein wesenltiches Merkmal für das Funktionieren von Beton-Verbundkonstruktionen". In: *Zement und Beton* 5, pp. 18–21.

Poston, R.W. and J.P. Wouters (1998). *Durability of precast segmental bridges.* Tech. rep.

Preinstorfer, P. and J. Kollegger (2017). "Doppelwand aus Ultrahochleistungsbeton - Planung und Bau von großformatigen Prototypen". In: *Beton- und Stahlbetonbau* 112. Jahrgang.Heft 6, pp. 346–354.

Radaj, D and M Vormwald (2007). *Ermüdungsfestigkeit-Grundlagen für Ingenieure, Dritte, neubearbeitete und erweiterte Auflage.* Springer-Verlag Berlin Heidelberg.

Ruhrberg, R. and H. Schumann (1982). "Schäden an Brücken und anderen Ingenieurbauwerken. Dokumentation 1982 vom Bundesmin. f. Verkehr". In:

RVS 15.02.2.12 (2018). *Bemessung und Ausführung von integralen Brücken.* Forschungsgesellschaft Straße - Schiene - Verkehr. [Standard].

Schacht, G. and S. Marx (2010). "Unbewehrte Betongelenke - 100 Jahre Erfahrung im Brückenbau". In: *Beton- und Stahlbetonbau* 105. Jahrgang.Heft 9, pp. 599–607.

Schläfli, M. (1999). "Ermüdung von Brückenfahrbahnplatten aus Stahlbeton". PhD thesis. Ecole Polytechnique Fédérale de Lausanne.

Schlappal, T., M. Schweigler, S. Gmainer, M. Peyerl, and B. Pichler (2017). "Creep and cracking of concrete hinges: insight from centric and eccentric compression experiments". In: *Materials and Structures* 50.6, p. 244.

Schmidt, S., F. Zierath, H. Amann, and H. Meyer (2012). "Die Rollbrücken der neuen Landebahn Nordwest am Flughafen Frankfurt/Main". In: *Beton- und Stahlbetonbau* 107. Jahrgang.Heft 3, pp. 164–174.

Siwowski, T. (1995). "Improving the durability of precast concrete bridges". In: *IABSE reports - Rapports AIPC - IVBH Berichte* 73-1-73-2.

Stark, J. and B. Wicht (2013). *Dauerhaftigkeit von Beton.* Springer.

Steiger, H., T. Zeißler, M. Bernhard, and H. Meyer (2012). "Integrale Großbrücken mit flexiblen Widerlagern". In: *Beton- und Stahlbetonbau* 107. Jahrgang.Heft 3, pp. 175–182.

Suza, D. and J. Kollegger (2015). *Stress redistribution in prestressed concrete bridges built with ultra-thin precast girders.* CONCREEP 10 - Vienna. [Proceedings].

Vennemann, K. (2005). “Fertigteile im Brückenbau auf neuen Wegen”. In: *Expertenforum Beton 2005 - Betonfertigteile im Brückenbau: sicher, schnell und innovativ.*

Wimmer, D. (2013). “Thin-walled precast concrete girders for bridge construction and civil engineering”. In: *BFT International* Volume 79, pp. 44–51.

Wimmer, D. (2016). “Entwicklung eines neuen Brückenbauverfahrens durch die Kombination von dünnwandigen Betonfertigteilen und Vorspannung”. PhD thesis. TU Wien.

Wimmer, D. and M. Kleiser (2013). “Precast Reloaded - Fertigteilträger im Brücken- und Ingenieurbau”. In: *F & E Schriftenreihe der ASFINAG* Nr. 7, pp. 19–26.

Yapparina (2013). [Date: 01.03.2018]. URL: `https://commons.wikimedia.org/wiki/File:Typical_reversed_fatigue_stress_cycles.png#file`.

Zilch, K. and G. Zehetmaier (2008). *Bemessung im konstruktiven Betonbau.* Springer-Verlag Berlin Heidelberg.

Appendix A
Detailed results of experimental preliminary tests of grout materials

A.1 Flexural tensile stength

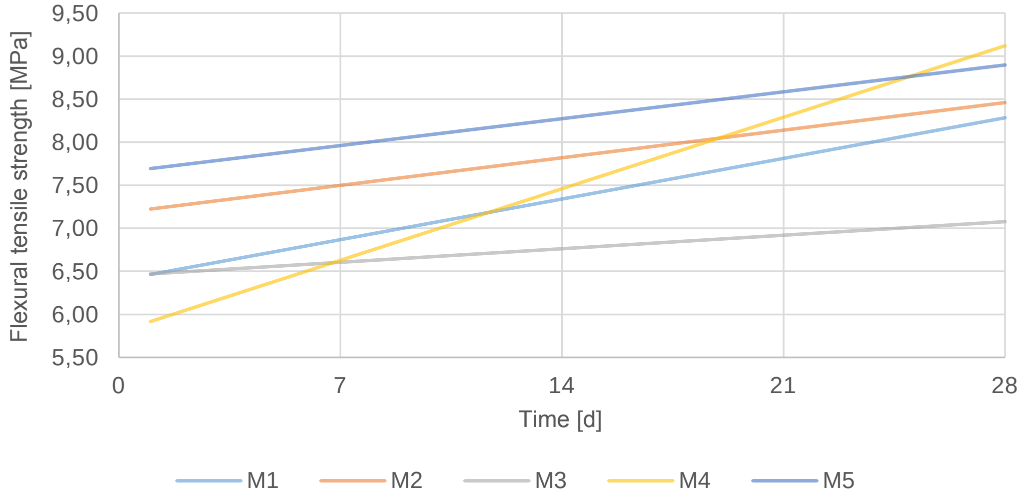


Fig. A.1: Flexural tensile strength as a function of time from 24 hours to 28 days of the five tested grouts (mean calculated from multiple specimens)

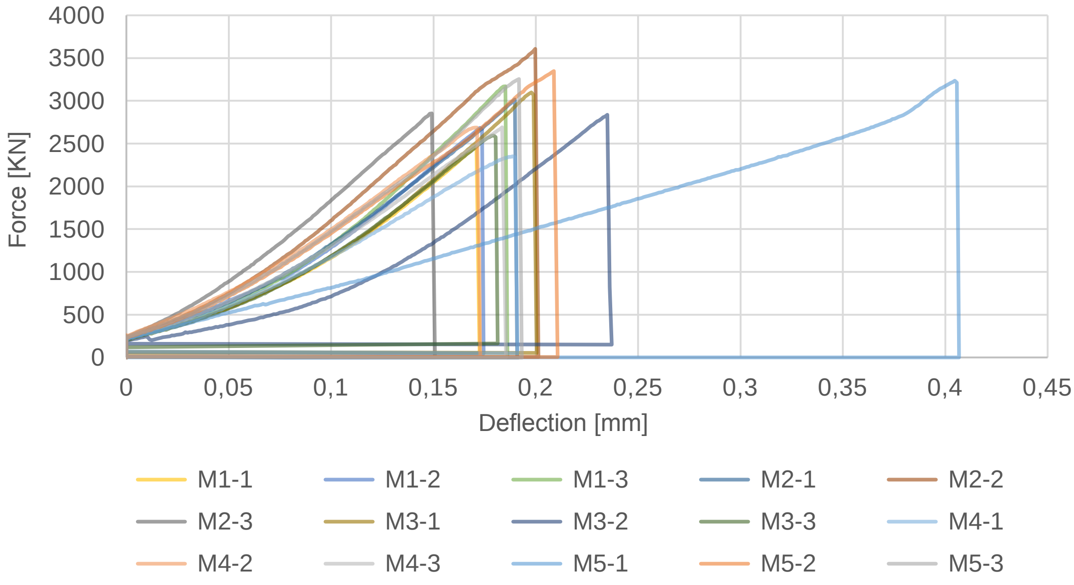


Fig. A.2: Flexural tensile strength test of the five tested grouts with three specimens each after 24 hours - Force-Deflection-Diagram

