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- Stefanie Poon

Longlisted World's OK-est Thesis Prize 2021

Glass that strengthens concrete, facilitates change, and tells stories

A love letter to the specters
of the Fenix II, Rotterdam

A perspective on heritage and future use

An investigation of structural glass
to strengthen historic concrete

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Bachelor of Science in Architecture
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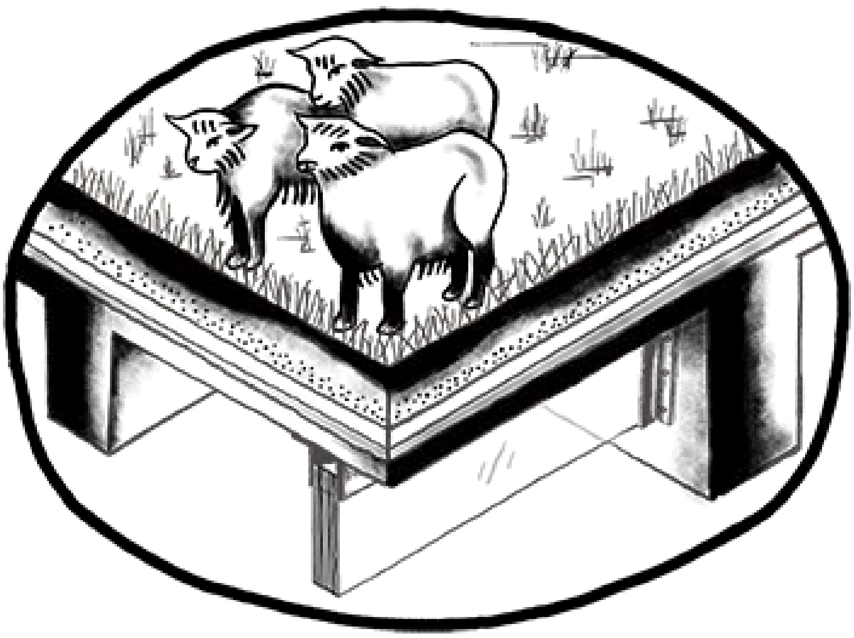
Abstract

This text is an investigation of process. It outlines the design development of a historic concrete slab retrofit using glass. By working through iterations that build off each other, it chronicles the steady unfolding of the concept coming into being. Materiality, statics, construction, and aesthetics are the parameters that are mediated in a case study intervention.

The divide between preservation and progress, between the currently clear-cut choices of conservation or structural design, is a powerful space for transformation. The research looks in depth into the existing historic structure, recalculating it using modern methods. In turn, this informs geometry and configuration, which are iteratively redesigned to be minimal, respectful, and surgical.

Foundationally, this project is a love letter to the ghosts in the hall, who are tasked with handing down their teachings to the future generations, so they better understand their responsibility to the universe and that all things past and present are equal parts of the whole.



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In all the younger versions of myself who couldn't wait to be me at this age right now, I never foresaw this trajectory – to a thesis, the spell of glass, and this particular project. I owe it to everyone who has been in my corner, the source of any strength, grit, and luck I might have.

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The low-tech SF that is *magic* - its fringe intellectuals, poets, artists, and revered eccentrics, who know glass and concrete better than any engineer. Together, they taught me to think of myself as a scientist, to listen for the fog horn, to be *brave*, and to clean my desk.

And my parents, who should've named me *Trouble*, who have given me more than they'll ever know.

ʻĀ ʻOIA!

Contents

1 Introduction

With glass as the protagonist, this thesis meditates on structural design in building reuse, drawing reflection on ways of preserving a place's ghosts and of tectonic conversation to provoke a metanoia, an addendum, a change of medium. There is a fascination with glass that has to do with strength and unpredictability and value and sculpture and its ability to *last* and age gracefully. Wow, I could be talking about myself here.

Enough small talk, now let's chop this chayote.

1.1 Background

The prevalence of glass in structural applications has become more apparent in the last decades. Its virtues, appealing in combination, are its durability, compressive strength, and that we can see through it. It can be said that its antonym is concrete, a material developed for industry over beauty, that came to characterize the design language of many 20th century factories, hangars, power stations, and warehouses. But the paradox of reinforced concrete, writes the architect François Botton, is a material that projects an image of solidity, but proves to be fragile. Concrete and glass together are a polemic, in that the two are architectural and material opposites, but also a lyric, in that the union of their contradictions brings about power and possibility.

1.2 Societal and Scientific Relevance

The subject of this thesis stands between the fields of structural design and heritage conservation. Its scope is underpinned by an egalitarian approach to technology and heritage, identifying the ways in which one discipline forgoes the other so that it can link the two.

The research aims to further the potential of glass in structural applications by promoting its use over other materials (typically concrete, FRP, or steel) in historic building retrofit. It challenges the conventional interpretations of contemporary conservation theory to introduce a fourth option, that is also appropriate as a load-bearing element, to the materiality debate. It reviews the existing literature on adaptive reuse, industrial architecture, the material glass, and historic concrete to form a layered perspective on the subject that pushes it toward feasible application.

It is also an appeal to stakeholders involved in redevelopment: to see old concrete buildings as culturally valuable, to take on the more difficult task of a sensitive retrofit, and ultimately to invest in architectural value that people will demand if it is granted to them. It includes a study of the international, national, and regional views toward how existing buildings, scheduled historic or not, should be regarded to argue for a structural glass intervention that is more consistent with conservation theory and more value-driven.

1.3 Problem Statement

1.3.1 Early Reinforced Concrete Structures for Reuse

Reinforced concrete as a building material came about in the late 19th century. The international scientific understanding of the composite behavior of concrete and steel developed concurrently as it was being applied in the built environment. While this is also true of steel structures, a distinction can be made in the monolithic nature of reinforced concrete in that it is an intrinsically continuous system, while steel can be more forgiving if connections are assumed to be hinged. The tendency toward fixed connections of early concrete design often led to stresses and vulnerabilities in unplanned places that engineers of the time did not fully understand (Friedman 2010). Additionally, an empirical application of the material commonly results in early degradation and corrosion. The legacy of historic concrete construction preceding theory is marked by adverse effects that are felt relatively prematurely, to an extent that concrete repair is its own industry.

The other facet of early reinforced concrete is the building typology for which they were originally designed. Many buildings and civil infrastructure of the Second Industrial Revolution up through World War I (1870s - 1914 in Europe) were built of the newly developed material, reflecting a wave of globalization and technology. By the 1970s and 1980s, Western Europe slid into deindustrialization and swaths of the socio-economic structure became obsolete. This can be attributed to the trend toward outsourcing industry and production to outside of Europe, exploiting cheaper labor in developing countries. Consequently, many of the structures that were built specifically to support industry, some stretching as far back as the first Industrial Revolution in the 1760s, were suddenly not needed anymore after a century of use. This is evident in the built environment by the abandonment of numerous industrial building typologies, that often occupied central locations in the urban core, to leftover landscapes that were left to decay (Chatzi Rodopoulou 2020).

The combination of inherent design flaws and outdated use leave many concrete buildings slated for demolition. But with the increasing recognition of buildings from the recent past as heritage worth preserving, there is also a growing appreciation for reinforced concrete as a novel construction material of the era (Heinemann, Van Hees, and Nijland 2008). Now that many have become decommissioned or at the end of their useful life, the question of what to do with them lingers. Reuse and concrete industrial buildings appear to go well together, since the structures were designed to carry substantial loads and can be adapted to fit a variety of new program. As the critic Ada Louise Huxtable writes, worthy old buildings are a psychological necessity for the physical continuity of a place and we must preserve them "not as pathetic museum pieces, but by giving them new uses."

1.3.2 Glass for Strengthening

When structural strengthening is required in existing building retrofit, it is almost always with opaque materials such as steel, concrete, and FRP. These methods are either irreversible, call for some removal of the existing material, or alter the experiential aesthetic of the historic fabric. While the treatment of heritage buildings is not written into the Codes, several international guidelines and conservation charters form the basis of how interventions are regulated. A building's identity and authenticity are the central issues, where the appropriate introduction of new material and the treatment of old material are the everlasting points of contention.

The trend of glass in contemporary structural applications pushes a transparent solution toward the field of heritage conservation. The transparency of glass, as an immaterial, can persuasively step around the question of materiality through an assembly of diaphanous elements that have the illusion of being delicate. The basic material properties of glass exceed those of unreinforced concrete, specifically of note in compression, which warrants an investigation into its potential across the spectrum of structural uses. Combine this with the number of derelict concrete buildings waiting to be adapted for new uses, and therein lies an opportunity to explore the possibility of glass to strengthen existing elements.

1.4 Research Questions

In a case for glass as a compelling structural medium in building reuse design,
the main research question is:

To what extent can structural glass components be used to strengthen concrete heritage buildings, in lieu of the prevailing conventional methods?

Then, the research can be framed by a layering of subquestions:

- How can early 20th century concrete structures be adaptively reused, where both the technical and historic context are mediated?
    - How is early reinforced concrete characterized?
    - How can the heritage value of historic concrete be evaluated in the Dutch context?
    - How have realized examples of historic concrete interventions addressed both structural performance and aesthetic compatibility? How have they not?
- How can the different material properties of glass and concrete be leveraged to create a structurally unified intervention?
    - How is the structural use of glass in historic buildings consistent with conservation theory?
    - What are the advantages and limitations of each material?
    - What might the load situations be and how can the intervention be adjusted to them?
    - How can the system be designed for non-destructive assembly and reversibility?
- How can the constructability, structural safety, and return on investment of the glass intervention set it on a path toward feasible implementation?
    - What is the geometry of the glass structure; how will it be fabricated, then installed?
    - What is the safety criteria the intervention should meet and what secondary mechanisms are in place?
    - What are the financial incentives that justify the use of glass to reinforce historic concrete?

1.5 Objectives

Through a case study, the objective is to depict how transparent load-bearing glass components can be designed, detailed, and validated to work in conjunction with an existing concrete slab, with the aim of understanding the ways in which glass can be used to adapt an existing structure in flexure for a change in use.

Constraints: The test case will not discuss historic concrete repair in depth. It is stipulated that the existing structure is in its original condition, and that the building was constructed per the original documentation and associated codes. This is not to say that these topics are irrelevant, but that their inclusion lengthens the project beyond the time scope.

1.6 Research Methodology

The research begins with a literature review that is the theoretical framework - to devise a methodology of assessing an existing building within its heritage and market context, to describe the state-of-the-art of intervention principles through precedent analysis, and to extract the potentials of each material and how they go together. It is further divided into:

- A survey of the definitions of industrial heritage, the challenges of reuse, and form the argument of using glass in historic buildings.
- A study of historic concrete, with an emphasis on the structural logic of monolithic reinforced concrete construction in the early 20th century.
- A study of glass in structural applications, with an emphasis on mechanical properties. This is to determine the parameters of the material that govern the design intervention.
- Unearthing documentation on the case study building, to contextualize its heritage value and understand the original structural design intent.

Then, the conclusions gathered from the literature review form the design, functional, and safety criteria that informs the first design concept. The configuration of the intervention is evaluated through analytical calculations. Also considering efficiency, installation, and feasibility, the results of the study are described and assessed for how well they meet the criteria, then fed back into the design development. This is an iterative process that is to end with an optimized glass intervention for the specific given condition of the test case.

Finally, the rational explanation and theory behind the solution is used to identify gaps in the research and suggest further investigation on the subject.

Theoretical Framework

2 Industrial Heritage Reuse

2.1 Heritage

Whether old buildings are judged to be "historic" or not, they are reflective of the ideas, values, and culture of a particular past, so any interventive decision should carry this weight. Their presence as part of the urban fabric adds a layer to the city that gives it depth and can be closely tied to its collective identity. This section presents a brief overview of the recent international discourse on heritage building care.

2.1.1 Terminology

Sometimes used interchangeably in everyday life, the terminology used in conservation denote different actions and treatments. A few definitions (Australia ICOMOS 2013) prior to further discussion:

Preservation is to retain the current state of a place to lessen further damage or decay. It is about stabilizing without the addition of new material, so it is about slowing the process of deterioration.

Restoration is to return a place to a specific past state, again without the addition of new material. Reassembly of existing elements and material is allowed. Any features not consistent with the intended time period may be removed.

Reconstruction is to return a place to a specific past state, but is distinct from restoration in that new material is allowed.

Adaptation, which is related to the subject of this thesis, is to alter a place according to its existing use or to a new use.

2.1.2 International Charters and Guidelines

The first modern concept of regulating built heritage care originated in the Athens Charter of 1931, which was the predecessor to the Venice Charter of 1964 that is still influential to this day. As definitions and the discipline of heritage conservation evolved, countries adapted the principles to their own regional contexts (Florence, Washington, Burra, Riga Charters, etc.).

But that the Venice Charter was regarded as the *mother* of all charters for many years is clear in many interventions that took its words as law. However, much is lost in the nuance of translation from the original French and its missing legal-style diction. Unintentionally vague language is one of the main culprits that lead to liberal interpretation, that stem from an apparent gap between the letter and the spirit of the document (Dawans and Houbart 2016).

Take the example of the infamous "contemporary stamp" introduced in Article 9, originally written as *la marque de notre temps* or *the mark of our time* (ICOMOS 1964). A loose description of the expression and the direction that new work is to reflect its own time gave way to interpretations that range from conservative (the consolidation of the Parthenon using like materials) to deconstructivist (Daniel Libeskind's *Bundeswehr* Military History Museum).

The reference to "harmony between old and new" of Article 12 is typically more precisely defined in country-specific translations. Article 10 of the 1987 Washington Charter also gives an elaboration: "the existing spatial layout should be respected, especially in terms of scale and lot size. The introduction of contemporary elements in harmony with the surroundings should not be discouraged since such features can contribute to the enrichment of an area" (ICOMOS 1987). Article 3 of the 2013 Burra Charter appears to be in agreement by stating that "changes to a place should not distort the physical or other evidence," but that interventions should change no more than what is necessary (Australia ICOMOS 2013).

Two other points relevant to this thesis are described in the 2003 Victoria Falls Charter. §3.7 states the preference for the interventive technique[1] that is least invasive, most compatible with the historic fabric, and best meets the safety and durability criteria. §3.9 states that reversibility must be designed into the intervention where possible, so that it can be replaced in the future, should the technology advance (ICOMOS 2003).

Reconstruction is not acceptable in the Venice Charter but is carefully proposed with conditions for its specific application in the 2000 Riga Charter, as Latvia is a place that has seen the trauma and erasure of monuments that come from a long history of occupation. The later loss of other culturally significant sites, whether due to deliberate destruction (Palmyra, Aleppo, the Bamiyan Valley) or natural phenomena led UNESCO to also accept reconstruction in exceptional circumstances "on the basis of complete and detailed documentation and to no extent on conjecture" (UNESCO 2019).

As countries wrote their own charters, views on conservation became more specific and the stance taken in the Venice Charter is currently regarded as outdated. Together, modern Charters appear to point toward interventions that are technically viable and telling of its time, yet subtle enough that the experiential aesthetic of the place and the scale of the building elements can be perceived as they were before.

2.2 Dutch Perspectives on Conservation

In the Netherlands, the first organization formed to conserve cultural identity and memory was established in 1875 as the *Kunsten en Wetenschap* (Arts and Sciences) department under the Ministry of Internal Affairs, which evolved into the present day *Rijksdienst voor het Cultureel Erfgoed* (RCE, Cultural Heritage Agency). The conservation of heritage sites was first written into law in the 1961 *Monumentenwet* (Monuments and Historic Buildings Act), which established an official roster of historic places. The Dutch were also among the first to accept its buildings from the recent past as historic, through the establishment of *Docomomo* in 1988

1. Article 10 of the Venice Charter shows a bias toward traditional techniques, which the Victoria Falls Charter supercedes by stating that"'traditional' and 'innovative' techniques should be weighed up on a case-by-case basis."

to protect Modern architecture and urbanism in the spirit of ICOMOS. While heritage care and classification in the Netherlands is multilayered, this section is mostly concerned with the lead-up to industrial heritage reuse.

The contemporary Dutch views on heritage conservation have broadened from modernist preservation-motivated approaches to an emphasis on managing change not only by combining the past with new uses, but also integrating them in the city's spatial planning process (Janssen et al. 2017). As such, it is not enough to reuse an individual building, but the urban landscape should be seen as a living archive where history, sustainability, and development are strongly connected.

(Janssen et al. 2017) believe that the post-War Dutch practices of incorporating conservation in planning can be categorized as: a *sector* where heritage is preserved and the dynamics of spatial planning are seen as a threat, a *factor* where heritage is an asset in spatial planning, and a *vector* where heritage is the determining factor in (re)development. Though the three approaches evolved consecutively, they did not replace each other, but they pushed forward based on different stakeholders. The complexity in evaluating the proper reuse of a building indicates that there is no generally accepted answer.

The trend in the Netherlands toward heritage as vector coincides with the international recognition of intangible values as part of historic significance, and the mutual understanding between developers and heritage groups that they have common interests. Conservation is no longer seen as a field that romanticizes history (conservation for the sake of conservation), rather it also serves a commercial interest that produces added value that can be financially measured (Meurs 2016).

The beginning of the 21st century saw the publication of the *Nota Belvedere* (Belvedere Memo) in 1999, a Dutch policy that had great influence on subsequent heritage management. It acknowledged the tension between heritage conservation and speculative real estate and sought to balance the two through the concept of *behoud door ontwikkeling* (conservation through development), which incentivized large-scale redevelopment of landscapes and structures through government subsidies (Ministerie van OCW 1999). This resulted in an increased partnership between the State and private stakeholders, at once stressing industrial sites as cultural heritage yet encouraging their commodification (Chatzi Rodopoulou 2020; Janssen et al. 2017).

Two other factors affect how Dutch built heritage is currently managed (Chatzi Rodopoulou 2020). First, the consolidation of various government heritage groups into the RCE in 2009 lessened State involvement in social and cultural matters. This decentralization of heritage management gave more power to regional stakeholders and pushed the field toward market-driven forces. Second, the financial crisis of 2008 paused the financial subsidies of redevelopment projects, which in turn went into decline.

To renew interest in the reuse of heritage sites, the Dutch government introduced two new policies of note. The 2009 *Modernisering Monumentenzorg* (MoMo, Modernization of Monument Care) stressed the conservation of cultural significance in the face of economic progress, reiterated the previous guidelines, and promoted reuse. The national program *Herbestemming* (Reuse) that ran from 2010-2015 involved a hierarchy of partners from the State to regional actors to contractors, with the aim of providing subsidies and educating other stakeholders on the subject, including the redevelopment of industrial areas and buildings.

An example can be made of the 1948 5-story *Ter Meulen* building in Rotterdam that was vertically extended and renamed as *de Karel Doorman*. Work started in 2006, was halted for two years in 2008 due to uncertainty in financing and construction issues, then finally completed in 2012. Though the existing concrete plinth is not a protected monument, the team proposed to restore it, remove the two upper floors added in 1977, and add a 70m residential tower above bearing completely on the existing structure. The new tower is wrapped in glass, playing a game with solid and void, tranparency and translucency. Like a veil, the fritted glass block is an abstract volume that blends with the sky beyond and recedes from the existing historic plinth below.

2.3 Industrial Structures as Cultural Heritage

The significance of industrial buildings is not so much in their association with certain people or events, though some were designed by prominent architects, but more so as evidence of a vernacular architecture forged from the production and economic past of a region. Mostly constructed for pragmatic purposes, they often contain vast, open spaces that are both bland and brazen in their departure from traditional design elements. It is this multiplicity that makes them so uninteresting yet charming at the same time.

The slow recognition of their worth is seen in the loss of many industrial buildings in Western Europe beginning in the 1970s, under the guise of urban renewal. The razing of the old Paris market halls, specifically Victor Baltard's *Les Halles* in 1971, was followed with immediate regret that made France one of the early advocates for the protection of industrial heritage. It was only at their destruction that their role in piecing together a complete urban history of Paris was sanctified, giving way to the view that the post-industrial city is ingrained with culturally significant artifacts accrued from a long tradition of use (Weiss 2009). This shift in attitude influenced the more recent redevelopment and reuse of *Gare d'Orsay* and of many warehouses in the 13th arrondissement bordering the Seine, where an expansive former industrial neighborhood stretches from *Gare d'Austerlitz* to the eastern end of the city center.

In Eastern Europe and the Baltic states, particularly those that were under the influence of the former USSR, deindustrialization came later and was more related to the political shift in 1989, and less so because of outsourced industry (Šolks, Dejus, and Legzdinš 2012). What used to be the city fringes and now the center of Riga grew reflexive of a very rapid industrialization from the second half of the 19th century up to World War I. The decreased reliance on manufacturing and the railroad belt in the 1990s, and the end of occupation left many working-class neighborhoods abandoned or derelict. In all this, private developers saw two advantages: large plot areas in a central location and that though the industrial sites were recognized as culturally significant, they were not subjected to the same laws as other listed monuments - paving the way for the profit-driven redevelopment of the buildings into new residential and commercial functions (Šolks, Dejus, and Legzdinš 2012).

But an official recognition of industrial cultural heritage was not until recently and the first international guideline for their conservation was issued in 2003 as the Nizhny Tagil Charter for the Industrial Heritage (TICCIH 2003). A more comprehensive document came about in 2011 as the Principles for the Conservation of Industrial Heritage Sites, Structures, Areas and Landscapes (referred to as the Dublin Principles) and was jointly issued by (ICOMOS-TICCIH 2011). Besides providing basic definitions and directions for conservation, the two guidelines also point out technological and scientific values that are unique to the industrial typology.

The Dublin Principles specifically calls for conservation of significant features and patterns of circulation and activity (Art. III.10), documentation of changes (Art. III.11), reversibility (Art. III.11), and functional integrity and machinery conservation (Art. II.9). In accordance with Article III.10, and as exemplified in built applications, the "appropriate original or alternative and adaptive use is the most sustainable way of ensuring the conservation of Industrial Heritage sites or structures" (ICOMOS-TICCIH 2011). The word *sustainable* appears to be consistent with the shift in expectation that industrial relics have a role in urban regeneration and that the pool of stakeholders is expanded to include governmental and private Actors, which the scholar Françoise Choay calls the *industrialization* of heritage (Chatzi Rodopoulou 2020, citing Miles Glendinning). So, it can be concluded that the alteration of the former use of an existing industrial building is the most appropriate way of ensuring its survival.

The early instances of adaptive reuse in Western Europe in the 1980s and Eastern Europe in the 1990s saw little care for heritage conservation, and therefore took place in industrial sites that were perceived to be of negligible historic value. The proliferation of market-driven reuse in the early 2000s not only saved the industrial buildings from demolition, but also often resulted in their treatment as vessels for architectural experimentation and destructive speculative redevelopment (Chatzi Rodopoulou 2020). Nevertheless, the place of industrial architecture in heritage conservation was recognized around the 1970s and solidified in the 21st century through the publication of the Nizhny Tagil Charter and Dublin Principles.

2.4 Building Reuse

The main goal of existing building adaption is not to preserve or restore, rather it aims to extend the life of the building by modernizing it for new uses (Chatzi Rodopoulou 2020; Hein and Houck 2008). To this end, there is still a bias toward minimal intervention as specified in the Charters and a recognition that the making of the historical significance of a building is through its sense and spirit of place, or *genius loci*, and authenticity. So, these characteristics should be defined before any intervention design.

2.4.1 Sense and Spirit of Place

The story of the deep-rooted tie between people and place has been told many times. The writing of Bachelard in *The Poetics of Space*, Heidegger in "Building Dwelling Thinking," and Borges in "El Sur" come to mind. But a building's identity is easily threatened in changes of use.

The identifying values of a place have been difficult to scientifically analyze because of the complex relationship between the tangible and intangible. The spirit of a place can be defined as emotion materialized in the architecture that is outside us, while sense of place is the feeling that the architecture provokes (Yazdani Mehr and Wilkinson 2020). Both can change over time and mean different things to different people (Yazdani Mehr and Wilkinson 2020). Perhaps it is similar to the concepts of *voice* and *tone* in literary theory.

An example:
Built in 1907 as a public pool and reconstructed in 1960, the *Stadtbad Wedding*[1] in Berlin was reincarnated as *Stattbad* [2] in 2009 into a palace of art, exhibitions, performance, and its most beloved feature, a nightclub. As such, it was a portrait of the city at its most imaginative. Though the new program deviated from the former, the original feeling of Swimming Pool did not equivocate (Fig. 2.1). The *genius loci* of the place was preserved through conspicuous hints to its past life: large glass observation panels, signage pointing to the *Erfrischungsraum* (cafeteria) that is no longer there, and the way the light hit the weathered and dated finishes.

Fig. 2.1: Left: Robert Montgomery: All Palaces, LED, Berlin DE, 2012. Right: The *Schwimmenmeister's* office as a kitchen, Stattbad (Smith & Peony Press, 2012)

2.4.2 Authenticity

The other totem of building identity, authenticity, is related to form, materials, function, setting, workmanship, and spirit and feeling (ICOMOS 1994; UNESCO 2019). Rather than just the original construction, a number of scholars believe this to also include all subsequent modifications over time, tangible and intangible, which can be used to convey sense and spirit of place (Jokilehto 2002, 2013). In the case of reuse, the original authenticity of a place will change as time and function also change. The preservation of authentic features and values can protect the spirit of a place, but sense of place is a more subjective feeling dependent on people and requires community engagement to identify. When assessing the authenticity of a place for reuse, (Yazdani Mehr and Wilkinson 2020) suggest to consider whether:

- The style (Art Deco, Modernist, Romanesque, etc.) and materials (clay, wood, concrete, ornamentation, etc.) are representative of a specific time period
- The new function is based on the original use and if the original use was of significance
- Previous adaptations represent a specific time period (the Tower of London that was a prison that became a royal palace, then a tourist attraction)
- The construction represents a technique of a specific time period (heights, spans, etc.)

The successful preservation of a building's *genius loci* and authenticity is contingent on the understanding that the two are linked.

1. The building has since been demolished after it was shut in 2015 due to noncompliant egress paths, lack of emergency preparedness, and what the City deemed an inappropriate use of the building layout for a nightclub.

2. The new name, where *statt* means "instead of" and *bad* is "pool," alludes to the former.

2.4.3 Structural Sustainability and Construction

Beyond conserving a building's heritage values, the reuse of an existing building reduces the environmental impact of demolition and new construction by lessening demand on material and transport. If the system can be methodically disassembled to maximize the number of salvaged components while retaining their value, it can be 1. more likely to be adapted rather than replaced, 2. able to be reused in other applications, and 3. easily separated into specific waste streams (Kestner and Webster 2010; Maydl 2006). Because structure can be thought of as a skeleton that will remain after all other building systems are gone, the order of deconstruction should also be related to the order of construction (Le Ricolais 1996) and consider access, maintaining safety, and the necessary equipment. While conservation theory already emphasizes reversibility, it also facilitates the overall sustainability of the intervention.

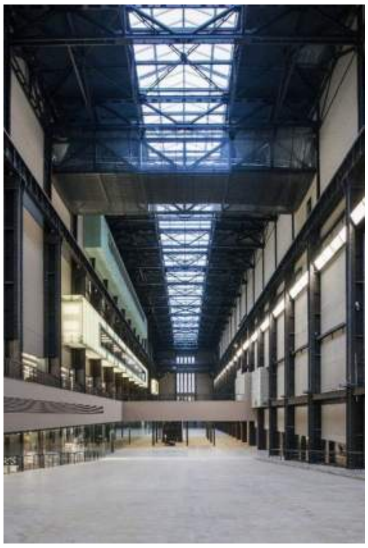
However, the issues involved in adapting an older structure can be more expensive and time-consuming than starting over. The jurisdiction, the associated preservation groups, and the owner must agree on what is kept, what may be demolished, and the extent of the intervention to make the project feasible. The impact of the intervention on structural capacity and heritage values must be identified (Hein and Houck 2008). Changes in use should also consider modernizing other building systems, architectural interiors, and fire safety[1] and evacuation measures. Additionally, pre-construction investigations may reveal that significant work would be required at existing elements to comply with modern codes. The discovery of toxic building components, hidden structural conditions, and surgical cutting and patching around historic features can all result in added cost and time delays. As such, the act of intervening often leads to accompanying work.

Discrepancies are common in the historic methods of calculating and dimensioning structure. In some cases, elements were over-designed based on proven past performance rather than scientific understanding, resulting in very large cross-sections with high residual capacity. The structural integrity of the building should be assessed as a whole to ensure adequate strength and stability of materials, gravity, and lateral systems. Foundations are likely to require retrofit if higher loads are proposed, as they are often designed with the least bearing capacity compared to other structural elements when calculated using modern methods (see §5.1.2).

The Tate Modern (Fig. 2.2) in London is a precedent for the long-term successful adaptation of an industrial building. The site of the former Bankside Power Station on the South Bank is significant as it is directly across the River Thames from St. Paul's Cathedral and as of 2000, adjacent to the Millennium Bridge by Foster + Partners. When the power station was decommissioned in 1981, it was left to rust for years without a conservation plan. A debate ensued over whether to redevelop or demolish the building. The historian Gavin Stamp wrote numerous editorials and produced a BBC documentary begging for it to be saved, and was ultimately successful in 1993 (Beaton 2006). The £134 million cost of conversion is offset by the 6 million annual visitors, making it one of the most visited museums in the world. It was a very deliberate choice of the city of London to allow an industrial relic to face an iconic religious monument, which anchored the redevelopment of the surrounding neighborhood.

1. In Oakland, the 2016 Ghost Ship fire occurred in a concrete-block warehouse informally converted to ateliers, residences, and an event space. Probable explanations of the high loss of life include an occupancy type that the building was not designed for, electrical problems, and aggregated detritus that contributed to the fuel load.

(a) North facade from the River Thames, with the former Bankside Power Station in the foreground and the 11-story extension in the background



(b) The Turbine Hall, former location of the power station generators

Fig. 2.2: Herzog & de Meuron: Tate Modern conversion, London GB, 2000

2.5 Transparency and Glass

As implied in the Charters and Guidelines, the goals in conservation are minimal intervention, reversibility, compatibility, durability (ICOMOS 1964; ICOMOS-TICCIH 2011; TICCIH 2003). Transparent interventions meet this criteria by emphasizing the existing elements and features of a building to provide a view into its past (Barou et al. 2018). Without the introduction of new solidity, the hierarchy of existing elements and their relation to each other is preserved. Glass has a long history in tectonic theory, where its intangible appearance is used as a communicative tool.

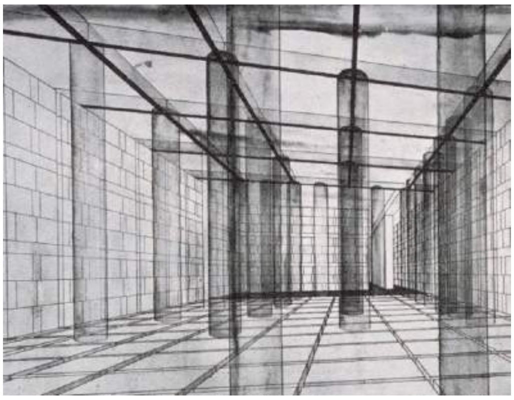
First, a definition of transparency (Rowe and Slutzky 1963).

1. Literal transparency: the see-through kind of material transparency that leaves little to the imagination.

2. Phenomenal transparency: an ordering of planes suggestive of volume and shape, where the observer is asked to complete the thought.

2.5.1 As Dematerialization

The characteristic of glass that separates it from all other materials is that it is an immaterial. It adopts the shape that is given to it, so this shape must be in agreement with its nature. In this way, glass is not subjected to the laws of tectonics, where a physical material is raised to the meta-physical world. The presence of glass is inherent in the material. Its tectonic expression will always be an allusion, said the theorist Karl Bötticher of the shape of glass, that is both its artificial and core shape.



(a) Giuseppe Terragni: Danteum, Rome IT, 1938

(b) Machado Silvetti: Menokin House, Virginia US, 2020

Fig. 2.3: Glass as a tectonic metaphor: Those beams, columns, and walls are not beams, columns, or walls.

For as long as it was established as a building material and not just ornamentation, architects have been bewitched by the formal possibility and transparency of glass:
- As a sublimation of the building envelope in Mies van der Rohe's entry for a skyscraper on the Friedrichstraße in Berlin that foreshadowed the design of every tall building to come.
- As a total-media glass structure in Terragni's draft for the Danteum in Rome (Fig. 2.3a).
- As a suggestion of what used to be, in Machado Silvetti's infill of the Menokin House ruins in Virginia (Fig. 2.3b), which captures the principles of conservation without mimicking or reenacting the past. The decay is stabilized but unresolved, the marks and flaws of the brickwork are laid bare, delivering the ruins "in the full richness of their authenticity" (ICOMOS 1964), yet delineating space and allowing full understanding of the place at the same time.

Attaining transparency is not always through (im)material, rather by the design of voids. The wire-frame objects of Robert le Ricolais, embodied at building scale in the architecture of Toyo Ito[1], have a skeletal character that comes from the manipulation of negative space. Not only through the distribution of elements, but also through the hollowness of the element itself, so as to have no weight. Or as (Le Ricolais 1996) describes it, "the art of structure is how and where to put the holes." To design strength out of what appears to be fragile is the delight of the Sendai Mediatheque's structural diagram (Fig. 2.4), where floors seem to float and dare to answer the question: You can't do that with a bunch of hollow sticks, can you?
Well, a 9.0 earthquake was no match for it.

In a related field, the notion of form through line and void has been employed in historic building interventions. The immaterial, the invisible, and the metaphor are the drivers behind the reconstruction of monuments by dematerialization (Barou et al. 2018). Edoardo Tresoldi's wire mesh representation of the missing pieces of the *Basilica di Siponto* (Fig. 2.5) is a conceptual rebuilding that uses phenomenal transparency to augment the ruins and preserve its sense of place and authenticity.

Glass as a trick. Transparency as a trick.

