The reuse of cast-in-situ concrete elements in new building structures

A contribution to sustainable structural engineering with a focus on reconnecting reusable elements

Master thesis by
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Preface

This thesis is the final step in completing my Master's degree in Structural Engineering at Delft University of Technology. The research was conducted during my graduation internship at Witteveen+Bos, where I had the opportunity to explore a topic that merges sustainability with structural engineering: the reuse of cast-in-situ concrete elements in new building structures.

The motivation behind this study comes from the increasing urgency of sustainable construction practices. With the growing focus on circularity and material reuse, it is crucial to investigate whether structural concrete elements from buildings scheduled for demolition can be effectively reused into new designs. This research contributes to that goal by exploring practical reconnection methods for reused elements.

I would like to thank my supervisors at TU Delft, Sander Pasterkamp and Sandra Barbosa Nunes, for the insightful discussions and valuable feedback throughout this process. I am also thankful to my sunpervisor at Witteveen+Bos, Maartje Dijk, for her practical experience, guidance and for providing an inspiring environment in which to develop this research. Lastly, I would like to thank my friends and family for their support and encouragement throughout my studies.

I hope this thesis contributes to the further development of sustainable structural engineering solutions and provides a foundation for future research in this field.

Femke Middeldorp
Delft, March 2025

Summary

The built environment plays a crucial role in shaping our daily lives, but it also significantly contributes to resource consumption and environmental pollution. With the housing shortage in the Netherlands, construction activities continue to grow, making the need for sustainable solutions increasingly urgent. The Dutch government aims to be climate-neutral by 2050, with an interim goal of at least 55% CO_2 reduction by 2030. One potential strategy to support these objectives is the reuse of structural elements from existing buildings, reducing the demand for new materials and minimizing construction waste.

This research explores the feasibility of reusing cast-in-situ concrete elements in new building structures. These elements were not originally designed for disassembly and reuse, making their integration into a new structural system challenging. The study identifies structural and practical challenges, including the effects of cutting, transport, as well as the changes in support conditions, force distribution and connection detailing required for reintegration.

To address these challenges, a literature review was conducted, complemented by a theoretical analysis of the behavior of structural concrete elements to assess the effects of reuse on element performance. Additionally, two existing cast-in-situ buildings were analyzed to evaluate the theoretical challenges in real-world cases. Various reconnection methods were designed and evaluated using a Multi-Criteria Analysis (MCA), considering element modification, the applicability of standard techniques, construction effort, overdimensioning of reused elements within the new design and required precision during assembly. The results demonstrate that both fixed and hinged connections, either with reused elements or in combination with new ones, can be successfully implemented. However, for initial implementation, hinged connections that combine reused and new elements are recommended as the most practical solution for application in the current construction industry.

The study concludes that reusing cast-in-situ concrete elements can be a viable strategy within circular construction practices, provided that careful consideration is given to reconnection detailing and structural adaptation. However, for large-scale implementation, further research is needed to develop standardized assessment methods, ensure compliance with modern design codes, and validate structural performance through full-scale testing. Additionally, exploring digital tools for inventory management and adaptive design could further support the feasibility of reuse. Addressing these challenges will enable structural reuse to become a standard practice in the construction industry, enhancing its role in circular building strategies.

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1 Introduction

1.1. Research context

Civil engineering is one of the most impactful human activities affecting the natural environment (Tiza et al., 2022). A significant contributor to this impact is the extensive use of concrete, which is after water the most consumed material in the world (Monteiro et al., 2017). This huge consumption highlights the urgent need for sustainable practices in concrete constructions to limit environmental degradation. Circular Economy strategies help limit the demand for new raw materials. As a result, the amount of construction waste can be decreased, which promotes more sustainable building practices (Kupfer et al., 2023). Circular Economy has been divided into 10 strategies known as the R-ladder, ranging from $R0$ to $R9$ (Reike et al., 2017), shown in figure 1.1.

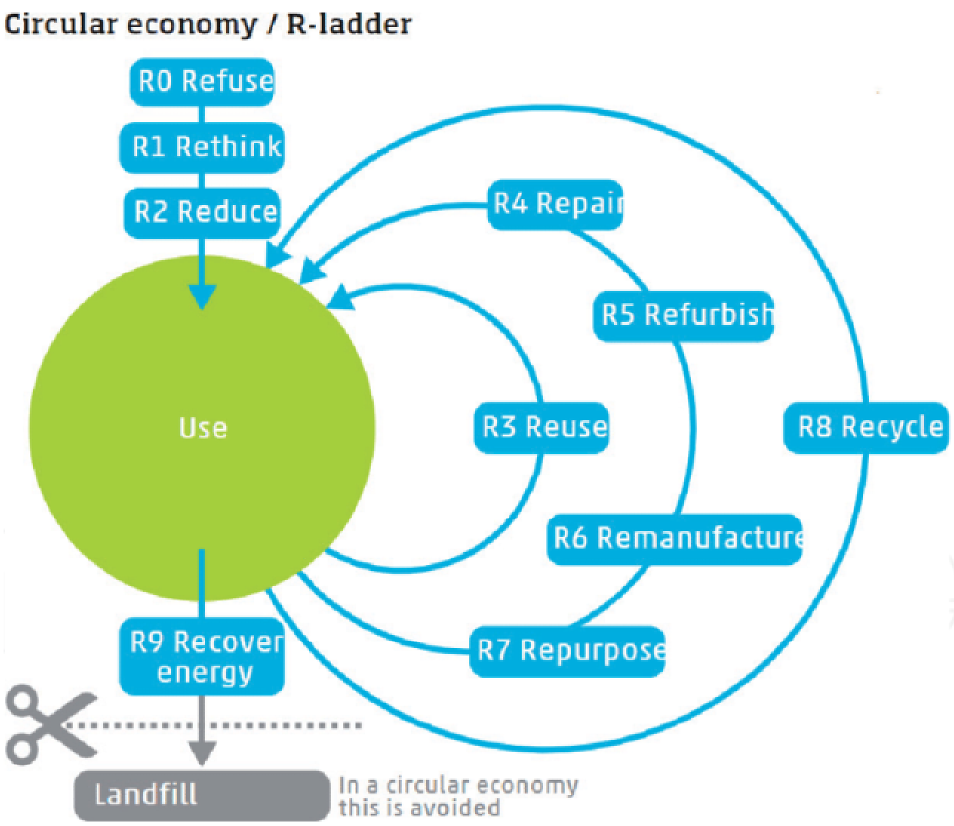


Figure 1.1: R-ladder of circularity strategies (Minguez et al., 2021)

In the context of building structures, the first three strategies focus on extending the use of structures as long as possible without modification. The third strategy emphasizes that if building removal is unavoidable, deconstruct it and reuse its pieces in another project with minimal reprocessing. The last strategies involve demolishing components that are not reusable into the manufacture of a similar or different product (Kupfer et al., 2023). The reuse of load-bearing components, the third circular strategy, has been shown to offer significant environmental benefits. Various projects and simulation studies have already demonstrated its potential to reduce integrated greenhouse gases, making it an area of focus in sustainable building practices (Küpfer et al., 2021; Röck et al., 2020). Therefore, this research focuses on the reuse of structural components from concrete building demolition in new projects. The reuse of these elements involves carefully disassembling old structures into usable structural elements for reassembly in a new design with minimal alterations to their geometric and mechanical properties (Suchorzewski et al., 2023).

1.2. Research problem

Numerous studies demonstrate applications for reusing structural components, both in scientific literature and in practical applications. However, these advertised applications mainly concern structures made of steel or timber (Kupfer et al., 2023). For cast-in-situ concrete elements, reuse presents additional challenges due to their original monolithic nature and lack of demountability. While there is increasing interest in the reuse of concrete elements, research has mainly focused on assessing their structural feasibility and environmental benefits rather than their direct integration into new structures. For the reuse of concrete elements, guidelines and standards are needed to ensure that practices are consistent across various construction projects (Jabeen, 2020; Monteiro et al., 2017; Widmer, 2022). Further research should focus on the development of suitable reconnection details that maintain structural integrity and enable effective reuse in new designs (Kupfer et al., 2023; Volkov, 2019).

1.3. Research objectives

The objective of this research is to identify the most suitable load-bearing concrete elements from existing cast-in-situ structures for reuse and to develop effective reconnection details that facilitate their integration into new structural systems. The study aims to provide practical solutions for reusing these elements while ensuring structural integrity, feasibility and alignment with conventional construction practices.

1.4. Research scope

This research focuses on the element reuse of cast-in-situ concrete buildings. Specifically, it involves the direct reuse of concrete elements from cast-in-situ structures without significant modifications. Unlike recycling or demolishing, element reuse involves preserving the structural integrity of concrete element while preparing them for integration into new projects. This includes the necessary preparation of reconnection details to ensure their compatibility and functionality in new structural designs.

The scope is limited to load-bearing elements and excludes the reuse of components designed to be used multiple times and precast concrete components. Additionally, the study only considers the reuse of concrete from existing structures, not new concrete. It is assumed that the elements have already been tested and approved for reuse, meeting all required structural and performance standards for their new application.

1.5. Research questions

To achieve the goals outlined in the previous sections, specific research questions were formulated. This study is structured to address these questions and provide insights into their answers. The main research question is:

”*Which load-bearing concrete elements from cast-in-situ existing structures are most suitable for reuse and how can their reconnection details be designed to enable effective reuse?*”

The main question is divided into sub-questions, which have been addressed to provide a clear answer. These include:

- *Which structural elements from cast-in-situ construction in the Netherlands are suitable for reuse?*
- *What are the structural and practical challenges in adapting cast-in-situ elements for reuse in prefabricated systems?*
- *What existing and innovative connection details are suitable for ensuring the structural integrity of reused load-bearing concrete elements from cast-in-situ structures?*

1.6. Methodology

This research follows a structured methodology to assess the feasibility of reusing cast-in-situ concrete elements in prefabricated systems. The study begins with a literature review to identify existing knowledge on cast-in-situ concrete structures and the challenges associated with their reuse. This is followed by a theoretical analysis to quantify the structural implications of reusing these elements. To bridge theoretical insights with practical applications, two real-world case studies of existing cast-in-situ buildings in the Netherlands are analyzed. Based on these analyses, various reconnection strategies are designed and evaluated through a multi-criteria analysis, ultimately providing an informed answer to the main research question.

2
Literature review

The reuse of concrete elements in the construction industry is an interesting topic in sustainable building practices, aligning with circular economy principles. This literature review aims to provide an overview of the current state-of-the-art in this field, the methodologies already used to address the problem and identifying research gaps.

2.1. Introduction to reuse of cast-in-situ concrete elements

Monteiro et al. (2017) highlight the urgent need for sustainable practices in concrete production due to its significant environmental impact. Concrete is the most widely used material in the world after water and its production is resource-intensive. The main contributor to its environmental footprint is cement production, which accounts for approximately 8% of global CO_2 emissions. This is primarily due to the calcination of limestone and the combustion of fossil fuels, processes that together release nearly one ton of CO_2 per ton of cement produced. Their study also projects that global cement production will increase by 50% by 2050, making it crucial to reduce concrete production to achieve the Dutch government goal of being climate neutral by 2050 (Rijksoverheid, 2024). According to Kupfer et al. (2023), the reuse of concrete components can significantly reduce environmental pollution in the construction industry. Their study provides an overview of 77 case studies on the reuse of concrete elements, including both precast and cast-in-situ components. Cast-in-situ elements are mostly reused for foundations or pavements, highlighting a gap in their application for new building structures. This is mainly due to the lack of standardized reconnection methods, challenges in reinforcement continuity and limited knowledge on reuse strategies. Foundations and pavements typically require fewer structural modifications than load-bearing structures, as they are primarily subjected to compressive forces and can sometimes tolerate minor surface degradation. To enable a greater contribution to circular construction, further research is needed on the reuse of structural concrete elements into new building structures, particularly regarding their reconnection methods and structural performance (Kupfer et al., 2023).

2.2. Environmental and economic benefits of reuse

Jabeen (2020) explored the economic feasibility of reusing structural concrete components, creating a feasibility tool. The findings indicate on-site reuse is the most cost-effective approach, as it eliminates transportation costs and logistical complexities which are major cost drivers in reuse. In contrast, off-site reuse and storage-based reuse are less favorable. The research also highlights that the costs of selective deconstruction for reuse vary significantly by component, with floor slabs accounting for 42% of total deconstruction costs, followed by walls for 15%, columns for 9% and beams for 8%. To facilitate reuse, the study of Jabeen (2020) emphasizes the role of planning and policy instruments. Higher landfill taxes on non-reusable products could stimulating the use of secondary materials by making them more competitive in the market. The financial benefits of reuse primarily stem from savings on material costs, as fewer new concrete and reinforcement materials are needed and transportation costs in on-site reuse scenarios.

Beyond direct financial savings, reuse also reduces shadow costs (the indirect environmental costs associated with raw material extraction), energy consumption and waste disposal. The research by Glias (2013) found that the shadow cost of reusing secondary components is 75% lower than using new components, significantly improving the economic feasibility of reuse (Jabeen, 2020). This highlights the broader economic and environmental benefits of structural reuse. Kupfer et al. (2023) further emphasize the environmental benefits of concrete reuse, including the conservation of natural resources, waste reduction and lower energy demand for new material production. By extending the lifespan of structural elements, reuse minimizes the demand for virgin materials, contributing to more sustainable and circular construction practices.

More studies show that structural reuse of concrete can significantly reduce environmental impact. Widmer (2022) demonstrated that reusing sawn cast-in-situ concrete elements can lead to a 71% reduction in CO_2 emissions compared to conventional construction methods. Similarly, Naber (2012) found that reusing hollow-core slabs resulted in a $53 - 56\%$ lower carbon footprint compared to newly produced prefabricated slabs. These findings underscore the potential of structural reuse in reducing raw material extraction, energy consumption and overall environmental impact.

2.3. Design proposals for the reuse of concrete elements

Volkov (2019) identifies specific challenges of reconnection methods for reused concrete elements, particularly in ensuring reinforcement continuity, adapting elements to new connections and managing design complexity. His research proposes detailed reconnection designs to facilitate the reuse of elements, treating all elements as precast in a new design. He developed several proposals for four connection details: column to foundation block, column splice joint, column to beam node and a floor to beam connection. Since this research focuses on the reuse of cast-in-situ elements, only the reconnection details from Volkov (2019) that are potentially applicable to cast-in-situ concrete structures are discussed below. The other solutions in the study are designed for precast elements, which differ in their reinforcement layout and fabrication process, making them less suitable for direct application to cast-in-situ components.

In figure 2.1, four proposals of Volkov (2019) are presented for connecting a reused column to a newly constructed foundation block. In proposals (a) and (b), an overlap is created with the existing vertical reinforcement in the column, with (a) allowing the bars to be welded together. In (c), the reused column is inserted into a foundation pocket. Lastly in (d), a head plate is welded onto the existing vertical reinforcement of the column, enabling connection to the foundation using bolts.

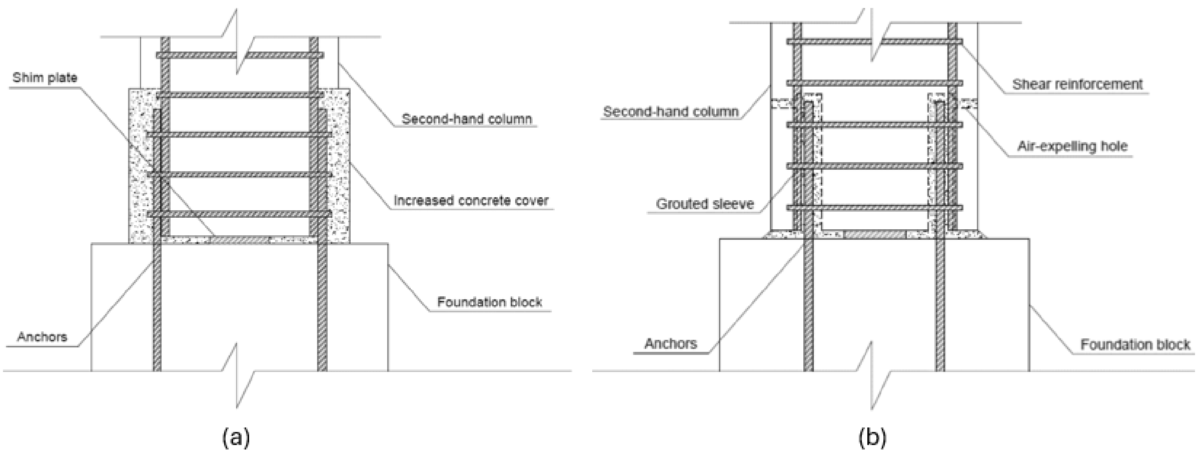


Figure 2.1: Design proposals for reconnecting a reused column to a foundation block (Volkov, 2019) (continued on next page)

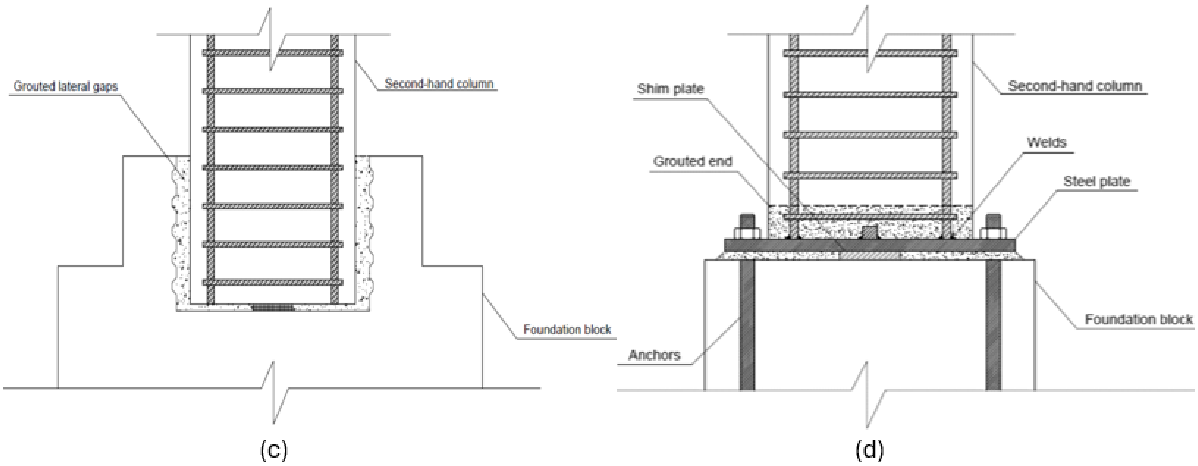


Figure 2.1: Design proposals for reconnecting a reused column to a foundation block (Volkov, 2019) (continued)

Figure 2.2 presents two of Volkov (2019) proposals for reconnecting a reused column to a new column using a splice joint. In (a), a grouted sleeve connection is used, where connection bars are inserted into grouted sleeves to ensure structural continuity. In (b), a bolted splice connection is applied, with a steel plate placed on the existing column, to which the second-hand column is bolted using steel bolts.

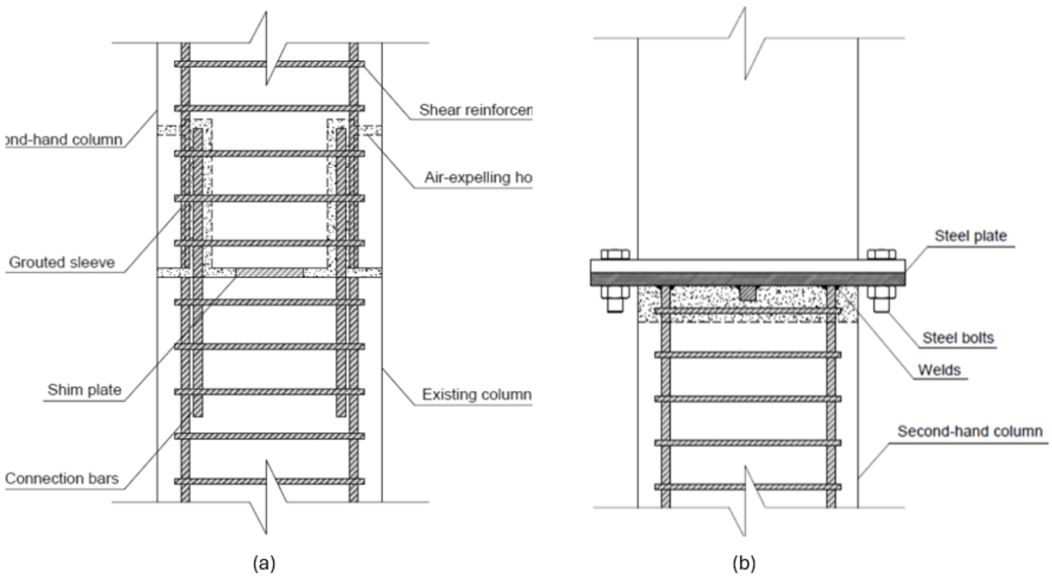


Figure 2.2: Design proposals for reconnecting a reused column to new column with a splice joint (Volkov, 2019)

Figure 2.3 (a) and (b), on the following page, illustrates two proposals for reconnecting reused beams. Both solutions involve exposing the original reinforcement by selectively removing concrete at the beam ends, allowing for a welded connection with the longitudinal reinforcement. In (b), an additional stirrup is incorporated between the beams to enhance shear force transfer, improving the load-bearing capacity of the connection.

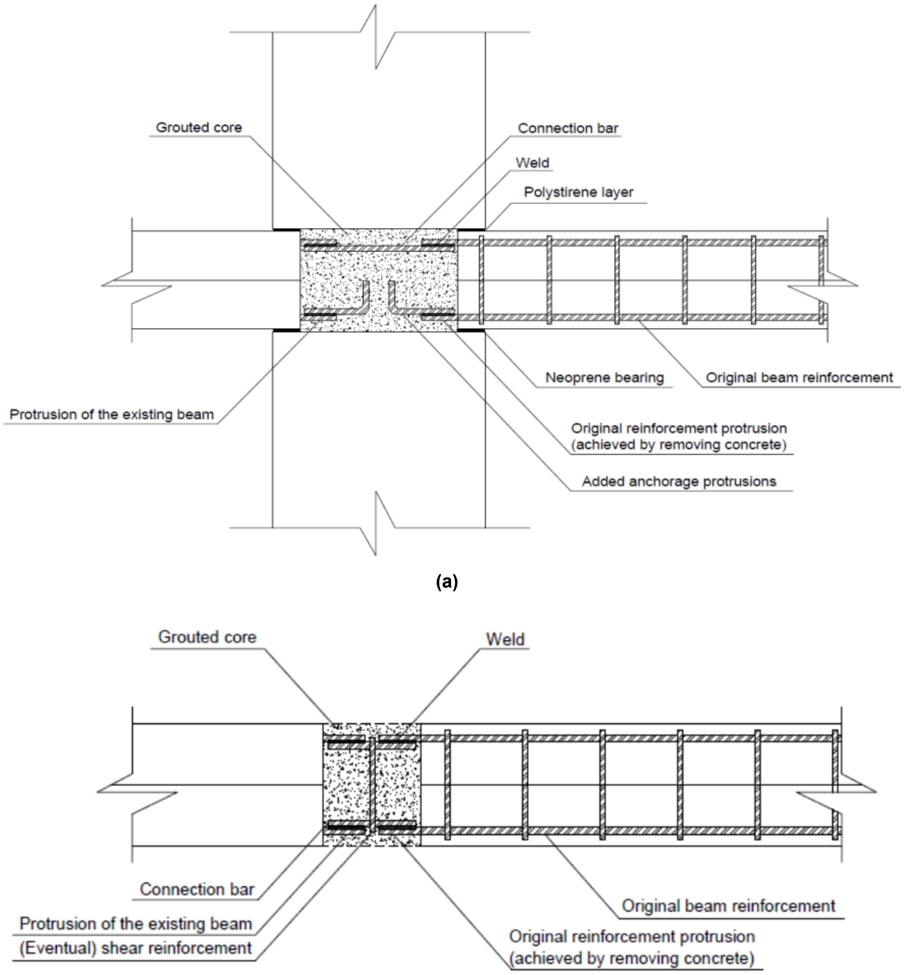


Figure 2.3: Design proposal for reconnecting reused beams (Volkov, 2019)

In figure 2.4 a reconnection detail for reusing wide-slabs with an new prefabricated beam is shown. The reused floor is supported on a neoprene bearing and the voids are filled with grout to create structural continuity. A connection bar is integrated within the newly cast top of the beam, ensuring force transfer between the reused floor segments.

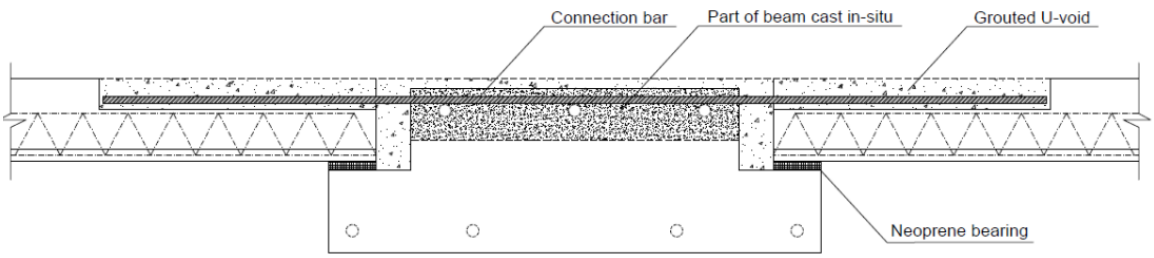


Figure 2.4: Design proposal for reconnecting wide-slab floor with a new poured beam (Volkov, 2019)

These proposals build on traditional reconnection methods and inform the development of standardized reconnection techniques for cast-in-situ concrete components. While traditional reconnection methods rely on mechanical and grouted connections, emerging materials offer alternative solutions to improve durability and performance.

Monteiro et al. (2017) suggest exploring advanced materials such as self-healing concrete and alkali-activated binders to ensure the reliability of reused concrete components. They emphasize the need for standardized guidelines and long-term durability data to facilitate widespread adoption and ensure structural integrity in reuse applications. Additionally, Widmer (2022) propose the use of Ultra-High Performance Fiber-Reinforced Concrete (UHPFRC) to enhance tensile force transfer in reconnection zones. This is achieved through its superior bond performance and increased crack resistance, which improve stress redistribution. Furthermore, UHPFRC's high compressive strength makes it a suitable material for integrating reused cast-in-situ elements into new structures. Additionally, it can strengthen aged or deteriorated elements, restoring their load-bearing capacity and improving their durability in reconnection zones. However, its high material costs and specialized application process may limit feasibility in large-scale reuse. Further research is needed to assess its cost-effectiveness and practical implementation (Widmer, 2022).

2.4. Methodologies for assessing reusability

The assessment of reusability for cast-in-situ concrete elements requires systematic evaluation methods that account for both structural integrity and long-term durability. Several methodologies have been developed to facilitate this process, incorporating damage classification, service life prediction and structural integration into new designs.

Suchorzewski et al. (2023) developed a methodology for assessing the structural state of existing buildings and selecting elements suitable for reuse. Their framework includes guidelines for dismantling, storage and installation. The proposed classification system for concrete elements is based on parameters including remaining service life and degree of cracking, which are essential for establishing assessment criteria in the reuse of cast-in-situ concrete components, as they directly influence the structural reliability and durability of the reused elements. A simplified mathematical model with two stages of degradation, coating degradation and reinforcement corrosion, is developed and validated on real buildings. This model accounts for degradation parameters including exposure to moisture, carbonation depth and environmental stressors. The model is implemented in their designed service life calculation tool for reused elements in Excel. The results of their study highlighted the importance of considering the new design and intermediate storage conditions for the lifetime of concrete elements. For example, exposure to humidity, temperature fluctuations and mechanical damage during storage can significantly reduce reusability in the new structure.

Devènes et al. (2024) contribute to the assessment of load-bearing reinforced concrete components from cast-in-situ buildings by introducing a methodology that evaluates their long-term durability and reusability. Their approach classifies components into a damage class (figure 2.5), use class (figure 2.6) and intervention class (figure 2.7) to determine a reusability grade of the concrete elements with the matrix in figure 2.8. With this method it is possible to evaluate the reuse potential of the concrete components. The damage class is based on visual inspection, assessing the damage to the elements that may affect their serviceability and structural resistance. The size of the damage is evaluated in relation to the overall dimensions of the element. The damage classification is shown in figure 2.5 on page 9.

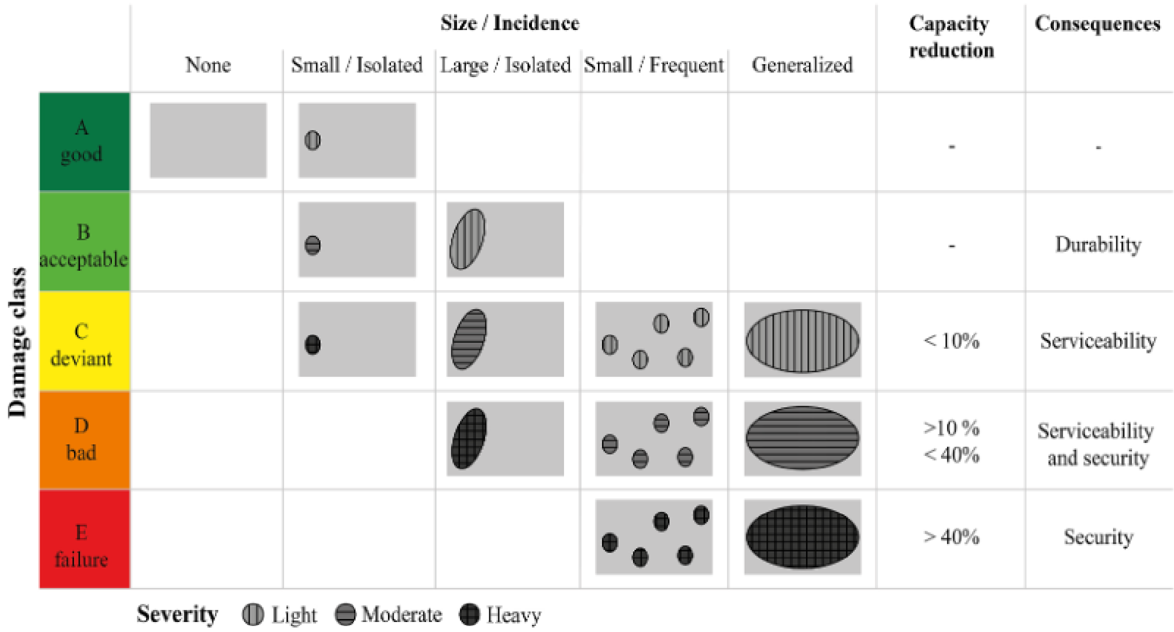


Figure 2.5: Damage classification by visual inspection (Devènes et al., 2024)

The use class categorizes reinforced concrete elements based on their structural stability requirements and exposure to water in their new application. The classification ranges from I to V, where class I represents elements with minimal stability criteria and low exposure, while class V includes components that must remain stable under external loads and are highly exposed to water. This classification helps determine whether a reused element meets both structural and durability requirements in its intended function.

Use class assignment based on the stability and water exposition.

Use class	Lightly exposed	Moderately exposed	Highly exposed
No stability criteria	I	II	III
Self-stable	II	III	IV
Stable under external loads	III	IV	V

Figure 2.6: Use classification (Devènes et al., 2024)

The intervention class defines the extent of modifications, strengthening, or rehabilitation required for a reused reinforced concrete component before its integration into a new structure. It is determined during the design phase of the receiving project and must remain economically and environmentally proportional. Class a requires no intervention beyond extraction, b involves preventive maintenance or minor strengthening with simple cutting, while c includes significant rehabilitation or reinforcement with complex cutting or modifications. If the necessary interventions exceed reasonable economic or environmental costs, reuse should be reconsidered.

Intervention class definition.

Intervention class	Maintenance measures	Geometry modifications
a	No action	No further cutting after extraction
b	Preventive maintenance, light strengthening	Simple cutting
c	Curative maintenance, rehabilitation, medium to important strengthening	Complex cutting or modification

Figure 2.7: Intervention classification (Devènes et al., 2024)

The reusability grade of a reused element is determined by combining its damage class, use class and intervention class, categorizing elements on a five-level scale. Components with a grade of 1 or 2 are suitable for reuse, while those graded 3 may require further evaluation. Elements with a grade of 4 or 5 are considered unsuitable for reuse due to safety concerns. The grading decision matrix is shown in figure 2.8. This methodology, tested on three case studies in Switzerland, demonstrated its robustness. After classification, elements with marginal scores may require further material testing, such as compressive strength tests or reinforcement scanning, to confirm their usability in new structural applications. This method helps determine whether components are suitable for reuse. Further research into how to implement these components in a new design is still necessary (Devènes et al., 2024).

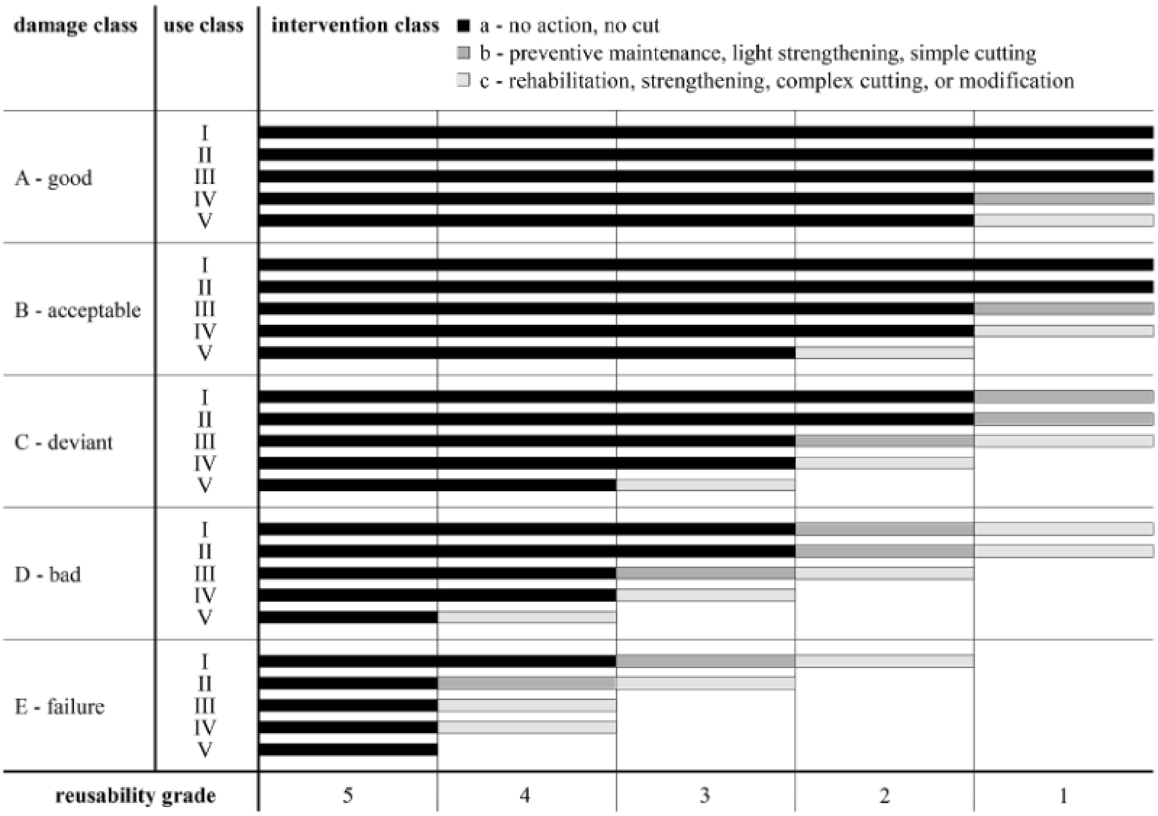


Figure 2.8: Reusability grading decision matrix (Devènes et al., 2024)

This classification system is relevant for the reuse of concrete elements, as it provides a structured approach to assessing their condition and reusability. The damage class helps evaluate whether an element retains sufficient structural integrity, while the use class determines its suitability for various applications based on stability and exposure criteria. Additionally, the intervention class indicates the level of modification required before reintegration into a new structure. By systematically grading reused elements, this methodology enables more informed decision-making on reuse feasibility, necessary strengthening measures and appropriate reconnection techniques. While effective, this method requires specific considerations for cast-in-situ elements, particularly regarding reinforcement continuity and reconnecting challenges. Unlike prefabricated components, they often need additional evaluation and adaptation to ensure structural integrity in their new application. Additionally, marginally graded elements need further testing, increasing implementation complexity. However, its structured approach enables early-stage reusability assessment, supporting informed decision-making before deconstruction. Future research should refine this framework for cast-in-situ applications, addressing challenges in reinforcement exposure and reconnection detailing.

Widmer (2022) demonstrated the feasibility of constructing buildings from reused cast-in-situ reinforced concrete elements, comparing it to conventional construction methods. The assessment considered structural performance, economic feasibility and environmental impact compared to conventional methods. The study outlines a 12-step design process, developed by the researcher;

- step 1: Analyze the source buildings
- step 2: Define a preliminary floor plan of the target building
- step 3: Divide the target floor slab into elements, which are sought to be built in one piece
- step 4: Allocation of source slab elements to target slab
- step 5: Check deflections of the reassembled slab
- step 6: Allocation of vertical load-bearing elements
- step 7: Check columns for second order effects
- step 8: Check punching resistance of reassembled floor slab
- step 9: Design connections for gravity loads
- step 10: Design strengthening measures
- step 11: Seismic verifications
- step 12: Assessment and comparison to conventional construction method

This approach offers a systematic strategy for integrating reused cast-in-situ elements into new structures. It highlights critical aspects such as connection detailing, load redistribution and structural performance verification, which are important considerations in ensuring the feasibility of reuse.

Xia et al. (2022) introduce a sustainability-based reliability design paradigm for assessing the reusability of cast-in-situ concrete elements, integrating machine learning and physical modeling. The approach estimates carbonation-induced degradation by combining non-destructive testing (NDT) and climate data, allowing for a data-driven evaluation of reinforcement corrosion risks. By incorporating past service life data, this method predicts long-term durability under changing environmental conditions, ensuring that reused elements meet structural requirements. Factors influencing the degradation assessment are carbonation-induced reinforcement corrosion, freeze-thaw cycles and exposure to aggressive environments. The sustainability-based reliability design paradigm is illustrated in figure 2.9, outlining the calibration of degradation models, reliability design and validation steps.

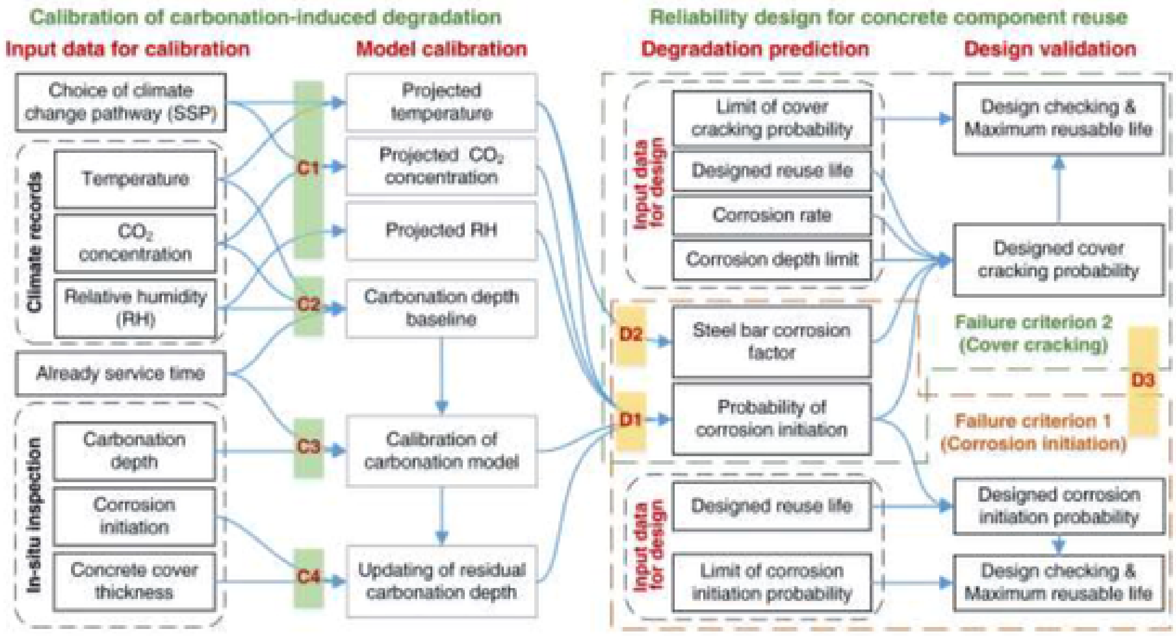


Figure 2.9: The workflow of sustainability-based reuse design paradigm (Xia et al., 2022)

This methodology combines non-destructive testing with climate records to estimate carbonation depth and the probability of reinforcement corrosion. This helps determine whether existing reinforcement remains adequate or if additional protection (coatings or extra concrete cover) is needed. The model also considers climate change impacts, predicting long-term durability under new conditions. By checking Failure Criterion 1 for corrosion initiation and Failure Criterion 2 for cover cracking, it assesses whether a cast-in-situ element is suitable for reuse or requires strengthening. This approach provides a structured method for evaluating the remaining lifespan of reused elements, ensuring safe and durable integration into prefabricated systems.

All the previously mentioned methodologies provide valuable insights into the structured assessment of reusing cast-in-situ concrete elements. However, to enable large-scale reuse in the Netherlands, these methods must be integrated into a unified framework that ensures consistency, structural reliability and practical feasibility.

2.5. Barriers to innovation in the construction industry

While the reuse of concrete elements aligns with sustainable practices and circular economy principles, its adoption in the construction industry remains challenging. According to Arnoldussen et al. (2017), various factors hinder the implementation of innovative building methods, including concrete element reuse, despite its environmental and economic benefits. One barrier to innovation in the construction industry is the regulatory environment, which prioritizes new materials over reused ones, making it easier to comply with established practices than to adopt new methods like reusing cast-in-situ elements. Strict quality and certification standards require reused materials to meet the same performance criteria as new ones, yet clear assessment and certification protocols for reused concrete elements are often lacking (Icibaci, 2019). This uncertainty complicates approval processes and discourages stakeholders from adopting reuse.

