Proceedings of the International fib Symposium on the Conceptual Design of Structures

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introduction
The conference is organised by the Swiss group of the International Federation for Structural Concrete (fib).

The fib, Fédération internationale du béton, is a not-for-profit association formed by 41 national member groups and approximately 1000 corporate and individual members. fib’s mission is to develop at an international level the study of scientific and practical matters capable of advancing the technical, economic, aesthetic and environmental performance of concrete construction.

The fib was formed in 1998 by the merger of the Euro-International Committee for Concrete (the CEB) and the International Federation for Pre-stressing (the FIP). These predecessor organizations existed independently since 1953 and 1952, respectively. Today, the fib is the main international organization dedicated to concrete construction.

The fib-CH group achieves fib’s goals in Switzerland by locally disseminating information and knowledge obtained through the association’s international activities.
The conceptual design of structures is at the heart of the design process and when the most fundamental and influential decisions are taken for a project. It merges experience, intuition, tradition, site constraints, technical solutions and, above all, the genius and sensitivity of the designers.

The aim of the International fib Symposium on Conceptual Design of Structures 2021, which continues a series of symposia opened by fib and whose first edition was held in Madrid in 2019, is to generate a fruitful exchange event for academics and practitioners from engineering, architecture and other disciplines on the topic of the conceptual design of structures. The focus is placed on experiences made particularly during the design process. The discussions reflect how a project emerges, how design decisions are taken, how responsibilities are distributed, how obstacles and constraints are handled, how fundamental design principles are applied and the way the individual members of the design team collaborate.

Taking place in September 16-18, 2021, the Symposium follows an hybrid in-person/online format. The in-person events take place at the Attisholz Areal, close to the city of Solothurn (Switzerland). This reconverted industrial venue, which was originally used as a cellulose factory, witnesses the tremendous architectural potential for reuse of existing structures. More information on the venue and its access is available at https://www.attisholz-areal.ch/.
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themes & contributions

The contributions in these proceedings are organized according to the four main topics of the Symposium, as follows:

**exposed or concealed**: the interaction between structure and architecture.
How does structural design shape the overall concept?

**challenging gravity**: contemporary structures for our built environment.
How can structures challenge gravity with new systems, materials and construction technologies?

**rediscovering the past**: forgotten structures and concepts to rethink the future.
How can projects and concepts from the past be a valuable source of inspiration and knowledge for future projects?

**behind the curtain**: the creative role of structural engineers and architects in the 21st century.
Which responsibilities do structural engineers and architects face and which skills will they require in the future with respect to society, economy, and environment?

Authors had the choice between submitting a paper or a video. Papers are provided in full in these proceedings. Videos and a digital version of the proceedings are available on the website of the Swiss Society for the Art of Engineering:

[https://www.ingbaukunst.ch/de/veranstaltungen/conceptual-design-of-structures/](https://www.ingbaukunst.ch/de/veranstaltungen/conceptual-design-of-structures/)
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exposed or concealed
Embodied structural ambivalence: a neurophysiological perspective on structural expression

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Abstract
The paper attempts to provide a neuroscientific perspective on the discussion of structural expression. Taking human perception as the clue, this article starts with the notion of strong structures, take it as the base to review and analyses the principles of structural perception under the neurophysiological perspective. Based on the findings from mirror neuron and embodied simulation, this article further reviewed the impact of embodied structural ambivalence on structural expression and human perception from the concrete example to the theoretical implication. Therefore provides a neuroscience-based scientific perspective on the research of structural expression.

1 Intro: the question of structural expression
In the discourse since the enlightenment of European architectural modernity, Structural Expression has always been a vital issue in bridging construction technology and constructive culture. The perennial controversy over it stems from the game between technology and art. The Italian engineer and architect Pier Luigi Nervi has described the dual meaning of architectural phenomena: physical structures constructed in obedience to material requirements and constraints, and aesthetic significance aimed at generating subjective feelings [1]. These dualism comprehensions revealed the complex and indispensable relationship between the material and expressive dimensions of the structural design.

Subsequently, Eduard F. Sekler, in his discussion of the relationship between structure, construction, and tectonics, describes structures as “the more general and abstract concept refers to a system or principle of arrangement destined to cope with forces at work in building” [2, p. 89]. As an “intangible concept”, the structure “is realised through construction and given visual expression through tectonics.” [2, p. 92]. Therefore, the structures, on the one hand, constrained by the construction technique and, on the other hand, bonded to the perceptual representation of the tectonic form. This tension between technology and art forms the tension between structure and its’ expression, which is a crucial topic when discussing the experience and perception of the architectural space. From the appeal for structural rationality in the enlightenment period of modernity to the introspection on the relationship between structure and ornaments in the postmodern period, then to the discussion of structural performance in contemporary architecture. All of them are exploring the various degrees between “the truth” of structure and its expression. In other words, the degree of exposing or concealing structure in architecture.

2 The secrets in “Strong Structures”
On the description for using the supporting structure as a medium for architectural expression, Arthur Rüegg proposed the notion of Strong Structures (Starke Strukturen) [3]. It was defined as “load-bearing structures that do not secretly fulfil their function by carrying loads to the ground as discretely as possible, but instead make architecture out of this existentialist theme, this drama.” [4, p. 2] Strong structures describe structural thinking that focuses not only on a structure’s physical properties but also on the structural visual performance and spatial concepts. At the end of the article, Arthur Rüger described the current trend towards the strong structures: “However, their emphatically structural buildings are not all that simple to decipher precisely like those of their predecessors. They contain secrets that can only be unlocked by employing a certain amount of patience...This concealment of the flow of forces is combined with a love of abstraction and the elimination of the boundary between inside and outside space that has flared up at regular intervals since the time of early modernism.” [3, p. 11] In contrast to
the obsession with the authentic expression in tectonic design in the discourse of structural rationalism, the article also uses the Leutschenbach School and the New residential and commercial building at Ottoplatz as examples to explain that the structural logic they express does not emphasise a clear and direct physical and visual correspondence between the structural representation and the force flow, and can even be interpreted as concealment of the structural logic.

![Fig. 1](image1.png)

**Fig. 1** (a): Leutschenbach School, Zurich, 2009, Architect Christian Kerez, Engineer Joseph Schwartz. Architect. (b): The structure model for the misaligned and repeated trusses in Leutschenbach School © Walter Mair

In the case of the Leutschenbach School, due to the limitation of the site, Kerez wanted to retain a large green space area alongside the new functions. Based on the concept of superimposition from Kerez, Schwartz cleverly transforms these constraints into an essential part of the structural expression and gives the building a sense of lightness and floating (Fig. 1a). From the outside, the repetitive trusses on the façade disappeared from the first and fifth floors of the building, making the whole building seem to be levitated. Due to the partial absence of the structure, people cannot read the logic of the forces throughout the building in a continuous manner, making for an intriguing and unusual structural expression. However, the structure of the whole building is apparent and logical - the structural anomalies are on the one hand due to the inward movement of parts of the structure; on the other hand, the same truss-like forms are composed regarding different types of force, but their similarity and repetitiveness confused the human understanding of the structural logic (Fig. 1b). A similar expression also could be seen from the building at Ottoplatz. Although it looks very different from the Leutschenbach School, the structural system between the two is actually very similar - the hidden force flow behind this facade is similar to the standard V-shaped truss (Fig. 2b). In the design of the building at Ottoplatz, in order to maintain continuity with the tessellated façade layout of the surrounding buildings. The openings of windows on the façade are presented in many rectangular solid panels and openings at each level (Fig. 2a). The structure of the building at Ottoplatz also leaves the first floor of the building “missing”, but
the overall structure expression is very dissimilar to the Leutchenbach School - the form of the structure in the building at Ottoplatz does not directly correspond to the truss-like form of the internal force flow, and the internal force flow only controls the dislocation of the solid and opening parts on the deep beam. The contrast between these two cases clearly shows how to execute the construction, which is the additional layer beside abstract structural logic, also strongly influences people’s understanding and perception of structure.

It is worth distinguishing that these concealments are not the additional cladding to obscure or decorate the structure [5]. On the contrary, the concealment in these two examples corresponds more to the intuition of the people’s perception and design intention of structure. In short, the structural expressions sought by strong structures do not care whether the logic of force or construction can be quickly understood through the observation of the structure. In addition to the clarity and authenticity, strong structures also embrace the structural thinking that seeks to create an ambiguous “uncertain certainty” that is even atectonic: the structure is exposed without revealing all the logic [6].

Then why do these architects unanimously choose to create this ambiguity or concealments in the structural expression? And how people read these expressions and be able to feel into them? To answer these questions, we could get back to the former discussed Intangible part of the structure, where Sekler’s focus on the “visual” level of the structure reveals the essential medium for the experience of the structure - perception. Intriguingly, Sekler draws an analogy between tectonics and the expression of artworks in his description of structural perception, arguing that structural expression is an Einfühlung (Empathy) between the built environment and the human body [2] [7], therefore intensifying one’s experience of the internal forces in structural forms. Based on the human body and experience, this article will rethink the relationship between the technical and expressive dimensions of structures, trying to provide an interpretation to create such secrets in strong structures from the perspective of embodied perception.

3 Embodied perception

In the preface to Style, Harry Francis Mallgrave explains that Semper’s discussion of the reading of the internal forces in forms led to the germination of empathy in the field of architecture and art at the end of the 19th century [8]. It grew a long time scientific interests and debates on the human psychological and physiological perception of form and structures [9] [10] [11] [12]. These researches are all attempt to explain that the body for us is the most direct way we understand space and architecture - they are the ontological principle about how humans read the emotion or meaning from the structures.

3.1 Neurophysiological embodiment

However, the theory of empathy from the early twentieth century was mainly driven by technology-oriented formalism. It is essentially a semiotic approach that lacked scientific physiological or psychological basics to unambiguously explain or demonstrate how we feel or project our emotions into the behaviour of others or the built environment and resonate with their forms [12]. Therefore, the theory of empathy was mainly used as an explanatory approach instead of a design approach to inform a pre-reflective embodied design method. The recent developments in the relatively new discipline of Cognitive Neuroscience open up a new way of conceptualising structural design that expands our theoretical framework through the notion of embodiment and embodied simulation. With the aim to investigate representational and artistic mechanisms of perception, cognitive neuroscience provides an egocentric perceptive on human physiological or psychological perception and understanding of structures [13]. Neuroscience’s findings can provide a rigour explanation and design factors for the ambiguity mentioned above towards structural reading and design in terms of embodied perception.