1. and perhaps also the *strandbeesten* by Theo Jansen, the Golgi Structure by Fumihiko Maki, and the radio towers by Vladimir Shukhov

Fig. 2.4: Toyo Ito: Sendai Mediatheque, Acrylic scale model 1:150, MoMA NYC, 2001

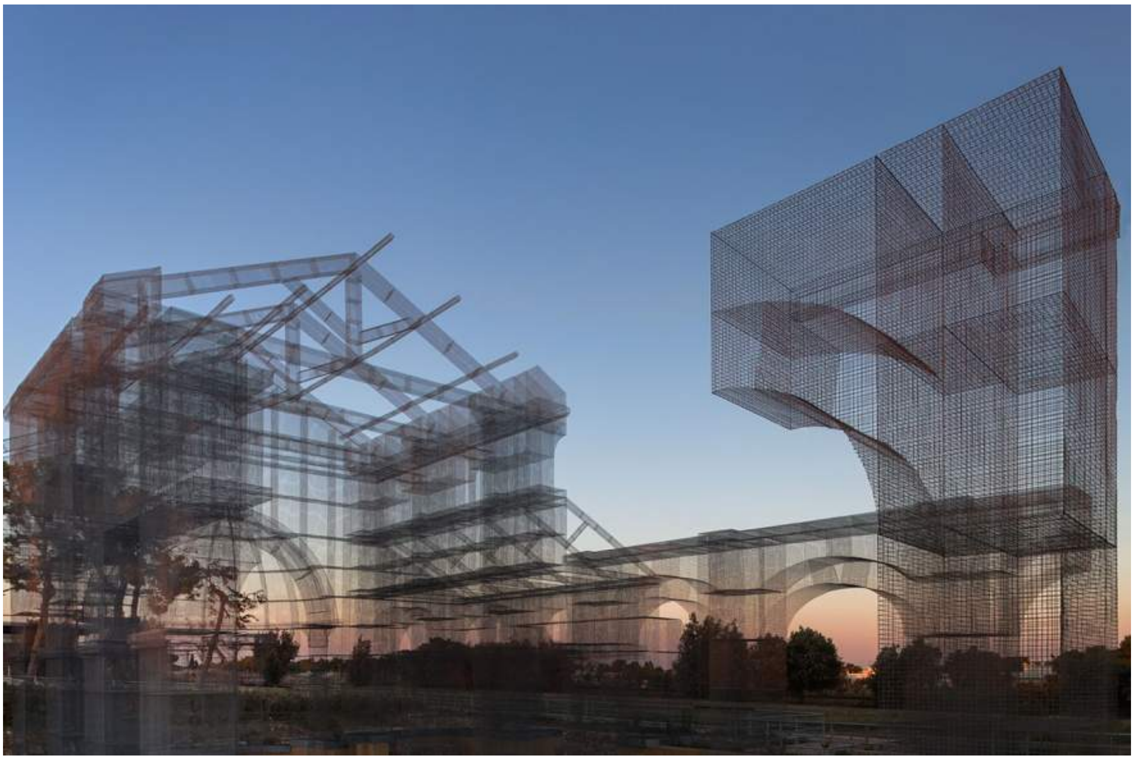


Fig. 2.5: Edoardo Tresoldi: Basilica di Siponto, Wire mesh, Manfredonia IT, 2016

2.5.2 As a Political Statement

The presence and absence of glass has also been used in political gestures.
Here, we should revisit Berlin and the resurrection of the post-DDR *Reichstag*. In contrast to the heavy-handed, megalomaniac vision of Albert Speer's Berlin, the designers of the newly reunified German Republic sought to conjure openness and accountability in the restitching of the new capital (Koepnick 2001). Glass became an exalted construction material in the architecture of post-Wall Berlin, a direct rejection of the giant stone blocks of *Germania* (Koepnick 2001). A well-known instance of this is Norman Foster's 1992 Reichstag dome (Fig. 2.6), where the public walks up a spiral ramp and looks down to the *Bundestag* below - an affirmation that the people are always above the government.

Fig. 2.6: *Das Reichstagsgebäude* from the Platz der Republik, Berlin DE (Kay Nietfeld, 2018)

Of note, Foster's dome is not a complete departure from Paul Wallot's original dome. The colossal steel frame and glass assembly is evocative not only of the massive *Gründerzeit* architecture that was born from the Industrial Revolution, but also the materials of early Modernism (Koepnick 2001). Foster's dome can be read as a palimpsest of the original building, that goes beyond the wars of the 20th century, that actually *adds* a ghost to the city's heritage landscape.

The glass addition is clearly distinct from the existing building and its form does not attempt to reenact the original, so it makes the contemporary statement that is needed to distinguish between old and new. But it does not deviate so far from the original form and proportion so as to tell a separate story. That the original dome was destroyed in war is also significant. The transparency of Foster's dome in contrast to the rest of the building recognizes that something of the original is lost, does not try to erase history, sees the missing dome as part of its evolving authenticity, and conserves this by choosing not to recreate what is gone.

Glass as moving on without forgetting, in response to a painful history.

2.6 Discussion

All adaptations to an existing building for new uses are rooted in a reality that does not only concern the built heritage, but also socio-economic, cultural, and political factors (Hein and Houck 2008). Appropriate reuse is tailored to its regional context, but also value-based (Maydl 2006). Reuse is a practical alternative that is a more sustainable intermediate between demolition and historic designation (Hein and Houck 2008; Weiss 2009). But construction implications of intervening in existing buildings can also have expensive consequences. This thesis is mainly concerned with structural design, but a holistic adaptation may also include retrofitting the plumbing, electrical, mechanical, and other environmental systems of the building. The feasibility of an adaptation hinges on the existing value, estimated cost of construction, and anticipated worth after construction.

The Netherlands has shown an interest in conserving its heritage buildings through development and considers structures from its recent past as culturally valuable (Chatzi Rodopoulou 2020; Janssen et al. 2017; Meurs 2016). Subsidy programs first introduced in the *Nota Belvedere* were the catalyst for the redevelopment of many post-industrial areas and buildings. But the recognition of industrial structures as cultural heritage is a relatively recent concept. The publication of guidelines specifically related to industrial heritage in combination with the other well-known international guidelines will provide the conservation theory basis behind the design intervention.

The points of conservation related to reuse are minimal intervention, reversibility, durability, and respecting significant material (ICOMOS 1964; ICOMOS-TICCIH 2011; TICCIH 2003). This is not to say that it is best if nothing is done; even scholars generally agree that the identity and authenticity of a place is subjective and can change over time (Jokilehto 2002, 2013; Yazdani Mehr and Wilkinson 2020). But a satisfying integration of heritage value conservation and technical performance is missing in practice.

Conservation opinions on interventive approaches according to these principles most often fester into a debate over materiality and how it affects the perception of a space (Barou et al. 2018). Traditional conservation materials that are either similar or identical to the existing do not unequivocally separate the intervention from the historic fabric. Conversely, the separation in structurally driven interventions is too clear. This debate would be over if, like witchcraft, a solution can intervene without resembling traditional or existing building elements and without undermining the authenticity of original and all subsequent alterations, thereby making a sensitive distinction between old and new and avoiding conjecture.

Here, the nature of glass provides a cogent and convincing argument. Transparent glass does not have an inner consciousness[1]. It does not engage in the way a material like copper does, where color and texture can connote place or the passing of time. So then, the un-characteristics of glass make it ideal for use in historic building interventions because it does not push its own parallel narrative, rather it extends the existing one. Ultimately the successful conservation of a place's identity, that also reflects the progress of its time, results in a historically and financially valuable prerequisite that gives cities fundamental dimension, plurality, and a density of experience.

1. Specifically in reference to clear glass in its most basic, archetypal form. For instance, not the Gothic use of glass and color that makes light almost tangible.

3 Historic Concrete as a Construction System

With the ongoing conversation around evaluating the historic significance of aging 20th century buildings, the identifying values and features of a built work that tie it to its era should be defined. Current interventive techniques typically gloss over conservation principles by approaching it as a purely structural puzzle (Heinemann, Van Hees, and Nijland 2008). This integration is missing because concrete falls under "new heritage" and few experts are knowledgeable in both fields.

The development of reinforced concrete and its proliferation at the turn of the century are telling of a style, function, and technique that are specific to the time. In the Netherlands, the concrete structures of the first decades of the 20th century are particularly interesting, as they were the experiments of a material still in development and thereby physical evidence of a society's first advances in a certain technological innovation. The design intervention will concern such a structure, so it is necessary to understand the context and logic behind it.

3.1 Early 20th Century Reinforced Concrete in the Netherlands

Germany, France, and the United Kingdom led the European development of concrete construction. The Netherlands followed their progression, but lagged a little behind. By the end of the 19th century, the Dutch started experimenting more frequently with the material in building applications. It was thought to be maintenance free, had stunning fire resistance, and unleashed a universe of construction potential (Heinemann 2013). The first uses of reinforced concrete came about in defense facilities, warehouses, factories, watertowers, bridges, and public buildings. An even more courageous act, they used it for large infrastructure projects, such as the Hofplein viaduct in Rotterdam (1908). Though it was not so much because of enthusiasm for the material, but an incomplete understanding of its characteristics.

Because of lack of resources within the national boundaries, the Netherlands did not have its own manufacturing facilities and much of the raw material for concrete production was imported from Belgium and Germany. Regulation was difficult in the first decades of the 20th century where tampering and inconsistency was common, particularly with suppliers cutting cement with lesser ingredients (Heinemann and Redactie Betoniek 2012). Because of these issues, there were wide variations in quality until the first Dutch cement factories were established in the 1930s.

The Netherlands also depended on foreign knowledge for calculation and construction methods, as it was initially not easily available. The *Technische Hogeschool Delft* (today *Technische Universiteit Delft*) started teaching reinforced concrete as part of its engineering curriculum in 1905. But vague knowledge around calculating statically indeterminate structures and no references for the elastic theory of concrete mix led to approximate and inaccurate results. Part of this was due to the prevalence of proprietary systems that shrouded calculation methods in secrecy and ambiguity (Heinemann 2013; Marcos et al. 2017).

3.2 Components of the Concrete

The constituents of historic concrete in the Netherlands were initially imported from Belgium and Germany. Few natural resources led to some additions to the concrete mix pre-1930s that were unique to the Netherlands, such as trass which was good in marine environments but was ultimately replaced by *hoogovencement* (blast furnace slag cement). While other cements were available, the most common and only type allowed until the *GBV 1930* (Dutch code of practice for reinforced concrete) was Portland cement, though its detailed chemical properties and composition were not fully known then. It became the preferred cement in reinforced concrete structures by the end of the 19th century due to its high early strength. If no binder is documented, it is most likely to be Portland cement (Heinemann 2013).

The other main ingredient, aggregates are responsible for the strength and durability of concrete through size, geometry, and grading. The knowledge surrounding aggregates were one of the later developments and the material properties were not thought to be of significance in the early stages (Heinemann 2013). The *GBV 1912* only addresses general dimensions, benefits, and sources. Sand should be no larger than 5 mm and only clean river sources are allowed (Article 3). The size of gravel was chosen based on use, but must be between 5-60 mm (Article 4). Contamination with other sands, clay, silt, or organic matter was discouraged, to achieve a better bond with the concrete. Fine sand was not common; it was understood that if aggregates were too fine, they would agglomerate with each other rather than with the binder, leading to a weaker composition. Size and limiting contamination were the main concerns. Shape and texture were not addressed. Aggregates were combined by empirically determined ratios (for instance, 1:2:4), without reference to grading of fine vs. coarse material.

Proportioning was either volume based (1:x:y) or by kg cement content per volume aggregate. (High) water and cement content were influenced by workability, since admixtures and additives were not widely used yet. Concrete mix design was generally empirical and often varied per the site superintendent's preferences until the effects of a good mix became apparent in the 1930s.

Steel with a circular cross-section is used as reinforcement in most present day structures. But an assortment of ferrous metals of varying cross-sections can be seen in historic concrete. The *GBV 1912* requires main reinforcement to be *vloei- ijzer/staal* (ingot iron or steel[1]) and allows *welijzer* (wrought iron) as secondary reinforcement. Plain, round bars without surface texture were most prevalent, with ends bent for better slip resistance. Rectangular flat bars were sometimes used as main reinforcement, but more known as a characteristic of secondary reinforcement of Hennebique patent systems. Other patents were known to texture their bars for better bond as well as branding. Transverse and longitudinal bars were twist-tied together by hand, but this was not always done and sometimes bars were not connected at all.

Corrosion due to concrete carbonation and chlorides was not discovered until the 1960s, and the placement of main reinforcement was based on the understanding that it should sit within the tensile zone, but it must be protected from fire so it cannot be placed totally at the bottom. The *GBV 1912* specifies 10 mm of cover at slabs, and 15 mm at beams and columns (Article 15).

1. Terminology based on production process: considered *staal* (steel) if cooled rapidly from high temperatures, ending with a higher hardness (Heinemann 2013).

3.3 Mechanical Properties of Historic Concrete

Early applications of reinforced concrete were based on empirical knowledge and the progression to scientific understanding can be seen in the evolution of the Codes. The first Dutch reinforced concrete regulations were published in 1912 as the *Gewapend Beton Voorschriften* (GBV), which aptly means reinforced concrete regulations. The *GBV* was issued by the *Koninklijk Instituut van Ingenieurs* (KIVI, Royal Institute of Engineers), so had no legal basis but served as a guideline. The document addressed material, execution, and made calculation methods widely accessible. Before this, Dutch engineers followed German regulations. The *GBV* went through five more iterations until being replaced by the *Voorschriften Beton* in 1974 and 1986, which was followed by the adoption of Eurocode 2 (EC2) in 2001. This section describes the mechanical properties of concrete (Table 3.1) and reinforcement (Table 3.2) as stated in the *GBV 1912*, which will apply to the design case study.

Table 3.1: Properties of concrete, based on *GBV 1912*

Concrete	Composition	Average cube strength	Allowable compressive stress	Equivalent safety factor
	-	[kg/cm^3]	[kg/cm^3]	-
*1 hl = 125 kg sand + gravel	135 kg cement, 5-6 hl aggregate	150	30	5
	135 kg cement, 4-5 hl aggregate	200	35	5.7
	135 kg cement, 4 hl aggregate	250	40	6.25

In present day EC2 notation (NEN-EN 1992-1-1), concrete strength classes are indicated by *C*, followed by the characteristic compressive strength, f_{ck}, and the characteristic compressive cube strength, $f_{ck,cube}$. For instance, C30/37 has a f_{ck} of 30 MPa and a $f_{ck,cube}$ of 37 MPa. The *GBV 1912* used a different method, by requiring a composition based on cement content (kg per volume aggregate) then obtaining the average compressive strength of three 200 mm cubes to represent the design strength (Article 22.3). Water content was specified for plasticity, but not concrete mix design.

Eurocode requires partial factors to be applied to characteristic values for materials, γ_M, and loads, $\gamma_{Q;G}$, for safety, reliability, and variation in construction methods. The *GBV 1912* built in safety by allowing a max compressive stress based on the average of the cube sample compressive strength tests. The safety factor is not explicitly mentioned, but the values can be inferred by dividing the average cube strength by the allowable compressive stress. The resulting values do not correlate to modern standards, so should be read as an indication of how the material was understood and not as a direct comparison to EC2 (Florisson 2013). But it is clear that the first concrete codes were very conservative, which was needed as quality of concrete was not consistent (Heinemann 2013). Generally, the magnitude of the safety factors decreased over time with the progression of scientific knowledge.

The *Rijkswaterstaat* (Directorate-General for Public Works and Water Management) translates the concrete strengths into Eurocode notation beginning from *GBV 1930*, which equates to C8/10. Because of the initial lag in concrete development, C8/10 may also apply to *GBV 1912*, but should be verified per project. Where mechanical properties are not documented, preliminary use of the lowest strength class provided under the original building standard is allowed (*Rijkswaterstaat Richtlijnen Beoordeling Kunstwerken (RBK 1.1), 2013*).

Table 3.2: Properties of reinforcement steel, based on *GBV 1912*

Steel	Surface	Ultimate yield strength	Allowable stress - beams	Allowable stress - slabs	Equivalent safety factor
	-	[MPa]	[MPa]	[MPa]	-
	smooth	370	80	100	4.6
	smooth	440	90	110	4.9
	smooth	500	120	120	5.0

EC2 denotes steel reinforcement quality by *FeB*, followed by the 2% yield strength, $f_{y;k}$. So for FeB500, the $f_{y;k}$ is 500 MPa. *GBV 1912* uses the ultimate yield strength and specifies allowable tensile stress values for beams and slabs (Article 22.1). Again, equivalent safety factors are high compared to current standards.

As with concrete, lowest strength values for steel may be used in the absence of documentation (*RBK 1.1, 2013*). The present day equivalent set by *Rijkswaterstaat* begins from *GBV 1918*, where the lowest value is FeB220, but (Florisson 2013) notes that early reinforcement steel is generally of low quality and that actual values may differ greatly. The parallel metals guideline of the time, the *Algemeene Voorschriften Ijzer 1911* (AVIJ 1911), classifies reinforcement steel quality by class (1-3) and further subdivided by letters (A-E). The *GBV 1912* does not reference the *AVIJ 1911*, rather it stipulates that the minimum tensile strength should be 37 kg/mm^2, which would eliminate anything under class B1. But note that 37 kg/mm^2 is an ultimate value, not the 2% yield strength.

3.4 Principles

The beginning uses of reinforcement in combination with concrete was driven by fire-protection and making impermeable objects[1]. A well-known name in reinforced concrete development, Joseph Monier's first patent in 1864 was for concrete pots, with wire mesh to support their form. This understanding of reinforcement is clear through other early applications, where its placement is in the middle of the object rather than the tensile zone. The Wilkinson patent is credited to be the first to use reinforcement to carry tensile loads, through iron rods and hoop iron strips (Heinemann 2013). By 1892, the Hennebique patent introduced monolithic reinforced concrete structures so that floors, walls, and columns are structurally unified, even if the walls and columns are made of other material.

The first iterations imitated more familiar materials, such as masonry (heavy exterior load-bearing walls), wood (columns to girders to joists), and steel (self-supporting I-beams) before developing into a concrete construction based on its own characteristics (Friedman 2010). Typical issues of the earliest systems, like the first version of the Monier patent[2], reflected the newness of the technology and resulted in no clear separation of compression and tension roles, inadequate shear reinforcement, and discontinuous rebar in successive spans. The European development of concrete into its own construction typology is often credited to the Swiss engineer Robert Maillart for his flat slab system that sits directly on columns with curved capitals, together reducing floor-to-floor heights (Slaton et al. 2014).

1. The invention of the first RC object is attributed to Joseph-Louis Lambot for a boat in 1844.
2. Monier was a gardener, not an engineer.

Reinforced concrete before 1950 is often characterized by rigid frames that integrate both horizontal and vertical load transfer. Stability was not fully understood, which is apparent in the drawings by the lack of calculations. However, there are indications to stabilizing elements, but they were incorporated into the building based on intuition. While today's multi-story structures typically have separate stability and gravity systems, where lateral forces are transferred to stability elements through the diaphragm, (Florisson 2013)'s case studies in the Netherlands indicate that the transfer is by the beam to column connections or the column to floor connections.

Without advanced knowledge of material and structural behavior, one may question how the early structures were able to function. This can be ascribed to the crude and conservative calculation methods, seen in the high safety factors of §3.3, that they did not consider linear strength behavior, and that they only designed in the elastic zone, all of which do not come near to using the total strength of reinforced concrete (Friedman 2010; Marcos et al. 2017). Also, that many were intended for industrial uses meant that imposed loads were often uniform and predictable.

3.5 Structural Typologies

The proliferation of reinforced concrete began in the mid-1800s through proprietary patents (Fig. 3.1), developed by industrialists who prioritized protection and profit of their systems over science. Key individuals are Monier and Hennebique in France, Ransome and Kahn in the US, Wayss in Germany, and Wilkinson in Britain. By the 1920s, the influence of patented systems went into decline as little transparency behind calculations and some structural failures led to distrust, along with the rise of a new generation of educated engineers who designed with theoretical grounding (Marcos et al. 2017).

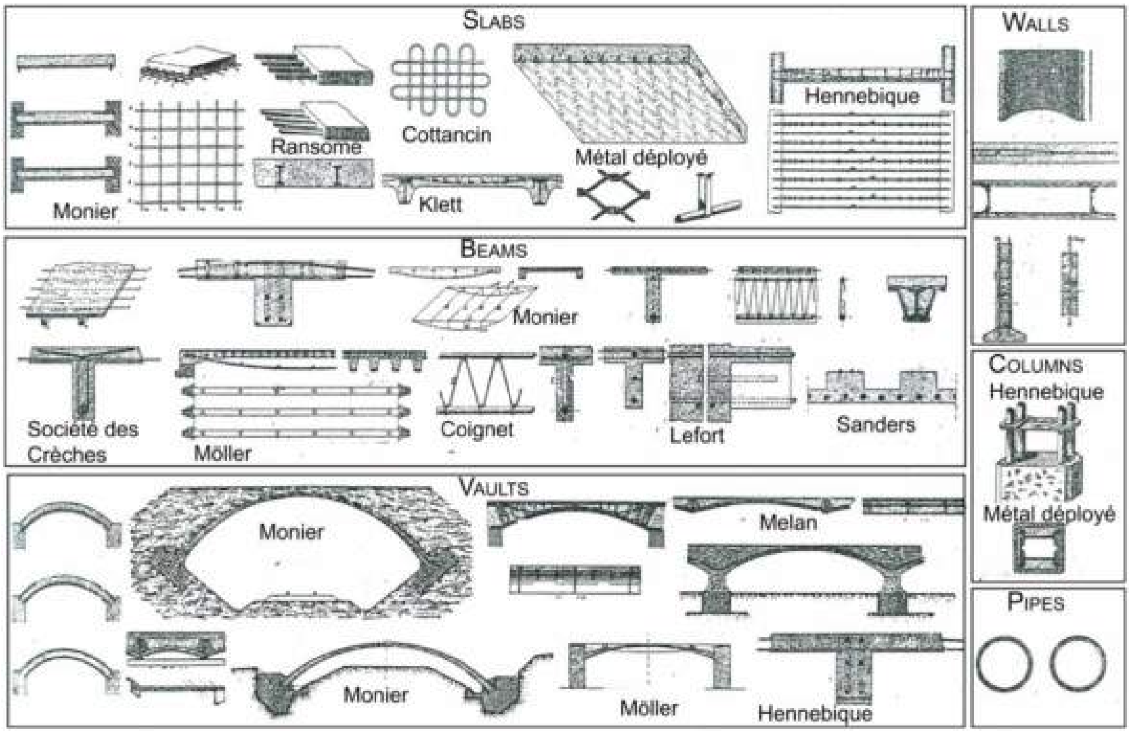
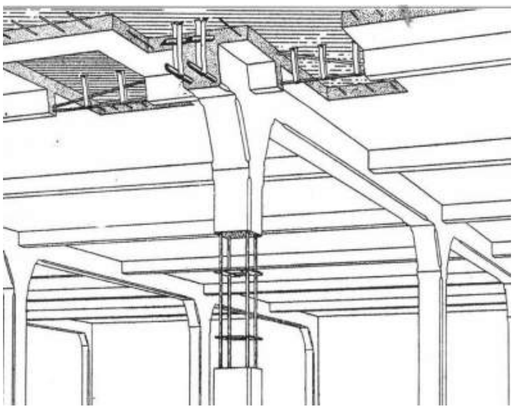
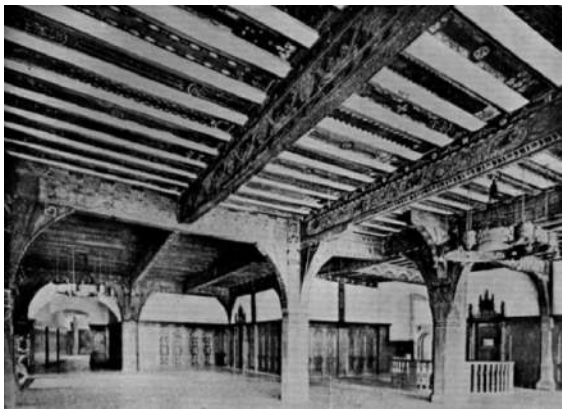


Fig. 3.1: Range of patents, from *Le béton armé et ses applications* by Paul Christophe, 1899

Many of the earliest reinforced concrete structures in the Netherlands were based on the Hennebique patent, with several examples located in the Rotterdam harbor including the 1901 New York warehouse, the 1914 St. Job warehouse, the 1916 San Francisco warehouse, and the 1931 HaKa mixed-use building (De Winter 1982). While the patent provided the design basis, it did not address every detail so the principles were adapted to each project by the local builder. The prevalence of the Hennebique patent (Fig 3.2a) in historic concrete was not just in the Netherlands, but internationally, so this section goes over its characteristics.



(a) Hennebique system (Hellebois 2013)



(b) Timber structure at *Rathaus Heilbronn*, DE (Rehm 2019)

Fig. 3.2: Comparison of Hennebique system with timber

(Rehm 2019) draws a parallel between the Hennebique systems and medieval timber ballrooms (Fig. 3.2b), which was noted by scholars as early as 1911. The imitation of conventional timber construction is seen in the almost vault-like beam to column connections that ensure rigidity and the clear hierarchy of main and secondary beams, not only optimizing use of material but perhaps also intending to market the unfamiliar material by evoking more familiar ones.

The main principles of the Hennebique system until the 1920s are main and secondary beams that transfer to rectangular columns, that together create moment frames. Beams are structurally connected to slabs and act together as a continuous T-section system. Plain round bars are placed in the tensile zone, following the path of the governing bending moment and inclining at around one third of the span (Fig. 3.3). Where the inclining bars reach the mid-supports, the (short) overlap is connected through wire-ties. Bar ends are bent or fish-tailed for better bond and slip resistance. Vertical stirrups, often flat rectangular strips, are spaced according to the shear force and do not cross over the top, so are U-shaped.

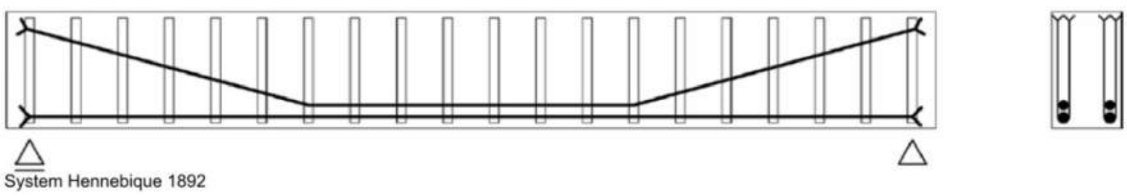


Fig. 3.3: Hennebique system beam diagram (Rehm 2019)

Typical issues of the system are too short overlap lengths at anchorage and at main bars as they meet at the supports. When three separate 6m spans of 1904 Belgian Hennebique concrete were lab tested in ultimate limit state, insufficient overlap of rebar (0.5 m in one case and 0.43 m in the other) made it so that bending capacity was lost at negative moments of continuous T-sections, and sudden failure occurred due to slip between concrete and rebar (Hellebois and Espion 2013). Also, Hennebique designs were calculated based on linear continuous behavior, where concrete strength was assumed to be uniform over the neutral axis, so safety intent is not accurately reflected in the resulting capacity (Marcos et al. 2017). Any future planned increases in load may be required to consider strengthening.

3.6 Typical Degradation Mechanisms and Repairs

Incomplete understanding of the material and its application in the pre-1950s wild west era of early concrete development led to degradation that became apparent only decades after construction. The usual issues are related to too little concrete cover of reinforcement, high chloride content, other inappropriate contents in the concrete mix, and unawareness about environmental classes. This section briefly summarizes the types of degradation, since the topic is well-documented and durability of concrete is not directly related to this thesis.

Based on the EU project Conrepnet, the most common damage processes observed in 215 concrete structures located in Europe are (in order): corrosion, frost, and cracks (Hellebois 2013). Alkali-aggregate reaction (AAR), deteriorated concrete, and poor construction are less common. The repair techniques are often not designed for historic concrete which is distinct from modern concrete, ultimately damaging the building and removing some of its significance (Heinemann 2013). The usual damage processes and how they are addressed:

- When rebar corrodes, it expands and makes concrete crack and spall. Concrete protects rebar in part through its alkalinity, which can be lost through carbonation (reaction of atmospheric CO_2 with water), from chlorides added to the concrete mix as an accelerator, seawater, or deicing salts. Damage from corrosion is traditionally fixed by patch repairs using a compatible shrinkage- compensated mortar. Some removal of the original material is required to prepare the substrate for a clean bond. Cathodic protection is an additional technique that turns an entire bar into a cathode (positive), so that corrosion can only occur at an exterior added anode (negative). Corrosion inhibitors can be applied, but are not reversible and their efficiency is still debated (Botton 2019).

- Cyclic freezing and thawing (a soaked concrete surface in frosty weather) cause scaling and cracking that can extend below the surface. Portland cement contains microscopic air bubbles that mitigate this, and new concrete construction often includes air-entraining agents against frost. Coatings or sealers typically alter appearance.

- Active cracks should be repaired at the damage origin. Inactive cracks are often addressed by a compatible material injection, but this is not a reversible technique and is difficult to re-treat.

At their least damaging, these issues are a matter of surface-level fix, but at their worst, they lead to corroded reinforcement, reduced effective cross-sections, or loss of bond between concrete and reinforcement. If removal of material is involved, the scale of intervention is always a question in conservation, where durability is directly proportional to the extent of replacement and authenticity is inversely proportional.

3.7 Strengthening of Reinforced Concrete in Flexure

Choosing an appropriate remedial action to strengthen an existing reinforced concrete structure is based on the scale of the carrying capacity increase, if damage must be addressed, technical feasibility (global structural behavior, installation, maintenance), safety criteria, life-cycle costs, and future use (Hellebois 2013). Lower magnitudes of capacity increase can typically be mediated by increases in the cross-section or bonding by steel plates or fiber reinforced polymer (FRP). More substantial interventions warrant the addition of new structural elements, such as steel beams and girders.

3.7.1 Increase Cross-Section

The section of an existing slab or beam can be extended through the addition of concrete layers, or jackets. The concrete strength of the jacket should match existing to avoid structural compatibility issues. If drilling is required, existing elements should be scanned so that existing reinforcement is not damaged and new bars can be lapped. The substrate should be roughened for proper bonding, allowing the old and new layers to act as a composite. Enlargement of the section increases load-carrying capacity while preserving original material at the same time. But this approach is not reversible and it alters the original form of the element.

3.7.2 Externally Bonded Fiber-Reinforced Polymer (FRP)

Applied to buildings since the 1980s, glass or carbon fiber reinforced polymers (GFRP, CFRP) are continuous systems that are epoxy bonded to the surface of a substrate. The thermosetting or thermoplastic matrix protects and binds the fibers and distributes loads across an even surface area. Usually, the process is a wet lay-up system that is installed and cured on site (Fig. 3.4). The anistropic nature of the fabric requires that it be laid out unidirectionally or that a product with fibers going in at least two directions is specified. FRP is limited by its linear-elastic behavior leading to brittle failure and low-failure strains in comparison to steel (Hellebois 2013). Care should be given when choosing CFRP in the case of adjacent anodic metals, due to risk of corrosion from galvanic action. This section goes over typical techniques used in concrete strengthening.

FRP is often justified as an appropriate remedial action for significant 20th century reinforced conrete buildings in that it meets the conservation principles of no destruction of existing material, nearly unchanged size of components, and reversible when designed to be contact (not bond) critical - though this can result in insufficient structural improvement. (Karydis 2006), citing the Venice Charter, Burra Charter, and British conservation guidelines, argues that FRP is more of a minimal intervention than with concrete or steel as it less invasive and is easier to conceal.

Fig. 3.4: GFRP by *Simpson Strong-Tie* bonded to a URM wall

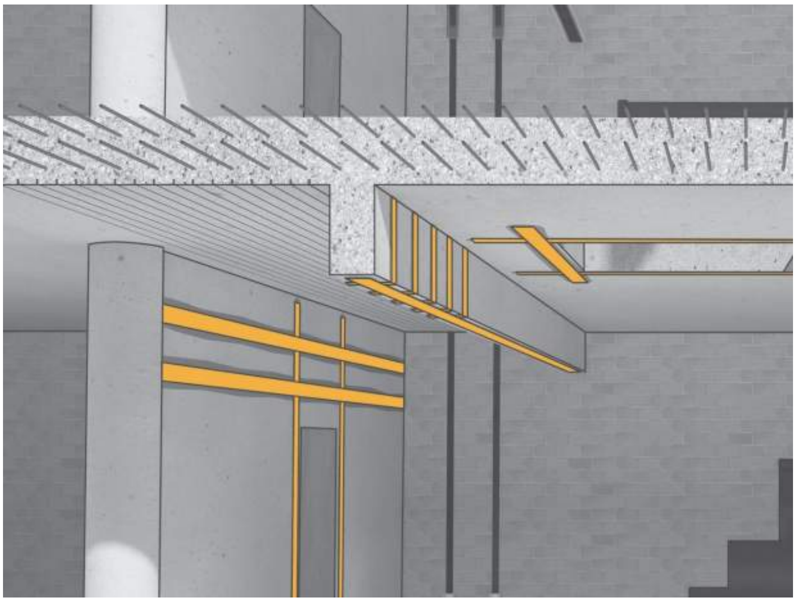


Fig. 3.5: Diagram of steel plate bonding, from *Sika AG*

3.7.3 Steel Plate Bonding

Another option, steel plates can be attached to concrete through mechanical bolting or adhesive bonding. The plate is usually adhered to a roughened substrate through an epoxy resin, with supplementary bolts to prevent delamination (Hellebois 2013). The technique increases flexural strength, stiffness, and load-carrying capacity, making it a cost-effective intervention if existing bending capacity is insufficient as well as for reinforcing openings. Care should be given to protection of the steel from corrosion. There is a limitation on the effective dimension of plates, usually 6 mm thick by 6-8 m long, so interventions typically look like long strips (Fig. 3.5). The resulting visual effect certainly changes the perception of the space and it may not be architecturally acceptable to leave the plates exposed in certain use types.

3.7.4 Structural Steel Beams and Girders

New steel elements can be introduced to reduce spans or they can be added to existing elements as a supplement. The steel is mechanically attached to the substrate, so it is reversible. Anyhow, it is distinct in appearance from the original structure, perhaps to the extent of changing how the space was originally understood. Often it is the only option if the scale of load-increase is high.