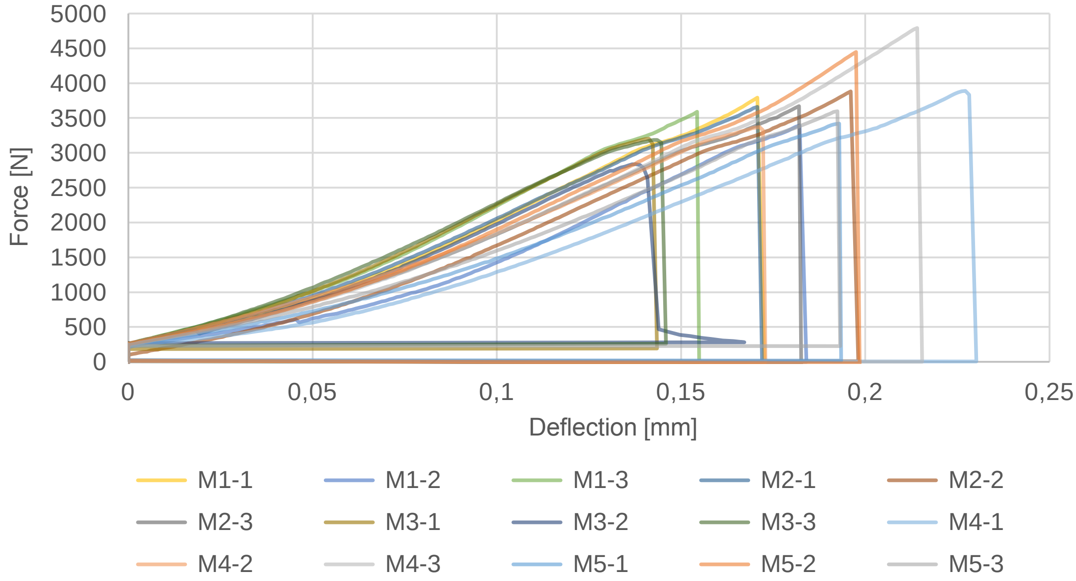


Fig. A.3: Flexural tensile strength test of the five tested grouts with three specimens each after 28 days - Force-Deflection-Diagram

A.2 Compression strength

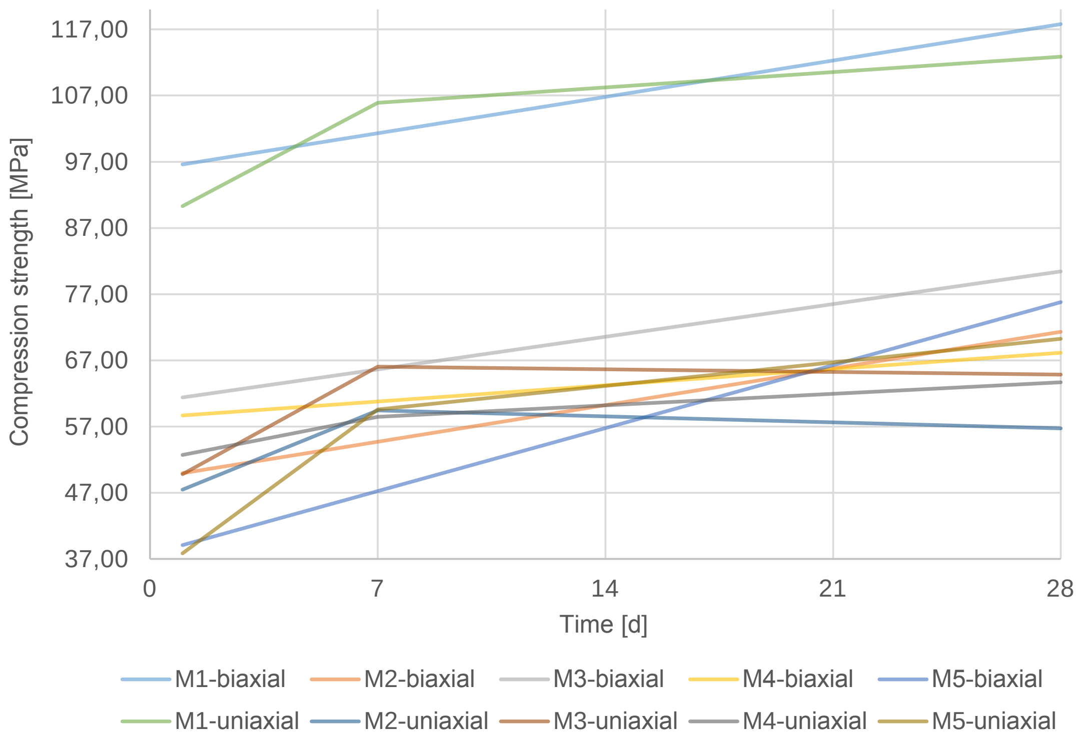


Fig. A.4: Flexural tensile strength (uniaxial and biaxial) as a function of time from 24 hours to 28 days of the five tested grouts (mean calculated from multiple specimens)

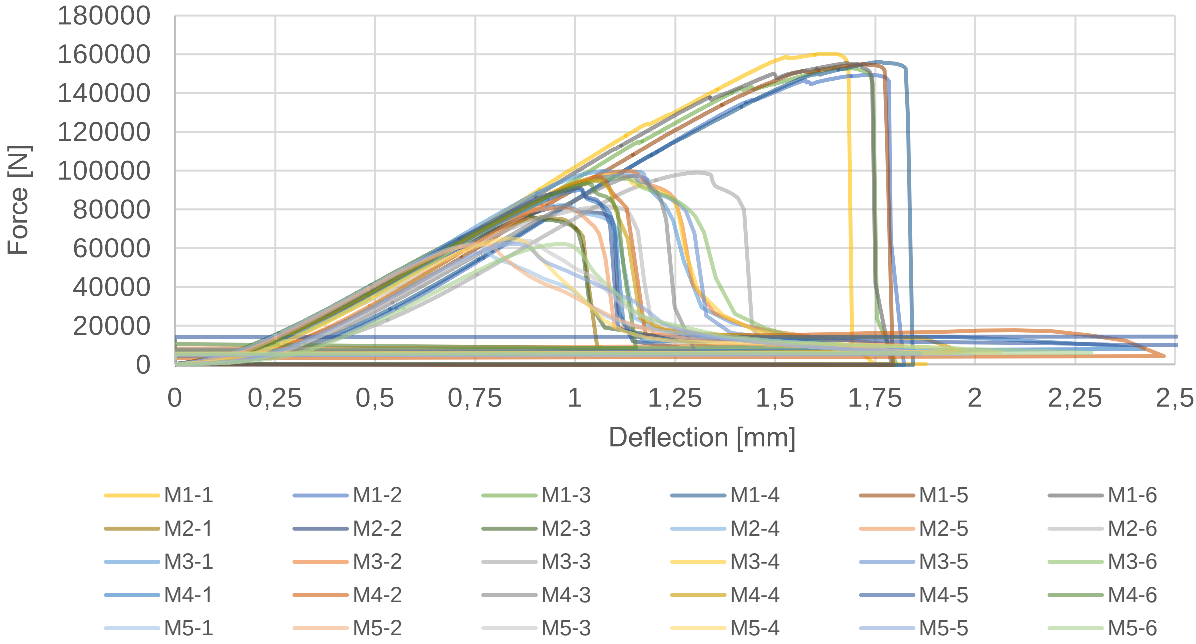


Fig. A.5: Biaxial compression strength test of the five tested grouts with six specimens each after 24 hours - Force-Deflection-Diagram