Another challenge is the lack of standardization in assessing and designing with reused elements. Unlike prefabricated concrete, which follows strict manufacturing standards, cast-in-situ components vary significantly in quality and reinforcement detailing, making it difficult to develop universal reuse guidelines (Icibaci, 2019). Furthermore, outdated regulations are largely based on conventional construction methods, failing to account for circular strategies such as structural reuse (Wessels, 2020).

Economic factors also play a role, as discussed in section 2.2. The costs associated with dismantling, processing and reusing concrete elements can sometimes exceed those of new materials. This is compounded by the lack of an established market for reused structural components, limiting financial incentives for reuse (Knuttson, 2023).

A further obstacle is the risk-averse behavior within the construction industry. Reuse introduces uncertainties regarding structural performance, cost variability and long-term durability, which can make stakeholders hesitant to implement new approaches (Arnoldussen et al., 2017). Without clear precedent or established practices, designers and contractors often prefer conventional solutions that offer predictability and regulatory certainty. This reluctance is compounded by the lack of technical expertise in assessing and designing with reused elements, particularly in areas such as structural integrity and reconnection detailing. The study by Kupfer et al. (2023) illustrates this hesitation: despite analyzing 77 case studies of concrete reuse, only 9 projects involved the reuse of structural elements from cast-in-situ buildings. Among these, only 3 cases reused walls, columns, or beams for new housing structures, highlighting the rarity of full structural reuse and the industry's prevailing caution in adopting such approaches.

The barriers identified highlight the importance of developing simple, standardized solutions that are easy to implement. Such approaches would lower the threshold for adoption by addressing uncertainties and simplifying compliance with existing regulations. Pilot projects and demonstration cases could help establish best practices and reduce industry hesitancy.

2.6. Conclusion and research gaps

The literature review highlights that significant progress has been made in understanding the feasibility of reusing cast-in-situ concrete components. Research has provided methodologies for assessing the structural integrity of elements, evaluating their environmental and economic viability and determining the potential benefits of reuse within the context of sustainable construction. Tools such as damage classification systems, service life models and machine learning frameworks have proven effective in identifying elements suitable for reuse and optimizing their integration into new designs.

Despite these advancements, several critical gaps remain. Standardization of reconnection methods is still lacking, which is essential to ensure the structural reliability and practical implementation of reused cast-in-situ components. Current reconnection proposals are primarily based on precast concrete principles and require further adaptation for cast-in-situ applications. In addition, regulatory and economic barriers hinder large-scale adoption. Quality certification and approval processes for reused structural elements remain underdeveloped, creating uncertainty in compliance with building regulations. Furthermore, economic feasibility depends on dismantling efficiency and market availability, which are not yet optimized. The risk-averse behavior within the industry further slows adoption, as stakeholders remain cautious about deviating from conventional construction methods.

These literature review highlight the need for further research into standardized and practical reconnection solutions, as well as methods to identify the most suitable elements for reuse. By addressing these gaps, this research aims to contribute to the practical implementation of concrete reuse in structural design.

3

Preconditions and design framework

The reuse of structural elements from cast-in-situ concrete structures involves a complex process that extends beyond the scope of this research. To enable effective reuse, it is essential to address the challenges related to testing the original elements, assessing their current state and ensuring their suitability for new applications. This includes evaluating remaining service life, structural performance and compliance with modern design standards.

While these preconditions lie outside the scope of this study, they are crucial to ensure that the elements are ready for integration into new designs. This research focuses specifically on the design of reconnection techniques and assumes that all necessary testing and preparations have been successfully completed. By framing the broader process, this chapter highlights the foundational steps required for reuse and establishes the context in which this study operates.

3.1. Preconditions for structural reusability

To ensure that the reused structural elements are suitable for the new design, it is essential to conduct a series of assessments addressing durability, structural performance and compliance with design requirements. In the following subsections these aspects are outlined.

3.1.1. Concrete strength over time

The compressive strength (f_{ck}) of the concrete may have increased over time due to hydration (Betonhuis, 2021), but long-term exposure to environmental factors or previous loads could lead to degradation (Ismail et al., 2010). In contrast, the tensile strength tends to decrease over time due to micro cracking, aging effects and external influences (Betonhuis, 2021). Both parameters must be carefully assessed to ensure that the element can safely resist the newly applied loads and maintain its structural integrity in the new design.

3.1.2. Characteristic yield strength

Corrosion can reduce the yield strength (f_{yk}) of the reinforcement. It causes a reduction in the cross-sectional area of the steel and depending on the extent of the corrosion, it can also weaken the material itself (Lee et al., 2023). This parameter must be sufficient to withstand the newly applied loads and maintain the structural reliability of the element in its new design.

3.1.3. Deformation due to creep

The element may have already experienced deformation due to creep during its original use, which can influence its capacity to handle deformations in the new design. These pre-existing deformations must be considered as initial imperfections in the structural analysis, as they directly influence the magnitude of second-order effects. If these second-order effects exceed acceptable limits, the element may not be suitable for reuse in the intended application.

Furthermore, the effective creep coefficient (φ_{ef}) is typically lower for reused elements because the age of the concrete (t_0) is higher. As shown in figure 3.1 of Eurocode 2 (NEN-EN 1992-1-1+C2, Section 3.1.4; European Committee for Standardization, 2017), creep deformation decreases significantly as the concrete matures. While this may reduce the potential for additional creep in reused elements, the pre-existing deformations remain critical to assess.

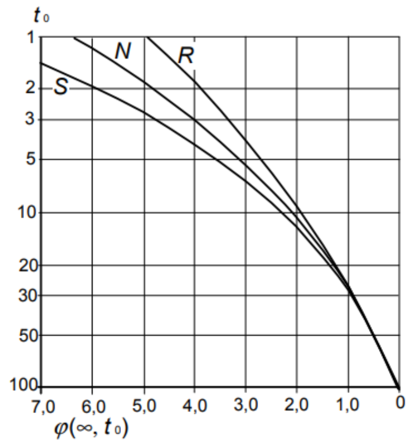


Figure 3.1: Determination of the creep coefficient $\varphi(\infty, t_0)$ (European Committee for Standardization, 2017)

3.1.4. Cracks

Inspection for cracks resulting from previous loads, removal, or transportation of the element is essential to ensure its suitability for reuse. According to Eurocode 2 (NEN-EN 1992-1-1+C2, Section 7.3; European Committee for Standardization, 2017), allowable crack widths are defined based on the exposure class and structural function of the element. All cracks should be thoroughly documented, including their width, depth and location, to verify compliance with these limits. Any cracks exceeding the permissible widths must be evaluated for their impact on the element's structural performance and durability and appropriate repair measures should be considered if reuse is still intended.

3.1.5. Concrete cover

According to Eurocode 2 (NEN-EN 1992-1-1+C2, Section 4.4; European Committee for Standardization, 2017), the thickness of the concrete cover is a critical factor in providing sufficient protection against carbonation-induced corrosion. The existing elements must have enough concrete cover to meet current standards for carbonation resistance, ensuring that the embedded reinforcement remains adequately protected from environmental effects over its intended lifespan. It is important to note that older structures often had smaller concrete covers, as the requirements for carbonation resistance were less stringent in the past. Additionally, the concrete cover must comply with the fire resistance requirements outlined in Eurocode 2 (NEN-EN 1992-1-2+C1, Section 5.3; European Committee for Standardization, 2017) to maintain structural integrity under fire conditions. If the existing concrete cover is insufficient for either carbonation resistance or fire resistance, solutions exist to add additional cover to the element. Protective coatings, such as epoxy or hydrophobic treatments, can reduce carbonation and moisture ingress. Alternatively, additional concrete layers incorporating silane additives or hydrophobic cement mixes can be bonded to the surface to improve resistance. In cases of advanced carbonation, chemical or electrochemical realkalization may restore alkalinity and slow reinforcement corrosion (Bauden, 2024).

3.1.6. Damage assessment

A comprehensive damage assessment is crucial for reuse suitability, including chloride ingress, frost-thaw cycles and chemical exposure. Additionally, phenomena such as alkali-silica reaction and carbonation must be checked, as they can compromise the structural integrity of the element. Porosity and water absorption should also be measured, as these influence the potential for ongoing degradation.

The preconditions discussed above serve as fundamental criteria for determining whether a cast-in-situ concrete element is suitable for reuse. Existing methodologies, such as those proposed by Suchorzewski et al. (2023) and Devènes et al. (2024), incorporate similar parameters into structured assessment frameworks.

3.2. Logistic planning

Reusing structural elements presents both technical and practical challenges that must be addressed for successful integration into new designs. Beyond ensuring that the elements meet the necessary structural standards, practical issues such as matching, storage and transportation must be carefully managed.

3.2.1. Matching

A complex aspect of reusing the elements is ensuring that they fit into the new design. This will require early coordination between architects and engineers to adapt the design to the dimensions and characteristics of the available elements. To coordinate this matching effectively, it can be useful to employ advanced algorithms (Kookalani et al., 2024). Such algorithms consider factors such as element dimensions, structural capacity and architectural constraints, helping to optimize the reuse process and reduce waste.

3.2.2. Storage

Reusing elements on the same site where they were removed is preferred, as this minimizes the need for transport and reduces the risk of damage (Jabeen, 2020). However, this is not always feasible, making it essential to identify a suitable storage facility. The storage location should be easily accessible to the project site while providing environmental conditions that protect the elements from carbonation and corrosion. Exposure to moisture or fluctuating temperatures can accelerate carbonation, lowering the pH of the concrete and leading to reinforcement corrosion. To mitigate these risks, indoor storage with controlled temperature and humidity is recommended. For outdoor storage, additional protective measures, such as sealing or wrapping the elements, may be required (Bauden, 2024).

3.2.3. Transportation

Transporting reused structural elements presents additional challenges, not only due to the risk of damage to the elements during handling and movement but also because of the negative environmental impact and logistical complexity. Keeping the transport distance as short as possible is essential. As it will be discussed in section 4.2.1, there are strict limitations on the size of elements that can be transported without requiring special permits or equipment, adding another layer of complexity. These constraints must be considered early in the design process to optimize logistics. Furthermore, elements may be sensitive to damage during transportation, such as cracks or chipping from improper handling, which can compromise their structural integrity in the new design. These risks, along with the dynamic forces acting on elements during transport, are further discussed in section 6.1.

3.3. Design assumptions

To focus this research on the design and feasibility of reconnecting reused elements, certain assumptions are made regarding the condition and applicability of these elements. It is assumed that a donor project has already been identified, meaning that the structural elements selected for reuse have been tested and deemed suitable for reintegration into the new structural system. This includes verification of their mechanical properties, durability and compliance with relevant structural codes. Additionally, it is assumed that the applied loads in the new design have been assessed and validated against the capacity of the reused elements. As a result, this research does not evaluate the general feasibility of reusing structural elements but instead focuses on the development and assessment of connection solutions to facilitate their effective integration into prefabricated systems.

4

Cast-in-situ construction methods and reusable elements

The reuse of structural elements from cast-in-situ buildings offers a sustainable alternative to demolition, but its feasibility depends on various factors. This chapter explores different cast-in-situ construction methods in the Netherlands and identifies which structural components can be extracted for reuse. Additionally, it examines the practical constraints affecting their extraction, transportation and compliance with modern building regulations.

4.1. Cast-in-situ building systems in the Netherlands

Cast-in-situ is widely used in the Netherlands for serial housing construction and non-residential buildings (Controleplannen.nl, 2024). For serial housing construction, tunnel formwork and wide-slab systems are frequently used, as these methods provide efficient and continuous building processes (Betonhuis, 2020; Gietbouwcentrum, 2020). Non-residential buildings often feature line-supported or point-supported structures. These support systems allow for more flexibility in floor plans and are particularly suited to the functional requirements of commercial, office and industrial buildings.

4.1.1. Tunnel formwork

Tunnel formwork is a construction method where concrete walls and floors are cast in a single operation, creating a monolithic structure (Betonhuis, 2024), as shown in figure 4.1. This construction type is often used in the Netherlands and is known for its fast production method (van der Vegte, 2008). The elements considered for reuse from tunnel formwork include walls and floors.

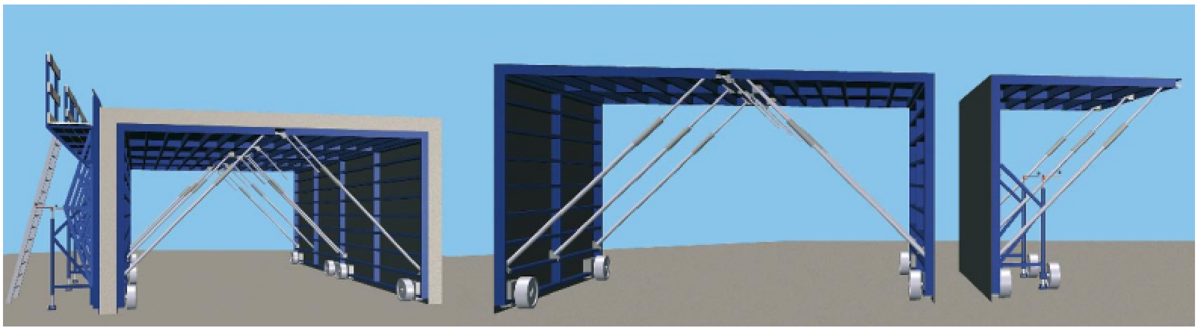


Figure 4.1: Tunnel formwork (Gietbouwcentrum, 2007)

4.1.2. Wide-slab system

The wide-slab system involves the use of cast-in-situ concrete walls combined with wide-slab floors, which are prefabricated concrete slabs, as shown in figure 4.2. After positioning the slabs, an additional layer of concrete is poured over them on-site, creating a composite floor structure (Gietbouwcentrum, 2020). From this construction method, walls and floors can be identified as potential elements for reuse.

Figure 4.2: Wide-slab system (Gietbouwcentrum, 2020)

4.1.3. Line-supported system

In a line-supported system, the floors are supported by beams and columns. The beam-column connection can be constructed in three ways, as shown in figure 4.3. In option a), the beam and column have the same width, while in options b) and c), the widths differ (Gerrits, 2008).

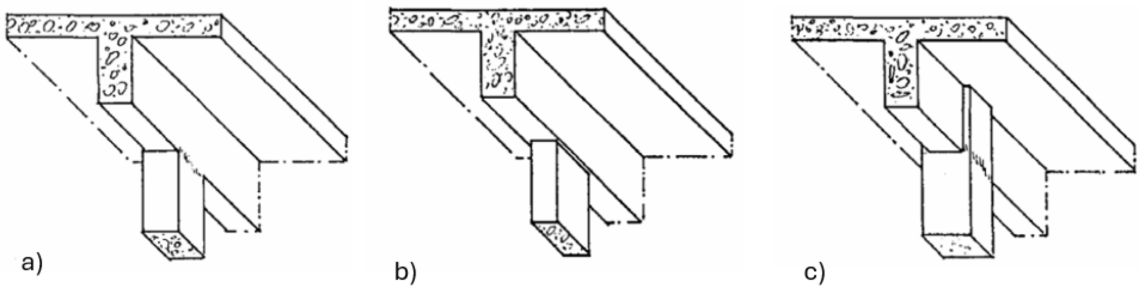


Figure 4.3: Line-supported system (Gerrits, 2008)

Beams, columns and floors are components that are potentially reusable from line-supported systems. Another option is to reuse a beam-column element, preserving the existing fixed connection between the beam and column. This can include, for example, a Pi or T element, as illustrated in figure 4.4. In this case, the advantages of the cast-in-situ concrete construction method are leveraged by utilizing the moment-fixed connections typical of cast-in-situ concrete buildings.

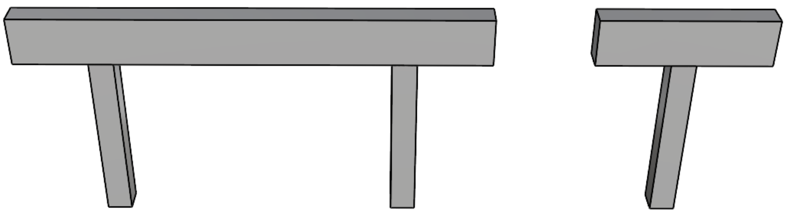


Figure 4.4: PI and T element

T-girders can also potentially be extracted from line-supported systems. They are commonly used in concrete construction due to their structural efficiency, which results from the combination of a high web and a wide flange. The top flange, together with the bottom chord, primarily resists bending moments, while the high web enhances shear resistance, making these elements especially suitable for long-span applications (Betonhuis, n.d.).

4.1.4. Point-supported system

In a point-supported system, the floors are supported by columns and they span in two directions. The connection of the floor to the column occurs in the following ways: with a column head, with a column plate, with both a column head and plate and without a column head or plate (Gerrits, 2008), as shown respectively in figure 4.5. From this construction method, columns and floors can be identified as potential elements for reuse.

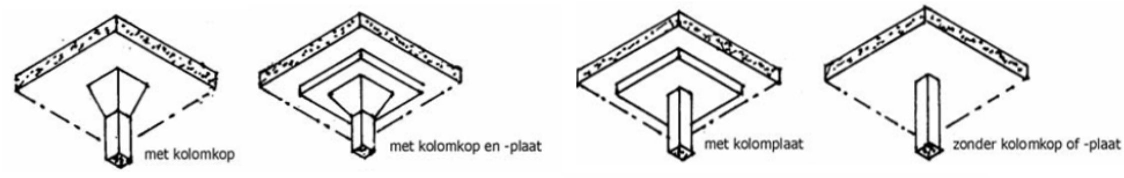


Figure 4.5: Point-supported system (Gerrits, 2008)

4.2. Constraints on the geometry of the elements

The reuse potential of these elements is not only determined by their extraction feasibility but also by practical constraints such as transportation, storage and compliance with modern building codes. These constraints discussed in this subsection can impact the geometry of the elements discussed in section 4.1.

4.2.1. Transportation

Transportation is a constraint in the process of reusing cast-in-situ concrete elements. The size and weight of structural components present logistical challenges that must be addressed to ensure safe and efficient movement from the deconstruction site to the new location. The maximum dimensions and weights for transportation in the Netherlands are specified in 'Regeling Voertuigen' (Dutch government, 2011). The size limits for standard transportation are as follows: maximum height is 4 meters and maximum length is 12 meters. This regulation outlines the legal limits for the size of vehicles to ensure safe transportation on public roads. However, in exceptional cases, larger elements can be transported with special permits, though this increases logistical complexity and costs. The maximum weight for road transportation, as described in the 'Regeling Voertuigen,' is 50 tons. However, the actual maximum weight also depends on the hoisting capacity of the cranes used to lift the elements, which will be discussed in section 4.2.2.

4.2.2. Hoisting capacity

The maximum hoisting capacity of construction cranes in the Netherlands depends on the type of crane and its specific configuration. Cranes come in various sizes and capacities, ranging from small mobile cranes to large tower cranes. Mobile cranes are known for their flexibility and heavy-duty performance. These cranes can lift from 40 tons up to 800 tons depending on the model and specific setup (Barneveldse Kraanverhuur, 2024; Bulten Materieel, 2024). On the other hand, tower cranes, which are widely used in high-rise construction, offer a different set of advantages. Although their lifting capacities are generally lower, ranging between 1.8 tons to 18 tons, tower cranes can lift to greater heights and over longer distances (Aboma, 2024; Beequip, 2024).

4.2.3. Current guidelines Besluit bouwwerken leefomgeving

In 'Besluit bouwwerken leefomgeving Artikel 4.28', specific guidelines are established for habitable spaces. This section specifies the minimum ceiling height of 2,6 m (Rijksoverheid Nederland, 2025) for habitable spaces to ensure they are functional and meet safety and health standards. When vertical elements are extracted from a building for reuse, they must have sufficient height to meet the requirement for habitable spaces.

4.3. Conclusion

This chapter has explored the feasibility of reusing structural elements from cast-in-situ buildings, considering different building systems, geometric constraints and logistical limitations. Various cast-in-situ construction methods commonly used in the Netherlands, such as tunnel formwork, wide-slab systems, line-supported systems and point-supported systems, provide potential sources for reusable elements. However, the extraction and reuse of these elements are subject to practical constraints, including transportation regulations, hoisting capacity and compliance with current building guidelines. Considering these factors, the structural elements most suitable for reuse include columns, beams, floors, walls and T-shaped columns. While T-girders and Pi-frames were initially considered, they were ultimately excluded due to practical constraints. T-girders pose challenges in extraction due to complex cutting requirements, making them less feasible for reuse. Pi-frames, on the other hand, are typically too large for efficient transportation, adding significant logistical difficulties. The selection of the identified elements is based on their ability to be effectively cut, transported and reintegrated into a new structural system. By identifying these elements, this chapter addresses the first sub-question regarding which structural components from cast-in-situ construction in the Netherlands are suitable for reuse. The reusable elements are presented in figure 4.6 below.

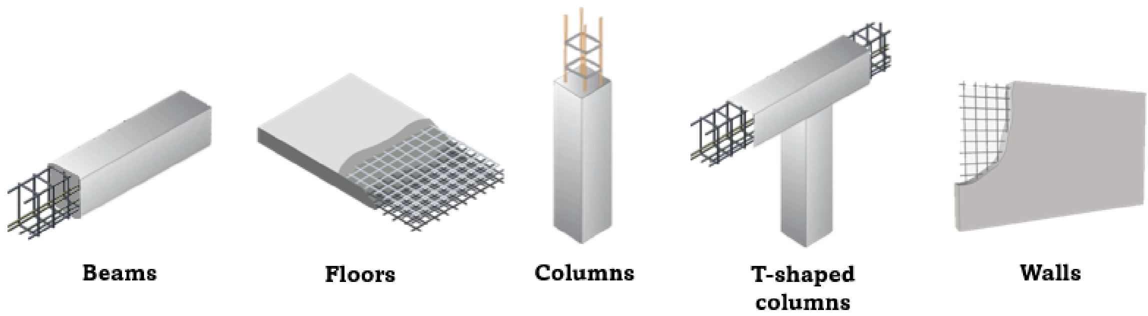


Figure 4.6: Identified reusable structural elements

5

Cutting the elements

When adapting cast-in-situ elements for reuse, the process of cutting these elements presents structural and practical challenges. Cutting disrupts the original continuity of the structural system, leading to the loss of original structural properties. These changes must be carefully assessed to ensure that the remaining elements retain sufficient structural integrity for reuse. In the following sections, considerations such as cutting methods, structural impacts and specific design limitations are discussed to guide effective reuse of cut elements in prefabricated configurations.

5.1. Cutting process

In cast-in-situ designs, elements are poured as part of a continuous structure, making it necessary to cut through reinforced concrete. Diamond saws are highly effective for cutting reinforced concrete. The diamond blade allows for precise, smooth cuts through both the concrete and the embedded steel reinforcement. However, alternative methods, such as wire saws or hydrodemolition, may be considered depending on the accessibility of the element and the need to preserve the remaining reinforcement (Bortolussi et al., n.d.; Sokolov, n.d.).

Cutting at locations where the moment and associated stresses are highest, such as near mid-span or supports, can pose challenges. High bending moments in these regions create a risk of excessive cracking or reinforcement detachment during the sawing process. Therefore, it is preferable to saw at locations where the moment is zero, reducing the risk of damaging the reusable element. If there is a need to make the cut at a location with high stresses, temporary supports will be required to reduce the stresses at the cutting location (Li et al., 2024). This can be achieved through external strut systems, tension release techniques, or temporary prestress to minimize stress concentrations before cutting.

5.2. General challenges after an element is cut

5.2.1. Anchorage length

Due to the cutting of the elements, some or all of the anchorage length originally required in the design will no longer be present in the reusable element. Additionally, it is important to note that in the past, smooth reinforcement bars were commonly used (BetonLexicon, 2023). According to NEN 8702, a factor of 0.5 is applied for the anchoring strength of smooth reinforcement bars, indicating that the required anchorage length must be twice as long compared to ribbed bars to achieve the same level of reinforcement anchorage (European Committee for Standardization, 2017). Given these considerations, it is crucial to develop a solution in the new connection to ensure adequate anchorage length for the tensile reinforcement. This is particularly important as a common problem in detailing reinforcement is that the tension bar is often not properly anchored (Hordijk & van der Vossen, 2024). Without ensuring the necessary anchorage length, there is a significant risk of structural failure due to slippage of the reinforcement bars.

5.2.2. Edge Reinforcement in floors and walls

Edge reinforcement is added to absorb stress concentrations that occur at the edges of the elements. Both horizontal and vertical bars are utilized to reinforce these areas. When the element is cut into reusable sections, this edge reinforcement will be lost, which reduces the element's ability to handle stress concentrations at the edges. Therefore, it is important to account for the loss of this reinforcement during the redesign process.

For floors, the loss of edge reinforcement alters their load-bearing behavior. Floors in cast-in-situ designs often span in two directions. When the edge reinforcement is lost, the floor can no longer transfer forces in the transverse direction. This means that the floor can be applied in a new building as a one-way spanning element, provided it is fully supported in the spanning direction. This makes the reuse of the floor feasible, as long as the design is adapted accordingly. According to Eurocode 2 (NEN-EN 1992-1-1+C2, Section 9.3.1.1; Nederlands Normalisatie-instituut (NEN), 2011), in one-way spanning slabs, the transverse reinforcement should be at least 20% of the main reinforcement. Near the supports, transverse reinforcement for the top main bars is not required if no bending moment occurs in the transverse direction. For reused floors, this rule is advantageous as it reduces the need for additional reinforcement in areas without transverse bending moments, aligning well with the structural characteristics of one-way spanning elements.

For walls, edge reinforcement is critical for maintaining in-plane and out-of-plane stability. Walls in cast-in-situ designs rely on edge reinforcement to handle bending moments and shear forces effectively. When this reinforcement is lost due to cutting, the wall's capacity to resist bending and shear stresses is significantly reduced, making it more likely to become unstable and develop cracks at the edges. However, with appropriate strengthening or redesign of the edge reinforcement, the reuse of the wall remains feasible.

5.2.3. Cross-sectional reinforcement ratio

When reusing concrete elements, it is important to consider the reinforcement ratio limits specified in Eurocode 2 (NEN-EN 1992-1-1+C2, Section 9.2.1.1; Nederlands Normalisatie-instituut (NEN), 2011). According to these guidelines, the reinforcement ratio in a cross-section must not exceed 4%, while in overlap splices it must not exceed 8% of the concrete cross-sectional area. These limits ensure proper force transfer and prevent excessive reinforcement, which could lead to stress concentrations and adversely affect the structural performance of the reused element. Additionally, maintaining an appropriate reinforcement ratio is crucial for ensuring sufficient ductility. A reinforcement ratio that is too high can reduce the element's ability to undergo controlled deformation, which is essential for redistributing forces and preventing brittle failure in the new structural system. When designing connections or making modifications, it is essential to verify that the existing reinforcement and any additional reinforcement comply with these limits to maintain the structural performance of the reused elements.

5.2.4. Standard design lengths

When elements are cut for reuse, they will result in shorter lengths that may no longer conform to standard design lengths, heights and spans used in construction. This is relevant for floors and beams, where maintaining standard spans is crucial for efficient integration into new structures. For columns and walls, height is crucial to comply with the 'Besluit bouwwerken leefomgeving' discussed in section 4.2.3. The challenges related to length and span will affect architects, engineers and contractors, as the reduced dimensions could limit the elements' suitability for their intended applications in the new design. It is advisable to incorporate additional length into the new connection design to ensure proper functionality and compliance with structural requirements.

5.3. Impact of the cutting location

The location of the cut significantly influences the remaining capacity of a reused element and its compatibility with new support conditions. In horizontal elements such as beams and floors, cutting near moment-zero points or near the supports has distinct advantages and disadvantages. Cutting at the moment-zero point in beams reduces internal stresses during the cutting process, simplifying the sawing operation. This approach preserves the positive bending capacity of the element, as the dominant bottom reinforcement remains intact, requiring the element to be supported as a hinged connection in the new design. However, it can result in a significant loss of shear reinforcement, which is primarily concentrated near the supports. Additionally, this method shortens the remaining beam length, which may limit its reuse potential.

Conversely, cutting near the supports allows for the retention of shear reinforcement, essential for the load-bearing capacity of the element. It also preserves more of the original length, making reuse in new structures easier. Additionally, the dominant top reinforcement remains intact at the end of the element, requiring it to be reinstalled with a fixed support condition in the new design. However, cutting in high-stress areas requires additional support measures to prevent cracking or failure during the cutting process.

In the end, the optimal cutting location depends on the intended support conditions of the reused element. Cutting near the supports maintains top reinforcement, making fixed connections required, while cutting at moment-zero points preserves bottom reinforcement, requiring hinged support conditions.

5.4. Conclusion

The process of cutting cast-in-situ elements for reuse introduces several structural and practical challenges that must be carefully addressed to ensure their stability and functionality in a new prefabricated system. This chapter has outlined important considerations, including the loss of anchorage length, edge reinforcement and the impact on reinforcement ratios and element dimensions. The location of the cut plays a crucial role in determining the structural integrity of the reused element. Cutting at the moment-zero point in beams reduces internal stresses during the cutting process and can preserve the fixed connection behavior between beam and column (the T-shaped element). It requires hinged connections in the new structural system. However, this approach can lead to a significant loss of shear reinforcement and results in shorter element lengths, potentially limiting reuse applications. In contrast, cutting near the supports helps retain shear reinforcement and preserves more of the original element length. It required fixed connections in the new structural system. However, it introduces challenges related to high stress concentrations during the cutting process, requiring temporary supports to prevent damage. Addressing these challenges is essential to facilitate the reuse of cast-in-situ elements in prefabricated structures. This will require appropriate connection strategies that compensate for lost reinforcement and altered load-bearing behavior.

6
From cast-in-Situ to prefabricated

When reusing cast-in-situ elements, the new structure effectively becomes a prefabricated system. Transitioning cast-in-situ elements to a prefabricated system introduces various structural challenges. Unlike cast-in-situ systems, where elements are specifically designed for their original placement and fully monolithic behavior, prefabricated systems rely on jointed connections and often involve hinged supports. This change fundamentally changes the way forces and moments are distributed through the structure, requiring a re-evaluation of the design and performance of the elements.

This chapter examines the considerations when adapting cast-in-situ elements for reuse in prefabricated systems. The challenges in this transition include the impact of transport and handling stresses, as cast-in-situ elements were never intended to withstand the dynamic forces encountered during lifting, transport and erection. Additionally, changing support conditions from fixed to hinged alters bending moment distribution, affects buckling behavior and influences overall structural stability. Finally, the introduction of jointed connections in place of monolithic cast-in-situ connections results in stress concentrations and changes in load paths, necessitating specific reinforcement detailing to ensure structural integrity. This chapter examines these challenges in detail, identifying critical structural considerations to facilitate the safe and effective reuse of cast-in-situ elements in prefabricated systems.

6.1. Transport and handling stresses

Since cast-in-situ construction involves creating elements directly at the location where they will perform their structural function, specific considerations for transport and handling are unnecessary. However, with reusing cast-in-situ elements in a new prefabricated system, specific considerations for transport and handling become essential. It is important to evaluate the impact of the processes involved in relocating these reused elements to their new positions. This includes addressing the effects of transport, erection and assembly forces on elements that were not originally designed to endure such dynamic and handling loads (Concrete NZ, 2015).

6.1.1. Element transportation

During transport, the elements will be subjected to dynamic forces. These loads introduce significant stresses to elements originally designed for a cast-in-situ context, where dynamic transportation forces are not considered. Additionally, differential road cambers can introduce torsional loads in long concrete elements, further stressing sections that were not designed for such dynamic effects. As a result, cast-in-situ elements may lack the necessary reinforcement or design features to withstand the impact and vibration and tilting stresses experienced during movement. To ensure the structural integrity of the elements during transport, it is crucial to implement protective measures such as temporary strengthening or the use of supportive transport frames. Proper handling procedures and pre-transport inspections should also be conducted to identify and address any vulnerabilities in the elements.

6.1.2. Element lifting

Reusing cast-in-situ elements requires careful planning for lifting, as these elements typically lack lifting inserts or clutch points found in prefabricated components. Without these built-in lifting aids, alternative methods such as slings or clamps must be used, which can create localized stress. These concentrated lifting forces at specific points can introduce extra stress on parts of the element that were not originally designed to bear these loads, increasing the potential for minor cracking or other local damage. To mitigate these risks, it is essential to carefully distribute lifting forces. To mitigate these risks, lifting forces should be carefully distributed using techniques such as soft slings to reduce pressure points. Additionally, selecting appropriate lifting points and reinforcing critical areas with temporary supports can further minimize stress concentrations and prevent unintended damage.

6.1.3. Element erection

During erection, cast-in-situ elements encounter forces that weren't part of the original design. Erection forces include loads that arise when positioning, tilting, or adjusting the element. Since these elements were designed for static placement, they will not be optimized to handle these loads experienced during erection. In a prefabricated design, temporary support forces play a role before the element is fully secured in its final position. These will also introduce concentrated point loads to which the element may not be resistant. To address these challenges, it is critical to carefully plan erection procedures, use adequate temporary supports to distribute loads evenly and assess the element's capacity to withstand these additional forces before construction begins.

The relocation of cast-in-situ elements into a prefabricated system introduces a range of dynamic, handling and concentrated point loads that these elements were not originally designed to withstand. Unlike prefabricated components, cast-in-situ elements lack specific design features, needed to manage the stresses from transport, lifting and erection processes. For successful reuse, it is important to consider design adaptations to prevent potential cracking, deformation, or localized damage in the prefabricated system.

6.2. Change in support conditions

When adapting an element from a cast-in-situ structure into a prefabricated system, the type of support conditions may change from fixed to hinged. However, the element can also be re-implemented with fixed supports, depending on the design of the new connections. This section identifies the challenges that arise when transitioning from fixed to hinged support conditions, providing insights into whether hinged connections are viable or if fixed supports might be a better choice.

The shift from fixed to hinges affects how forces and deformations are distributed within the element, impacting bending moments, buckling length and overall rotation and deformation behavior. This discussion focuses specifically on the changes occurring at the element level, rather than the global structural system. Understanding these effects is critical to ensuring the element is effectively adapted to its new conditions and can perform as intended within the prefabricated design.

6.2.1. Bending moment

A bending moment occurs in a structural element when external forces cause it to bend around an axis. The distribution of bending moments depends on the type of supports such as fixed or hinged. Changing the support reactions from fixed to hinged affects the location and magnitude of the bending moments along the element. As shown in figure 6.1, the central bending moment will be higher with hinged connections (A1) than with the original fixed configuration (A3). Consequently, elements reused in the new prefabricated design with hinged connections will have to withstand lower forces than in the original fixed configuration, making it crucial to assess and adjust these elements for their new support conditions.

In addition to the increased central bending moment in hinged configurations, the bending moments at the supports also differ significantly. In a fixed system, a negative moment develops at the supports, requiring additional top reinforcement in slabs and beams to resist these forces. This design typically results in less bottom reinforcement near the supports, as the tensile stresses in that region are minimal for a fixed system. When a beam or slab is reused at its full original length from the old system, the existing top and bottom reinforcement is configured to handle negative moments at the supports.

This means that the reinforcement layout in the elements may not be optimal for a new hinged connection, where positive moments dominate directly after the supports. This highlights the potential advantages of recreating a fixed connection in the new system. By re-implementing fixed supports, the original reinforcement layout designed to handle negative moments can be fully utilized, reducing the need for additional modifications or strengthening. Practical testing has demonstrated that the negative moment capacity of precast concrete beams made continuous with high-tensile reinforcement bars placed in the topping is at least 85% of the moment resistance of a fully monolithic connection. This reduction in capacity suggests that the connection exhibits less rotational stiffness, potentially altering the internal force distribution and reducing the overall load-carrying capacity when the beam is reused in a fixed system (du Béton, 2008).

Conversely, adapting the element for a hinged connection might require significant adjustments, such as adding bottom reinforcement to withstand the increased central moments. However, when an element is not reused at its full original length, for instance if only the segment between the moment zero points is retained, the reinforcement is typically well-suited for hinged connections. This is because such segments primarily contain reinforcement designed to handle positive moments, which dominate in hinged configurations.

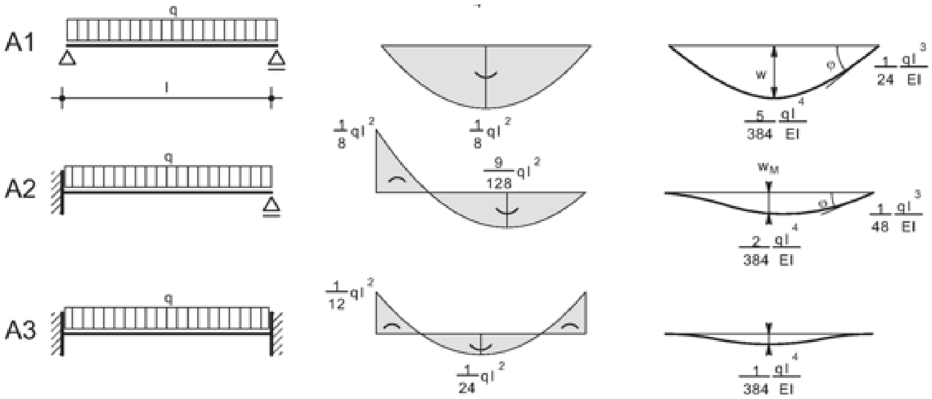


Figure 6.1: Moment and deflection behavior for different support conditions

6.2.2. Buckling lengths

When transitioning from fixed to hinged connections, the effective buckling length of vertical elements increases significantly, as shown in figure 6.2.

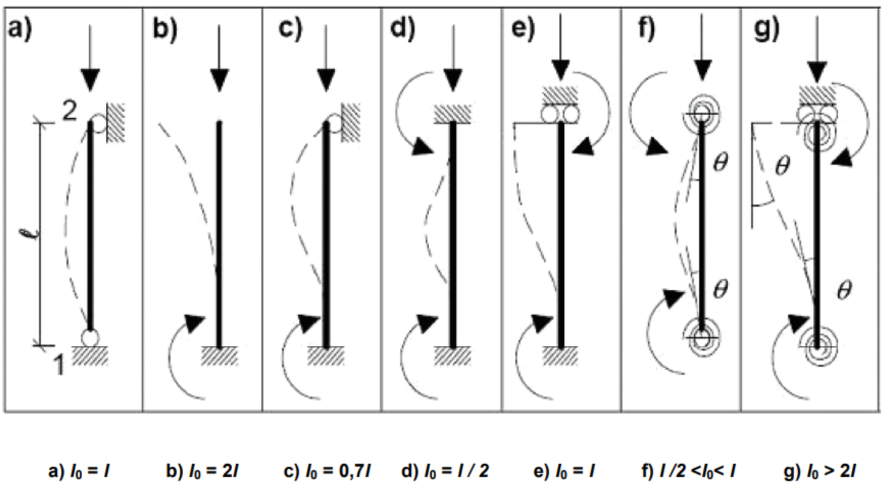


Figure 6.2: Buckling lengths (European Committee for Standardization, 2017)

This change occurs because hinged connections allow rotation at the ends of the columns, reducing the lateral restraint that would otherwise help resist bending. A longer buckling length has a direct impact on element dimensioning. As the buckling length increases, columns and walls become more sensitive to buckling under compressive loads, reducing the critical buckling load they can withstand. Therefore, to maintain structural integrity within a hinged connection design, the applied loads on these columns must be reduced, as they may not be able to handle the same load-bearing capacity as in a fixed connection configuration.

6.2.3. Rotation and deformation

In a structure with fixed connections, the element contributes to the overall stiffness, which limits displacements and rotations. In a structure with hinged connections, however, that same element will experience greater deformations, as it is no longer rigidly fixed and can rotate at the connections. The element should therefore be evaluated for its capacity to accommodate larger deformations or rotations without incurring damage.

6.2.4. Redistribution of forces

With hinged connections, the way forces are transmitted through the structure changes, causing the load paths to adjust. In a fixed connection, the joints resist both rotation and translation, allowing forces like bending moments to be distributed through the connected elements. However, in a hinged connection, rotation is allowed, so the connection cannot transfer bending moments; it only transmits axial (compression or tension) forces. This means that forces once managed by the fixed joint are now redistributed to other parts of the structure, often increasing axial loads on adjacent elements (du Béton, 2008). Since the elements cannot be resized, it may be necessary to reduce the applied forces to maintain structural stability within the new configuration.

When an element is reused with the same support conditions as in its original design, the internal force distribution remains similar to the original configuration. This reduces potential issues, as the existing reinforcement is already suited to resist the bending moments and shear forces for these conditions. As a result, reusing an element with its original support conditions reduces the need for extensive design modifications or additional reinforcement, thereby simplifying the adaptation process while maintaining structural reliability. However, even when reused in a fixed configuration, the load-carrying capacity may be reduced, as practical testing has shown that the negative moment capacity of precast beams is approximately 85% of that in a fully monolithic connection. When transitioning to a hinged configuration, the redistribution of internal forces eliminates the negative moment at the supports, increasing the positive moments in the span. This shift requires careful evaluation of the existing reinforcement to ensure it remains adequate for the new force distribution.

6.3. From monolithic to jointed connections

When reusing cast-in-situ elements in a prefabricated design, there are specific challenges that arise due to the fact that these elements were not originally designed for jointed, prefabricated conditions. This section discusses challenges associated with transitioning to jointed connections, including stress concentrations, differential deformations and splitting failure and their implications for the structural performance of reused elements.

6.3.1. Concentrated stresses

In cast-in-situ constructions, the connections are monolithic, meaning the concrete is poured and hardens as a continuous unit. This results in a uniform stress distribution across the connections, as there are no joints or interruptions. Each force acting on the cast-in-situ connection is smoothly transferred through the entire cross section of the structure. A prefabricated concrete design involve joints between connected elements, leading to stress concentrations at support points. Unlike cast-in-situ connections, which rely on distributed reinforcement throughout the monolithic section, prefabricated connections depend on concentrated tie bars to transfer loads. This makes prefabricated joints more vulnerable to stress concentrations and requires specific reinforcement detailing to prevent localized failures. When cast-in-situ elements are reused in a prefabricated system, it must be considered that the element was not originally designed to handle these stress concentrations.

To transfer these concentrated stresses through the connection, the end of a prefabricated element is designed using a strut-and-tie model. This end region is also referred to as the D-region (Disturbed region), where stress concentrations are managed by specific reinforcement detailing. In this method, struts (compression paths) and ties (tension paths) represent the internal force flow within a concrete element, effectively transferring concentrated stresses through the connection to the supporting structure (du Béton, 2008). In figure 6.3, the stress analysis of a slab-wall connection is shown. In b), it can be seen that the stresses are concentrated through the connection. In c), the strut-and-tie method shows the location of the tie bars needed to ensure equilibrium in the connection.