The First National State Bank in New Jersey, designed by Cass Gilbert in 1912, is a designated historic building and was retrofitted in 2014 by Pennoni. It is a typical example of a skeleton construction system: a 12-story structure with reinforced concrete slabs, steel I-beams, and built-up steel columns (Stuart and Cahan 2014). The new program called for a conversion of the mostly vacant building into a mixed-use facility containing a hotel, restaurant, and ground level retail. The roof of the building was strengthened to become an occupiable terrace. Because the existing concrete-encased steel beams were not sufficient in the heavier load situation, new steel beams (Fig. 3.6a) were added to reduce the tributary width (Stuart and Cahan 2014). The other option was to weld steel sections to the underside of the existing beams, but this would require removal of the concrete encasement and ultimately re-fireproofing.

Another example can be seen at Hudson Commons, a 1962 8-story reinforced concrete warehouse in New York that was converted by KPF into offices by a 17-story extension in 2019. Even though there was a load "credit" from the less demanding use, there was still a need to reinforce the roof so that it could be accessible. The original design specified CFRP, but with the addition of a green roof with a 4 ft (1.2 m) max soil profile, it was not possible. Instead, a secondary system of steel beams was installed (Fig. 3.6b).

(a) First National State Bank (Stuart and Cahan 2014)

(b) Hudson Commons (Provenza et al. 2019)

Fig. 3.6: Steel beam interventions at bottom of slab

3.8 Discussion

The composition and mechanical properties of Dutch historic concrete in the first decades of the 20th century could be inconsistent due to dependence on imported sources. The secrecy surrounding empirically designed patents added to the volatility of construction quality of the era. As design and construction moved away from proprietary systems to generally available knowledge in the 1920s, the science had far to advance to today's reinforced concrete standards in the progression of calculation methods, study of material properties, and clarity around the causes and effects of corrosion. Historic concepts of safety are considered inaccurate in comparison to the modern Eurocode theory of partial factors, so structural interventions should always evaluate the actual safety of the existing structure.

The first indication of a joint effort to organize reinforced concrete design in the Netherlands came about in 1906. Until the first codes of practice were established, Dutch engineers referred to German regulations for standardization. The lowest allowable strength values from the first guideline on reinforced concrete design and execution in the Netherlands, *GBV 1912*, will provide the basis of structural analysis of the test case where information is not available in the original documentation.

While the historic significance of a concrete building as a whole may be recognized, the value of the material still lacks awareness. Few examples remain of early reinforced concrete in Europe, so interventions in such structures should be judicious in how they address both technical and conservation questions. There is precedence in the Netherlands, through the designation of monuments such as the *Fort Bezuiden Spaarndam*, that historic concrete is culturally valuable as tangible documentary evidence of the early stages of the material's technological development.

Additionally, the prevailing strengthening methods with concrete, steel, and FRP add solid elements to the structure that alter the perceived scale of the space, changes its identity and authenticity. It could be that these methods meet the conservation guidelines as the only techniques currently available, so they are acceptable and they represent a reasonable compromise. To date, FRP appears to be the least invasive technology that follows conservation theory, but is often concealed and is not appropriate for larger load increase situations. If glass adds a transparent possibility, then it would better meet the criteria of respecting significant material and components, spirit and feeling, and no distortion of the physical evidence, as recommended in the Dublin Principles Article III.10 and Burra Charter Article 3.

4 Glass as a Structural Material

4.1 Compositions

Glass is liquid in a solid state. It is amorphous. When a liquid cools and changes phase into a solid, the molecules arrange themselves into a neat, orderly pattern. But the molecular structure of glasses do not have a coherent crystal lattice as found in other solids (Fig. 4.1). During solidification, the molecules of glass simply jiggle slower and slower until they stop moving, freezing in a state somewhere between liquid and solid (Schittich et al. 2007). No internal grain boundaries are formed. This is why glass is transparent. There are many theories about the glass transition[1], but that is a topic for another day.

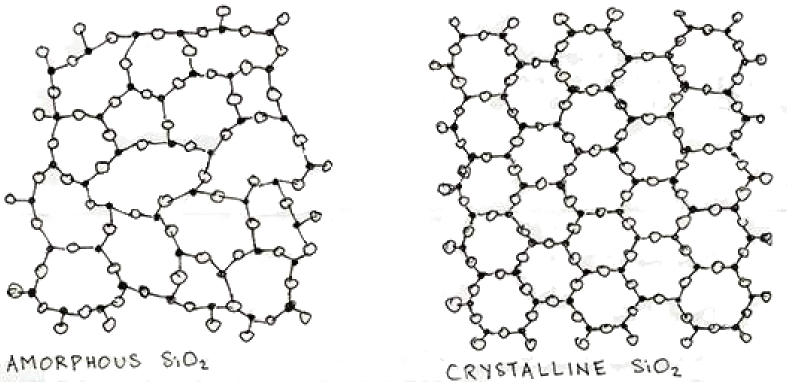


Fig. 4.1: Silicon dioxide tetrahedra: glass (left), quartz (right)

The most common and least expensive type of glass used in building applications, windshields, bottles, and tableware is soda-lime. It is usually composed of silica (SiO_2) which is the *former* that makes up most of the ingredients to be melted, soda (Na_2CO_3) which is the *flux* that lowers the melting temperature of the former, and lime ($CaCO_3$) which is the *stabilizer* that makes glass strong and water-resistant (Corning Museum of Glass 2011). Smaller amounts of other constituents are added as fining agents or to influence certain properties, such as coefficient of thermal expansion or color.

Soda-lime accounts for 90% of glass production, due to a good balance between low cost of raw materials, satisfactory strength and durability, and easy manufacturing. Borosilicate glass is known for its high thermal fatigue resistance, so it is often used in labware, cookware, and lighting. Lead glass is relatively soft, has a low temperature working viscosity, high electrical resistance, and can be spliced, making it favorable in glass tube bending, other fine art disciplines, and some tableware.

1. No one knows exactly what it is, except maybe Dr. Peter Wolynes, Professor of Chemistry at Rice University.

The chemical compositions of soda-lime, borosilicate, and lead glass are depicted in Table 4.1. Other types are aluminosilicate, 96% silicate, and fused silica glass, which are used in applications such as aerospace products, phone screens, fiberglass, and in situations that must withstand high operating temperatures (Oikonomopoulou 2019), so they are prohibitively expensive and are not so relevant to this thesis.

Table 4.1: Constituents of soda-lime, borosilicate, and lead glass, based on EN 572-1:2012, CES (Granta Design Ltd, 2019) and (Oikonomopoulou 2019)

	Composition			Uses	Price [€/kg]
Soda-lime	Silicon	Si	32-35%	windows and facades	1.30
[EN 572 -1:2012]	Calcium	Ca	3.5-10.1%	windshields	
Note: magnitude of proportion by mass of element	Sodium	Na	7.4-11.9%	bottles	
	Magnesium	Mg	0-3.7%	tableware	
	Aluminum	Al	0-1.6%		
	Others	-	<5%		
Borosilicate	Silicon dioxide	SiO_2	80%	labware	4.35-5.38
	Boric oxide	B_2O_3	13%	cookware	
	Sodium oxide	Na_2O	4%	lighting	
	Aluminum oxide	Al_2O_3	2.3%		
	Potassium oxide	K_2O	0.1%		
Lead silicate	Silicon dioxide	SiO_2	63%	neon tubes	3.64-4.46
	Lead oxide	PbO	21%	art glass	
	Sodium oxide	Na_2O	7.6%	tableware	
	Potassium oxide	K_2O	6%	x-ray absorption	
	Calcium oxide	CaO	0.3%		
	Magnesium oxide	MgO	0.2%		
	Boric oxide	B_2O_3	0.2%		
	Aluminum oxide	Al_2O_3	0.6%		

The softness and susceptibility to scratches of lead glass, caused by lead oxide (PbO) but is not an alkali flux, renders it inappropriate for structural uses. In borosilicate glass, boric oxide ($B_2O_{3)}$) not only acts like a former with the silica, but also reduces the percentage of alkali fluxes (Na_2CO_3) in the recipe, which in turn increases the mechanical strength of the composition and makes it more difficult to melt (Borax 2018). The addition of borates also increases chemical durability. But because borosilicate is not common in building applications, is it likely that soda-lime glass is the most fitting option as it is also the most cost-efficient.

4.2 Production Methods

Manufacturing processes are based on the form of the specified end product. In structures, this usually means float glass, but there are also instances of cast and extruded glass.

4.2.1 Float

Because most building glass is flat, the dominant type of glass production is the float process. The demand for glass rose after World War II, when the prevailing process was twin-grinding which was slow, costly, and required labor-intensive post-processing. The *Pilkington Brothers* developed the float process in 1959 as a method to make optically transparent flat sheets by floating molten glass over a tin bath that is smoothed by gravity and surface tension, eliminating the need for grinding and polishing (Corning Museum of Glass 2011).

Today's float lines are automated to operate continuously, where a river of glass flows from furnace to cutting, along a path of 300-400 meters. There are five steps (Fig. 4.2) as described by (Le Bourhis 2014; Saint-Gobain, n.d.):

1. Batch mixing: the raw materials, following a recipe in compliance with EN 572-1:2012 in the case of soda-lime, are continuously fed into a furnace and heated.
2. Melting and refining: the ingredients melt at around 1550°C, where the molten glass is homogenized through convection and fined to remove bubbles. Clear and clean (Class A) cullet is added to aid in melting.
3. The float bath: the molten glass moves onto the liquid tin bath at a temperature of around 1000°C, where it floats because of its lower density, forming a perfectly flat ribbon that is an equilibrium thickness of around 6.5 mm. The top rollers (Fig. 4.3) that pull the ribbon through the tin bath can be adjusted to control width and thickness. 2-25 mm is commercially standard, but 4-19 mm is typical. The ribbon is cooked and leaves the tank at around 600°C.
4. Annealing: the ribbon is in a near solid state and is transferred to the lehr for controlled cooling to prevent distortion and the creation of residual stresses. The glass transition occurs. A sulphur dioxide (SO_2) atmosphere reduces the friction between the ribbon and the rollers, and reacts with the glass to form a protective lubricant layer. At around 250°C, the ribbon exits the lehr.
5. Cutting: the glass is inspected for defects and then cut around them using diamond edged wheels. The most common size is 6x3.21 meters. The edges that have been in contact with the top rollers are cut off. Then it is stacked for shipping, or sent to further processes for coatings, lamination, or bending.

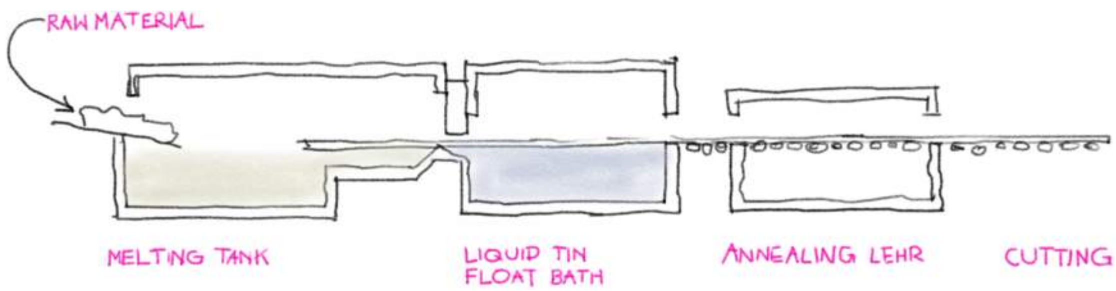


Fig. 4.2: Diagram float glass production, based on (Schittich et al. 2007)

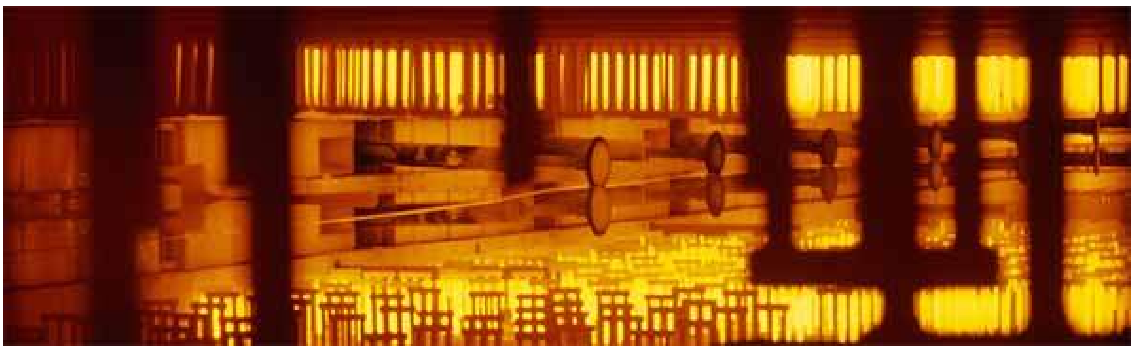


Fig. 4.3: Float tank interior (https://www. cmog.org/article/pool-tin-float-glass)

4.2.2 Cast

Casting is the most ancient way of producing glass. It is the only method of making large, monolithic objects or those with unique and complex geometry, so it is often seen in astronomy and art. The long annealing times of large objects make it so that the most feasible scale of cast glass building components to date are about the size of a masonry block (Oikonomopoulou 2019; Paech and Göppert 2008). The two ways to cast glass are:

- Primary casting by hot-forming: the raw materials are melted in a furnace, ladled into a mold, then annealed in a second furnace.
- Secondary casting by kiln-casting: in a single kiln, existing cullets are remelted into a mold below and the temperature is lowered for annealing.

4.2.3 Extruded

Extrusion is used to produce glass with constant cross-sections, such as tubes and rods. Typical applications are in pharmaceutical products, architectural interiors, lighting, and art. Because of its use in industrial settings, it is often composed of borosilicate glass, giving it high thermal shock resistance, precise tolerances, chemical resistance, and high transparency (Schott North America 2018). The most common method is the Danner process, where molten glass is continuously drawn down through a rotating mandrel, which blows air into the glass and makes it hollow as it moves along rollers.

4.2.4 3D Printed

Though it has not yet been applied outside the realm of research, a process for optically transparent 3D printed glass objects (*G3DP*) was introduced in 2015 by the Mediated Matter group at MIT Media Lab, the MIT mechanical engineering department, the MIT Glass Lab, and the Wyss Institute at Harvard in a joint effort. The team developed a steel and aluminum printer of two chambers, where the upper contains a crucible that feeds molten glass through a nozzle into an annealing chamber below (Klein et al. 2015). The project was followed up in 2018 with *G3DP2*, which demonstrated the technology's ability to produce columns and other architectural scale products.

Because the design intervention is in flexure, it is likely that it will use the conventional float process and that the posed problem can be solved without exploiting the three-dimensional possibilities of other types of glass. Material efficiency and complex annealing schedules of large monolithic glass are also factors, even if solutions can be found using all processes previously described. For these reasons, the following sections continue under the assumption that float glass is most applicable to the test case, and that cast, extruded, and 3D printed glass do not need to be discussed further.

4.3 Strength of Glass

Glass in its most basic form is brittle, isotropic, almost perfectly linear-elastic. It is more unforgiving than other widely used structural materials such as steel, concrete, and wood. Because glass breaks within the elastic limit before any permanent deformation, peak stresses cannot be redistributed by internal forces, leading to fracture without warning.

Table 4.2: Material properties of glass per EN 572-1:2012 and CES (Granta Design Ltd, 2019). Strength values vary based on the literature.

Type	Tensile strength	Compressive strength	Hardness (Knoop)	Young's modulus	Poisson's ratio	Density at 18°C	Coefficient of thermal expansion
	[MPa]	[MPa]	[GPa]	[GPa]	[MPa]	[kg/m^3]	[10^{-6}/K]
Symbol	$f_{gl.t}$	$f_{gl.c}$	$HK_{0.1/20}$	E	v	ρ	α
Soda-lime	30-60	300-420	6	70	0.23	2500	9
Borosilicate	22-32	260-350	4.5-6	60-70	0.2	2200-2500	3.1-6.0 (Class 1-3)

The material properties of soda-lime and borosilicate glass are shown in Table 4.2. The characteristic tensile strength of glass is 30-60 MPa, or 45 MPa as stated in EN 572-1:2012. In contrast, the theoretical strength required to break the interatomic bonds of soda-lime glass is 32 GPa, when E=70 GPa. However, this is never reached in practice (Le Bourhis 2014). The reason for this discrepancy in theoretical and actual strength is related to the presence of micro-flaws and defects on the surface of glass (and all other brittle materials) that are both intrinsic and extrinsic. Since glass cannot redistribute the local stresses by compensating with plastic strain, fracture propagates from the flaws when stress concentrations exceed the theoretical strength.

In compression, glass has a higher characteristic strength because the flaws do not directly induce failure. But the design of a glass element in pure compression will always be governed by tensile strength, as buckling or the Poisson effect will occur before complete utilization of compressive strength (Oikonomopoulou 2019). The strength of glass is always dependent on tensile stresses, which are caused by external actions, environment, size of element and cross-section, and production and processing quality.

The unpredictability of glass is reflected in statistical testing, showing a large scatter in strength and failure that does not follow any probability distribution. As such, it is probable that a glass element fails before reaching any characteristic strength, with a coefficient of variation that is 25% higher than other typical structural materials (O'Regan 2014).

4.4 Types of Float Glass

Float glass has different bending strengths based on type. Secondary processes allow prestressing by heat-strengthening or toughening. The variants of glass (O'Regan 2014) listed in ascending strength:

- Annealed: the basic type of glass as it leaves the float process, described in §4.2.1. It is vulnerable to thermal shock and uneven heating. It breaks into large shards.

- Heat-strengthened (HSG) or semi-tempered: annealed glass is reheated to 620°C then jet-cooled by air. This cools and solidifies the surface more quickly than the interior. As the interior gradually cools and tries to shrink, tension increases and it pulls the less dense surface into compression, usually between 24-52 MPa in Europe (Fig. 4.4a). When heat strengthened glass fails, it also breaks into shards like annealed glass. Because of the slow cooling process, the thickness of HSG is limited to 12 mm or less.

- Toughened or fully-tempered: the process is similar to heat-strengthening but the surface is cooled more rapidly. This puts the interior in tension, causing surface compressive stress, usually between 80-150 MPa in Europe. Then, the glass is stronger in bending, as both the added compressive stress and the original annealed strength make up the total strength. Toughened glass fractures into small cubes that separate upon impact. Generally, the number of fragments increases with higher surface compressive stress.

  Naturally occurring nickel sulfide (NiS) inclusions in glass sometimes cause spontaneous fracture in toughened glass. During prolonged heating, NiS undergoes a phase change and expands. If the glass contains a large volume of NiS, which is rare, it will shatter or crack sometime between hardening and the first few years after production. Heat-soaking the glass can accelerate the phase change so the panes that have high NiS inclusions can be identified during production.

- Chemically toughened: the composition of the surface is altered by bathing the glass in a potassium nitrate solution. The smaller sodium ions on the surface are exchanged for larger potassium ions through electrolysis (Fig. 4.4b). Because extreme heat and cooling are not applied, there is less deformation in the process. But the resulting compressed layer is thinner and less robust than in thermal toughening, so it is typically only applied to thin glass.

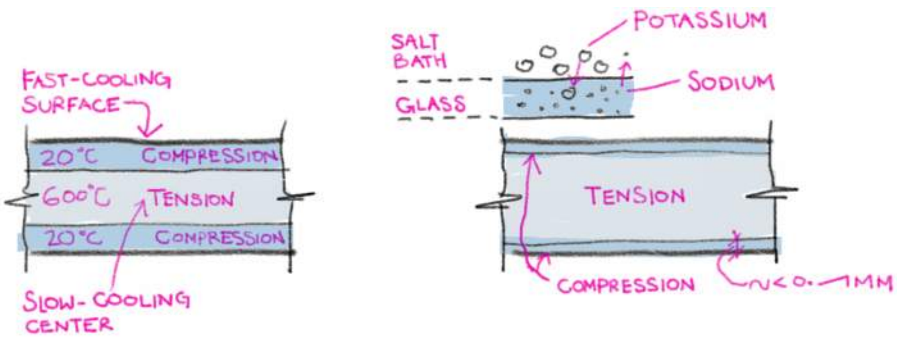


Fig. 4.4: Left: thermal tempering. Right: chemical tempering

In heat-strengthening and toughening, holes with a diameter no larger than the thickness of the glass are acceptable, but all cutting and drilling must occur before tempering so that the compressive layer is not disturbed. Tempering also causes two types of distortion: 1. as the heated glass moves along rollers, it tends to deform from its own weight and ends up with a slightly wavy surface or *roller waves*. 2. opalescent effects that are visible under certain lighting conditions, especially polarized light.

Annealed, heat-strengthened, and toughened glass can be laminated for better post-fracture performance, impact resistance, and fire resistance. Laminated glass is two or more pieces bonded by an interlayer made from polyvinyl butyral (PVB), thermoplastic polyurethane (TPU), ethyl vinyl acetate (EVA), polyester (PET), resin, or ionoplast. Because monolithic glass is subject to brittle fracture, it is often laminated in structural elements so there is still residual resistance when it breaks (Sjögren 2012). The process: the assembly is heated and pressed, then fully bonded through elevated temperatures and pressures in an autoclave.

Laminated annealed or heat-strengthened glass retain some structural capacity in a fractured state since they break into large fragments, but not as well as laminated toughened glass. However, the high capacity and impact resistance of laminated toughened glass is considered negligible when it loses its stiffness by breaking into small pieces. In a horizontally mounted structural glass element, such as a floor, the issue of a total toughened glass assembly crumbling out of its frame after fracture can be mediated by substituting at least one of the layers with heat-strengthened glass (O'Regan 2014).

4.5 Connection Types

Connection design for materials such as steel and wood are typically more concerned with areas of maximum stress and shear. But the inability of glass to yield and dissipate forces do not allow localized stresses to be ignored and precise fit is required. So, connections are a core part of structural glass design. Connection types have trended toward reducing the size of fixing elements and increasing the loads that the glass must carry. This development is apparent in the progression from early linearly supported glazing to more recent adhesive joints.

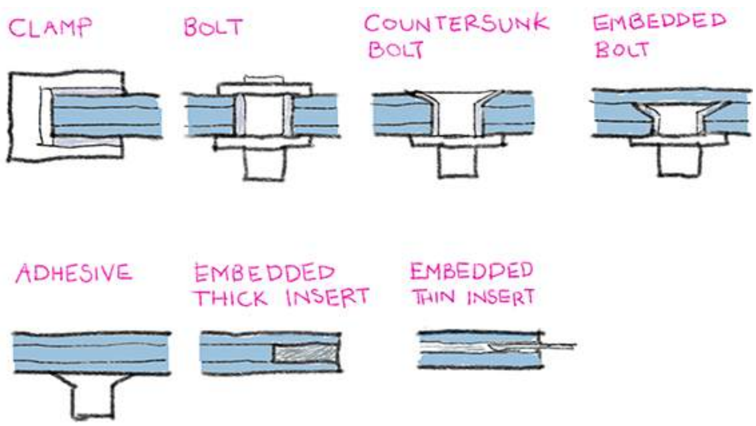


Fig. 4.5: Overview of connection types, based on (Bedon and Santarsiero 2018)

4.5.1 Mechanical-Fixing

4.5.1.1 Linear-Support

Continuous linearly supported glazing is usually found in curtain wall systems, where a glazed panel rests on setting blocks that transfer its weight to a supporting frame. Localized stresses from the blocks are not of great concern because the load type they are designed for is only a matter of the panel's self-weight (Haldimann, Luible, and Overend 2008). Lateral loads are transferred to the frame sides by clamp and glazing bead.

4.5.1.2 Clamping

Clamps have less of a visual impact than continuous supporting frames. They are usually small, local supports, but can also be designed to be continuous. They fix the glass to either a substructure or to glass fins. In the case of local low-friction clamps (Fig. 4.6), the principle is to transfer perpendicular loads to the glass. Clamping plates are typically screwed or bolted together to hold the glass in place, separated by an intermediate material. Local bending strength and stiffness of the component is critical in design. The relation between the stiffness of the clamp and local stresses that develop around the glass it is holding is usually the governing factor (O'Regan 2014). This can be established by determining the magnitude of the applied stresses, comparing them to the design strength of the glass, and building in an allowance for displacement.

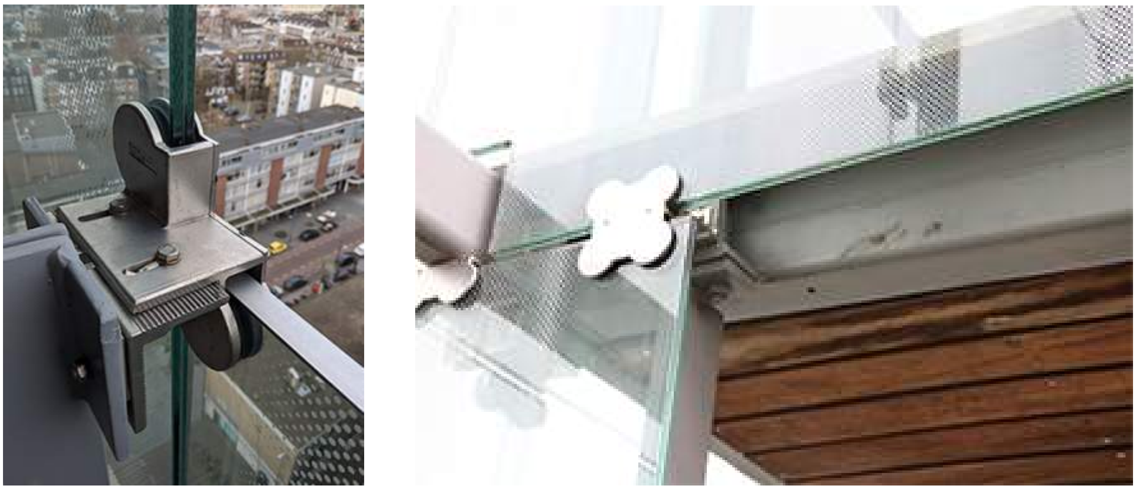


Fig. 4.6: Low friction clamp allowing movement in 3 directions, guardrails at *de Karel Doorman*, Rotterdam

Friction-grip connections are designed to transfer in-plane loads through a tight clamp, where it is to have tension, shear, and compression capacity additional to a gravity load. They are often used in splice plate connections, where in-plane tensile loads are spread across the surface area of the plate and the bolt does not bear on the glass. The assembly includes the glass, steel plates on either side, gaskets as an interlayer between the plate and glass, and bolts to clamp the plates together. If the required clamp tightness under maximum design load disrupts the interlayer, which can cause unacceptable local stresses in the glass, a common solution is to include pure aluminum plates at the same thickness of the resin interlayer within the laminated glass at the locations of the splice plates (Hansen Group, n.d.).

4.5.1.3 Bolting

Bolted connections (Fig. 4.7) are more minimal than friction-grip connections, but they are susceptible to local peak stresses that cannot be redistributed across a plate and increased flaws, from the drilling process, that act as stress concentrators. Fit and avoiding direct contract between the glass and the fixing bolt is critical. Gaskets of a lower Young's modulus are used as an intermediate between the two materials. The placement of the holes and their spacing should be modelled using non-linear finite element methods to accurately depict the distribution of stresses. Bolting usually requires the use of tempered glass in response to the high stresses generated near the holes. Installation and assembly on site require skilled knowledge about fit.

Fig. 4.7: von Gerkan Marg & Partner (gmp): Berlin *Hauptbahnhof* (Marcus Bredt, 2006; (Gugeler et al. 2006))

4.5.1.4 Embedded Connections

Embedded connections involve either a thick or thin metal insert (typically titanium for a similar coefficient of thermal expansion with glass) within the laminated layers. Thin inserts require the addition of structural interlayers to match the thickness of the insert, so no cutting or drilling is needed. Thick inserts (Fig. 4.8) require cutting before embedment and the quality control of the cut edges is an automated process. Much of the work occurs during the lamination process, so the mostly pre-fabricated components make installation on site more straightforward than with the other connection types (Bedon and Santarsiero 2018).

Fig. 4.8: Thick insert embedded connection at stair tread to balustrade, Apple San Francisco (*seele*, 2016)

4.5.2 Adhesive-Fixing

The appeal of adhesive-fixing is that the connector is invisible. Adhesively bonded connections distribute stresses more uniformly over a larger surface area than in mechanical connections. They do not require drilled holes. The two types are soft elastic (structural silicone sealants) and rigid adhesives (epoxy, acrylic, polyester resins). All adhesives are polymers and their thermomechanical properties dictate whether they can be repeatedly heated and cooled or if they are irreversible once set.

Soft elastic adhesives are good in uniform tension, but susceptible to deformation under shear. Structural silicone is traditionally used to bond glass to frame substructures in curtain wall systems, but are increasingly applied to other structural elements: glass corners, glass fins to vertical glazing. The main limitation of structural silicone is their low Young's modulus, making them unable to transfer the high shear forces of built-up structural glass elements (Haldimann, Luible, and Overend 2008).

Rigid adhesives create composite action between elements. But their application demands precision: the margin of tolerance in the substrate dimensions is very narrow due to the thin adhesive (especially *contact adhesives* that are typically <1 mm thick), the surface area of the substrate must be clean so that the adhesive adheres, sharp edges and uneven surfaces must be removed so that stress concentrations do not occur, staging of the work area must account for the required temperature, humidity, and lighting conditions during setting and curing. The low viscosity of the adhesive may prevent its application on anything other than horizontal surfaces. The highly controlled installation procedure affects the feasibility of deconstruction, whether the adhesive is reversible or not (Oikonomopoulou 2019).

4.6 Glass in Flexure

The nature of glass makes it an inherently unsafe structural material. Certain measures can increase the safety of all-glass structural systems. Because the design intervention will be installed at the bottom of slab of a roof system, the following subsections survey the strategies that can be applied to elements in flexure.

Even if monolithic glass can work in short spans or low load situations, it is discouraged because of the lack of redundancy in cases of spontaneous breakage that can be caused by impact or environmental factors (Johnson 2014). Due to the brittle character of glass damage and failure, optimizing redundancy by increasing post-failure resistance will increase the safety of the system. In partial damage, the main point of concern is the ratio of residual resistance to what is lost. In failure state, glass cannot carry tensile loads so a second path of load transfer is needed, which makes any flexural system with post-failure capacity a composite (Bos 2009). This can be done through laminating sacrificial layers or metals between layers of glass, reinforcing beams with a stronger and stiffer material, or combining glass with another material to create built-up hybrid elements. Typical structural interlayers in laminated glass are either PVB, which is traditionally more common, or ionoplast, such as SentryGlas (SG) which is stiffer and has better thermal and moisture resistance.

Though not often applied in practice yet, several concepts to reinforce, post-tension, or other ways to build in post-failure capacity have been developed in glass beam research (Fig. 4.9).

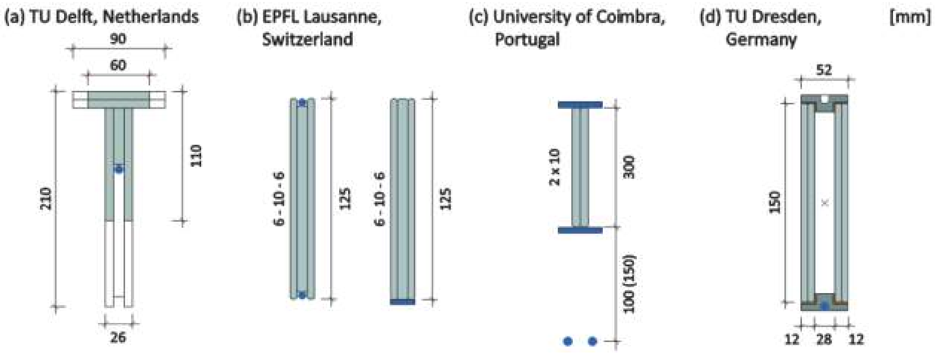


Fig. 4.9: "Case Studies of Reinforced and Post-tensioned Glass Beams" by Engelmann & Weller, 2017 (steel in blue)

Glass beams with GFRP, CFRP, or stainless steel rods laminated within SG interlayers have been studied for their residual capacity and ductility (Bedon and Louter 2019). The rods act as reinforcement tendons that still transfer tensile forces while in a cracked glass state. The results show stronger resistance to initial cracking, when compared to unreinforced laminated glass, and all three materials exhibit increased post-fracture deformation from the first crack, with steel at the highest.

As an alternative to rod embedment, SG interlayer laminated glass beams post-tensioned through a mechanically or adhesively bonded (or both) steel tendon to the bottom or top (or both) of a glass beam edge were studied by EPFL Lausanne (Fig. 4.9b) (Cupac, Louter, and Nussbaumer 2021). It is a relatively simple manufacturing process that provides substantial increase in load capacity and redundancy over laminated glass without reinforcement. Compared to rod reinforcement, exterior post-tensioning uses the material more efficiently while maintaining high level of redundancy in post-fracture state. Flexure is generally governing in failure under 4-point bending, but some specimens failed from lateral torsional buckling or by shear prior to the tendon yielding.

The two previous beam types use annealed glass for its tendency to break into large shards at failure. The low strength is compensated by the reinforcement. The disadvantage of this approach is that annealed glass loses its strength over time due to static fatigue, so some kind of tempered glass may be more suited to long-term loading. A study by TU Dresden developed a beam design (Fig. 4.9d & 4.10) that consists of two sets of parallel laminated HS glass, with a steel cable tendon in between and adjusted by stainless steel connectors at the top and bottom of the beam (Engelmann and Weller 2018a).