As the milestone of neuroscience, the discovery of the Mirror neuron in the mid-1990s, reveals that our neural circuits, in which we simulate the actions of others, are in the same areas of the brain through which we undertake our own actions [14]. It explains that the same neural structures involved in our own body-related experience contribute to conceptualising what we perceive from the world both visually and motorically [15]. This embodied perception process is based on the mechanism that humans will start the non-conscious or precognitive type of perception before consciously analysing it. The former based on the activation of our previous bodily experience and its related emotions [16]. Therefore, the mirror perception system is “a direct form of ‘experiential understanding’ of others, achieved by modelling their behaviours as intentional experiences, based on the equivalence between what the others do and feel and what we do and feel.” [17] This means that the human body is an essential basis
for our perception to have an empathetic relationship with the world besides vision [14]. In brief, the mechanism of mirror neurons is based on people’s memory of past bodily experiences and feelings. When people observe a gesture similar to that bodily memory or repeat an action similar to that memory later, they directly and unconsciously evoke the past bodily experience and mood corresponding to this bodily gesture, thereafter realising our ability to read into things. This could explain the perceptual commonality between the structural engineer and a random individual with no structural background – their first unconscious perception of structure is almost identical since they share a nearly identical body. The structural engineer’s knowledge or other people from different educational/psychological backgrounds and different cultural sensibilities, will appear in the conscious and analytical reading of the structural system after the unconscious impression.

Based on the mirror neuron, the notion of “embodied simulation” was proposed as an extension to explain the mechanism of how humans not only “see” the built environment but also feel and simulate emotions and actions from the world through the medium of the body [17]. The findings of the embodied simulation were based on the premise that perception and cognition inherently depend upon the organisms’ interaction with their environment [18] [19]. Which meanings, human perception emerges from the active dynamic interaction and movement, and thus get the “meaning” through the evocation of similar experiences of our bodies, which is a form of “experiential understanding” of the environment [17] [20]. The recent studies also show that our embodied simulation is not restricted to the social world. Humans have the “precognitive capacity to mirror the tactile values of all objects or forms in our environments, both living and non-living” [21]. This provides a solid theoretical basis for our embodied understanding of the built environment like structures. The embodied thinking can vindicate the discussions of embodied perception in the theory of empathy and phenomenology from a scientific level.

Therefore, based on the embodied neuroscientific perspectives, we can extensively reveal the structural perception logics of the Leutschenbach School. As embodied simulation emphasises, our perception of the built environment is primarily based on our previous bodily experience, which means that the people’s “imagination” of the force flow behind the structure is also based on how our motor-sensory felt the forces when we project ourselves into it. However, the partial “missing” in the overall structural relations of the Leutschenbach School direct interrupting the perceptual continuity. From the human bodily experience, all gravity associated with our bodies is transmitted continuously from head to the ground. Suppose we jump up and off the ground, as the body can only remain floating for an instant due to gravity. Therefore, our bodily muscle memory will perceive this movement as a state of instability. This means that embodiment will imply unstable and instant emotions if people do not see this continuity, then they get incomprehensible, curious, abnormal, and nervous. They are the main reasons for the feeling of “secrets” in the structural expression of the Leutschenbach School.

### 3.2 Neurophysiological ambivalence

It is also noteworthy that the repetition of the same truss-like elements in Leutschenbach School is intended to stimulate the perception of the missing part in the holistic structural continuity, thus reinforcing the mystery of “floating”. From the perspective of embodied perception, these structures deliberately set up the anomalous expressions to our everyday bodily motor experience, thus evoking our curiosity and intensifying our attention. This approach reveals, intentionally or unintentionally, that the activation of embodied mapping is a very crucial part of embodied perception. Our embodied perception can only occur if our perception is activated and then trigger the starting point for the process of embodied perception. This eliciting of the embodied perception by the structural ambivalence can be more precisely explained under another notion from cognitive neuroscience: arousal, which is central to the emergence of embodied perception [22] [23] [24] [25] [26].

Noteworthy, the balance between the everydayness and the abnormal of the perception stimulation is also crucial and subtle. The findings from arousal indicate that either too little or too much stimulation tends to be disregarded by the individuals [26] [27]. For example, in the case of Leutschenbach School, besides its’ unusual structural expression of continuity, the chosen specific structural elements are the common and standard trusses. It is the perception that these structural elements are familiar and understandable in the first place that accentuates their relationship. If the single structural element were already complex and alienated, it would instead interfere and even deprive the possibility of embodiment.

The above neuroscientific research and case studies could show that the appropriate structural embodiment could stimulate and evoke corresponding bodily sensations, emotions, and movements, thus directly shaping people’s perception of space and atmosphere.
4 Embodied structural ambivalence

Not only strong structures emphasised the ambivalence perspective on designing structures, Pérez-Gómez, for example, used to define that architecture is not just about imitating a comfortable environment to suit our needs, and it is about to arouse our embodied pre-reflective perception to feel the presence. He extensively reveals that rather than pleasing us, architecture is like a “heteropoietic system” that challenges our perception to get more imagination [28]. According to embodiment simulation, the human body is the primary medium through which we experience gravity and force. Many design projects have already practised similar structural thinking using perceived “force” and ambivalence within structures to evoke bodily experience memories. For example, Antoine Picon’s interpretation reveals that the structural forms designed by Eduardo Torroja are intended to play with the radical structural expression regarding the static equilibrium to challenge the embodied feeling [29].

However, in using similar bodily implication principles to create the drama of strong structures, different architects have different ways and aims. In the case of the Leutschenbach School, the embodied structural ambivalence is more likely to create a sense of lightness, openness, and levitation, thus challenging the visual-body perception. The appropriate combination of embodied structural ambivalence and architectural concepts can further enhance and convey the design intentions by implying and guiding the human embodied perception. For example, in the case of Tanikawa House and Plantahof Auditorium, both architects claimed that they intended to create a sense of spatial ambivalence, which successfully shaped the spatial tension with structures. Although their structural expressions are all related to embodiment, the underlying reasons and design methods are different from a neuroscientific point of view.

The Tanikawa House (Fig. 3a) was designed in 1974 by Kazuo Shinohara. To create a dialogue inside the building with its’ nature context, Shinohara exposed all the structures, including a large area of natural soil inside the house. Inside the building, the white roof and dark earth slope, the slenderness of the column, the proportion between the open space and the living area, was design to augment the contradictions between functional and non-functional, balanced and unbalanced, artificial and natural, inside and outside [30]. This opposition-oriented ambivalence in structural expression could stimulate human’s embodied perception in twofold. First, the seemingly unbearable scale of the slender columns. Standing inside, the exaggerated height and the pristine white roof make the ceiling appear almost flat, reinforcing its sense of sky-like scale from the human perspective. The exaggerated sense of scale is reinforced by the two slim columns that hold up the roof, enhancing the sense of instability of the roof and structure. From the perspective of embodied perception, one can directly imagine the bodily oppression generated by two skinny people attempting to hold up a sky with their arms. Secondly, the interior of the building leaves almost three-quarters of the area to the exposed earth but compresses the living space into the second storey on one side of the building (Fig. 3b). In contrast to the traditional way of habitability, Tanikawa House creates a spatial tension between the everydayness and the non-everydayness, functional and non-functional, that could make people feel nervous. Therefore, “people began to walk back and forth, trying to relieve their anxiety. There are many kinds of ways to interact, from which a variety of meanings will be generated.” [31] With this movement, people start walking on the soft earthy ground, and the feeling of the gentle slope activates the bodily experience of being outside and nature. Therefore, it generated a body-oriented mental connection between being inside and outside beside the physical approach. Consequently, in the Tanikawa House, the structural expression is conveyed through ambivalence-evoked bodily memory. The exaggerated and unstable naturalistic sense of scale inside the building, the direct exposure of the earth, and the inversion between the artificial and natural space are all intended to convey people’s previous bodily experiences in nature. The building creates a connection and dialogue with the architectural context and atmosphere from an embodiment perspective. Therefore, the structural ambivalence becomes the medium to connect the contextual concepts and concrete building in Kazuo Shinohara’s design.

The structures in Plantahof Auditorium designed by Valerio Olgiati have different motives for creating structural ambivalence expression. In Plantahof Auditorium, Olgiati designed a diagonal brace with an exaggerated scale to support the roof. This giant diagonal brace was also designed to be in an unusual misaligned position - it pierces through the vertical wall and sticking out of the exterior, only exposing a short portion of the support (Fig. 4a). Like the Tanikawa House, this dramatic inclined structure and unreadability in structural relations could stimulate human’s embodied perception from twofold. First, the diagonal brace activates the human bodily experience of an inclined bodily gesture and carrying loads. From the exterior of the building, this 45 degree inclined bracing appears to support
the massive wall load with its exposed relatively small volume. Therefore, one’s understanding of this structural expression would directly correspond to the bodily experience of the difficulty in maintaining stability when our bodies are tilted to 45 degrees, which consequently evokes a sense of imbalance. Furthermore, the contrast in scale between the vast wall and the smaller exposed part of the structure directly evokes the bodily experience of attempting to hold up a massive and heavy object with our bodies, thus experiencing a sense of oppression. Secondly, as this inclined bracing penetrates the wall, people can only partially read the structure both from inside and outside (Fig. 4b). This fragmented representation of the structural system is very similar to interrupting the expression of the structural continuity in the building at Otttoplatz and the Leutschenbach School; it makes the overall structural logic unclear and unreadable and thus creates a sense of confusion and curiosity. Olgiati explained the structure thinking of Plantahof Auditorium as: “the Plantahof Auditorium has an outer shape that does not allow one to understand the entire building organism. From the outside it seems that this building does not need and supports. On the other hand, if we only see the building from the inside, it also seems as if no restraints are necessary. Only when we see the entire building, do we begin to recreate it in our mind and understand why it has supports, why they have the dimensions they do, and why they are positioned as they are… This is all the more so because of the specific location of these structural elements in the room…I am convinced that if people are confronted with something that resembles nothing and something that they cannot yet handle, they begin to fathom this and ultimately experience it positively…” [32, p. 64] This ambivalence structural relations makes the structure becomes a trigger to encourage people to move and interact with the building actively - in Olgiati’s words, it could “stimulate thought”, thus allowing people to be involved both physically and mentally.

Fig. 3 (a) Tanikawa House, Karuizawa, 1974, Kazuo Shinohara. (b): The floor plan of Tanikawa House, the left narrow part is the living room; the right empty part is the exposed earth.