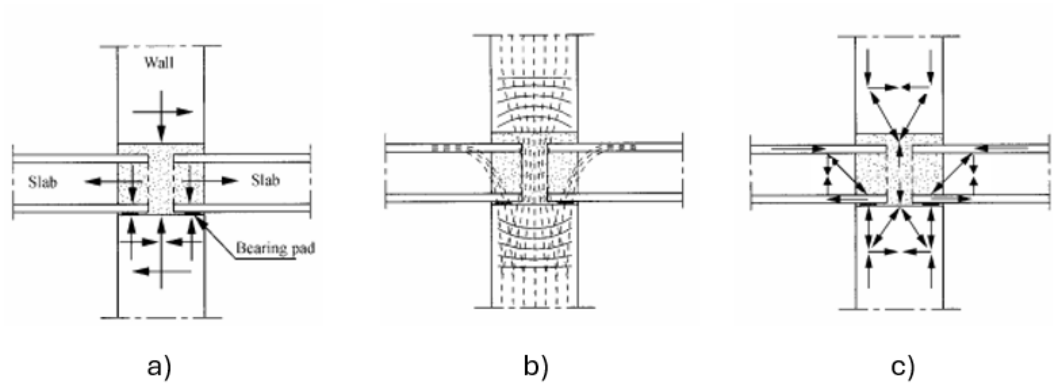


Figure 6.3: Slab-wall connection a) forces, b) simplified stress analysis, c) strut-tie method (du Béton, 2008)

In figure 6.4, a column analysis is shown. This figure has been compiled based on the study by Sahoo and Varghese (Sahoo & Varghese, 2018), highlighting the differences between cast-in-situ and prefabricated column designs. a) indicates how the column is designed when it is part of a cast-in-situ structure, where stresses are evenly distributed throughout the column. In contrast, b) shows how the column is designed using the strut-and-tie method when it is part of a prefabricated structure, resulting in concentrated stresses that need to be managed specifically at the connections. In this figure, the B-region (Bernoulli region), where stresses are more uniform and D-region, discussed earlier, are also visible.

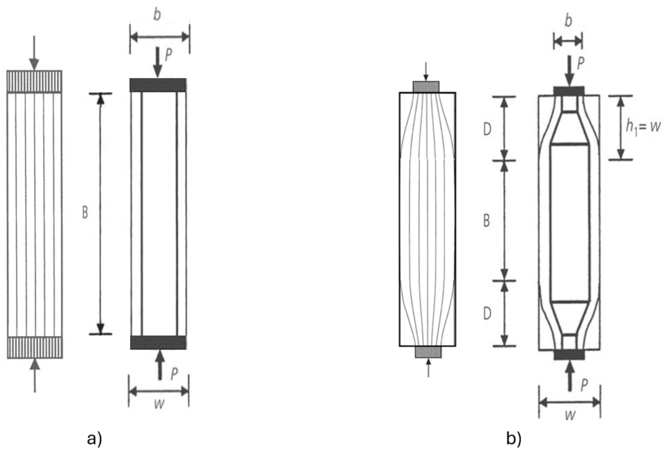


Figure 6.4: Column analysis a) evenly distributed stress, b) concentrated stress

For partially continuous areas where $b \leq \frac{H}{2}$, which applies to a column, the tensile force T in this D-region can be determined according to Eurocode 2 (NEN-EN 1992-1-1+C2, Section 6.5.3; Nederlands Normalisatie-instituut (NEN), 2011) using equation (6.1). The reinforcement required to resist T may be distributed over a length $h = b$.

$$T = \frac{1}{4}\frac{w-b}{b}F \tag{6.1}$$

where:

- T: tensile force,
- w: width of the column,
- b: width of the load-bearing area (width of the support),
- F: applied concentrated force from the beam.

Based on this knowledge, the reinforcement in the column head of a prefabricated concrete column is designed as shown in figure 6.5.

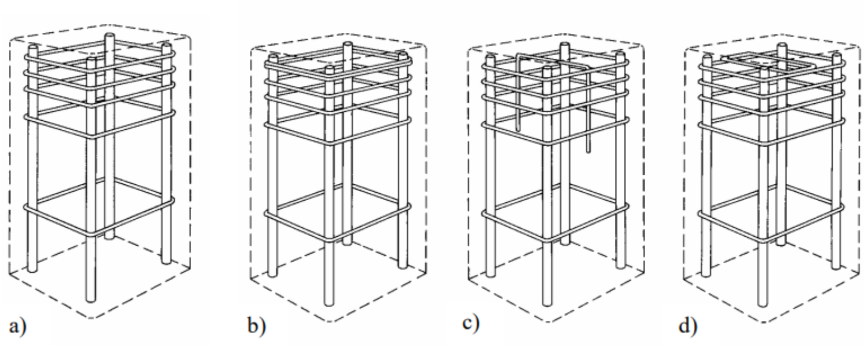


Figure 6.5: Column head splitting reinforcement, a) normal, b) additional tie, c) U-shaped tie, d) special tie (du Béton, 2008)

Additionally, figure 6.6 illustrates how a wall element behaves under concentrated loading conditions. In the case of a wall, the entire section can be considered as a D-region, meaning that stress concentrations are managed throughout the entire height.

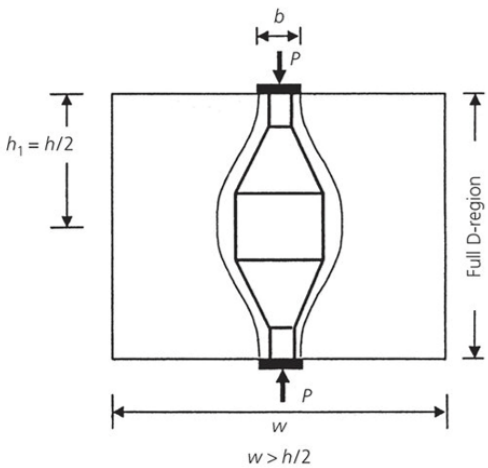


Figure 6.6: Wall analysis concentrated stress (Sahoo & Varghese, 2018)

In the case of fully discontinuous regions, where b exceeds half of the element height $\left(b > \frac{H}{2}\right)$, which applies to walls, the tensile force T can be calculated with equation (6.2) according to Eurocode 2 (NEN-EN 1992-1-1+C2, Section 6.5.3; Nederlands Normalisatie-instituut (NEN), 2011):

$$T = \frac{1}{4}\left(1 - 0.7\frac{b}{h}\right)F \tag{6.2}$$

where:

- T: tensile force,
- w: width of the column,
- b: width of the load-bearing area (width of the support),
- F: applied concentrated force from the beam.

To ensure that concentrated stresses are effectively managed in practical applications, bearing pads must be designed to accommodate different force transfer mechanisms. Figure 6.7 presents key requirements for proper bearing pad placement, preventing local failure and ensuring the intended load distribution.

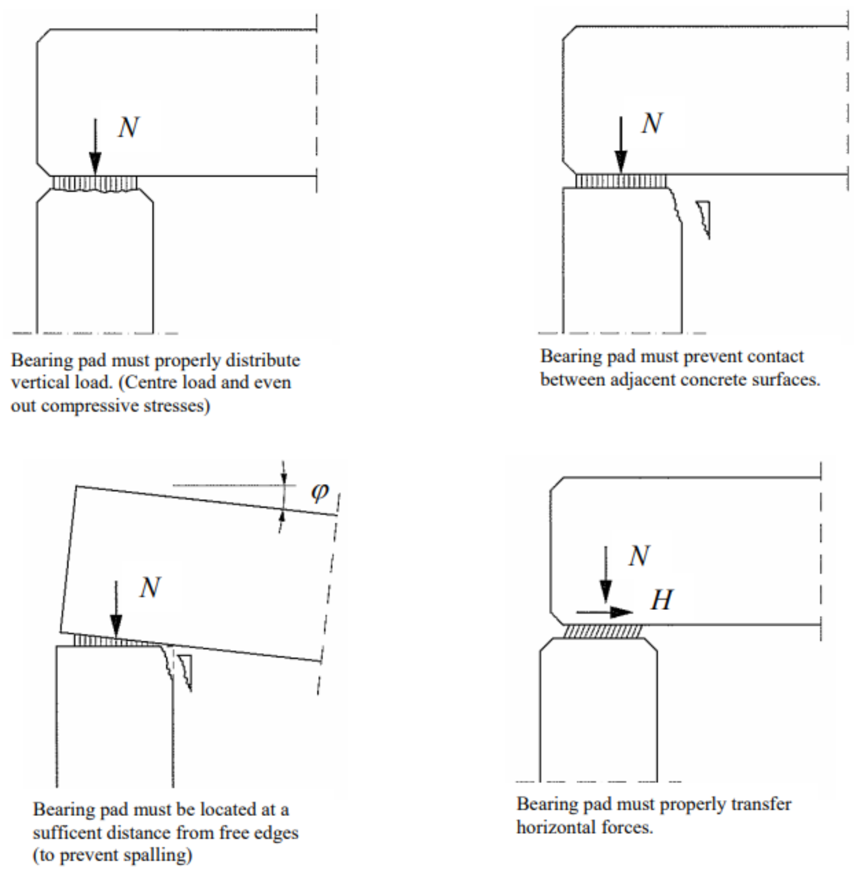


Figure 6.7: Reasons for using bearing pads (du Béton, 2008)

The choice of bearing material can determine the degree of concentrated stresses in a jointed connection. Higher stiffness materials, such as steel, provide a more uniform load distribution and minimize local stress peaks, whereas softer materials like mortar can lead to stress concentrations at contact interfaces. Softer materials tend to deform more, which can lead to concentrated stresses at specific points, especially at the surface of the adjacent concrete. The choice of bearing material can also result in tensile stresses within the concrete. figure 6.8 highlights this difference. Materials with a higher Poisson's ratio/E-modulus ($\frac{\nu}{E}$), such as steel, expand less under the same compressive force compared to concrete, creating compressive stresses at the surface. In contrast, mortar has a lower Poisson's ratio/E-modulus ($\frac{\nu}{E}$), causing it to expand more quickly than concrete under the same compressive force, which leads to tensile stresses at the surface. When tensile stresses arise in the concrete, special reinforcement is required, which is typically absent in reused elements. For this reason, using steel as a bearing material offers advantages when designing connections.

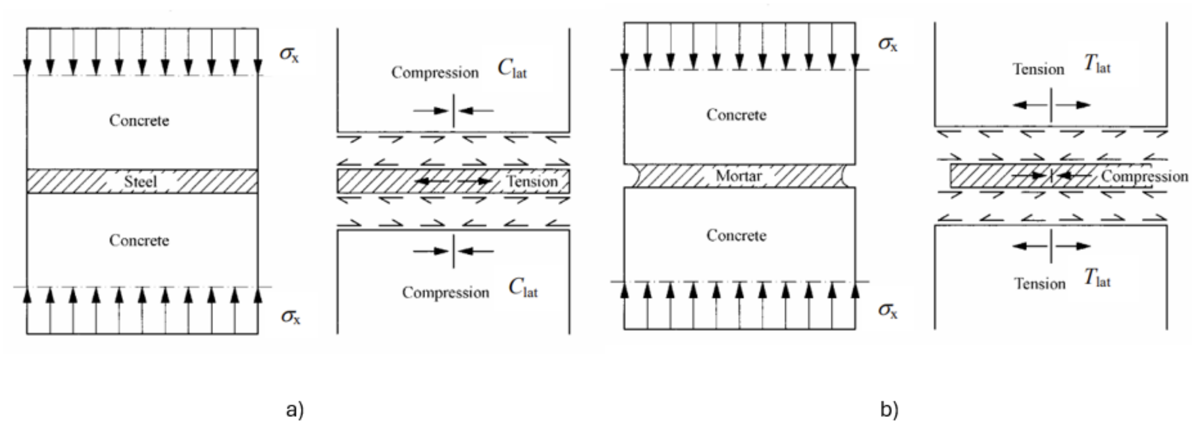


Figure 6.8: Stresses caused by the choice of bearing material, a) higher $\frac{\nu}{E}$, b) lower $\frac{\nu}{E}$ (du Béton, 2008)

The challenge of concentrated stresses is especially relevant for vertical elements, as they often carry the highest points loads. Horizontal elements, such as beams and floors, generally experience more uniformly distributed loads, which reduces the likelihood of stress concentrations at the connection points. However, both vertical and horizontal prefabricated elements are affected by static discontinuities at their connection zones, which can lead to stress concentrations that need to be carefully managed. Because of static discontinuities in prefabricated connections, the connections zones should be considered to be D-regions (du Béton, 2008). The reused elements in this research were originally designed for cast-in-situ conditions, where no D-regions are required. Without specific reinforcement in the D-region to handle concentrated stresses, there is a risk of cracking or deformation, as the concrete and reinforcement are not originally intended to manage the stress peaks effectively.

6.3.2. Requirement for wall-floor connections

When reusing cast-in-situ walls in a prefabricated system, it is essential to ensure that the new connections can safely transfer loads without causing local failures. Eurocode 2 (NEN-EN 1992-1-1+C2, Section 10.9.2; Nederlands Normalisatie-instituut (NEN), 2011) specifies a requirement regarding the maximum load per unit length in a wall-floor connection when no special reinforcement is provided at the base, which is the case with reused elements. Therefore, when designing a wall-floor reconnection, the maximum allowable load given by $\leq 0.5 \cdot h \cdot f_{cd}$. An example of such special reinforcement required when the applied load exceeds this limit is shown in figure 6.9.

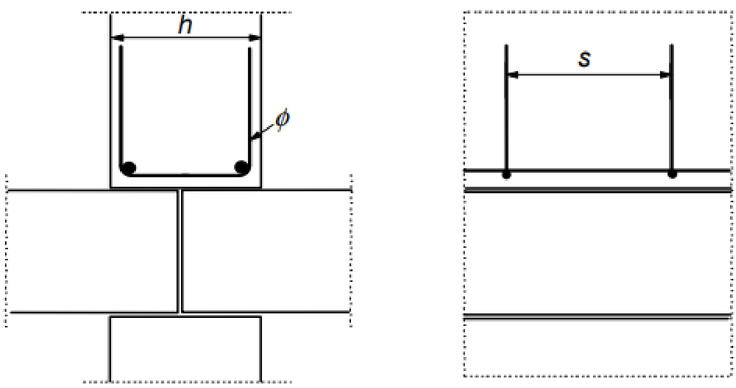


Figure 6.9: Example of required reinforcement in a wall above two floor slabs when the applied load exceeds the limit of $0.5 \cdot h \cdot f_{cd}$ (NEN-EN 1992-1-1+C2, Section 10.9.2; Nederlands Normalisatie-instituut (NEN), 2011)

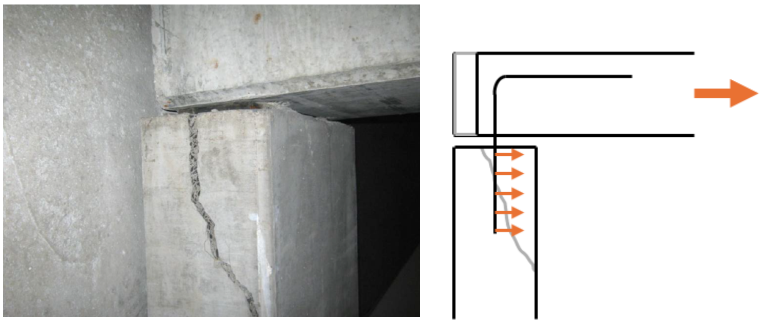
6.3.3. Stability requirements for connections

Fixed connections, by design, are capable of transferring both vertical and horizontal forces without the need for additional measures. They are often employed in stability systems due to their ability to resist moments and provide robust horizontal force transfer.

Although hinged connections are typically not part of the primary stability system, there are cases where they are required to transfer limited horizontal forces, such as wind loads, seismic loads, or asymmetrical forces. Horizontal forces are transferred through diaphragm action from the floor to stability walls or columns. Additionally, beams must also transfer horizontal forces to the column if they are part of the secondary stability system. Hinged connections must provide sufficient stiffness to transfer these horizontal forces. A lack of horizontal force transfer can lead to cracking, instability, or even failure of the stability system. Horizontal forces can be transferred through friction between contact surfaces. This is effective when there is sufficient vertical pressure and the contact surface is adequately rough. If friction is insufficient, mechanical solutions such as bolts, steel brackets, or tie rods can be used in the connection.

6.3.4. Casting joints

The different elements connected in a prefabricated system often experience uneven deformations. This issue does not occur in monolithic cast-in-situ designs, where the continuous nature of the structure ensures uniform deformation. Uneven deformations are caused by shrinkage, creep and temperature differences and will lead to crack formation (Hordijk & van der Vossen, 2024). A common form of damage in prefab concrete structures due to restricted uneven deformations is shown in figure 6.10.



(a) Spalling failure in practice (Hordijk & van der Vossen, 2024) **(b)** Spalling failure in theory

Figure 6.10: Spalling failure due to restricted uneven deformations

This issue is typically present in prefabricated constructions, but it is expected to become more problematic when reused cast-in-situ elements are incorporated into a prefabricated system, especially when these reused elements are combined with newly manufactured prefabricated components. Cast-in-situ elements have likely already undergone much of their shrinkage and creep deformation over time (NEN-EN 1992-1-1+C2, Section 3.1.4; Nederlands Normalisatie-instituut (NEN), 2011), whereas the newly prefabricated components will still experience shrinkage and creep. This difference in deformation increases the probability of this failure mechanism to occur. Consequently, this must be carefully considered when designing the new connections, as standard design choices may not be sufficient to accommodate these increased risks.

6.3.5. Splitting failure

Splitting failure occurs when forces, generated by tensile-loaded fasteners, create stress that causes the concrete to crack and split apart. This failure mode is typically the result of tensile stresses that act perpendicular to the primary load direction, leading to cracks that compromise the integrity of the concrete member (Hüer & Eligehausen, 2007). This occurs particularly when the steel reinforcement is significantly stronger than the surrounding concrete. Stronger reinforcement transfers higher forces to the concrete. Since prefabricated concrete is typically stronger than cast-in-situ concrete, special attention must be given to the strength of the connection reinforcement when reusing cast-in-situ elements. Using the same high-strength reinforcement commonly applied in prefabricated elements can increase the risk of splitting failure due to shear forces. Figure 6.11 illustrates a schematic representation of a splitting failure.

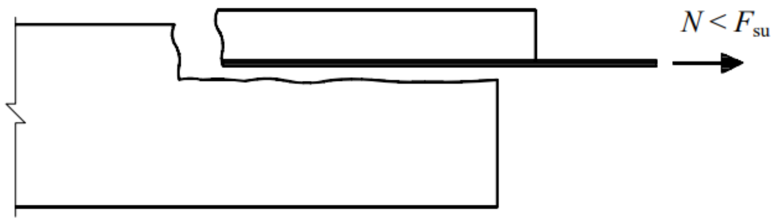


Figure 6.11: Schematic representation of splitting failure (du Béton, 2008)

6.3.6. Concrete cone failure

Concrete cone failure occurs when tensile forces from fasteners or anchors cause a cone-shaped portion of the concrete to separate and fail. This failure mode is influenced by the tensile strength of the concrete, which determines its ability to resist the stresses transferred by the fastener (du Béton, 2008). When reusing cast-in-situ elements, the concrete is often less strong than in new prefabricated components, due to factors such as aging, lower original concrete grades, or micro cracking caused by cutting and handling. The reduced concrete strength increases the likelihood of concrete cone failure. To mitigate this risk, it is essential to evaluate the strength of the reused concrete and adjust the connection design accordingly. This can include reducing anchor forces, increasing the embedment depth of fasteners, or using supplementary measures such as reinforcement around the anchorage zone to enhance load distribution. Figure 6.12 illustrates a schematic representation of a concrete cone failure.

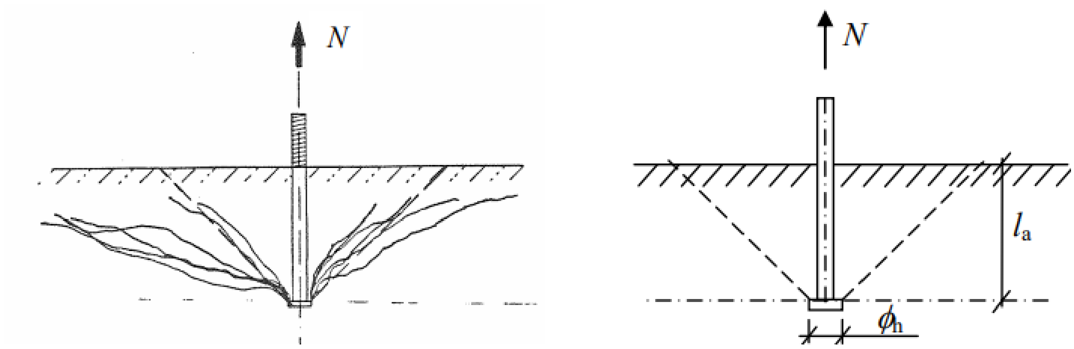


Figure 6.12: Schematic representation of concrete cone failure (du Béton, 2008)

6.4. Conclusion

Reusing cast-in-situ concrete elements in a prefabricated system introduces significant structural challenges that must be carefully addressed to ensure safe and effective integration. This chapter has highlighted key considerations that influence the feasibility of reusing these elements, including transport and handling stresses, changes in support conditions and transitioning from monolithic to jointed connections. In the following chapters, the focus is on developing and evaluating reconnection solutions that address these structural challenges, ensuring that reused elements can perform effectively in a prefabricated system.

From this chapter, the following challenges are carried forward into the next stages of the research:

- Jointed connections create stress concentrations, requiring an assessment of existing reinforcement detailing. Unlike monolithic cast-in-situ systems, prefabricated designs rely on strut-and-tie models to transfer concentrated stresses at connections. Without dedicated reinforcement in the D-region, reused elements may experience cracking or structural deficiencies.
- Stability and force redistribution must be reconsidered, particularly when transitioning from fixed to hinged connections. Hinged systems limit moment transfer but require sufficient horizontal force transfer to maintain overall stability. In fixed connections, reinforcement detailing must be adapted to compensate for the potential reduction in negative moment capacity due to the loss of monolithic continuity.
- Differential deformations and potential failure mechanisms, such as splitting and concrete cone failure, introduce additional constraints on connection design. Differences in shrinkage and creep between reused and newly cast elements can lead to cracking, while high-strength reinforcement in prefabricated elements may induce localized stress concentrations in the reused elements.
- The choice of cutting location, either near the original fixed supports or at the moment-zero points, significantly influences the possible support conditions for beams and floors in their new application, as discussed in chapter 5. Although the cutting location imposes limitations on the type of support condition in the new design, it also ensures that the original force distribution is maintained, which helps reduce the risk of unexpected failure.
- The maximum allowable load in the wall-floor connection is given by $\leq 0.5 \cdot h \cdot f_{cd}$.

These findings, combined with insights from chapter 5, address the structural and practical challenges of reusing cast-in-situ elements in prefabricated systems, answering the second sub-question.

7

Anchorage and reconnection solutions

In the previous chapters, the structural challenges associated with reusing cast-in-situ concrete elements have been identified, particularly the loss of anchorage and the need to connect reinforcement between adjoining elements. This chapter explores practical solutions to these challenges by examining various anchorage techniques and reinforcement connection methods applicable to reconnection designs. By evaluating different strategies, this chapter provides a foundation for selecting appropriate solutions that facilitate the integration of reused elements into new structural systems.

7.1. Hydrodemolition

Hydrodemolition is an effective technique for exposing reinforcement in concrete elements, which is essential for utilizing the existing reinforcement in the reconnection of the reused elements. This method employs high-pressure water jets to remove concrete without damaging the embedded reinforcement bars or causing micro-cracks in the remaining concrete (Michael, 2013; The Constructor, 2020). To ensure proper bonding between the reused elements and newly cast concrete, the surface must be sufficiently rough. According to Eurocode 2 (NEN-EN 1992-1-1+C2, Section 6.2.5; Nederlands Normalisatie-instituut (NEN), 2011), a roughness of at least 3 mm with a spacing of approximately 40 mm, achieved by raking, exposing of aggregate or other methods is required. Experimental studies have shown that hydrodemolition produces a rough surface comparable to mechanically roughened surfaces, such as scarification (Courard et al., 2003; Michael, 2013). Since scarification is classified as a ”rough” surface in Eurocode 2 (NEN-EN 1992-1-1+C2, Section 6.2.5; Nederlands Normalisatie-instituut (NEN), 2011), hydrodemolition can also be considered to meet this criterion when the resulting roughness profile falls within the required limits. Since hydrodemolition does not induce micro cracking, it further enhances bond strength, as microcracks can disrupt adhesion between the existing and newly cast concrete (Courard et al., 2003). Additionally, research investigated the bond strength of hydrodemolition treated concrete and found an average bond strength of $1.04MPa$ (Wenzlick, 2002). This further supports the effectiveness of hydrodemolition in creating a structurally reliable interface between old and new concrete.

7.2. Mechanical couplers

Mechanical reinforcement couplers ensure direct force transfer between reinforcement bars, independent of the surrounding concrete (nVent, 2013). Using these mechanical coupling systems, the reinforcement of one element can be directly connected to another. A minimum reinforcement length of 80 mm must be exposed to allow for proper connection using these couplers (nVent, 2013; Regbar, 2019). This requires the removal of 80 mm of surrounding concrete from the reused beam using hydrodemolition.

Mechanical coupling of reinforcement bars requires 125% to 150% more capacity compared to overlap lengths (nVent, 2013; Regbar, 2019), which ensures a high level of reliability and safety in force transfer. This makes mechanical couplers particularly suitable for applications where traditional overlap lengths are impractical or insufficient due to the limited available lengths in reused elements or constraints imposed by the original design. The selection of a specific type of coupler depends on the forces that need to be transferred and the feasibility of installation on-site. This section discusses mechanical coupling systems suitable for connecting two existing reinforcement bars in reused elements, where both bars are already embedded in concrete. The inability to rotate or significantly adjust the bars limits the available solutions. The couplers discussed in this section have been chosen for their suitability in accommodating these limitations, making them viable options for the reconnection of reinforcement in reused concrete elements.

7.2.1. Lock coupler

The lock coupler is a mechanical connection system that enables the joining of two reinforcement bars without requiring modifications to the bars themselves. The system works by allowing a sleeve to be fully slid over one of the bars first. Once the bars are positioned against each other, the sleeve can be moved back over the connection point. The connection is then secured by tightening bolts that are drilled into the reinforcement bars, ensuring mechanical interlock. This makes it a suitable solution for re-connecting reinforcement in reused elements. This system meets or exceeds all requirements in international construction standards, including NEN 6720 and 6723, DIN EN-1992-1-1, BS EN1992-1-1 and CalTrans IBC®, IAPMO®-UES and ACI® 318 Type 2. An advantage of this system is that no threading or welding of the existing reinforcement is required, which simplifies installation on-site and minimizes the risk of damaging the original bars. Despite the fact that drilled bolts introduce stress concentrations in the reinforcement, the lock coupling still provides structural performance comparable to that of a continuous reinforcement bar (nVent, 2013). Figure 7.1 illustrates the lock coupling system. The main image shows how the sleeve is positioned over the joint of the bars, while the inset provides a closer view of the internal bolting mechanism.

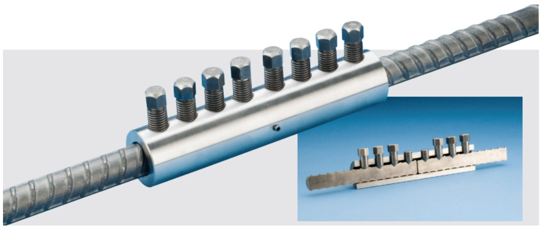


Figure 7.1: Lock coupler (nVent, 2013)

7.2.2. Quick wedge coupler

The quick wedge coupler is a mechanical reinforcement connection system designed for efficient and straightforward installation. This system does not require precise alignment of the reinforcement bars being connected, simplifying the installation process. The coupler accommodates reinforcement bars with diameters ranging from 12 to 20 mm and provides a tensile strength comparable to that of traditional overlap connections (nVent, 2013). The mechanism relies on a wedge-locking system that securely grips both bars without requiring threading or welding. Figure 7.2 illustrates the quick wedge coupler system and shows how the coupler clamps onto the two reinforcement bars.

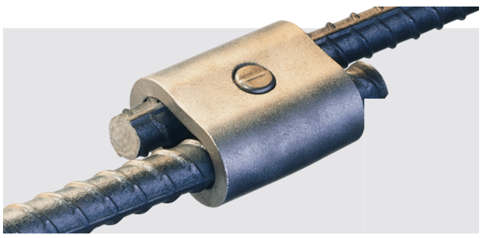


Figure 7.2: Quick wedge coupler (nVent, 2013)

7.2.3. Interlock coupler

The interlock coupler is a mechanical connection system that allows for the secure joining of existing reinforcement bars through a combination of threaded and grouted connections. One side of the coupler features a threaded connection, while the other side consists of a sleeve that is later filled with grout. A requirement for this system is that threads must be cut into one of the existing reinforcement bars before installation. Additionally, precise alignment of the bars is essential to ensure a proper connection. Despite these constraints, it provide a reliable solution for reconnecting reinforcement in reused concrete elements. Interlock couplers are available for reinforcement sizes ranging from 20 mm to 57 mm, making them suitable for various reinforcement diameters. Furthermore, they offer structural performance equivalent to a continuous reinforcement bar, ensuring that the connection maintains the necessary load-bearing capacity without compromising structural integrity (nVent, 2013). Figure 7.3 illustrates the working principle of the interlock coupler, showing the threaded engagement and grout-filled sleeve that secures the connection.

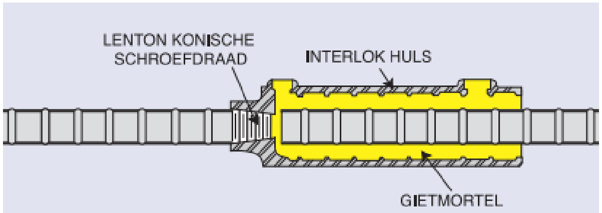


Figure 7.3: Interlock coupler (nVent, 2013)

7.2.4. Comparison of the mechanical couplers

Table 7.1 provides a comparative overview of the three mechanical coupler systems discussed in this section. This table serves as a reference for selecting the most suitable coupler based on project-specific constraints and requirements.

Coupler type	Modification required	Installation complexity	Bar alignment requirement	Diameter range (mm)	Structural performance
Lock Coupler	No modifications needed	Medium	Precise	Not specified	Comparable to a continuous bar
Quick Wedge Coupler	No modifications needed	Low	Less precise	12 - 20	Comparable to traditional overlap connections
Interlock Coupler	Threading	High	Precise	20 - 57	Comparable to a continuous bar

Table 7.1: Comparison of Mechanical Couplers

7.3. Overlapping with existing reinforcement

Another option for connecting reinforcement, besides direct connection using mechanical couplers, is creating an overlap. In overlapping connections, the force is transferred between the reinforcement bars through bond stress with the surrounding concrete. This requires a sufficient overlap length to fully develop the strength of the bars through the concrete between them, as specified in Eurocode 2 (NEN-EN 1992-1-1+C2, Section 8.7.3; Nederlands Normalisatie-instituut (NEN), 2011). More details on the required overlap length will be discussed in chapter 8. However, when reusing cast-in-situ concrete elements, direct overlapping of two existing reinforcement bars can be impractical. The required overlap length may be too long, necessitating excessive hydrodemolition to expose sufficient reinforcement which can result in an insufficient remaining concrete length of the element. This section discusses overlap options that are feasible with reused elements: post-installed reinforcement, memory steel and carbon fiber reinforced polymer (CFRP) lamellas. These methods provide alternative solutions to establish a strong and reliable connection between reused elements by enabling force transfer through overlapping reinforcement.

7.3.1. Post-installed reinforcement

Post-installed reinforcement is used to establish an overlapping connection with an existing reinforcement bar in situations where no prior provisions for such connections were made in the original design. These bars are anchored into existing concrete through drilled holes and bonded using structural adhesives, ensuring force transfer through bond stress with the surrounding concrete. This method allows for the introduction of new reinforcement to extend or strengthen existing elements, making it a practical solution for reconnecting reused structural components (Hilti Corporation, 2017).The maximum practical embedment depth commonly used for post-installed reinforcement bars is approximately 600 mm (Hilti Corporation, 2023). However, embedment depths ranging from 600 mm to 1000 mm are also feasible, although they considerably increase the complexity of installation. The maximum achievable depth depends on factors such as the type of mortar, the reinforcement properties and the ambient temperature. The requirements for drilling near existing reinforcement are specified in the ETA-certificate and illustrated in figure 7.4.

However, the feasibility of this method depends on the available concrete volume in the reused element and the required overlap length with the existing reinforcement. In conventional reinforcement splices, the overlap length is determined by the requirements of both reinforcing bars. However, experimental studies by Rovnak and Brozovsky (2014) have shown that high-strength adhesives can reduce the overlap length needed for post-installed reinforcement to values smaller than those specified in Eurocode 2 (NEN-EN 1992-1-1+C2, Section 8.4.4; Nederlands Normalisatie-instituut (NEN), 2011). Specifically, adhesives such as hybrid mortars, which combine organic and cementitious components and high-bond epoxy injection mortars allow for anchorage lengths to be reduced to below max$[15 \cdot \phi, 200$ mm$]$ (Rovnak & Brozovsky, 2014). As a result, in such cases, the required overlap length is primarily governed by the overlap length of the existing reinforcement.

Drilling method	Reinforcing bar ∅ [mm]	c_{min} [mm] without drilling aid	c_{min} [mm] with drilling aid
Hammer drilling or diamond core drilling	≤ 20	30mm + 0,06 l_{bd}*) $\geq 2\varnothing$	30mm + 0,02 $l_{bd} \geq 2\varnothing$
	≥ 25	40mm + 0,06 $l_{bd} \geq 2\varnothing$	40mm + 0,02 $l_{bd} \geq 2\varnothing$
Compressed air drilling	≤ 20	50mm + 0,08 l_{bd}	50mm + 0,02 l_{bd}
	≥ 25	60mm + 0,08 $l_{bd} \geq 2\varnothing$	60mm + 0,02 $l_{bd} \geq 2\varnothing$

*) l_{bd} = design value of anchorage (development) length, ∅ = diameter of reinforcing bar.

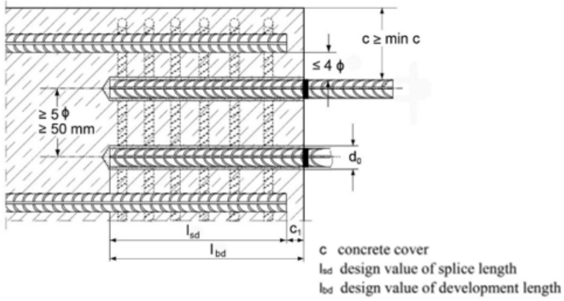


Figure 7.4: Requirements for post-installed reinforcement (Fuchs & Hofmann, 2020)

7.3.2. Carbon fiber reinforced polymers (CFRP)

When direct overlapping of reinforcement is not feasible in reused elements, carbon fiber-reinforced polymers (CFRP) lamellas offer an alternative means of strengthening connections. Unlike steel reinforcement, CFRP is applied externally, transferring tensile forces through bonding with the concrete surface. With a tensile strength of 3500 N/mm^2, CFRP can significantly enhance the load-bearing capacity of structural elements (Sika Nederland B.V., 2024). However, the effectiveness of CFRP is highly dependent on the method of application. When CFRP lamellas cannot be fully wrapped around an element, such as in a U-shaped configuration, the full strength of the material cannot be utilized. In such cases, force transfer relies only on bonding, which limits the usable strength to $40 - 50$% of the ultimate capacity, providing only a marginal 5% increase in strength of the element. To address this limitation, carbon fiber anchoring has been developed to improve bonding and enable CFRP to achieve its full strength. This method involves drilling holes into the concrete, inserting carbon fibers into the holes and spreading them over the externally applied lamellas. This anchoring technique enhances adhesion and ensures that CFRP can reach its full load-carrying potential, even when complete wrapping is not possible (Hub, 2023). In figure 7.5 an example of CFRP lamellas used to strengthen a concrete bridge is shown.

Figure 7.5: Example of CFRP lamellas used to strengthen a concrete bridge (Simpson Strong-Tie, 2024)

7.3.3. Memory steel

In addition to using CFRP lamellas to create an external overlap, Memory steel can also be used for this purpose. Memory steel is an innovative material made from iron-based shape memory alloys (Fe-SMA) designed for strengthening and repairing concrete and masonry structures. Its unique property is the ability to return to a predefined shape when heated, enabling active prestressing without the need for external tension devices. The material achieves tensile strengths of approximately 950 MPa and yield strengths around 400 MPa, with an elastic modulus of 160 GPa. These properties allow Memory steel to effectively improve structural performance while maintaining ductility, as it exhibits elongation at break beyond 20%. The prestressing process involves attaching Memory steel elements, such as re-bars or plates, to the reused elements, either by bonding or mechanical anchorage. Once installed, the material is heated to 160°C–200°C, triggering the shape memory effect. Upon cooling, the material contracts, inducing prestressing forces directly into the structure. This process eliminates the need for traditional prestressing jacks and simplifies installation (re-fer AG, 2021). An example of the use of memory steel to strengthen a concrete floor is shown in figure 7.6.

Figure 7.6: Example of memory steel used to strengthen a concrete floor(re-fer AG, 2021)

7.3.4. Comparison of the overlapping techniques

Table 7.2 provides a comparative overview of the three options to create an overlap with the existing reinforcement in reused elements, discussed in this section. This table serves as a reference for selecting the most suitable method based on project-specific constraints and requirements.

Method	Mechanism of Force Transfer	Required Modifications	Installation Complexity	Internal/External Application
Post-installed reinforcement	Bonding with surrounding concrete	Requires drilling holes and adhesive anchoring	Medium	Internal
CFRP lamellas	External bonding to concrete surface	Requires surface preparation and optional fiber anchoring	Medium to high	External
Memory steel	Activation of shape memory effect for prestressing	Requires heating to activate prestress	High	External

Table 7.2: Comparison of Overlapping Methods for Reinforcement Connections

7.4. Anchorage solutions

When reusing cast-in-situ concrete elements, the loss of anchorage due to cutting is a major challenge as discussed in the previous chapters. Anchorage solutions are required to ensure sufficient force transfer between the existing reinforcement bars and the surrounding concrete in the reused element. This section explores suitable anchorage solutions that can be used in reconnection designs to compensate for lost anchorage. The solutions include T-heads, bend anchorage, MBT mechanical end coupler and a threaded disk. At the end of this section, a comparative table summarizes the specifications and characteristics of each anchorage solution, providing a reference for selecting the most suitable option.

7.4.1. T-heads

T-heads are steel plates that are rigidly connected to the end of the existing reinforcement bars using friction welding, as shown in figure 7.7. These T-heads offer an alternative to traditional anchoring by replacing the required anchoring length with a direct force transfer mechanism at the end point of the rebar. Since the heads are welded onto the reinforcement bar, they ensure a strong and reliable connection without reducing the bar's cross-sectional strength (HRC Europe, 2024; Mavotrans, 2024). When using this solution, the T-heads must be embedded in newly cast concrete, as securing them tightly against the existing concrete surface is not feasible.

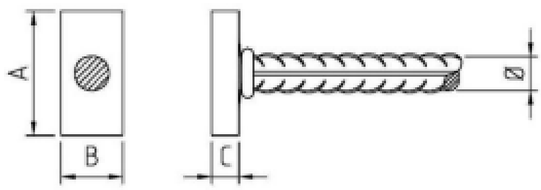


Figure 7.7: HRC T-heads anchorage system (HRC Europe, 2024)

7.4.2. Bend anchor

A bend reinforcement bar can be connected to the existing reinforcement using a mechanical coupling system, as shown in figure 7.8. This method replaces the traditional straight anchorage length by utilizing the bent shape of the reinforcement to transfer forces into the concrete (Halfen, 2017). According to Eurocode 2 (NEN-EN 1992-1-1+C2, Section 8.4.3; Nederlands Normalisatie-instituut (NEN), 2011), the anchorage length in such cases may be measured along the centerline of the bend. The bent anchorage must be embedded in newly cast concrete to ensure proper force transfer. Threads must be created in the existing reinforcement to connect with the bend anchorage.

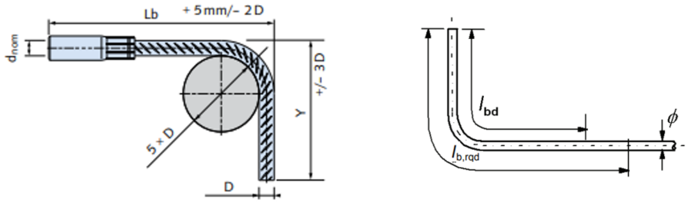


Figure 7.8: Bend anchor coupling (Halfen, 2017)

7.4.3. MBT mechanical anchorage

The MBT end anchorage coupler is a mechanical anchorage system designed to provide sufficient anchorage where traditional embedment lengths are not feasible. As shown in figure 7.9, the system consists of a sleeve that is pushed over the ends of the reinforcement bars to be connected. The connection is secured by tightening bolts, which grip the reinforcement and lock the coupler in place (Leviat, 2016; nVent, 2013). This method eliminates the need for additional anchorage length, making it a suitable solution for re-anchoring cut reinforcement bars in reused concrete elements. However, proper embedment in new concrete is required to ensure load transfer from the reinforcement into the surrounding structure.

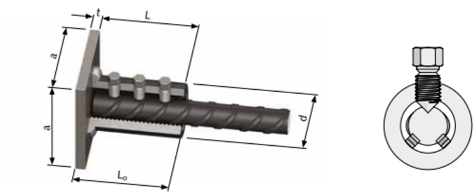


Figure 7.9: Anchorage coupler (Leviat, 2016)

7.4.4. Max Frank disk

The Max Frank end anchorage disk is a threaded anchorage system designed to provide efficient force transfer for reinforcement bars with limited anchorage length. As shown in figure 7.10, the system consists of a threaded disk that is attached to the end of the existing reinforcement bar. This disk increases the bearing area, improving anchorage capacity without requiring an extended embedment length. Unlike the other anchorage systems that must be fully embedded in newly cast concrete, the Max Frank disk can be installed directly against the existing concrete surface (MAX FRANK, 2024). However, a protective layer is required to prevent corrosion and ensure durability in long-term applications. This topic is further explored in the research by E. Bauden (Bauden, 2024). For outdoor elements, a cement-based cover layer for protection is needed.