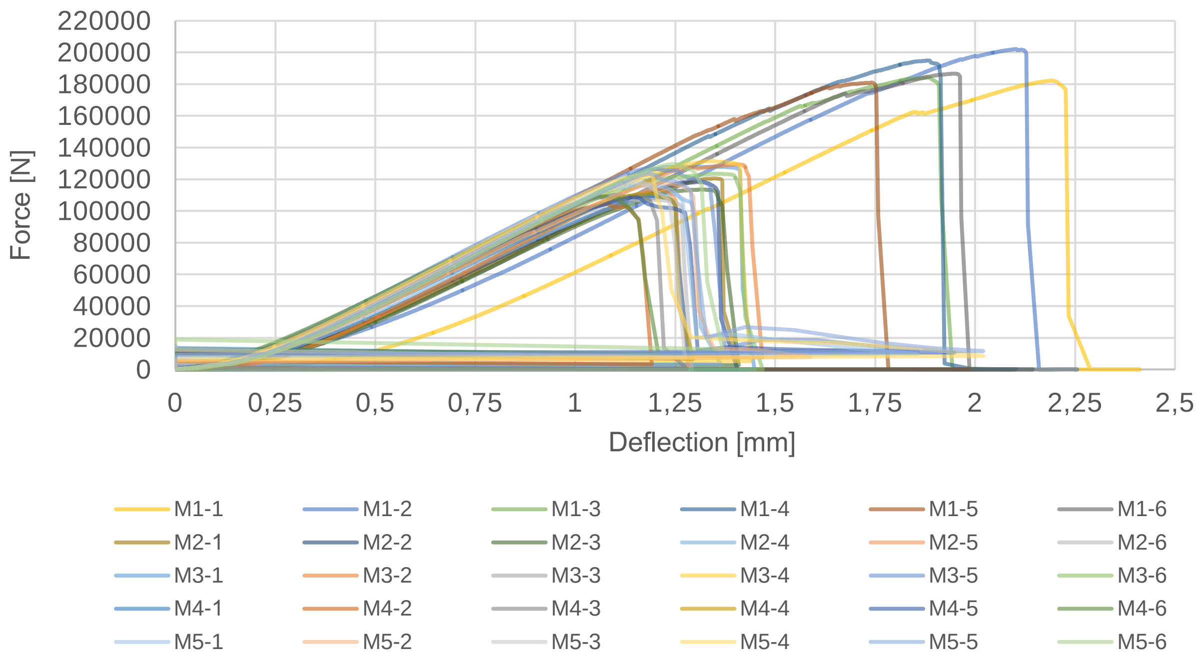


Fig. A.6: Biaxial compression strength test of the five tested grouts with six specimens each after 28 days - Force-Deflection-Diagram

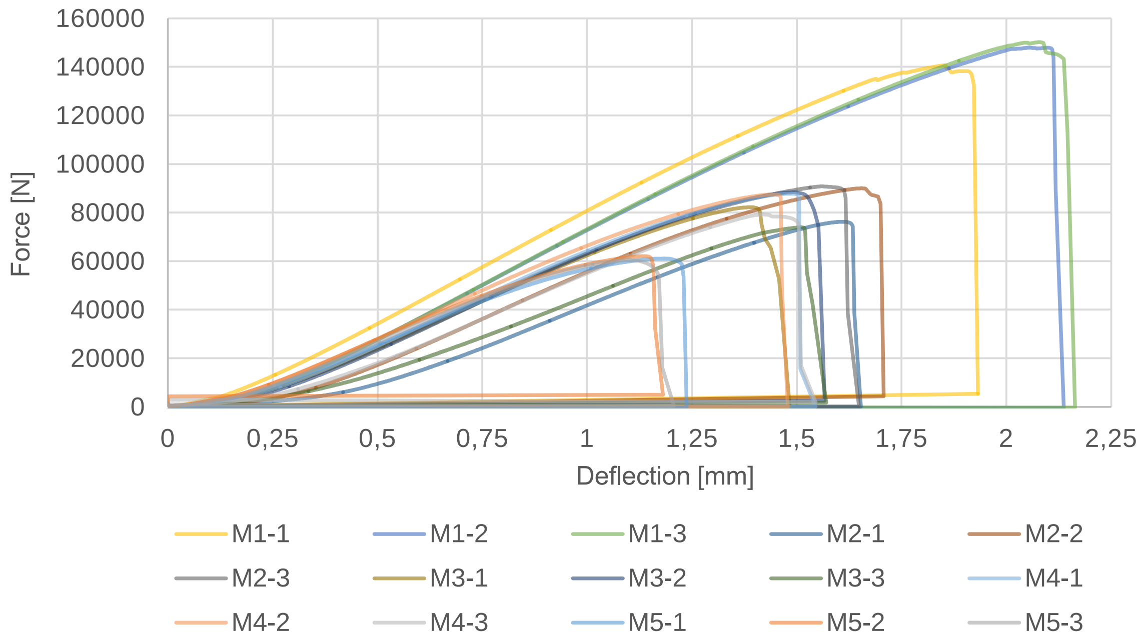


Fig. A.7: Uniaxial compression strength test of the five tested grouts with three specimens each after 24 hours - Force-Deflection-Diagram

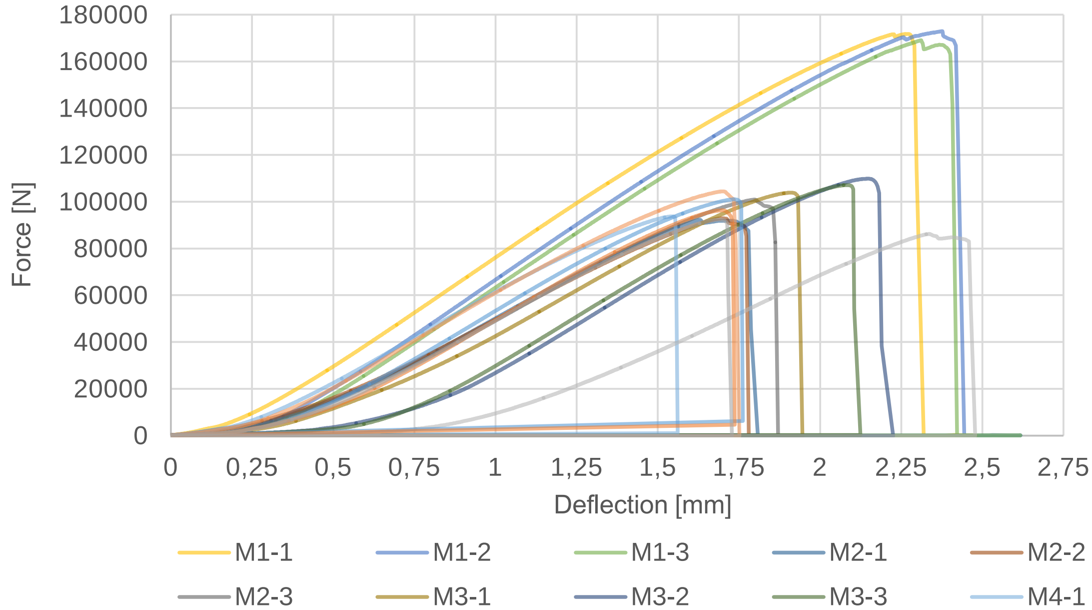


Fig. A.8: Uniaxial compression strength test of the five tested grouts with three specimens each after 7 days - Force-Deflection-Diagram