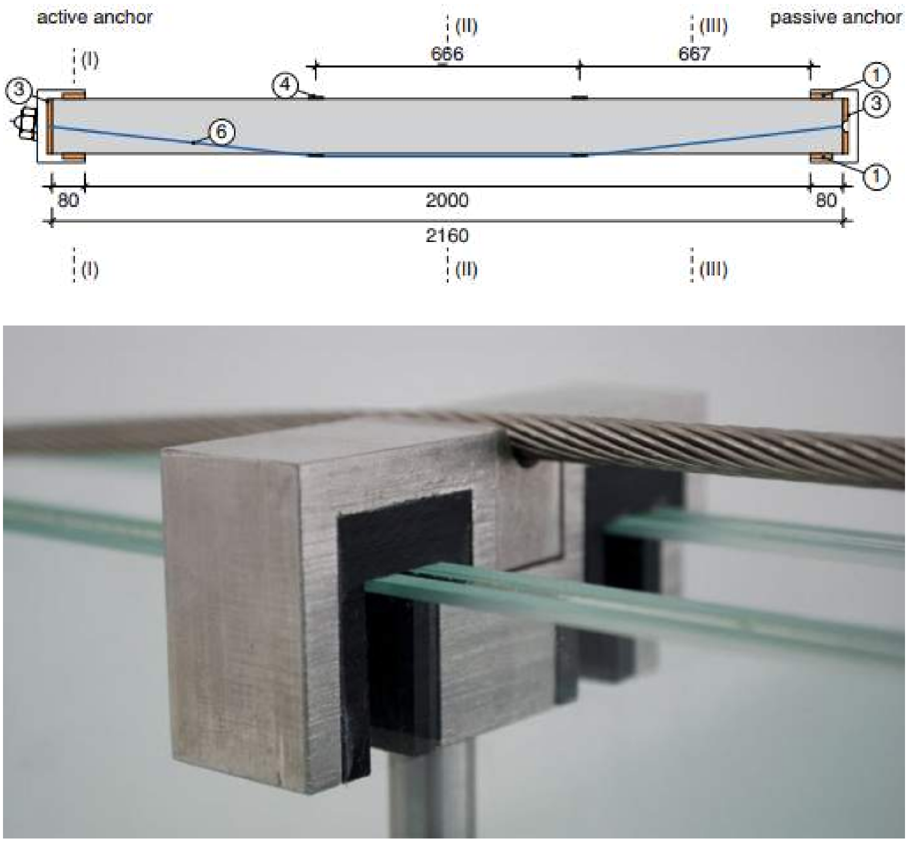


Fig. 4.10: Post-tensioned glass beam (*Spannglas)* by (Engelmann and Weller 2018a)

Another option to build in ductile behavior of glass components in flexure is to substitute some parts for other materials. Hybrid beams with steel flanges and end plates adhesively bonded to a glass web, with external 10 mm steel cables below for eccentric pre-stressing (Fig. 4.9c) (Firmo et al. 2015), and beams with wood flanges adhesively bonded to a glass web have both been developed (Kozłowski, Serrano, and Enquist 2014). In the case of the glass-timber beam, monolithic glass was used and two situations using annealed glass and heat-strengthened glass were tested.

4.7 Recyclability

Glass is an inert material that is infinitely recyclable. In the realm of building glass, this is almost never done. Using recycled glass cullet as part of the float glass process is already standard across all major European manufacturers. Cullet can come from internal sources (cut-offs or rejects during processing) or from post-consumer sources. The average percentage of cullet in European float glass recipes is 20-25%, with Saint Gobain UK among the highest at 30%, but it is likely that up to 50% is possible while maintaining the existing manufacturing process and quality (DeBrincat and Babic 2019; Saint-Gobain, n.d.). However, cullet must be rated at the highest level of quality with zero contamination to become building glass again. If not, it is relegated to downcycling. Present demand far outstrips material supply.

Laminated glass does not usually meet the zero contamination criteria to be remelted into float glass because of two main reasons: the interlayers and adhesives are difficult to completely remove and the pulverizing process to delaminate glass turns it into pieces that are too small to be ideal in float glass manufacturing (DeBrincat and Babic 2019). So if not directed to landfill, laminated glass cullet is most likely downcycled to glass wool insulation or colored container glass. The solution can be approached either by rethinking the use of laminated glass for safety or by investing in developing a new delamination process. There is interest from both the auto and building industries to push this forward.

Also of note, the trend toward low-iron recipes, which lessens the greenish tint of standard glass, may affect its recyclability. The addition of older glass cullet may not be possible in low-iron glass batch mixes due to mismatching elemental compositions. Though the demand for clean cullet may drive the development of laminated or adhesively-bonded glass cleaning technology, the ability to disassemble a building component into separate material streams is currently the critical factor in ensuring its recyclability.

4.8 Standardization for Glass

The rapid progression of structural glass development in the past decades has led to several European standards (prEN) and certain country-specific guidelines. Different design approaches and reservations among European member states have not yet yielded an official or unified set of standards. The European Committee for Standardization (CEN) established a structural glass committee in 2016, CEN/TC 250, to work on a general European standard for structural glass designated to become Eurocode 10 when complete. It is projected to address:

- General rules: basis of design of glass structures, materials, and products

- Design of secondary structural components, like plates and associated supports and fixing elements
- Design of primary structural components, like beams, columns, shear elements, special joints

The European standards and guidelines most relevant to this thesis and glass design in the Netherlands are:

- The Dutch norm, NEN 2608:2014 *Vlakglas voor gebouwen - Eisen en bepalingsmethode* (Flat glass in buildings - requirements and determination methods), which covers the three sections currently under development as Eurocode 10 and scenarios from low to high consequence classes, such as breakage of load-bearing glass elements.
- The Dutch norm, NEN-EN 572-1+A1:2012 *Glas voor gebouwen – Basisproducten van natronkalkglas – Deel 1: Definities en algemene fysische en mechanische eigenschappen* (Definitions and physical and mechanical properties of soda-lime silicate glass)
- The 2018 pre standard for Eurocode 10, CEN TS "Structural Glass – Design and Construction Rules"

4.9 Glass in Structural Interventions

In 2017, the *seele group* was commissioned to engineer a 82 sqm glass wall (Fig. 4.11) at the chapel of St. Patrick's Cathedral in New York City. The intent was to acoustically, but not visually separate visitors from the rest of the cathedral. The upper section is three panes 10.7 x 24 m and made of 3 x 12 mm laminated tempered glass, weighing 2.5 tons and resting on one 7 m 8-ply beam. The lower section is composed of double and single-leaf swing doors. The glass attaches to the existing structure by a brass trim custom-fitted to the columns.

Fig. 4.11: Interior glass wall at St. Patrick's Cathedral, US (Stephen Walker, 2018)

The *Tempio di Augusto* near Naples dates to 194 BCE and was most recently restored in 2003 by Marco Dezzi Bardeschi, with Faraone as the glass subcontractor. The intervention involved a tie rod and strut system spidered to a glass infill of the space between the remaining columns, seen on the right side of Fig. 4.12, and a glass fin to glass facade system, seen on the left side of Fig. 4.12. The fins are cutouts in the image of the columns, which one may argue goes into the sphere of conjecture.

Fig. 4.12: Glass facade and column representation at *Tempio di Augusto di Pozzuoli*, IT (Enrico Colosimo, 2019)

The intervention at the Menokin House ruins (Fig. 4.13, see also §2.5.1) by Machado Silvetti, in collaboration with Eckersley O'Callaghan, includes replacement of missing portions of the walls and roof with structural glass. A transparent catwalk is planned for the upper level void that looks all the way down to the basement. To date, this project does not yet have funding secured for the glass intervention.

Fig. 4.13: Menokin House, US - Left: ruins in current condition; Right: section axon by Machado Silvetti

4.10 Glass and Concrete Together

Precedents of glass and concrete acting together structurally do not exist. But the two share basic material properties: strength in compression that far exceeds strength in tension, reflected by a linear stress-strain ratio that ends in brittle failure by tensile stresses. To maximize utilization in tension, concrete is always reinforced and the reinforcement of glass is the subject of a number of current research activities. The most notable difference is tensile bending stress in the cross-section of glass that makes it vulnerable in stability, which puts calculations in the realm of second order theory. The coefficients of thermal expansion for glass and concrete are not so significantly far apart that their combination would lead to large and unwanted movements.

(Engelmann and Weller 2018b) liken glass to a transparent kind of concrete and believes that the safe design of glass in bending can take cues from reinforced concrete design. A comparison of the properties of both materials (Table 4.3):

Table 4.3: Soda-lime glass and concrete, comparison of basic material properties (Engelmann and Weller 2018b)

Property	Glass	(Unreinforced) Concrete	Ratio
Tensile strength [f_t]	45-120 MPa	1.0-8.3 MPa	5.4-120
Compressive strength [f_c] *DIN 1249-10, DIN 51067-1	500-900 MPa*	12-100 MPa	5.0-75
$f_c : f_t$	4.2-20	7.5-12.5	0.56-1.67
Young's modulus	70 GPa	27-44 GPa	1.6-2.6
Poisson's ratio	0.2-0.23	0,20 +/- 0,05	1.00-1.15
Coefficient of thermal expansion [10^{-6}/K]	9	10	0.9

4.11 Discussion

Precedence shows that standard float glass can be combined and configured to carry substantial loads in bending, where the focus of current research has been on increasing initial crack strength and ductile post-fracture behavior. Besides flexure, the slenderness of planar float glass make lateral torsional buckling another common cause of failure. A reasonable progression in the research is to determine how glass behaves when acting with other materials. Knowing that it is unable to dissipate stress concentrations through plastic strain, the detailing of joints and connections should be approached with tact. The application of glass in historic buildings holds the additional stipulations of reversibility, minimal intervention, and protection of identity and place. The point of using glass would be lost if it is overpowered by an assembly of opaque supporting elements that only serve a technical purpose.

Building with glass is not intuitive, because it is about turning something that does not want to be a structure into a structure. There is a challenge in finding configurations that maximize the utilization of material while maintaining transparency and minimizing the addition of supporting elements. Some theoretical grounding can be found in reinforced concrete design. But a warning can also be found in this analogy, in that the first concrete structures were based on little knowledge surrounding proprietary patents, followed by extensive research into the topic. 100 years later, it appears that the state of structural glass is in the latter stage and it is the task of today's researchers to elevate and standardize a design philosophy.

Case Study
Design & Validation

5 Case Study: Fenix II, Rotterdam

The previous sections discussed the conservation theory behind industrial heritage reuse, early concrete as a historically significant material and construction type, and gave an overview of how glass can be made structural. To study how glass can be used to strengthen historic concrete, the research is applied to the specific conditions of a test case from here on. The selected building is the Fenix II warehouse (*Fenixloods-II)*, originally called the *San Franciscoloods*, in Rotterdam, Netherlands. Located in what was once the fringe of the city, the site is now considered prime real estate in the center of the Rotterdam metropolitan area. It is a textbook opportunity for adaptive reuse.

5.1 Context and Historic Significance

With the rise of international shipping and transshipment companies, Rotterdam rapidly expanded from the extents of its historic city center in 1825. The port of Rotterdam outpaced that of Amsterdam and became a hub of trade between the US, UK, and the industrial Ruhr valley of Germany. As such, the growth of Rotterdam in the early 20th century was centered around new port infrastructure to meet the demands of new industry, and warehouses featured prominently on the quays (De Winter 1982). This era of the Second Industrial Revolution coincided with the development of reinforced concrete, which was preferred for its fire resistance and low construction costs in comparison to steel. Many contractors who built the concrete warehouses were also holders of the Hennebique patent. One of the first such buildings was the 2-story New York warehouse in 1901 on the *Wilhelminakade*, a 5 minute walk from the Fenix warehouse.

Built in 1916-1922, the Fenix warehouse was commissioned by the transatlantic shipping company *Holland-Amerika Lijn* (HAL) when their fleet expanded to include more lines and more storage was needed (Fischer 2014). Construction was delayed due to the scarcity of material during and after World War I. Besides trade, the building also has a place in the history of European migration. Millions of emigrants left from the ports of Rotterdam headed for Ellis Island and beyond, and the Fenix warehouse stored their belongings.

The building is situated in *Katendrecht* (Fig. 5.1), a peninsula neighborhood that is part of the larger *Feijenoord* district on the south bank of the River Maas. The site lies on the *Veerlaan*, with the *Rijnhaven* as the northern border. The *Maashaven* forms the southern border of Katendrecht. The *Rijnhaven* and *Rijnhaven* were dug around 1895, which turned *Katendrecht* into a busy port area (Van Winsen et al. 2018). The shipping trade brought foreign Chinese sailors to the area, who turned the neighborhood into one of Europe's first Chinatowns in the first decades of the 20th century. From WWII until the 1980s, the demographic expanded to include a variety of international sailors, stevedores, and associated entertainment. The following transition of port activities to *Hoek van Holland* marked a shift in the character of the neighborhood, which then took on a reputation for violence and crime.

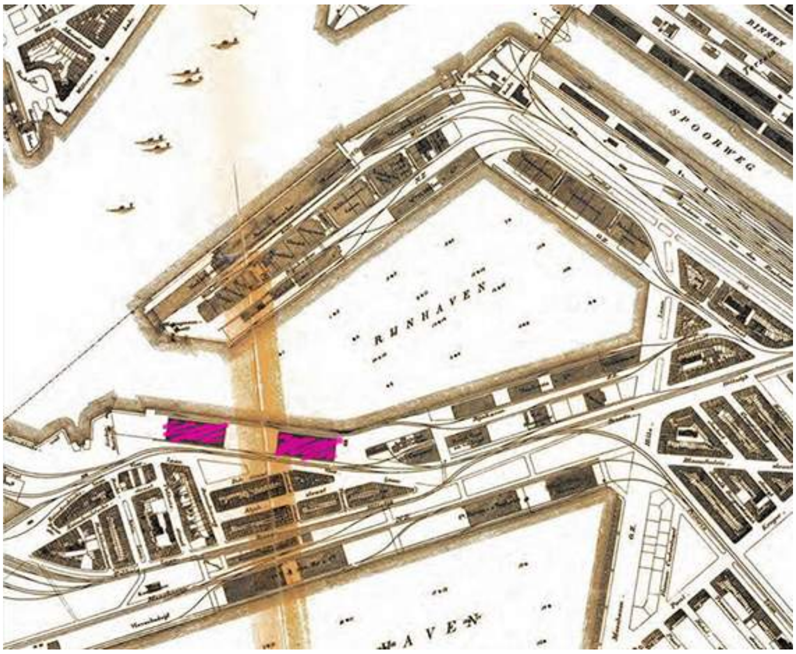


Fig. 5.1: The *Rijnhaven* in 1914 (Van Winsen et al. 2018), with the site of the Fenix warehouses highlighted

Today, the Fenix buildings are easily accessible and well-connected to the rest of the city: a Metro stop is down the way and a pedestrian bridge connects *Katendrecht* to the *Wilhelminapier* to the north, which is a short distance to the Erasmus Bridge. The neighborhood is generally gentrified, particularly with the recent redevelopment of the buildings on the Wilhelminakade and the Fenix I within the last 10 years.

5.1.1 Morphology

Due to the bombing of Rotterdam on 14 May 1940, many of the construction drawings from 1905-1940 of the *Stadsarchief Rotterdam* no longer exist. This applies to the Fenix warehouse, so to form a holistic view of the building's history, the incomplete file is supplemented with photographic evidence and construction tender documents from the HAL archives. The formal syntax of the building and its evolution is closely tied to its story, reflexive of political agenda and place. Here is a quick history.

The situation on the *Veerlaan* began soon after the creation of the *Rijnhaven* in 1895, taking the form of two wooden sheds, very much like where Fenix I and II are today. The HAL leased the property from the City in 1915 with plans to develop it into the *San Franciscoloods* (Fig. 5.2). Its roof carried a series of cranes (Fig. 5.3) that moved on tracks at the waterside so that loading could take place directly from the ship to the ground or first floor. The architect was Cornelis Nicolaas van Goor, who was also responsible for assorted civic buildings in Rotterdam - several of which have since been designated historic at both national and municipal levels. The contractor was the *Hollandsche Maatschappij tot het maken van Werken in Gewapend Beton* (later known as HBM), who specialized in reinforced concrete construction. The building was one of the largest warehouses in the world at the time of building: 360 m long, 75 m wide, and 2-stories high. The interior is rational and repetitive (Fig. 5.4), with wide spans that maximize storage capacity.

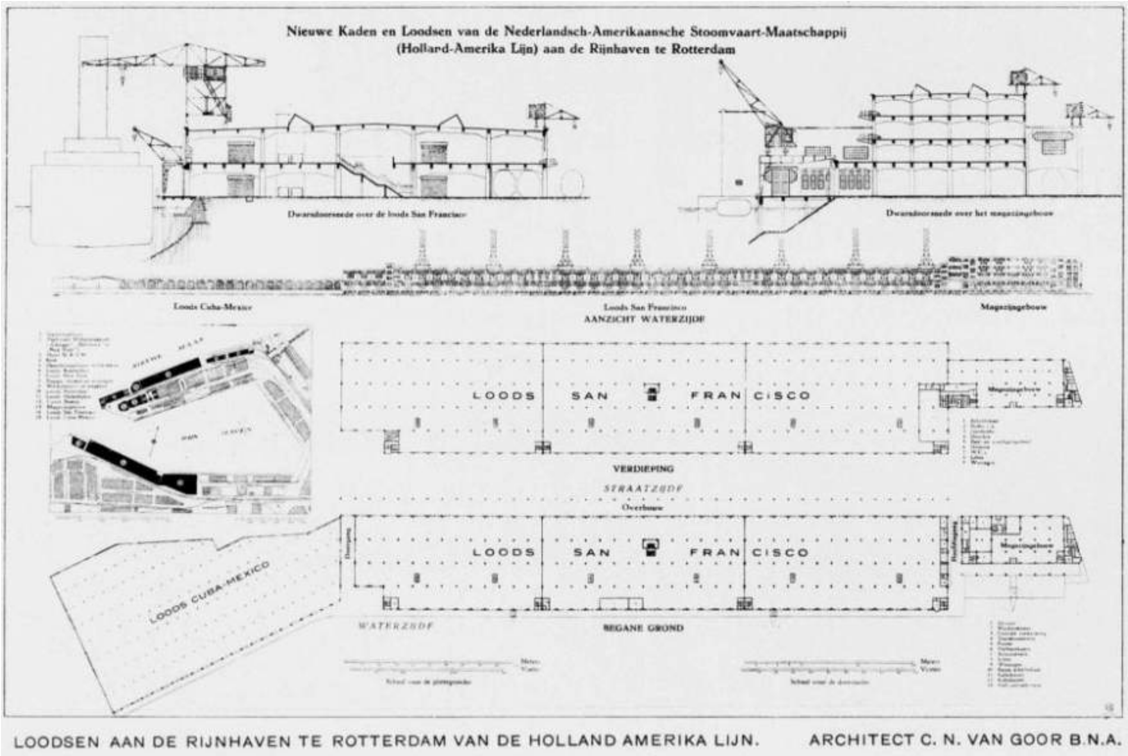


Fig. 5.2: The *San Franciscoloods* by C.N. van Goor (*Bouwkundig Weekblad Architectura*, 1931)

On 19 September 1944, the warehouses along the *Rijnhaven* were bombed and the *San Franciscoloods* was partially destroyed on the quay side. Then, as a mirror of how the city of San Francisco burned to the ground three times in its history, a major fire in 1947 collapsed the middle section of the *San Franciscoloods*. And as a the city of San Francisco has a phoenix on its flag, the *San Franciscoloods* was rebuilt as the *Fenixloodsen* from then on. Perhaps there is no correlation.

The destruction from the war and fire was the catalyst that divided the building into two separate structures in 1954: the Fenix I and Fenix II, with the new Fenixplein in between. The two northernmost bays on the waterside were destroyed and reconstructed as one bay with an inclining and chamfered roof. The quay was extended and the extra space allowed the cranes to be moved from the roof to ground level. As port activities moved further west toward *Hoek van Holland* and the North Sea, the Fenix warehouses were decommissioned by the 1980s.

Though the warehouses are not considered a *Rijksmonument* nor a *Gemeentelijk Monument*, the city of Rotterdam has shown a sensitivity to the history of the structure that should be preserved. Heavy bombardment from World War II destroyed much of Rotterdam's built history and what remains of it must be regarded with care.

Fig. 5.3: The Fenix warehouse from Katendrecht, pre-WWII (from the HAL archives at *Stadsarchief Rotterdam*)

Fig. 5.4: First floor of the Fenix warehouse, pre-WWII (from the HAL archives at *Stadsarchief Rotterdam*)

5.1.2 Fenix I

The owner and contractor of Fenix I, *Heijmans*, began plans for redevelopment in 2009. The city of Rotterdam was interested in redeveloping the *Kattendrecht* neighborhood by building new housing and mixed-use program. *Heijmans* and Rotterdam's planning department decided to add a new residential volume above the existing HAL warehouse, which was to house new creative, culinary, and cultural activities. A parking structure within the warehouse was also added as a requirement proportional to the number of units, which was specified by *Heijmans* to make the entire project financially feasible. The project team included Mei architects and planners, ABT Delft, CSM Steelstructures, and Techniplan.

The finished Fenix I complex consists of three structural layers (Fig. 5.5):

- The original concrete warehouse forms the base, which was retrofitted to adapt to the new mixed-use program.
- Then, a new steel table structure is the intermediate layer that transfers the new volume above through the warehouse and to new piles below. This supporting structure earned the *Nationale Staalprijs 2020*, required a kiloton of steel that was welded on-site, and had its own foundation woven between the existing.
- Above, the new residential structure was designed in concrete to meet the construction timeline and to build in enough mass to meet the Dutch acoustic requirements. It is eight stories on the waterside and slopes to four facing the Veerlaan, to mimic the aspect ratio of the surrounding streets. The addition of height also required the addition of new stability elements at the interior to withstand the resulting horizontal forces.

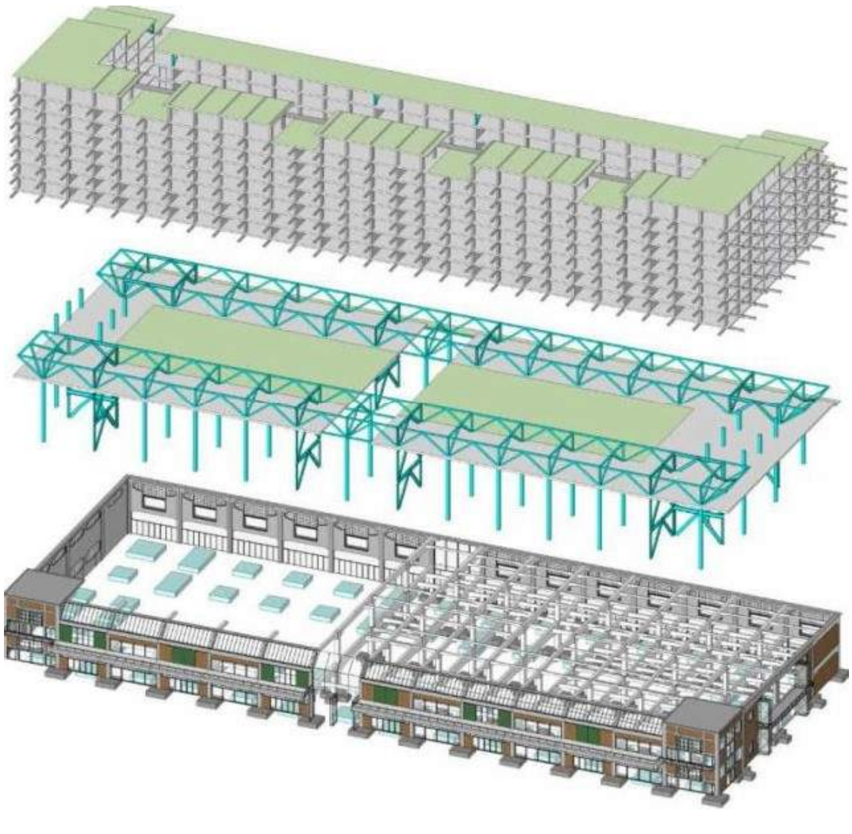


Fig. 5.5: Fenix I redevelopment structural design (Courtesy: *ABT*)

ABT conducted drilled core testing to determine the capacity of the existing superstructure columns, beams, and floors (Table 5.1). Most of the construction was found to be of low strength concrete. When calculated per the current standards, the design load of the floor appeared to be 5 kN/m^2. Several columns were retrofitted by jacketing to accommodate the new mixed-use program, which called for large holes in the floor and deleted columns. The condition of the interior structure was generally good, with most of the degradation occurring at the exterior south facade.

Table 5.1: Fenix I concrete strength classes in Eurocode notation (From research by *ABT*)

Structural element	Building portion	Concrete strength class per Annex D, NEN-EN 1990
Columns	New	C35/40
	Old	C12/15
	Old at Veerlaan Façade	C12/15
Beams	New	C20/25
	Old	C15/20
	Old at Veerlaan Façade	C12/15
Floors	New	C25/30
	Old	C20/25

Because the building is not designated historic, the team took certain liberties that would not have been permitted otherwise. The floor and columns of nearly half of the warehouse were demolished to build the parking structure. The Veerlaan facade was reconstructed to appear like the original. These were deemed essential because without the parking program, there would not have been a project.

Where possible, the warehouse was restored and protected even if it came at a time and financial cost. This presented a technical challenge at the foundations (Fig. 5.6), which turned out to have the lowest bearing capacity of all existing structural elements due to historic methods of calculation that were not as stringent as current codes, especially concerning settlement, soil types, and high water levels. The base of certain columns were replaced without removing the column, so temporary shoring frames were installed to hold the columns in the air for the new base installation. In addition, the columns required new piles, which were driven adjacent to the existing within the interior four meter ceiling height.

Fig. 5.6: Left: Shoring frame at column base. Right: Piles driven at interior columns. (Courtesy: *ABT*)

5.1.3 Fenix II

The *Stichting Droom en Daad*, a non-profit foundation that invests in art and culture projects in Rotterdam, acquired the Fenix II and initiated plans for redevelopment in 2018. The ground floor is to house mixed-use creative, culinary, and cultural activities similar to Fenix I. The first floor is to become the Fenix Museum of Migration (*Landverhuizersmuseum*), which will exhibit art and history related to Rotterdam's role in European emigration. The project team includes MAD Architects as the lead designer, Bureau Polderman as the restoration architect, and IMd Raadgevende Ingenieurs as the structural engineer. The main design move is to add a spiral staircase and atrium at the center of the building that leads to an observation deck (Fig. 5.7). The roof will not be accessible, but plans include a replica of the old crane rails and an extensive green roof. Construction began in early 2020 and is projected to finish in 2023.

Fig. 5.7: MAD Architects: Fenix II museum and mixed-use conversion, Rendering, 2020

5.2 Structure Assessment

5.2.1 Methodology

The available documentation is investigated prior to the test case intervention design. The steps detailed below are used to assess the current condition of the existing building (Florisson 2013; Hellebois 2013).

1. Record drawings and archive documentation are collected. Direct and circumstantial documentation related to the original 1916 building is found in the HAL archives. This includes the original structural calculations and the drawings. Interviews with the architect, structural engineer, and historical consultant of the Fenix I renovation and extension are conducted as supplemental information.

2. The structure is appraised in its present condition and visually compared to the documentation. Site visits do not show significant pathologies or defects in the interior. Degradation is mostly concentrated at the exterior south facade. The structure appears to be in a good state of conservation. Locations where the slabs and beams have been cut are compared to the drawings. The strength and durability of materials are determined: the existing documentation does not describe the strength of the specified materials, so the testing data from Fenix I is used as a reference (Table 5.5) in addition to the lowest allowable values listed in the *GBV 1912* (Tables 3.1 & 3.2).

5.2.2 Vertical Load-Bearing Superstructure

The existing load-bearing structure consists of cast-in-place concrete columns supporting concrete beams and slabs. A typical column bay (Fig. 5.8) is 8.60 m in the east/west direction and 13.50 m in the north/south direction. Columns measure 500x600 mm at the first floor and 700x1000m at the ground floor. The *moerbinten* (main beams) follow the transverse direction to provide stability and are 400x1000 mm at the roof and 500x1800 mm at level one. In the longitudinal direction, *kinderbinten* (secondary beams) are placed 4.5 m on center and are 300x600 mm at the roof and 400x900 mm at level one. All beams have gusset-like brackets as they approach supports, for moment resisting connections. The roof slab is 100 mm and the level one slab is 130 mm.

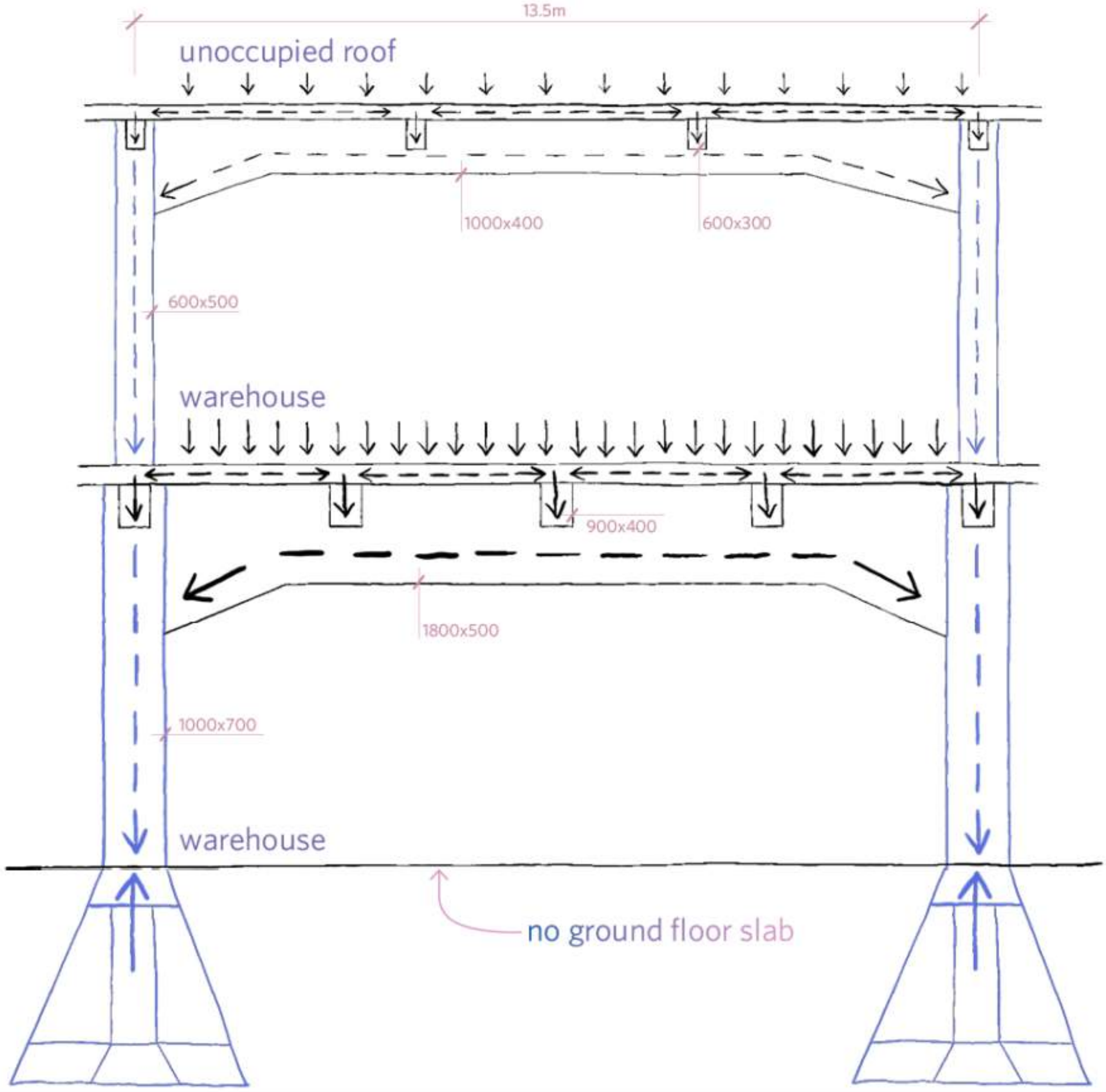


Fig. 5.8: Fenix II: typical column bay dimensions

Fig. 5.9: Fenix II: ground floor looking west, 1916 construction

There is no ground floor slab. When first in operation as a warehouse, a wood floor was installed. Over time, this was replaced with pre-fabricated reinforced concrete slabs (and easily removed for the current construction taking place, Fig. 5.9).

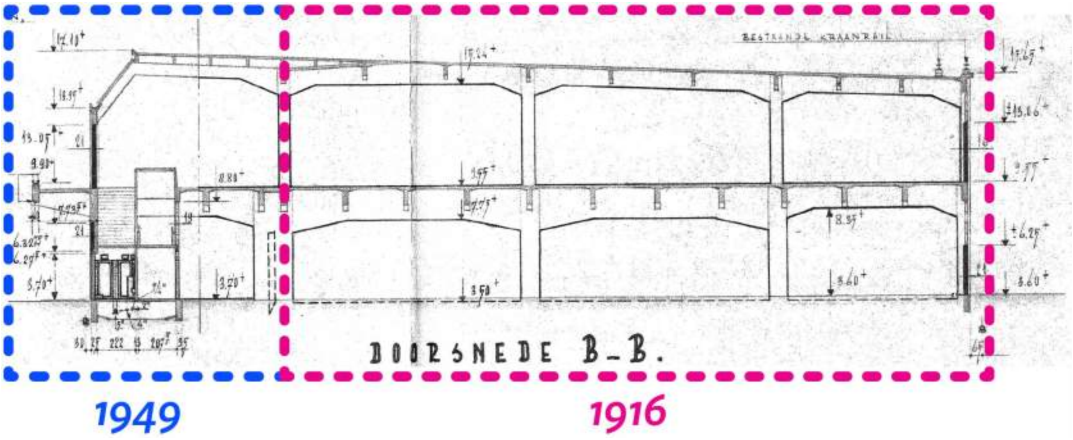


Fig. 5.10: Transverse section diagram: build-up from 1916 to 1949

The newer portion of the building that was rebuilt in 1949 (Fig. 5.10) generally follows the proportions of the original. Little documentation can be found and the calculation methods are unknown. However, the Fenix I material tests show that the concrete is consistently of a higher strength than in the 1916 construction.

The 1949 roof beams (Fig. 5.11) are pre-stressed concrete according to the Freyssinet system with improvements by Gustave Magnel (De Winter 1982). The post-War roof beams of the *San Francisco, New York*, and *Havana* warehouses that all belonged to the HAL are some of the first applications of prestressed concrete in the Netherlands.