Figure 7.10: Anchorage disk (MAX FRANK, 2024)

7.4.5. Comparison of the end anchorage solutions with specifications

The selection of an appropriate anchorage solution depends on various factors, including the available rebar length, space constraints and required force transfer. Table table 7.3 provides an overview of the key specifications for different anchorage methods. It compares the need for modifications to the existing rebar, the required anchorage length and surface area, the applicable rebar diameters and the necessary concrete strength. This comparison serves as a guide to determining the most suitable anchorage solution based on the constraints and conditions of the reused elements.

The specifications presented in this table are based on the information from the references provided in the subsections discussing each anchorage solution. The corresponding references can be found within the respective sections.

Specification	**T-heads**	**Bend anchor**	**MBT**	**Disk**
Modification of existing rebar	No	Yes, screw thread	No	Yes, screw thread
Required length (mm)	Max 25	Min 130 - 400	Min 85 - 262	Min 14 - 42.5
Required surface area (mm)	36 - 113	Negligible	70 - 150	45 - 130
Applicable rebar (ϕ)	$\phi 12 - \phi 40$	$\phi 12 - \phi 40$	$\phi 10 - \phi 40$	$\phi 12 - \phi 40$
Exposed rebar length (mm)	Minimal	Max 80	Max 250	Max 60
Concrete strength class	$\geq$ C30/37	Any concrete	**	**

** Concrete strength class is not defined in the ETA-certificate. For these solutions, the concentrated compressive force causes an internal cone of compressive stress. The compressive strength of the concrete at the end anchorage point must be carefully considered.

Table 7.3: Comparison of anchorage solutions for reused elements

7.5. Conclusion

This chapter explored various anchorage and reconnection solutions essential for integrating reused cast-in-situ concrete elements into new structural systems. Hydrodemolition was identified as an effective technique for exposing reinforcement while preserving structural integrity. Mechanical couplers offer a reliable method for reconnecting reinforcement, with different options available depending on alignment and spatial constraints. Overlapping techniques, including post-installed reinforcement, CFRP lamellas and memory steel, provide alternative solutions when direct coupling is not feasible. Additionally, various anchorage solutions were assessed to compensate for the loss of anchorage due to cutting, with the choice depending on factors such as available space and geometric constraints. The comparative tables provided an overview of the advantages and limitations of each method, highlighting that the optimal choice depends on the specific constraints of the reused element, such as available reinforcement bar length, installation feasibility and required force transfer capacity. The feasibility of these solutions is evaluated in the next chapter through real-world examples, assessing their application for each element in the case studies.

8

Case studies

This chapter analyzes two case studies of cast-in-situ concrete structures to bridge theoretical understanding and practical application through real-world examples. By examining the structural drawings of a parking garage and an office building constructed between 1960 and 1970, the aim is to determine whether the structural design and detailing of these buildings reflect the challenges identified in the preceding chapters. These examples provide initial insights into the feasibility of reconnecting the elements in new building systems. To systematically assess the feasibility of reconnecting these structural elements, various calculations are performed for each reusable element (columns, beams, floors and walls). Table 8.1 provides an overview of these calculations, specifying their purpose.

Table 8.1: Overview of calculations per structural element and their purpose.

Element	**Calculation**	**Purpose of the calculation**
Columns	Tensile capacity of horizontal reinforcement in the D-region	Verify whether the existing stirrups provide sufficient tensile capacity to resist T caused by point loads.
	Overlap length l_0 of vertical reinforcement	Assess whether post-installed reinforcement can be used for reconnection.
Walls	Overlap length l_0 of vertical reinforcement	Assess whether post-installed reinforcement is feasible for reconnection.
	Available space around the horizontal reinforcement bars	Determine which solutions are feasible to compensate for the lost edge reinforcement.
Beams	Length between the moment-zero points	Determine if a usable beam length remains after cutting at the moment-zero points.
	Shear capacity of a vertical joint	Assess whether the shear joint between the reused beam and new concrete provides adequate capacity to transfer the compression diagonal.
	Reinforcement ratio in the cross-section	Asses whether post-installed reinforcement is feasible for reconnection.
	Overlap length l_0 of horizontal reinforcement	Assess whether reconnecting reinforcement using an overlap is feasible for achieving a moment-fixed connection.

Continued on next page

Element	Calculation	Purpose of the calculation
	Anchorage length l_{bd} of horizontal reinforcement	Evaluate whether modifications are required to achieve sufficient anchorage for reinforcement in a hinged connection.
	Available space around the horizontal reinforcement bars	Determine which solutions are feasible to compensate for the lost anchorage lengths.
Floors	Length between the moment-zero points	Determine if a usable beam length remains after cutting at the moment-zero points.
	Reinforcement ratio in the cross-section	Asses whether post-installed reinforcement is feasible for reconnection.
	Overlap length l_0 of longitudinal reinforcement	Assess whether reconnecting reinforcement using an overlap is feasible for achieving a moment-fixed connection.
	Anchorage length l_{bd} of longitudinal reinforcement	Evaluate whether modifications are required to achieve sufficient anchorage for reinforcement in a hinged connection.
	Available space around the horizontal reinforcement bars	Determine which solutions are feasible to compensate for the lost anchorage lengths.

The calculations are based on Eurocode 2 and the used sections are listed below:

- The calculations of the **overlap lengths (l_0)** are based on the formulas in NEN-EN 1992-1-1+C2, Section 8.7; Nederlands Normalisatie-instituut (NEN), 2011.
- The calculations of the **anchorage lengths (l_{bd})** are based on the formulas in NEN-EN 1992-1-1+C2, Section 8.4; Nederlands Normalisatie-instituut (NEN), 2011.
- The calculations of the **shear capacity** in vertical joints are based on the formulas in NEN-EN 1992-1-1+C2, Section 6.2.5; Nederlands Normalisatie-instituut (NEN), 2011.
- The calculations of the **tensile capacity in the D-region** are based on the formulas in NEN-EN 1992-1-1+C2, Section 6.5; Nederlands Normalisatie-instituut (NEN), 2011.
- The **length between the moment zero points** is based on the analytical approach discussed in appendix A.3.

The following sections apply these calculations to the structural elements of the Munthof parking garage and the office building at Stationsplein 107, evaluating their feasibility for reconnection within new building systems.

8.1. Munthof parking garage

Figure 8.1: Munthof parking garage - Amsterdam (BouwTotaal, 2020)

The Munthof parking garage, built between 1966 and 1969, has five parking levels and two office floors (BouwTotaal, 2020). Its structure consists of a cast-in-situ concrete skeleton with beams and columns supporting the floors. Additionally, several concrete shear walls are present to provide structural stability. In the following subsections, the structural elements, including columns, beams, walls and floors, are analyzed to assess how the findings in the preceding chapters align with the existing design and reinforcement detailing. Since not all structural elements are identical throughout the building, the analysis focuses on the most frequently occurring configurations.

8.1.1. Columns

The challenges addressed in the theoretical part of this research for columns include the development of a D-region due to concentrated stresses, as discussed in section 6.3.1. Additionally, when reconnecting columns with other elements, determining the required overlap length of the existing vertical reinforcement is relevant in case the element would be reused under the same loading conditions. This helps determine whether the necessary length can be achieved using post-installed reinforcement or if alternative solutions, as discussed in section 7.3, should be considered.

8.1.1.1. Tensile capacity of horizontal reinforcement in the D-region

This calculation is performed to determine whether the existing stirrups in the D-region of the Munthof columns, can provide sufficient tensile capacity to resist point loads caused by two beams supported on the column. This is a common connection in prefabricated structures and figure 8.2 shows a schematic representation of this scenario.

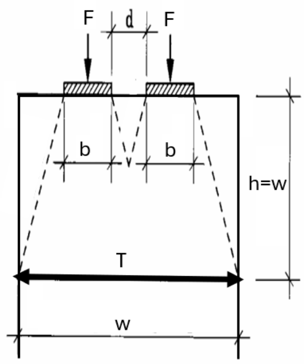


Figure 8.2: Schematic representation of two concentrated forces on top of a column

Most columns in the Munthof parking garage have a width (w) of 400 mm and contain ribbed horizontal reinforcement of type QR 40 (yield strength = 400 MPa) with a bar diameter of 12 mm (ϕ) and a spacing (s) of 200 mm. The tensile force T that develops due to the concentrated forces (F) can be determined using the following formula, derived from equation (6.1).

$$T = \frac{1}{4}\frac{w - (2 \cdot b)}{w}F \tag{8.1}$$

To determine the maximum force (F) that the column can withstand, equation (8.1) is rearranged to equation (8.2). Since a hinged support requires a gap (d) between the beams to allow rotation, the support width ($2 \cdot b$) cannot equal the full column width (w). Additionally, as illustrated in figure 6.7, proper bearing pad placement is necessary to prevent spalling. Therefore, the support width ($2 \cdot b$) is expressed as a percentage (P) of the total column width, which results in the following rearranged formula. The derivation of equation (8.2) is detailed in appendix A.1.2.

$$F = \frac{4}{1 - P}T \tag{8.2}$$

The reinforcement required to resist T may be distributed over a D-region with length $h = w$ (NEN-EN 1992-1-1+C2, Section 6.5.3; Nederlands Normalisatie-instituut (NEN), 2011). In this case, the tensile capacity of the reinforcement present in the region h = 400 mm provide the capacity T. To calculate the tensile capacity of the existing stirrups in the D-region of the column, the following formula is used:

$$N_t = 2 \cdot \frac{A_s}{s} \cdot h \cdot f_{yd} \tag{8.3}$$

The tensile capacity (N_t) is then substituted for T in equation (8.2), determining the point load capacity of the column, which is 943 kN with 67 % support area. To verify whether this capacity is sufficient to withstand realistically applied forces in a parking garage, the applied shear force (V_{Ed}) is determined using the ULS loads on the beams from the current design of the Munthof. The detailed verification can be found in appendix A.1. The calculated shear force (V_{Ed}) from two beams on the top of the column is 572 kN, which is significantly lower than the allowable point load (F) on the column (943 kN). This confirms that the existing reinforcement in the D-region of the columns provides sufficient tensile capacity to accommodate realistic loads in a parking garage, ensuring structural safety in the proposed reuse scenario. As a result, no external reinforcement solution is required for the column to support two beams in a hinged connection. To optimize this capacity, it is preferable to support beams along the longest column side, maximizing the resistance provided by the horizontal reinforcement.

8.1.1.2. Overlap length (l_0) of vertical reinforcement

The required overlap length (l_0) of the vertical reinforcement is determined to assess whether the necessary length can be achieved using post-installed reinforcement or if external solutions, as discussed in section 7.3, are required for connecting the reinforcement of the column to other elements by an overlap. To determine the overlap length, the base anchorage length ($l_{b,rqd}$) must first be calculated as it forms the basis for ensuring proper force transfer between overlapping reinforcement bars. The base anchorage length is calculated based on the bond strength (f_{ctd}) between the reinforcement and the surrounding concrete, which is expressed as:

$$f_{bd} = 2.25 \cdot \eta_1 \cdot \eta_2 \cdot f_{ctd} \tag{8.4}$$

Using this bond strength, the required base anchorage length can be determined as follows:

$$l_{b,rqd} = \frac{\phi}{4} \cdot \frac{\sigma_{sd}}{f_{bd}} \tag{8.5}$$

The actual overlap length is obtained by applying modification factors that account for specific conditions affecting the anchorage performance:

$$l_0 = \alpha_1 \cdot \alpha_2 \cdot \alpha_3 \cdot \alpha_5 \cdot \alpha_6 \cdot l_{b,rqd} \tag{8.6}$$

Finnaly, to verify the minimum overlap length ($l_{0,min}$) the following verification is used:

$$l_{0,min} \geq \max\{15 \cdot \phi, 200\,\text{mm}, 0.3 \cdot \alpha_6 \cdot l_{b,rqd}\} \tag{8.7}$$

The vertical reinforcement in the columns of Munthof consists of ribbed QR 40 bars with $\phi = 28$ mm. The concrete quality is K300, equivalent to C25/30 under current standards. Table 8.2 summarizes the variables used in the calculations, along with their corresponding values and the final results for the bond strength (f_{bd}), the base anchorage length ($l_{b,rqd}$) and the required overlap length (l_0). The detailed calculation and the description of the selected factors can be found in appendix A.2.

Variable	η_1	η_2	f_{ctd}	ϕ	σ_{sd}	α_1	α_2	α_3	α_5	α_6	$\boldsymbol{f_{bd}}$	$\boldsymbol{l_{b,rqd}}$	$\boldsymbol{l_{0,min}}$	$\boldsymbol{l_0}$
Value	1	1	1.2 N/mm^2	28 mm	349 N/mm^2	1	1	1	1	1	2.7 N/mm^2	905 mm	407 mm	1358 mm

Table 8.2: Overview of parameters and results for calculation of the overlap length of the column in Munthof

The calculated 1358 mm overlap length ensures proper force transfer if the column is reused under its original load conditions, assuming the reinforcement stress reaches yield strength. If the new design has significantly lower loads, this overlap length is conservative and may even become unnecessary. Moreover, 1358 mm exceeds the practical drill depth for post-installed reinforcement, which is typically limited to 600 mm (section 7.3.1). If vertical reinforcement connection is required, external strengthening solutions (e.g., CFRP lamellas, Memory Steel) can ensure structural integrity and load transfer under the original design loads. Alternatively, by reducing the applied loads, the required overlap length can be achieved with post-installed reinforcement or even eliminate the need for overlapping the vertical reinforcement, simplifying the reconnection.

8.1.2. Walls

Challenges in reusing wall elements have also been discussed in the theoretical part of this research. The required overlap length of vertical reinforcement is interesting or assessing how the wall elements can be reconnected with other elements. Additionally, cutting the walls removes edge reinforcement. The available space around the horizontal reinforcement is determined to assess the feasible solutions to compensate for the lost edge reinforcement.

8.1.2.1. Overlap length (l_0) of vertical reinforcement

The vertical reinforcement in the walls consists of ribbed QR 40 bars with $\phi = 12$ mm. The concrete of the walls is of quality K300, equivalent to C25/30 under current standards. The required overlap length of the vertical reinforcement is calculated in the same way as for the column. Specifically, equations (8.4) to (8.7) are used again, which define the bond strength (f_{ctd}), base anchorage length ($l_{b,rqd}$), the overlap length (l_0) and the minimum overlap length ($l_{0,min}$) respectively. The summary of the parameters and results for the overlap length is presented in table 8.3 and the detailed calculation with explanation of the values can be found in appendix A.10.

Variable	η_1	η_2	f_{ctd}	ϕ	σ_{sd}	α_1	α_2	α_3	α_5	α_6	$\boldsymbol{f_{bd}}$	$\boldsymbol{l_{b,rqd}}$	$\boldsymbol{l_{0,min}}$	$\boldsymbol{l_0}$
Value	1	1	1.2 N/mm^2	12 mm	349 N/mm^2	1	1	1	1	1.5	2.7 N/mm^2	388 mm	200 mm	582 mm

Table 8.3: Overview of parameters and results for calculation of the overlap length of the vertical reinforcement in the wall in Munthof

An overlap length of 582 indicates that creating an overlap using post-installed reinforcement is feasible. However, this length is close to the practical limit so alternative solutions for reconnecting the reinforcement may be desirable. The height of the wall is 2.8 m, while the minimum height required for a parking garage is 2.4 m (Rijksoverheid Nederland, 2025). This provides an opportunity to free up some reinforcement for mechanically reconnecting.

As with columns, the calculated overlap length assumes that the wall element is reused under the same load conditions as in its original design. Additionally, it is assumed that the reinforcement reaches its yield strength under the current load conditions. However, if the applied loads in the new structure are significantly lower, the required overlap length will also decrease and may even become zero. This highlights the conservativeness of the overlap length calculation and underlines the importance of assessing the actual forces in the new structural system to simplify the reconnection.

8.1.2.2. Available space around the horizontal reinforcement bars

For the walls in the Munthof garage, which have a thickness of 200 mm and a horizontal reinforcement spacing of 200 mm, several anchorage solutions remain feasible. Given that the reinforcement bars have a diameter of 8 mm, the T-heads, MBT couplers and threaded disk anchors can be accommodated within the available 200 mm spacing. In contrast, bend-anchorage may not be practical, as the required anchorage length could exceed the available wall thickness, making proper anchorage difficult to achieve. An alternative approach is to weld the reinforcement to that of an adjacent wall, ensuring structural continuity without the need for additional anchorage length. For both options, a part of the concrete would need to be removed using hydro-demolition to expose the reinforcement for connection.

8.1.3. Beams

Similarly, for beams, challenges related to their reuse have been discussed in the theoretical part of this research. An important factor is the location where it is cut, either near the support or at the moment-zero point. These differences impact the possible methods for reconnecting and support conditions in the new structure. Cutting near the existing fixed support results in a beam-end with dominant top reinforcement, requiring a fixed support condition for reuse. The required overlap length is calculated for this section to determine the necessary length for an overlap to reconnect the reinforcement, which is essential for achieving a new fixed connection. In contrast, cutting at the moment-zero point can lead to a greater loss of shear reinforcement and leaves the beam-end with dominant bottom reinforcement, making it more suitable for a hinged support. When choosing to cut at the moment-zero points, it is essential to assess whether the remaining length of the beam is still suitable for reuse. The anchorage length is determined for the moment-zero cross-section to establish the required anchorage length when the beam is hinged-supported in the new design. The possible anchorage solutions are evaluated by assessing the spacing of the longitudinal reinforcement. Additionally, it is important to evaluate whether the shear capacity of a joint between the reused beam and the newly poured concrete is sufficient, as this determines whether a continuous beam can be re-established in the new structural system.

The beam span of the Munthof garage between two continuous supports, which are assumed to act as fixed supports, is 10.32 m and is designed with a ULS load of 55.45 kN/m. The schematic representation of the beam in the Munthof garage is shown in figure 8.3

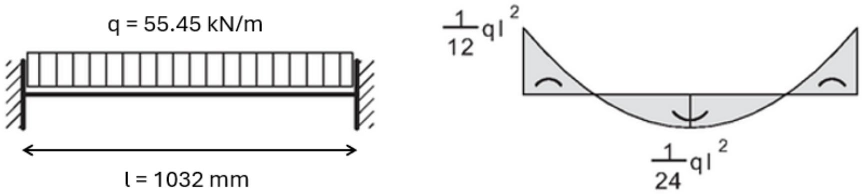


Figure 8.3: Schematic representation of the beam in Munthof

8.1.3.1. Length between the moment-zero points

Cutting at the moment-zero points is only practical if the remaining beam length fits the new design. In figure 8.3 a schematic representation of this beam is shown. The analytical derivation of the location of the moment-zero points in a fixed-fixed beam can be found in appendix A.3, which shows that the moment-zero points are located around $0.221 \cdot l$ and $0.789 \cdot l$. This means that when this beam is cut at these points, it results in a remaining element length of 5.8 m, which is still a feasible span for certain applications. The dimensions of the beam in the Munthof parking garage is 600 mm by 600 mm. The slenderness ratio of the beam is $5780/600 = 9.63$, indicating that it is relatively stiff for this span. Since its possible that more shear reinforcement is lost due to cutting at the moment-zero points, this additional stiffness may be beneficial in compensating for the reduced shear capacity.

8.1.3.2. Shear capacity of a vertical joint

The reconnection options for a beam that has been cut at the moment-zero points include a hinged support on a column or a connection to the T-shaped column. When the beam is connected to the T-shaped column, it is crucial to assess the shear strength of the vertical joint between the hydrodemolished surface of the reused beam and the newly poured concrete, as the shear joint is not supported. A schematic representation of the shear joint is shown in figure 8.4.

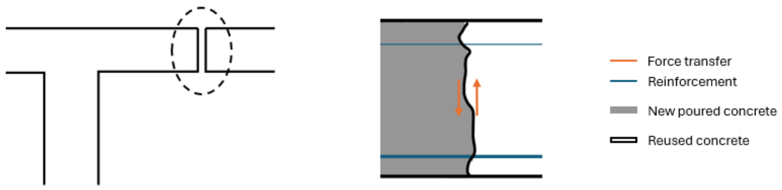


Figure 8.4: Schematic representation of a reconnection of a T-shaped column and a beam

Since the interface is assumed to be rough, as described in section 7.1, the shear verification is performed according to Eurocode 2 (EN 1992-1-1+C2, Section 6.2.5; Nederlands Normalisatie-instituut (NEN), 2011), considering adhesion ($c = 0.4$), friction ($\mu = 0.7$) and reinforcement crossing the joint. For the Munthof garage, the beam has a cross-section of 600 mm $\times$ 600 mm (as shown in figure 8.5) and contains ribbed QR 40 reinforcement with a diameter of 26 mm. In this section, 8 reinforcement bars are in tension.

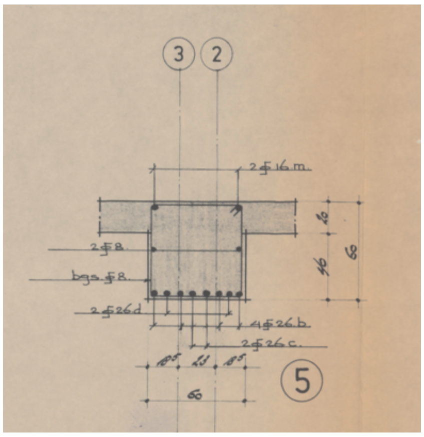


Figure 8.5: Cross-section moment-zero point in the beam of Munthof

To calculate the design shear strength the following formula is used:

$$v_{Rdi} = c \cdot f_{ctd} + \mu \cdot \sigma_n + \rho \cdot f_{yd}(\mu \sin \alpha + \cos \alpha) \tag{8.8}$$

A detailed verification and step-by-step calculation can be found in appendix A.21. It confirms that the actual shear stress at the joint ($v_{Edi} = 0.347$ N/mm^2) does not exceed the design shear strength ($v_{Rdi} = 3.66$ N/mm^2), indicating that no additional shear reinforcement through the joint is required to ensure structural integrity. This means that the shear force can be transferred diagonally through the joint. However, a stirrup in the newly poured concrete is required to anchor this force. To accommodate this, at least one $\phi 12$ mm bar ($A_s = 113.10$ mm^2) should be placed in the new concrete along the shear force trajectory. Since this corresponds to the $21.8°$ diagonal force transfer, it should be positioned at a distance of 0.24 m to ensure proper anchorage and force transfer.

8.1.3.3. Reinforcement ratio in the cross-section

An other important factor influencing the feasibility of reconnecting a beam is the reinforcement ratio in its cross-section. As discussed in section 5.2.3, Eurocode 2 (NEN-EN 1992-1-1+C2, Section 9.2.1.1; Nederlands Normalisatie-instituut (NEN), 2011) specifies that the reinforcement ratio in a beam's cross-section must not exceed 4%, while in overlap splices it must not exceed 8% of the concrete cross-sectional area. If the existing reinforcement ratio is already near these limits, it may restrict the possibility of using post-installed reinforcement for reconnection. The cross-section with the most tensile reinforcement is the section located near by the fixed support and shown in figure 8.6.

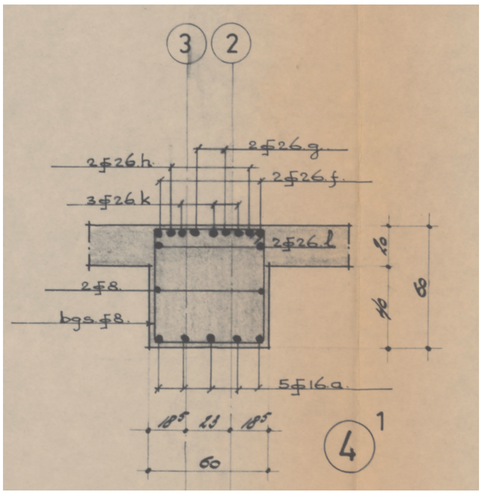


Figure 8.6: Cross-section near by the support in Munthof

To assess the reinforcement ratio in this cross-section, the effective tensile zone is considered. The effective reinforcement ratio is calculated using the tensile reinforcement area relative to the effective concrete area, as detailed in appendix A.4. This results in an effective reinforcement ratio of approximately 1.75%. Since the current ratio is well below the threshold of 4% and 8%, additional reinforcement can be drilled and installed if required to facilitate proper connections in this case.

8.1.3.4. Overlap length (l_0) of horizontal reinforcement

The overlap calculation is performed to determine the required length for reconnecting post-installed reinforcement with the existing reinforcement of the beams through an overlap. This reconnection is necessary to ensure a moment-fixed support condition, allowing the beam to transfer bending moments effectively. With this reconnection, the anchorage of the horizontal reinforcement is ensured. The calculation is conducted for the beam cross-section near the supports, as shown in figure 8.6, since this is the section where the top reinforcement is dominant, making it suitable for a moment-fixed connection. The tensile reinforcement in this cross-section consists of 11 ribbed QR 40 bars with a diameter of 26 mm. The required overlap length for the horizontal reinforcement is determined using the same formulas as applied in the column and wall calculations in sections 8.1.1 and 8.1.2. Specifically, equations (8.4) to (8.7) are used, which define the bond strength (f_{ctd}), base anchorage length ($l_{b,rqd}$) the overlap length (l_0) and the minimum overlap length ($l_{0,min}$) respectively. The reinforcement stress (σ_{sd}) used to determine the base anchorage length is calculated based on the current load conditions and span of the beam. A schematic representation of this span and loading configuration is shown in figure 8.3.

The summary of the parameters and results for the overlap length is presented in table 8.4 and the detailed calculation with explanation of the values can be found in appendix A.5.

Variable	η_1	η_2	f_{ctd}	ϕ	σ_{sd}	α_1	α_2	α_3	α_5	α_6	$\boldsymbol{f_{bd}}$	$\boldsymbol{l_{b,rqd}}$	$\boldsymbol{l_{0,min}}$	$\boldsymbol{l_0}$
Value	0.7	1	1.2 N/mm^2	26 mm	165 N/mm^2	1	1	1	1	1.5	1.89 N/mm^2	567 mm	390 mm	851 mm

Table 8.4: Overview of parameters and results for calculation of the overlap length in the beam section near the support in Munthof

The required overlap length of 851 mm can be realized through two approaches. The first approach involves hydrodemolition, where 851 mm of existing concrete is removed to facilitate the overlap of the reinforcement bars. However, this significantly reduces the effective beam length, which may compromise the feasibility of the new structural design. Alternatively, post-installed reinforcement could be used to maintain the existing beam length. However, a 851 mm drill depth exceeds typical practical limits for post-installed reinforcement, making this solution potentially unfeasible without modifications to the anchorage of the existing reinforcement.

If the beam is reused in a new structural system with lower loads, the required overlap length will decrease, as the reinforcement stress governing the base anchorage length will be lower. Consequently, the need for hydrodemolition or deep post-installed reinforcement will be reduced.

8.1.3.5. Anchorage length (l_{bd}) of horizontal reinforcement

As discussed in section 5.2.1, the anchorage length (l_{bd}) of the reinforcement in the beams is lost when they are cut from an existing building. For a fixed connection, the reinforcement must be continuous, meaning its anchorage is inherently ensured. However, for a hinged connection, the reinforcement is interrupted, making it necessary to explicitly determine the required anchorage length. Therefore, the anchorage length of the existing reinforcement bars is calculated in the moment-zero cross-section of the beams. This section contains dominant bottom reinforcement, allowing the beam to be supported as a hinged connection. The cross-section is shown in figure 8.5.

With the bond strength (f_{bd}) and base anchorage length ($l_{b,rqd}$), calculated with equations (8.4) and (8.5), the anchorage length can be calculated using the following formula:

$$l_{bd} = \alpha_1 \cdot \alpha_2 \cdot \alpha_3 \cdot \alpha_4 \cdot \alpha_5 \cdot l_{b,rqd} \tag{8.9}$$

To verify the minimum anchorage length ($l_{b,min}$), the following verification is used:

$$l_{b,min} \geq \max\{10 \cdot \phi, 100\,\text{mm}, 0.3 \cdot l_{b,rqd}\} \tag{8.10}$$

The summary of the parameters and results are presented in table 8.5 and the detailed calculation with explanation of the values can be found in appendix A.6.

Variable	η_1	η_2	f_{ctd}	ϕ	σ_{sd}	α_1	α_2	α_3	α_4	α_5	$\boldsymbol{f_{bd}}$	$\boldsymbol{l_{b,rqd}}$	$\boldsymbol{l_{b,min}}$	$\boldsymbol{l_{bd}}$
Value	1	1	1.2 N/mm^2	26 mm	29.02 N/mm^2	1	1	1	1	1	2.7 N/mm^2	70 mm	260 mm	70 mm

Table 8.5: Overview of parameters and results for calculation of the anchorage length in the beam section at the moment-zero point in Munthof

Since the calculated anchorage length l_{bd} is lower than the minimum anchorage length $l_{b,min}$, the required anchorage length is governed by $l_{b,min} = 260$ mm. When a reused beam is hinged-supported in a new structural system, direct anchorage of the reinforcement after the support is essential to ensure effective force transfer. An anchorage length of 260 mm may be insufficient due to limited available space on the column, necessitating an extended anchorage solution, as discussed in section 7.4. To determine which of these solutions are feasible, the available space is analyzed in the following section.

8.1.3.6. Available space around the horizontal reinforcement bars

To determine whether each solution described in section 7.4 is feasible, the spacing between the horizontal reinforcement in the section shown in figure 8.5 is determined. The spacing is found to be 37 mm. Given this spacing and a beam thickness of 600 mm, only the bend anchor and T-heads, discussed in section 7.4, can be used for the end-anchorage of the longitudinal reinforcement in this beam.

8.1.4. Floors

Finally, the challenges related to reusable floor elements, as discussed in the theoretical part of this research, are examined. The remaining span between the moment-zero points is determined to assess whether it is sufficient for reuse in a new structural system with hinged connections. The reinforcement ratio in the cross-section is evaluated to determine whether post-installed reinforcement can be used for reconnection by an overlap. Similar to walls, the edge reinforcement of the original two-way spanning floor is removed after cutting. However, after cutting, the reused element spans in only one direction, making the lost edge reinforcement redundant. If the floor element is reused in a moment-fixed connection, an overlap length (l_0) is calculated determine the feasibility of reconnecting the reinforcement by an overlap. Alternatively, if the element is hinged-supported, the required anchorage length (l_{bd}) must be provided to secure the reinforcement in the new design. Lastly, the available space of the longitudinal reinforcement is analyzed to identify feasible anchorage solutions for securing the connection.

The ULS load on the floor of the Munthof garage is 8.78 kN/m^2 and the span is 6.35 m. The reinforcement layout in the floors is shown in figure 8.7

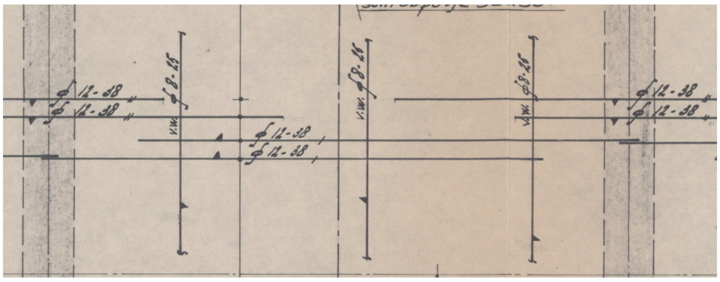


Figure 8.7: Reinforcement layout in the floors in Munthof

8.1.4.1. Length between the moment-zero points

Cutting at the moment-zero points is only a viable option if the remaining floor element length is suitable for reuse in a new design. The analytical derivation of the moment-zero points in a fixed-fixed slab, as shown in appendix A.3, indicates that they are located at approximately $0.221 \cdot l$ and $0.789 \cdot l$. Applying this to a floor element with an initial span of 6.35 m, these points are positioned at 1.4 m and 5 m from one end. This results in a remaining span of approximately 3.6 m, which is often too short for reuse in new designs.

8.1.4.2. Reinforcement ratio in the cross-section

Similar to the beams, analyzing the reinforcement ratio in the concrete cross-section of floor elements is essential for assessing the feasibility of implementing post-installed reinforcement in reconnection designs. This analysis considers a 1 m wide floor element that can be cut from the Munthof parking garage floors. The reinforcement layout is shown in figure 8.7. The longitudinal reinforcement consists of bars with a diameter of 12 mm, placed at a spacing of 190 mm. In appendix A.7, it is calculated that the reinforcement ratio is 0.45%, which does not exceed the Eurocode 2 (NEN-EN 1992-1-1+C2, Section 9.2.1.1; Nederlands Normalisatie-instituut (NEN), 2011) limits of 4% for the cross-section and 8% for overlap splices. Therefore, the implementation of post-installed reinforcement for reconnecting the floor element is feasible within these limits.

8.1.4.3. Overlap length (l_0) of the longitudinal reinforcement

While the reinforcement ratio in the floor allows for additional post-installed reinforcement, its feasibility for reconnecting reinforcement bars by an overlap also depends on the required overlap length. The overlap length for the longitudinal reinforcement is determined using the same approach as for the column, wall and beam calculations in sections 8.1.1 to 8.1.3, applying equations (8.4) to (8.7), which define the bond strength (f_{ctd}), base anchorage length ($l_{b,rqd}$), overlap length (l_0) and the minimum overlap length ($l_{0,min}$) respectively. The calculations are carried out for the cross-section near the supports with top reinforcement, as its continuity is essential for achieving a moment-fixed connection. The reinforcement consists of ribbed QR 40 bars with a diameter of 12 mm. A summary of the parameters and results for the overlap length is presented in table 8.6, with a detailed calculation and justification of the values provided in appendix A.8.

Variable	η_1	η_2	f_{ctd}	ϕ	σ_{sd}	α_1	α_2	α_3	α_5	α_6	$\boldsymbol{f_{bd}}$	$\boldsymbol{l_{b,rqd}}$	$\boldsymbol{l_{0,min}}$	$\boldsymbol{l_0}$
Value	0.7	1	1.2 N/mm^2	12 mm	357 N/mm^2	1	1	1	1	1.5	1.89 N/mm^2	567 mm	255 mm	851 mm

Table 8.6: Overview of parameters and results for calculation of the overlap length in the floor section near by the support in Munthof

The required overlap length of 851 mm presents some challenges for reconnection. One approach involves hydrodemolition, where 851 mm of existing concrete is removed to facilitate the overlap of the existing reinforcement bars. However, this would substantially reduce the effective element length, which may make reuse impractical in the new design. Alternatively, post-installed reinforcement could be used to maintain the original element length. However, drilling to a depth of 851 mm exceeds standard practical limit of 600 mm, making this method unfeasible without modifications for the anchorage of the existing reinforcement. If the floor element is reused in a structural system with lower loads, the required overlap length may decrease, reducing both the need for extensive hydro-demolition and the drilling depth for post-installed reinforcement.

8.1.4.4. Anchorage length (l_{bd}) of longitudinal reinforcement

As with beams, the anchorage length (l_{bd}) of the longitudinal reinforcement in floors will also be lost after cutting the elements, as discussed in section 5.2.1. When a floor element cut at the moment-zero points is reused, proper anchorage of the horizontal reinforcement is essential to ensure effective force transfer directly next to the new hinged supports. Therefore the anchorage length is calculated. Equations (8.4), (8.5), (8.9) and (8.10) are used to calculate the effective bond strength (f_{bd}), the base anchorage length ($l_{b,rqd}$), the anchorage length (l_{bd}) and the minimum anchorage length ($l_{bd,min}$) respectively. The overview of the variables and values for this calculation is shown in table 8.5 and the detailed calculation with value explanation can be found in appendix A.9.

Variable	η_1	η_2	f_{ctd}	ϕ	σ_{sd}	α_1	α_2	α_3	α_4	α_5	$\boldsymbol{f_{bd}}$	$\boldsymbol{l_{b,rqd}}$	$\boldsymbol{l_{b,min}}$	$\boldsymbol{l_{bd}}$
Value	1	1	1.2 N/mm^2	12 mm	27.52 N/mm^2	1	1	1	1	1	2.7 N/mm^2	61 mm	120 mm	61 mm

Table 8.7: Overview of parameters and results for calculation of the anchorage length in the floor section at the moment-zero point in Munthof

Since the calculated anchorage length l_{bd} is lower than the minimum anchorage length $l_{b,min}$, the required anchorage length is governed by $l_{b,min} = 120$ mm. Anchorage is required immediately after the support in the case of a hinged connection. However, if the required anchorage length of 120 mm can be fully accommodated above the supports, alternative anchorage solutions, as discussed in section 7.4, may not be required.

8.1.4.5. Available space around the horizontal reinforcement bars

The feasibility of the anchorage solutions discussed in section 7.4 depends on reinforcement spacing. The spacing between the longitudinal reinforcement of the floor elements is 190 mm. Given a bar diameter of 12 mm, this spacing is sufficient to accommodate the T-heads, threaded disk and the MBT end-anchorage, ensuring their feasibility for implementation.

8.2. Stationsplein 107

Figure 8.8: Stationsplein 107 - Leiden

The building at Stationsplein 107, originally constructed in 1965 for office use, features a cast-in-situ concrete skeleton with columns and beams supporting the floors. Additionally, several concrete shear walls are present to provide structural stability. In the following subsections, the structural elements, including columns, beams, walls and floors, are analyzed to evaluate how the existing design and reinforcement detailing align with the challenges and insights identified in the theory. The same methodological approach as for the Munthof parking garage and as discussed in table 8.1 is applied to each element. Since not all structural elements are identical throughout the building, the analysis focuses on the most frequently occurring configurations.

8.2.1. Columns

The challenges addressed for the columns in the Munthof garage included verifying whether the existing horizontal reinforcement in the D-region of the column provides sufficient capacity to resist realistic vertical point loads. For Stationsplein 107, this capacity is also evaluated. Additionally, the overlap length of the vertical reinforcement is calculated to assess whether post-installed reinforcement can be used for reconnection by an overlap.

8.2.1.1. Tensile capacity of horizontal reinforcement in the D-region

The tensile capacity of the horizontal reinforcement in the D-region of the columns is calculated using the same method as applied for the Munthof parking garage. The schematic representation of the scenario of this calculation is shown in figure 8.2. Most columns in Stationsplein 107 have a width (w) of 300 mm, which, according to Eurocode 2 (NEN-EN 1992-1-1+C2, Section 6.5.3; Nederlands Normalisatie-instituut (NEN), 2011), defines the height of the D-region as 300 mm. The horizontal reinforcement in these columns consists of smooth QR 24 (yield strength = 240 MPa) bars with a diameter of 8 mm (ϕ) and a spacing of 200 mm. To determine the maximum point load (F) that the column can withstand, equation (8.2) is used, where N_t, as calculated with equation (8.3), is substituted for T. The detailed calculation can be found in appendix A.11, with the maximum point load (F) that the column can withstand being 185 kN. To verify whether this capacity is sufficient for realistically applied loads in an office building, the ULS load on the beams from the current design of Stationsplein 107 is used to determine the applied shear force (V_{Ed}). The calculated V_{Ed} in appendix A.11 is 262 kN, which exceeds the column's load-bearing capacity. Therefore, column head strengthening with CFRP lamellas or memory steel, is required for the column to support two beams in a hinged connection under the original design loads. However, if the applied loads in the new design are lower, additional strengthening may not be necessary.

8.2.1.2. Overlap length (l_0) of vertical reinforcement

The required overlap length (l_0) has also been calculated for the vertical reinforcement in the columns of Stationsplein 107, to assess whether the necessary length can be achieved using post-installed reinforcement or if solutions other than overlapping are required for reconnecting. It is calculated in the same way as before with equations (8.4) to (8.7). The vertical reinforcement in the column consists of ribbed QR 42 (yield strength = 420 MPa) with a diameter of 12 mm. The summary of the variables and values is shown in table 8.8 and the detailed explanation can be found in appendix A.12.

Variable	η_1	η_2	f_{ctd}	ϕ	σ_{sd}	α_1	α_2	α_3	α_5	α_6	$\boldsymbol{f_{bd}}$	$\boldsymbol{l_{b,rqd}}$	$\boldsymbol{l_{0,min}}$	$\boldsymbol{l_0}$
Value	1	1	1.2 N/mm^2	12 mm	365 N/mm^2	1	1	1	1	1.5	2.7 N/mm^2	406 mm	200 mm	609 mm

Table 8.8: Overview of parameters and results for calculation of the overlap length of the vertical reinforcement in the columns in Stationsplein 107

With an overlap length of 609 mm, the required length slightly exceeds the practical drillable depth for using post-installed reinforcement to reconnect the reinforcement. Therefore, if the reinforcement needs to be connected, alternative solutions such as CFRP lamellas or memory steel, as described in section 7.4, can be considered. As with the columns in Munthof, this calculation assumes that the reinforcement reaches its yield strength under the current load conditions. If the applied loads in the new structure are lower, the required overlap length will also decrease, making this a potentially conservative estimate. In some cases, a significant reduction in load may render the vertical reinforcement structurally unnecessary, effectively reducing the required overlap length to zero.

8.2.2. Walls

The walls of Stationsplein 107 are analyzed in the same way as those in the Munthof. First, the required overlap length (l_0) of the vertical reinforcement in the wall is calculated to determine whether it can be achieved using post-installed reinforcement. Next, the available space around the horizontal reinforcement is determined to assess which anchorage solutions are feasible. This is necessary because the edge reinforcement of the walls has been lost due to cutting.

8.2.2.1. Overlap length (l_0) of vertical reinforcement

The vertical reinforcement in the walls of Stationsplein 107 consists of ribbed QR 42 with a diameter of 10 mm and the concrete is of quality C$25/30$. The overlap length is calculated in the same way as before, with equations (8.4) to (8.7) and can be found in detail in appendix A.19. The summary of the variables and values are listed in table 8.9.

Variable	η_1	η_2	f_{ctd}	ϕ	σ_{sd}	α_1	α_2	α_3	α_5	α_6	$\boldsymbol{f_{bd}}$	$\boldsymbol{l_{b,rqd}}$	$\boldsymbol{l_{0,min}}$	$\boldsymbol{l_0}$
Value	1	1	1.2 N/mm^2	10 mm	365 N/mm^2	1	1	1	1	1.5	2.7 N/mm^2	338 mm	200 mm	507 mm

Table 8.9: Overview of parameters and results for calculation of the overlap length of the vertical reinforcement in the walls in Stationsplein 107

The overlap length of 507 mm is achievable with post-installed reinforcement but is at the upper limit of practical feasibility. The height of the walls is 3.3 m, while the minimum height required for a living space is 2.6 m (Rijksoverheid Nederland, 2025). This provides a significant opportunity to free up some reinforcement for the reconnection, as part of the wall height can be removed to fit the new design. As with the columns, this calculation assumes that the reinforcement reaches its yield strength under the current load conditions, making it a conservative estimate. If the applied loads in the new structure are lower, the required overlap length will decrease or may not be needed at all.