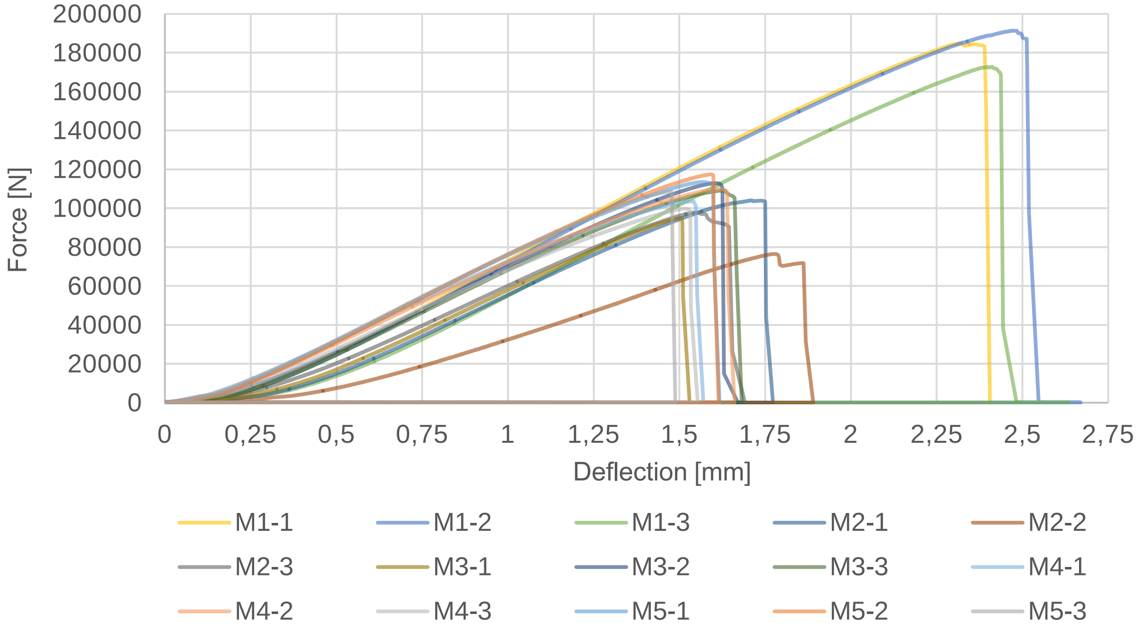


Fig. A.9: Uniaxial compression strength test of the five tested grouts with three specimens each after 28 days - Force-Deflection-Diagram

A.3 Flow consistency and viscosity

Tab. A.1: Average compression and flexural tensile strength in [MPa] assessed in the tests

	V-funnel test*	**Spread test****
Grout 1	7.00 sec	365 mm
Grout 2	8.50 sec	310 mm
Grout 3	4.16 sec	290 mm
Grout 4	no flow	100 mm
Grout 5	26.23 sec	187.5 mm

* Time measured for grout to flow through the apparatus

* Starting diameter of the cone: 100 mm

A.4 Shrinkage

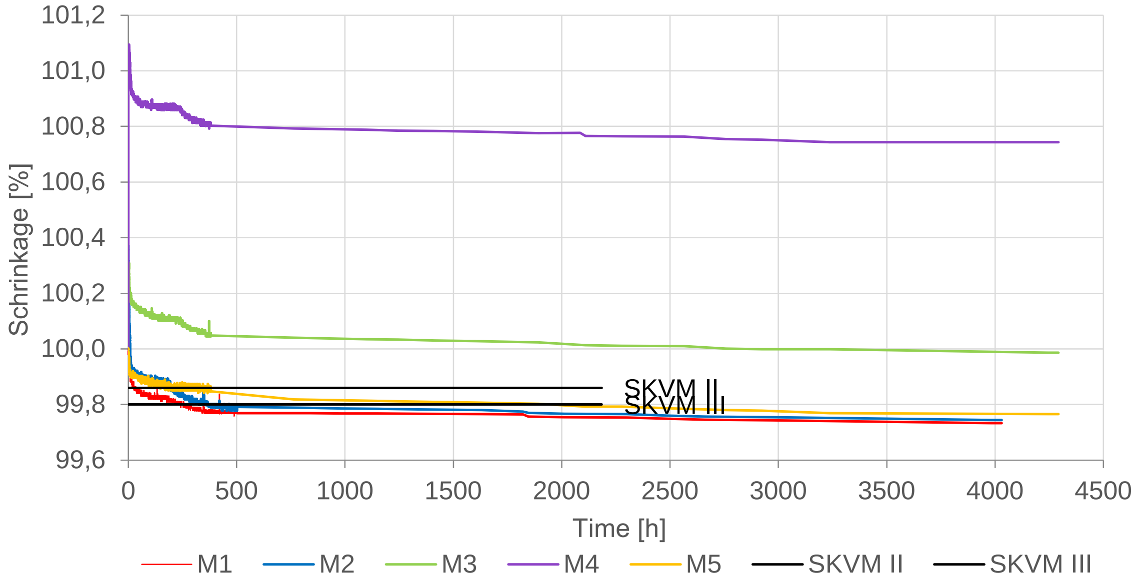


Fig. A.10: Expansion and Shrinkage [%] of the five tested grouts over time

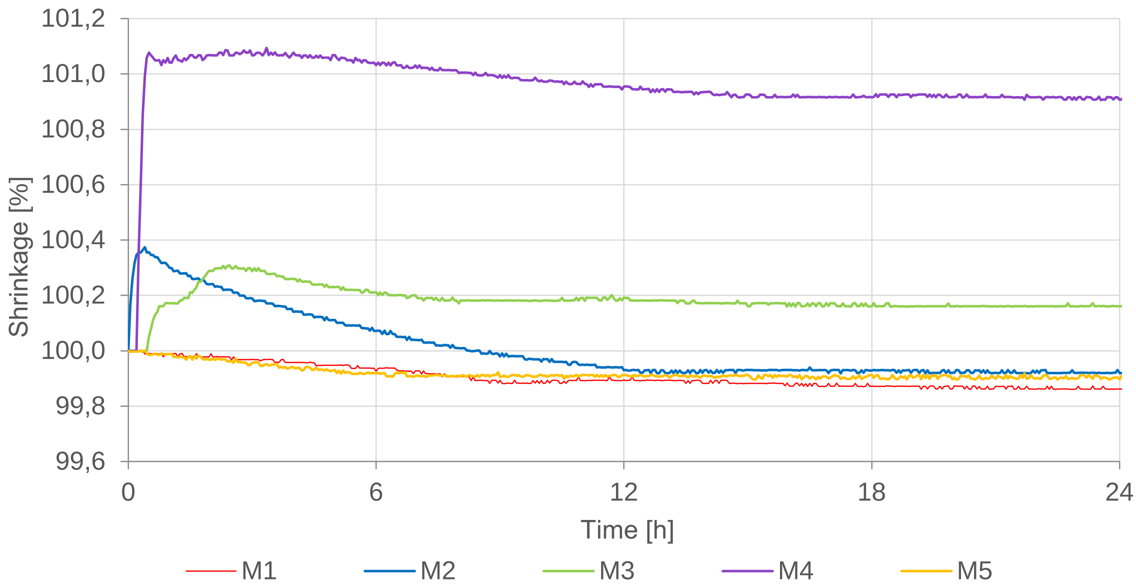


Fig. A.11: Expansion and Shrinkage [%] of the five tested grouts in the first 24 hours