Fig. 4 (a) Plantahof Auditorium, Landquart, 2010, Valerio Olgiati. © Javier Miguel Verme (b): The interior of Plantahof Auditorium. © Javier Miguel Verme

Whether it is the Leutschenbach School, Tanikawa House, or Plantahof Auditorium, the design of the structures in these projects is to find the balance of statics, dissolving the conflict of gravity and force; whereas their structural expression are all contrarily tending to creates an unbalanced, discontinuous,
and even disordered spatial condition to stimulate human perceptions. Their simultaneous consideration of embodiment and ambivalence implicated in structural expression expressed a paradoxical structural condition that is logically balanced in statics but unbalanced in its expression. Both of these approaches contribute to creating the drama of strong structures and successfully convey their architectural concept by considering bodily experience.

5 Conclusion

What is significant about the research of neuroscience in architecture is that it reveals the natural principles of perception. It allows us to re-examine the question of structural expression in more rigour and scientific way. From a perceptual point of view, the perspectives of mirror neuron and embodied simulation seem to answer the question that architects and structural engineers have debated for many years: why and whether to express structure authentically in architectural space [11].

In the light of neuroscience, the architecture may or may not be a wholly authentic expression of structural logic. The exposure of the structure is not the aim or the criterion. From the neurophenomenological perception point of view, the appropriate conveyance of the structure expression to the embodiment of bodily gesture and muscle experience, and the stimulation of bodily movement and interactions, are the true objectives of structural design - to positively influence and enhance design intentions. What noteworthy is, only the appropriate degree of perception stimulation in the light of arousal could positively enhance the design intention instead of overload it.

Indeed, structural clarity is the basis for readability, but above this clarity, the degree of embodiment of the structural form is the determining factor in the embodied experience of structural expression. What truly matters is whether the degree of embodiment of structural expression can act as attention or stimulation cues under human embodiment principles.

This awareness is not limited to the interpretation of structural expressions but can also influence the future of structural design. Like the vision that strong structures are chasing, neurophysiological perspectives allow the structures to go beyond their load-bearing properties and be associated with human perception, emotion, and behaviour, becoming an embodied structure.

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Design principles providing solutions to multiple engineering tasks

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Abstract

Conceptual design in structural and civil engineering involves balancing interdependent multiples tasks in order to provide answers and interdependent trade-offs in form of best possible solutions. There are several ways to accomplish this goal, however the principles of the conceptual design should be followed. By developing the design process of structures, eminent factors like project constraints, use, existing environment, aesthetic value and owner goals should be considered. Consequently, the shape, the materials and the structural system have to be analysed and carefully chosen to deliver a precise and efficient solution within the project constraints.

In this contribution we outline three different approaches that were pursued by our design team with the aim to provide efficient solutions and satisfy the acceptance of users and owners. The author's intent is to give a brief overview on these classical engineering tasks, in order to show the benefits of in-depth conceptual design in the early project stage. The essential issues and the few principles analysed and defined by the conceptual design were successfully applied in the following design stages and during execution.

The structure of the West Campus in Lugano was planned to fulfil multiple tasks and serve multiple users (Students, teaching, laboratories, workshops, events). Therefore, the structure was designed in order to minimize the presence of vertical structural members in the interior spaces. Precast and pre-stressed deck elements were designed in order to be slender, to speed up construction time, to integrate installations without additional space and to provide excellent flexibility for future conversions. The facade concept makes use of a stabilizing concrete frame structure, allowing to overcome wide spans over the entrances and the major interior spaces. The conceptual design of the load-bearing structure merges into architectural expression and becomes part of the urbanistic value of the large-scale building.

The Passwang pass road was originally built in 1930s during severe economic depression as a part of social plans investments (job creation scheme). The refurbishment of the road in 2014-2019 had to be planned considering several local constraints like variable topography, permanent road traffic during construction, difficult geological conditions, natural hazards and short construction times owing to alpine weather conditions. In 2019 the project was awarded with the heritage prize from Heimatschutz Solothurn, recognizing the social value of the construction and the wise design in harmony with the landscape.

The new Viaduct Klus with a length of 296 m leads the new bypass road over a dense built environment with several pre-existing infrastructure. The skew crossing of the complex environment with an industrial site, the railway line as well as natural obstacles impose strict constraints. The proposed design solution with a slender steel and concrete composite bridge and precisely placed pillars results directly from these constraints. The new Viaduct Klus becomes part of the landscape and is an indicator of technical evolution with a significant and multifunctional impact on the life of this area.
1 Concrete shell in urban landscape

The structure of the West Campus in Lugano is the result of an international competition for Architects and Engineers held in 2010. The University Campus is a dynamic facility, who needs to react to future needs and to meet requirements for future conversions. Therefore, the flexibility of interior spaces should be an essential feature.

Flexibility can be achieved by keeping the amount of vertical structure low, which was the aim of the proposed solution.

The West Campus is one of the largest building in Lugano. The "doughnut"- shaped building around an inner square and its organisation in volumes with different heights has an expressive urban planning impact. The interior public square is the largest in the city of Lugano.

Fig. 1 Flight view of the construction site in direction south (left) and from the top (right).

The vertical structure is characterised by a peripheral perforated concrete shell which is completed by internal cores holding elevators, building installations and other similar services. This tube-in-tube structure is designed to bear gravity loads and lateral forces due to wind and earthquake. Lateral forces are transferred from the outer structure to the core through the interconnecting floor structure. The floors consist of precast concrete slabs, which are connected by a top layer with site-poured concrete. The combination of external frame and internal core action permits open architectural spaces and provides high horizontal stiffness.

Fig. 2 Floor plan P1 (left) and cross section of the building (right).

The peripheral concrete frame structure allows to overcome wide spans between point supports over the entrance zones, wide facade openings and large interior spaces. The structural behaviour of is shown for the facade side pointing to the South over the main entrance. This part of the facade is 34.5 m large and the spans between the supports are of 12 m and 3 x 7.5 m, respectively. The behaviour of the façade-structure over the main entrance can be compared with a slender deep beam with large openings, which is placed on point supports. The structure is considered as two-dimensional plane shell with a thickness of 25 cm and subjected to plane stress condition. For the design, the shell behaviour was analysed using stress fields [1].
In the design phase the size and the position of the openings were discussed and investigated balancing between high vertical loads from precast long span deck elements, architectural design requirements, geometrical constraints, natural lighting conditions and energy labels regulations. The regular alternance of the openings and the overlap of the single plate elements allow to design vertical struts, inclined diagonals and horizontal strings in order to transmit shear, tension and compression forces. The application of stress fields theory [1] allows the investigation of the distribution of the internal forces in slender elements and the design of vertical and horizontal reinforcements as well as critical joint elements. The investigation showed that the lower zone of the facade structure, a continuous string element under the openings and over the single supports, is subjected to very high tensile and shear stresses. This requires special reinforcements with welded steel beams set in concrete (see Fig. 3). Due to the high slenderness of the facade, post-tensioning systems and stirrup reinforcements cannot be sensible used.

In the sectors where the facade structure is monolithically fixed to the basement structure, its behaviour can be described as atypical multi-storey shear wall with large openings, loaded by dead and live loads as well as horizontal forces (wind loading, earthquakes). The facade acts together with the internal cores like a tube-in-tube system and provides very high horizontal stiffness to the structure (see Fig. 4).

The floor system of the building is made by precast and prestressed deck elements with span lengths between 8 and 14 m. The structural members are designed in order to be slender, to speed up construction time and to integrate installations without additional space. The openings in the web of the decks
are designed to fit different layers of technical installations. The long span deck elements allow for adapting or expanding technical installations and provide excellent flexibility for future conversions.

![Stress fields and reinforcements](image1)

**Fig. 5** Stress fields (i-concrete, EPFL) and reinforcements in the supporting brackets (left). View of the main entrance of the Campus West (right).

The precast deck elements are mounted on thin wall brackets (see Fig. 5) and connected by a site-poured reinforced concrete layer that provides a shear-resistant structure without joints.

Due to the high ratio of slenderness and large opening in the webs of the precast beams, computational fire simulations were carried out at the ETH Zurich in order to investigate the fire resistance of the elements.

The example shows how conceptual design could be applied in building conception achieving expressive and rational architectural design.

### 2 In symbiosis with the landscape

The Passwang pass road was built in 1930s during sever economic depression period as a part of social plans investments. About 300 peoples were involved in the construction over a time of 5 years. Among them about 270 were unemployed, mostly from the watch industry. The Passwang pass road is characterized by the typical elements that were used in the early 20th century. The road with switchbacks was laid in the steep slopes maintaining a maximum slope of 12%. The excavated materials were reused to construct valley-side embankments in order to minimize the transportation of new material. Moderate geotechnical structures like retaining walls were built in order to minimize the construction costs. These structures were designed very cautiously and uniformly and adapted to the terrain. Thanks to these principles the road results well embedded and in symbiosis with the landscape.

![Passwang pass road](image2)

**Fig. 6** Passwang pass road with switchbacks [2] (left) and view of the refurbished road on the upper part (right).
The heavier traffic loads but especially the difficult geological conditions have recently led to increasing damages and much larger maintenance requirement. Considerable settlements could be observed, especially along the valley side lane. However, the road and the slopes on the mountain side still didn't show any sign of instability. Accurate geotechnical investigations as well displacements and settlements monitoring were carried along the road over several years in order to collect important informations for the elaboration of a consistent refurbishment project.

The refurbishment of the road had to be planned considering several constraints like topography, road traffic during construction, landscape protection, difficult geological conditions, natural hazards (rockfall and landslides) and short construction times characterized by alpine weather conditions. The project was optimised to keep maintenance costs and intervals as low as possible.

The location of the Passwang pass road in the Jura protection Zone and the fact that the road is registered in the federal inventory of historical traffic routes IVS as an object with national importance increased the importance of a wise conceptual design. The special landscape maintenance conditions led to solutions with slightly visible impact.

The stability of the slopes had also to be considered. The stability was verified using limit analysis and soil plasticity theory for different construction stages according to Chen [3]. The construction methods were carefully implemented according to the geotechnical constraints, with consideration of the landslide hazard.

The cross-section of the Passwang pass road is no longer compatible to the requirements of today's traffic and must be widened to ensure adequate road safety. The widening of the road was planned to take place on the valley side, so that no significant interventions in the landscape were necessary.