8.2.2.2. Available space around the horizontal reinforcement bars

For the walls in Stationsplein 107, which have a thickness of 200 mm and a spacing of 200 mm, the feasibility of anchorage solutions is similar to that of the Munthof garage. With a reinforcement diameter of 10 mm, T-heads, MBT couplers and threaded disk anchors remain viable options within the available 200 mm space. As in the previous case, bend-anchorage is unlikely to be practical, as the required anchorage length may exceed the wall thickness. Welding the reinforcement to that of an adjacent wall element is also a viable solution. For both options, hydro-demolition is required to expose the reinforcement for connection.

8.2.3. Beams

The same challenges related to beam reuse, as discussed in the theoretical part of this research, are evaluated for the beams in Stationsplein 107. The suitability of the remaining beam length and the shear capacity of a vertical joint with newly poured concrete are assessed. Additionally, the overlap length of the horizontal reinforcement, in the cross-section near by the supports, is calculated to determine the required length for reconnecting the existing reinforcement through an overlap in a new moment-fixed connection. Finally, the anchorage length is determined for the moment-zero cross-section to determine the required anchorage length when the beam is hinged-supported in the new design. The feasibility of anchorage solutions is evaluated by analyzing both the anchorage length and the reinforcement spacing. The span of the beam is 8.15 m long and is designed with an ULS load of 32.07 kN/m. The reinforcement layout of the beam in Stationsplein 107 is shown in figure 8.9 and the schematic representation is shown in figure 8.10.

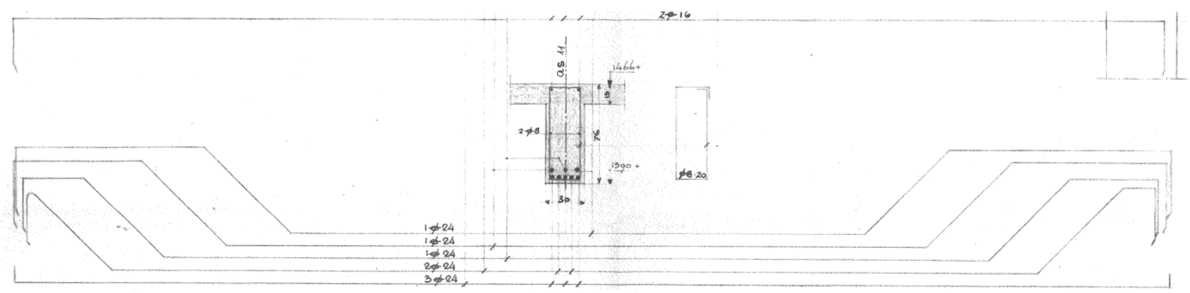


Figure 8.9: Stationsplein 107 - Leiden beam drawing

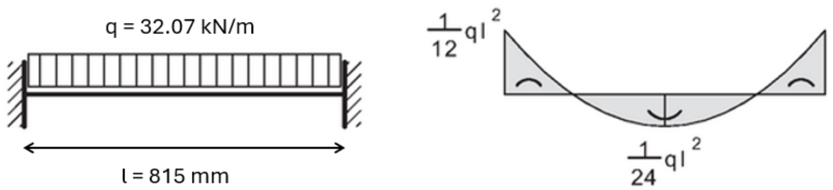


Figure 8.10: Schematic representation of the beam in Stationsplein 107

8.2.3.1. Length between the moment-zero points

For the beams in Stationsplein 107, it is assessed whether a sufficient beam length remains after cutting at the moment zero points. This approach enables the beam to be supported in a hinged configuration in the new design, as the dominant reinforcement at the beam ends will then be located at the bottom. The figure below shows the schematic representation of the beam in Stationsplein 107. As shown in appendix A.3, the moment zero point will approximately be at $0.221 \cdot l$ and $0.789 \cdot l$. After cutting at the moment zero points, the remaining beam length is 4.5 m, which is relatively uncommon for typical office spans. This may pose challenges in integrating the reused elements into a new hinged connection with a column. However, this challenge is not present when the beam is reconnected to a T-shaped column (the schematic representation is shown in figure 8.4, as this allows the full original span to be maintained.

8.2.3.2. Shear capacity of a vertical joint

For a T-shaped column with beam connection, it is crucial to assess the shear strength of the vertical joint between the hydrodemolished surface of the reused beam and the newly poured concrete, as this joint is not supported. The schematic representation of this type of reconnection is shown in figure 8.4. As in the Munthof case, the shear verification follows Eurocode 2 (EN 1992-1-1+C2, Section 6.2.5;Nederlands Normalisatie-instituut (NEN), 2011), assuming a rough interface as described in section 7.1, with adhesion ($c = 0.4$), friction ($\mu = 0.7$) and reinforcement crossing the joint. The beam has a cross-section of 300 mm $\times$ 760 mm (as shown in figure 8.9) and contains ribbed QR 42 reinforcement with a diameter of 24 mm. At the moment zero point, 8 rebars are in tension. To calculate the design shear strength again equation (8.8) is used and a detailed verification and step-by-step calculation can be found in appendix A.22. It confirms that the actual shear stress at the joint ($v_{Edi} = 0.35$ N/mm^2) does not exceed the design shear strength ($v_{Rdi} = 4.49$ N/mm^2), indicating that no additional shear reinforcement is required through the joint to ensure structural adequacy. This means that the shear force can be transferred diagonally through the joint. However, a stirrup in the newly poured concrete is required to anchor this force. To accommodate this, at least one $\phi 8$ mm bar ($A_s = 50.27$ mm²) should be placed in the new concrete along the shear force trajectory. Since this corresponds to the $21.8°$ diagonal force transfer, it should be positioned at a distance of 0.30 m to ensure proper anchorage and force transfer.

8.2.3.3. Reinforcement ratio in the cross-section

The reinforcement ratio of the beam, shown in figure 8.9, is checked to assess the feasibility of reconnecting them with post-installed reinforcement. The cross-section with the highest concentration of tensile reinforcement is presented in figure 8.9. To evaluate the reinforcement distribution in this cross-section, the effective tensile zone is taken into account. The effective reinforcement ratio is determined by comparing the tensile reinforcement area to the effective concrete area and is calculated in appendix A.13. This calculation yields an effective reinforcement ratio of 1.76%. Since this value remains well below the Eurocode 2 (NEN-EN 1992-1-1+C2, Section 9.2.1.1;Nederlands Normalisatie-instituut (NEN), 2011) limit of 4% and 8%, additional reinforcement can be introduced through post-installed techniques if necessary to ensure proper connections.

8.2.3.4. Overlap length (l_0) of horizontal reinforcement

As with the beams in the Munthof garage, the overlap calculation is conducted to determine the required length for reconnecting the existing reinforcement between two beams with an overlap through a new fixed connection. Therefore, this calculation is performed for the cross-section near the supports, where the dominant reinforcement is located at the top, as can been seen in figure 8.9. This overlap also guarantees proper anchorage of the horizontal reinforcement. The overlap length is calculated using the same formulas, equations (8.4) to (8.7), as those applied in section 8.2.1 and the elements of the Munthof parking garage. The summary of the values and variables used for the beams in Stationsplein 107 is shown in table 8.10 below. The detailed calculation can be find in appendix A.14.

Variable	η_1	η_2	f_{ctd}	ϕ	σ_{sd}	α_1	α_2	α_3	α_5	α_6	$\boldsymbol{f_{bd}}$	$\boldsymbol{l_{b,rqd}}$	$\boldsymbol{l_{0,min}}$	$\boldsymbol{l_0}$
Value	0.7	1	1.2 N/mm^2	24 mm	75.34 N/mm^2	1	1	1	1	1.5	1.89 N/mm^2	239 mm	360 mm	358 mm

Table 8.10: Overview of parameters and results for calculation of the overlap length in the beam section near the support in Stationsplein 107

Since the calculated overlap length l_0 is lower than the minimum overlap length $l_{0,min}$, an overlap length of 360 mm is needed. The same two approaches as discussed for the Munthof garage can be applied. Hydrodemolition can remove 360 mm of existing concrete, allowing the existing reinforcement bars to overlap. However, this would shorten the effective beam length, which may be undesirable. Alternatively, post-installed reinforcement offers a solution to maintain the existing beam length, as the reinforcement ratio allows for additional bars and the required overlap length falls within a practical drillable depth of 600 mm. If the beam is reused in a system with lower loads, the required overlap decreases, reducing the amount of concrete removal or drilling depth. The most suitable approach depends on structural feasibility and design requirements, ensuring proper reinforcement continuity.

8.2.3.5. Anchorage length (l_{bd}) of horizontal reinforcement

The anchorage length at the moment-zero point is also calculated for Stationsplein 107. When a beam cut at these points is reused, proper anchorage of the horizontal reinforcement is essential to ensure effective force transfer at the new hinged supports. This cross section is shown in figure 8.9. The anchorage length is calculated using the same formulas, equations (8.4), (8.5), (8.9) and (8.10), as those applied in section 8.1.3. The summary of the values and results is shown in table 8.11 and the detailed calculation can be found in appendix A.15.

Variable	η_1	η_2	f_{ctd}	ϕ	σ_{sd}	α_1	α_2	α_3	α_4	α_5	$\boldsymbol{f_{bd}}$	$\boldsymbol{l_{b,rqd}}$	$\boldsymbol{l_{b,min}}$	$\boldsymbol{l_{bd}}$
Value	1	1	1.2 N/mm^2	24 mm	19.52 N/mm^2	1	1	1	1	1	2.7 N/mm^2	43 mm	240 mm	43 mm

Table 8.11: Overview of parameters and results for calculation of the anchorage length in the beam section at the moment-zero point in Stationsplein 107

Since the calculated anchorage length l_{bd} is lower than the minimum anchorage length $l_{b,min}$, the required anchorage length is governed by $l_{b,min} = 240$ mm. When a reused beam is hinged-supported in a new structural system, an anchorage length of 240 mm may be insufficient due to limited available space on the column to develop the required bond strength and reinforcement capacity, necessitating an extended anchorage solution, as discussed in section 7.4. To determine which of these solutions are feasible, the spacing is analyzed in the following section.

8.2.3.6. Available space around the horizontal reinforcement bars

The different end-anchorage options discussed in section 7.4 must be evaluated based on the available space to determine which solutions are feasible for this beam. The spacing between the top reinforcement at the end of the beam shown in figure 8.9 is 36.5 mm. With a reinforcement bar diameter of 24 mm, the remaining space for end-anchorage using the threaded disk, as discussed in section 7.4, is insufficient. Similarly, the MBT end-anchorage requires even more space and will also not fit. In this beam with a hight of 760 mm, only the bend-anchorage and the T-heads are feasible for end anchoring the reinforcement.

8.2.4. Floors

For the floors in Stationsplein 107, the challenges related to their reuse, as discussed in the theoretical part of this research, are examined. For a moment-fixed connection, an overlap length (l_0) is required to reconnect the reinforcement. In contrast, for a hinged-supported element, the necessary anchorage length (l_{bd}) must be provided to secure the reinforcement, while also ensuring that the remaining span is sufficient for integration into a new design. Lastly, the available space of the longitudinal reinforcement is analyzed to identify feasible anchorage solutions. Similar to the floors in Munthof, cutting removes the edge reinforcement of the original two-way spanning floor. However, since the reused element spans in only one direction, this lost reinforcement becomes redundant. Consequently, only the anchorage of the longitudinal reinforcement requires consideration, as previously discussed.

The ULS load on the floor of stationsplein 107 is 7.88 kN/m^2 and the span is 5.85 m. The reinforcement layout of most of the floors is shown in figure 8.11.

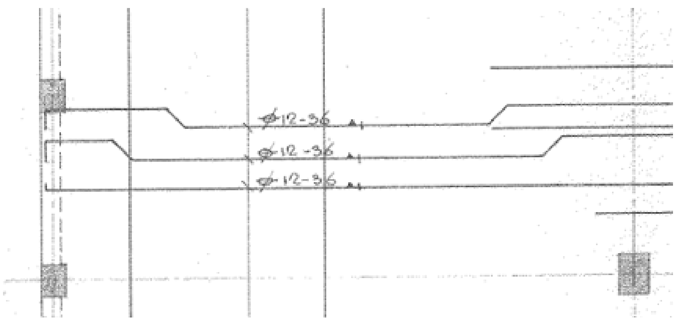


Figure 8.11: Reinforcement layout floor span Stationsplein 107

8.2.4.1. Length between the moment-zero points

Cutting at the moment-zero points is only a viable option if the remaining floor element length aligns with the new design requirements. The analytical derivation of these points in a fixed-fixed slab, as shown in appendix A.3, indicates that they are positioned at approximately $0.221 \cdot l$ and $0.789 \cdot l$. For a floor element with an initial span of 5.85 m, this results in a remaining span of around 3.26 m, which is an uncommon floor span in a new design.

8.2.4.2. Reinforcement ratio in the cross-section

The reinforcement in the floors of Stationsplein 107 consists of ribbed QR 42 bars with $\phi = 12$ mm in the primary span direction. Similar to the beams in this building, the reinforcement in the span direction is not uniformly positioned at the top and bottom but curves in response to the moment distribution. The spacing of the longitudinal reinforcement is 120 mm in the most concentrated section, the thickness of the floor is 150 mm and a 1 m wide element is cut. The floor slab consists of 9 reinforcement bars. The detailed calculation for the reinforcement ratio of a 1 m wide floor element can be found in appendix A.16. The resulting reinforcement ratio is 0.68%, which remains below the specified limits, indicating that post-installed reinforcement can be considered for reconnection.

8.2.4.3. Overlap length (l_0) of longitudinal reinforcement

To determine the overlap length required to reconnect the existing longitudinal reinforcement at the top of the floor for a moment-fixed connection, l_0 must be calculated at the section near by the supports. This has been calculated using the same approach as in the previous sections. By applying equations (8.4) to (8.7) the values in the table below have been determined. The detailed calculation can be found in appendix A.17.

Variable	η_1	η_2	f_{ctd}	ϕ	σ_{sd}	α_1	α_2	α_3	α_5	α_6	$\boldsymbol{f_{bd}}$	$\boldsymbol{l_{b,rqd}}$	$\boldsymbol{l_{0,min}}$	$\boldsymbol{l_0}$
Value	0.7	1	1.2 N/mm^2	12 mm	156 N/mm^2	1	1	1	1	1.5	1.89 N/mm^2	247 mm	200 mm	371 mm

Table 8.12: Overview of parameters and results for calculation of the overlap length of the top reinforcement in the beam in Stationsplein 107 near by the supports

The required overlap length of 371 mm can be achieved using post-installed reinforcement, which remains a feasible option. Alternatively, removing 371 mm of concrete to overlap the existing reinforcement may result in elements that are too short for reuse. If the floor element is reused in a system with lower loads, the required overlap length will decrease, reducing both the need for hydro-demolition or the drilling depth for post-installed reinforcement.

8.2.4.4. Anchorage length ($l_b d$) of longitudinal reinforcement

The required anchorage length (l_{bd}) for the described floor element is calculated in the same way as the sections before, using equations (8.4), (8.5), (8.9) and (8.10). The values are summarized in table 8.13 and the detailed calculation can be found in appendix A.18.

Variable	η_1	η_2	f_{ctd}	ϕ	σ_{sd}	α_1	α_2	α_3	α_4	α_5	$\boldsymbol{f_{bd}}$	$\boldsymbol{l_{b,rqd}}$	$\boldsymbol{l_{b,min}}$	$\boldsymbol{l_{bd}}$
Value	1	1	1.2 N/mm^2	12 mm	14.02 N/mm^2	1	1	1	1	1	2.7 N/mm^2	31 mm	120 mm	31 mm

Table 8.13: Overview of parameters and results for calculation of the anchorage length of the top reinforcement in the floor in Stationsplein 107 at moment zero points

Since the calculated anchorage length l_{bd} is lower than the minimum anchorage length $l_{b,min}$, the required anchorage length is governed by $l_{b,min} = 120$ mm. For floors, anchorage is typically required directly after the support when they are hinged-supported. However, if the required 120 mm anchorage length can be accommodated above the supports, additional anchorage solutions, as described in section 7.4, may not be necessary for this floor element.

8.2.4.5. Available space around the horizontal reinforcement bars

To determine which end-anchorage solution is feasible, it is essential to examine the spacing between the reinforcement bars. The spacing of the reinforcement is 120 mm, which, with a bar diameter of 12 mm, allows for the implementation of T-heads, disk anchors and MBT anchorage. However, with a floor thickness of 150 mm, the required anchorage length exceeds this thickness, making bend-anchorage infeasible.

8.3. Conclusions

This section summarizes the findings for the structural elements analyzed in the case studies.

8.3.1. Columns

For the columns, the existing horizontal reinforcement in the D-region can provide sufficient tensile capacity to resist the force T generated by point loads from two beams hinged on top of the columns. This confirms that additional reinforcement is not necessarily required to strengthen the top of reused columns. However when this capacity is insufficient, the applied loads must be reduced or additional strengthening must be introduced. To optimize the capacity, it is preferable to support beams along the longest column side, maximizing the resistance provided by the horizontal reinforcement.

When it is necessary to connect the vertical reinforcement of the columns, overlap using post-installed reinforcement may be constrained by drillable depth limitations. If the required overlap length exceeds practical limits, alternative reconnection methods, such as mechanical couplers or external strengthening with CFRP or memorysteel, should be considered. However, if the applied loads in the new structural system are significantly lower than in the original design, the required overlap length will decrease. In some cases, the vertical reinforcement may no longer be structurally necessary, eliminating the need for reconnection.

8.3.2. Walls

The removal of edge reinforcement in the walls due to cutting necessitates a solution to ensure proper anchorage for resisting tensile forces. T-heads, MBT couplers and threaded disk anchors are viable due to the available reinforcement spacing in both case studies. Welding reinforcement between adjacent walls also provides an alternative. All these solutions require exposing the bars through hydrodemolition to ensure proper anchorage.

For the vertical reinforcement, overlap using post-installed reinforcement remains feasible, but the available height of the walls influences whether existing reinforcement can be freed up for this overlap. In some cases, minimum height requirements in the new design limit the amount of reinforcement that can be exposed. Additionally, if the applied loads in the new structural system are significantly lower than in the original design, the vertical reinforcement may no longer be structurally necessary, eliminating the need for reconnection. The feasibility of reconnecting vertical reinforcement is therefore highly dependent on the new structural design requirements.

8.3.3. Beams

For the beams, the reinforcement ratio is within the Eurocode 2 (NEN-EN 1992-1-1+C2, Section 9.2.1.1; Nederlands Normalisatie-instituut (NEN), 2011) limits, allowing for the use of post-installed reinforcement. However, in a moment-fixed connection, the required overlap lengths can exceed the feasible drilling depth for post-installed bars, making direct overlap unfeasible without modifying the anchorage of the existing reinforcement.

Beams cut at the moment-zero points result in remaining spans that may not always be suitable for reuse in standard building designs. In some cases, the remaining length is significantly shorter than typical spans, limiting their applicability. Alternatively, reconnecting these beams using a vertical shear joint with a reused T-shaped column is a structurally viable solution, as the shear joint has sufficient strength. To ensure effective force transfer, an appropriate stirrup must be placed at the correct distance to anchor the diagonal shear force. Regarding end-anchorage, the available reinforcement spacing restricts the feasibility of certain solutions. The bend-anchorage and T-heads remain viable, while MBT couplers and disk anchors, may not be applicable due to spacing constraints.

8.3.4. Floors

For floors cut at the moment-zero points, the remaining span must be evaluated for reuse. In some cases, the remaining length is still sufficient for integration into new designs, but in others, it may be too short for standard applications. Using bend-anchorage to anchorage the horizontal reinforcement is not feasible due to the limited floor thickness. However, alternative anchorage solutions, including T-heads, MBT couplers and threaded disk anchors, are feasible within the available reinforcement spacing. In some cases, end-anchorage may not be required at all if the support length is sufficient to accommodate the necessary anchorage length.

The reinforcement ratio in the floors is well below the Eurocode 2 (NEN-EN 1992-1-1+C2, Section 9.2.1.1; Nederlands Normalisatie-instituut (NEN), 2011) limits, making post-installed reinforcement a feasible reconnection method. However, similar to the beams, the required overlap length can exceed practical drillable depths, making overlap connections challenging. If the applied loads in the new structure are lower than in the original design, the required overlap length decreases, potentially improving the feasibility of post-installed reinforcement.

9

Design proposals

To better understand the specific challenges of integrating reused cast-in-situ elements into a new building construction, prefabricated connection details are analyzed in this chapter. The aim is to determine how these connections can be designed with reused elements, identifying which components of traditional design solutions can be utilized and which aspects require alternative approaches. While designing the connections, the goal is to propose simple solutions with minimal element modifications, as concluded from the literature review. The design proposals are developed in accordance with the rules outlined in Eurocode 2 (NEN-EN 1992-1-1+C2; Nederlands Normalisatie-instituut (NEN), 2011). The connections analyzed are schematized in figure 9.1, where the gray elements are new prefabricated concrete elements and the white elements are reused. Through this analysis, the feasibility of each connection type can be evaluated and it becomes possible to determine which solutions are most suitable with reused elements and have the greatest potential for successful implementation in practice.

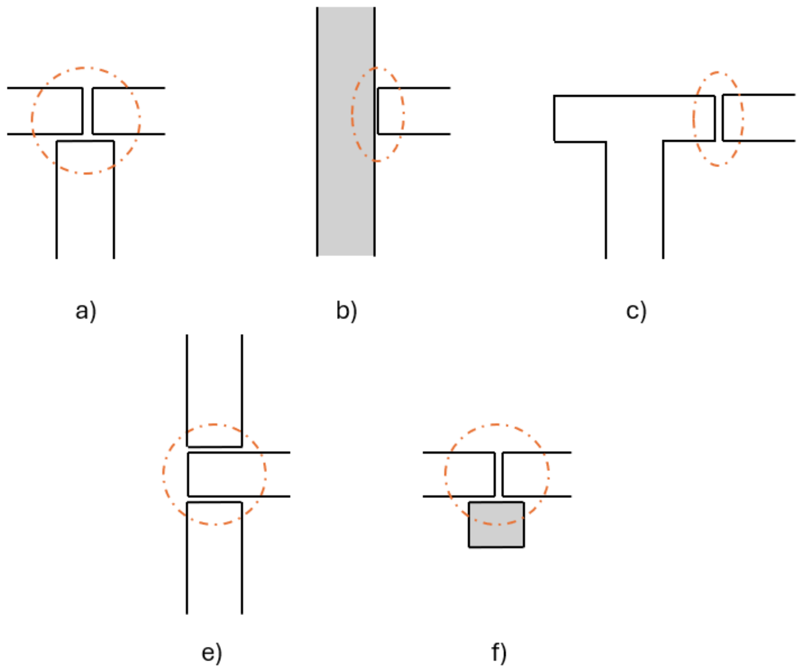


Figure 9.1: Connection schemes to be analyzed, a)double beam-to-column connection, b)Prefab column-beam connection c)T-shaped column-beam connection d)column-foundation connection e)wall-slab connection f)beam-slab connection

9.1. Double beam-to-column connection

In this section, the focus is on developing a double beam-to-column connection, as illustrated in figure 9.1 a). This connection can be classified as either moment-resisting or hinged, each requiring distinct design approaches. A moment-resisting connection must be capable of transferring both bending moments and shear forces, ensuring structural continuity and rigidity. In contrast, a hinged connection is designed to transfer axial and shear forces while allowing rotational flexibility to accommodate structural movements. In the next subsections, both moment-resisting and hinged connections are designed according to the specified requirements.

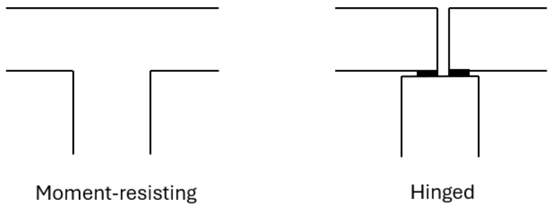


Figure 9.2: Sketch of the two designed double beam-to-column connections.

9.1.1. Moment-resisting connection

In prefabricated systems, a moment-resisting connection is often not implemented. However, when the original full length of the beam from a cast-in-situ fixed system is reused, the dominant top reinforcement at the beam-ends makes it suitable for a fixed connection, as discussed in chapter 8. Thereby preserving a load distribution more similar to the original structural behavior reduces the risk of sudden failure. Therefore, the connection of two full length reused beams, so the cuts are made near by the original fixed supports, with a reused column is developed below.

To create a moment-resisting connection, the column must be considered as an intermediate support. Eurocode 2 (NEN-EN 1992-1-1+C2; Section 10.9.4.5 Nederlands Normalisatie-instituut (NEN) (2011)) specifies that the top reinforcement in the beams must remain continuous across the connection at intermediate supports to ensure effective moment transfer. In chapter 8, it was concluded that overlapping between the existing reinforcement is not practical, as it results in a excessive loss of usable load-bearing concrete. Furthermore, creating an overlap between the existing reinforcement and the post-installed reinforcement, without modifications for end anchorage, may also be unfeasible. However, if anchorage is provided at the ends of the existing bars, the required overlap length would only depend on the anchorage length of the post-installed reinforcement. This length is with the use of high-strength adhesives, specified in section 7.3.1, shorter than max $[15 \cdot \phi;\ 200$ mm]. It was also noted in chapter 8 that, in the case of beams, there is insufficient space to install individual end-anchorage systems for each reinforcement bar. Therefore, it is now proposed to expose the reinforcement at the end of the beam using hydrojetting and weld a single steel plate to all the exposed bars. This process wouldn't take place on the construction site but rather at the storage location where the beams are kept before being incorporated into the new design, ensuring better control over execution and quality of the welds. Figure 9.3 shows the solution of end-anchorage for the beams. To ensure proper force transfer, the welded steel plate must be anchored in newly poured concrete, as explained in section 7.4.

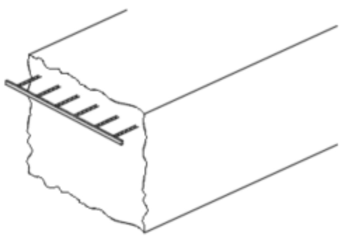


Figure 9.3: End-anchorage solution for fixed reconnections

There are three possible options for connecting the existing top reinforcement of the beams. In the first case, a post-installed bar is drilled into both beams to achieve the connection. The distance between the existing reinforcement and the post-installed bar must comply with the limits specified in Eurocode 2 (NEN-EN 1992-1-1+C2, Section 8.7.2; Nederlands Normalisatie-instituut (NEN), 2011), which are min[$4 \cdot \phi$; 50 mm]. The post-installed bars must be connected using one of the mechanical couplers discussed in section 7.2, as there is insufficient space to accommodate an overlap between the post-installed bars at the connection. Furthermore, Eurocode 2 (NEN-EN 1992-1-1+C2, Section 9.2.1.5; Nederlands Normalisatie-instituut (NEN), 2011) requires that the bottom reinforcement must have an overlap length (l_0) equal to the anchorage length (l_{bd}) to accommodate potential positive moments caused by explosions, ensuring the robustness of the connection. The same method as with the top reinforcement can be applied to the bottom reinforcement, ensuring compliance with all design requirements. If no bottom reinforcement is present at the end of the beam, robustness must be ensured elsewhere in the new design. A schematic representation of this solution is shown in figure 9.4.

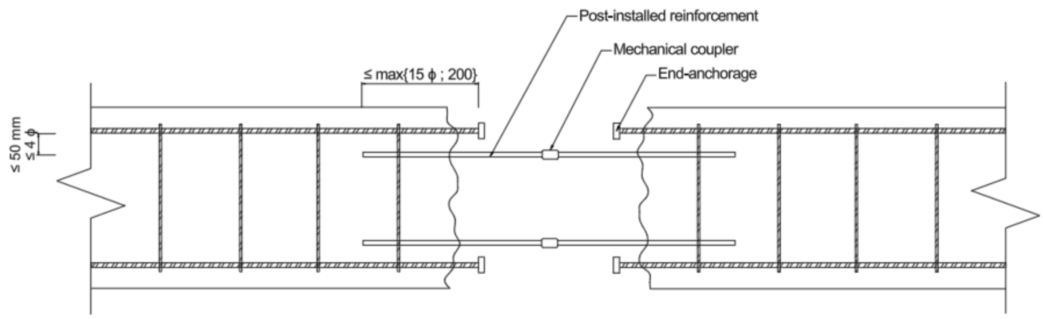


Figure 9.4: Reinforcement reconnecting proposal with post-installed reinforcement

Another option to connect the existing reinforcement of the beams is to use mechanical couplers for direct connection. To preserve as much beam length as possible, it can be beneficial to insert a short intermediate reinforcement bar between the two beams. Since the reinforcement is directly connected, its anchorage is also ensured. When this method is applied to the top reinforcement, it can similarly be implemented for the bottom reinforcement, if present, thereby simultaneously fulfilling both requirements, as shown in figure 9.5.

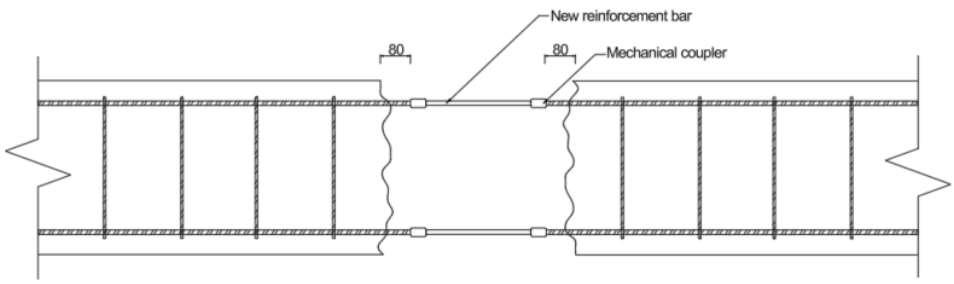


Figure 9.5: Reinforcement reconnecting proposal with direct mechanical couplers

In section 7.3, external overlapping with existing reinforcement was discussed as an alternative approach for reconnecting existing reinforcement by an overlap. This method can also be applied for reconnecting these beams in a fixed connection. The distance between overlapping reinforcement bars is ideally not more than 50 mm, specified in Eurocode 2 (NEN-EN 1992-1-1+C2, Section 8.7.2; Nederlands Normalisatie-instituut (NEN), 2011). With the use of external strengthening, this implies that the concrete cover should preferably be less than 50 mm to minimize the distance to the existing reinforcement, ensuring effective force transfer. Existing concrete structures often have a lower concrete cover, as this was permitted under previous design standards.

However, if the concrete cover exceeds 50 mm, the overlap length should be increased by a length equal to the spacing where it exceeds 50 mm. When the anchorage of the existing reinforcement is ensured like proposed in figure 9.3, the overlap length will only depend on the anchorage of the external reinforcement and whether the concrete cover exceeds 50 mm or not. By reducing the dependency on the overlap length of the existing reinforcement, this approach minimizes the need for large amounts of expensive external reinforcement materials. CFRP lamellas or memory steel is easily applicable to the top of the beams due to the available space. In contrast, for the bottom reinforcement, the presence of the column below restricts the space available for creating an external overlap necessary to ensure continuous reinforcement. This requirement is set by Eurocode 2 (NEN-EN 1992-1-1+C2, Section 9.2.1.5; Nederlands Normalisatie-instituut (NEN), 2011) to ensure robustness. It is possible to achieve this robustness outside the connection within the building.

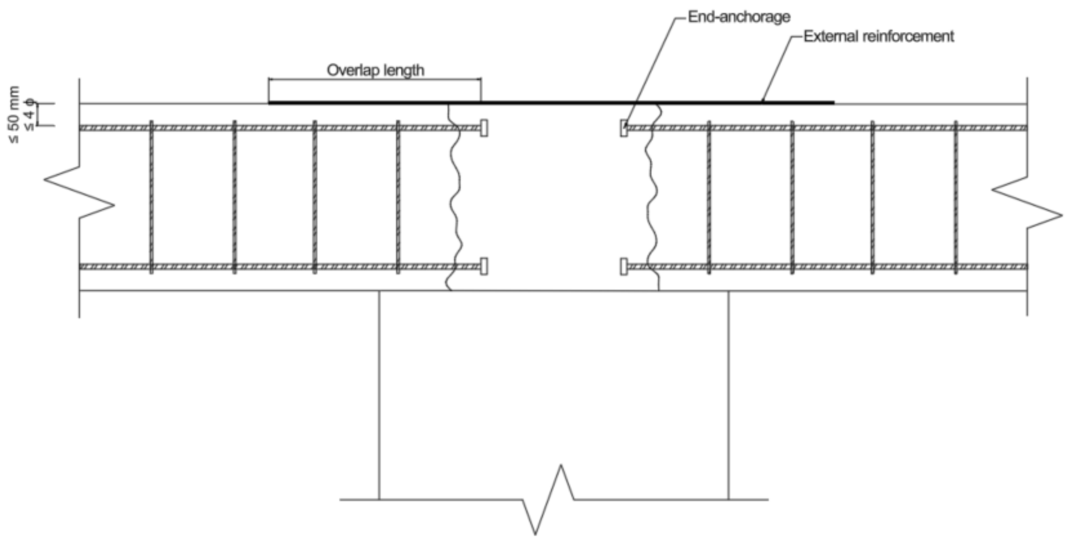


Figure 9.6: Reinforcement reconnecting proposal with external strengthening

Among the three proposed solutions, the second option using mechanical couplers for direct connection is identified as the most feasible and effective solution. This method minimizes the number of additional components required, thereby simplifying the construction process and reducing potential vulnerabilities within the connection. Furthermore, the direct connection of reinforcement ensures complete force transfer, mitigating uncertainties associated with anchorage or overlap effectiveness. Consequently, this approach provides both structural robustness and efficient execution, making it the preferred solution where applicable.

In addition to ensuring the continuity of the longitudinal reinforcement for moment transfer, it is also crucial to ensure the effective transfer of shear forces within the connection. In the reused beams, shear reinforcement is still present, enabling the force to travel vertically upward and then downward through a concrete compression diagonal. The reconnection between the two reused beams and column must allow the shear force to follow this path effectively. From chapter 8, it was concluded that the shear joint between the reused beam and the newly poured concrete is strong enough to allow the diagonal force to pass through. It was also determined that stirrups are required in the newly poured concrete to anchor this diagonal force. However, since the shear joint is supported by a column, additional stirrups are unnecessary, as the compressive diagonal can directly be transferred to the column. Although chapter 8 confirms that the existing horizontal reinforcement in the column can resist the tensile forces from non-uniform load transfer, maintaining a uniform force distribution remains beneficial. This approach helps preserve the original load distribution, reducing the risk of stress concentrations. A layer of mortar is applied on top of the column to distribute forces evenly, compensate for surface irregularities and improve adhesion. Additionally, post-installed anchors extend from the column into the newly poured concrete, providing anchorage and enhancing connection stability for a moment-resisting connection. The design proposal for the described moment-resisting reconnection is shown in figure 9.7.

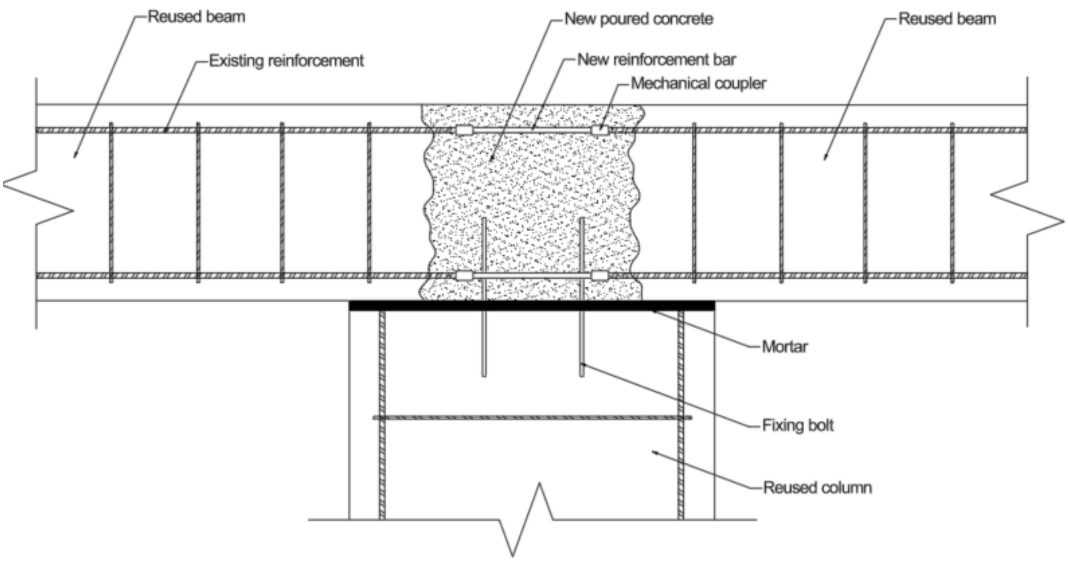


Figure 9.7: Design proposal moment-resisting double beam-to-column reconnection

Although moment-resisting connections are not commonly used in prefabricated structures due to their complexity and additional design requirements, this approach provides a significant advantage when reused beams are cut near their original fixed supports. By maintaining the original loading conditions, this solution ensures that the forces remain aligned with the initial design, thereby minimizing the risk of unexpected structural behavior and maximizing the reuse potential of the elements. Moreover, the existing reinforcement layout is already designed to accommodate these moment-fixed conditions, further supporting the feasibility of this approach.

9.1.2. Hinged connection

In contrast to moment-resisting connections, hinged connections are more commonly used in prefabricated structures due to their simplicity and cost efficiency. These connections transfer axial and shear forces while allowing rotational freedom. However, when reusing beams, a hinged connection can only be achieved if the element is cut at the moment-zero points of the originally fixed beam. Cutting near the original supports would result in the absence of dominant bottom reinforcement at the beam-end, which is essential for resisting the positive moment in a hinged support configuration. In this section, the hinged connection with reused beams on a column is developed.

As mentioned earlier in chapter 8, the existing horizontal reinforcement in reused columns can provide sufficient capacity to resist the tensile force resulting from two point loads at the top of the column in a realistic scenario. This solution assumes that the applied external loads on the beam are selected in such a way that the column has sufficient capacity to withstand the resulting tensile force with the current reinforcement. The bearing pad between the beams and column must be located at a certain distance of the column edge to prevent for spalling as discussed in chapter 6. Tests have shown that modified mortar material enhanced with polymers and fibers effectively distributes stresses and accommodates small rotations between elements, which is important for hinged connections (El Debs et al., 2003). In chapter 8, it was concluded that end-anchorage is required for connecting reused beams in a hinged configuration. Here too, this is addressed by welding a single steel plate to all the exposed bars, as described in section 9.1.1 and figure 9.3. An in-situ infill is not used in a hinged connection, as it would restrict the freedom of rotation and limit horizontal deformation at the support. Therefore, it is proposed to cast a concrete extension using a mold to embed the end-anchorage in the new concrete, as shown in figure 9.8.

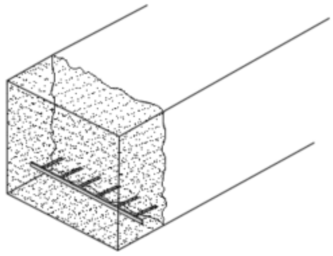


Figure 9.8: End-anchorage solution for hinged reconnections

To transfer the shear force from the beam to the column in a hinged connection, it is important that the compression diagonal aligns with the center of the load bearing area. The distance lh from the existing stirrup to the center of the support has a minimum and maximum value, as defined in appendix A.20 and equation (9.1). In figure 9.9 the design proposal of the hinged reconnection between two reused beams and a column is shown.

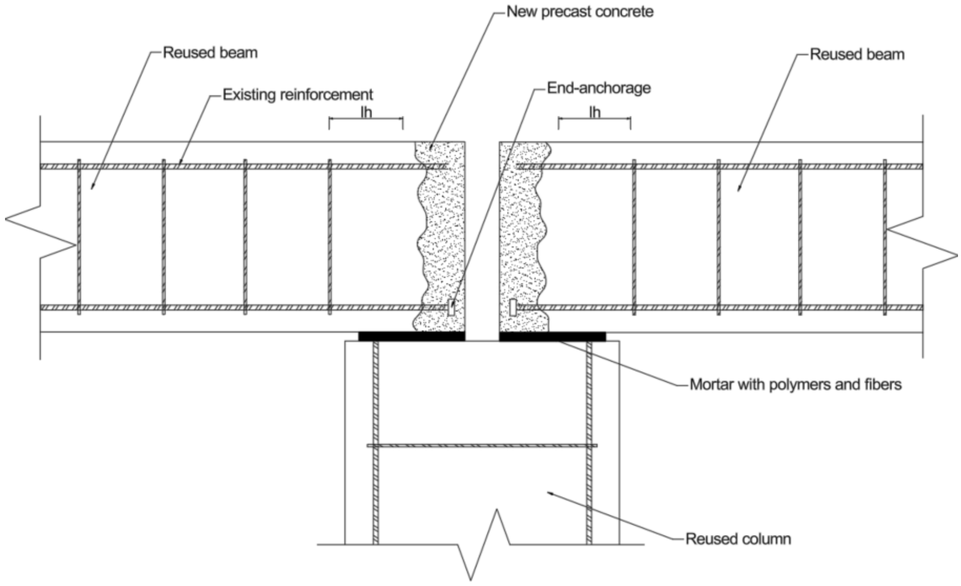


Figure 9.9: Design proposal hinged double beam-to-column connection

This proposed hinged connection is a practical and feasible solution for reusing structural beams and columns, allowing for effective vertical transfer while accommodating the necessary rotation and deformation at the hinged support. If horizontal force transfer is required, for example, when the elements contribute to the secondary stability system as discussed in section 6.3.3, additional measures can be implemented to ensure sufficient horizontal resistance. In this case, the preferred solution is the implementation of external angle brackets, which provide horizontal stability. The proposed solution is shown in figure 9.10. The angle brackets, which contain slotted bolt holes, are introduced to allow the connection to transfer limited horizontal forces, providing additional stability in scenarios where such force transfer is required. According to standards in NEN-EN 1090-2, normal clearance holes should be 1 to 2 mm larger than the bolt diameter. For increased movement, over-sized holes of 3 to 6 mm larger than the bolt diameter or slotted holes are recommended, depending on the degree of rotation required. Additionally, these slotted holes ensure the proper fixation of the beams to the column by accommodating minor shifts or adjustments during assembly, resulting in a secure and reliable connection between the reused beams and the column.