A.5 Temperature development

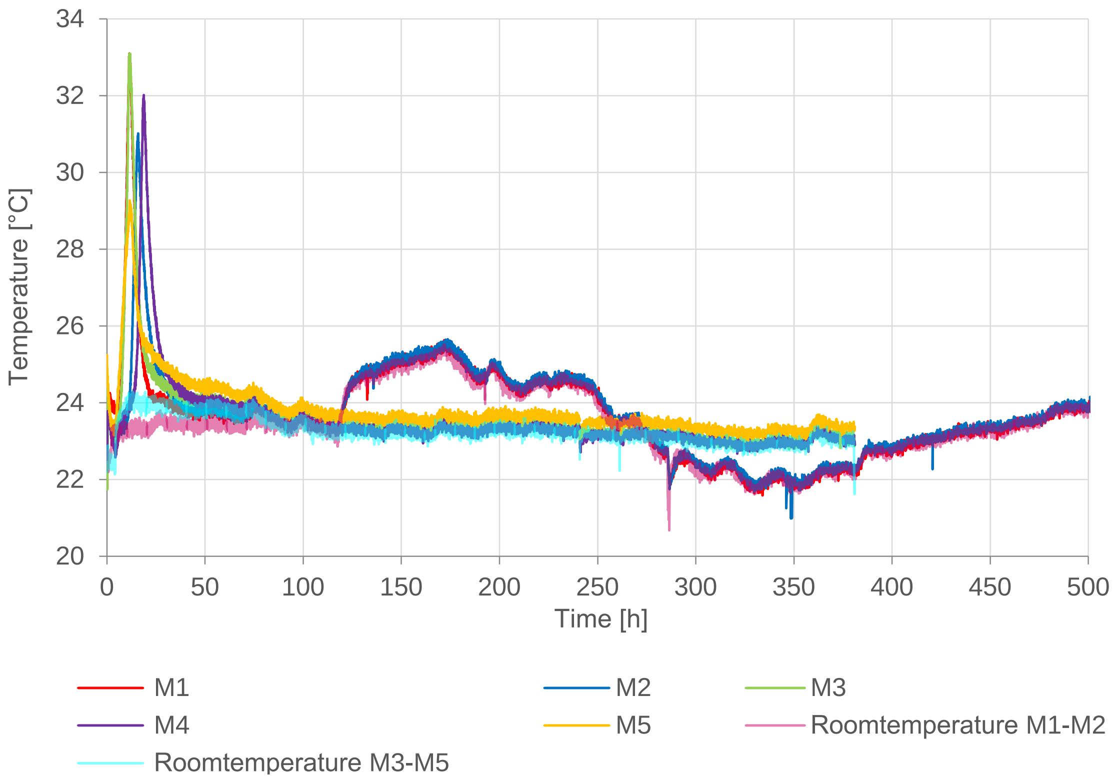


Fig. A.12: Temperature development of the five tested grouts over time

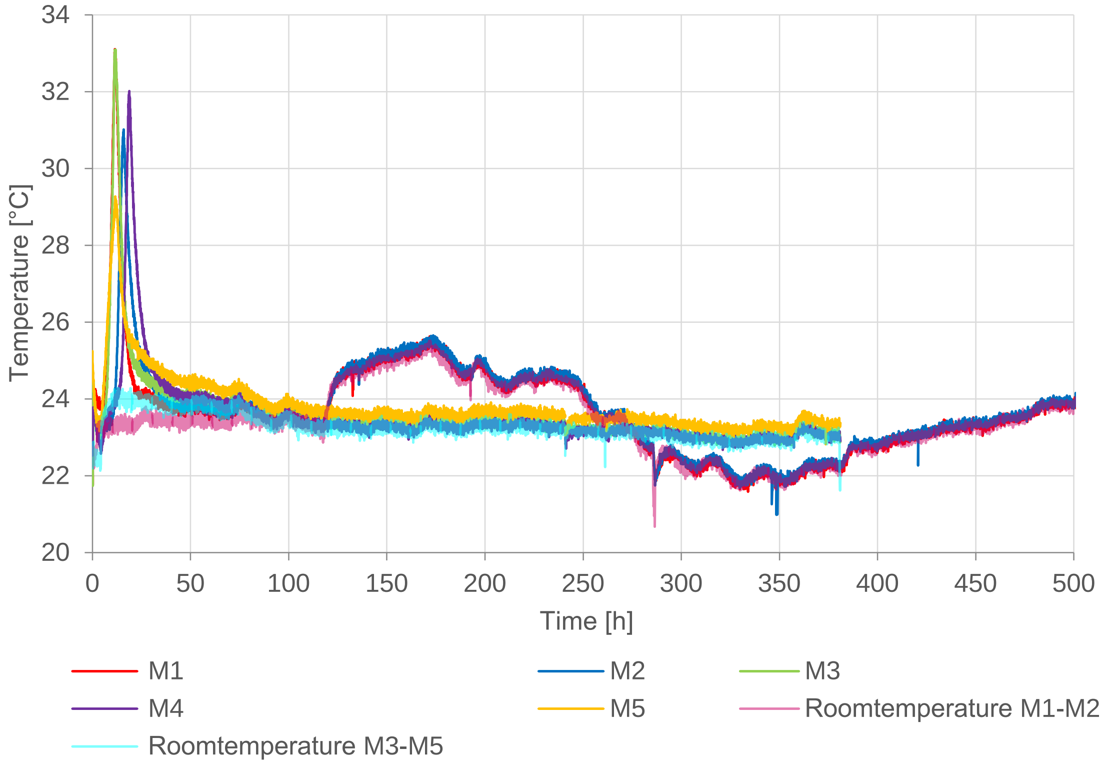
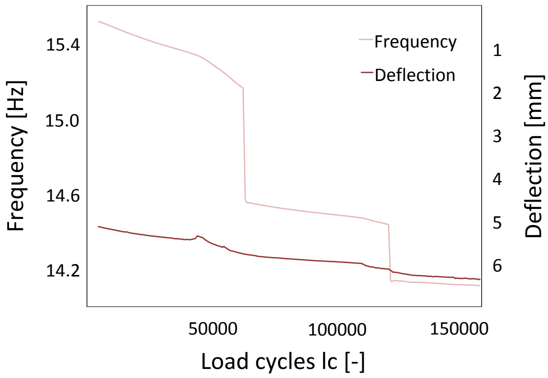
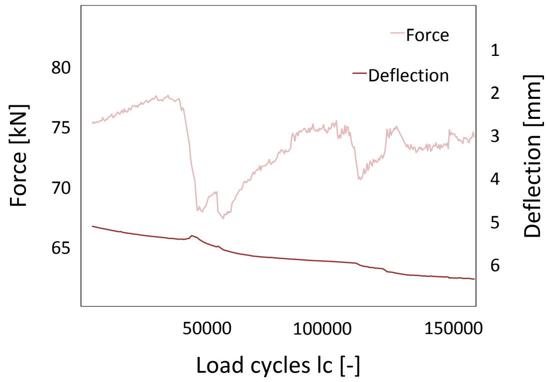


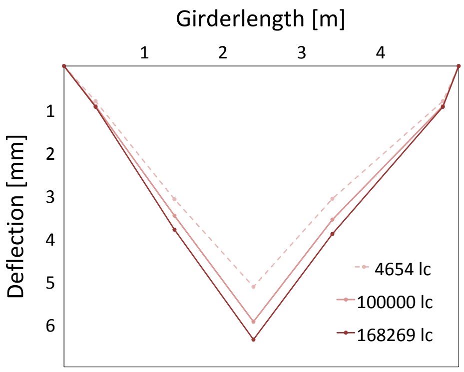
Fig. A.13: Temperature development of the five tested grouts in the first 24 hours

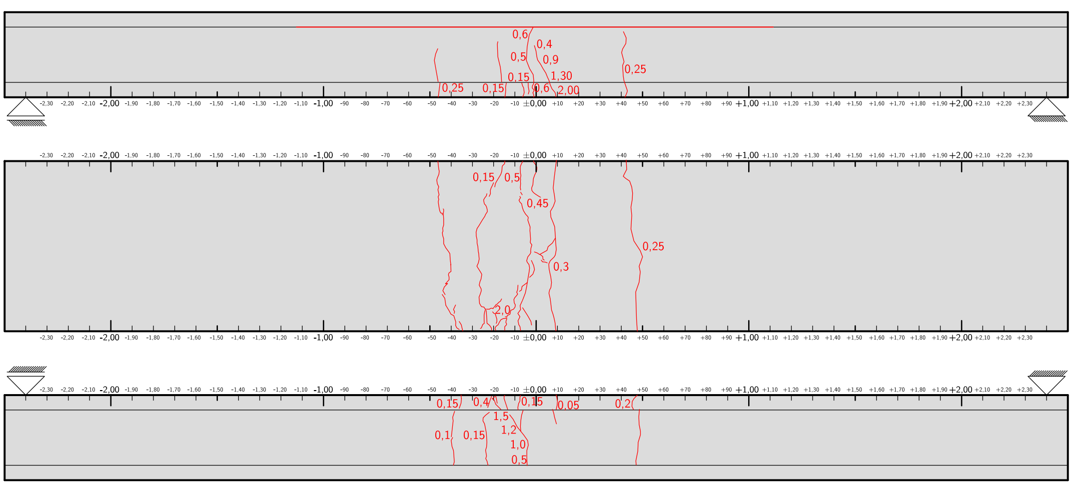
Appendix B
Detailed results of experimental tests of fatigue behavior of thin-walled precast elements