In the upper section of the 3.5 km long road, the geotechnical conditions are characterized by the presence of rock and debris. Due to the angular shape fragments and the high angle of internal friction of the debris the angle inclination of the slope is up to 45°, which is why the risk of falling rocks in this zone is considerable. In this section, a new monolithically jointless slope viaduct with a total length of 540 m was built in order to support the valley side of the road, see Fig. 7. Due to the limited space, restricted access and difficult topography the slope viaduct is founded on micropiles, which requires small and light drilling machines. By that, the risks due to the realisation of a drilling planum on steep slope and high costs for drilling scaffolding or platform needed by big larger diameter piles could be avoided. The jointless concrete construction of 540 m is unique for swiss standards. In order to limit the distribution and the width of cracks along the cross section, proper longitudinal reinforcements were designed [4] and construction method was adapted, building the slope viaduct in alternating segments with the help of special developed formworks.
In the following section, outside the steepest section, the topography becomes more favourable and the road runs within slope wash and opalinus clay. Particular attention must be paid to slope drainage to avoid the risk of landslides and creeping movements. Thanks to the more favourable space conditions, the deep foundation of the geotechnical support structures on the valley side could be carried out with the help of large diameter bored piles. This construction method is more economical than the one with micropiles and enables rapid construction times. In order to increase stability and torsional stiffness the piles were bored with varying offset.

The retaining walls on the mountain side are mostly stable over the entire section of the road, although due to the low quality of concrete and not adequate foundations a complete refurbishment is needed. At first the damaged retaining walls where back-anchored and the concrete surface was roughened. After this a 30 cm thick concrete shell with independent foundation was poured over the walls. The surface structure of the retaining walls is uniform and realised with vertical formwork panels. The original appearance of the retaining walls on the mountain side is maintained by the inconspicuous reinforcement design and the slight elevation of the new walls.

As a result of the careful conceptual design, the new constructions appear as a slim uniform line leaned on the slope along the entire road. Thanks to the slightly shape of the wall console, the perceived height of the retaining walls has a very low impact on the view and can be seen as a narrow string cutting the slope.

In 2019 the project was awarded with the swiss heritage prize from Heimatschutz Solothurn, recognizing the social value of the construction and the wise design according to the landscape.

3 Composite bridge as a solution in complex environment

Bridges are spatial structures designed to overcome physical obstacles. There are several ways to accomplish the goal depending on the physical obstacle, how the bridge will be used, and pre-existing
environmental or built structures. Consequently, the bridge shape, material, and structural system have to be analysed and carefully chosen to deliver a precise and efficient solution within the project constraints.

The Klus from Oensingen to Balsthal in the Jura is a topographical bottleneck for road and rail traffic. Due to the heavy commuter traffic and existing railroad level crossing, daily traffic jams occur. The village of Klus especially suffers from these circumstances. The life quality is impaired by noise and exhaust gases. In order to decisively increase the quality of stay at this location, a bypass road for trough traffic is planned.

For the new bypass road, a new routing with the aim of the best possible integration in the pre-existing landscape was studied. The solution resulting from the intense conceptual design process is shown in Fig. 10. A new viaduct with a total length of 296 m spans over the existing infrastructure in the valley of the Klus at a height of 6 to 10 m. The viaduct crosses the Schmelzihof area, the railroad tracks, the river Dünnern and the Klus industrial zone. From the west of the viaduct the road is led into the Guntenflüeli tunnel, which bypasses the rock hill to the west.

The viaduct, is designed as a slender frame structure with 10 fields and variable spans between 21.6 m and 32.9 m. The substructure of the viaduct is made by slim pairs of pillars made of Corten steel, whereby the rust colored steel set a reference to the important steel production tradition in the area of the Klus. The chosen alternate arrangement of the frame legs, that follow the axis of the Dünnern creek and of the railway line, results from the local space constraints and from the aim to provide a minimum of obstacles under the bridge.

Fig. 10 Situation and view of the viaduct Klus.

Fig. 11 View from Bahnhofplatz Klus
The pairs of frame legs and the abutments are skew and parallel to the railroad tracks and to the Dünnern creek, with a very acute angle of 29.5 ° in plan with respect to the bridge axis. Through the dissolution of the substructure into very slender elements, the transparency is maximized at the terrain level and at the same time, the impact on the land area by the pillar foundations is minimized. In the longitudinal direction of the bridge, the frame effect of the offset pillars can be recognized.

The bridge superstructure is designed as a steel-concrete composite construction and consists of two longitudinal girders in Corten steel with a box cross-section and a roadway slab above them that acts as a composite (see Fig. 12). The very slender roadway slab is designed for two-lane road traffic and have on both sides a cantilever beam, equipped with glass sound barriers. The design elements of the bridge cantilevers are perpetuated in the adjoining retaining walls, creating continuity along the new infrastructure. Thanks to the steel-concrete composite solution, the viaduct appears as a slight transparent line. The composite construction also offers the advantage of a very economical construction process. The viaduct can be erected without falsework and by that to minimise the impact and the time of the construction process.

4 Conclusions

Although projects like buildings, geotechnical structures and bridges have the task to fulfil precise and simple functions, examples show how different and complex could be the design approach in order to provide viable solutions. The search of efficient and suitable answers to engineering tasks according to the given constraints is an integral part of conceptual design. This process starts with the first draft of the structure, choice of material and construction process. The aim of conceptual design applied to classic engineering tasks is to deliver the best available solution within project constraints. The constructions costs, low maintenance requirements and durability of the projects should always be on the background.

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Designing for innovation: from model-use to thinking in models

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Abstract

As typical problem solvers, structural engineers often face new and complex problems, recently for instance due to digitalization or climate change. Often, such problems cannot be approached using established methods. Instead, structural engineers need to rethink existing problem-solving processes and develop new approaches. In this paper, we propose that an inductive approach of thinking in models is essential to address complex new problems in structural engineering and generate innovations. Based on a literature review, we explore the meaning and importance of this approach and illustrates it with two examples of practice. Hereby, we aim to provide an account of the potentials of this approach in structural engineering to design for innovations and seek to contribute to a change of mind-set in engineering practice as well as in research and education.

1 Introduction

Engineers are typically perceived as “problem solvers” [1]. Depending on the nature of the problem, the problem-solving process varies. While narrow and well-defined problems can usually be tackled in a deductive manner applying known methods and following established procedures, new and ill-defined problems often first require an inductive development of new methods and processes before they can be solved. Examples for such new and ill-defined problems are the ones that emerge due to climate change or digitalization. How can we built more resource-efficient, minimise emissions and create flexible structures which can be adapted to changing climate conditions? How does the digitalization affect the work flows, creative processes and collaboration in structural engineering?

In this paper, we do not want to focus on the specific solutions to these problems. Instead, we aim to contribute to a meta-discussion on different approaches to problem-solving. Hereby, we focus on the essential role of models in the problem-solving process: For centuries, engineers have tackled both established problems and new challenges, as well as generated approaches, methods and solutions for them by translating real-world problems into model-world problems. Duddeck refers to working with models as the core of the engineering profession [2, p. 90]. We propose that there are two modes of working with models which come to use in problem-solving processes: a mainly deductive model-use and a mainly inductive thinking in models. In the latter, the problem-solving process itself is adapted to the respective problem and its challenges, which makes inductive thinking in models essential for the generation of new ideas and innovative solutions.

Based on a literature review, the paper explores the meaning and importance of thinking in models for approaching complex and new problems in structural engineering, such as the ones emerging due to climate change and digitalization. In the next section, the role of models for structural engineering and the two modes of working with models, a mainly deductive model-use and a mainly inductive thinking in models, are described. In the third section, the concept of thinking in models is illustrated with two examples of practice, which show how it can manifest in concrete techniques and methods as well as in different contexts and scales. Section four highlights key characteristics of thinking in models which become apparent in the examples. We conclude that teaching and training thinking in models should play a vital role in engineering education. Furthermore, it is essential that engineers better understand and question their methods and approaches, particularly their use of models. The chosen examples of practice should inspire new generations to think conceptually and develop their own methods and approaches by thinking in models. The paper aims to provide an account of the potentials in structural engineering to design for innovations and seeks to contribute to a change of mind-set in engineering practice as well as in research and education.
2 Models in Structural Engineering: Model-Use versus Thinking in Models

Due to the fact that their structures are always prototypes and cannot be tested as a whole before they are put to use, working with models is common practice for structural engineers. Models are used to transform complex realities into illustrative, simplified and abstract representations that only account for the essential phenomena and therefore make the reality understandable and manageable [2, p. 166]. Schlachter states that without “models of abstraction and simplification, [the engineer] would be completely subject to trial and error” [3], and several other engineers, both in academia and in practice, have also highlighted the importance of models for structural engineering [2, pp. 165-166], [4], [5]. Models are used in multiple contexts such as design and validation, and are a vital part of the daily work for most structural engineers. However, as models are often employed in a routine manner, most engineers, whether practicing or researching, view models as tools [6] that are simply applied whenever needed, and do not actively think about their epistemological or conceptual value. Thus, Duddeck has raised the question if the structural engineer’s “use of models is too naïve, non-reflective” [2, p. 181].

In this paper, with respect to recent transformations and new challenges in structural engineering, we seek to reflect on how engineers work with models. We argue that there are two different modes of working with models, and that the appropriate mode depends on whether the problem engineers are facing can be tackled with known methods or requires new approaches, but also on the engineer’s general approach and mind-set towards problem-solving. The first mode of working with models is a creative and inductive thinking in models, whereby models, which translate complex realities into simplified understandable and manageable representations, are developed and applied. The second mode is a deductive model-use, which refers to applying well-established models.

By inductive thinking in models we mean an open approach to a problem that includes the development of models by transforming complex realities into illustrative, simplified and abstract representations [2, p. 166]. Thinking in models enables the engineer to inject ideas, visions, fantasy and intuition into the problem-solving process, which is vital to develop new ideas and innovation [2, p. 162]. Particularly in the conceptual design stages, which define the range of possible solutions for a problem, in research as well as in practice, inductive elements are needed which enable to adjust the design processes and the models used within to fit the problems at hand. The ability to think in models and abstract complex realities is essential for instance for the task of structural design, as it cannot be completed solely with deductive methods, but instead requires fantasy, intuition and sensitivity [2, p. 47]. Hereby, the whole structure as well as the sum of possible effects on it have to be considered in one integral creative design [2, p. 47]. Aspects to be considered are loads acting on the structure, static systems and their mechanical behaviour, the behaviour of the building materials, dimensions and aesthetic appearance. Schlachter states that “the translation of a reality, which up to then existed only in the engineers mind, into the right models to predict the utility, durability, economy and beauty of the structure to be built, is one of the main challenges to the structural engineer, a semi-rational intuitive step” [3].