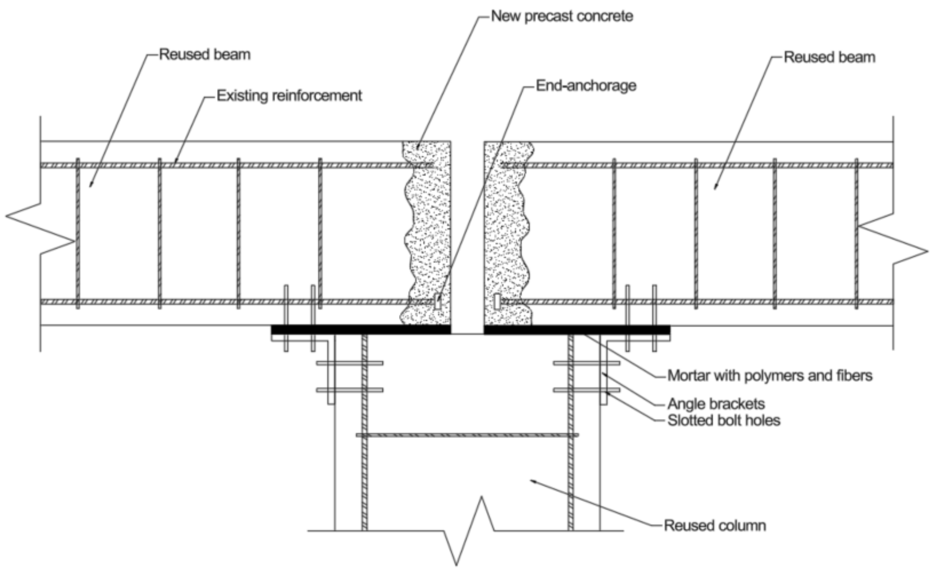


Figure 9.10: Design proposal hinged double beam-to-column connection with stability assurance

The traditional method of connecting prefabricated beams to a column with a bolt extending from the column into a prefabricated sleeve cast in the beam is not feasible for reused beams. These elements lack the necessary embedded provisions. In contrast, when connecting a reused column to newly fabricated beams, the bolt can be post-installed in the reused column, while the embedded provisions can be incorporated into the newly cast beam. As a result, this approach may be implemented more quickly compared to the hinged connection involving both a reused beam and column, given the challenges and limitations discussed in section 2.5.

9.2. Prefab column-beam connection

In this section, the focus is on developing a prefabricated column-beam connection, as illustrated in figure 9.1 b). In prefabricated construction systems, continuous column-to-beam connections are typically executed using three primary methods, each with distinct structural and practical considerations. These options are presented in figure 9.11 and analyzed based on their feasibility, constructability and structural performance when combining a prefabricated continuous column with a reused beam.

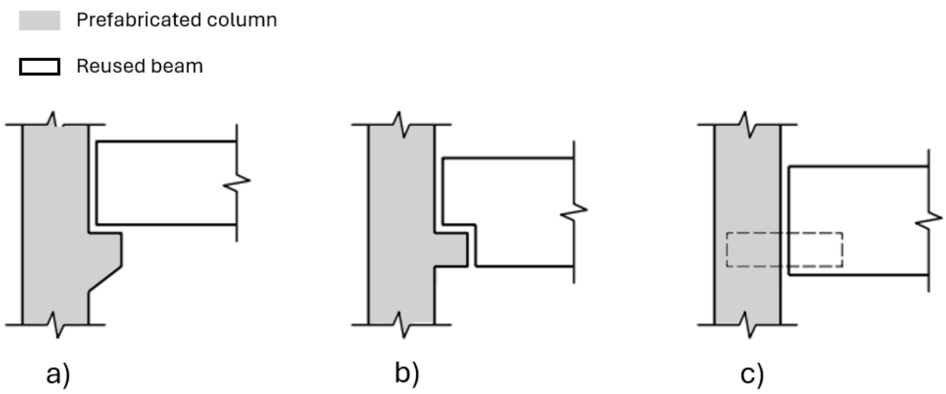


Figure 9.11: Schematic representation of the prefabricated column-beam solutions, a) corbel connection, b) toothed connection, c) steel anchoring connection

9.2.1. Corbel connection

In a corbel connection, the concept of shear force transfer is similar to the hinged connection in figure 9.9. The distance between the last stirrup in the reused beam and the center of the support is referred to lh, as shown in appendix A.20. The choice of bearing material is again modified mortar material enhanced with polymers and fibers, which accommodates small rotations between elements, an important feature in hinged connections.

For reused beams, the corbel must be designed larger than in connections with new beams. This requirement arises from the additional section of newly poured concrete at the beam's end, which is necessary to accommodate the anchorage of the longitudinal reinforcement, as discussed in figure 9.8. To ensure effective load transfer and preserve the structural integrity of the connection, a significant part of the support must be positioned beneath the original concrete of the beam.

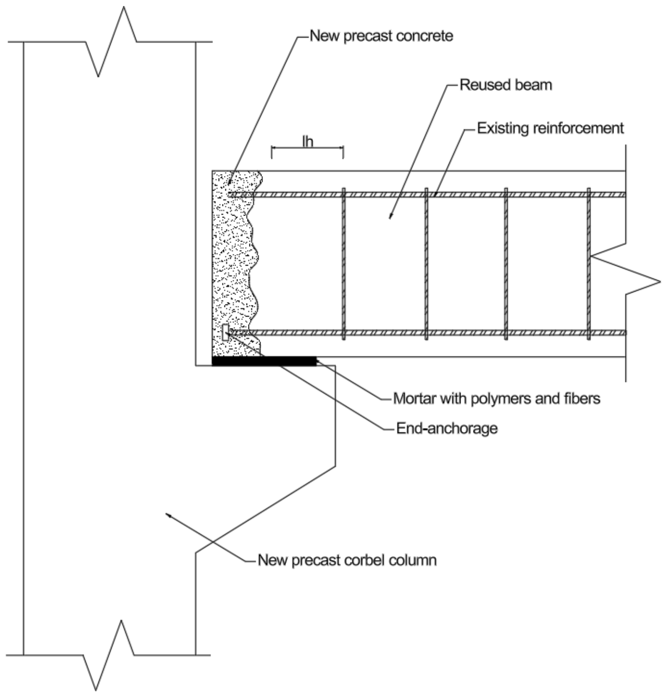


Figure 9.12: Design proposal of the corbel connection with reused beam

If horizontal force transfer is required, when the elements contribute to the building's secondary stability system, this can be achieved by installing a post-installed reinforcement bar into the reused beam. This reinforcement bar is embedded within the beam and extends into a prefabricated hole in the new precast corbel, where it is secured by bolting it at the bottom. To accommodate construction tolerances and minor structural deformations, the hole in the corbel is designed to be slightly larger than the bar itself. This ensures that the connection can accommodate limited movement and minor rotations, which are essential for a hinged connection to function properly, while still allowing for the transfer of limited horizontal forces when required. Structurally, this approach closely resembles the traditional method of bolted connections but with a reversed configuration, where the bolt is embedded in the beam instead of being anchored in the corbel. The proposed solution is shown in figure 9.13 at page 70.

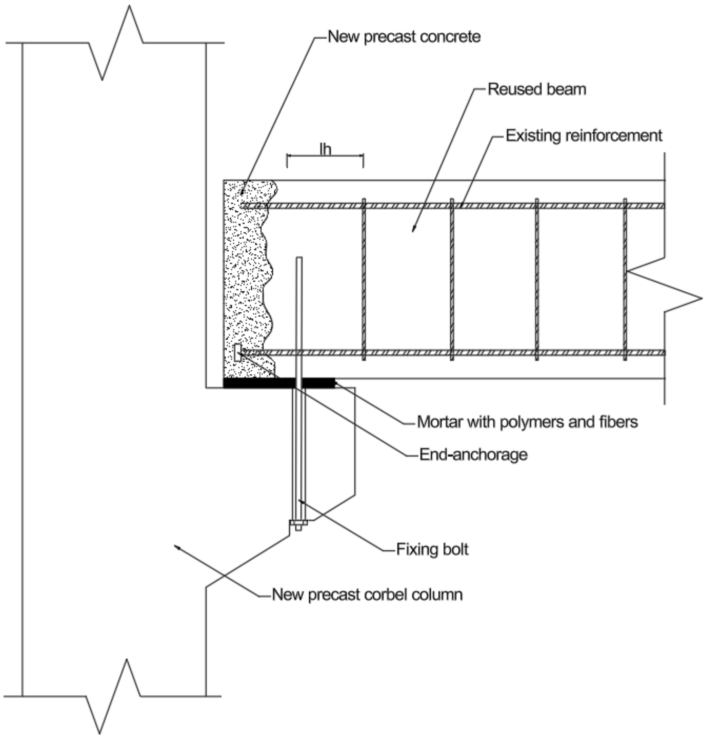


Figure 9.13: Design proposal of the corbel connection with stability assurance

The corbel connection with a reused beam element provides a practical and feasible solution for reusing beams in new structural buildings. The approach closely aligns with existing connection techniques, ensuring familiarity with the design and its implementation.

9.2.2. Toothed connection

To create a toothed connection, the reused beam will need to be modified. It is possible to create the toothed shape using hydrojetting (Aggregate Technologies, 2024). The exposed reinforcement can be trimmed so that a small portion remains, allowing it to be anchored as discussed in figure 9.8. Figure 9.14 provides a schematic representation of the reinforcement that could be present in this configuration.

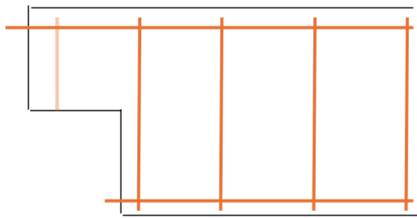


Figure 9.14: Reused tooth shape sketch of the existing reinforcement

To effectively transfer the shear force through the support, specially designed reinforcement is required. Figure 9.15 illustrates two strut-and-tie models that represent the load transfer mechanisms within the support region. These models highlight the paths for compressive and tensile forces, ensuring an efficient and safe transfer of shear forces in the structure (Wijte et al., 2010).

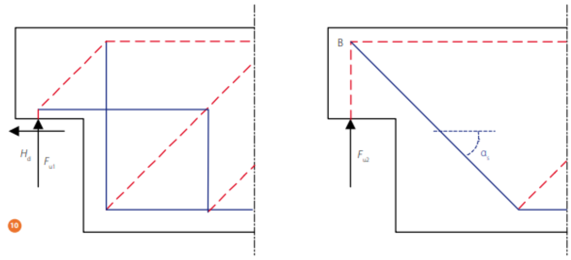


Figure 9.15: Strut and tie models to design the reinforcement in a tooth joint (Wijte et al., 2010)

To ensure that the shear force is effectively transferred to the support in the absence of the required internal reinforcement, alternative solutions such as post-installed reinforcement, CFRP lammellas or memory steel must be considered. This solution is significantly more complex and labor-intensive compared to the corbel connection, making it unlikely to be implemented in practice. However, certain beams, such as those in Stationsplein 107, contain diagonal reinforcement that could be utilized for this type of connection using an appropriate strut-and-tie model. If the existing beam contains favorable reinforcement for the strut-and-tie models required in this design, this connection method could still be considered. However, since this research focuses on identifying the most efficient connection methods in general, this option is excluded due to its impracticality application.

9.2.3. Steel anchoring connection

In prefabricated systems, anchored steel elements are cast into the column during its construction. These are then connected to fastening provisions that are pre-cast into the beam. However, when the beam is a reused element, these provisions are absent, requiring an alternative solution to establish a reliable connection. Adapting a reused beam to connect with cast-in fasteners in a column is impractical due to several challenges. Connecting the existing reinforcement to the cast-in fasteners is not feasible, as these reinforcement bars cannot be screwed or bolted. Achieving such a connection would require extensive modifications to the beam, such as creating slots for steel plates or installing beam shoes to allow the beam to be bolted to the column. These modifications are labor-intensive and contradict the goal of designing simple, practical connections, ultimately making this approach inefficient and impractical for most applications.

9.3. T-shaped column-beam connection

In this section, the focus is on analyzing a T-shaped column-beam connection, as illustrated in figure 9.1 c). This connection is located at the moment zero point of the originally clamped beam. Therefore, the connection is only required to transmit shear force. Consequently, the reused beam must be cut at its moment zero points to ensure proper alignment of the reinforcement layout with the reconnection, thereby maintaining a similar force distribution. In prefabricated systems, this connection is often executed using a toothed joint or by means of mechanical connectors such as beam shoes or steel brackets. However, as discussed in sections 9.2.2 and 9.2.3 both these solutions are too complex and labor-intensive for practical implementation. Therefore, a monolithic connection remains the only feasible option.

To achieve a monolithic connection between the beam and the T-shaped column, the reinforcement must be connected and the shear force must be effectively transferred through the connection. The reinforcement connection can be implemented in the same way as described in figure 9.5, where the reinforcement is directly connected using mechanical couplers and an intermediate reinforcement segment. Since in this case the connection is not supported by a column, stirrups in the newly poured concrete between the beam and the T-shaped column will be required to anchor the compressive diagonal. Eurocode 2 (NEN-EN 1992-1-1+C2, Section 6.2.3; Nederlands Normalisatie-instituut (NEN) (2011)) specifies a minimum and maximum angle for the compression diagonal, which simultaneously imposes limits on the spacing of the stirrups.

For the design of the connection, it is important that the new placed stirrups in the newly poured concrete between the reused beams satisfy the following requirement:

$$l_{h,\text{min}} - z \leq x \leq l_{h,\text{max}} - z \tag{9.1}$$

where $l_{h,\text{min}}$ and $l_{h,\text{max}}$ are calculated based on the limits of the compression diagonal angle, as detailed in appendix A.20, z is the offset caused by the existing position of the stirrup and x is the effective placement of the stirrups, as showed in figure 9.16. By ensuring the correct placement of stirrups, connecting the reinforcement and achieving sufficient bond strength between the existing and newly poured concrete, s concluded to be achievable in chapter 8, the T-shaped connection can effectively transfer the shear forces while maintaining structural integrity.

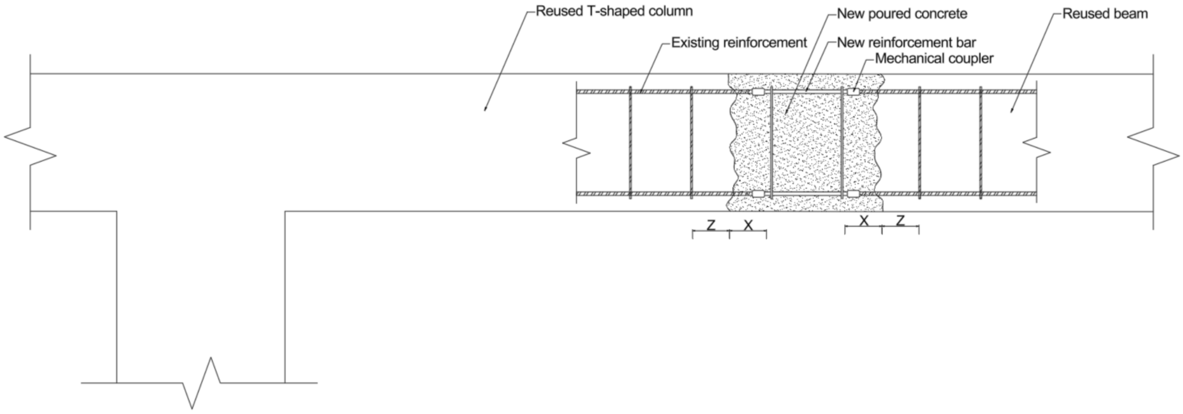


Figure 9.16: Design proposal of the reused T-shaped column-beam connection

By maintaining this connection, the structural behavior of the elements remains consistent with their original design and stability is preserved through the strong reused moment-resisting connection between the beam and column, making it a practical and efficient solution for reusing concrete elements.

9.4. Wall-slab connection

In this section, the focus is on analyzing the wall-slab connection, as illustrated in figure 9.1 d). The goal is to explore how a reused monolitic floor element can be connected to reused structural walls. While the reuse of wide-slab elements is certainly feasible, as described in section 4.1.2, connection designs for these elements have already been developed by Volkov (2019) shown in figure 2.4 and are therefore not included in this study.

For the design of the wall-slab connection, the findings from the case studies are applied. It was concluded in chapter 8 that when the load on the walls is reduced compared to the original design, the vertical reinforcement does not need to be connected, simplifying the connection. Additionally, it was determined that there is sufficient space to individually anchor the longitudinal reinforcement in floor elements. Except for the bent anchorage, the T-heads, MBT mechanical anchorage and the threaded disk, discussed in section 7.4, are viable alternatives for implementation. These insights form the basis for the analysis and design of practical wall-slab connections discussed in this section.

To minimize modifications to the elements and ensure optimal force transfer, the design assumes that the load on the walls in the new structure is significantly lower than in the original design, eliminating the need for post-installed reinforcement to connect the walls. The most practical and effective solution for the wall-slab reconnection is the one represented in figure 9.17. The anchorage of the reconnection follows the same design approach as illustrated in figure 9.3, in which a single plate is welded to the reinforcement ends. This anchorage simplifies the anchorage process compared to using multiple individual anchorage points an meets the Eurocode 2 (NEN-EN 1992-1-1+C2, Section 9.2.1.4; Nederlands Normalisatie-instituut (NEN) (2011)) requirement for anchoring above an fixed end-support, as it provides immediate anchorage through the end-anchorage system.

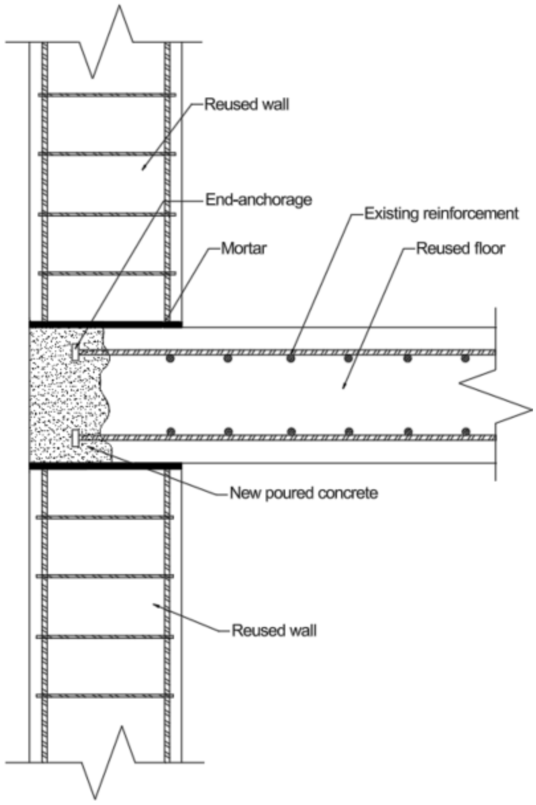


Figure 9.17: Design proposal of the wall-slab connection

In this reconnection proposal, the floor is considered as clamped, maintaining the same load distribution of a floor element cut near the original supports. The weight from the upper wall contributes to the frictional capacity of this connection, which is assumed to be sufficient to transfer horizontal forces. This frictional force provides additional stability to the design, ensuring the connection effectively supports the overall structural system. With a reused wall-slab connection, a hinged connection is not a viable option. Similar to section 9.2.3, too many modifications would need to be made to the floor and wall elements, making it neither a straightforward nor efficient method for reuse.

9.5. Beam-wall connection

The focus in this section is on the structural integration of reused floor elements with a new prefabricated beam, as illustrated in figure 9.1 e). The first analysis examines a fixed connection. When floor elements are cut near their supports, a fixed connection preserves the force distribution. Moreover, the reinforcement layout aligns with fixed connection requirements.

In a floor, forces are more evenly distributed compared to a beam. However, for a moment-resisting connection, all top reinforcement bars must be properly connected. Overlapping the existing reinforcement with post-installed bars and mechanically connecting them provides an efficient and practical solution for achieving a moment-resisting connection between a reused floor and a new beam. In chapter 8 it was concluded that overlapping with post-installed and the existing reinforcement can be not feasible when no modifications for anchorage are made to the existing bars. Therefore, Theads, MBT mechanical anchorage or the threaded disk can be used, as previously concluded to be feasible for floors in chapter 8. Additionally, the same method described in figure 9.8, involving welding a single plate to the reinforcement ends, can also be used to accelerate the process. The top of the prefabricated beam is cast later to anchor the post-installed reinforcement. The in-situ infill ensures monolithic behavior, ensured by the rough surface from hydrodemolition. This design is shown in figure 9.18 at page 74.

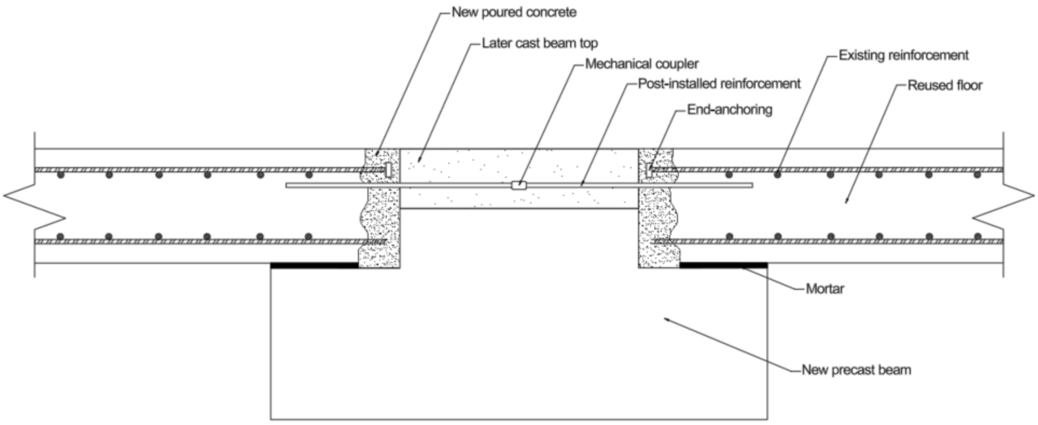


Figure 9.18: Design proposal of a fixed beam-slab connection option 1

Another option is to directly connect the existing and post-installed reinforcement bars using mechanical couplers, shown in figure 9.19. This approach ensures full force transfer and continuity of the reinforcement without requiring an overlap length. By eliminating the need for post-installed anchorage and end-anchorage, this method simplifies the reconnection while maintaining direct structural integrity of the moment-resisting connection.

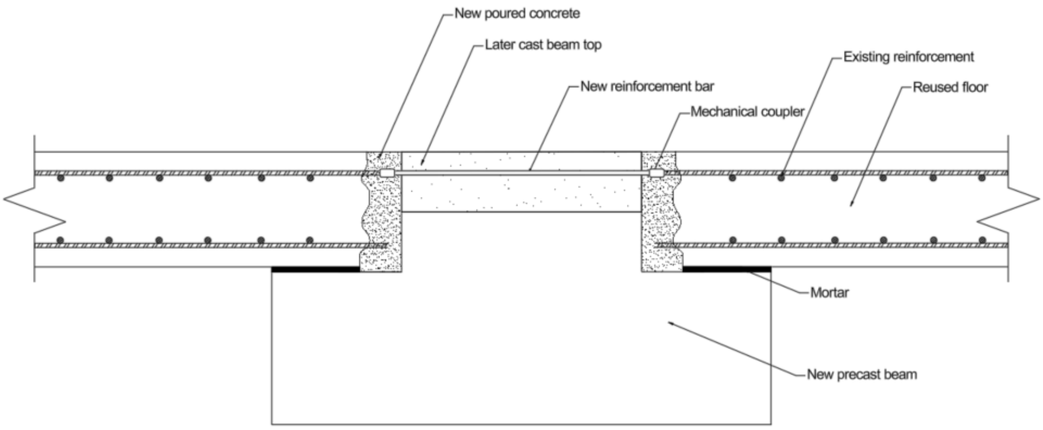


Figure 9.19: Design proposal of a fixed beam-slab connection option 2

Comparable connection details are commonly implemented in practice. Based on prior experience, cracking in such connections is often unavoidable. Consequently, it is advisable to integrate flexible materials to form expansion joints within the connection, thereby accommodating potential movements and mitigating the impact of cracking.

Analyzing a hinged connection provides a simpler, more cost-efficient solution, which is commonly used in prefabricated systems. However, the reused floor element must have adequate bottom reinforcement, meaning it should be cut at the moment-zero points. In some cases, this may not be feasible, as the remaining length could be too short for a viable new design. In chapter 8, it was concluded that no additional end-anchorage is required, as a sufficiently large support can be created to accommodate the anchorage. The bearing area on the prefabricated beam should be at least equal to the minimum required anchorage length of $10 \cdot \phi$. A high-quality bearing material, such as mortar with polymers and fibers, should be used to ensure effective load distribution and prevent stress concentrations. This approach allows the vertical forces from the floor element to be reliably transferred to the beam. The proposed design is shown in figure 9.20 at page 75.

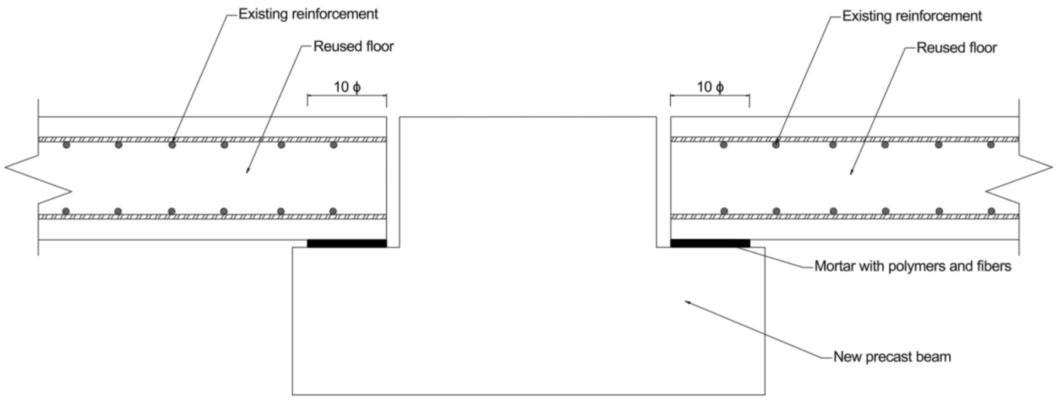


Figure 9.20: Design proposal of a hinged beam-slab connection

If diaphragm action is required and friction is insufficient for horizontal force transfer, additional measures are needed. One option is to install mechanical connectors, such as dowels, bolts, or steel brackets, as shown in figures 9.21 and 9.22. Alternatively, a cast-in-situ compression layer can enhance stiffness, strength and continuous force transfer, effectively unifying the elements into a single structural system.

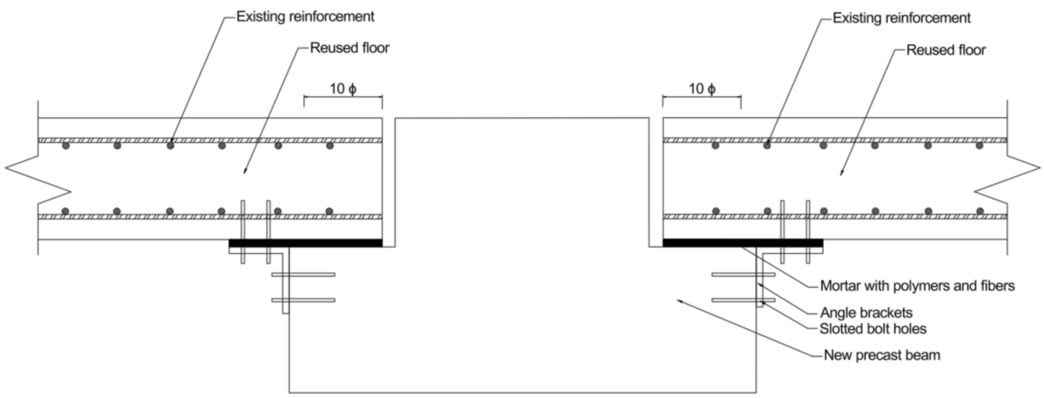


Figure 9.21: Design proposal of the beam-slab connection with stability assurance by angle brackets

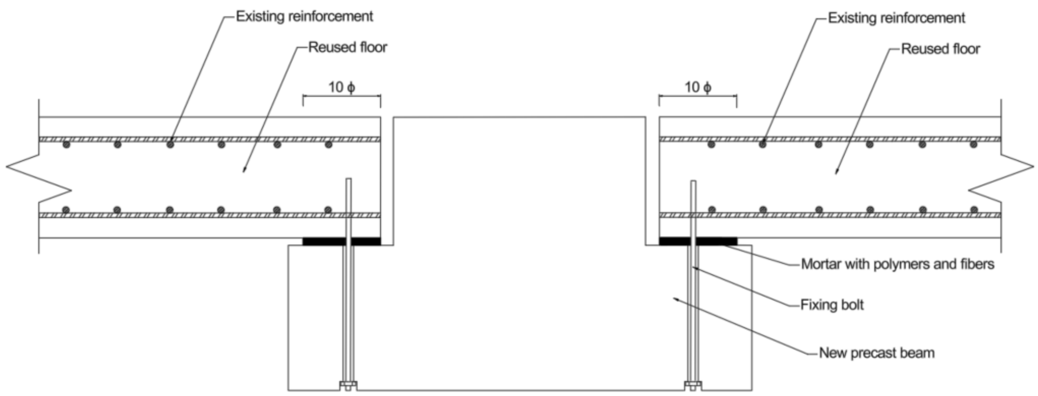


Figure 9.22: Design proposal of the beam-slab connection with stability assurance by bolts

9.6. Multi-criteria analysis

This chapter has shown that cast-in-situ concrete elements can be successfully reused in new prefabricated systems through well-designed connection solutions. While each connection offers potential for reuse, their feasibility varies in terms of required modifications, constructability and overdimendioning. To systematically assess the proposed reconnection methods, a multi-criteria analysis (MCA) is conducted, comparing their feasibility across these factors. By scoring these criteria, the analysis identifies the most practical and efficient solutions, which should be prioritized for initial implementation to facilitate the adoption of reuse in the construction industry. The MCA is based on five criteria:

1. **Element modification;** the extent of modifications required to the elements before installation on-site.
2. **Standard techniques;** whether conventional prefabricated construction methods can be used.
3. **Construction effort;** the amount of work and steps required to assemble the connection on-site.
4. **Overdimensioning;** assessing to what extent the original element's dimensions and structural capacity exceed the requirements of the new design.
5. **Required precision;** the tolerance level needed for a successful placement.

The weighting of these criteria reflects the focus of this study: identifying which load-bearing concrete elements from cast-in-situ existing structures are most suitable for reuse. As highlighted in the literature review, practical feasibility is crucial for successfully implementing reuse strategies in the construction industry. If a reconnection method is too complex, requires a lot of modifications, or does not fit common construction techniques, its applicability remains limited. Additionally, overdimensioning can result in inefficient reuse of materials and increased structural weight. While overdimensioning alone does not prevent reuse, it affects the practicality of certain reconnection methods. The assigned weights are as follows:

1. Element modification **25 %**; The extent of modifications required before installation directly affects feasibility. If many modifications are needed, the process becomes complex and impractical, making it unlikely that this reuse approach will be adopted in practice. Since excessive modifications must be addressed before elements can be installed, this factor is one of the most critical in determining feasibility.
2. Standard techniques **25 %**; Ensuring that reconnections can be executed using well-established connection methods improves the potential for industry adoption. If a solution requires highly specialized or unfamiliar techniques, its applicability is significantly reduced, even if it performs well structurally. Since constructability plays a crucial role in reuse, this criterion is weighted equally to element modifications.
3. Construction effort **20 %**; The amount of on-site work directly impacts feasibility. Reducing labor-intensive assembly steps increases efficiency. However, construction effort is slightly less critical than element modifications and well-established connection techniques, as on-site labor can often be optimized through planning.
4. Overdimensioning **10 %**; High overdimensioning lead to inefficient reuse of materials, increased weight and unnecessary structural capacity, which can complicate handling and integration. However, since it does not directly determine feasibility, reuse remains possible even if an element is overdimensioned. This factor is therefore assigned a lower weight.
5. Required precision **20 %**; High tolerance requirements can complicate execution. Ensuring that the connection allows for reasonable construction tolerances is important for practical feasibility. Unlike construction effort, which concerns the amount of labor required, precision requirements determine the level of accuracy needed for a successful placement of the elements. If tolerances are too strict, even well-planned construction processes may lead to misalignment issues, affecting the structural integrity and applicability of the reconnection method.

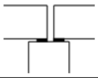
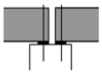
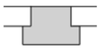
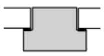
Connection type		Element modification	Standard techniques	Construction effort	Over-dimensioning	Required precision	Score
	Fixed reused column - two reused beams	Medium	No	High	Low	High	3.2
	Hinged reused column - two reused beams	High	Yes	Low	Substantial	Medium	2.15
	Hinged reused column - two new beams	Low	Yes	Medium	Low	Medium	1.4
	New column corbel - reused beam	High	Yes	Medium	Substantial	Medium	2.35
	Reused T-shaped column - reused beam	Medium	No	High	Low	High	3.2
	Reused walls - reused slab	Substantial	Yes	Substantial	Medium	Low	2
	Fixed new beam - reused slabs	Medium	No	High	Low	High	3.2
	Hinged new beam - reused slabs	Low	Yes	Low	Medium	Medium	1.3

Table 9.1: Multi-criteria analysis of the proposed connections

All criteria, except standard connection techniques, are assessed using a gradual scale of low, medium, substantial and high, where lower scores indicate a more favorable outcome. For standard connection techniques, a simple Yes/No system was used, with yes being the preferred option. The final scores for each connection range from 1 to 4, with 1 representing the most favorable overall outcome and 4 the least favorable. A more detailed explanation of the scoring methodology and the reasoning behind each score can be found in appendix B. The full scoring breakdown is shown in table 9.1.

The MCA results indicate that the lowest scores correspond to the most favorable reconnection solutions, making them the most feasible for implementation in the construction industry. The best-performing connection is the hinged reused floor slab connected with a new prefabricated beam. Additionally, the hinged reused column connected with new prefabricated beams also receives a very low score. Both solutions combine reused and new elements while utilizing standard construction techniques, making them highly practical and efficient. As a result, these solutions are the most likely to be adopted quickly in practice. These findings directly address sub-question 3, which explores suitable existing and innovative connection details for ensuring the structural integrity of reused load-bearing concrete elements from cast-in-situ structures.

10 Conclusion

This research has demonstrated that cast-in-situ concrete elements can be effectively reused in new building structures when appropriate reconnection solutions are applied. The findings contribute to more sustainable construction practices by addressing the technical and practical challenges of integrating reused components. As the literature review has shown, the reuse of concrete elements remains limited due to the lack of standardized reconnection methods, risk-averse behavior in the construction industry, restrictive regulations and limited expertise. Given these constraints, this research focused on developing practical reconnection methods that prioritize simplicity, feasibility and alignment with traditional construction techniques while minimizing modifications to the original elements, allowing the reuse of concrete elements to be effectively implemented in real-world construction projects. To address these challenges, the study was guided by the following main research question:

Which load-bearing concrete elements from cast-in-situ existing structures are most suitable for reuse and how can their reconnection details be designed to enable effective reuse?

This question is further explored through three sub-questions:

1. Which structural elements from cast-in-situ construction in the Netherlands are suitable for reuse?
2. What are the structural and practical challenges in adapting cast-in-situ elements for reuse in prefabricated systems?
3. What existing and innovative connection details are suitable for ensuring the structural integrity of reused load-bearing concrete elements from cast-in-situ structures?

The first step in enabling reuse is ensuring that structural elements can be extracted from existing buildings while maintaining their integrity. Diamond sawing is a proven technique in the construction industry, capable of making precise cuts through reinforced concrete with minimal damage, particularly when cutting in a section with low internal stresses during cutting. Among the cast-in-situ construction methods analyzed, straight columns, T-shaped columns, beams, floors and walls were identified as suitable for reuse in new building structures. However, cutting unavoidably results in the loss of critical reinforcement, including anchorage in all elements and edge reinforcement in walls and floors. For horizontal elements, the location of the cut significantly impacts their structural performance in the new design. It determines whether the dominant reinforcement at the end of the element remains at the top or bottom of the element. This directly influences the type of support conditions (e.g., hinged or fixed) and the corresponding moment distribution the element can withstand in the new design. If the cut is made near the original supports, the reinforcement remains at the top, making the elements more suitable for fixed connections. In contrast, if the cut is made at a moment-zero point, the reinforcement is located at the bottom, making it more suitable for hinged connections. In both cases, the original force distribution is preserved, reducing the risk of unexpected structural behavior. These findings directly answer the first two sub-questions by confirming which elements are best suited for reuse and identifying the key structural challenges in adapting them to prefabricated systems.

Based on these theoretical insights, practical reconnection solutions, such as post-installed reinforcement and end-anchorage, were identified and analyzed. To validate feasibility in real-world applications, two case studies were conducted. Supporting calculations were performed for each reusable structural element, leading to different conclusions about their reconnection feasibility. The specific findings for each structural element are outlined below.

- **Columns:** To optimize the reuse of columns, the applied load should be sufficiently reduced to eliminate the need for overlap with the vertical reinforcement and ensuring that the horizontal reinforcement provides adequate tensile capacity to resist point loads. To enhance this capacity, supporting beams along the longest side of the column is recommended.
- **Walls:** To simplify the reuse of walls, the applied load should be reduced enough to remove the necessity for overlap with the vertical reinforcement. To compensate for the lost edge reinforcement, various anchorage solutions can be used, including T-heads, MBT couplers, threaded disk anchors and welded reinforcement between adjacent walls, all requiring hydrodemolition to expose the reinforcement bars.
- **Beams:** To maximize the feasibility of reusing beams, it is essential to carefully determine the cutting locations. For hinged reconnections, the beam should be cut at the moment-zero points, though this may result in spans that are too short for feasible new designs. To anchor the reinforcement in this connection, a single plate will be welded to the existing reinforcement. For moment-fixed reconnections, the beam should be cut near the original supports, preserving the total length of the element. To ensure reinforcement continuity in this connection, the reinforcement can be directly connected using mechanical couplers. Reconnecting hydrodemolished beams via a vertical shear joint with new concrete is structurally viable, provided that additional stirrups are added in the newly poured concrete to anchor the compressive diagonal.
- **Floors:** The reuse of floors is also influenced by the cutting location, as it determines the type of new supports needed. For floors cut at moment-zero points, end-anchorage is not required if the support length is sufficient for anchorage, but the remaining span may be too short for standard designs. For floors cut near the original supports, the reinforcement can be directly connected using mechanical couplers to transfer moments and it preserves the full length of the element.

Building on theoretical insights and case study findings, reconnection solutions were developed for five common prefabricated connection types, ensuring representation of all identified reusable structural elements. These designs address case study limitations, enabling the integration of reused cast-in-situ concrete elements while preserving structural integrity. The solutions combine existing prefabricated connection methods with innovative techniques to overcome the challenges of reusing cast-in-situ elements. These proposed reconnections answer the third sub-question on suitable connection details for ensuring structural integrity. After developing and assessing the reconnection proposals, their feasibility was systematically compared through a multi-criteria analysis (MCA). The results confirm that hinged connections combining reused and new prefabricated elements are the most effective solutions for integrating cast-in-situ structural components in the current structural industry. These reconnections require minimal modifications, align with conventional construction techniques and allow for efficient force transfer, making them both structurally reliable and practical for implementation. In contrast, connections requiring fixed supports, more modifications, or significant precision are less favorable due to their increased complexity and lower feasibility in practice. However, when floors or beams are cut near their original supports, preserving their full length, maintaining a fixed connection can still be preferable to ensure structural integrity and efficient load transfer. These findings directly answer the main research question by identifying which load-bearing concrete elements are most suitable for reuse and how their reconnection details can be designed for effective reuse.

This research demonstrates that the reuse of cast-in-situ concrete elements is technically feasible when connection methods are carefully designed to meet structural and practical constraints. By developing solutions that require minimal modifications, use conventional construction techniques and optimize material efficiency, the integration of reused elements into new building structures becomes significantly more viable. The findings show that the most suitable load-bearing elements for reuse are those that can be incorporated into hinged connections, particularly when combined with new elements. By developing feasible reconnection solutions, this research provides a foundation for standardizing concrete element reuse in the industry, contributing to a more sustainable and circular construction industry.

11 Discussion

This chapter critically examines the findings of this research by discussing methodological limitations, the feasibility of the proposed reconnection solutions and the broader implications for sustainable construction. While this study demonstrates that cast-in-situ concrete elements can be successfully reused in new structures, several challenges remain regarding standardization, structural validation and real-world implementation.

11.1. Critical reflection on the methodology

This research focuses on the reconnection of pre-qualified structural elements, assuming that all necessary durability and structural assessments have been completed. However, in practice, ensuring the suitability of reused elements requires extensive testing for compressive strength, reinforcement corrosion, creep deformations and potential damage. Since no standardized approach for pre-qualification exists, the condition of the reused element could impact the feasibility of reuse in ways that were not accounted for in this study. Additionally, regulatory challenges remain, as existing building codes do not always accommodate reused elements, particularly in structural applications. The lack of clear design guidelines for reused concrete elements may limit their approval in building projects unless additional safety margins are applied. This means that the effectiveness of the proposed reconnection solutions depends on reliable assessment methods, which are currently lacking. Without a standardized framework for evaluating reused elements, the practical implementation of these solutions remains uncertain.

To further explore the theoretical insights on reusing cast-in-situ concrete elements and to assess the feasibility of reconnection methods within new structures, this research examined two case studies. Based on these case studies, assumptions regarding reinforcement layouts, available anchorage space and required overlap lengths guided the development of the reconnection solutions. While these cases provided valuable insights in practice, they do not fully represent the diversity of cast-in-situ structures. Differences in concrete quality, reinforcement layouts and original design intentions can affect the generalization of the findings. The extent to which variations in these parameters affect the applicability of the proposed solutions remains uncertain and requires further investigation.

The multi-criteria analysis in this study provided a structured way to compare different reconnection solutions, but it has its limitations. Not all possible connection types in a structure were analyzed in this study, meaning that certain solutions, such as connections to foundations, were not included in the assessment and therefore could not be considered as potential outcomes. These exclusions were necessary to maintain a manageable scope. Similarly, not all criteria relevant to reconnection of elements were included. Instead, only the most critical ones for this research were considered. For instance, economic feasibility was not explicitly assessed, even though cost is a major factor influencing whether reuse is prioritized in practice. This study primarily focused on technical feasibility and constructability. Furthermore, the MCA evaluated the feasibility of reconnection solutions rather than the overall reusability of individual elements.