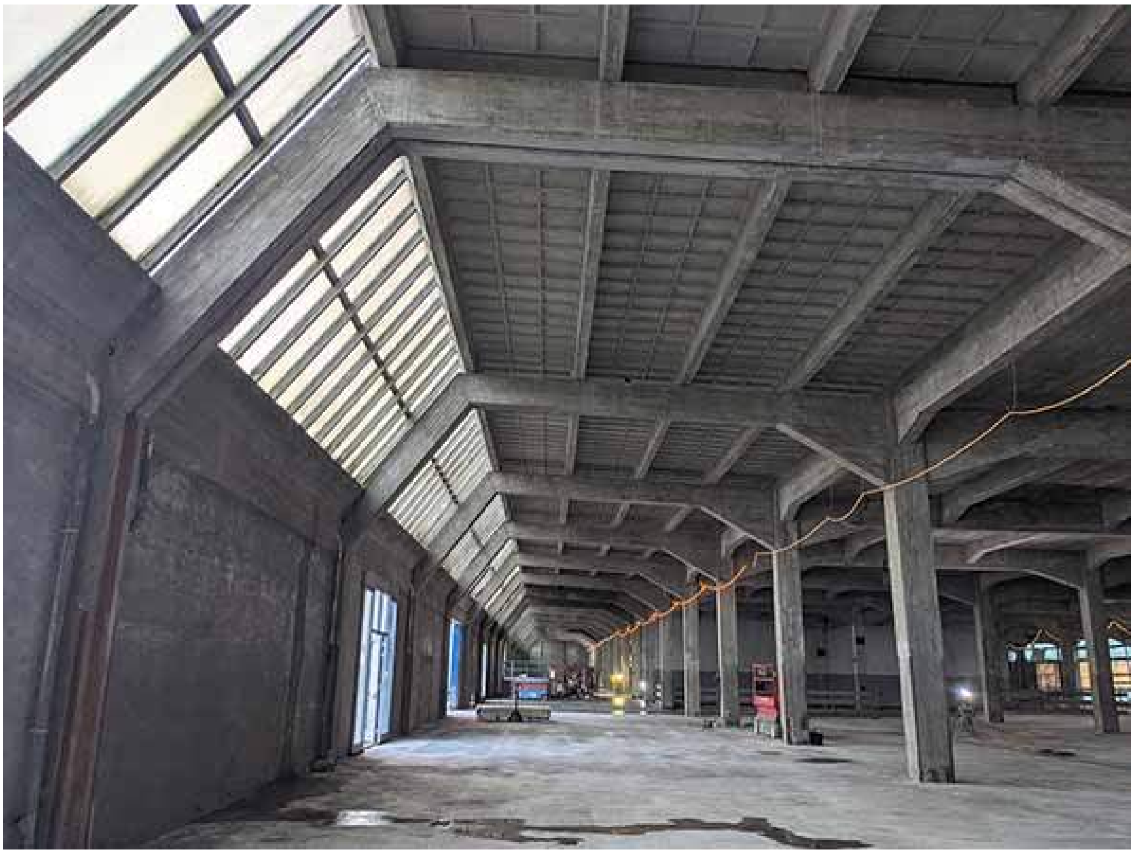


Fig. 5.11: Fenix II: first floor looking east, 1949 construction

5.2.3 Reinforcement

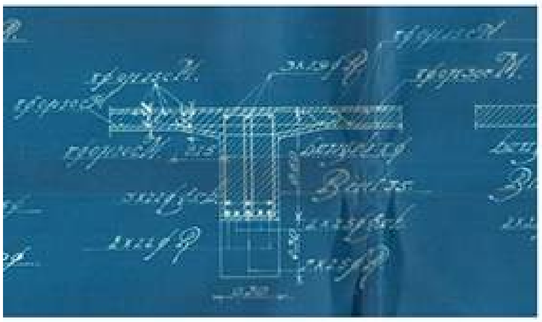
The main reinforcement is from plain smooth-surface round bars, which is typical for the 1920s. Interior columns with exposed rebar show a combination of horizontal flat bars and thinner round bars as secondary reinforcement (Fig. 5.12a), which is typical of Hennebique structures. Facade columns show horizontal round bars with a smaller diameter (Fig. 5.12b). In both cases, the vertical and horizontal bars are wire tied together. Reinforcement at beams (Fig. 5.13) do not appear to deviate from the drawings: two rows at the bottom, one row at mid-height, and one row of smaller bars at the top. No stirrups at the beams are visible, but the drawings indicate round bars, not the flat strips that are often seen in Hennebique systems.

(a) Level 1 interior column

(b) East facade column

Fig. 5.12: Column reinforcement at Fenix II

Fig. 5.13: Reinforcement at cut beam at Fenix II



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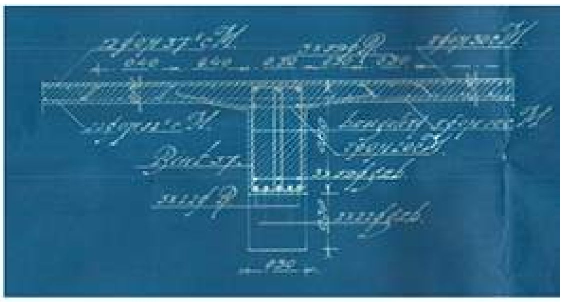

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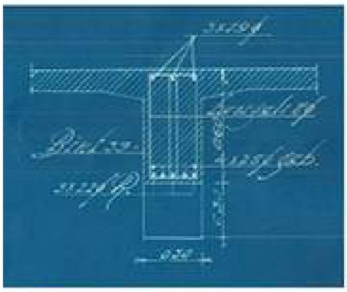

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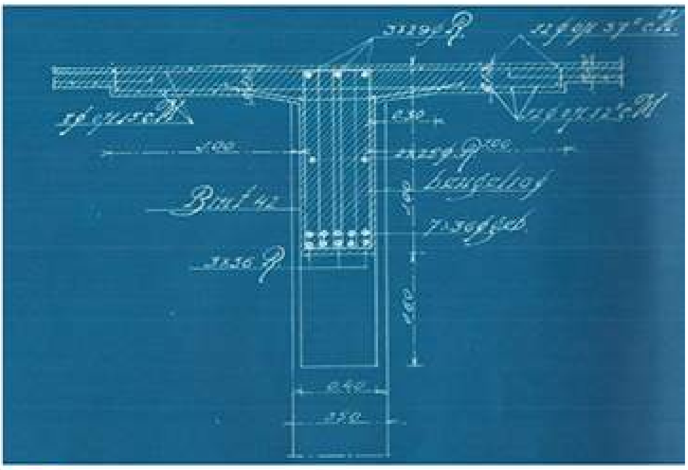

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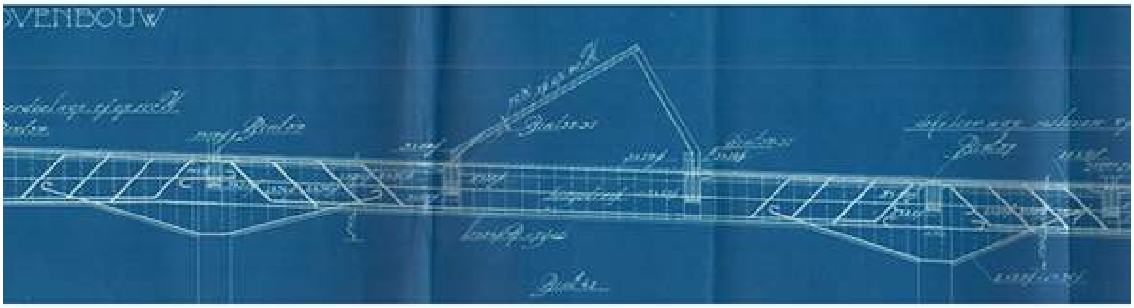

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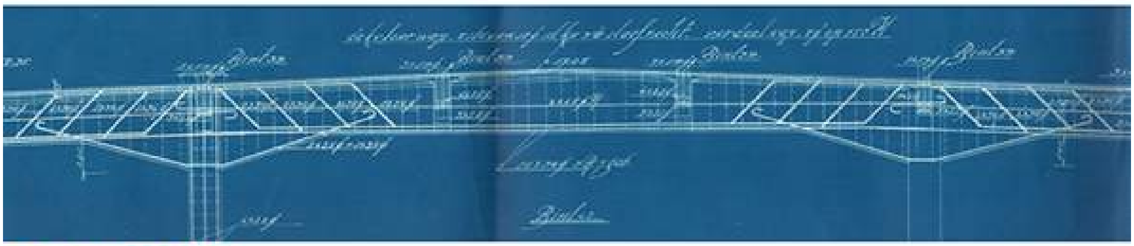

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Fig. 5.14: Reinforcement drawings - beams, *San Franciscoloods*

Reinforcement drawings of roof slab beams at the original 1916 building (Fig. 5.14), keyed to plan on following page (Fig. 5.15).

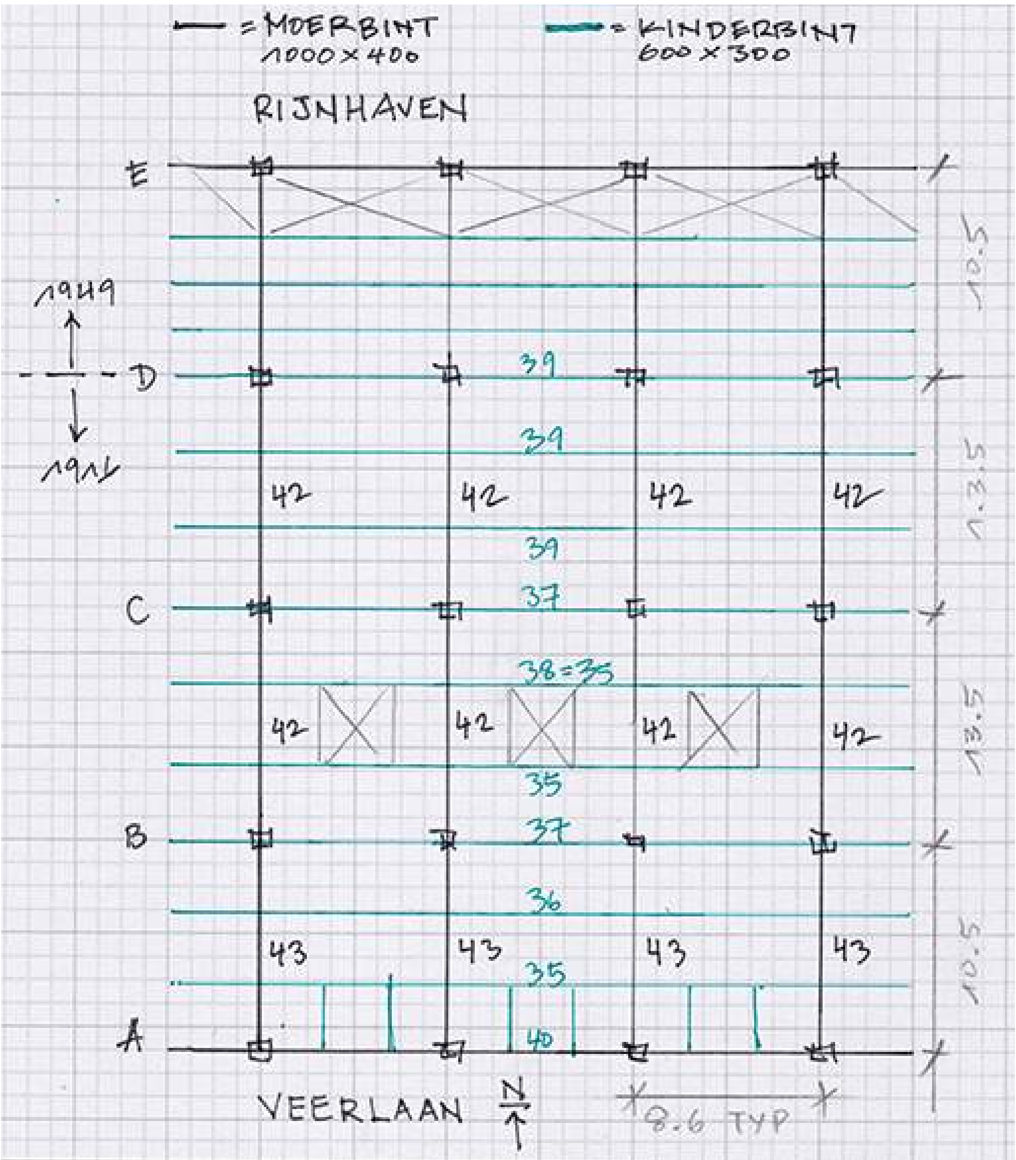


Fig. 5.15: Excerpt plan of roof slab, hatched magenta on Fig.5.16

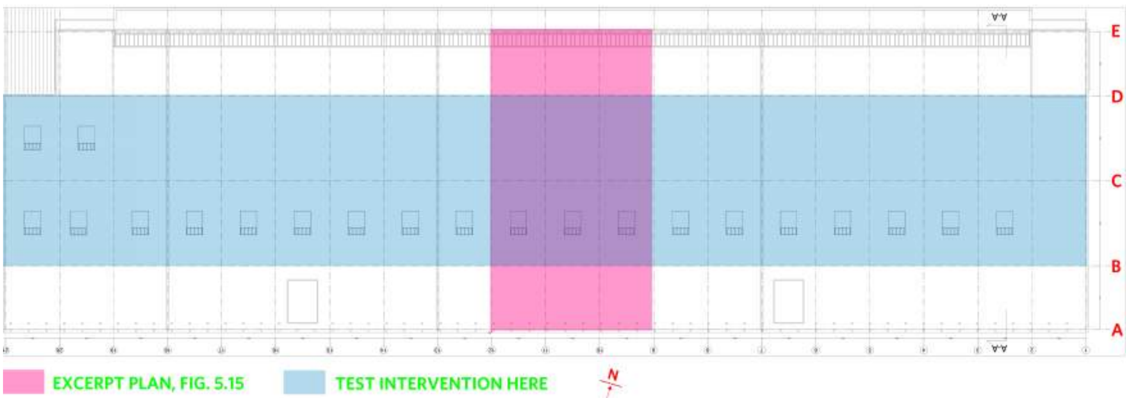


Fig. 5.16: Fenix II: plan diagram intervention area

5.2.4 Existing Capacity and Strength of Materials

Because the Level 1 slab is of a robust construction originally meant for warehouse activities of 5 kN/m^2, most new uses would not require strengthening. However, the roof was designed for a much lower 1 kN/m^2, mostly for maintenance, service, rain, and snow loads. For this reason, it is the focus of the design intervention, as its smaller elements were not intended to carry the loads of any occupiable use type.

Of the four column bays in the longitudinal (east/west) direction, the most appropriate spot to intervene would be between lines B and D (Fig. 5.16), since this portion is from the original 1916 construction. Skylights are centrally located in each bay between lines B and C (Fig. 5.9). Lines D to E were completely built in 1949, and lines A to B are not the typical 13.5 m span.

As any future alterations to the building should comply with present day Eurocode, the existing construction is also assessed by the same standard. Using the original design loads, the structural capacity of the reinforced concrete can be later compared to the effects of the new applied loads in the intervention design situations.

The strengths of the existing concrete and reinforcement are not stated in the available calculations, so the first step is to assess the plausibility of the lowest allowable values per the *GBV 1912*, the national concrete guideline at the time of original construction. When translated into Eurocode 2 notation, these values are C8 concrete and B250 reinforcement (§3.3). The roof slab, beams, and steel reinforcement are calculated in two limit states, as described in NEN-EN 1990: Ultimate Limit State (ULS), which is related to safety and strength, and Serviceability Limit State (SLS), which is related to usability. Detailed calculations of the existing roof structure can be found in Appendix A.

The distinction between ULS and SLS is defined as:

$$ULS : q = \gamma_G \times G_k + \gamma_{Q;1} \times Q_{1;k} + \Sigma(\gamma_{Q;i} \times \psi_{0;i} \times Q_{i;k}) \tag{5.1}$$

$$SLS : q = G_k + \gamma \times Q_{1;k} + \Sigma(\psi_{0;i} \times Q_{i;k}) \tag{5.2}$$

Other variable loads, $Q_{i;k}$, are reduced by their associated combination factor, $\psi_{0;i}$.

Characteristic loads, G_k & Q_k, are increased by the following partial factors:
γ_G = 1.2 permanent loads (unfavorable), which is reduced from 1.35 by a factor, ξ=0.89
γ_Q = 1.5 variable loads (leading and other)

Characteristic strengths, f_k, are reduced by material partial factors, γ_M (fundamental combinations only; use γ_M = 1.0 for serviceability). To determine design strengths, f_d, the Dutch national annex to EC2 specifies γ_M as 1.5 for concrete and 1.15 for reinforcement:

$$f_d = \frac{f_k}{\gamma_M} \tag{5.3}$$

5.2.4.1 Roof Slab

The ratio of the boundary dimensions and presence of hinged line supports along all four edges allow the slab to be analyzed as a one-way slab (Fig. 5.17), though some two-way behavior is expected toward the supports in the direction of the shorter span. The distribution of reinforcement is not completely uniform; additional bars are added to the top of slab over the supports along column lines, indicating that it is designed to take areas of negative moment. However, with the exception of columns, the absence of walls or other structural elements either above or below the supports make it so that the boundary conditions cannot be considered fully clamped.

The largest stresses are expected to be at the positive moment at mid-span between supports. With the original roof live load of 1.4 kN/m^2 and concrete self-weight at 2.4 kN/m^2, the design bending moment of a unit strip width of 1 m is calculated using the theory of elasticity, like a simply-supported beam:

$$M_d = \frac{1}{8} \times q \times L_y^2 \tag{5.4}$$

The design moment is divided by the ultimate bending capacity of the section:

$$M_u = K_{lim} \times f_{cd} \times b \times d^2 \tag{5.5}$$

where:
K_{lim} limiting factor of 0.167 to ensure ductile failure
f_{cd} design strength of concrete
b 1 m slab design strip width
d slab depth to center of tension reinforcement

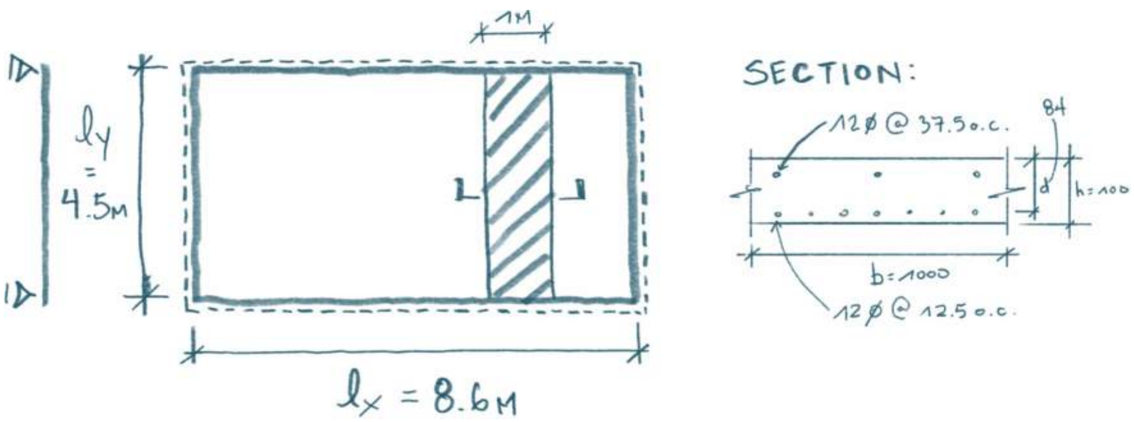


Fig. 5.17: Typical one-way roof slab, static scheme, and analyzed section

The resulting unity check using C8 concrete is 2.01 (detailed calculations in Appendix A.1), a utilization that far exceeds the moment capacity. This suggests beyond a reasonable doubt that the concrete is not of such low strength; the slab would not have spanned for 100 years otherwise. Because the Fenix I material tests showed C20/25 concrete at the 1916 portions of the level one slab, this value (along with the same B250 steel) is input into the moment, reinforcement, deflection, and shear calculations.

Existing reinforcement is reverse calculated based on formulas derived from the simplified stress block described in EN 1992-1-1 §3.1.7(3) and (IStructE 2006). The compression reinforcement, 37.5 cm on center, exceeds the maximum allowed spacing (EN 1992-1-1:2011 §8.2(2)) and can be considered nonstructural. The lever arm, z, between the center of the compressive force and the center of the tension reinforcement is found using the following quadratic equation:

$$z = d \times \left(0.5 + \sqrt{0.25 - \frac{K_{lim}}{1.134}}\right) \tag{5.6}$$

Then the area of required steel, A_s, is given by Eq. 5.7, where f_{yd} is the design yield strength of reinforcement steel.

$$A_s = \frac{M_d}{0.87 \times f_{yd} \times z} \tag{5.7}$$

Deflection, w_{max}, is calculated as a simply-supported beam using SLS loads and is limited to 0.004 times the span:

$$w_{max} = \frac{5 \times q \times L^4}{384 \times E \times I} \tag{5.8}$$

By inspection, shear reinforcement for the 100mm thick slab is probably not needed. The design shear force, V_{Ed}, is checked against the shear resistance of the slab without reinforcement, $V_{Rd,c}$.

With:

$$V_{Ed} = \frac{q \times L}{2} \tag{5.9}$$

$$V_{Rd,c} = 0.12k(100 \times \rho \times f_{cd})^{1/3} \times bd, but \leq 0.035 \times f_{cd}{}^{0.5} \times k^{1.5} \times bd \tag{5.10}$$

where:

k	$1 + \sqrt{\frac{200}{d}}$
ρ	$\frac{A_{s,prov}}{bd}$
$A_{s,prov}$	provided area of tension reinforcement steel

As shown in Appendix A.1, assuming a lowest allowable strength of C8 by the 1912 guidelines results in a slab that is unable to support its original design loads. In contrast, the higher C20 strength appears consistent with the loads and is corroborated by the Fenix I material test results. In the absence of any specific testing and with the knowledge that they were originally one building, it is then reasonable to assume that the concrete quality of the Fenix I first floor is the same as at the Fenix II roof. From here, C20 concrete is used in all subsequent roof slab calculations.

The results using the higher strength are summarized in Table 5.2. When the original design loads are applied to the existing slab and calculated using present-day methods, there is almost no excess capacity. Reinforcement is governing, so if the loads of a new imposed use exceed that of the original, the slab is expected to fail as the steel yields.

Table 5.2: Existing roof slab utilization using C20/25 concrete and B250 reinforcement, original design loads

C20/25*	Max moment	Reinforcement	Deflection	Shear
UC	0.8	1.07	0.53	0.18

*f_{cd} = 13 *MPa* ; f_{yd} = 217 *MPa* ; q = 4.98 *kN/m*^{2}

5.2.4.2 Roof Beams

The existing concrete beams are integrated into the slab construction, indicating that they were poured monolithically and are of the same strength. Together with an adjacent section of the slab on either side where simplified stress conditions can be assumed (EN 1992-1-1:2011 §5.3.2.1), the beams and their effective flange widths are considered as T-sections.

The geometry of the T makes it so that the neutral axis lies within the web, and the stress block (Fig. 5.18) is partially located in the downstand beam. Based on the strength of concrete (C20) and the distance of the neutral axis in relation to the top, a rectangular portion of the beam can be assumed to be in uniform compression. The compressive force in the flange and web are then balanced by the steel reinforcement at the tension zone in the base. Because the beam engages the slab to act as one section, the ultimate moment capacity of the T is higher than if the beam is considered as just a rectangle.

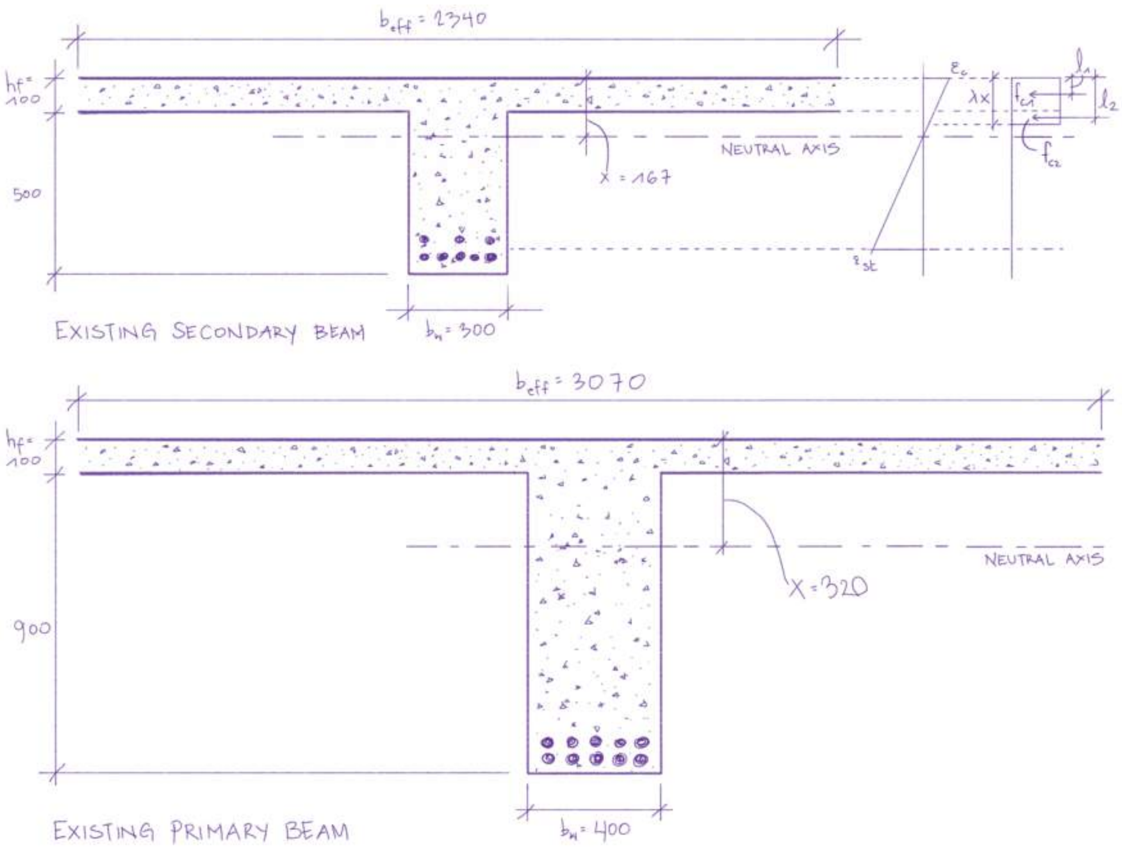


Fig. 5.18: Existing concrete beam section, strains, and stress block

Then as with the slab, the secondary and primary beams are checked using similar methods and equations as in §5.2.4.1. The unity checks for moment, reinforcement, and deflection are summarized in Table 5.3, with the full calculations in Appendix A.2-3. Both sets of beams show that while they are still governing in ULS, the utilizations are less than in the slab.

Table 5.3: Existing secondary and primary roof beam unity checks, original design loads

Secondary beams	Max moment	Reinforcement	Deflection
UC	0.85	0.73	0.3

Primary beams	Max moment	Reinforcement	Deflection
UC	0.41	0.48	0.26

5.3 Conclusions

The Fenix II is a logical candidate to be retrofitted for reuse. It pre-dates World War II, which is not typical for Rotterdam. The positive characteristics, such as substantial original design loads and high ceilings, of the building as a relic of industry make it advantageous to speculative developers. Additionally, the centrally located site make reuse a financially interesting option, as space available for new construction is limited.

To determine whether (and to what degree) the existing roof would require strengthening in the event of new imposed loads, the elements were reverse calculated from the available given data. The necessary information was extracted from the original calculations and drawings: design loads, area of provided reinforcement, and dimensions. The strengths of concrete and reinforcement were derived from the minimum values as recommended in the *GBV 1912* or from the Fenix I tests, whichever is greater. Though the documents also include historic methods of calculation, these were not needed because 1. they used (now) outdated coefficients and equations that are difficult to decipher and 2. the structure was calculated according to the current version of Eurocode 2.

The resulting unity checks from the original load calculations showed that while the roof slab is certainly not oversized, the secondary beams do have some excess capacity, and the primary beams even more so. Because of the near utilization of the slab, the strengthening of it in response to different load increases will be the focus of intervention in the following chapter. How the loads are directed to the network of existing beams will also be reflected in the efficiency of the intervention section design and configuration.

6 Case Study: Design Intervention

Now, let us decide the future of the roof. For the aim of this study, the new scenarios should be heavy enough to require strengthening but knowing the innate low strength of historic concrete foundations, it should not be so heavy so as to turn the project in a geotechnical direction.[1] The new uses should also relate to the neighborhood and add value to the building.

6.1 Precedents

Because of the large surface area (155x48 m) of the Fenix II roof, a mixed-use program with varied load requirements provides the opportunity to test the intervention for different degrees of capacity increase. Some that may go well together are:

- Green roof with a more substantial soil profile than proposed in the new Fenix II design
- Nightclub at night and event space / cafe during day

Some precedents of occupiable roofs over industrial or historic buildings are:

- *Klunkerkranich* (Fig. 6.1) in Berlin-Neukölln: a biergarten, a green garden, cafe, bar, and nightclub above a parking garage
- SoHo House in Los Angeles' Art's District: with a pool and trees above a former plumbing and metal supply warehouse
- Suicide Club in the center of Rotterdam: restaurant and bar above the *Groothandelsgebouw* - a listed postwar concrete office building

Fig. 6.1: *Klunkerkranich*, Berlin-Neukölln (https://klunkerkranich.org/)

1. Unfortunately, this excludes all swimming pool designs. Sad!

6.2 Load Tracking

Assemblies and load combinations are defined for the new use types, then applied to slab and beam areas to:

- Determine element weight and distribution
- Design elements for gravity loading

6.2.1 Speculative Future Use

The proposal (Fig. 6.2) is to allow public access to a roof garden, skateboarding activities, and a cafe/bar - all of which contribute to reducing energy bills, promoting biodiversity, increasing the well-being and productivity of people, providing jobs, generating additional revenue, and making the museum below more attractive to visitors.

The **roof garden** is *not* a grass field full of photovoltaic panels. It is an intensive green roof system that supports assorted vegetables, grasses, shrubs, rocks, and mud piles. This choreographed landscape also houses a symphonic mix of mid-sized animals, bugs, and non-poisonous reptiles.

Whether one skates or not, the **skatepark** is a safe haven for people to get in trouble, fall in love, and find personal sovereignty. It is here where we remember to forget what Veronica Sawyer calls our "teenage angst bullshit." I'll leave it at that and let you fill in the rest.

The **cafe/bar** is a place where hardworking people skip work, talk about things that do not matter, half-watch an outdoor film screening, and stare at the river. It is easy and uncomplicated, where people are friendly yet well-dressed. The space is programmed all day and night. You can bring your kids, dog, and mother.

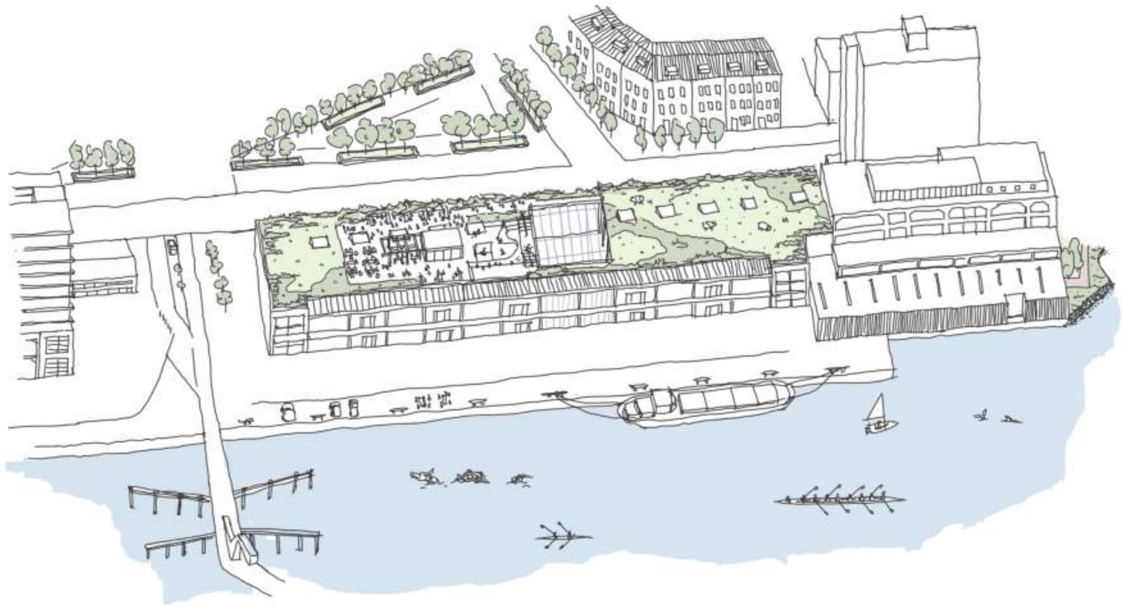


Fig. 6.2: Overview of design intervention and site. The River - who knows what lies - beneath its surface? - monsters? - answers? - forgotten gold? - it's the great unknown - that's what it is

6.2.2 Material Weights & Assemblies

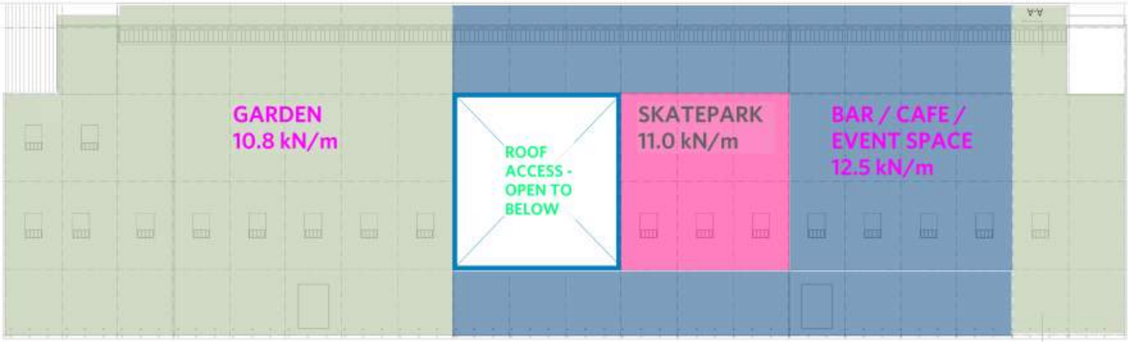


Fig. 6.3: Plan diagram: new uses and loads

The new uses are laid out per Fig. 6.3 and the intervention is designed according to the loads specified in Table 6.1. With safety factors included, the original design load of 4.98 kN/m^2 is more than two times less than the lightest of the proposed scenarios (garden). In adapting a previously unoccupied roof for new uses, replacement of roofing is an expected part of the complete retrofit scope of work, so an estimated weight is added to the dead loads in all scenarios. The garden is not intended to hold crowds of people in addition to the planted areas, so it only includes a small service path live load. The cafe is assumed to include some partition walls, shelter, and shading - this is added as a movable live load.