B.1 Fatigue test 1

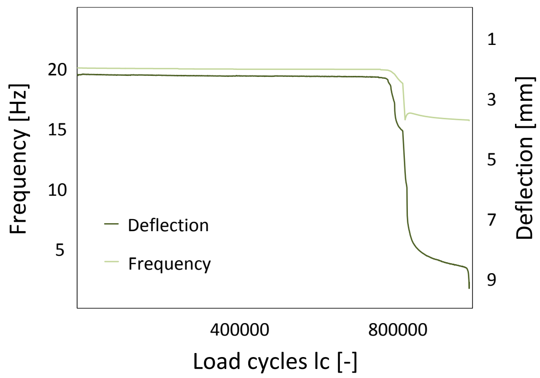


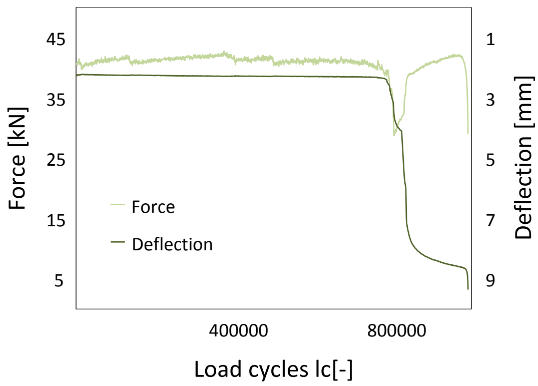


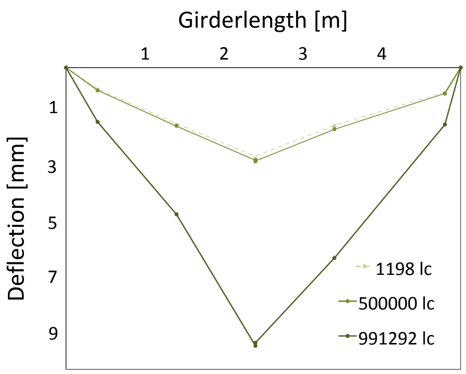


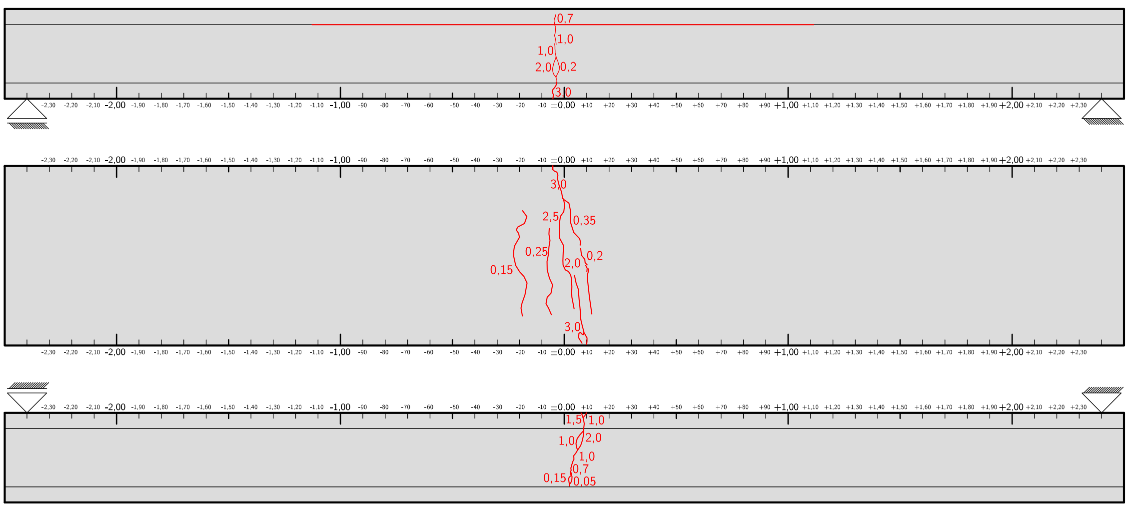


B.2 Fatigue test 2

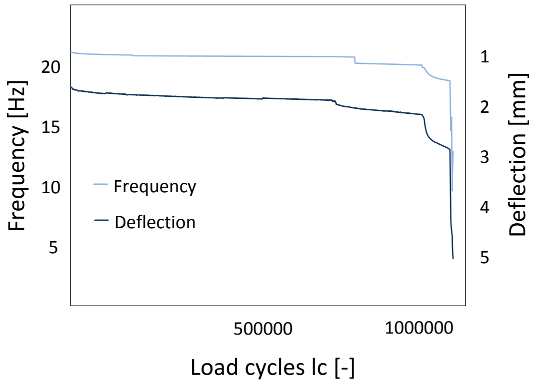


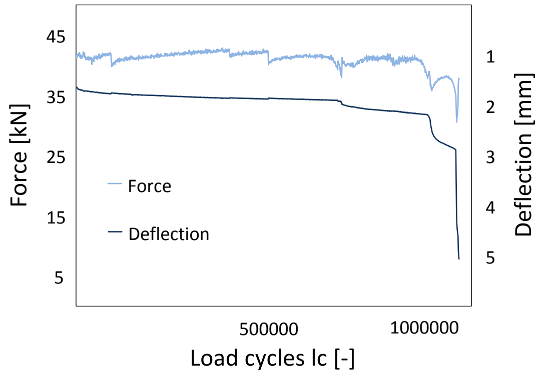


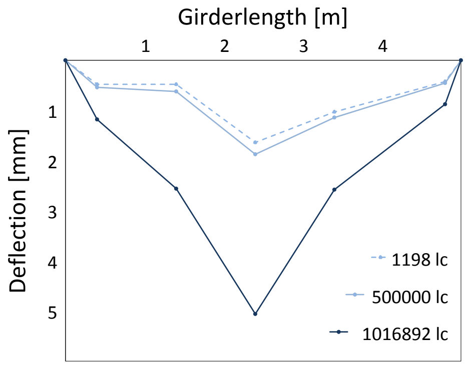