As a result, factors such as the influence of the cutting location on an element's suitability for reuse were not incorporated. For example, elements cut at moment-zero points may not always be structurally viable for reuse due to an insufficient remaining design length, while those cut near their original supports are more likely to be applicable.

Long-term durability and maintenance costs, which can impact life-cycle performance, were also not factored into the MCA scoring. While this approach ensures a focused comparison, it also means that the MCA does not capture all aspects that may influence the practicality and sustainability of reusing cast-in-situ concrete elements. Additionally, the relative weight assigned to each criterion was determined throughout the research process, incorporating insights from discussions and guidance, which introduces a degree of subjectivity. A sensitivity analysis of the weight distribution could clarify how different criteria influence the final rankings. If minor weighting adjustments result in significantly different rankings, it may indicate that some solutions are particularly sensitive to specific design assumptions.

One of the main drivers for this research is sustainability, but the actual environmental impact depends on several factors. While the reuse of structural elements reduces raw material consumption, the additional processing required for cutting, transportation and reconnection introduces new resource demands. This study primarily focuses on structural reuse, whereas alternative circular strategies, such as down cycling into aggregates or reusing elements for non-structural applications, may sometimes be more practical and sustainable depending on project constraints. For example, if a project requires extensive modifications to integrate reused elements, the overall carbon savings might be lower than simply using recycled concrete aggregate in a new mix. The optimal circular strategy depends on the specific trade-off between processing effort, transportation distances and overall life-cycle impact. These aspects should be considered when evaluating the true sustainability benefits of structural reuse.

11.2. Interpretation of the results

While the proposed reconnection solutions are structurally viable, their implementation in real projects depends on more detailed structural analyses and design considerations than those discussed in this research. The effectiveness of these solutions is influenced by factors such as element-specific load conditions, second-order effects, long-term behavior under sustained loads and the structural system in which the element is utilized. Additionally, the force transfer mechanisms assumed in the design process require further validation through refined numerical modeling and experimental testing to assess actual bond behavior, anchorage performance and load redistribution effects. Force transfer in post-installed reinforcement under dynamic loading conditions remains also an uncertainty. Similarly, the actual bond behavior between hydrodemolished surfaces and new concrete requires experimental validation. To address these uncertainties, bond-slip tests could assess the interface behavior between new concrete and the hydrodemolished surface of existing elements, providing insight into load transfer efficiency. Additionally, durability tests could evaluate the long-term performance, while cyclic loading tests could assess the structural response of the connections under repeated loading. Full-scale load tests could further provide insights into the overall performance of reconnected elements in real-world conditions. Without experimental validation, the performance of these solutions remains theoretical.

While this study demonstrates that hinged connections combining reused and new elements provide the most practical solution, their applicability across different project types remains to be explored. It is important to assess how these connections perform in various structural configurations, such as low-rise versus high-rise buildings, different load-bearing systems and varying functional requirements. Additionally, the ability to integrate reused elements into entire structural systems, rather than just individual connections, must be further examined. While individual reconnections may be feasible, their impact on global structural behavior, including lateral stability and load redistribution, remains uncertain. Furthermore, a potential limitation of hinged connections is that the remaining element length may become too short for standard structural designs, which could restrict their feasibility in certain applications.

11.3. Practical challenges beyond structural feasibility

While this research demonstrates that cast-in-situ concrete elements can be successfully reused with well-designed reconnection methods, real-world implementation faces challenges beyond structural feasibility. Adoption depends on logistics, construction workflows and regulatory acceptance, which influence whether reuse strategies can be scaled effectively.

Logistical complexity arises from the deconstruction, transportation, storage and on-site adjustments required for reused elements. Variations in dimensions, reinforcement exposure and surface conditions can lead to mismatches or rework. Limited storage space, especially in urban areas, adds further constraints. Efficient tracking and planning are essential to minimize disruptions and ensure proper sequencing. Workflow adaptation poses another challenge. Prefabrication relies on efficiency and standardization, while reused elements introduce variability that may require adjustments in positioning, tolerance management and connection detailing. Despite aligning with conventional techniques, reuse may still demand additional on-site modifications, potentially increasing assembly time and reducing contractor willingness to adopt these methods. Regulatory acceptance remains a key barrier. Building codes and design standards primarily address new materials, making approval processes for reused elements uncertain. The lack of standardized assessment procedures can lead to additional testing and administrative burdens, while liability concerns may discourage adoption. Despite the technical feasibility of reconnection methods, large-scale implementation depends on establishing clear qualification guidelines to streamline regulatory approval and reduce liability concerns. Without such guidelines, approval remains uncertain, limiting widespread adoption. To enable large-scale reuse, integrated planning, adaptive workflows and regulatory support are crucial. Without addressing these constraints, reuse may remain limited to niche applications rather than becoming a viable strategy for circular construction.

12 Recommendations

While this research provides valuable insights into the feasibility of reusing cast-in-situ concrete elements, several areas require further investigation to facilitate real-world implementation. The following recommendations focus on research directions for future studies that aim to contribute to the development of structural concrete reuse.

12.1. Evaluate applicability in a complete construction

While this research validates individual reconnection solutions, their feasibility within a complete structural system remains untested. The findings indicate that reusing floor slabs and columns with new beams are the most practical and feasible solutions. Future research should focus on developing a full structural design that integrates these elements to assess whether a safe construction can be realized in accordance with current regulations. This evaluation should consider aspects such as global load distribution, stability, second-order effects and constructability.

To bridge the gap between theoretical feasibility and real-world application, incorporating data from existing donor projects is recommended. By selecting an actual donor building and designing a new structure based on its available elements, researchers can simulate the full reuse process from element extraction and reconnection to structural validation. This approach would provide insight into practical constraints such as variability in element dimensions, logistics of material handling and regulatory challenges.

Potential thesis topic: *”Assessing the structural integrity of reused concrete slabs and columns in a new building design”*

12.2. Experimental validation of reconnection solutions

While this study provides theoretical and practical validation of reconnection solutions, experimental research is needed to confirm their real-world structural behavior. Load tests are necessary to assess durability, failure mechanisms and overall structural integrity under realistic conditions. These tests should evaluate how reconnected elements perform under repeated loading, varying support conditions and how they adapt to load distributions different from those in the original design, to ensure their reliability in practical applications.

Potential thesis topic: *”Experimental validation of reconnected reused concrete elements: Structural integrity under varying load conditions”*

12.3. Life cycle assessment (LCA) of reuse strategies

A life cycle assessment (LCA) is essential to quantify the environmental impact of different reuse strategies for cast-in-situ concrete elements. While reuse reduces raw material consumption, additional processes such as cutting, transportation and reconnection introduce new energy demands and emissions. An LCA can provide a comparative analysis of embodied carbon, energy use and overall sustainability between reused structural elements, conventional new construction and alternative circular strategies such as recycling into aggregates. By demonstrating the potential carbon savings and resource efficiency of reuse, an LCA can highlight its role in achieving the 2050 climate goals, emphasizing the necessity of structural reuse as a key strategy for reducing emissions in the built environment.

Future research should assess whether the material savings from reuse outweigh the additional resource inputs and identify the conditions under which reuse offers the most environmental benefits. This should include analyzing transportation distances, required modifications and the lifespan extension achieved through reuse. Additionally, evaluating the potential of hybrid strategies, such as combining reused elements with new materials, could provide a more flexible and optimized approach to sustainable construction. Quantifying the trade-offs between different reuse approaches will help policymakers and industry stakeholders make informed decisions about integrating reuse into mainstream construction practices.

Potential thesis topic: *”Life cycle assessment of reused cast-in-situ concrete elements: Quantifying environmental benefits and trade-offs in circular construction”*

12.4. Standardized assessment of qualify structural elements

A barrier to large-scale reuse of structural concrete elements is the lack of standardized assessment methods to determine their suitability for reuse. Current evaluation practices vary widely, with no uniform guidelines for assessing mechanical properties, durability and structural integrity. Developing a standardized framework for evaluating reused concrete elements is important to ensure safety, streamline approval processes and facilitate widespread adoption in the construction industry.

Future research should focus on establishing clear assessment protocols for key parameters such as compressive strength, reinforcement corrosion, creep behavior and potential damage. These protocols should define acceptable limits, testing methods and classification criteria to determine whether an element can be reused structurally or requires alternative applications. Additionally, aligning these assessment methods with existing building regulations and Eurocode standards will be essential for regulatory acceptance. By creating a standardized qualification process, the uncertainty surrounding the reliability of reused elements can be reduced, making structural reuse more predictable and scalable. This would not only increase industry confidence in reuse strategies but also contribute to the development of a circular construction economy.

Potential thesis topic: *”Developing a standardized assessment framework for qualifying reused cast-in-situ concrete elements in structural applications”*

12.5. Comparison of old and modern design codes

Reused concrete elements were originally designed under older structural codes, which may differ significantly from current Eurocode requirements and standards. These differences can affect aspects such as material safety factors, reinforcement detailing and durability requirements. To facilitate structural reuse, a systematic comparison of old design codes and new regulations is necessary. Identifying changes will help determine whether reused elements require modifications or recalculations to meet today’s performance criteria.

Moreover, regulatory constraints often hinder the widespread adoption of reuse. Many current building regulations are primarily designed for new materials and structures, making it challenging to certify reused components. By identifying differences between past and present codes, it becomes possible to pinpoint specific regulatory barriers that restrict reuse. This knowledge can inform policy recommendations aimed at facilitating circular construction practices.

Future research should focus on mapping out these regulatory differences and assessing their implications for reuse. This includes analyzing load assumptions, safety margins and material degradation allowances in both past and current standards. In addition, developing guidelines for adapting reused elements to meet contemporary standards could help engineers integrate reuse strategies into new construction projects.

Potential thesis topic: *"Bridging the gap between legacy and modern structural codes: Adapting reused concrete elements for compliance with Eurocode standards"*

12.6. Digital tools and automation for efficient reuse integration

One of the key challenges in structural reuse is the difficulty of identifying suitable donor projects and matching available elements with new designs. Variations in dimensions, reinforcement layouts and material properties complicate the integration of reused elements into new structures. Manual assessments and redesign efforts can be time-consuming and inefficient, limiting the scalability of reuse strategies.

Future research should explore the potential of digital tools to enhance the reuse process, from material inventory and structural analysis to automated redesign strategies. Parametric design and AI-driven material mapping could facilitate reuse by generating new structural configurations based on available donor elements. Additionally, investigating how BIM can integrate structural data from donor buildings and support adaptive design approaches will be crucial for scaling up reuse.

Potential thesis topic: *"Optimizing structural reuse through digital tools: The role of BIM and automation in integrating reused concrete elements into new designs"*

The reuse of cast-in-situ concrete elements presents a significant opportunity for sustainable construction, yet critical challenges remain. Future research should focus on integrating these reused elements into complete structural systems, experimentally validating reconnection methods, and developing standardized assessment and regulatory frameworks. Advancing digital tools and conducting life cycle assessments will further facilitate large-scale implementation. Addressing these challenges is crucial for advancing innovation in circular building construction and determine structural reuse as a viable and standardized approach in building engineering.

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A

Calculations

A.1. Verification of horizontal reinforcement capacity in the columns of the Munthof parking garage

The following calculations verify whether the existing stirrups in the D-region of the Munthof columns provide sufficient tensile capacity to resist the horizontal forces introduced by two beams.The existing horizontal reinforcement in the collumns consists of ribbed QR 40 reinforcement stirrups with $\phi 12$. The D-region has the length of $h = 400$ mm.

A.1.1. Tensile capacity calculation

The maximum tensile force N_t of the stirrups in the top region of the column with length h = 400 mm is calculated using:

$$N_t = 2 \cdot \frac{A_s}{s} \cdot h \cdot f_{yd} \tag{A.1}$$

where:

- $A_s = \frac{\pi}{4} \cdot 12^2 = 113$ mm^2 cross-sectional area per leg
- $s = 200$ mm stirrup spacing
- $h = 400$ mm evaluated length

Substituting the values:

$$N_t = 2 \cdot \frac{113}{200} \cdot 400 \cdot 349 \approx 158\,\text{kN}$$

The maximum tensile capacity, further mentioned as T, of the reinforcement bars in the top region of 400 mm is approximately 158 kN.

A.1.2. Allowable concentrated point load on a column

To determine the allowable shear force F that can be resisted by the column, equation (8.1) is used:

$$T = \frac{1}{4} \frac{w - (2 \cdot b)}{w} F \tag{A.2}$$

Rewriting the formula to solve for F:

$$F = \frac{4w}{w - 2b} T \tag{A.3}$$

Assuming that a percentage P of the column width w is used for the supports:

$$b = \frac{P}{2} \cdot w \tag{A.4}$$

Substituting this into the formula:

$$w - 2b = w(1 - P) \tag{A.5}$$

And further:

$$F = \frac{4}{1 - P} T \tag{A.6}$$

For example, if $P = 0.67$ ($\frac{2}{3}$ of the column cross-sectional area is used for support):

$$F = \frac{4}{0.67} T = 5.97 \cdot 158 \approx 943 \text{ kN}$$

The allowable force F with 67% support area is approximately 943 kN.

A.1.3. Verification if the stirrup can handle a realistic load in a parking garage

To verify if the horizontal reinforcement in the top regian of a column of the Munthof can handle a realistic applied load on a beam of a parking garage, the following calculations are performed.

Given data of the munthof:

- Beam span: $l = 10.32$ m
- ULS load: $q = 890$ kg/m^2 $= 8.73$ kN/m^2
- Floor width supported by the beam: $B = 6.35$ m
- Point load capacity: $F = 943$ kN

Line load calculation:

$$q_{\text{line}} = q \cdot B = 8.73 \cdot 6.35 = 55.45 \text{ kN/m} \tag{A.7}$$

Shear force calculation for a simply supported beam:

$$V_{Ed} = \frac{q_{\text{line}} \cdot l}{2} = \frac{55.45 \cdot 10.32}{2} \approx 286 \text{ kN} \tag{A.8}$$

The maximum shear force V_{Ed} from 2 beams on the column is calculated to be $286 \cdot 2 = 572$ kN.

The calculated shear force (572 kN) is well below the allowable point load capacity of the reinforcement bar (943 kN).

A.2. Overlap length of the vertical reinforcement in a column of the Munthof parking garage

The calculation below determines the overlap length (l_0) for the vertical reinforcement in a column of the Munthof garage. The vertical reinforcement consists of ribbed QR 40 rebars with $\phi = 28$ mm, and the concrete is of quality K300, equivalent to C25/30 under current standards.

A.2.1. Calculation of the effective bond strength (f_{bd})

$$f_{bd} = 2.25 \cdot \eta_1 \cdot \eta_2 \cdot f_{ctd} \tag{A.9}$$

where:

- $f_{ctd} = \frac{f_{ctk,0.05}}{\gamma_c} = \frac{1.8}{1.5} = 1.2$ (N/mm^2)
- $\eta_1 = 1$ for vertical reinforcement.
- $\eta_2 = 1$ for ribbed reinforcement bars.

This gives a bond strength $f_{bd} = 2.25 \cdot 1 \cdot 1 \cdot 1.2 = 2.7$ N/mm^2.

A.2.2. Calculation of the base anchorage length ($l_{b,rqd}$)

$$l_{b,rqd} = \frac{\phi}{4} \cdot \frac{\sigma_{sd}}{f_{bd}} \tag{A.10}$$

where:

- $\sigma_{sd} = 349$ (N/mm^2) (assumed equal to the yield strength; (Betonstaal.nl, 2024))
- $f_{bd} = 2.7$ (N/mm^2)
- $\phi = 28$ (mm)

$$l_{b,rqd} = \frac{28}{4} \cdot \frac{349}{2.7} \approx 905 \text{ mm}.$$

A.2.3. Verification of the minimum overlap length ($l_{0,min}$)

$$l_{0,min} \geq \max\{15 \cdot \phi, 200 \text{ mm}, 0.3 \cdot \alpha_6 \cdot l_{b,rqd}\} \tag{A.11}$$

$$l_{0,min} \geq \max\{15 \cdot 12, 200, 0.3 \cdot 1.5 \cdot 905\} = \max\{180, 200, 407\} = 407 \text{ mm}.$$

A.2.4. Calculation of the overlap length (l_0)

$$l_0 = \alpha_1 \cdot \alpha_2 \cdot \alpha_3 \cdot \alpha_5 \cdot \alpha_6 \cdot l_{b,rqd} \tag{A.12}$$

where:

- $\alpha_1 = 1$ Straight overlap.
- $\alpha_2 = 1$ Assumed low concrete cover.
- $\alpha_3 = 1$ Conservative choice.
- $\alpha_5 = 1$ Assuming that compressive stresses do not significantly contribute to the bond effect.
- $\alpha_6 = 1.5$ Overlap in the same cross-section.
- $l_{b,rqd} = 905$ mm.

$$l_0 = 1 \cdot 1 \cdot 1 \cdot 1 \cdot 1.5 \cdot 905 \approx 1358 \text{ mm}.$$

A.3. Derivation of the moment zero points for a fixed-fixed Beam

For a beam with fixed supports subjected to a uniformly distributed load (q), the bending moment at any point along the span can be determined using the differential equation of beam bending:

$$EI\frac{d^4v}{dx^4} = q \tag{A.13}$$

where:

- EI is the flexural rigidity of the beam,
- q is the uniform load,
- $v(x)$ is the deflection function along the beam,
- x is the position along the beam length.

Step 1: Moment Equation By integrating Equation (A.13) twice, we obtain the bending moment equation:

$$M(x) = \frac{q}{2}x^2 + C_1x + C_2 \tag{A.14}$$

where C_1 and C_2 are integration constants determined by the boundary conditions.

Boundary Conditions for Fixed Supports For a fixed-fixed beam, the boundary conditions are:

- No rotation at the supports: $\theta(0) = 0$ and $\theta(L) = 0$,
- No deflection at the supports: $v(0) = 0$ and $v(L) = 0$.

Solving for C_1 and C_2 leads to the moment equation:

$$M(x) = \frac{qL^2}{12}\left(6\frac{x}{L} - 6\left(\frac{x}{L}\right)^2\right) \tag{A.15}$$

where L is the beam span.

Finding the Zero Moment Points The zero moment points occur where $M(x) = 0$. Substituting into Equation (A.15):

$$\frac{qL^2}{12}\left(6\frac{x}{L} - 6\left(\frac{x}{L}\right)^2\right) = 0 \tag{A.16}$$

Dividing by $\frac{qL^2}{12}$:

$$6\frac{x}{L} - 6\left(\frac{x}{L}\right)^2 = 0 \tag{A.17}$$

Rearranging:

$$6\frac{x}{L} = 6\left(\frac{x}{L}\right)^2 \tag{A.18}$$

Factoring:

$$\frac{x}{L}(6 - 6\frac{x}{L}) = 0 \tag{A.19}$$

Solving for x:

$$\frac{x}{L} = 0 \quad \text{or} \quad \frac{x}{L} = 1 \quad \text{(trivial solutions at the supports)} \tag{A.20}$$

and the non-trivial solutions:

$$\frac{x}{L} = 0.211 \quad \text{or} \quad \frac{x}{L} = 0.789 \tag{A.21}$$

Thus, the exact zero moment points are:

$$x_{\text{zero1}} = 0.211L, \quad x_{\text{zero2}} = 0.789L \tag{A.22}$$

A.4. Cross-sectional reinforcement ratio of a beam in the Munthof parking garage

To determine the tensile reinforcement cross-sectional ratio of a beam in the Munthof parking garage, a cross-section near the support is selected, as this is where the most tensile reinforcement is concentrated. In figure A.1, the calculated cross-section is shown. The following calculations are performed.

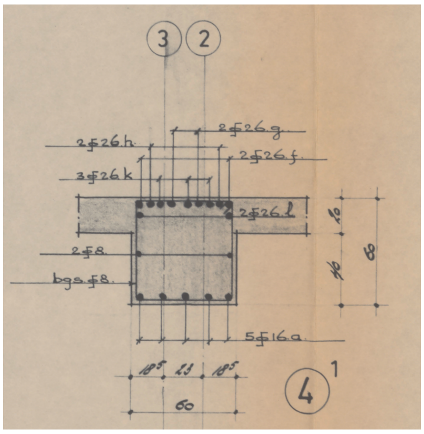

Figure A.1: Cross-section of a beam in Munthof above a support

The effective depth d of the tensile zone is calculated as:

$$d = h - c - \frac{\phi}{2} = 600 - 20 - \frac{26}{2} = 567 \text{ mm} \tag{A.23}$$

The effective concrete area $A_{c,\text{eff}}$ corresponding to the tensile zone is:

$$A_{c,\text{eff}} = b \cdot d = 600 \cdot 567 = 340200 \text{ mm}^2 \tag{A.24}$$

The total area A_s of the tensile reinforcement in the cross-section is calculated as:

$$A_s = \sum n \left(\frac{\pi}{4} \cdot \phi^2\right) = 2\left(\frac{\pi}{4} \cdot 8^2\right) + 11\left(\frac{\pi}{4} \cdot 26^2\right) \approx 5941 \text{ mm}^2 \tag{A.25}$$

The effective reinforcement ratio ρ_{eff} in the tensile zone is calculated as:

$$\rho_{\text{eff}} = \left(\frac{A_s}{A_{c,\text{eff}}}\right) \cdot 100 = \left(\frac{5941}{340200}\right) \cdot 100 \approx 1.75\% \tag{A.26}$$

This effective reinforcement ratio provides insight into the tensile reinforcement concentration, relevant for assessing re-connection strategies such as post-installed reinforcement.

A.5. Overlap length for the existing horizontal reinforcement in the beam of Munthof near by fixed support

The calculation below determines the overlap length (l_0). The horizontal reinforcement in the beams of the Munthof consist of ribbed QR 40 rebars with $\phi = 26$, and concrete of quality K300, which is equivalent to C25/30 under current standards. The concrete cover is 20 mm. The calculation is performed for the same cross-section as shown in figure A.1, considering the top reinforcement. The schematic representation of the beam is shown below.

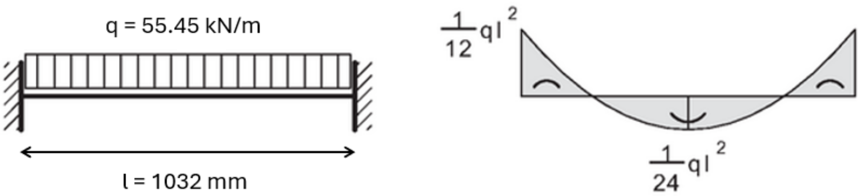

Figure A.2: Schematic representation of the beam in Munthof

A.5.1. Calculation of the effective bond strength (f_{bd})

The effective bond strength is calculated for the top reinforcement.

$$f_{bd} = 2.25 \cdot \eta_1 \cdot \eta_2 \cdot f_{ctd} \tag{A.27}$$

where:

- $f_{ctd} = \frac{f_{ctk,0.05}}{\gamma_c} = \frac{1.8}{1.5} = 1.2$ (N/mm^2)
- η_1 = 0.7 for reinforcement in the top.
- η_2 = 1 for ribbed reinforcement bars.

This gives a bond strength f_{bd} = 1.89(N/mm^2).

A.5.2. Calculation of the base anchorage length ($l_{b,rqd}$)

The base anchorage length is calculated using:

$$l_{b,rqd} = \frac{\phi}{4} \cdot \frac{\sigma_{sd}}{f_{bd}} \tag{A.28}$$

where:

- σ_{sd} = reinforcement stress
- f_{bd} = 1.89(N/mm^2)
- ϕ = 26(mm)

The σ_{sd} is calculated using:

$$\sigma_{sd} = \frac{M_{Ed}}{z \cdot A_s} \tag{A.29}$$

where:

- $M_{Ed} = \frac{1}{12}qL^2 = \frac{1}{12}55.45 \cdot 10.32^2 = 492.13 \cdot 10^6$Nmm
- z = $0.9 \cdot d = 0.9(600 - 20 - \frac{1}{2} \cdot 26) = 510$ mm
- A_s = $11(\frac{\pi}{4} \cdot 26^2) = 5840$ mm^2

This gives a σ_{sd} of 165 N/mm^2 which gives a base anchorage length $l_{b,rqd} \approx 567$ mm

A.5.3. Verification of the minimum overlap length ($l_{0,min}$)

The minimum overlap length is determined as:

$$l_{0,min} \geq \max\{15 \cdot \phi, 200\,\text{mm}, 0.3 \cdot \alpha_6 \cdot l_{b,rqd}\} \tag{A.30}$$

$$l_{0,min} \geq \max\{15 \cdot 26, 200, 0.3 \cdot 1.5 \cdot 567\} = \max\{390, 200, 255\} = 390\,\text{mm}$$

A.5.4. Calculation of the overlap length (l_0)

The overlap length is calculated using:

$$l_0 = \alpha_1 \cdot \alpha_2 \cdot \alpha_3 \cdot \alpha_5 \cdot \alpha_6 \cdot l_{b,rqd} \tag{A.31}$$

where:

- α_1 = 1 Straight overlap.
- α_2 = 1 Assumed low concrete cover.
- α_3 = 1 Conservative choice.
- α_5 = 1 Assuming that compressive stresses do not significantly contribute to the bond effect.
- α_6 = $1,5$ Overlap in the same cross section.
- $l_{b,rqd}$ = 567 mm.

$$l_0 = 1 \cdot 1 \cdot 1 \cdot 1 \cdot 1.5 \cdot 567 \approx 851\,\text{mm}$$

The requireed overlap length l_0 = 851 mm

A.6. Anchorage length for the existing horizontal reinforcement in the beam of Munthof at moment-zero point

The anchorage length is calculated for the reinforcement in the cross-section at the moment-zero point. This cross-section is shown in figure A.3. Since this concerns the same beam, the concrete and reinforcement properties remain identical to those specified in appendix A.5. The remaining beam length between the moment-zero points is 5.8m.

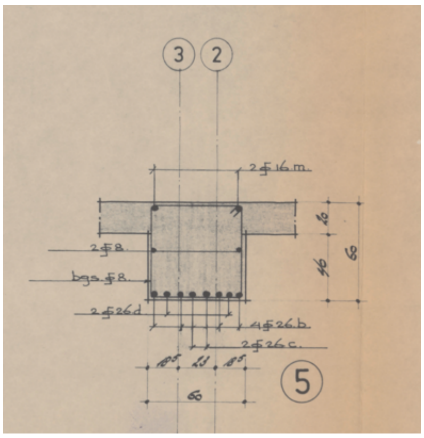

Figure A.3: Cross-section moment-zero point in the beam of Munthof

A.6.1. Calculation of the effective bond strength (f_{bd})

The effective bond strength is calculated for the top reinforcement.

$$f_{bd} = 2.25 \cdot \eta_1 \cdot \eta_2 \cdot f_{ctd} \tag{A.32}$$

where:

- $f_{ctd} = \frac{f_{ctk,0.05}}{\gamma_c} = \frac{1.8}{1.5} = 1.2$ (N/mm^2)
- η_1 = 1 for reinforcement in the bottom.
- η_2 = 1 for ribbed reinforcement bars.

This gives a bond strength f_{bd} = 2.7(N/mm^2).

A.6.2. Calculation of the base anchorage length ($l_{b,rqd}$)

The base anchorage length is calculated using:

$$l_{b,rqd} = \frac{\phi}{4} \cdot \frac{\sigma_{sd}}{f_{bd}} \tag{A.33}$$

where:

- σ_{sd} = reinforcement stress
- f_{bd} = 2.7(N/mm^2)
- ϕ = 26(mm)

The σ_{sd} is calculated using:

$$\sigma_{sd} = \frac{M_{Ed}}{z \cdot A_s} \tag{A.34}$$

Since this calculation is performed in the cross-section at the moment-zero point, the shifted moment line must be taken into account. The moment line is shifted over a distance $ai = d$ (NEN-EN 1992-1-1+C2; Section 9.2.1.3 (Nederlands Normalisatie-instituut (NEN), 2011)), which corresponds to the height of the beam. In this case, the shift is 0.6 m.

For a simply supported beam with a uniformly distributed load q, the bending moment at any position x along the span is given by:

$$M_{Ed}(x) = \frac{q \cdot x \cdot (L - x)}{2} \tag{A.35}$$

where:

- $q = 55.45$ kN/m (ULS line load)
- $L = 5.8$ m (remaining span of the beam)
- $x = 0.6$ m (shifted moment position)

Substituting these values:

$$M_{Ed}(0.6) = \frac{55.45 \cdot 0.6 \cdot (5.8 - 0.6)}{2} = 86.50\,\text{kNm}$$

Thus, the bending moment that must be considered for the reinforcement stress at the moment-zero point is 86.50 kNm. To calculate the stress in the reinforcement the following equation is used:

$$\sigma_{sd} = \frac{M_{Ed}}{z \cdot A_s} \tag{A.36}$$

where:

- $M_{Ed} = 86.50 \cdot 10^6$ N/mm
- z = $0.9 \cdot d = 0.9(600 - 20 - \frac{1}{2} \cdot 26) = 510$ mm
- A_s = $11(\frac{\pi}{4} \cdot 26^2) = 5840$ mm^2

This gives a σ_{sd} of 29.02 N/mm^2 which gives a base anchorage length $l_{b,rqd} \approx 70$ mm

A.6.3. Verification of the minimum anchorage length ($l_{b,min}$)

The minimum overlap length is determined as:

$$l_{b,min} \geq \max\{10 \cdot \phi, 100\,\text{mm}, 0.3 \cdot l_{b,rqd}\} \tag{A.37}$$

$$l_{b,min} \geq \max\{10 \cdot 26, 100, 0.3 \cdot 70\} = \max\{260, 100, 21\} = 260\,\text{mm}$$

A.6.4. Calculation value of the anchorage length (l_{bd})

The calculation value of the anchorage length needed at an end support is calculated using:

$$l_{bd} = \alpha_1 \cdot \alpha_2 \cdot \alpha_3 \cdot \alpha_4 \cdot \alpha_5 \cdot l_{b,rqd} \tag{A.38}$$

where:

- α_1 = 1 Straight overlap.
- α_2 = 1 Assumed low concrete cover.
- α_3 = 1 Conservative choice.
- α_4 = 1 No welded shear reinforcement.
- α_5 = 1 Assuming that compressive stresses do not significantly contribute to the bond effect.
- $l_{b,rqd}$ = 70 mm.

This gives an anchorage length $l_{bd} \leq l_{b,min}$, which results in an anchorage length of 260 mm

A.7. Cross-sectional reinforcement ratio of a 1 m floor element in the Munthof garage

To determine the reinforcement cross-sectional ratio of a 1 m wide floor element, the following calculations are performed. The reinforcement in the floors have a diameter of 12 mm and is spaced at 190mm between the reinforcement bars.

The total concrete area A_c of the floor element is calculated as:

$$A_c = b \cdot h = 1000 \cdot 150 = 150000\,\text{mm}^2 \tag{A.39}$$

The total reinforcement area A_s is calculated for longitudinal reinforcement:

$$A_s = n \cdot \left(\frac{\pi}{4} \cdot \phi^2\right) \tag{A.40}$$

The number of reinforcement bars n in a 1 m wide floor element is:

$$n = \frac{1000}{190} \approx 6\,\text{bars} \tag{A.41}$$

Substituting n and ϕ:

$$A_s = 6 \cdot \left(\frac{\pi}{4} \cdot 12^2\right) \approx 2 \cdot 6 \cdot 113 = 678\,\text{mm}^2 \tag{A.42}$$

The reinforcement cross-sectional ratio ρ is calculated as:

$$\rho = \frac{A_s}{A_c} \cdot 100 = \frac{678}{150000} \cdot 100 \approx 0.45\,\% \tag{A.43}$$

The reinforcement cross-sectional ratio of the 1 m wide floor element, considering both top and bottom reinforcement, is approximately 0.45%.

A.8. Overlap length for the existing longitudinal reinforcement in the floor of Munthof near by fixed support

The calculation below determines the overlap length (l_0) for the reinforcement in the floor of the Munthof. The concrete is of quality K300, equivalent to C25/30 under current standards. A 1 m wide slab consists of 6 ribbed QR 40 rebars with $\phi = 12$ mm. The calculation is done for the section near the supports. The span of the floor is 6.35 m and the ULS load on a 1 m wide slab is 8.73 kN/m. The thickness of the floor is 150 mm with a concrete cover of 10 mm.

A.8.1. Calculation of the effective bond strength (f_{bd})

The effective bond strength is calculated for the top reinforcement, as the reinforcement is in the top near by the supports.

$$f_{bd} = 2.25 \cdot \eta_1 \cdot \eta_2 \cdot f_{ctd} \tag{A.44}$$

where:

- $f_{ctd} = \frac{f_{ctk,0.05}}{\gamma_c} = \frac{1.8}{1.5} = 1.2\,$(N/mm^2)
- η_1 = 0.7 for reinforcement in the top.
- η_2 = 1 for ribbed reinforcement bars.

This gives a bond strength $f_{bd,top}$ = 1.89(N/mm^2).

A.8.2. Calculation of the base anchorage length ($l_{b,rqd}$)

The anchorage length is calculated for both the top and bottom reinforcement using:

$$l_{b,rqd} = \frac{\phi}{4} \cdot \frac{\sigma_{sd}}{f_{bd}} \tag{A.45}$$

where:

- $\sigma_{sd} =$ reinforcement stress
- $f_{bd} = 1.89\,$(N/mm^2)
- $\phi = 12\,$(mm)

The σ_{sd} is calculated using:

$$\sigma_{sd} = \frac{M_{Ed}}{z \cdot A_s} \quad \text{(A.46)}$$

where:

- $M_{Ed} = \frac{1}{12}qL^2 = \frac{1}{12}8.73 \cdot 6.35^2 = 29.33 \cdot 10^6$Nmm
- z = $0.9 \cdot d = 0.9(150 - 10 - \frac{1}{2} \cdot 12) = 121$ mm
- A_s = $6(\frac{\pi}{4} \cdot 12^2) = 679$ mm^2

This gives a σ_{sd} of 357 N/mm^2 which gives a base anchorage length $l_{b,rqd} \approx 567$ mm

A.8.3. Verification of the minimum overlap length ($l_{0,min}$)

The minimum overlap length is determined as:

$$l_{0,min} \geq \max\{15 \cdot \phi, 200\,\text{mm}, 0.3 \cdot \alpha_6 \cdot l_{b,rqd}\} \quad \text{(A.47)}$$

$$l_{0,min} \geq \max\{15 \cdot 12, 200, 0.3 \cdot 1.5 \cdot 567\} = \max\{180, 200, 255\} = 255\,\text{mm}$$

A.8.4. Calculation of the overlap length (l_0)

The overlap length is calculated using:

$$l_0 = \alpha_1 \cdot \alpha_2 \cdot \alpha_3 \cdot \alpha_5 \cdot \alpha_6 \cdot l_{b,rqd} \quad \text{(A.48)}$$

where:

- α_1 = 1 Straight overlap.
- α_2 = 1 Assumed low concrete cover.
- α_3 = 1 Conservative choice.
- α_5 = 1 Assuming that compressive stresses do not significantly contribute to the bond effect.
- α_6 = 1.5 Overlap in the same cross section.
- $l_{b,rqd}$ = 567 (mm).

This gives an overlap length of:

$$l_0 = 1 \cdot 1 \cdot 1 \cdot 1 \cdot 1.5 \cdot 567 = 851\,\text{mm}$$

This gives an overlap length l_0 of 851 mm.

A.9. Anchorage length for the existing longitudinal reinforcement in the floor of Munthof at moment-zero points

The anchorage length is calculated for the reinforcement in the cross-section at the moment-zero p points. Since this concerns the same floor as used in appendix A.8, the concrete and reinforcement properties remain identical. In the moment-zero section the reinforcement is located at the bottom and the remaining span length is 3.6 m.

A.9.1. Calculation of the effective bond strength (f_{bd})

The calculation is performed for the bottom reinforcement, as the reinforcement is located at the bottom in the moment-zero points.

$$f_{bd} = 2.25 \cdot \eta_1 \cdot \eta_2 \cdot f_{ctd} \tag{A.49}$$

where:

- $f_{ctd} = \frac{f_{ctk,0.05}}{\gamma_c} = \frac{1.8}{1.5} = 1.2$ (N/mm^2)
- $\eta_1 = 1$ for reinforcement in the bottom.
- $\eta_2 = 1$ for ribbed reinforcement bars.

This gives a bond strength $f_{bd} = 2.7$ (N/mm^2).

A.9.2. Calculation of the base anchorage length ($l_{b,rqd}$)

The base anchorage length is calculated using:

$$l_{b,rqd} = \frac{\phi}{4} \cdot \frac{\sigma_{sd}}{f_{bd}} \tag{A.50}$$

where:

- σ_{sd} = reinforcement stress
- f_{bd} = 2.7(N/mm^2)
- ϕ = 24(mm)

The σ_{sd} is calculated using:

$$\sigma_{sd} = \frac{M_{Ed}}{z \cdot A_s} \tag{A.51}$$

Since this calculation is performed in the cross-section at the moment-zero point, the shifted moment line must be taken into account. The moment line is shifted over a distance $ai = d$ (NEN-EN 1992-1-1+C2; Section 9.2.1.3 (Nederlands Normalisatie-instituut (NEN), 2011)), which corresponds to the height of the floor. In this case, the shift is 0.15 m.

For a simply supported beam with a uniformly distributed load q, the bending moment at any position x along the span is given by:

$$M_{Ed}(x) = \frac{q \cdot x \cdot (L - x)}{2} \tag{A.52}$$

where:

- $q = 8.73$ kN/m (ULS load on 1m wide floor slab)
- $L = 3.6$ m (remaining span of the beam)
- $x = 0.15$ m (shifted moment position)

Substituting these values:

$$M_{Ed}(0.6) = \frac{8.73 \cdot 0.15 \cdot (3.6 - 0.15)}{2} = 2.26\,\text{kNm}$$

Thus, the bending moment that must be considered for the reinforcement stress at the moment-zero point is 2.26 kNm. To calculate the stress in the reinforcement the following equation is used:

$$\sigma_{sd} = \frac{M_{Ed}}{z \cdot A_s} \tag{A.53}$$

where:

- $M_{Ed} = 2.26 \cdot 10^6$ N/mm
- z = $0.9 \cdot d = 0.9(150 - 10 - \frac{1}{2} \cdot 12) = 121$ mm
- A_s = $6(\frac{\pi}{4} \cdot 12^2) = 679$ mm^2

This gives a σ_{sd} of 27.52 N/mm^2 which gives a base anchorage length $l_{b,rqd} \approx 61$ mm

A.9.3. Verification of the minimum anchorage length ($l_{b,min}$)

The minimum anchorage length is determined as:

$$l_{b,min} \geq \max\{10 \cdot \phi, 100\,\text{mm}, 0.3 \cdot l_{b,rqd}\} \tag{A.54}$$

$$l_{b,min} \geq \max\{10 \cdot 12, 100, 0.3 \cdot 61\} = \max\{120, 100, 18.3\} = 120\,\text{mm}$$

A.9.4. Calculation value of the anchorage length (l_{bd})

The calculation value of the anchorage length is determined using:

$$l_{bd} = \alpha_1 \cdot \alpha_2 \cdot \alpha_3 \cdot \alpha_4 \cdot \alpha_5 \cdot l_{b,rqd} \tag{A.55}$$

where:

- α_1 = 1 Straight overlap.
- α_2 = 1 Assumed low concrete cover.
- α_3 = 1 Straight overlap.
- α_4 = 1 No welded shear reinforcement.
- α_5 = 1 Assuming compressive stresses do not significantly contribute to the bond effect.
- $l_{b,rqd}$ = 61 mm

This gives an anchorage length $l_{bd} \leq l_{b,min}$, which results in an anchorage length of 120 mm

A.10. Overlap length for the vertical reinforcement of the wall of Munthof

The calculation below determines the overlap length (l_0) for the vertical reinforcement in the wall of the Munthof. The concrete is of quality K300, equivalent to C25/30 under current standards. In most walls the vertical reinforcement consists of ribbed QR 40 rebars with $\phi = 12$ mm.

A.10.1. Calculation of the effective bond strength (f_{bd})

The effective bond strength is calculated for the vertical reinforcement.

$$f_{bd} = 2.25 \cdot \eta_1 \cdot \eta_2 \cdot f_{ctd} \tag{A.56}$$

where:

- $f_{ctd} = \frac{f_{ctk,0.05}}{\gamma_c} = \frac{1.8}{1.5} = 1.2$ (N/mm^2)
- $\eta_1 = 1$ for vertical reinforcement.
- $\eta_2 = 1$ for ribbed reinforcement bars.

This gives a bond strength $f_{bd} = 2.25 \cdot 1 \cdot 1 \cdot 1.2 = 2.7$ (N/mm^2).

A.10.2. Calculation of the base anchorage length ($l_{b,rqd}$)

The anchorage length is calculated using:

$$l_{b,rqd} = \frac{\phi}{4} \cdot \frac{\sigma_{sd}}{f_{bd}} \tag{A.57}$$

where:

- $\sigma_{sd} = 349$ (N/mm^2) (assumed equal to the yield strength; (Betonstaal.nl, 2024))
- $f_{bd} = 2.7$ (N/mm^2)
- $\phi = 12$ mm

$$l_{b,rqd} = \frac{12}{4} \cdot \frac{349}{2.7} \approx 388 \text{ mm}$$

A.10.3. Verification of the minimum overlap length ($l_{0,min}$)

The minimum overlap length is determined as:

$$l_{0,min} \geq \max\{15 \cdot \phi, 200 \text{ mm}, 0.3 \cdot \alpha_6 \cdot l_{b,rqd}\} \tag{A.58}$$

$$l_{0,min} \geq \max\{15 \cdot 12, 200, 0.3 \cdot 1.5 \cdot 388\} = \max\{180, 200, 175\} = 200 \text{ mm}$$

A.10.4. Calculation of the overlap length (l_0)

The overlap length is calculated using:

$$l_0 = \alpha_1 \cdot \alpha_2 \cdot \alpha_3 \cdot \alpha_5 \cdot \alpha_6 \cdot l_{b,rqd} \tag{A.59}$$

where:

- $\alpha_1 = 1$ Straight overlap.
- $\alpha_2 = 1$ Assumed low concrete cover.
- $\alpha_3 = 1$ Conservative choice.
- $\alpha_5 = 1$ Assuming that compressive stresses do not significantly contribute to the bond effect.

- $\alpha_6 = 1.5$ Overlap in the same cross-section.
- $l_{b,rqd} = 388$ mm

$$l_0 = 1 \cdot 1 \cdot 1 \cdot 1 \cdot 1.5 \cdot 388 = 582\,\text{mm}$$

A.11. Verification of horizontal reinforcement capacity in the columns of Stationsplein 107

The following calculations verify if the existing smooth QR 24 reinforcement stirrups with $\phi 8$ in the columns of Stationsplein 107 can handle the horizontal force expected in a repurposed simply supported system of a parking garage. The width of most of the columns is 300 mm so the length of the D-region is also 300 mm.