However, inductive thinking in models is not the adequate mode when it comes to tasks such as calculating, evaluating and verifying structures or proving their feasibility. For this purpose, a closed approach rather than an open one is needed, which means working with readily available models which can be deductively employed following a known sequence of steps. While still requiring knowledge, expertise and diligence, deductive model-use ensures that structural engineers can work productively and economically to finalize designs. A deductive approach is for instance taken on when known solutions are transferred to similar contexts, for instance in the verification of a structure according to previously developed procedures and using well-established models. Hereby, aspects which are not already accounted for in the models cannot be integrated, and the models often do not yield making synthesized judgements, for instance if problems could have been avoided with a different choice of design, or whether certain actions are sensible and reasonable at all [2, p. 189]. Furthermore, a deductive approach can be used in research with respect to further development or refinement of known models or solutions, therefore iteratively improving what is already there. However, it usually only yields incremental developments rather than innovations [2, p. 163].

Thus, both modes of working with models are essential in research as well as in practice, and ideally complement each other in an iterative process. This can be compared with the notions of a divergent, lateral mode of problem-solving on the one hand and a convergent, vertical mode on the other hand (see e.g. [7]). Yet, as conditions for structural engineers change and the profession faces new challenges, the inductive mode of thinking in models needs to gain more attention, as deductive models usually do
not suitably deal with the core of the new and unknown problems. However, as Duddeck stresses, thinking in models is not at all a natural process or not self-evident, but instead requires a huge amount of effort and the ability of abstraction [2, pp. 182-183]. Thus, in the remainder of this article, we focus on this inductive mode of thinking in models. To illustrate the idea behind it, we use two examples of practice of how this general attitude of thinking in models has manifested itself in certain techniques and methods in the past. The two examples—the working methods of Frei Otto and the development and use of strut-and-tie models for the design of structural concrete—initially seem quite divergent: while the first one deals with a general approach to design which includes the experimental development of physical scale models, the second one describes the development and use of a very specific method for the detailed design of concrete structures. However, they were chosen intentionally: they both contain elements of thinking in models and showcase how this mode of working brought about new techniques and methods how to design structures. Furthermore, both examples contain elements of general approaches as well as specific methods, thus demonstrating how thinking in models can be applied on different scales. In each example, the respective mode of working with models or the model itself is described and characterized first. Then, the elements important with respect to thinking in models are highlighted. These are abstracted into more general characteristics of thinking in models in section 4.

3 Thinking in Models: Examples of Practice

3.1 Frei Otto’s Working Methods

Frei Otto is one of the most inspiring personalities in the world of architecture and structural engineering. His work led to a 25-year long period of innovation in the field of light-weight, planar and wide-spanning structures [8]. His original and inventive working practices, approaches and methods have been described in multiple publications, e.g., [9]–[11]. Hereby, three aspects are particularly relevant with respect to thinking in models: his extensive use of physical models, his approach of finding form based on physical techniques, and his integral and collaborative approach to all projects.

3.1.1 Frei Otto’s Use of Physical Models and General Approach to Design

Frei Otto made extensive use of physical models in all of his projects (see Figure 1 left). His daughter describes the eminent role of physical models in Frei Otto’s working practices in the following way [9]: “Our work and life were determined by thinking with models, thinking around models, understanding by models, feeling for models, discovering, researching and finding form with models, testing, verifying, proving and proofing with models, measuring and iterating in models, simulating, calculating and visualizing with models and convincing with models.” The aim of working with physical models was not only to find and define the form of structures, but also to investigate the evolution process of the form [8, p. 111], [12]. For this purpose, Frei Otto invented several techniques to build models, often inspired by building processes in nature [13].

Figure 1: left: Physical Model of the Mechtenberg Bridges, 1997 right: Form-finding soap film model, 1963 (both taken from [9])

This relates to the second aspect important to thinking in models, which is his approach to form-finding. Frei Otto did not believe in simply deciding on forms for his structures but instead aimed at finding geometries that yielded minimal material solutions for his structures to minimize resource input [9].
For this purpose, he made use of different form-finding processes, which were inspired by self-formation processes in nature [13] and based on basic physical principles and forces such as surface tension (see Figure 1 right), adhesive power, magnetic or electrostatic forces, pressure differences, gravity or friction [9].

The third aspect is Frei Otto’s integral and collaborative approach to projects. This manifested for one in an approach to deal with architectural, urban and landscape development problems in an integral way [14, p. 77], considering all relevant and at times conflicting aspects, a process which was strongly aided by his work with models [9]. For another, this meant that he did not expect to solve problems arising in projects on his own—instead, he tried to incorporate experts to work with him early on in his projects following a collaborative approach [15]. In the case of the eco-houses in Berlin, this led to the incorporation of the later residents of the houses into the planning process to find appropriate solutions [9] (see Figure 2 left).

![Collaborative working approach: Planning Meeting for the Eco-Houses in Berlin](image1)

![Castle in the air? Project study “City in the Arctic”](image2)

3.1.2 Frei Otto’s Way of Thinking in Models

With respect to the inductive approach of thinking in models, we can find at least four ways in which it manifests in Frei Otto’s practices. First, what characterizes his work most is the experimental approach. This, however, was not done arbitrarily but can be compared to a fundamental research process, making use of experience, experiment, and mathematics [15]. In each project, he always tried not only to solve the specific problem, but instead to generalise and abstract the situation, generating ideas and visions by thinking in models—an approach that sometimes resulted in general recommendations, design techniques and new empirical findings with often ground-breaking quality [15].

Second, he made use of the slow technique of physical modelling to really understand the projects and the respective problems he was working on, literally translating complex reality into models [16]. In this way, he was able to integrate diverse aspects into one integral model, which is consistent to his integral and collaborative working approach [9]. This was his way of making the design task known to himself, but also to his collaborators, to be able to subsequently solve it [8, p. 111].

Third, thinking in models was also necessary for transferring the self-formation processes of nature into his experimental form-finding models for structures. Inspired by the observation of structures and their evolution in nature, he strived to abstract them to apply their essential mechanisms to the design of the built environment. Hereby, using self-formation processes and openly experimenting led to the discovery of completely new phenomena, which were then analysed and were the source for the development of many of his innovations.

While the first three aspects refer to the practices he carried out, the last one rather describes a more general mind-set or approach, namely that he oftentimes detached himself from the pressure of having to realize each project [15], and instead followed his idealistic preconceptions. Otto himself stated that he built little, but “constructed many castles in the air” [17]. With this mind-set of thinking in models, he was able to leave pre-defined ways and develop original visions. These include for instance adaptable structures, eco-houses, convertible rooftops, the idea of dematerialized architecture or a “City in the Arctic” (see Figure 2 right). By detaching himself from the pressure to realize every idea, he was able to develop and investigate rather radical visions, instead of incrementally improving on and optimizing established ways of working [15].
3.2 Strut-and-Tie Models for the Design of Structural Concrete

3.2.1 Purpose, Development and Use of Strut-and-Tie Models

Strut-and-tie models (Figure 3) were developed for the design and dimensioning of concrete structures in general as well as for the specific cracked state. The basic idea behind these models is first, to understand and visualize the flow of forces in a concrete structure and second, to enable the designing engineer to appropriately dimension the geometry and reinforcement of the concrete structure. To achieve this, the compressive stresses in the concrete are idealized with straight compression struts that form a framework together with the tensile rods of the reinforcement. Based on that, the reinforcement can be easily calculated. The strut-and-tie model builds upon the truss model, which was developed by Ritter in 1899, and later refined mainly by Mörsch and Leonhardt [18]–[20], and explains the internal forces in beams. The strut-and-tie model then is a generalization of the truss model and can be applied to all kinds of concrete structures.

Starting point for the development and advancement of strut-and-tie models in the 1980s was the circumstance, that even though structural concrete was already the predominant material used in construction, concepts for the dimensioning of concrete structures were only available for few simple structural elements. Other, more complex elements were mostly dimensioned on the basis of rules of thumb, which were derived from empirical findings instead of being founded on consistent models that account for the complex behaviour of the material [21], [22]. According to Schlaich and Schäfer, the mere application of these rules of thumb can lead to misinterpretations and even fatal errors, as it suppresses the engineer’s ability to transfer findings to new applications in a creative way [23]. Furthermore, detailing was considered to be entirely based on experience, even to be an art—something which is reserved to experienced and creative engineers and cannot be taught—which led to an underestimation of its role [24]. Hence, the vision was to develop a consistent model for dimensioning structural concrete which was clear and simple in order to avoid rules which are not understood by the designing engineer [22]. The general and reproducible model should be able to capture the behaviour of structures in a consistent and illustrative fashion [21], and thus demonstrate that “detailing is not an art but follows from clearly and gap free describing the local flow of forces in order to materialize it” [24].

![Figure 3](image.png)

Figure 3: Example of a strut-and-tie model based on linear-elastic stress trajectories (taken from [21])

Strut-and-tie models were a breakthrough in the design of concrete structures. First and foremost, they capture and illustrate the flow of forces, and therefore lead to a better understanding of concrete structures [22]. By producing necessary knowledge and understanding, they support the engineer to design good and harmonious structures with appropriate and functional conceptual designs [23]. They can be used to develop structural details, and often provide simple solutions [22], [25]. Additionally, the strut-and-tie models replace extensive calculations and at the same time provide sufficiently accurate results [25]. All in all, the use of strut-and-tie models leads to safer structures, as the better understanding of the behaviour of the structure helps to prevent mistakes, and also as detailing becomes less vulnerable to the subjective influence of the structural engineer’s personal experience or judgement [23]. Furthermore, with these models, it is possible to systematically teach and train detailing of concrete structures [26].
3.2.2 Thinking in Models with Strut-and-Tie Models

With respect to the inductive approach of thinking in models, three aspects should be highlighted. First, designing with strut-and-tie models promotes thinking in models, as it requires the engineer to develop one simple fitting model of the flow of forces which is appropriate to the respective structure. By developing a simple model of internal forces in the structure, the engineer gains an own understanding for the specific structure. Similar to the effect of creative structural design—yet on a different scale—by using strut-and-tie models the engineer is able to direct the forces in an appropriate manner inside of the structure. According to Schlaich and Schäfer, “developing a strut-and-tie model is comparable to the task of choosing an overall static system, both in terms of knowledge and experience and of its relevance and significance for the structure itself” [23].

Second, as there is a clear procedure that can be followed for developing a strut-and-tie model, the model development can be systematically taught and trained. This is essential due to two aspects. According to Schlaich and Schäfer, when developing and using strut-and-tie models regularly, the understanding for the structural behaviour of the material concrete is not limited to the specific structure the strut-and-tie model is generated for [23]. Instead, through practicing this approach, the engineer gains a general understanding of different structural systems and their behaviour. Furthermore, training thinking in models on this small scale encourages thinking in models also on other scales, for instance in the conceptual structural design. Teaching strut-and-tie models and training to develop them thus is a suitable way to incorporate the concept of thinking in models in engineering education [27].