Table 6.1: Dead and live loads at proposed new uses

Garden			
Permanent	Roofing	0,5 kN/m^2	estimated
	0,3 m dirt	4,5 kN/m^2	packed + saturated earth
Variable	Service path + maintenance	1,0 kN/m^2	EN 1991, H: roofs
	Some animals	0,3 kN/m^2	unit weight 2 pigs
Skatepark			
Permanent	Roofing + finishes	0,5 kN/m^2	estimated
Variable	Areas w/ physical activities	5,0 kN/m^2	EN 1991, C4: congregation
	Point load	7,0 kN	EN 1991, C4
Café/bar			
Permanent	Roofing + finishes	0,5 kN/m^2	estimated
Variable	Areas w/ physical activities	5,0 kN/m^2	EN 1991, C4: congregation
	Partitions	1,0 kN/m^2	estimated
	Point load	7,0 kN	EN 1991, C4

Self-weight of materials:
ρ_c = 24 kN/m^3 normal weight concrete
ρ_g = 25 kN/m^3 normal weight glass

6.3 Design Development

The boundaries of the study area are the extents of a typical column bay: 13.5 m x 8.6 m (Fig. 6.4). Secondary beams spanning 8.6 m occur in the short direction at 4.5 m on center. The design principle is to find a solution that is minimally invasive, that works with and optimizes the utilizations of existing elements, that can be surgically installed without effecting historic characteristics. This is studied through different configurations of new elements and through the composite design of existing concrete together with glass.

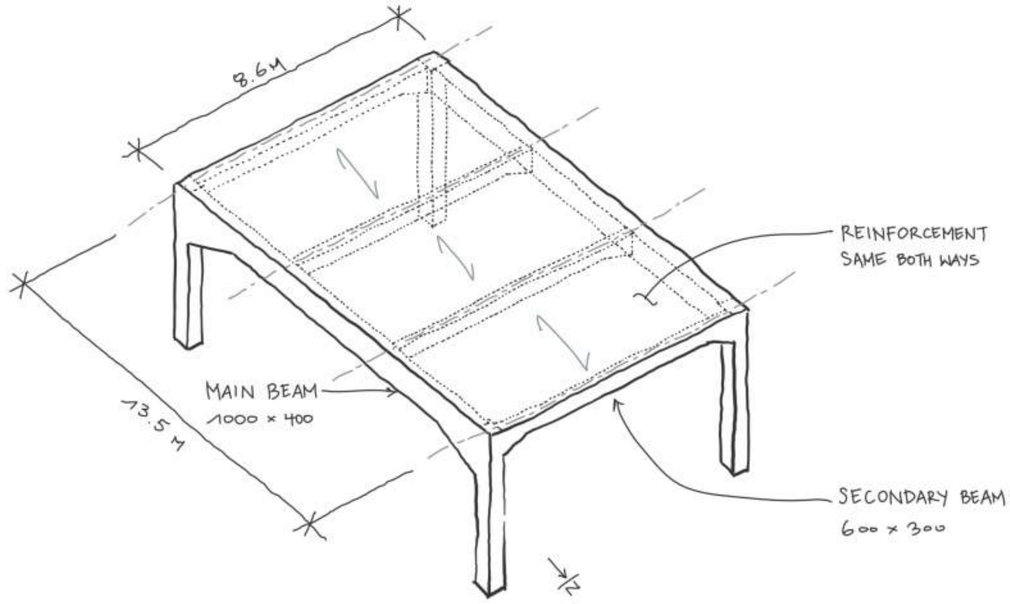


Fig. 6.4: Axon diagram: existing typical column bay at roof

The unity checks of the existing roof structure show utilizations of 100% at the slab, 85% at the secondary beams, and 48% at the main beams. Further increases in structural load will require strengthening at the slab and likely also at the beams.

6.3.1 Configurations

The initial design iterations add a third series of beams to the existing hierarchy (Fig. 6.5), reducing the slab tributary width. Since the existing reinforcement is the same both ways, it is possible to change the direction of the slab span. The loads are first transferred to the secondary beams, then the primary beams. Because of the order of construction in a retrofit situation, because the slab will not be shored or jacked up, and to simplify the process of this first study, the glass beams are calculated to accept all of the new loads. It is also assumed that the self-weight of concrete is carried by the existing concrete elements.

The minimum number of added beams is determined based on whether the slab can still span between the new glass beams, with the loads from the new uses. This eliminates any solution with only one beam. The dimensions and configurations of the beams per use scenario are summarized in Table 6.2. Taking the lowest floor loads of the garden as a representative case, detailed calculations can be found in Appendix B.1.

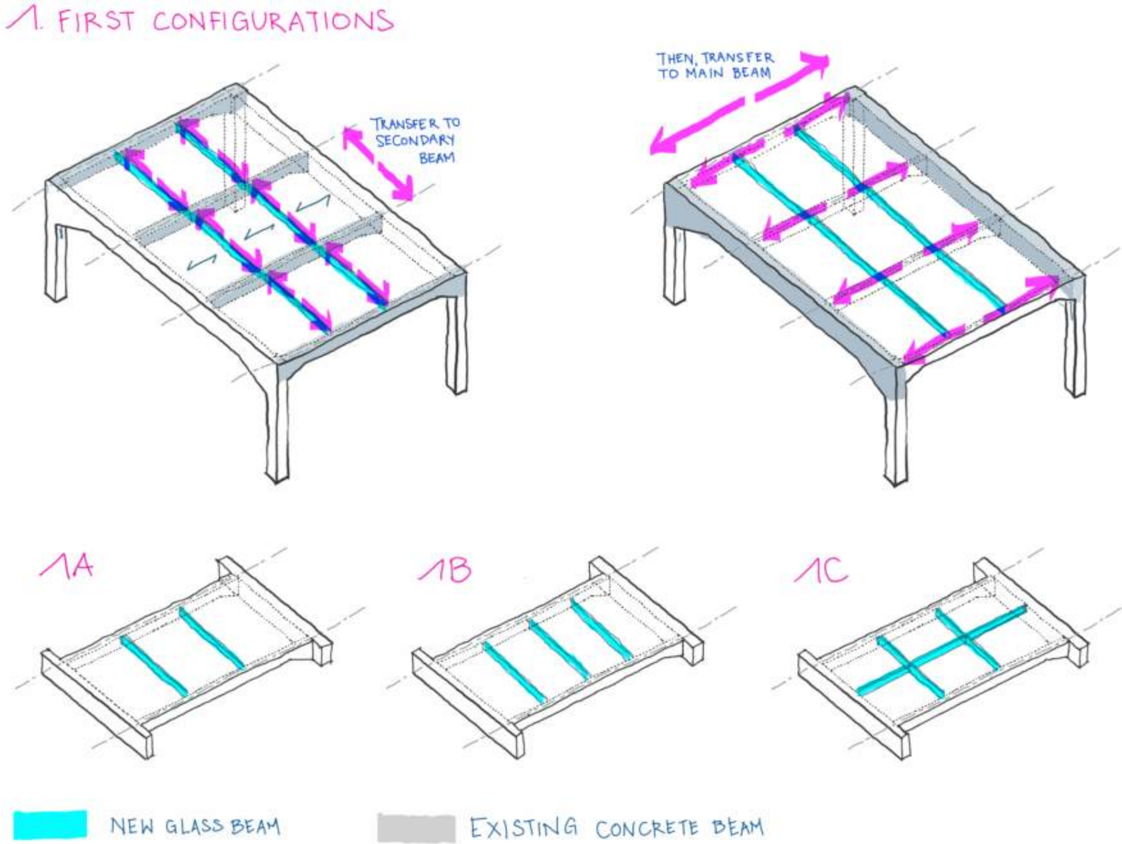
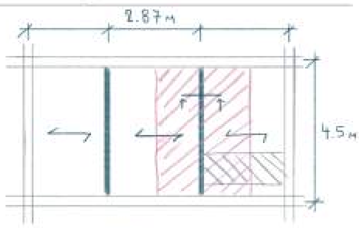
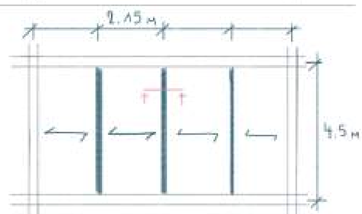
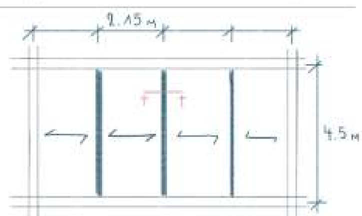


Fig. 6.5: First design: load transfer scheme and glass beam configurations

Table 6.2: First design: loads, geometry, and configurations, where glass elements accept all new loads

	Q_{ULS}	Geometry	Configuration
Roof Garden	25.13 *kN/m*	84x320 7-ply x 12 mm	
Skatepark	18.38 *kN/m*	75x285 mm 5-ply x 15 mm	
Cafe/Bar	21.68 *kN/m*	75x310 mm 5-ply x 15 mm	

Since the glass beams are installed at bottom of slab, which is 5.75 m above finish floor, it is unlikely that they would be accidentally knocked or tampered with at that height. A possible extraordinary load situation would be a fire. Fire limit state can be written as:

$$q = G_k + \psi_{2,1} \times Q_{k,1} + \Sigma(\psi_{1,i} \times Q_{k,i}) \tag{6.1}$$

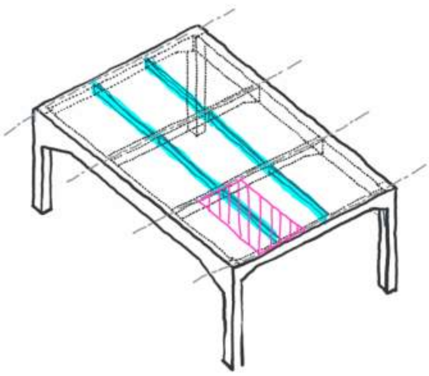
where:

G_k characteristic permanent load
$\psi_{2,1}$ reduction factor at leading variable load (0.6 at congregation areas, 0 at roofs)
$Q_{k,1}$ leading variable load
$\psi_{1,i}$ reduction factor at other variable loads (0.7 at congregation areas, 0 at roofs)
$Q_{k,i}$ other variable loads

The glass beams are calculated as if two of their plies were taken away, to account for fire on either side that reduces the effective cross-section. When the fire moves past the outer layers of glass, a transparent intumescent interlayer can act as a fire shield. As calculated, fire is not governing in any case (the bottom of the glass beam is not protected), so there is no change to the section. However, without a clear national standard for designing under fire, further development should be coordinated with the City.

6.3.2 Composite Interaction

To see what reduction in the cross-section would be possible if the glass beam engages the concrete slab to act together structurally, they are calculated as a T-section using the transformed method area for composites then input into the flexure formula (Fig. 6.6). In the absence of any directly relevant codes, the design basis for the effective width of the concrete flange is taken from Eurocode 4: Design of composite steel and concrete structures (EC4). Refer to Appendix B.2.



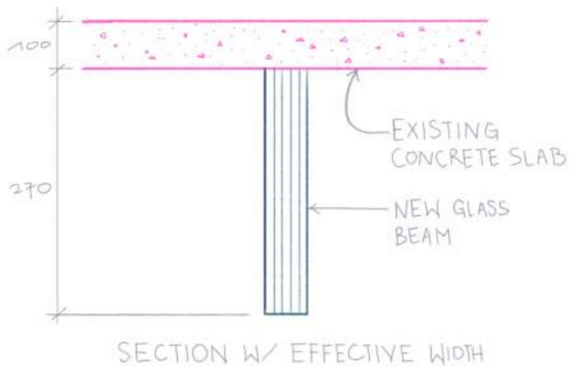


Fig. 6.6: Transformed slab section for composite action

Sizing the beam as a composite reduces the section by 30-40% (Table 6.3). It should be noted that the calculations do not consider the existing steel reinforcement at slabs, which is shown to be at total utilization in previous analysis. The results also show that deflection is governing, rather than bending. Still, full composite action is dependent on the connection design of the beam to slab.

Table 6.3: Comparison glass beam with and without composite interaction

	Separate	Composite	Reduction
Roof garden	84x320 7-ply x 12 mm	60x270 mm 5-ply x 12 mm	40%
Skatepark	75x285 mm 5-ply x 15 mm	60x250 mm 5-ply x 12 mm	30%
Bar	75x310 mm 5-ply x 15 mm	60x270 mm 5-ply x 12 mm	30%

6.3.3 Existing Beam Strengthening

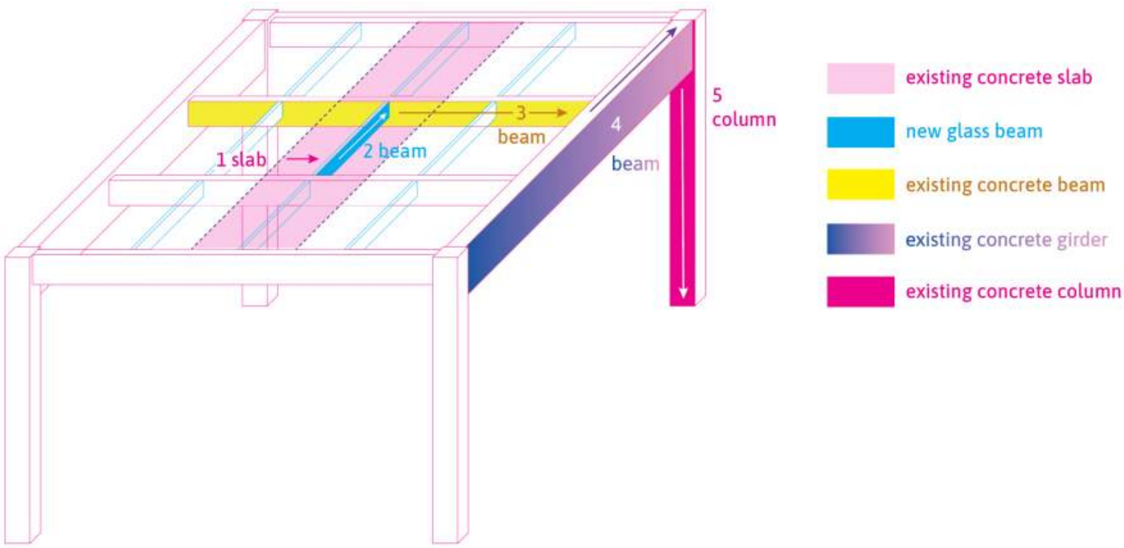


Fig. 6.7: First design: gravity load path

Recall that the utilizations of the floor elements vary (Tables 5.2 & 5.3) and that the existing beams may need to be strengthened. Because the order of force transfer (Fig. 6.7) goes from slab - glass beam - concrete beam - concrete girder - column, the existing concrete beams are checked for whether the available carrying capacity is exceeded.

First, the effects of the lowest load scenario on the secondary beams are calculated. Even though there is some room in the existing concrete calculations, it is not enough to compensate for the added loads and the utilization is decisively exceeded at 204% (Appendix B.3). As a result, the secondary beams must be strengthened, so new beams are added below and designed as a transformed section with their associated effective widths. The high utilization also indicates that it is likely that the main beams would also require reinforcement, ultimately ending with required strengthening of all floor elements. For consistency, glass is used as the strengthening material. (Fig. 6.8).

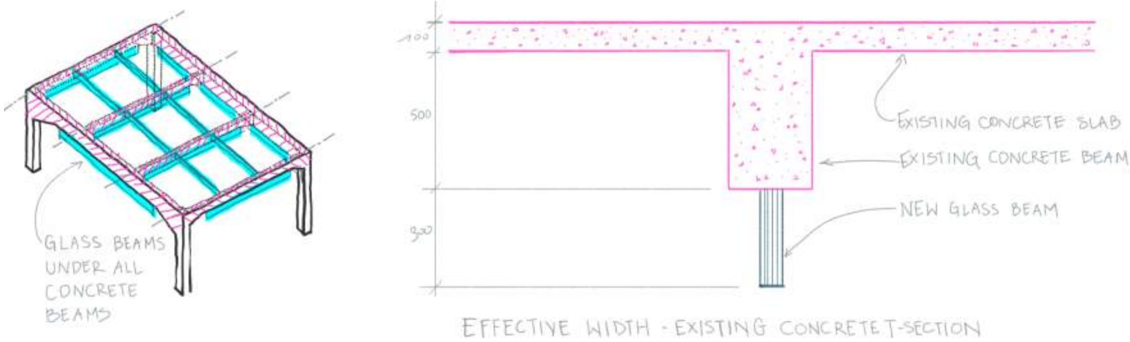


Fig. 6.8: Existing beam strengthening with glass

6.3.4 Evaluation of Initial Studies

The initial design studies (Fig. 6.5) looked for a configuration that used the least amount of material while providing adequate strengthening of the slab. Option 1A was used for the garden, where the minimum addition of two beams was due to the floor slab governing in bending. Option 1B, adding three beams, was used for the heavier skate and bar area, providing smaller sections and a greater level of safety than if it was just two. Option 1C, adding an additional longitudinal beam, reduced the sections of all new elements but ultimately resulted in more material overall. It also requires a glass-to-glass connection design at the intersection of beams, which may be a more complex detail than at the other glass-to-concrete options, so it is no longer considered.

Because the roof garden is the lightest of the new proposed uses yet uses 204% of the existing beam capacity, it is reasonable to project that the other heavier uses will also require strengthening of both primary and secondary beams in the current scheme. Ultimately, this strategy is not preferred: the number of new elements results in a visually cluttered intervention that contradicts the design parti of historically sensitive retrofit, altering the existing hierarchies and perceptions of space. Additionally, the associated work and amount of material needed to strengthen every existing concrete beam would extend the construction schedule and require more complicated assembly. For these reasons, orienting the glass beams toward the secondary beams is architecturally and financially unacceptable.

6.4 Final Design

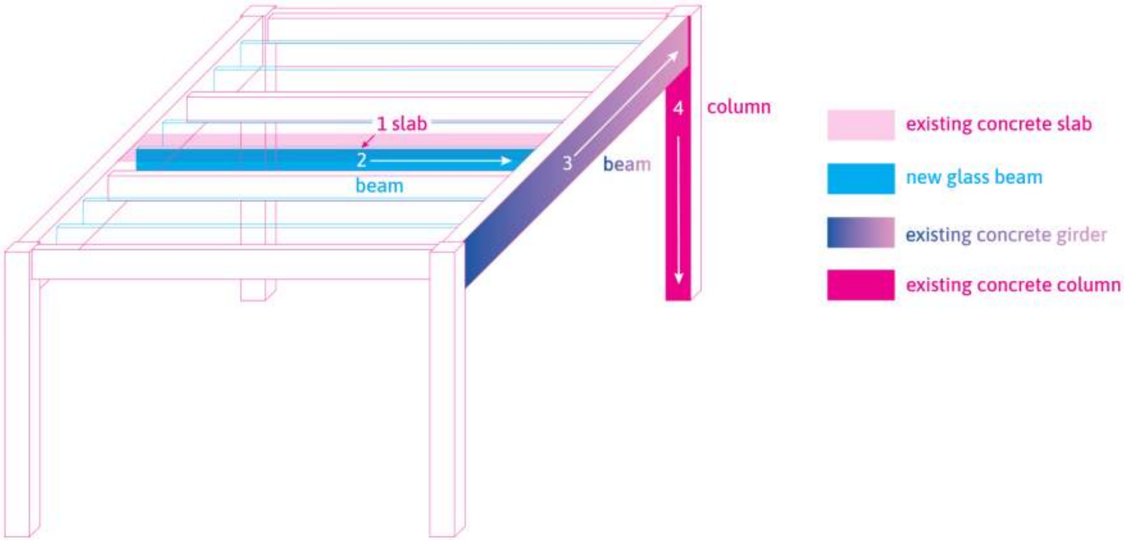


Fig. 6.9: Final design: gravity load path

6.4.1 Glass Beam Design and Results

In the final iterations, the new beams are configured to transfer directly to the existing primary beams, bypassing the strengthening of the secondary beams and reducing the total amount of reinforcement required (Fig 6.9). For ease of construction and because the amount of anchorage needed for fully composite action may not be worth the savings in material, they are designed to be independent from the slab.

With the exception of the green roof, the intervention calls for the addition of two new glass beams per bay (Fig. 6.10). Options 2A and 2B provide the most minimal intervention, with two added beams giving a higher level of safety. 2B is more appropriate for the heavier bar and skate uses, where there is also a greater risk of injury to occupants in the event of failure. Option 2C is to place two new beams parallel and close to the face of the existing secondary beams, then add a series of perpendicular beams that transfer to the glass rather than the concrete. This requires more material and introduces a new glass-to-glass connection type that adds complexity to the construction. Though it is not the most efficient solution for a typical bay, it may be appropriate where the skylights occur so that the beams do not fly across the center of the void in the slab.

The final sizes and placement of elements are stated in Table 6.4, with detailed calculations in Appendix C. As with the previous studies, the slab is checked for whether it can still span, and the maximum bending moments of the new glass elements and their deflections. The thickness of the plies are determined based on the largest that is commercially and readily available, so should not exceed 15mm. The number of plies is an odd number so that glass, and not the interlayer, is at the center of the section. The beams are calculated as separate from the slab and are sized to accept the entirety of the new loads. Note that the rotated orientation of the new beams, in comparison to the initial studies, increases the span to 8.6 m, resulting in a much deeper beam of 410-510 mm.

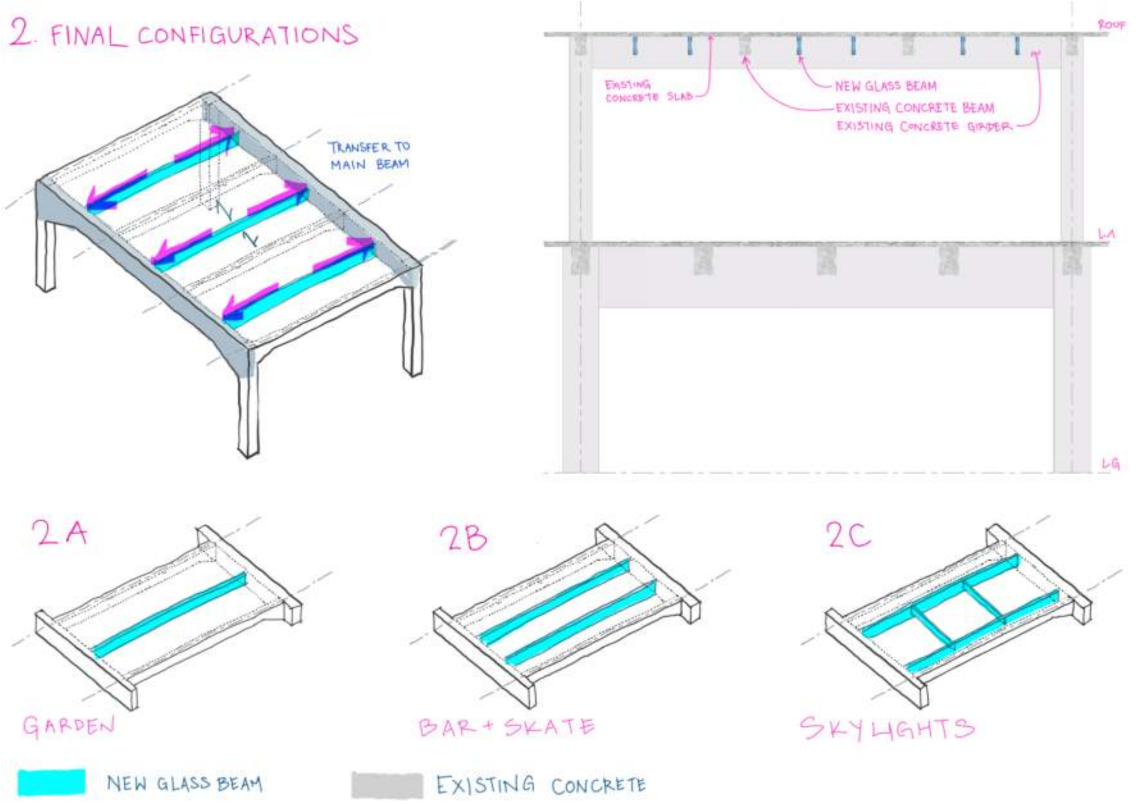
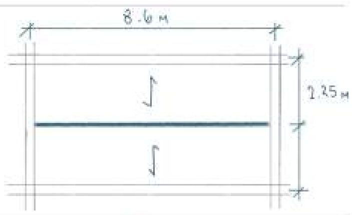
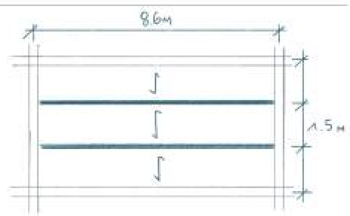
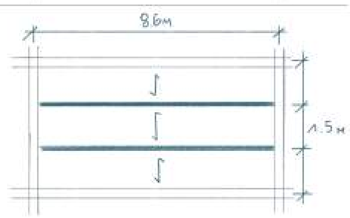


Fig. 6.10: Final design: load transfer scheme, section, and glass beam configurations

Table 6.4: Final design: loads, geometry, and configurations, where glass elements accept all new loads

	Q_{ULS}	Geometry	Configuration
Roof Garden	11.42 *kN/m*	84x410 mm 7-ply x 12 mm	
Skatepark	12.98 *kN/m*	84x450 mm 7-ply x 12 mm	
Cafe/Bar	15.36 *kN/m*	84x500 mm 7-ply x 12 mm	

6.4.2 Existing Beam Strengthening

The degrees to which the primary beams should be strengthened are summarized in Table 6.5 and the full calculations are in Appendix C under their associated use scenario. It falls out of the scope of this thesis to solve the strengthening of the existing beams. It may be conducive to the conservation argument to wrap them with FRP so that the cross-section remains roughly the same. For more demanding uses such as at the bar and skate areas, where the utilizations are 140-160%, it is doubtful that FRP would be sufficient and the beams may need to be reinforced with new steel elements or perhaps with new (glass?) columns.

Table 6.5: Percentage over capacity at existing concrete beams, with new loads

	New M_d	% over
Roof garden	2070 kNm	27%
Skatepark	2350 kNm	40%
Bar	2790 kNm	61%

6.4.3 Connection Design

The structural transfer of loads is reflected in the detailing. The main connections are at top of beam to bottom of slab and at end of beam to face of existing beam (Fig. 6.11).

Fig. 6.11: Final design: detail key

6.4.4 Connection Design

Due to the way it is fastened, the detailing does not allow the glass beam to act in conjunction with the concrete slab as a composite. The beam sits in a steel shoe but is not glued or bolted to it, so that there is a tolerance for movement (Fig. 6.12).

The steel brackets that are bolted to the slab, that flank both sides of beam, restrain it and prevent lateral torsional buckling (Fig. 6.13). Because the surface of the existing concrete slab cannot be assumed to be completely flat or uniform, the beam is levelled then installed with a gap above it - which is packed with mortar. The slab rests on the beam and the forces are transferred through the intermediate layer of mortar. Where the beam meets the brackets, it also has a pre-laminated steel track that is the interface between the glass and the other materials it contacts.

The length of the beam is measured in field before manufacture so that there is enough of an installation tolerance, but is precise enough that it is in minimally adjusted or altered on site (Fig. 6.14).

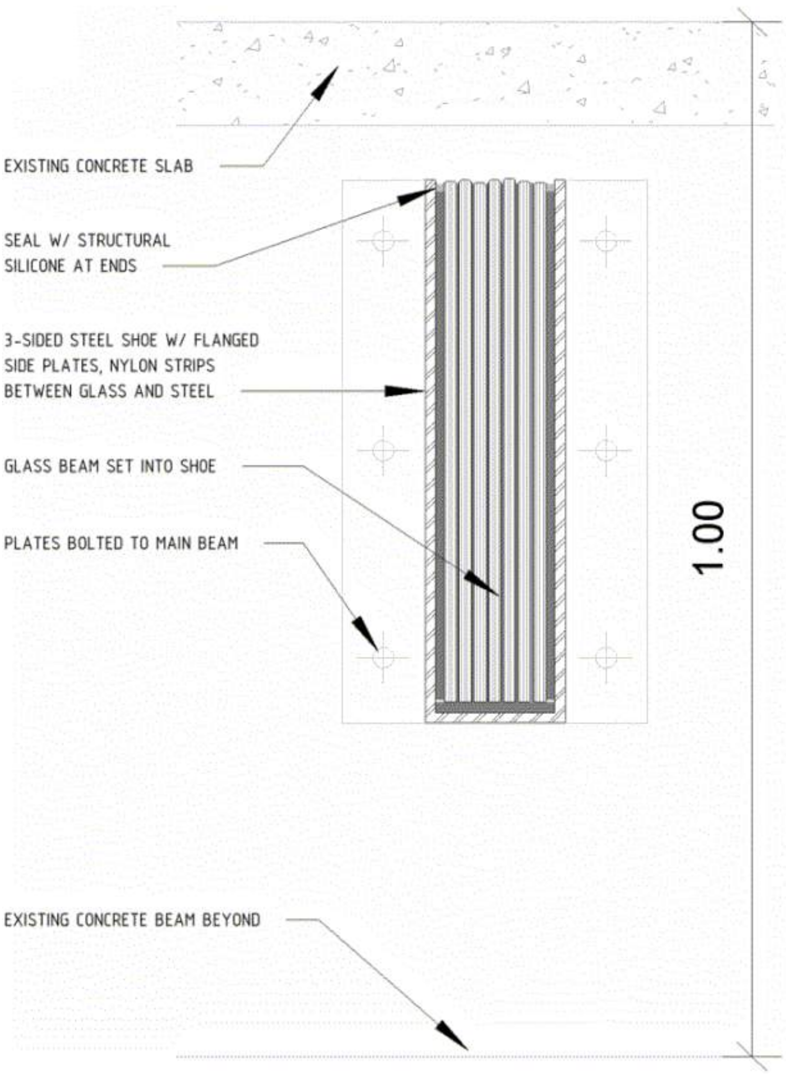


Fig. 6.12: Section detail at end of glass beam to existing concrete beam

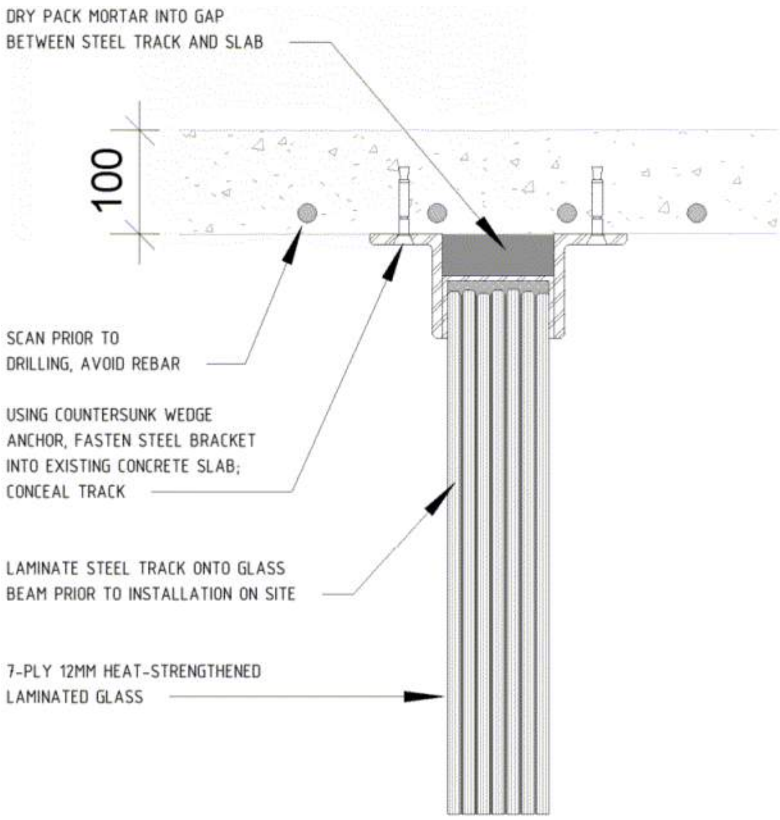


Fig. 6.13: Section detail at mid-span glass beam

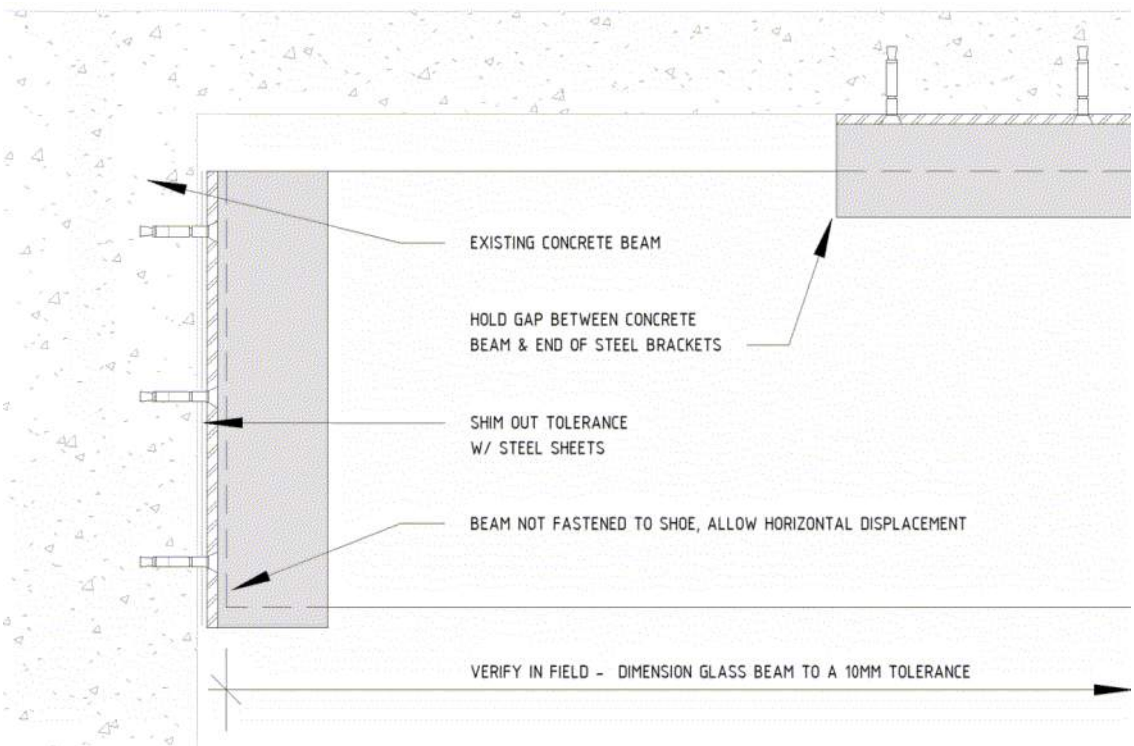


Fig. 6.14: Elevation detail at end of glass beam to existing concrete beam

6.5 Constructability and Lifespan

6.5.1 Assembly

The glass beam and connections are designed for a simple installation (Fig. 6.15). There are no holes or bolting of the glass that demand more specialized labor. The beam with the steel shoes is raised to the specified height, it is supported until the shoes are bolted to the concrete beams, a steel bracket is installed on one side, providing the fence to pack mortar from the other side, the other bracket is installed, then the seams are sealed with structural silicone.