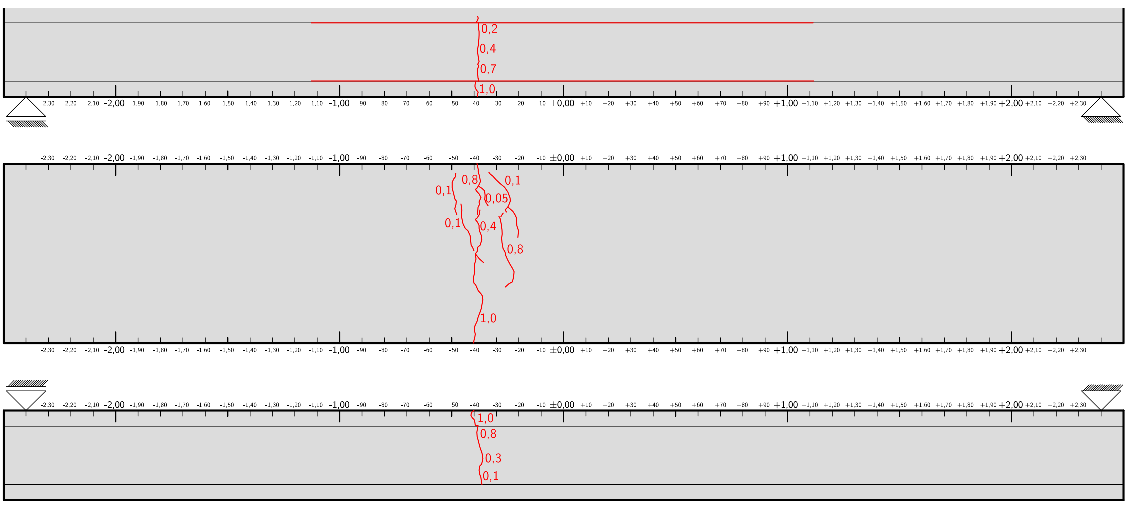


B.3 Fatigue test 3

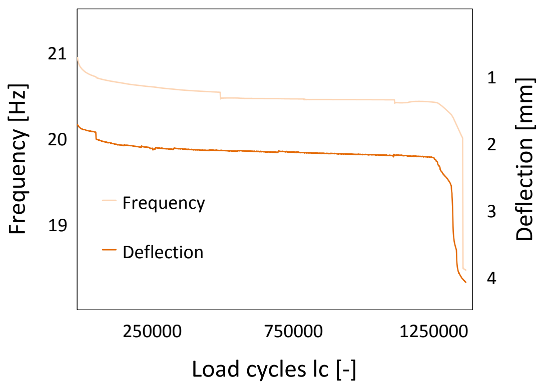


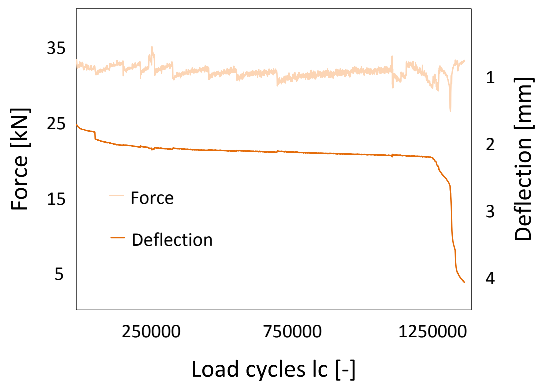


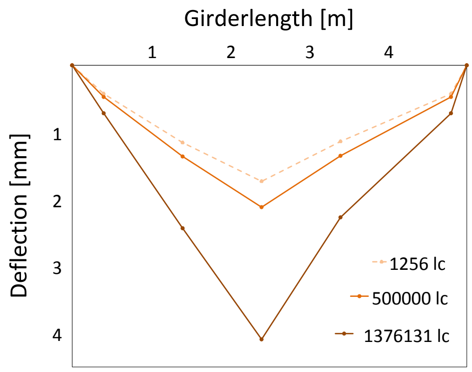


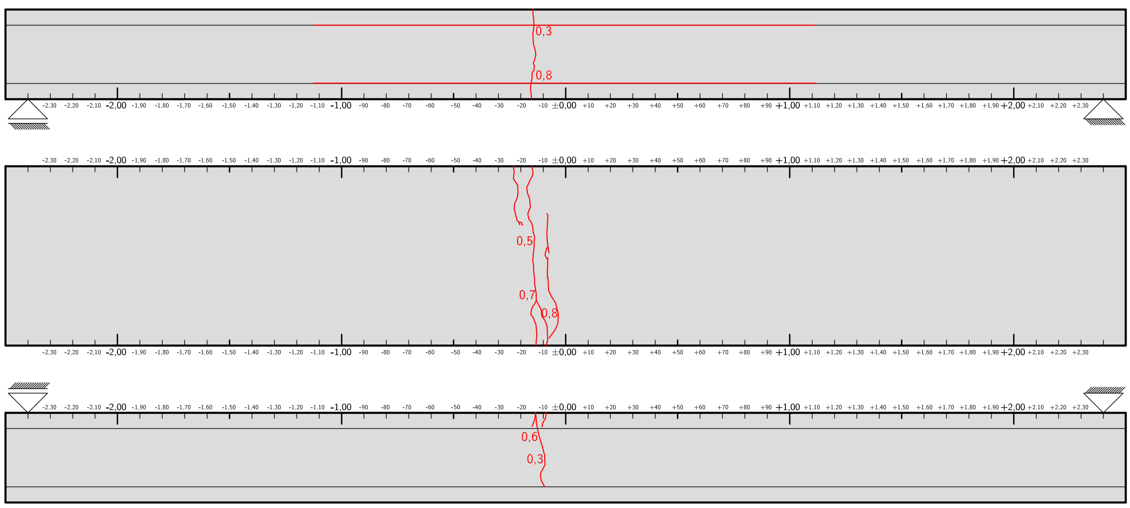


B.4 Fatigue test 4

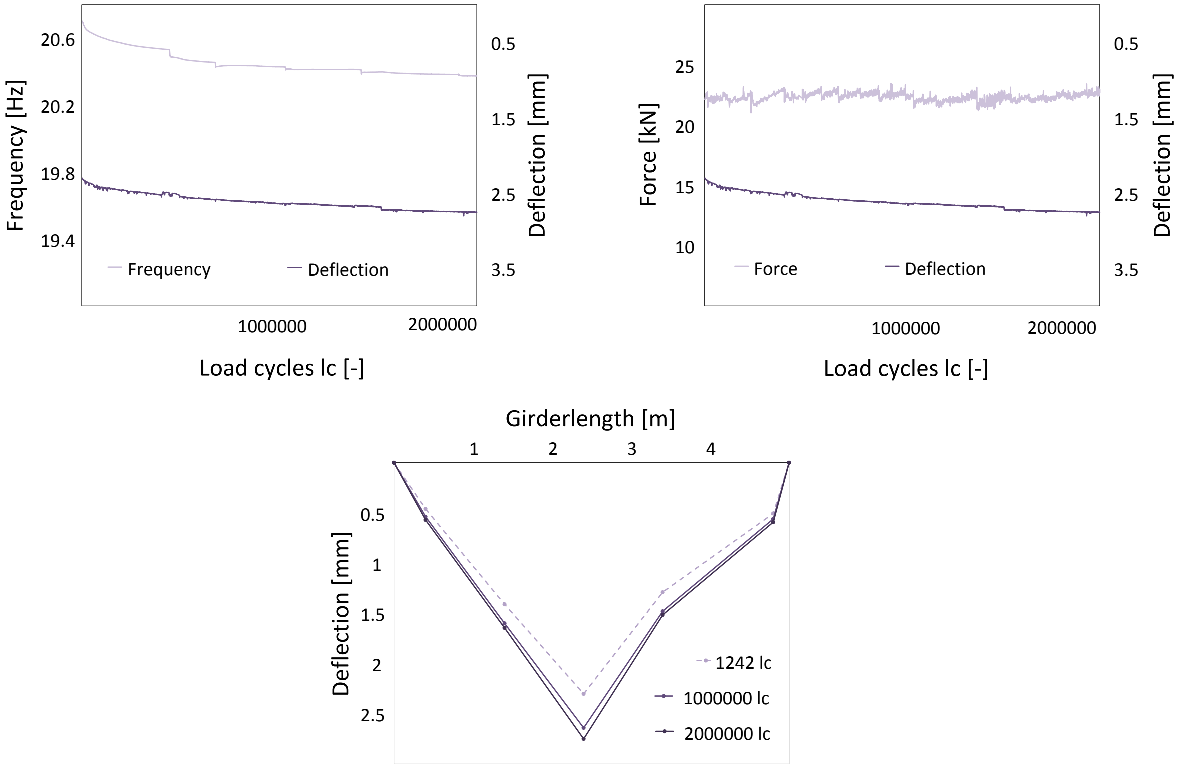




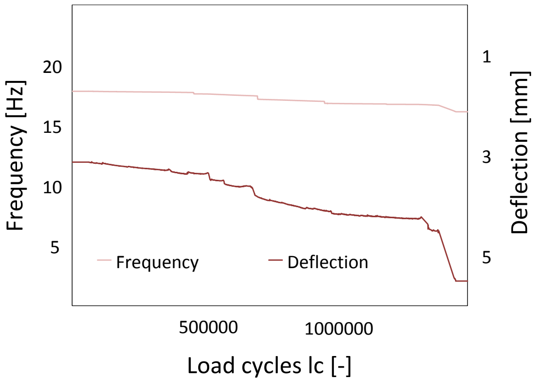


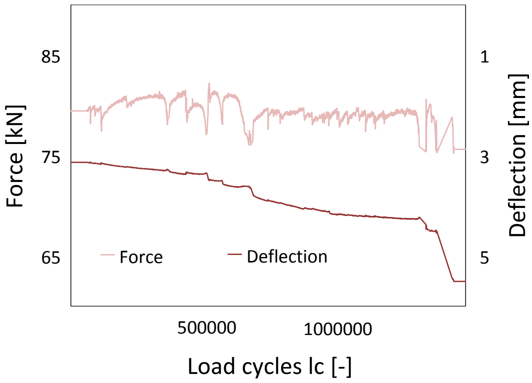