Third, the development of the strut-and-tie models itself can be seen as an act of thinking in models. First of all, the development was triggered by the new problem of how to deal with the new material structural concrete and the lack of consistent models for its behaviour. In consequence, Schlaich developed the vision of a general, reproducible and consistent model for dimensioning structural concrete. As described above, strut-and-tie models were developed as a generalization of the truss model. This required the transfer of known findings to new ground, while abstracting and reducing reality.

4 Characteristics of Thinking in Models

In this section, the elements of thinking in models found in the two examples and described in sections 3.1.2 and 3.2.2, respectively, are jointly analysed. As already mentioned above, the two examples initially seem quite divergent, the first one referring to a more general approach and the second one to a rather specific method. However, a close examination of the examples reveals several common features which are in the following abstracted into general characteristics that serve to further describe the approach of thinking in models.

First, both examples describe the development of a method used in design processes. Hereby, particularly the example of the strut-and-tie model illustrates the path from inductive thinking in models used to develop a model or method to the deductive model-use enabled with the model.

Second, in both examples, the mode of inductive thinking in models is supported by a rather slow and intuitive technique. In the example concerning Frei Otto, this is the slow technique of working and building physical models—in the strut-and-tie example, it is the generation of one’s individual strut-and-tie model. Through these slow and intuitive techniques, the structure or design becomes gradually known to the designing engineer and can be thoroughly understood by them, thus the task of designing becomes accessible to the engineer and subsequently solvable. A central aspect of this process is experimenting. For Frei Otto, this meant experimenting in a literal sense with materials and structures on a small scale. For the development of a strut-and-tie model, this means iteratively generating multiple models in order to eventually find the best one out of multiple possibilities. Hereby, observations are made, reality is conceived, and ideas are developed. This can lead to the discovery of new phenomena, for instance new forms of buildings with planar shapes, or the visual and haptic understanding of the behaviour of structural concrete structures.

Third, both examples also demonstrate how thinking in models can change between different scales. The example of Frei Otto shows how a general attitude of thinking in models can eventually manifest itself in specific working practices, for instance physical modelling techniques. Vice versa, the example of the strut-and-tie models illustrates how thinking in models can be trained on a small scale by applying methodological procedures for a very specific situation, which then subsequently inspires the adoption of a more integral mind-set and attitude towards the design of structures.

In general, the examples showed that thinking in models essentially comprises the thought processes that happen when one tries to develop an own model for a complex reality. This development
and advancement of a new model by inductive thinking in models can be driven by pressing problems such as the environmental impact or the safety of structures. What is more, the examples also show that thinking in models requires a specific mind-set, an open approach to problems instead of following pre-defined rules. For one, particularly Frei Otto’s example illustrates that this mind-set is not tied to or dependent on a certain technology but instead autonomous and therefore timeless in the face of technological changes. For another, the strut-and-tie models are a good example for how this kind of thinking can be transferred in an efficient method and thus can be taught, trained and nourished in engineering education.

5 Concluding Remarks

The engineering profession is regularly affected by changing circumstances, most recently transformations due to climate change and the digitalization. Universities have to empower future engineers adapt to such new circumstances and deal with their challenges in order to achieve the necessary development impulses [28]. Hereby, the generation of innovation goes hand in hand with the understanding and the formation of new models. In this regard, Duddeck states that while “it is impossible to teach future technologies which are yet non-existent,” it could instead be “fruitful to teach general ways of thinking and working, that will also be of value in the future” [2, p. 104]. Therefore, placing more emphasis on the inductive mode of thinking in models in addition to the deductive mode of model-use is of utmost importance with respect to the orientation of engineering education. As thinking in models is a rather abstract concept, two examples of practice were used in this paper to illustrate the approach. The examples are not intended as guidelines of practice. Rather, they serve as illustrations of how the general attitude of thinking in models has manifested itself in certain techniques and methods in the past, and should inspire new generations to think conceptually and develop their own methods and approaches by thinking in models. To achieve this, it is essential that engineers understand and question their general methods and approaches to problem-solving processes and particularly the use of models in this process. In their dissertations, both Lachauer [29] and Ohlbrock [30] provide extensive overviews of modelling techniques used in structural design, which provide fruitful starting points to the development of a general and conceptual understanding of what characterizes engineering models and a more profound conceptual reflection how structural engineers interact with them.

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On sustainable structural design

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Link to the video: https://youtu.be/fqB-hBri5el
Forces behind the scene: concealed structural systems in large-span building structures

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Abstract
This paper describes examples from the authors’ practice to discuss some design concepts and technologies in concrete and steel concrete composite structures that are fruitfully used to achieve large-span spaces with discrete, mostly concealed structural elements. In the first example, the headquarter building of the First Advisory Group in Vaduz (FL), floor-deep diaphragm wall girders with a scattered distribution across the building were used to create spaces with extremely flexible use. In the second example, the “BSCW” school building in Munich, it is shown how free spans of up to 14.1m with significant variable loads due to the presence of heavy machinery, could be achieved through the use of steel-concrete composite beams. Finally, the concluding parts of the proposed paper will be dedicated to a more general description and design considerations of the technology and most suitable application of various types of slim-floor beams in steel-concrete composite structures.

1 Introduction

For structural engineers, it is tempting to regard those building structures as most appealing that prominently feature the load-carrying structure in its visible exterior or interior. In spite of this, not rarely some of the most elegant architectural forms can be achieved when the structure is not easily readable and is concealed behind the “scene” presented by the architectural space. The examples from the authors’ practice and research discussed in this paper achieved large-span spaces with discrete, mostly concealed structural elements using different design concepts and technologies in concrete and steel concrete composite structures. Some key aspects of the employed structural solutions are highlighted in this paper.

2 Belvedere Building in Vaduz (FL)

This office and administration building for the First Advisory Group in Vaduz (FL) was designed for 450 workplaces and includes event halls, an atrium and a theatre workshop [1]. Some key data are presented in Fig. 1. The floor plan area in the basement is 60x100m, while in the upper floors it is 31x85m. The principal conceptual challenges in the structural layout were presented by the maximum ceiling cantilevers of 5.6m, a maximum building cantilever of 18m, ceiling spans of max 15.5m, and a maximum span in the main event hall of 35.5m.
This project is an example of how maximum flexibility and architecturally effectful cantilevers (see Fig. 2) and lightness can be implemented by prestressing slabs and wall-type girders and by attributing smart multiple load-carrying functions to elements traditionally employed only for load transfers in either horizontal or vertical directions.

2.1 Structural system - overview

The load-bearing structure of the building is formed exclusively by vertical and horizontal diaphragms. The building as a whole has a structural dimension and function, where the spatial distribution of walls and ceilings is an indissoluble part of the load-bearing structure. Particularly in the upper floors, all existing wall diaphragms and ceilings are activated in the global structural system and become interactive load-bearing elements with several static functions. On the one hand, the concrete floor slabs take on the normal function of transferring vertical loads to the walls; on the other hand, they also act in combination with the walls as shells, by activating the horizontal shell effect and thus acting as horizontal “flanges” of a multi-layer, building-sized beam system. This structural choice results in corresponding free spaces on the first floor, in the “belvedere” and in the basement, which allow the client to achieve the desired freedom in its intended use and layout over the coming decades. The minimization of the load-bearing structure, made possible by prestressing of ceilings and walls, also means a minimization of the mass and the CO₂ consumption of the building structural system.
2.2 Prestressed slabs

The floor slabs have a uniform thickness of 40cm and were designed as flat slabs in cast-in-place concrete with prestressing with subsequent bonding, with an exemplary tendon layout shown in Fig. 3. This means that - even with the existing spans and cantilevers - no additional floor beams or intermediate supports are required. The ceiling surface and soffit were designed as closed flat surfaces. This allows for a maximum of openness and the greatest possible flexibility in floor plan design and cable layout for the building services. The prestressing of the slabs was also necessary in order to support the architecturally sophisticated glass facade with acceptably low deformations.

Fig. 3: Arrangement of tendons in ceiling above second floor

2.3 Prestressed spatially scattered wall-type girders

Horizontal bracing is ensured by the three continuous concrete cores. On the ground floor, selectively placed additional wall elements form the support for the checkerboard-like wall-like girders, shown in Fig. 4. These represent the key structural feature of this structure which, while largely hidden from view, achieves the very large spans and cantilevers of the building. The wall diaphragms acting as main load-bearing members over all floors have a uniform thickness of 35 cm. Due to layout constraints, they needed to feature many openings, which are placed in a scattered manner above each other. This allowed for the passage of prestressing tendons in narrow corner areas between only minimally overlapping walls. Strut and tie models were used along with extensive and novel finite element models to design the walls and nodes. The detailing of the nodes required great attention in both the design phase and the execution phase on site, as positioning deviations had to be avoided.

Fig. 4: a) simplified strut and tie model for one wall axis, b) full prestressing tendons-layout in shear walls
3 Berufsschule für Farbe und Gestaltung in Munich (DE)

The structural engineering department of the municipal building department is commissioning and supervising the new construction of the vocational schools for colour and design on Carl Wery Strasse (BSCW) in Munich on behalf of the Bavarian State Capital Munich. Approximately 1000 students will be taught at these vocational schools. The key data can be found in Fig 5.

A structure with 5 full floors, a reduced roof floor with terrace and enclosed technical center and 2 basement floors are planned. The dimensions of the above-ground structure are 34.0x88.0m in plan and 24.25m in height. The school is to be erected in a very confined building area between a busy road, a railway line and existing buildings, which resulted in many constraints for the conceptual design in light of the complex space allocation program for the building. The vocational school is intended to enrich the neighbourhood as an identity-forming building and includes, in addition to a 2-fold gymnasium at basement level two, an underground garage at basement level one. The facade is planned as a ventilated metal facade (VHF). The building is designed as a very compact rectangular body and thus makes optimal use of the very confined building site with an overall A/V ratio of 0.156.