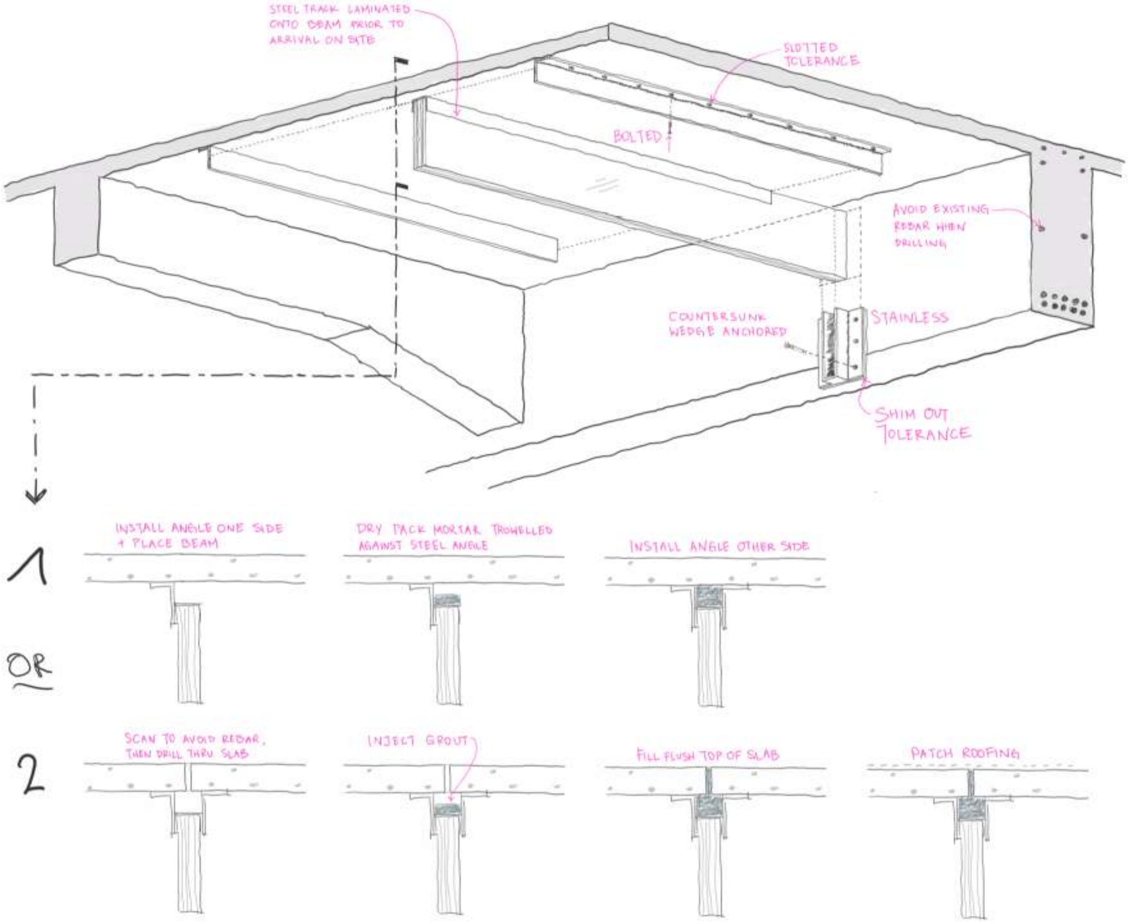


Fig. 6.15: Installation of glass beam and components

6.5.2 Maintenance

Since the glass beam intervention is in an interior, conditioned space, it is not subjected to harsh environments that make it vulnerable to deterioration. The high 5.75 m ceiling also limits access from occupants of the building. However, it is still part of a public space, with areas of high pedestrian traffic both above and below it. The glass should be inspected periodically for typical failure or deterioration mechanisms, such as: delamination, twisting or bowing, corrosion of steel components, busted up plies, and cracking. If the beam or its components should be repaired, replaced, or removed, the order of construction also facilitates disassembly. Other than the steel track laminated to the top edge, the beam itself is not glued or bolted to anything. The steel components can be easily unbolted and separated. At end-of-life, the laminated interlayers are the main hindrance to recycling of the glass beam.

7 Conclusions

This thesis studied the possibility of using glass to strengthen historic concrete, in the event of a change in use. Because the proposal was to alter an existing roof to become accessible to the public, the resulting new loads were substantial in scale as they corresponded to uses that congregated people or introduced heavy landscaping. To direct the research, a main question was formulated then divided into more specific sub-questions.

7.1 Research Question

To what extent can structural glass components be used to strengthen concrete heritage buildings, in lieu of the prevailing conventional methods?
Glass can certainly be used as a reinforcement medium in existing buildings, notably in specific applications that are architecturally driven. But the slender nature and depth of a typical float glass beam can turn the intervention in a direction that takes away from the aesthetic intent.

The challenge is also in the progressive strengthening of existing elements, whether other materials or perhaps introduction of different structural components make for a more appropriate solution, and when to make this call. This project studied glass beams, but reducing the grid through columns or walls would also serve to lessen the demands on the existing structure. It may be that strengthening is not economical, but that a separate structure should be inserted into the existing - though this results in a more invasive intervention that is less respectful of the historic material. The decision to use glass should be balanced by the degree of strengthening that is needed (which is related to the scale of program added to make the project financially viable) and the excess capacity of the existing elements.

7.2 Sub-questions

The main research question is further divided into:

How can early 20th century concrete structures be adaptively reused, where both the technical and historic context are mediated?
There is precedent in the Netherlands that the development of reinforced concrete, physically documented in the survival of early 20th century structures, is regarded as culturally valuable. This is evidenced through the designation of several structures as officially protected monuments. At the same time, Dutch planning policy has also trended toward the redevelopment of buildings from the recent past, which is a middle-ground between historic designation and demolition. Examples of 20th century concrete buildings that have undergone

deep renovations and significant technical interventions include the *Karel Doorman* and Fenix I, both in Rotterdam. However, these projects were either able to bear completely on the existing structure or included a new and separate structure.

How can the different material properties of glass and concrete be leveraged to create a structurally unified intervention?
The core material properties of concrete and glass are similar, indicating that there is potential that they can work together structurally. Both are excellent in compression but tend to fail without warning, particularly when unreinforced. It is the transparency and durability of glass that makes it unique and valuable, that also makes it intriguing for use in historic buildings. To truly test how far glass can be used to retrofit a concrete building, the future use scenarios should impose substantially higher loads onto the existing structure.

How can the constructability, structural safety, and return on investment of the glass intervention set it on a path toward feasible implementation?
The decision to engineer and pay for a glass intervention is tied to the state of the building as a whole. It may not be enough to strengthen the slab, but also the beams, columns, and foundations will be affected. The case study involves a glass beam construction that uses standard and typical manufacturing processes, that is detailed to be installed and disassembled without complication, and is sized so that transport should not be a logistical issue. Safety is generally addressed through the addition of redundant layers, but this results in the use of more material.

8 Recommendations

8.1 Limitations

The extent of the thesis has been limited to calculations that are preliminary and conceptual in scope. This likely gives results and element sizes that are conservative and can be reduced upon further analysis. The studied beam design is monolithic, with a total glass section that may not completely contribute to structural performance.

The scale of the complete retrofit of the structure, and thereby its feasibility, is not known. The columns and foundations were assumed to be adequate. It is certain, though, that strengthening the slab is not enough, but that at least the main beams will be affected.

At times, unreliable access to electricity, internet, or a table subsequently drove the graphic style of this thesis. I ask the reader to recognize the significance of receiving a digital edition of the text, rather than a mixed-media stack of typewritten chapters, scribbles on the backs of receipts, watercolors, phone screenshots, stories deeper than the human mind can comprehend hand-etched onto meteorites, and so on.

8.2 Recommendations for Further Research

8.2.1 Composite Interaction

Because there is little precedent of glass beams attached to concrete in both research and practice, further study into how to fasten the beam to the slab so that they are integrally connected is a logical extension of the topic. The calculations show that a 30-40% reduction in the section is possible when designed for full composite action. Perhaps a T-plate can be laminated into the beam, then sufficiently anchored into the slab. If it was not a retrofit situation, this can probably be done through the inclusion of plates laminated into the glass that are embedded into the concrete as it is cast, similarly to how it is done in steel-concrete structures.

8.2.2 Section Design

Several research labs have developed glass beam designs, with the aim of optimizing efficiency and manufacturing post-failure ductility. This warrants more study into the beam's section design and whether it can withstand the same loads more efficiently either through geometry or by reinforcement with other materials. If it is to be used in a historic building, the visual effect should also be considered for whether it detracts from the architecture.

8.2.3 Fire Resistance

The thesis goes so far as to ensure that the beam section does not break when two plies are taken away in an exceptional loading condition, such as a fire. However, because pre-stressed glass tends to crack when exposed to high heat, this topic can be further developed on ways to protect the section so that sacrificial layers are not needed.

8.2.4 Experimental Testing

More complete research should include systematic experimental testing at preliminary design stages on concrete substrates that replicate existing conditions. Ideally, testing should be conducted at real scale on actual building pieces. The tests should be repeated so that they have statistical relevance and can be studied for trends and compared to analytical and numerical models, ultimately validating the design by more than one method.

8.2.5 Other Applications

Beyond retrofit situations, the glass beam can also be applied to new construction concrete buildings that are after a certain aesthetic effect that cannot be achieved with traditional, opaque materials such as steel. There is also a novelty in using structural glass that is unexpected, that adds experiential and architectural value to the building.

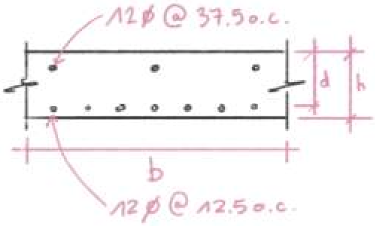
Appendices

A Existing Concrete Roof Calculations

A.1 Slab

Existing Concrete Strength C8

- Assess plausibility of lowest strength values for concrete (C8) using original design loads



<u>Geometry + Material Properties</u>

$f_{ck} := 8\ MPa$	lowest allowed concrete compressive strength
$\gamma_c := 1.5$	partial factor, concrete
$f_{cd} := \dfrac{f_{ck}}{\gamma_c} = 5.33\ MPa$	design strength, concrete
$b := 1000\ mm$	slab design strip width
$h := 100\ mm$	slab depth
$d := 84\ mm$	slab depth to tension rebar w/ 10mm cover (min. at slabs per *GBV 1912*)
$\rho_c := 24\ kN \cdot m^{-2}$	unit weight concrete
$L := 4.5\ m$	slab span

<u>Loads</u>

$G_k := \rho_c \cdot h = 2.4\ kN \cdot m^{-1}$	DL (self-weight only)
$Q_k := 1.4\ kN \cdot m^{-1}$	LL per existing calculations
$\gamma_G := 1.2$	partial factor, permanent loads
$\gamma_Q := 1.5$	partial factor, imposed loads
$Q_{ULS} := \gamma_G \cdot G_k + \gamma_Q \cdot Q_k = 4.98\ kN \cdot m^{-1}$	ULS design load

<u>Moment</u>

$M_d := \dfrac{Q_{ULS} \cdot L^2}{8} = 12.61\ kN \cdot m$	design moment
$K_{lim} := 0.167$	factor limit to ensure ductile failure
$M_u := K_{lim} \cdot f_{cd} \cdot b \cdot d^2 = 6.28\ kN \cdot m$	ultimate bending capacity w/o compression reinforcement
$UC := \dfrac{M_d}{M_u} = 2.01$	**utilization exceeds capacity 2x, slab strength is likely not C8**

Existing Concrete Strength C20

- Use concrete strength from Fenix I testing (C20/25) and reinforcement strength from *GBV 1912* (B250),
- Consider top reinforcement as nonstructural, since spacing exceeds maximum allowed per EC2

Geometry + Material Properties

$f_{ck} := 20\ MPa$ concrete strength from Fenix I

$\gamma_c := 1.5$ partial factor, concrete

$f_{cd} := \frac{f_{ck}}{\gamma_c} = 13.33\ MPa$ design strength, concrete

$f_{yk} := 250\ MPa$ lowest allowed steel tensile strength

$\gamma_s := 1.15$ partial factor, reinforcement

$f_{yd} := \frac{f_{yk}}{\gamma_s} = 217.39\ MPa$ design strength, reinforcement

$b := 1000\ mm$ slab design strip width

$h := 100\ mm$ slab depth

$d := 84\ mm$ eff. depth to tension rebar w/ 10mm cover (min. at slabs per *GBV 1912*)

$E_c := 30\ GPa$ Young's modulus, uncracked C20/25

$\rho_c := 24\ kN \cdot m^{-2}$ unit weight concrete

$L := 4.5\ m$ slab span

Loads

$Q_{ULS} := \gamma_G \cdot G_k + \gamma_Q \cdot Q_k = 4.98\ kN \cdot m^{-1}$ ULS design load

$Q_{SLS} := G_k + \gamma_Q \cdot Q_k = 4.5\ kN \cdot m^{-1}$ SLS design load

Moment

$M_d := \frac{Q_{ULS} \cdot L^2}{8} = 12.61\ kN \cdot m$ design moment

$K_{lim} := 0.167$ factor limit to ensure ductile failure

$M_u := K_{lim} \cdot f_{cd} \cdot b \cdot d^2 = 15.71\ kN \cdot m$ ultimate bending capacity w/o compression reinforcement

$UC := \frac{M_d}{M_u} = 0.8$ **OK**

Deflection

$I := \frac{b \cdot h^3}{12} = \left(8.33 \cdot 10^7\right)\ mm^4$ moment of inertia

$w_{max} := 0.004 \cdot L = 18\ mm$ max deflection allowed at floors

$w := \frac{5 \cdot Q_{SLS} \cdot L^4}{384 \cdot E_c \cdot I} = 9.61\ mm$ **OK**, <18mm

Reinforcement

$z := d \cdot \left(0.5 + \sqrt{0.25 - \frac{K_{lim}}{1.134}}\right) = 68.92\ mm$ lever arm

$A_s := \frac{M_d}{0.87 \cdot f_{yd} \cdot z} = 967.02\ mm^2$ area tension reinforcing steel

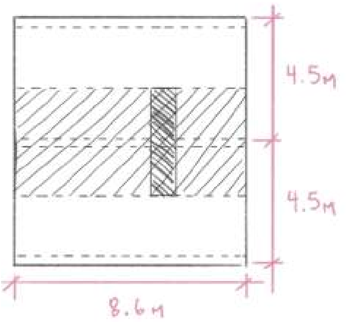
$A_{s;prov} := 904\ mm^2$ area reinforcement provided

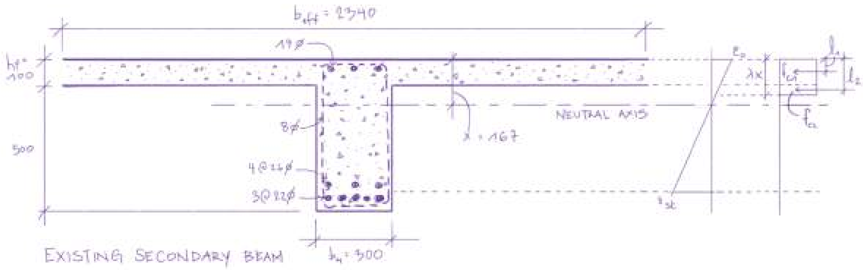
$UC := \frac{A_s}{A_{s;prov}} = 1.07$ **barely OK**

A.2 Secondary Beams

Existing Secondary Beam

- Using C20/25 for concrete and B250 for steel, check moment, reinforcement, and deflection



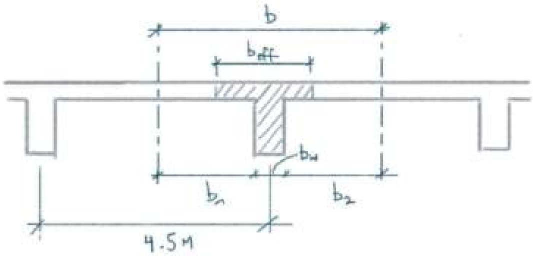


Loads

$\rho_c := 24\ kN \cdot m^{-3}$	unit weight concrete
$G_{beam2} := 0.3\ m \cdot 0.5\ m \cdot \rho_c = 3.6\ kN \cdot m^{-1}$	beam self-weight
$G_{slab} := 4.5\ m \cdot 0.1\ m \cdot \rho_c = 10.8\ kN \cdot m^{-1}$	adj. slab self-weight
$G_k := G_{beam2} + G_{slab} = 14.4\ kN \cdot m^{-1}$	characteristic dead load
$Q_k := 1.4\ kN \cdot m^{-2} \cdot 4.5\ m = 6.3\ kN \cdot m^{-1}$	characteristic live load
$Q_{ULS} := \gamma_G \cdot G_k + \gamma_Q \cdot Q_k = 26.73\ kN \cdot m^{-1}$	ULS design load
$Q_{SLS} := G_k + \gamma_Q \cdot Q_k = 23.85\ kN \cdot m^{-1}$	SLS design load

Geometry + Material Properties

$f_{ck} := 20\ MPa$	concrete strength from Fenix I
$\gamma_c := 1.5$	partial factor, concrete
$f_{cd} := \dfrac{f_{ck}}{\gamma_c} = 13.33\ MPa$	design strength, concrete
$f_{yk} := 250\ MPa$	lowest allowed steel tensile strength
$\gamma_s := 1.15$	partial factor, reinforcement
$f_{yd} := \dfrac{f_{yk}}{\gamma_s} = 217.39\ MPa$	design strength, reinforcement
$b := 300\ mm$	beam width
$h := 600\ mm$	beam depth
$d := 566\ mm$	eff. depth to tension rebar w/ 15mm cover (min. at beams per *GBV 1912*)
$d' := 44.5\ mm$	eff. depth to compression rebar, this is mostly made up
$L := 8.6\ m$	beam span



Effective width - exisiting secondary beam

$b_w := 300\ mm$ — width concrete beam

$b_1 := \frac{(4.5\ m - b_w)}{2} = 2.1\ m$ — slab tributary width

$L_0 := 0.7 \cdot 8.6\ m = 6.02\ m$ — contraflexure distance, interior span

$b_{eff;1} := 0.2 \cdot b_1 + 0.1 \cdot L_0 = 1.02\ m$ — effective width, each side

Check: — EN 1991-1-1 (5.3.2.1)

$b_{eff,i} \le b_i$

$b_{eff;i} \le 0.2 \cdot L \ge 0$

$1.02 < 1.2$ — **taker lesser value 1.02m**

$b_{eff;2} := b_{eff;1}$

$b_{eff} := b_w + b_{eff;1} + b_{eff;2} = 2.34\ m$ — effective width, concrete T-flange
EN 1992-1-1 (Eq. 5.7)

Moment - T-beam

$M_d := \frac{Q_{ULS} \cdot L^2}{8} = 247.12\ kN{\cdot}m$ design moment

$A_{sprov} := 3954\ mm^2$ area of provided reinforcement at beam

$d := 539\ mm$ distance from top to center of tension reinforcement

$\lambda := 0.8$ factor for eff ht of compression zone if fck<50 MPa

$x := 167.185\ mm$ neutral axis from top

$\lambda x := \lambda \cdot x = 133.75\ mm$ depth rectangular compression stress block from top

$h_f := 100\ mm$ height concrete flange

$l_1 := \frac{h_f}{2} = 50\ mm$ centroid of compression w/in flange

$l_2 := \frac{(\lambda x - h_f)}{2} + h_f = 116.87\ mm$ centroid of compression w/in web, from top

$f_{c1} := f_{cd} \cdot b_{eff} \cdot h_f = (3.13 \cdot 10^3)\ kN$ compressive force w/in flange

$f_{c2} := f_{cd} \cdot b_w \cdot (\lambda x - h_f) = 134.99\ kN$ compressive force w/in web

$F_c := f_{c1} + f_{c2} = (3.26 \cdot 10^3)\ kN$ total compressive force

$F_s := f_{yd} \cdot A_{sprov} = 859.57\ kN$ tension force provided in steel

$M_u := F_s \cdot d - f_{c1} \cdot l_1 - f_{c2} \cdot l_2 = 291.26\ kN \cdot m$ moment capacity

$UC := \frac{M_d}{M_u} = 0.85$

Compression Reinforcement

$M_d := \frac{Q_{ULS} \cdot L^2}{8} = 247.12\ kN \cdot m$ design moment

$K_{lim} := 0.167$ factor limit to ensure ductile failure

$M_u := K_{lim} \cdot f_{cd} \cdot b \cdot d^2 = 194.07\ kN \cdot m$ ultimate bending capacity w/o compression reinforcement

$UC := \frac{M_d}{M_u} = 1.27$ **need compression reinforcement**

$A'_s := \frac{M_d - M_u}{0.87 \cdot f_{yd} \cdot (d - d')} = 567.24\ mm^2$ area compression reinforcement required

$A'_{s;prov} := 850\ mm^2$ area compression reinforcement provided

$UC := \frac{A'_s}{A'_{s;prov}} = 0.67$ **OK**

Tension Reinforcement

$z := d \cdot \left(0.5 + \sqrt{0.25 - \frac{K_{lim}}{1.134}}\right) = 442.26\ mm$ lever arm

$A_s := \frac{M_u}{0.87 \cdot f_{yd} \cdot z} + A'_s = \left(2.89 \cdot 10^3\right)\ mm^2$ area tension reinforcement required

$A_{s;prov} := 3954\ mm^2$ area tension reinforcement provided

$UC := \frac{A_s}{A_{s;prov}} = 0.73$ **OK**

Deflection

$I := \frac{b \cdot h^3}{12} = \left(5.4 \cdot 10^9\right)\ mm^4$ moment of inertia

$w_{max} := 0.004 \cdot L = 34.4\ mm$ max deflection allowed at floors

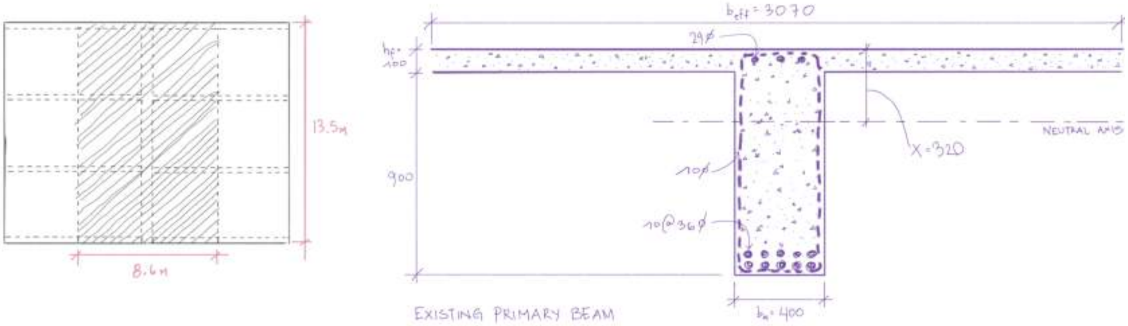
$w := \frac{5 \cdot Q_{SLS} \cdot L^4}{384 \cdot E_c \cdot I} = 10.49\ mm$ **OK**, <34.4mm

$UC := \frac{w}{w_{max}} = 0.3$

A.3 Primary Beams

Existing Primary Beam

- Using C20/25 for concrete and B250 for steel, check moment, reinforcement, and deflection



<u>Loads</u>

$G_{beam1} := 0.4\ m \cdot 0.9\ m \cdot \rho_c = 8.64\ kN \cdot m^{-1}$	primary beam self-weight
$G_k := G_{beam2} + G_{slab} + G_{beam1} = 23.04\ kN \cdot m^{-1}$	add to s-w from secondary beam calcs
$Q_{ULS} := \gamma_G \cdot G_k + \gamma_Q \cdot Q_k = 37.1\ kN \cdot m^{-1}$	ULS design load
$Q_{SLS} := G_k + \gamma_Q \cdot Q_k = 32.49\ kN \cdot m^{-1}$	SLS design load

<u>Geometry</u>

$b := 400\ mm$	beam width
$h := 1000\ mm$	beam depth
$d := 921\ mm$	eff. depth to mid of tension rebar w/ 15mm cover (min. at beams per *GBV 1912*)
$d' := 44.5\ mm$	eff. depth to compression rebar, this is mostly made up
$L := 13.5\ m$	beam span

<u>Effective width - exisiting secondary beam</u>

$b_w := 400\ mm$	width concrete beam
$b_1 := \dfrac{(4.3\ m - b_w)}{2} = 1.95\ m$	slab tributary width
$L_0 := 0.7 \cdot 13.5\ m = 9.45\ m$	contraflexure distance, interior span
$b_{eff;1} := 0.2 \cdot b_1 + 0.1 \cdot L_0 = 1.34\ m$	effective width, each side

Check: EN 1991-1-1 (5.3.2.1)

$b_{eff;i} \le b_i$

$b_{eff;i} \le 0.2 \cdot L \ge 0$

$1.34 < 1.89$ **taker lesser value 1.34m**

$b_{eff;2} := b_{eff;1}$

$b_{eff} := b_w + b_{eff;1} + b_{eff;2} = 3.07\ m$ effective width, concrete T-flange EN 1992-1-1 (Eq. 5.7)

Moment - T-beam

$M_d := \frac{Q_{ULS} \cdot L^2}{8} = 845.14\ kN \cdot m$	design moment
$A_{sprov} := 12160\ mm^2$	area of provided reinforcement at beam
$d := 921\ mm$	distance from top to center of tension reinforcement
$\lambda := 0.8$	factor for eff ht of compression zone if fck<50 MPa
$x := 320\ mm$	neutral axis from top
$\lambda x := \lambda \cdot x = 256\ mm$	depth rectangular compression stress block from top
$h_f := 100\ mm$	height concrete flange
$l_1 := \frac{h_f}{2} = 50\ mm$	centroid of compression w/in flange
$l_2 := \frac{(\lambda x - h_f)}{2} + h_f = 178\ mm$	centroid of compression w/in web, from top
$f_{c1} := f_{cd} \cdot b_{eff} \cdot h_f = \left(4.09 \cdot 10^3\right)\ kN$	compressive force w/in flange
$f_{c2} := f_{cd} \cdot b_w \cdot (\lambda x - h_f) = 832\ kN$	compressive force w/in web
$F_c := f_{c1} + f_{c2} = \left(4.93 \cdot 10^3\right)\ kN$	total compressive force
$F_s := f_{yd} \cdot A_{sprov} = \left(2.64 \cdot 10^3\right)\ kN$	tension force provided in steel
$M_u := F_s \cdot d - f_{c1} \cdot l_1 - f_{c2} \cdot l_2 = \left(2.08 \cdot 10^3\right)\ kN \cdot m$	moment capacity
$UC := \frac{M_d}{M_u} = 0.41$	**OK**

Compression Reinforcement

$M_d := \frac{Q_{ULS} \cdot L^2}{8} = 845.14\ kN \cdot m$ design moment

$K_{lim} := 0.167$ factor limit to ensure ductile failure

$M_u := K_{lim} \cdot f_{cd} \cdot b \cdot d^2 = 755.5\ kN \cdot m$ ultimate bending capacity w/o compression reinforcement

$UC := \frac{M_d}{M_u} = 1.12$ **need compression reinforcement**

$A'_s := \frac{M_d - M_u}{0.87 \cdot f_{yd} \cdot (d - d')} = 540.73\ mm^2$ area compression reinforcement required

$A'_{s;prov} := 1981\ mm^2$ area compression reinforcement provided

$UC := \frac{A'_s}{A'_{s;prov}} = 0.27$ **OK**

Tension Reinforcement

$z := d \cdot \left(0.5 + \sqrt{0.25 - \frac{K_{lim}}{1.134}}\right) = 755.7\ mm$ lever arm

$A_s := \frac{M_u}{0.87 \cdot f_{yd} \cdot z} + A'_s = (5.83 \cdot 10^3)\ mm^2$ area tension reinforcement required

$A_{s;prov} := 12160\ mm^2$ area tension reinforcement provided

$UC := \frac{A_s}{A_{s;prov}} = 0.48$ **OK**

Deflection

$I := \frac{b \cdot h^3}{12} = (3.33 \cdot 10^{10})\ mm^4$ moment of inertia

$w_{max} := 0.004 \cdot L = 54\ mm$ max deflection allowed at floors

$w := \frac{5 \cdot Q_{SLS} \cdot L^4}{384 \cdot E_c \cdot I} = 14.05\ mm$ **OK**

$UC := \frac{w}{w_{max}} = 0.26$

B Initial Design Calculations

B.1 Roof garden

- Glass elements to accept all new loads, existing self-weight goes to existing concrete elements.