A.11.1. Tensile capacity calculation

The maximum tensile force N_t of two reinforcement bars is calculated using:

$$N_t = 2 \cdot \frac{A_s}{s} \cdot h \cdot f_{yd} \tag{A.60}$$

Where:

- $A_s = \frac{\pi}{4} \cdot 8^2 \approx 50$ mm^2 cross-sectional area per leg
- $f_{yd} = 209$ N/mm^2 QR 24 Betonstaal.nl, 2024
- $d = 8$ mm diameter of the reinforcement bar

Substituting into the formula:

$$N_t = 2 \cdot \frac{50}{200} \cdot 300 \cdot 209 \approx 31 \text{ kN} \tag{A.61}$$

The maximum tensile force of two reinforcement bars is approximately 31 kN.

A.11.2. Allowable concentrated point load on the column

With the formula derived in appendix A.1.2:

$$F = \frac{4}{0.67} T = 5.97 \cdot 31 = 185 \text{ kN} \tag{A.62}$$

The allowable force F with 67% support area is approximately 185 kN.

A.11.3. Verification if the stirrup can handle realistic a load in an office building

To verify if the reinforcement can handle a realistic shear force in a beam of the Stationsplein 107 office building, the following calculations are performed.

Given data of Stationsplein 107:

- Beam span: $l = 8.15$ m
- ULS load: $q = 7.88$ kN/m^2
- Floor width supported by the beam: $B = 4.07$ m
- Point load capacity: $F = 192.1$ kN

Line Load Calculation:

$$q_{\text{line}} = q \cdot B = 7.88 \cdot 4.07 \approx 32.07\,\text{kN/m} \tag{A.63}$$

Shear Force Calculation for a simply supported beam:

$$V_{\text{Ed}} = \frac{q_{\text{line}} \cdot l}{2} = \frac{32.07 \cdot 8.15}{2} \approx 131\,\text{kN} \tag{A.64}$$

The maximum shear force V_{Ed} from 2 beams on the column is calculated to be $131 \cdot 2 = 262$ kN. The calculated shear force (262 kN) does exceeds the allowable point load capacity of the reinforcement bar (185 kN).

A.12. Overlap length for the vertical reinforcement in the columns of Stationsplein 107

The calculation below determines the minimum overlap length (l_0). The vertical reinforcement in the columns of Stationsplein 107 consists of ribbed QR 42 rebars with $\phi = 12$ mm, and the concrete quality is C25/30.

A.12.1. Calculation of the effective bond strength (f_{bd})

The effective bond strength is calculated for the vertical reinforcement.

$$f_{bd} = 2.25 \cdot \eta_1 \cdot \eta_2 \cdot f_{ctd} \tag{A.65}$$

where:

- $f_{ctd} = \frac{f_{ctk,0.05}}{\gamma_c} = \frac{1.8}{1.5} = 1.2$ (N/mm^2)
- $\eta_1 = 1$ for vertical reinforcement.
- $\eta_2 = 1$ for ribbed reinforcement bars.

This gives a bond strength $f_{bd} = 2.7$ (N/mm^2).

A.12.2. Calculation of the base anchorage length ($l_{b,rqd}$)

The anchorage length is calculated using:

$$l_{b,rqd} = \frac{\phi}{4} \cdot \frac{\sigma_{sd}}{f_{bd}} \tag{A.66}$$

where:

- $\sigma_{sd} = 365$ (N/mm^2) Betonstaal.nl, 2024
- $f_{bd} = 2.7$ (N/mm^2)
- $\phi = 12$ (mm)

$$l_{b,rqd} = \frac{12}{4} \cdot \frac{365}{2.7} \approx 406\,\text{mm}$$

A.12.3. Verification of the minimum overlap length ($l_{0,min}$)

The minimum overlap length is determined as:

$$l_{0,min} \geq \max\{15 \cdot \phi, 200\,\text{mm}, 0.3 \cdot \alpha_6 \cdot l_{b,rqd}\} \tag{A.67}$$

$$l_{0,min} \geq \max\{15 \cdot 6, 200, 0.3 \cdot 1.5 \cdot 406\} = \max\{90, 200, 183\} = 200\,\text{mm}$$

A.12.4. Calculation of the overlap length (l_0)

The overlap length is calculated using:

$$l_0 = \alpha_1 \cdot \alpha_2 \cdot \alpha_3 \cdot \alpha_5 \cdot \alpha_6 \cdot l_{b,rqd} \tag{A.68}$$

where:

- $\alpha_1 = 1$ Straight overlap.
- $\alpha_2 = 1$ Assumed low concrete cover.
- $\alpha_3 = 1$ Straight overlap.
- $\alpha_5 = 1$ Assuming that compressive stresses do not significantly contribute to the bond effect.
- $\alpha_6 = 1.5$ Overlap in the same cross section.
- $l_{b,rqd} = 406$ (mm)

$$l_0 = 1 \cdot 1 \cdot 1 \cdot 1 \cdot 1.5 \cdot 406 = 609 \text{ mm}$$

A.13. Reinforcement cross-sectional ratio of a beam in Stationsplein 107

To determine the reinforcement cross-sectional ratio of a beam in Stationsplein 107, the cress section with the most concentrated tensile reinforcement is analysed. In figure A.4 the technical drawing of the beam is shown.

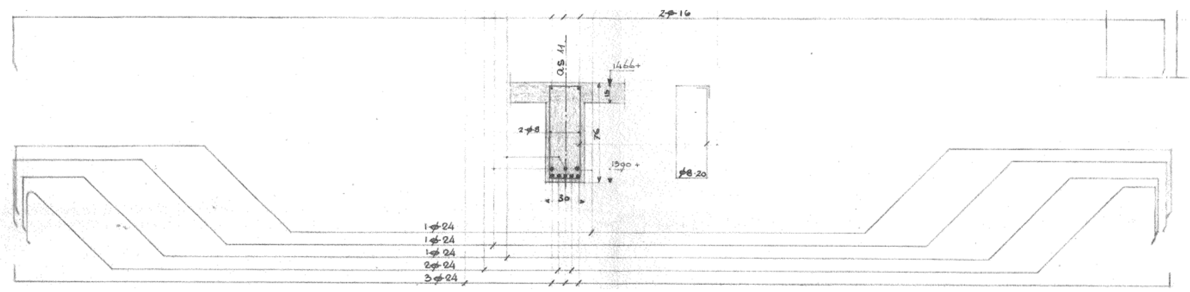

Figure A.4: Cross-section of a beam in stationsplein 107

The effective depth d of the tensile zone is calculated as:

$$d = h - c - \frac{\phi}{2} = 760 - 25 - \frac{24}{2} = 723 \text{ mm} \tag{A.69}$$

The effective concrete area $A_{c,\text{eff}}$ corresponding to the tensile zone is:

$$A_{c,\text{eff}} = b \cdot d = 300 \cdot 723 = 216900 \text{ mm}^2 \tag{A.70}$$

The total area A_s of the tensile reinforcement in this cross-section is calculated by:

$$A_s = \sum n \left(\frac{\pi}{4} \cdot \phi^2 \right) = 8 \left(\frac{\pi}{4} \cdot 24^2 \right) \tag{A.71}$$

$$A_s = 8 \cdot 452.4 \approx 3619 \text{ mm}^2 \tag{A.72}$$

The reinforcement cross-sectional ratio is calculated by:

$$\rho = \left(\frac{3619}{216900} \right) \cdot 100 \approx 1.67\% \tag{A.73}$$

A.14. Overlap length for the existing horizontal reinforcement in the beam of Stationsplein 107 near by fixed support

The calculation below determines the overlap length (l_0) of the horizontal reinforcement in the cross-section near by the fixed support. The reinforcement layout is shown in figure A.4. The horizontal reinforcement in the beams of Stationsplein 107 consists of ribbed QR 42 rebars with $\phi = 24$, and concrete of quality C25/30. The concrete cover is 25 mm and the dimensions of the beam are 300 mm x 760 mm. The schematic representation of the beam is shown below.

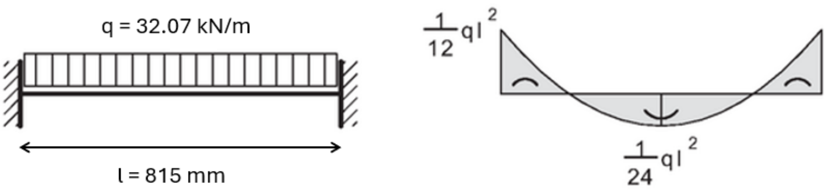

Figure A.5: Schematic representation of the beam in stationsplein 107

A.14.1. Calculation of the effective bond strength (f_{bd})

The calculation is performed for the top reinforcement, as the reinforcement is located at the top near by the supports.

$$f_{bd} = 2.25 \cdot \eta_1 \cdot \eta_2 \cdot f_{ctd} \tag{A.74}$$

where:

- $f_{ctd} = \frac{f_{ctk,0.05}}{\gamma_c} = \frac{1.8}{1.5} = 1.2$ (N/mm^2)
- $\eta_1 = 0.7$ for reinforcement in the top.
- $\eta_2 = 1$ for ribbed reinforcement bars.

This gives a bond strength $f_{bd} = 1.89$ (N/mm^2).

A.14.2. Calculation of the base anchorage length ($l_{b,rqd}$)

The base anchorage length is calculated using:

$$l_{b,rqd} = \frac{\phi}{4} \cdot \frac{\sigma_{sd}}{f_{bd}} \tag{A.75}$$

where:

- σ_{sd} reinforcement stress
- $f_{bd} = 1.89$ (N/mm^2)
- $\phi = 24$ (mm)

The σ_{sd} is calculated using:

$$\sigma_{sd} = \frac{M_{Ed}}{z \cdot A_s} \tag{A.76}$$

where:

- $M_{Ed} = \frac{1}{12}qL^2 = \frac{1}{12}32.07 \cdot 8.15^2 = 177.5 \cdot 10^6$ Nmm
- z = $0.9 \cdot d = 0.9(760 - 25 - \frac{1}{2} \cdot 24) = 651$ mm
- A_s = $8(\frac{\pi}{4} \cdot 24^2) = 3619$ mm^2

This gives a σ_{sd} of 75.34 N/mm^2 which gives a base anchorage length $l_{b,rqd} \approx 239$ mm

A.14.3. Verification of the minimum overlap length ($l_{0,min}$)

The minimum overlap length is determined as:

$$l_{0,min} \geq \max\{15 \cdot \phi, 200\,\text{mm}, 0.3 \cdot \alpha_6 \cdot l_{b,rqd}\} \tag{A.77}$$

$$l_{0,min} \geq \max\{15 \cdot 24, 200, 0.3 \cdot 1.5 \cdot 239\} = \max\{360, 200, 108\} = 360\,\text{mm}$$

A.14.4. Calculation of the overlap length (l_0)

The overlap length is calculated using:

$$l_0 = \alpha_1 \cdot \alpha_2 \cdot \alpha_3 \cdot \alpha_5 \cdot \alpha_6 \cdot l_{b,rqd} \tag{A.78}$$

where:

- $\alpha_1 = 1$ Straight overlap.
- $\alpha_2 = 1$ Assumed low concrete cover.
- $\alpha_3 = 1$ Conservative choice.
- $\alpha_5 = 1$ Assuming that compressive stresses do not significantly contribute to the bond effect.
- $\alpha_6 = 1.5$ Overlap in the same cross section.
- $l_{b,rqd} = 239$ (mm)

$$l_0 = 1 \cdot 1 \cdot 1 \cdot 1 \cdot 1.5 \cdot 239 = 358\,\text{mm}$$

So the overlap length $l_0 \leq l_{0,min}$ which means that the required overlap length is 360 mm

A.15. Anchorage length for the existing horizontal reinforcement in the beam of Stationsplein 107 at moment-zero point

The anchorage length is calculated for the reinforcement in the cross-section at the moment-zero point. This cross-section is shown in figure A.4. Since this concerns the same beam, the concrete and reinforcement properties remain identical to those specified in appendix A.14. The remaining beam length between the moment-zero points is 4.5m.

A.15.1. Calculation of the effective bond strength (f_{bd})

The calculation is performed for the bottom reinforcement, as the reinforcement is located at the bottom in the moment-zero section.

$$f_{bd} = 2.25 \cdot \eta_1 \cdot \eta_2 \cdot f_{ctd} \tag{A.79}$$

where:

- $f_{ctd} = \frac{f_{ctk,0.05}}{\gamma_c} = \frac{1.8}{1.5} = 1.2$ (N/mm^2)
- $\eta_1 = 1$ for reinforcement in the bottom.
- $\eta_2 = 1$ for ribbed reinforcement bars.

This gives a bond strength $f_{bd} = 2.7$ (N/mm^2).

A.15.2. Calculation of the base anchorage length ($l_{b,rqd}$)

The base anchorage length is calculated using:

$$l_{b,rqd} = \frac{\phi}{4} \cdot \frac{\sigma_{sd}}{f_{bd}} \tag{A.80}$$

where:

- σ_{sd} = reinforcement stress
- f_{bd} = 2.7(N/mm^2)
- ϕ = 24(mm)

The σ_{sd} is calculated using:

$$\sigma_{sd} = \frac{M_{Ed}}{z \cdot A_s} \tag{A.81}$$

Since this calculation is performed in the cross-section at the moment-zero point, the shifted moment line must be taken into account. The moment line is shifted over a distance $ai = d$ (NEN-EN 1992-1-1+C2; Section 9.2.1.3 (Nederlands Normalisatie-instituut (NEN), 2011)), which corresponds to the height of the beam. In this case, the shift is 0.76 m.

For a simply supported beam with a uniformly distributed load q, the bending moment at any position x along the span is given by:

$$M_{Ed}(x) = \frac{q \cdot x \cdot (L - x)}{2} \tag{A.82}$$

where:

- $q = 32.07$ kN/m (ULS line load)
- $L = 4.5$ m (remaining span of the beam)
- $x = 0.76$ m (shifted moment position)

Substituting these values:

$$M_{Ed}(0.6) = \frac{32.07 \cdot 0.76 \cdot (4.5 - 0.76)}{2} = 45.58\,\text{kNm}$$

Thus, the bending moment that must be considered for the reinforcement stress at the moment-zero point is 45.58 kNm. To calculate the stress in the reinforcement the following equation is used:

$$\sigma_{sd} = \frac{M_{Ed}}{z \cdot A_s} \tag{A.83}$$

where:

- $M_{Ed} = 45.58 \cdot 10^6$ N/mm
- z = $0.9 \cdot d = 0.9(760 - 25 - \frac{1}{2} \cdot 24) = 651$ mm
- A_s = $8(\frac{\pi}{4} \cdot 24^2) = 3619$ mm^2

This gives a σ_{sd} of 19.52 N/mm^2 which gives a base anchorage length $l_{b,rqd} \approx 43\,$mm

A.15.3. Verification of the minimum anchorage length ($l_{b,min}$)

The minimum anchorage length is determined as:

$$l_{b,min} \geq \max\{10 \cdot \phi, 100\,\text{mm}, 0.3 \cdot l_{b,rqd}\} \tag{A.84}$$

$$l_{b,min} \geq \max\{10 \cdot 24, 100, 0.3 \cdot 43\} = \max\{240, 100, 13\} = 240\,\text{mm}$$

A.15.4. Calculation value of the anchorage length (l_{bd})

The calculation value of the anchorage length needed at an end support is calculated using:

$$l_{bd} = \alpha_1 \cdot \alpha_2 \cdot \alpha_3 \cdot \alpha_4 \cdot \alpha_5 \cdot l_{b,rqd} \tag{A.85}$$

where:

- $\alpha_1 = 1$ Straight overlap.
- $\alpha_2 = 1$ Assumed low concrete cover.
- $\alpha_3 = 1$ Conservative choice.
- $\alpha_4 = 1$ No welded shear reinforcement.
- $\alpha_5 = 1$ Assuming that compressive stresses do not significantly contribute to the bond effect.
- $l_{b,rqd} = 43$ (mm)

This gives an anchorage length $l_{bd} \leq l_{b,min}$, which results in an anchorage length of 240 mm

A.16. Cross-sectional reinforcement ratio of a 1 m floor element from Stationsplein 107

To determine the reinforcement cross-sectional ratio of a 1 m wide floor element, the following calculations are performed. The thikness of the floor is 150 mm.

The total concrete area A_c of the floor element is calculated as:

$$A_c = b \cdot h = 1000 \cdot 150 = 150000\,\text{mm}^2 \tag{A.86}$$

The total reinforcement area A_s is calculated for longitudinal reinforcement with a diameter of $\phi = 12$ mm spaced at 108 mm.

$$A_s = n \cdot \left(\frac{\pi}{4} \cdot \phi^2\right) \tag{A.87}$$

The number of reinforcement bars n in a 1 m wide floor element is:

$$n = \frac{1000}{120} \approx 9\,\text{bars} \tag{A.88}$$

Substituting $n = 9$ and $\phi = 12$:

$$A_s = 9 \cdot \left(\frac{\pi}{4} \cdot 12^2\right) \approx 9 \cdot 113 \approx 1017\,\text{mm}^2 \tag{A.89}$$

The reinforcement cross-sectional ratio ρ is calculated as:

$$\rho = \frac{A_s}{A_c} \cdot 100 = \frac{1017}{150000} \cdot 100 \approx 0.68\,\% \tag{A.90}$$

The reinforcement cross-sectional ratio of the 1 m wide floor element, considering only bottom reinforcement, is approximately 0.68%.

A.17. Overlap length for the existing longitudinal reinforcement in the floor of Stationsplein 107 near by fixed support

The calculation below determines the overlap length (l_0) of the longitudinal reinforcement in the floor of Stationsplein 107. A 1 meter wide slab consists of 10 ribbed QR 42 rebars with $\phi = 12$ mm, and concrete of quality C25/30. It is calculated for the section near the supports, so the reinforcement is located at the top. The span of the floor is 5.58 m and the ULS load on the 1 m wide slab is 7.88 kN/m. The thickness of the floor is 150 mm with a concrete cover of 15 mm.

A.17.1. Calculation of the effective bond strength (f_{bd})

The effective bond strength is calculated for the reinforcement in the top.

$$f_{bd} = 2.25 \cdot \eta_1 \cdot \eta_2 \cdot f_{ctd} \tag{A.91}$$

where:

- $f_{ctd} = \frac{f_{ctk,0.05}}{\gamma_c} = \frac{1.8}{1.5} = 1.2$ (N/mm^2)
- $\eta_1 = 0.7$ for top reinforcement.
- $\eta_2 = 1$ for ribbed reinforcement bars.

This gives a bond strength $f_{bd} = 2.25 \cdot 0.7 \cdot 1 \cdot 1.2 = 1.89$ (N/mm^2).

A.17.2. Calculation of the base anchorage length ($l_{b,rqd}$)

The anchorage length is calculated using:

$$l_{b,rqd} = \frac{\phi}{4} \cdot \frac{\sigma_{sd}}{f_{bd}} \tag{A.92}$$

where:

- $\sigma_{sd} =$ reinforcement stress
- $f_{bd} = 1.89$ (N/mm^2)
- $\phi = 12$ (mm)

The σ_{sd} is calculated using:

$$\sigma_{sd} = \frac{M_{Ed}}{z \cdot A_s} \tag{A.93}$$

where:

- $M_{Ed} = \frac{1}{12} q L^2 = \frac{1}{12} 7.88 \cdot 5.58^2 = 20.45 \cdot 10^6$Nmm
- $z = 0.9 \cdot d = 0.9(150 - 15 - \frac{1}{2} \cdot 12) = 116$ mm
- $A_s = 10(\frac{\pi}{4} \cdot 12^2) = 1131$ mm^2

This gives a σ_{sd} of 156 N/mm^2 which gives a base anchorage length $l_{b,rqd} \approx 247$ mm

A.17.3. Verification of the minimum overlap length ($l_{0,min}$)

The minimum overlap length is determined as:

$$l_{0,min} \geq \max\{15 \cdot \phi, 200\,\text{mm}, 0.3 \cdot \alpha_6 \cdot l_{b,rqd}\} \tag{A.94}$$

$$l_{0,min} \geq \max\{15 \cdot 12, 200, 0.3 \cdot 1.5 \cdot 247\} = \max\{180, 200, 111\} = 200\,\text{mm}$$

A.17.4. Calculation of the overlap length (l_0)

The overlap length is calculated using:

$$l_0 = \alpha_1 \cdot \alpha_2 \cdot \alpha_3 \cdot \alpha_5 \cdot \alpha_6 \cdot l_{b,rqd} \tag{A.95}$$

where:

- $\alpha_1 = 1$ Straight overlap.
- $\alpha_2 = 1$ Assumed low concrete cover.
- $\alpha_3 = 1$ Conservative choice.
- $\alpha_5 = 1$ Assuming that compressive stresses do not significantly contribute to the bond effect.
- $\alpha_6 = 1.5$ Overlap in the same cross section.
- $l_{b,rqd} = 247$ (mm)

$$l_0 = 1 \cdot 1 \cdot 1 \cdot 1 \cdot 1.5 \cdot 247 = 371 \,\text{mm}$$

This gives an overlap length l_0 of 371 mm

A.18. Anchorage length of the existing longitudinal reinforcement in the floor of Stationsplein 107 at moment-zero point

The anchorage length is calculated for the reinforcement in the cross-section at the moment-zero point. Since this concerns the same floor as used in appendix A.17, the concrete and reinforcement properties remain identical. The reinforcement in the span direction is not uniformly positioned at the top and bottom but curves in response to the moment distribution, which means that the same reinforcement near by the supports at the top, are at the bottom at the moment-zero points. The remaining beam length between the moment-zero points is 3.26m.

A.18.1. Calculation of the effective bond strength (f_{bd})

The calculation is performed for the bottom reinforcement, as the reinforcement is located at the bottom in the moment-zero points.

$$f_{bd} = 2.25 \cdot \eta_1 \cdot \eta_2 \cdot f_{ctd} \tag{A.96}$$

where:

- $f_{ctd} = \frac{f_{ctk,0.05}}{\gamma_c} = \frac{1.8}{1.5} = 1.2$ (N/mm^2)
- $\eta_1 = 1$ for reinforcement in the bottom.
- $\eta_2 = 1$ for ribbed reinforcement bars.

This gives a bond strength $f_{bd} = 2.7$ (N/mm^2).

A.18.2. Calculation of the base anchorage length ($l_{b,rqd}$)

The base anchorage length is calculated using:

$$l_{b,rqd} = \frac{\phi}{4} \cdot \frac{\sigma_{sd}}{f_{bd}} \tag{A.97}$$

where:

- σ_{sd} = reinforcement stress
- f_{bd} = 2.7(N/mm^2)
- ϕ = 24(mm)

The σ_{sd} is calculated using:

$$\sigma_{sd} = \frac{M_{Ed}}{z \cdot A_s} \tag{A.98}$$

Since this calculation is performed in the cross-section at the moment-zero point, the shifted moment line must be taken into account. The moment line is shifted over a distance $ai = d$ (NEN-EN 1992-1-1+C2; Section 9.2.1.3 (Nederlands Normalisatie-instituut (NEN), 2011)), which corresponds to the height of the floor. In this case, the shift is 0.15 m.

For a simply supported beam with a uniformly distributed load q, the bending moment at any position x along the span is given by:

$$M_{Ed}(x) = \frac{q \cdot x \cdot (L - x)}{2} \tag{A.99}$$

where:

- $q = 7.88$ kN/m (ULS load on 1m wide floor slab)
- $L = 3.26$ m (remaining span of the beam)
- $x = 0.15$ m (shifted moment position)

Substituting these values:

$$M_{Ed}(0.6) = \frac{7.88 \cdot 0.15 \cdot (3.26 - 0.15)}{2} = 1.84\,\text{kNm}$$

Thus, the bending moment that must be considered for the reinforcement stress at the moment-zero point is 1.84 kNm. To calculate the stress in the reinforcement the following equation is used:

$$\sigma_{sd} = \frac{M_{Ed}}{z \cdot A_s} \tag{A.100}$$

where:

- $M_{Ed} = 1.84 \cdot 10^6$ N/mm
- z = $0.9 \cdot d = 0.9(150 - 15 - \frac{1}{2} \cdot 12) = 116$ mm
- A_s = $10(\frac{\pi}{4} \cdot 12^2) = 1131$ mm^2

This gives a σ_{sd} of 14.02 N/mm^2 which gives a base anchorage length $l_{b,rqd} \approx 31$ mm

A.18.3. Verification of the minimum anchorage length ($l_{b,min}$)

The minimum anchorage length is determined as:

$$l_{b,min} \geq \max\{10 \cdot \phi, 100\,\text{mm}, 0.3 \cdot l_{b,rqd}\} \tag{A.101}$$

$$l_{b,min} \geq \max\{10 \cdot 12, 100, 0.3 \cdot 44\} = \max\{120, 100, 13\} = 120\,\text{mm}$$

A.18.4. Calculation value of the anchorage length (l_{bd})

The calculation value of the anchorage length needed at an end support is calculated using:

$$l_{bd} = \alpha_1 \cdot \alpha_2 \cdot \alpha_3 \cdot \alpha_4 \cdot \alpha_5 \cdot l_{b,rqd} \tag{A.102}$$

where:

- $\alpha_1 = 1$ Straight overlap.
- $\alpha_2 = 1$ Assumed low concrete cover.
- $\alpha_3 = 1$ Conservative choice.
- $\alpha_4 = 1$ No welded shear reinforcement.
- $\alpha_5 = 1$ Assuming compressive stresses do not significantly contribute to the bond effect.
- $l_{b,rqd} = 44$ (mm)

This gives an anchorage length $l_{bd} \leq l_{b,min}$, which results in an anchorage length of 120 mm

A.19. Overlap length for the vertical reinforcement of the wall of Stationsplein 107

The calculation below determines the overlap length (l_0) for the vertical reinforcement in the wall of Stationsplein 107. The concrete is of quality C25/30. The reinforcement consists of ribbed QR 42 rebars with $\phi = 10$ mm.

A.19.1. Calculation of the effective bond strength (f_{bd})

The effective bond strength is calculated for the vertical reinforcement.

$$f_{bd} = 2.25 \cdot \eta_1 \cdot \eta_2 \cdot f_{ctd} \tag{A.103}$$

where:

- $f_{ctd} = \frac{f_{ctk,0.05}}{\gamma_c} = \frac{1.8}{1.5} = 1.2$ (N/mm^2)
- $\eta_1 = 1$ for vertical reinforcement.
- $\eta_2 = 1$ for ribbed reinforcement bars.

This gives a bond strength $f_{bd} = 2.25 \cdot 1 \cdot 1 \cdot 1.2 = 2.7$ (N/mm^2).

A.19.2. Calculation of the base anchorage length ($l_{b,rqd}$)

The anchorage length is calculated using:

$$l_{b,rqd} = \frac{\phi}{4} \cdot \frac{\sigma_{sd}}{f_{bd}} \tag{A.104}$$

where:

- $\sigma_{sd} = 365$ (N/mm^2) Betonstaal.nl, 2024
- $f_{bd} = 2.7$ (N/mm^2)
- $\phi = 10$ mm

$$l_{b,rqd} = \frac{10}{4} \cdot \frac{365}{2.7} \approx 338 \text{ mm}$$

A.19.3. Verification of the minimum overlap length ($l_{0,min}$)

The minimum overlap length is determined as:

$$l_{0,min} \geq \max\{15 \cdot \phi, 200 \text{ mm}, 0.3 \cdot \alpha_6 \cdot l_{b,rqd}\} \tag{A.105}$$

$$l_{0,min} \geq \max\{15 \cdot 10, 200, 0.3 \cdot 1.5 \cdot 338\} = \max\{150, 200, 152\} = 200 \text{ mm}$$

A.19.4. Calculation of the overlap length (l_0)

The overlap length is calculated using:

$$l_0 = \alpha_1 \cdot \alpha_2 \cdot \alpha_3 \cdot \alpha_5 \cdot \alpha_6 \cdot l_{b,rqd} \tag{A.106}$$

where:

- $\alpha_1 = 1$ Straight overlap.
- $\alpha_2 = 1$ Assumed low concrete cover.
- $\alpha_3 = 1$ Conservative choice.
- $\alpha_5 = 1$ Assuming that compressive stresses do not significantly contribute to the bond effect.
- $\alpha_6 = 1.5$ Overlap in the same cross-section.
- $l_{b,rqd} = 338\,\text{mm}$

$$l_0 = 1 \cdot 1 \cdot 1 \cdot 1 \cdot 1.5 \cdot 338 = 507\,\text{mm}$$

A.20. Required distance between shear stirrups

The slope of the compression diagonal is limited by $1.0 \leq \cot\theta \leq 2.5$ Nederlands Normalisatie-instituut (NEN), 2011, which corresponds to $21.8° \leq \theta \leq 45°$. The length l_h between the stirrups is determined by the formula $l_h = d \cdot \tan\theta$, where d is the effective height of the beam. Based on the limits of the compression diagonal angle:

$$l_{h,\text{min}} = d \cdot \tan(21.8°) = d \cdot 0.4 \tag{A.107}$$

$$l_{h,\text{max}} = d \cdot \tan(45°) = d \cdot 1 \tag{A.108}$$

A.21. Shear strength verification of a vertical joint with a beam from Munthof parking garage

This verification is performed to assess whether the shear strength of a vertical joint is sufficient in a connection with a beam and a T-shaped column at the moment zero point.

A.21.1. Design shear strength calculation

The shear resistance of the rough vertical joint is calculated as:

$$v_{Rdi} = c \cdot f_{ctd} + \mu \cdot \sigma_n + \rho \cdot f_{yd}(\mu \sin\alpha + \cos\alpha) \tag{A.109}$$

The concrete used in the Munthof garage is C$25/30$, while the beam contains ribbed reinforcement of QR 40 with a diameter of 26 mm. At the moment zero point, 8 rebars are in tension. The ultimate limit state (ULS) load on the beam is $55,45$ kN/m and the remaining length between the moment zero points is 5.8 m. The beam has a cross-section of 600 mm $\cdot$ 600 mm. As described in section 7.1, the joint is assumed to be rough. The cross-section of the beam in the moment-zero point is shown in figure A.6.

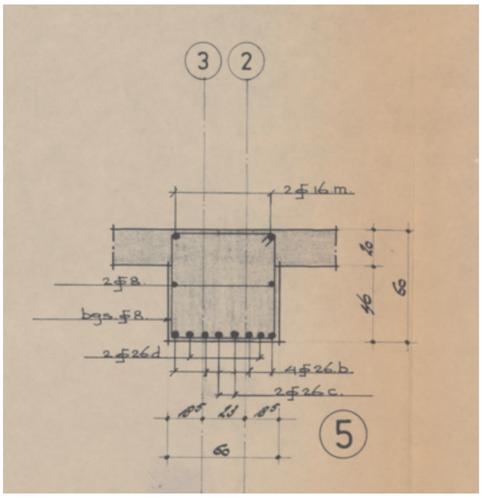

Figure A.6: Cross-section of the beam of Munthof at the shearjoint

where:

- $c = 0.4$ (coefficient for rough interfaces)
- $f_{ctd} = \frac{f_{ctk,0.05}}{\gamma_c} = \frac{1.8}{1.5} = 1.2\,$N/mm^2
- $\mu = 0.7$ (friction coefficient for rough interfaces)
- $\sigma_n = \frac{N}{A_i} = \frac{0}{0.6 \cdot 0.6} = 0\,$N/mm^2 (assuming no axial pressure)
- $\rho = \frac{A_s}{A_i}$, with $A_s = 8 \cdot \frac{\pi}{4}(26^2) = 4.69 \cdot 10^{-3}$ m^2 (only tensile reinforcement considered) and $A_i = 0.6 \cdot 0.6 = 0.36\,$m^2 so $\rho = 0.01303$
- $f_{yd} = 349 N/mm^2$ (QR 40 steel)
- $\alpha = 90° \Rightarrow \sin\alpha = 1, \cos\alpha = 0$ (angle between reinforcement and shear joint)

Substituting these values:

$$v_{Rdi} = 0.4 \cdot 1.2 + 0.7 \cdot 0 + 0.01303 \cdot 349 \cdot (0.7 \cdot 1 + 0) = 3.66\text{N/mm}^2$$

A.21.2. Verification of actual shear stress

The actual shear stress at the joint is calculated as:

$$v_{Edi} = \beta \frac{V_{Ed}}{z \cdot b_i} \tag{A.110}$$

where:

- $V_{Ed} = \frac{q \cdot L}{2} = \frac{55.45 \cdot 5.8}{2} = 160.81$ kN
- $z = 0.9 \cdot h = 0.9 \cdot 0.6 = 0.54$ m
- $\beta = 0.7$ (partial composite action, rough interface)

Thus:

$$v_{Edi} = 0.7 \cdot \frac{160.81 \cdot 10^3}{0.54 \cdot 0.6 \cdot 10^6} = 0.347\,\text{N/mm}^2$$

A.21.3. Final verification

The joint satisfies the shear strength requirement if:

$$v_{Edi} = 0.347\,\text{N/mm}^2 \leq v_{Rdi} = 3.66\,\text{N/mm}^2$$

The shear stress at the joint does not exceeds the design shear strength, indicating that additional shear reinforcement through the joint is not required.

A.21.4. Shear reinforcement design

Since the shear joint is verified to have sufficient capacity, the next step is to ensure that the diagonal force is effectively transferred into the new concrete. To ensure adequate shear transfer, stirrups are introduced in the newly poured concrete, as shown in figure A.7. The required stirrup reinforcement will be determined based on the force distribution in the joint.

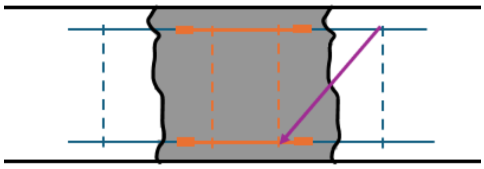

Figure A.7: Scheme of the shear reinforcement design

The required tensile force in the stirrups is determined as:

$$T = \frac{V_{Ed}}{\sin\theta} \tag{A.111}$$

where:

- $V_{Ed} = 15.19$ kN (shear force at the joint)
- $\theta = 21.8°$ (conservative angle for shear force path)

Substituting these values:

$$T = \frac{15.19}{\sin 21.8°} = 40.90\,\text{kN}$$

The required reinforcement area is calculated as:

$$A_s = \frac{T}{f_{yd}} \tag{A.112}$$

where $f_{yd} = 500$ MPa (design yield strength of B500B steel, added in the joint). Substituting:

$$A_s = \frac{40.90 \cdot 10^3}{500} = 81.81\,\text{mm}^2$$

Thus, one $\phi 12$ mm bar ($A_s = 113.10$) should be placed in the new poured concrete to ensure structural adequacy.

A.22. Shear strength verification of a vertical joint with a beam from Stationsplein 107

This verification is performed to assess whether the shear strength of a vertical joint is sufficient in a connection with a beam and a T-shaped column at the moment zero point. The cross-section of the beam in Stationsplein 107 is considered at the moment zero point.

A.22.1. Design shear strength calculation

The shear resistance of the rough vertical joint is calculated as:

$$v_{Rdi} = c \cdot f_{ctd} + \mu \cdot \sigma_n + \rho \cdot f_{yd}(\mu \sin\alpha + \cos\alpha) \tag{A.113}$$

The concrete used in the Stationsplein structure is C$25/30$, while the beam contains ribbed reinforcement of QR 42 with a diameter of 24 mm. At the moment zero point, 8 rebars are in tension. The ultimate limit state (ULS) load on the beam is 32.07 kN/m and the remaining length between the moment zero points is 4.5 m. The beam has a cross-section of 300 mm $\cdot$ 760 mm. As described in section 7.1, the joint is assumed to be rough.

where:

- $c = 0.4$ (coefficient for rough interfaces)
- $f_{ctd} = \frac{f_{ctk,0.05}}{\gamma_c} = \frac{1.8}{1.5} = 1.2$ N/mm^2
- $\mu = 0.7$ (friction coefficient for rough interfaces)
- $\sigma_n = \frac{N}{A_i} = \frac{0}{0.3 \cdot 0.76} = 0$ N/mm^2 (assuming no axial pressure)
- $\rho = \frac{A_s}{A_i}$, with $A_s = 8 \cdot \frac{\pi}{4}(24^2) = 3.62 \cdot 10^{-3}$ m^2 (only tensile reinforcement considered) and $A_i = 0.3 \cdot 0.76 = 0.23$m^2 so $\rho = 0.0157$
- $f_{yd} = 365$ MPa (QR 42 steel)
- $\alpha = 90° \Rightarrow \sin\alpha = 1, \cos\alpha = 0$ (angle between reinforcement and shear joint)

Substituting these values:

$$v_{Rdi} = 0.4 \cdot 1.2 + 0.7 \cdot 0 + 0.0157 \cdot 365 \cdot (0.7 \cdot 1 + 0) = 4.49 \text{ N/mm}^2$$

A.22.2. Verification of actual shear stress

The actual shear stress at the joint is calculated as:

$$v_{Edi} = \beta \frac{V_{Ed}}{z \cdot b_i} \tag{A.114}$$

where:

- $V_{Ed} = \frac{q \cdot L}{2} = \frac{32.07 \cdot 4.5}{2} = 72.16$ kN
- $z = 0.9 \cdot h = 0.9 \cdot 0.76 = 0.684$ m
- $\beta = 0.7$ (partial composite action, rough interface)

Thus:

$$v_{Edi} = 0.7 \cdot \frac{72.16 \cdot 10^3}{0.684 \cdot 0.3 \cdot 10^6} = 0.35 \text{ N/mm}^2$$

A.22.3. Final verification

The joint satisfies the shear strength requirement if:

$$v_{Edi} = 0.35\,\text{N/mm}^2 \leq v_{Rdi} = 4.49\,\text{N/mm}^2$$

The shear stress at the joint does not exceeds the design shear strength, indicating that additional shear reinforcement through the joint is not required.

A.22.4. Shear Reinforcement Design

Since the shear joint is verified to have sufficient capacity, the next step is to ensure that the diagonal force is effectively transferred into the new concrete. To ensure adequate shear transfer, stirrups are introduced in the newly poured concrete, as shown in figure A.7. The required stirrup reinforcement will be determined based on the force distribution in the joint.

The required tensile force in the stirrups is determined as:

$$T = \frac{V_{Ed}}{\sin\theta} \tag{A.115}$$

where:

- $V_{Ed} = 5.32$ kN (shear force at the joint)
- $\theta = 21.8^\circ$ (conservative angle for shear force path)

Substituting these values:

$$T = \frac{5.32}{\sin 21.8^\circ} = 14.33\ \text{kN}$$

The required reinforcement area is calculated as:

$$A_s = \frac{T}{f_{yd}} \tag{A.116}$$

where $f_{yd} = 500$ MPa (design yield strength of B500B steel, added in the joint). Substituting:

$$A_s = \frac{14.33 \cdot 10^3}{500} = 28.65\ \text{mm}^2$$

Thus, one Ø8 mm bar ($A_s = 50.27$ mm^2) should be placed in the new poured concrete to ensure structural adequacy.

B

Additional explanation of MCA scoring criteria

This appendix provides an explanation of the MCA scores. The majority of the criteria are evaluated using a qualitative scale of Low, Medium, Substantial and High. Below, each criterion is elaborated in more detail, explaining the reasoning behind the assigned scores and their implications.

B.1. Element modification

Definition: This criterion refers to any adjustments applied to a reused structural element before it can be integrated into the new connection. These modifications are carried out off-site in a controlled environment.

- **Low:** Minimal modifications, limited to drilling for post-installed reinforcement.
- **Medium:** Medium modifications involve slightly more adjustments. In this case, it refers to exposing reinforcement using hydrodemolition.
- **Substantial:** Substantial modifications involve exposing reinforcement using hydrodemolition, along with welding a single plate at the ends for anchorage.
- **High:** High modifications involve exposing reinforcement using hydrojetting, welding a steel plate at the ends for anchorage and casting additional concrete extension to secure the anchorage.

B.2. Standard connection technique

Definition: This criterion assesses whether the proposed connection method aligns with conventional construction practices commonly used in the industry. A connection is considered ”standard technique” if it relies on assembly methods that do not require innovative construction approaches. It specifically considers the way the connection is put together on-site, excluding any modifications required for the reused elements themselves.

For this criterion, the scoring is binary: ”Yes” indicates that the connection follows traditional techniques, while ”No” signifies that it involves unconventional or new methods that may require additional expertise or adjustments in construction processes.

B.3. Construction effort

Definition: This criterion evaluates the level of effort required to assemble the connection on-site, considering factors such as labor intensity, complexity, and time consumption. It specifically focuses on the ease or difficulty of executing the connection once all necessary modifications to the reused elements have been completed.

- **Low:** The elements are simply placed on top of each other and mechanically connected, requiring minimal additional work.

- **Medium:** : The elements require some alignment adjustments during placement before being mechanically connected.
- **Substantial:** The connection requires anchorage techniques on-site, but only minimal in-situ casting is needed to complete the assembly.
- **High:** The connection demands significant construction effort, involving multiple steps as in-situ casting of structural elements and anchorage techniques at the construction site.

B.4. Overdimensioning

Definition: Overdimensioning assesses to what extent the original element's material properties and dimensions exceed the requirements of the new design. High overdimensioning can lead to inefficiencies such as excessive material usage, increased weight, and unnecessary structural capacity, which may complicate handling and integration. A low level of overdimensioning indicates that the reused element closely matches the structural demands, minimizing excess material and optimizing its performance in the new context.

- **Low:** The reused element closely matches the structural demands in the new design, with no significant unnecessary capacity.
- **Medium:** The reused element has some excess capacity, but this does not significantly impact efficiency or integration.
- **Substantial:** The element has considerable excess material or capacity, leading to noticeable inefficiencies in structural utilization and integration.
- **High:** The element is significantly overdimensioned, resulting in major inefficiencies, excessive material use, and challenges in integration.

B.5. Required precision

Definition: This criterion assesses the level of accuracy needed during the placement, and assembly of the connection to ensure proper fit and structural performance.

- **Low:** The connection allows for substantial tolerances in element positioning and no reinforcement alignment is needed.
- **Medium:** Some precision is required, but small adjustments can be made on-site to correct misalignment due to slotted holes in the mechanical fasteners.
- **Substantial:** The connection requires careful placement with limited tolerances, including alignment of some reinforcement connectors.
- **High:** Very tight tolerances are necessary for a lot of reinforcement alignment.