3.1 Structural system

In light of the client's decision to relocate the sports hall to the basement levels, a structural system with low dead loads and high span widths had to be selected for a column-free sports hall and further desired column freedom on the ground floor (exhibition hall, painting hall). A steel-concrete composite structure with stiffening reinforced concrete cores and no expansion joints, wall-like beams (exposed concrete walls) and steel concrete composite slabs was selected as the most economical and functionally-structural system. This decision was formed over the course of the preliminary design stage, considering the following boundary conditions:

- Column-free areas on the ground floor (entrance area, painting hall and underground car park below): thus, no load-bearing partition walls were possible on the upper floors (axes C and D non-load-bearing).
- Resulting large spans in the upper floors: 9.3/14.1/9.3m (three-span system): making use of the main load-bearing axes B and E as well as the outer axes
- 2-fold sports hall in basement: required load support above the sports hall over 30m on the ground floor with door openings in the upper floors and required passage solutions in the ground floor: solved by storey-high steel trusses (one steel truss each in axis B and E)

The school building is thus designed as a composite structure in the upper floors, which "sits" on a solid concrete base (basement level one and two). The composite construction offers the following advantages:

- Large spans possible in the upper floors (solid concrete slabs with 14.1m span in comparison would be very uneconomical and not feasible without prestressing or massive beams)
- Strongly reduced dead loads for the load support on the ground floor and accordingly also for the foundation (more than 50% less dead weight of the composite floors compared to solid concrete floors with the same span width)
Design of the steel trusses on the ground floor as composite trusses with slabs connected to the upper and lower chords and possible "upward suspension" of the ceiling above sports hall to the trusses

- High degree of prefabrication in the factory; fast and dry assembly; highly developed steel construction connection technology; high load-bearing capacity with low weight

The 3 stair cores are of solid construction (reinforced concrete walls) and serve for the overall stiffening of the building (see Fig. 6).

Fig. 6: a) 3D-perspective of full structural IFC-model; b) 3D perspective of structural concrete part (not showing the composite slabs)

### 3.2 Long span steel concrete composite floors

The composite floors of the upper levels is formed by a concrete slab composed of semi-prefabricated elements with in-situ cast upper layer that span between continuous steel-concrete composite girders with free lengths of approx. 9.3/14.1/9.3m and run in the transverse direction of the building. In addition, various longitudinal transmission beams are deployed to cover the partially quite large openings for the atriums. A typical layout is shown in Fig. 7. In the two main axes B and E, the composite beams find their intermediate supports on wall-type girders, in recesses in the concrete wall that are subsequently grouted.

The horizontal distance between the beams is approx. 3m, thus allowing for the installation of the semi-prefabricated slabs without assembly supports in the as-built condition; this was a key advantage for the given site logistics and the desired construction times. The prefabricated elements have a thickness of 5cm. The remaining 10cm of slab thickness will be cast in-situ for the total 15cm concrete composite slabs. The steel beams also do not require any support in the as-built condition and are installed with a precamber for deformation reasons. The beams are fireproofed and installed behind a hanging ceiling, which also conceals the extensive HVAC piping needed in the structure.

The maximum structural ceiling depth is approx. 65cm in the center of the building (steel girders + concrete slab), which corresponds to a slenderness of h/L=1/22. Steel beams with smaller depth are provided all around in the two end bays so that the building services piping and electrical parts can be layed out below the composite slabs. In addition to the deformation limits for non-load bearing visible masonry walls, the vibrations of the large-span floors were of decisive importance for the design process, see section 3.4.

Fig. 7: Typical 3-span steel composite floor of upper floors with spans of 9.3/14.1/9.3m
3.3 Steel truss girder and concrete wall-type girders in composite interaction

The spanning of the 2-fold sports hall in the two basement levels of approx. 30m is solved by means of two visible steel truss girders on the ground floor, which are designed integrally with the wall-like exposed concrete girders above. It was a conscious design choice to make use of visible load-carrying elements on the ground floor, as this area also acts as an open space for school activities and lunch breaks. The open layout of the truss permits a “transparent”, yet perceptible separation of three distinct areas of this part of the floor: the central “open space” and the display areas for the painted vehicles and artworks towards the sides of the building. Nevertheless, from a structural point of view, the steel truss forms part of a complex system that takes into account both the final situation and the various building sequences.

Due to its significant stiffness, the steel truss carries a large part of the loads transferred from the load-bearing wall axes from the upper floors. In addition, the ceiling above the sports hall is suspended upwards in the truss. This allows a minimum cross-sectional height to be achieved for the ceiling above the sports hall, which in turn is very advantageous in order to keep the integration into the groundwater in basement level 2 as low as possible. This measure also ensured that the excavation base was above the measured groundwater level, thus eliminating the need for costly water management during construction. The steel truss girder has a very striking effect as a visible structural element on the ground floor and allows the viewer to see a clear load transfer and “read” the structure. At the same time, however, the concrete walls above the truss are also activated as wall-like girders and act, for the final configuration, in conjunction with the truss. Approximately 40% of the total loads above the sports hall are carried by the wall-type beams. In Fig. 8, in addition to the overall load transfer, the activated “compression arch” in the wall-type girder can be seen (see Fig. 8d: Idea Statica plot). By this co-activation of all existing components, it is possible to build the walls in exposed concrete, as the stiffness throughout all construction stages is large enough to control deformations and minimize the resulting cracks.

The load transfer in the two basement levels under the truss is carried out by means of composite columns cast in concrete, which transfer the high normal forces laterally via shear studs into the solid walls, thus avoiding a costly foundation.

Fig. 8: a) Integral truss with wall-type beam from 3D model, b) Main stresses on the overall system, c) Normal forces of the truss alone, d) Load path in the wall-type beam alone

3.4 Special considerations: Fire protection and vibrations

For the steel-concrete composite beams and the steel trusses, special attention must be paid to fire protection and its detailing. The trusses are protected with a reactive fire protection coating for F90, the steel ceiling girders are sheathed with fire protection plates on 3 sides, and the composite columns in the outer axes (designed as concreted steel hollow section columns, partly with steel cores) do not require any further fire protection measure.

The vibrations of the long-span composite slabs were already taken into account in the design phase and were also investigated in more detail in a bachelor thesis at TUM [2]. Only a 3D calculation in-
cluding the membrane effect of the cast-in-place concrete slabs allows for a realistic vibration evaluation of such composite girder grid systems, which can be evaluated e.g. on the basis of design diagrams from [3]. An impairment of the users by ceiling vibrations could thus be excluded for this structure.

![First Eigenmode](image1.png)

**Fig. 9: a) first eigenmode of composite floor, b) third eigenmode of composite floor, c) OS-RMS90 diagram from [3]**

### 4 Slim floor beams in steel concrete composite structures

In this final part of the paper, a more general description of the novel design concept, technology and most suitable application of various types of slim-floor beams in steel-concrete composite structures is presented. There is currently a noticeable push towards a wider application of this technology, made possible by innovations and new product developments. By its very nature, this constructional typology aims at concealing the load-bearing elements, facilitating the use of (partially) pre-cast elements, thus accelerating construction times and allowing for maximum floorplan flexibility.

Some developments that have entered the European product market for structural engineering solutions are shown in Fig. 9, with the relevant references given in the caption. While the traditional slim-floor beam, developed in Scandinavia in the latter half of the 20th century, essentially consisted of a steel beam with protruding lower flange, which served as support for prefabricated concrete elements, but did not otherwise structurally interact with them, newer systems like the ones depicted in Fig. 9 are characterized by a composite action between the steel and the concrete slab (prefabricated or cast in-situ) which can be exploited in design. In all depicted cases, structural steel is used for the beam itself. In the types a) and b), the composite action is created by an interlocking of steel and concrete in specific point or “concrete dowels”, which require transverse rebars to pass through openings in the webs. In the case of the Composite Steel Truss and Concrete system shown in Fig. 9c, the shear transfer is created by a direct interlocking of the concrete into the corner points of the diagonals [7]. Currently, experimental and numerical research work is being completed at ETH Zurich in order to develop a corresponding design model compatible with the Eurocode 4 framework [8].

For the structural designer performing the conceptual layout of a structure, these types of elements provide a number of advantages that may not be achieved by more traditional flooring solutions. These include:

i. the visual effect and architectural layout freedom of a concrete flat slab is achieved while simultaneously retaining significant aspects of the stiffening effect of floor beams;

ii. compared to equivalent solutions obtainable with prestressing tendons, the stiffness is present directly in the construction phase and does not vary over time.

iii. the fire performance of slim-floor beams is much easier to adapt to various project specifications through the use of internal, additional rebars, or a protection of the mere lower flange.
Overall, it is believed that these types of elements will gain further acceptance and use in the coming years, further helping overcome the challenges described in the two exemplary projects shown here.

![Fig. 10: Various slim-floor beam systems with composite action: a) CoSFB system by ArcelorMittal [4] b) Peikko DELTABEAM® system ([5], [6]) c) Tecnostrutture NPS® system [7].](image)

### 5 Summary

This article illustrated, by means of two building projects from the authors’ practice, how large spans, column-free areas and cantilevers can be achieved in concrete and composite construction without making the structural elements invasive or even specifically discernible parts of the structure. The paper showed which ideas underlined these particular structural designs and thereby demonstrated some ways of creating exciting structures despite, or rather because of, not always making the load transfer visible. Through further research and developments, structural engineering is currently further evolving and developing more and more options to give building project developers, architects and owners the choice to either reveal or conceal the “skeletons” at the heart of their developments.

In the authors’ view, the early design period has a decisive influence on the success of the optimal structural concept. Such designs are only possible in a fair cooperation with architects and experienced structural engineers. In addition to the purely technical level, this also requires a great deal of openness, mutual appreciation and communication within the specialist planning team, which ultimately leads to a successful design and an important contribution to building culture.

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The matter of form in structural invisible components: role of foundations

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Abstract

How do hidden structural elements (such as foundations) enable or influence the final appearance of visible elements (form)? This thesis aims to investigate the role of form in its generative relationship with structure in the contemporary Swiss cultural context, focusing on the relationship with the ground. The area where structure and ground meet is a point of comparison, more than ever between exposed or hidden. The aim is to analyze and show how foundations may or may not become part of the final perception of the architectural body, evaluating their formal influence. For this reason the intention is to make models/drawings of some case studies, showing also the foundations, imagining to extract them and lay them on the ground, like roots.

1 Introduction

The structure of the theme starts from a more conceptual dimension, to get closer and closer to the concrete and real aspect. Initially, the theme of foundations is treated as a visible/invisible element in architecture and how they influence the final formal perception of the building. After this first part of introduction, three components that revolve around the theme are analyzed in descending succession, following the force of gravity. The first concerns the line that divides what is visible from what is not (ground connection), and then analyzes the component of the foundations arriving to circumscribe the relationship that these have with the project site (topography).