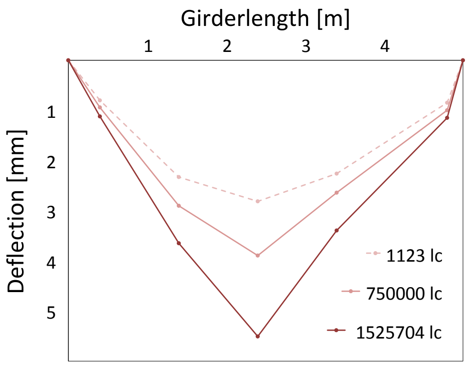
B.5 Fatigue test 5

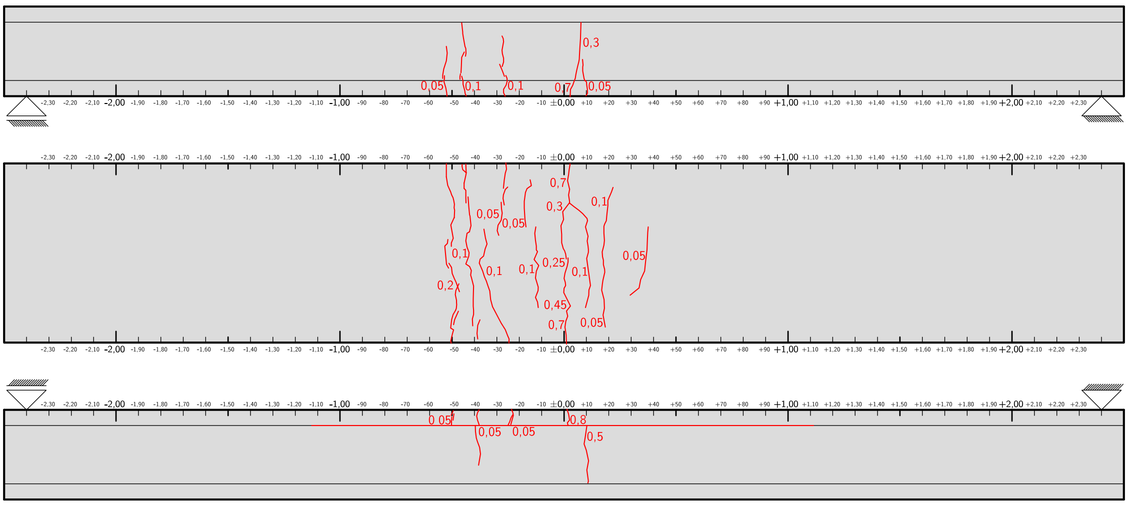


B.6 Fatigue test 6

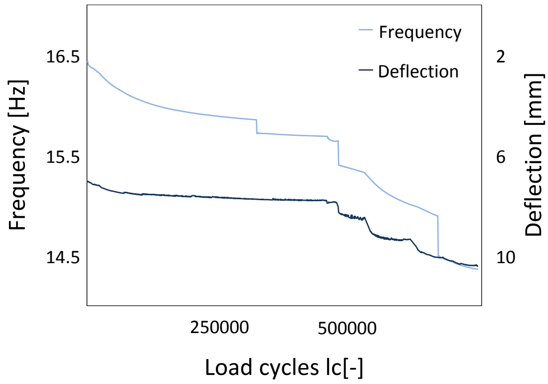


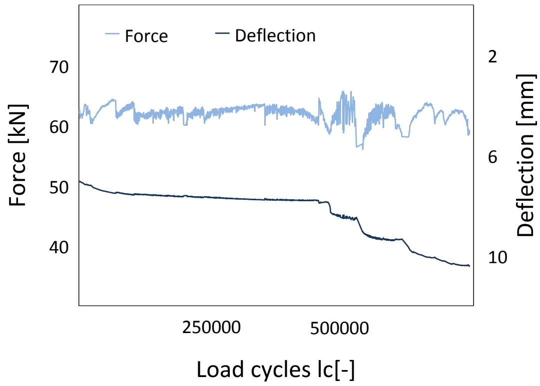


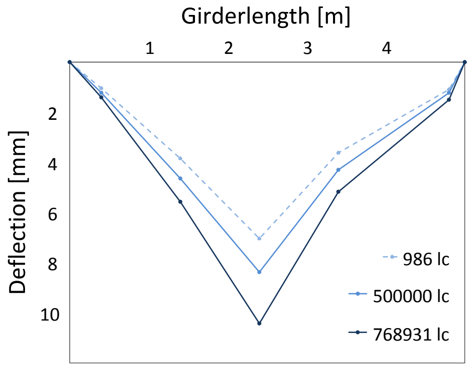


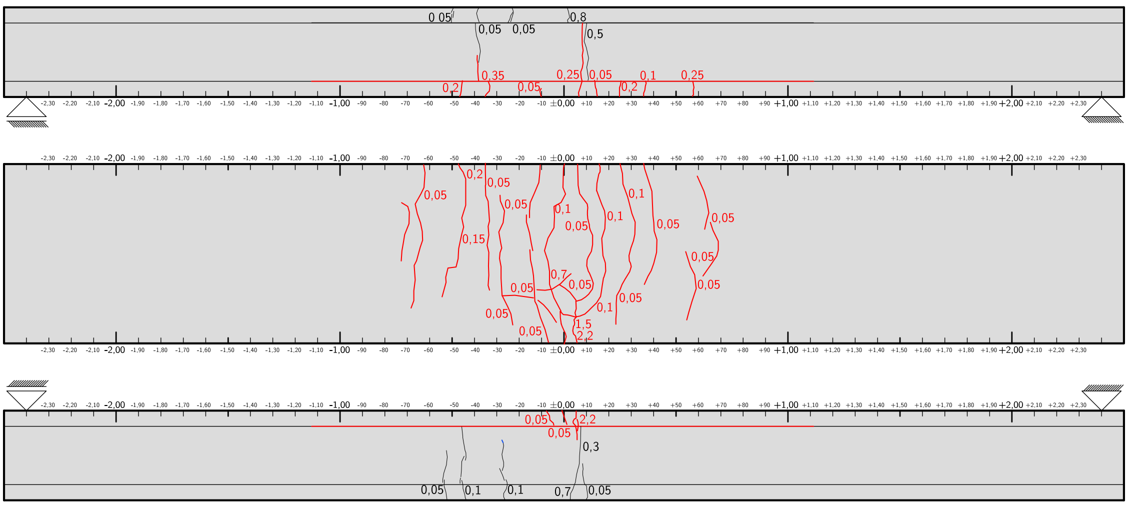


B.7 Fatigue test 7









B.8 Fatigue test 8