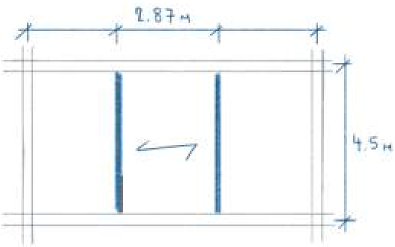
Material properties - glass

$f_{g;k} := 70\ MPa$	characteristic strength, HSG
$\gamma_{M;v} := 1.5$	material factor, pre-stressed glass
$f_{g;d} := \frac{f_{g;k}}{\gamma_{M;v}} = 46.67\ MPa$	design strength, HSG
$E_g := 70\ GPa$	elastic modulus, glass

Load tracking - garden

$G_{gdn} := 5\ kN \cdot m^{-2}$	garden dead load, with 0,5 for roofing and 4,5 for 0,3m soil profile
$G_{slab} := 2.4\ kN \cdot m^{-2}$	slab self weight
$kNm_{gdn} := 5\ kN \cdot m^{-1}$	linear DL
$kNm_{slab} := 2.4\ kN \cdot m^{-1}$	linear SW
$\gamma_G := 1.2$	partial factor, DL
$Q_k := 1.3\ kN \cdot m^{-1}$	live load, with 1,0 for service path and 0,3 for pigs
$Q_s := 0.56\ kN \cdot m^{-1}$	snow load, sloped roof 1,92°
$\Psi_0 := 0$	reduction factor for combination value of a snow load
$\gamma_Q := 1.5$	partial factor, LL
$\rho_g := 25\ kN \cdot m^{-3}$	unit weight glass

Add two beams in short direction



Check slab

$b := 1000\ mm$ slab design strip width

$h := 100\ mm$ slab depth

$d := 84\ mm$ effective depth

$L := 2.87\ m$ slab span

$G_k := kNm_{gdn} + kNm_{slab} = 7.4\ kN \cdot m^{-1}$ total dead load, garden

$Q_{ULS} := \gamma_G \cdot G_k + \gamma_Q \cdot Q_k + (\gamma_Q \cdot \Psi_0 \cdot Q_s) = 10.83\ kN \cdot m^{-1}$ ULS design load

$Q_{SLS} := G_k + \gamma_Q \cdot Q_k + (\Psi_0 \cdot Q_s) = 9.35\ kN \cdot m^{-1}$ SLS design load

Moment capacity - slab

$M_d := \frac{Q_{ULS} \cdot L^2}{8} = 11.15\ kN{\cdot}m$ design moment

$K_{lim} := 0.167$

$f_{cd} := 13.3\ MPa$

$f_{yd} := 217.39\ MPa$

$M_u := K_{lim} \cdot f_{cd} \cdot b \cdot d^2 = 15.67\ kN \cdot m$ bending capacity

$UC := \frac{M_d}{M_u} = 0.71$ **OK**

Reinforcement - slab

$z := d \cdot \left(0.5 + \sqrt{0.25 - \frac{K_{lim}}{1.134}}\right) = 68.92\ mm$ lever arm

$A_s := \frac{M_d}{0.87 \cdot f_{yd} \cdot z} = 855.41\ mm^2$ area reinforcement required

$A_{s;prov} := 904\ mm^2$ area reinforcement provided

$UC := \frac{A_s}{A_{s;prov}} = 0.95$ **OK**

Deflection - slab

$E_c := 30\ GPa$

$I := \frac{b \cdot h^3}{12} = (8.33 \cdot 10^7)\ mm^4$ moment of inertia

$w_{max} := 0.004 \cdot L = 11.48\ mm$ max deflection allowed at floors

$w := \frac{5 \cdot Q_{SLS} \cdot L^4}{384 \cdot E_c \cdot I} = 3.3\ mm$ **OK**

Geometry - glass beam

$b := 84\ mm$	beam width, w/ 7x15mm
$h := 320\ mm$	beam depth
$L_b := 4.5\ m$	beam span
$L_s := 2.87\ m$	adjacent slab span

Loads - glass beam

$kNm_{bm} := b \cdot h \cdot \rho_g = 0.67\ kN \cdot m^{-1}$	beam self-weight
$G_k := (kNm_{gdn} + kNm_{bm}) \cdot 2.87 = 16.28\ kN \cdot m^{-1}$	characteristic permanent load
$Q_{bm} := 1.3\ kN \cdot m^{-2}$	variable load at beam
$Q_k := Q_{bm} \cdot 2.87\ m = 3.73\ kN \cdot m^{-1}$	characteristic variable load
$Q_{ULS} := \gamma_G \cdot G_k + \gamma_Q \cdot Q_k = 25.13\ kN \cdot m^{-1}$	ULS design load
$Q_{SLS} := G_k + \gamma_Q \cdot Q_k = 21.88\ kN \cdot m^{-1}$	SLS design load

Moment - glass beam

$$M_d := \frac{Q_{ULS} \cdot L_b^{\ 2}}{8} = 63.61\ kN{\cdot}m$$ design moment

$$W := \frac{b \cdot h^2}{6} = (1.43 \cdot 10^6)\ mm^3$$ section modulus

$$\sigma_m := \frac{M_d}{W} = 44.37\ MPa$$ max bending stress at mid-span

$$UC := \frac{\sigma_m}{f_{g;d}} = 0.95$$ **OK**

Deflection - glass beam

$$I := \frac{b \cdot h^3}{12} = (2.29 \cdot 10^8)\ mm^4$$ moment of inertia

$$w_{max} := 0.004 \cdot L_b = 18\ mm$$ max deflection allowed at floors

$$w := \frac{5 \cdot Q_{SLS} \cdot L_b^{\ 4}}{384 \cdot E_g \cdot I} = 7.27\ mm$$ **OK**

Shear - glass beam

$$V_{max} := \frac{(Q_{ULS} \cdot L_b)}{2} = 56.54\ kN$$

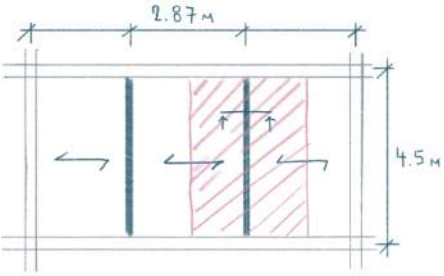
$$y := \frac{(h \div 2)}{2} = 0.08\ m$$

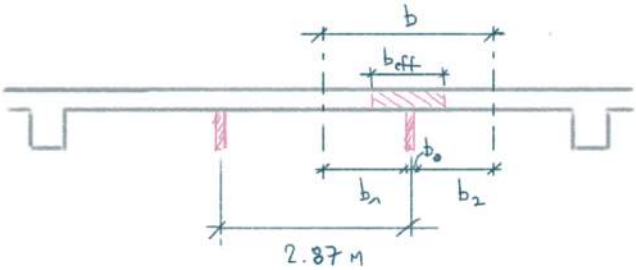
$$Q_{max} := y \cdot b \cdot \left(\frac{h}{2}\right) = (1.08 \cdot 10^{-3})\ m^3$$

$$\tau := \frac{(V_{max} \cdot Q_{max})}{I \cdot b} = 3.16\ MPa$$

B.2 Transformed glass section

Glass to strengthen slab at green roof





<u>Effective width of existing concrete flange</u>

$b_0 := 60\ mm$	width glass beam... width b/t axis of shear connectors
$b_1 := \dfrac{(2.87\ m - b_0)}{2} = 1.41\ m$	adjacent slab tributary width
$L_e := 0.7 \cdot 4.5\ m = 3.15\ m$	contraflexure distance, interior span
$b_{e1} := \dfrac{L_e}{8} = 0.39\ m$	effective width, each side
$b_{e2} := b_{e1}$	

Check:

$b_{ei} \le b_i$	EN 1994-1-1 (5.4.1.2)
$0.75 < 1.38$	**take lesser value 0.75m**
$b_{eff} := b_0 + b_{e1} + b_{e2} = 847.5\ mm$	effective width, concrete T flange EN 1994-1-1 (Eq. 5.3)

Moment capacity: slab + beam

- Transformed section, with Ec=30 GPa (uncracked C20) and Eg=70 GPa (glass)

Geometry - glass beam

$b := 60\ mm$	beam width, w/ 5x12mm
$h := 270\ mm$	beam depth
$L_b := 4.5\ m$	beam span
$L_s := 2.87\ m$	tributary width slab

Loads - glass beam

$kNm_{bm} := b \cdot h \cdot \rho_g = 0.41\ kN \cdot m^{-1}$	beam self-weight
$G_k := (kNm_{gdn} + kNm_{bm}) \cdot 2.87 = 15.51\ kN \cdot m^{-1}$	characteristic permanent load
$q_{bm} := 1.3\ kN \cdot m^{-2}$	variable load at beam
$q_k := q_{bm} \cdot 2.87\ m = 3.73\ kN \cdot m^{-1}$	characteristic variable load
$Q_{ULS} := \gamma_G \cdot G_k + \gamma_Q \cdot q_k + (\gamma_Q \cdot \Psi_0 \cdot q_s) = 24.21\ kN \cdot m^{-1}$	ULS design load
$Q_{SLS} := G_k + \gamma_Q \cdot q_k + (\Psi_0 \cdot q_s) = 21.11\ kN \cdot m^{-1}$	SLS design load

Moment - composite beam

$$M_d := \frac{Q_{ULS} \cdot L_b^{\,2}}{8} = 61.28\ kN{\cdot}m$$ design moment

$I := 5.2 \cdot 10^8\ mm^4$

$y_c := 91\ mm$ distance from top to neutral axis

$y_g := 183\ mm$ distance from bottom to neutral axis

$$\sigma_c := \frac{M_d \cdot y_c}{I} = 10.72\ MPa$$ max bending stress at concrete

$$UC := \frac{\sigma_c}{f_{cd}} = 0.8$$ **OK**

$$\sigma_g := \frac{M_d \cdot y_g}{I} = 21.57\ MPa$$ max bending stress at glass

$$UC := \frac{\sigma_g}{f_{g;d}} = 0.46$$ **OK**

Deflection - glass beam

$$I := \frac{b \cdot h^3}{12} = (9.84 \cdot 10^7)\ mm^4$$ moment of inertia

$w_{max} := 0.004 \cdot L_b = 18\ mm$ max deflection allowed at floors

$$w := \frac{5 \cdot Q_{SLS} \cdot L_b^{\,4}}{384 \cdot E_g \cdot I} = 16.36\ mm$$

$$UC := \frac{w}{w_{max}} = 0.91$$ **OK**

B.3 Existing beam strengthening

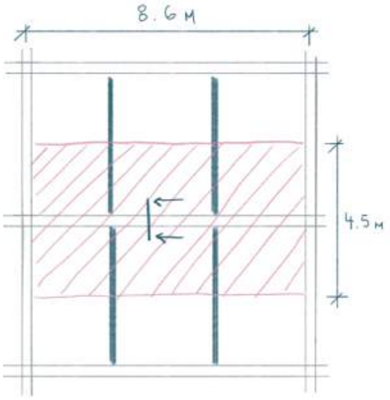
Glass to strengthen existing secondary beam

- use effective width of existing concrete flange from previous calcs

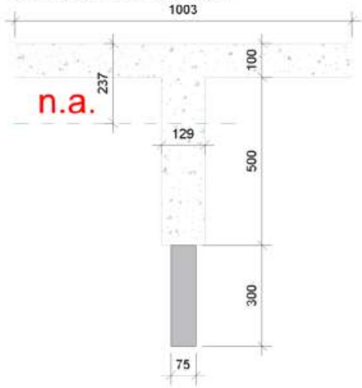
$b_{eff} := 2.34\ m$

$f_{g;d} := 46.67\ MPa$

<u>Add a new glass beam under it</u>



transformed section-



<u>Geometry - glass beam below secondary concrete beam</u>

$b_1 := 75\ mm$	beam width, 5 ply x 15mm
$h_1 := 300\ mm$	beam depth
$L_{bm1} := 8.6\ m$	beam span
$L_s := \frac{4.5\ m}{2} \cdot 2 = 4.5\ m$	adjacent slab tributary width

<u>Geometry - glass beam below slab</u>

$b_2 := 84\ mm$	beam width, 5 ply x 15mm
$h_2 := 400\ mm$	beam depth
$L_{bm2} := 4.5\ m$	beam span

<u>Loads</u>

$\rho_g := 25\ kN \cdot m^{-3}$	unit weight glass
$G_{gdn} := 5\ kN \cdot m^{-2}$	dead load at garden
$kNm_{bm1} := b_1 \cdot h_1 \cdot \rho_g = 0.56\ kN \cdot m^{-1}$	self-weight gl beam below existing beam
$kNm_{bm2} := b_2 \cdot h_2 \cdot \rho_g = 0.84\ kN \cdot m^{-1}$	self-weight gl beam below slab
$G_k := (G_{gdn} \cdot 4.5\ m) + kNm_{bm1} + kNm_{bm2} = 23.9\ kN \cdot m^{-1}$	characteristic dead load
$Q_k := 1.3\ kN \cdot m^{-2} \cdot 4.5\ m = 5.85\ kN \cdot m^{-1}$	characteristic live load
$Q := \gamma_G \cdot G_k + \gamma_Q \cdot Q_k = 37.46\ kN \cdot m^{-1}$	ULS design load

Moment - composite beam

$$M_{new} := \frac{\left(Q \cdot L_{bm1}^{\ 2}\right)}{8} = 346.3\ kN \cdot m$$ new design moment at glass beam

$M_d := 247.12\ kN \cdot m$ original design moment existing secondary conc beam

$M_u := 291.26\ kN \cdot m$ moment capacity existing secondary conc beam

$$UC := \frac{\left(M_{new} + M_d\right)}{M_u} = 2.04$$ **over capacity 100%**

$I := 1.2 \cdot 10^{10}\ mm^4$ moment of inertia transformed section

$y_c := 237\ mm$ distance from top to neutral axis

$y_g := 663\ mm$ distance from bottom to neutral axis

$$\sigma_c := \frac{\left(M_{new} + M_d\right) \cdot y_c}{I} = 11.72\ MPa$$ max bending stress at top of concrete

$$UC := \frac{\sigma_c}{f_{cd}} = 0.88$$ **OK**

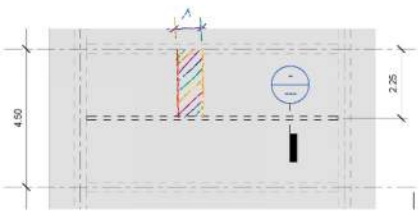
$$\sigma_g := \frac{M_d \cdot y_g}{I} = 13.65\ MPa$$ max bending stress at glass

$$UC := \frac{\sigma_g}{f_{g;d}} = 0.29$$ **OK**

C Final Design Calculations

C.1 Roof garden

Add one beam to split trib width in half



Check slab

$b := 1000\ mm$	slab design strip width
$h := 100\ mm$	slab depth
$d := 84\ mm$	effective depth to center of bottom bars
$L := 2.25\ m$	slab span

Loads

$G_{gdn} := 5\ kN \cdot m^{-2}$	garden permanent loads
$G_{slab} := 2.4\ kN \cdot m^{-2}$	slab self weight
$kNm_{gdn} := 5\ kN \cdot m^{-1}$	linear permanent load
$kNm_{slab} := 2.4\ kN \cdot m^{-1}$	linear s-w
$G_k := kNm_{gdn} + kNm_{slab} = 7.4\ kN \cdot m^{-1}$	total permanent load
$q_k := 1.3\ kN \cdot m^{-1}$	total variable load
$Q_{ULS} := \gamma_G \cdot G_k + \gamma_Q \cdot q_k = 10.83\ kN \cdot m^{-1}$	ULS design load
$Q_{SLS} := G_k + \gamma_Q \cdot q_k = 9.35\ kN \cdot m^{-1}$	SLS design load

Moment capacity - slab

$M_d := \dfrac{Q_{ULS} \cdot L^2}{8} = 6.85\ kN \cdot m$ design moment

$K_{lim} := 0.167$

$M_u := K_{lim} \cdot f_{cd} \cdot b \cdot d^2 = 15.71\ kN \cdot m$ ultimate bending capacity

$UC := \dfrac{M_d}{M_u} = 0.44$ **OK**

Reinforcement - slab

$z := d \cdot \left(0.5 + \sqrt{0.25 - \dfrac{K_{lim}}{1.134}}\right) = 68.92\ mm$ lever arm

$A_s := \dfrac{M_d}{0.87 \cdot f_{yd} \cdot z} = 525.74\ mm^2$ area reinforcement required

$A_{s;prov} := 904\ mm^2$ area reinforcement provided

$UC := \dfrac{A_s}{A_{s;prov}} = 0.58$ **OK**

Deflection - slab

$I := \dfrac{b \cdot h^3}{12} = \left(8.33 \cdot 10^7\right)\ mm^4$ moment of inertia

$w_{allow} := 0.004 \cdot L = 9\ mm$ max deflection allowed at floors

$w_{max} := \dfrac{5 \cdot Q_{SLS} \cdot L^4}{384 \cdot E_c \cdot I} = 1.25\ mm$ **OK**

Geometry - glass beam **calculated separate from concrete**

$b := 105\ mm$	beam width, w/ 7x15 mm
$h := 500\ mm$	beam depth
$L_b := 8.6\ m$	beam span
$L_s := 2.25\ m$	tributary width slab

Loads - glass beam

$kNm_{bm} := b \cdot h \cdot \rho_g = 1.31\ kN \cdot m^{-1}$	beam self-weight
$G_k := \left(G_{gdn} \cdot 2.25\ m\right) + kNm_{bm} = 12.56\ kN \cdot m^{-1}$	characteristic permanent load
$Q_{bm} := 1.3\ kN \cdot m^{-2}$	variable load at beam
$Q_k := Q_{bm} \cdot 2.25\ m = 2.93\ kN \cdot m^{-1}$	characteristic variable load
$Q_{ULS} := \gamma_G \cdot G_k + \gamma_Q \cdot Q_k + \left(\gamma_Q \cdot \Psi_0 \cdot q_s\right) = 19.46\ kN \cdot m^{-1}$	ULS design load
$Q_{SLS} := G_k + \gamma_Q \cdot Q_k + \left(\Psi_0 \cdot q_s\right) = 16.95\ kN \cdot m^{-1}$	SLS design load

Moment - glass beam

$M_d := \dfrac{Q_{ULS} \cdot {L_b}^2}{8} = 179.93\ kN \cdot m$	design moment
$W := \dfrac{b \cdot h^2}{6} = \left(4.38 \cdot 10^6\right)\ mm^3$	section modulus
$\sigma_m := \dfrac{M_d}{W} = 41.13\ MPa$	max bending stress at mid-span
$UC := \dfrac{\sigma_m}{f_{g;d}} = 0.88$	**OK**

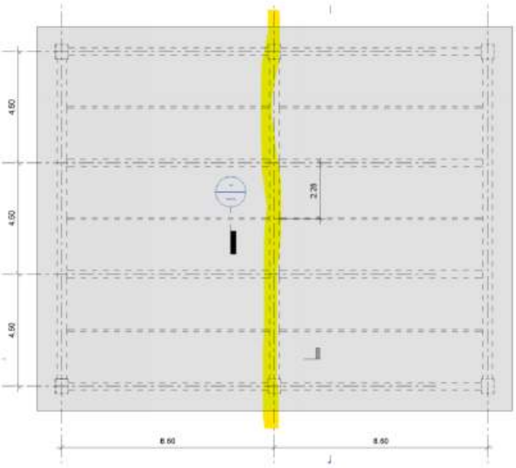
Deflection - glass beam

$I := \dfrac{b \cdot h^3}{12} = \left(1.09 \cdot 10^9\right)\ mm^4$	moment of inertia
$\delta_{max} := 0.004 \cdot L_b = 34.4\ mm$	max deflection allowed at floors
$\delta := \dfrac{5 \cdot Q_{SLS} \cdot {L_b}^4}{384 \cdot E_g \cdot I} = 15.77\ mm$	**OK**

Fire - glass beam

$Q_{FLS} := G_k = 12.56\ kN \cdot m^{-1}$	reduction factor $\psi_2 := 0$ at Category H: roofs
$b := 75\ mm$	delete 2-plies, w/ 5x17mm
$h := 500\ mm$	beam depth
$M_d := \dfrac{Q_{FLS} \cdot {L_b}^2}{8} = 116.14\ kN \cdot m$	design moment
$W := \dfrac{b \cdot h^2}{6} = \left(3.13 \cdot 10^6\right)\ mm^3$	section modulus
$\sigma_m := \dfrac{M_d}{W} = 37.16\ MPa$	max bending stress at mid-span
$UC := \dfrac{\sigma_m}{f_{g;d}} = 0.8$	**OK**

Degree of strengthening needed at primary beam



Geometry - new glass beams below slab

$b := 105\ mm$ beam width, 7 ply x 12mm

$h := 500\ mm$ beam depth

$L_b := 8.6\ m$ beam span

Loads

$\rho_g := 25\ kN \cdot m^{-3}$ unit weight glass

$L_s := \frac{8.6\ m}{2} \cdot 2 = 8.6\ m$ adjacent slab tributary width

$kNm_{gdn} := 5\ kN \cdot m^{-1}$ linear permanent load at garden

$kNm_{bm} := b \cdot h \cdot \rho_g = 1.31\ kN \cdot m^{-1}$ self-weight gl beam below slab

$kNm_{slab} := L_s \cdot 100\ mm \cdot \rho_c = 20.64\ kN \cdot m^{-1}$ self-weight slab

$G_k := (G_{gdn} \cdot 8.6\ m) + kNm_{bm} + kNm_{slab} = 64.95\ kN \cdot m^{-1}$ characteristic permanent load

$Q_{bm} := 2.93\ kN \cdot m^{-2}$ variable load at glass beam

$Q_k := Q_{bm} \cdot 8.6\ m = 25.2\ kN \cdot m^{-1}$ characteristic variable load

$Q := \gamma_G \cdot G_k + \gamma_Q \cdot Q_k = 115.74\ kN \cdot m^{-1}$ ULS design load

Moment - primary beam strengthening

$L := 13.5\ m$ span existing primary beam

$M_{new} := \frac{(Q \cdot L^2)}{8} = (2.64 \cdot 10^3)\ kN \cdot m$ new design moment

$M_d := 845.14\ kN{\cdot}m$ original design moment existing conc beam

$M_u := (2.08 \cdot 10^3)\ kN \cdot m$ moment capacity existing conc beam

$UC := \frac{M_{new}}{M_u} = 1.27$ **over capacity 27%**

C.2 Skatepark

Add two beams

Loads

$G_{sk} := 0.5\ kN \cdot m^{-2}$ skate permanent load

$G_{slab} := 2.4\ kN \cdot m^{-2}$ slab self weight

$kNm_{sk} := 0.5\ kN \cdot m^{-1}$ linear permanent load

$kNm_{slab} := 2.4\ kN \cdot m^{-1}$ linear s-w

$q_k := 5\ kN \cdot m^{-1}$ variable load

$q_s := 0.56\ kN \cdot m^{-1}$ snow load

$\Psi_0 := 0$ reduction factor

Check slab

$b := 1000\ mm$ slab design strip width

$h := 100\ mm$ slab depth

$d := 84\ mm$ effective depth to center of bottom bars

$L := 1.5\ m$ slab span

$G_k := kNm_{sk} + kNm_{slab} = 2.9\ kN \cdot m^{-1}$ total permanent load

$Q_{ULS} := \gamma_G \cdot G_k + \gamma_Q \cdot q_k + \left(\gamma_Q \cdot \Psi_0 \cdot q_s\right) = 10.98\ kN \cdot m^{-1}$ ULS design load

$Q_{SLS} := G_k + \gamma_Q \cdot q_k + \left(\Psi_0 \cdot q_s\right) = 10.4\ kN \cdot m^{-1}$ SLS design load

Moment capacity - slab

$$M_d := \frac{Q_{ULS} \cdot L^2}{8} = 3.09\ kN \cdot m$$ design moment

$K_{lim} := 0.167$

$M_u := K_{lim} \cdot f_{cd} \cdot b \cdot d^2 = 15.71\ kN \cdot m$ ultimate bending capacity

$$UC := \frac{M_d}{M_u} = 0.2$$ **OK**

Reinforcement - slab

$$z := d \cdot \left(0.5 + \sqrt{0.25 - \frac{K_{lim}}{1.134}}\right) = 68.92\ mm$$ lever arm

$$A_s := \frac{M_d}{0.87 \cdot f_{yd} \cdot z} = 236.9\ mm^2$$ area reinforcement required

$A_{s;prov} := 904\ mm^2$ area reinforcement provided

$$UC := \frac{A_s}{A_{s;prov}} = 0.26$$ **OK**

Deflection - slab

$$I := \frac{b \cdot h^3}{12} = \left(8.33 \cdot 10^7\right)\ mm^4$$ moment of inertia

$w_{allow} := 0.004 \cdot L = 6\ mm$ max deflection allowed at floors

$$w_{max} := \frac{5 \cdot Q_{SLS} \cdot L^4}{384 \cdot E_c \cdot I} = 0.27\ mm$$ **OK**

Geometry - glass beam

$b := 84\ mm$	beam width, w/ 7x12 mm
$h := 450\ mm$	beam depth
$L_b := 8.6\ m$	beam span
$L_s := 1.5\ m$	tributary width slab

Loads - glass beam

$kNm_{bm} := b \cdot h \cdot \rho_g = 0.95\ kN \cdot m^{-1}$	beam self-weight
$G_k := (G_{sk} \cdot 1.5\ m) + kNm_{bm} = 1.7\ kN \cdot m^{-1}$	characteristic permanent load
$Q_{bm} := 5\ kN \cdot m^{-2}$	variable load at beam
$Q_k := Q_{bm} \cdot 1.5\ m = 7.5\ kN \cdot m^{-1}$	characteristic variable load
$Q_{ULS} := \gamma_G \cdot G_k + \gamma_Q \cdot Q_k + (\gamma_Q \cdot \Psi_0 \cdot q_s) = 13.28\ kN \cdot m^{-1}$	ULS design load
$Q_{SLS} := G_k + \gamma_Q \cdot Q_k + (\Psi_0 \cdot q_s) = 12.95\ kN \cdot m^{-1}$	SLS design load

Moment - glass beam

$M_d := \dfrac{Q_{ULS} \cdot L_b^{\ 2}}{8} = 122.81\ kN{\cdot}m$	design moment
$W := \dfrac{b \cdot h^2}{6} = (2.84 \cdot 10^6)\ mm^3$	section modulus
$\sigma_m := \dfrac{M_d}{W} = 43.32\ MPa$	max bending stress at mid-span
$UC := \dfrac{\sigma_m}{f_{g;d}} = 0.93$	**OK**

Deflection - glass beam

$I := \dfrac{b \cdot h^3}{12} = (6.38 \cdot 10^8)\ mm^4$	moment of inertia
$w_{allow} := 0.004 \cdot L_b = 34.4\ mm$	max deflection allowed at floors
$w_{max} := \dfrac{5 \cdot Q_{SLS} \cdot L_b^{\ 4}}{384 \cdot E_g \cdot I} = 20.65\ mm$	**OK**

Fire - glass beam

$Q_{FLS} := G_k + 0.6 \cdot Q_k = 6.2\ kN \cdot m^{-1}$	reduction factor $\psi_2 := 0.6$ at Category C4: congregation
$b := 60\ mm$	delete 2-plies, w/ 5x12mm
$h := 450\ mm$	beam depth
$M_d := \dfrac{Q_{FLS} \cdot L_b^{\ 2}}{8} = 57.27\ kN{\cdot}m$	design moment
$W := \dfrac{b \cdot h^2}{6} = (2.03 \cdot 10^6)\ mm^3$	section modulus
$\sigma_m := \dfrac{M_d}{W} = 28.28\ MPa$	max bending stress at mid-span
$UC := \dfrac{\sigma_m}{f_{g;d}} = 0.61$	**OK**

Degree of strengthening needed at primary beam

Geometry - new glass beams below slab

$b := 84\ mm$	beam width, 7 ply x 12mm
$h := 450\ mm$	beam depth
$L_b := 8.6\ m$	beam span

Loads

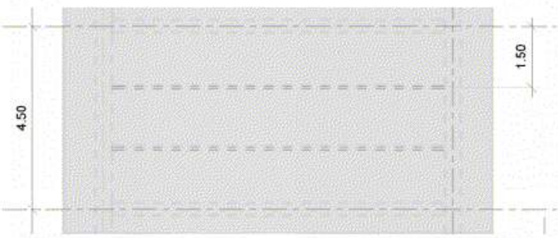
$\rho_g := 25\ kN \cdot m^{-3}$	unit weight glass
$L_s := \frac{8.6\ m}{2} \cdot 2 = 8.6\ m$	adjacent slab tributary width
$kNm_{sk} := 0.5\ kN \cdot m^{-1}$	linear permanent load at skatepark
$kNm_{bm} := b \cdot h \cdot \rho_g = 0.95\ kN \cdot m^{-1}$	self-weight gl beam below slab
$kNm_{slab} := L_s \cdot 100\ mm \cdot \rho_c = 20.64\ kN \cdot m^{-1}$	self-weight slab
$G_k := (G_{sk} \cdot 8.6\ m) + kNm_{bm} + kNm_{slab} = 25.89\ kN \cdot m^{-1}$	characteristic permanent load
$Q_{bm} := 7.5\ kN \cdot m^{-2}$	live load at glass beam
$Q_k := Q_{bm} \cdot 8.6\ m = 64.5\ kN \cdot m^{-1}$	characteristic live load
$Q := \gamma_G \cdot G_k + \gamma_Q \cdot Q_k = 127.81\ kN \cdot m^{-1}$	ULS design load

Moment - primary beam strengthening

$L := 13.5\ m$	span existing primary beam
$M_{new} := \frac{(Q \cdot L^2)}{8} = (2.91 \cdot 10^3)\ kN \cdot m$	new design moment
$M_d := 845.14\ kN{\cdot}m$	original design moment existing conc beam
$M_u := (2.08 \cdot 10^3)\ kN \cdot m$	moment capacity existing conc beam
$UC := \frac{M_{new}}{M_u} = 1.4$	**over capacity 40%**

C.3 Cafe/bar

Add two beams



Loads

$G_{bar} := 0.5\ kN \cdot m^{-2}$ bar dead load

$G_{slab} := 2.4\ kN \cdot m^{-2}$ slab self weight

$kNm_{bar} := 0.5\ kN \cdot m^{-1}$ linear dead load

$kNm_{slab} := 2.4\ kN \cdot m^{-1}$ linear SW

$q_k := 6\ kN \cdot m^{-1}$ live load, per C4 +1 for partitions

Check slab

$b := 1000\ mm$ slab design strip width

$h := 100\ mm$ slab depth

$d := 84\ mm$ effective depth to center of bottom bars

$L := 1.5\ m$ slab span

$G_k := kNm_{bar} + kNm_{slab} = 2.9\ kN \cdot m^{-1}$ total dead load

$Q_{ULS} := \gamma_G \cdot G_k + \gamma_Q \cdot q_k + (\gamma_Q \cdot \Psi_0 \cdot q_s) = 12.48\ kN \cdot m^{-1}$ ULS design load

$Q_{SLS} := G_k + \gamma_Q \cdot q_k + (\Psi_0 \cdot q_s) = 11.9\ kN \cdot m^{-1}$ SLS design load

Moment capacity - slab

$M_d := \dfrac{Q_{ULS} \cdot L^2}{8} = 3.51\ kN \cdot m$ design moment

$K_{lim} := 0.167$

$M_u := K_{lim} \cdot f_{cd} \cdot b \cdot d^2 = 15.71\ kN \cdot m$ ultimate bending capacity

$UC := \dfrac{M_d}{M_u} = 0.22$ **OK**

Reinforcement - slab

$z := d \cdot \left(0.5 + \sqrt{0.25 - \dfrac{K_{lim}}{1.134}}\right) = 68.92\ mm$ lever arm

$A_s := \dfrac{M_d}{0.87 \cdot f_{yd} \cdot z} = 269.26\ mm^2$ area reinforcement required

$A_{s;prov} := 904\ mm^2$ area reinforcement provided

$UC := \dfrac{A_s}{A_{s;prov}} = 0.3$ **OK**

Deflection - slab

$I := \dfrac{b \cdot h^3}{12} = (8.33 \cdot 10^7)\ mm^4$ moment of inertia

$w_{allow} := 0.004 \cdot L = 6\ mm$ max deflection allowed at floors

$w_{max} := \dfrac{5 \cdot Q_{SLS} \cdot L^4}{384 \cdot E_c \cdot I} = 0.31\ mm$ **OK**

Geometry - glass beam

$b := 84\ mm$ beam width, w/ 7x12 mm
$h := 500\ mm$ beam depth
$L_b := 8.6\ m$ beam span
$L_s := 1.5\ m$ tributary width

Loads - glass beam

$kNm_{bm} := b \cdot h \cdot \rho_g = 1.05\ kN \cdot m^{-1}$ beam self-weight
$G_k := (G_{bar} \cdot 1.5\ m) + kNm_{bm} = 1.8\ kN \cdot m^{-1}$ characteristic dead load

$Q_{bm} := 6\ kN \cdot m^{-2}$ live load at beam
$Q_k := Q_{bm} \cdot 1.5\ m = 9\ kN \cdot m^{-1}$ characteristic live load

$Q_{ULS} := \gamma_G \cdot G_k + \gamma_Q \cdot Q_k + (\gamma_Q \cdot \Psi_0 \cdot q_s) = 15.66\ kN \cdot m^{-1}$ ULS design load
$Q_{SLS} := G_k + \gamma_Q \cdot Q_k + (\Psi_0 \cdot q_s) = 15.3\ kN \cdot m^{-1}$ SLS design load

Moment - glass beam

$M_d := \dfrac{Q_{ULS} \cdot {L_b}^2}{8} = 144.78\ kN{\cdot}m$ design moment

$W := \dfrac{b \cdot h^2}{6} = (3.5 \cdot 10^6)\ mm^3$ section modulus

$\sigma_m := \dfrac{M_d}{W} = 41.36\ MPa$ max bending stress at mid-span

$UC := \dfrac{\sigma_m}{f_{g;d}} = 0.89$ **OK**

Deflection - glass beam

$I := \dfrac{b \cdot h^3}{12} = (8.75 \cdot 10^8)\ mm^4$ moment of inertia

$w_{allow} := 0.004 \cdot L_b = 34.4\ mm$ max deflection allowed at floors

$w_{max} := \dfrac{5 \cdot Q_{SLS} \cdot {L_b}^4}{384 \cdot E_g \cdot I} = 17.79\ mm$ **OK**

Fire - glass beam

$Q_{FLS} := G_k + 0.6 \cdot Q_k = 7.2\ kN \cdot m^{-1}$ reduction factor $\psi_2 := 0.6$ at Category C4: congregation
$b := 60\ mm$ delete 2-plies, w/ 5x12mm
$h := 500\ mm$ beam depth

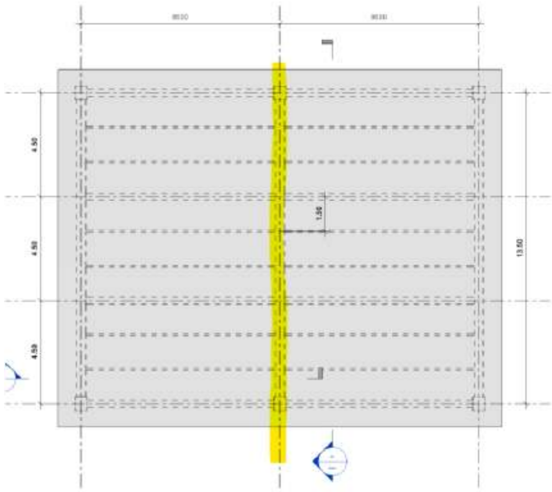
$M_d := \dfrac{Q_{FLS} \cdot {L_b}^2}{8} = 66.56\ kN{\cdot}m$ design moment

$W := \dfrac{b \cdot h^2}{6} = (2.5 \cdot 10^6)\ mm^3$ section modulus

$\sigma_m := \dfrac{M_d}{W} = 26.63\ MPa$ max bending stress at mid-span

$UC := \dfrac{\sigma_m}{f_{g;d}} = 0.57$ **OK**

Degree of strengthening needed at primary beam



Geometry - new glass beams below slab

$b := 84\ mm$	beam width, 7 ply x 12mm
$h := 500\ mm$	beam depth
$L_b := 8.6\ m$	beam span

Loads

$\rho_g := 25\ kN \cdot m^{-3}$ unit weight glass

$L_s := \dfrac{8.6\ m}{2} \cdot 2 = 8.6\ m$ adjacent slab tributary width

$kNm_{bar} := 0.5\ kN \cdot m^{-1}$ linear permanent load at bar

$kNm_{bm} := b \cdot h \cdot \rho_g = 1.05\ kN \cdot m^{-1}$ self-weight gl beam below slab

$kNm_{slab} := L_s \cdot 100\ mm \cdot \rho_c = 20.64\ kN \cdot m^{-1}$ self-weight slab

$G_k := \left(G_{bar} \cdot 8.6\ m\right) + kNm_{bm} + kNm_{slab} = 25.99\ kN \cdot m^{-1}$ characteristic dead load

$Q_{bm} := 9\ kN \cdot m^{-2}$ variable load at glass beam

$Q_k := Q_{bm} \cdot 8.6\ m = 77.4\ kN \cdot m^{-1}$ characteristic variable load

$Q := \gamma_G \cdot G_k + \gamma_Q \cdot Q_k = 147.29\ kN \cdot m^{-1}$ ULS design load

Moment - primary beam strengthening

$L := 13.5\ m$ span existing primary beam

$M_{new} := \dfrac{\left(Q \cdot L^2\right)}{8} = \left(3.36 \cdot 10^3\right)\ kN \cdot m$ new design moment

$M_d := 845.14\ kN \cdot m$ original design moment existing conc beam

$M_u := \left(2.08 \cdot 10^3\right)\ kN \cdot m$ moment capacity existing conc beam

$UC := \dfrac{M_{new}}{M_u} = 1.61$ **over capacity 61%**

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Codes and Guidelines

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