2 Visible - invisible

Normally the works are published, admired and studied for what emerges from the ground, without paying particular attention or wondering what is hidden underneath. The intention of the research instead is to analyze the relationship with the ground, revealing what is hidden below the ground line, investigating how a hidden structural component can influence the formal component of the visible building. This is because it is evident that the translations of an idea into a built architecture, structurally stable and adequate in its spatial location, finds a decisive moment in how the artifact touches and binds to the earth's crust, taking root. (1)

In the context of the relationship between form and structure, it is necessary to specify that the study of form, linked to the aesthetic components of a project, implies the need for visibility of the components. One does not pose aesthetic or compositional problems on components that no one ever sees. The point of discrimination between what is visible and what is invisible is identified by the line of intersection, (real or imaginary) of the surface of the building with the ground and the “force field that surrounds this line”. (2) Thus, a division into two is generated: a top and a bottom, visible and invisible, what supports and what is supported. A bit like what happens in the city of Eusapia described by Calvino in his famous book “Invisible Cities” (3), where a division is created, in which the visible part is a mirror of the invisible part, until the two components are completely mixed, making the difference and the position of the two unrecognizable. This highlights how on the one hand the invisible components are at least as important as the visible ones and how they perform important tasks silently underground. It
is as if the building were an element that represents the external reality, a fragment of a larger volume, but underneath it, is the presence of an underground world, not visible.

So the foundations become a spy, a "clue paradigm" (4) for research, an element that is commonly not analyzed, but able to highlight very relevant characters in the study and reading of the building. In fact this method has been pointed out by Carl Ginzburg, highlighting several examples of the application of this method in different fields. One of these is that used by the art-historian Giovanni Morelli, in the definition of a new paradigm for the association of works of art to their author. For this Morelli outlined a method that does not start from the most striking and recognizable characters of an artist but, on the contrary, from unusual characters, which are rarely observed (such as the representation of the hands or the ears) because it is there that the true artist is recognized. Similarly, the intention is to analyze the relationship between form and structure in important case studies, where the intention is not to start from the components where this relationship is clear and evident, but from a more hidden condition, such as foundations. Above the foundations, it is necessary to purely carry out a purpose, which is placed on the underground components, in a claim of purpose and visibility. And precisely with visibility comes the question of form, and with such topics as stylistic, historical, ideological and metaphysical implications, the whole complex of symbolism and meaning. (2) "No one ever pays special attention to the beauty of concrete piles in situ of a foundation, but as soon as the piles rise above the ground surface as pilotis, fundamental aesthetic and conceptual controversies arise over them". (2) The area where soil and structure meet is an area of difficult conditions. Underground the building faces the adversities of nature, dust, moisture, rot. Human has been given to live on the crust of the earth's mass. This tacit natural law is transgressed by the desire to establish an almost impossible, as necessary, presence in the very interior of the land. In antiquity, the subsoil represented another sphere that concealed forces and laws different from those that govern the external world. Outside, the daily acts of waking manifested themselves; inside, mystery and magic identified the sacred value of these places. (5) As in the case of the city of Argia, described by Calvino, where an entire city, with all its components (streets, buildings, crossroads) lives underground, facing difficult, humid, cold conditions. No one ever sees anything of this city from the surface, almost questioning its real existence, aware of its mysterious presence, even if invisible. (6)

In this sense are interesting the words of the philosopher Ludwig Wittgenstein, who states "The aspects of things that are most important for us are hidden because of their simplicity and familiarity" (7). With these words it is easy to understand how the components are not often seen, they are also those that are taken for granted, their fundamental existences are known, but nobody never question about their presence.

Although they play a crucial role in construction, foundations disappear beneath the ground, and perform their task in silence. But they are an indispensable element in the construction of the building. Once the construction process is over, they disappear underground and are no longer part of the perception of the finished work, but they are a necessary step, a fundamental and foundational (8) element in erecting what is admired. (9)

With the intention to analyze how a building appears if in its final perception we also include the view of the foundations, it is very interesting the work that Steven Holl does in the chapter "correlation programming" - contained in the volume Parallax. Here Holl shows what he defines as "primary relations" that is the basic relationships that a building establishes with the ground, dividing them into: under ground, in the ground, on the ground, over the ground. We can see how the same volume completely changes its perception according to the different relationships it can establish with the ground. Just think of a simple cube and imagine it completely underground (therefore invisible), partially embedded, simply resting on the ground or suspended on high piles. The architectural volume remains the same, what changes is the relationship with the ground that deeply affects the final perception we have of the project. (10)

This theme introduces the different perception we can have of a building if we imagine it completely underground or extracted from the ground with its foundations. The intention of the thesis is to analyze some case studies in this way, extracting them from the land, with their roots and rethinking them, analyzing "what would happen if" (24) the final perception of the building included the foundations. This type of exercise focuses attention on the ground line and its modification, trying to move it up or down, thus modifying the perception of the building and revealing or hiding certain components from time to time. Just as a spirit level measures the level of water in a tank, here the level of the land around the building is measured, testing its alteration. In this regard, it is important to analyze the design that Pierre Zoelly carries out on the Pantheon (11). Zoelly, in fact, reinterpret the section of the Roman
building, imagining it completely underground, leaving only the large central oculus to emerge. In this way, with its large walls, the building would be completely inside the earth, disappearing from view. But if we rewind the tape, vice versa re-observing it, we can consider it as a fossil extracted from the ground and placed on the public soil of Rome, and for this reason without any side openings with a single central cavity. Thinking about it in this way, it appears completely different, even though it has always been there, in front of our eyes.

Visual and spatial experiences are some of the fundamental elements in the perception of architecture. This makes even more evident the difficulty as much as the fascination in reading the foundations in the project. The eye becomes a kind of architectural position based on a phenomenon of spatial experience that must be reconciled with the concept and its absence of experiential spatiality. But what if there is no visual perception of certain components? (12)

When an observer looks at an object, from being empty, it becomes populated by his gaze. An impression of absence is activated, because now there is a consciousness that perceives it. Munoz Molina expresses this phenomenon very well by saying: "the gaze is a suspended life, a continuous questioning that is pleased with the surface of things and wants to go a little further, deeper, where light and darkness intertwine in the frontier of the penumbra, where knowledge is measured in fractions of a second, flashes of lightning, where what was known is unmasked, where certainty is cloaked in suspicion and the unknown becomes instantly familiar, déjà vu, pure surprise of an unexpected memory." The gaze is, at the same time, a perturbation and a necessity of the perception of vacancy due to absence. (13). In fact, when we observe an architecture, the eye moves, questioning everything it can see until it questions what it is not allowed to see. Underground structures are all around us, yet we hardly notice them, a situation that, depending on the circumstances, we find fascinating, obvious, or even questionable. Because it is invisible, the total or partial lack of knowledge as well as visibility of the underground structure leads to assumptions about the actual conditions. Thus by investigating structures, whose architectural value is found not least in the interaction between the building and its external spaces, new needs for space, for construction are met with "invisible", underground interventions (11)

Giacomo Leopardi in “The Infinite”, describes the importance of an object and its ability to hide the presence of something else, which is not granted to view, but exists. This shifts the dimension from real constraint to the symbolic possibility of the infinite. "Always dear to me was this steep hill, and this hedge, that from so much of the last horizon the view excludes" (14). In this sense Luigi Snozzi recognized a great symbolic value in the foundations, stating that they conceptually reach the center of the earth. So the idea of something that seems not to exist, nobody can see, with an almost mythological value, takes shape, but it is able to support our buildings.

3 Ground connection

“When this touches that, this in on top of that. This is not just "this-on-top-of-that", it enters into a relationship with that, touching makes the relation manifest.” (15)

The ground connection tends to incorporate new concerns in the desire to overcome the traditional notion of topography; it should be noted that the most interesting examples do not draw on a fixed repertoire of solutions already done, but propose innovations in terms of technology, structure and use of materials, giving the different projects new forms.

Intersection is used as an effective tool to reveal and synthesize all the intertwining of site and architecture. This is essential to understand the modalities of the encounter and to trace the materialization back to some basic situation. Tomà Berlanda describes and identifies different solutions where, depending on the circumstances, we find fascinating, obvious, or even questionable. Because it is invisible, the total or partial lack of knowledge as well as visibility of the underground structure leads to assumptions about the actual conditions. Thus by investigating structures, whose architectural value is found not least in the interaction between the building and its external spaces, new needs for space, for construction are met with "invisible", underground interventions (11)

Steven Holl identified the theme as one of the crucial nodes in the relationship between architecture and place. The zero point of that relationship is a section at the surface of the earth. This identifies all
the layers of soil with which the site relates, layers that are more or less dense, strong or weak, imposing different solutions to anchor the building to the ground. It shows how the building relates to the ground: in the ground, on the ground, or above the ground. (16)

4 Foundation

The term foundation, comes from the Latin "fundatio" - fundare, to found. These indicate all parts of a structure that are bound to the ground and whose function is to transmit loads to the soil. (17) Foundation, as a singular noun, refers to building elements that penetrate the ground to achieve a stable layer on which to transmit vertical loads. Foundation also has an origin meaning, marking the value that this component has - it is the first element that is made in the construction of a building. (18) When talking about this component, it is necessary to point out that its conformation is due in part to the architectural idea of the project above (philosophy of structure) and in part to the constraints of the site (type of soil). One has the value of the designed object, as an idea, the other gives the possibility to become a real construction, settling on the earth's crust. Luigi Snozzi gave great value to the conceptual as well as constructive importance of foundations, stating: "every building reaches the center of the earth". (19) In this way, foundations take on a symbolic value, a profound bond with the ground on which they are erected. Thus, the relationship that is structured from the ground line, with the foundations, establishes a deep relationship with all the layers of the earth's crust.

For Snozzi, foundations are capable of communicating the framework of the architectural idea and therefore of being a synthesis of the whole design process, defining how every building begins in its foundations (in Carl Ginzburg concept this become the clues paradigm) (4). Therefore, to understand an architecture it is enough to observe the foundations, because the most beautiful and mysterious project plans are those of the foundations, that for Snozzi is where the whole idea born. An interesting consideration about Luigi Snozzi is the one described by Alvaro Siza, who identifies how the Swiss architect meticulously searches for a desire for change in every single element such as foundations of ancient convents, rows of vines, walls, unevenness of the ground. This highlights the desire to think not with the "point of the pencil" but with "the feet". The ground is explored not as a mere technical operation of collecting quantitative data, but as a process of selection inseparable from the project. (20)

The archaeological role that foundations have should also be emphasized. In fact, these are almost always invisible, under the earth. The few moments in which they are visible coincide with crucial times for the building. Usually they are linked to construction or demolition, then birth or death. In the case of demolition in particular they have an archeological role; as an evidence that emerges from the ground, as a testimony, and tell of what the previous building was. In this concept it is interesting to trace the origin of the word plinth that leads to the concept of earth as the basis of architecture, going beyond the physical nature of the definition, to become a kind of stratified memory of the building. Considering the urban scale, this implies expanding the survey area beyond a single building footprint. (17) Referring in fact to what Snozzi says, the foundations alone are able to completely describe the idea and composition of the missing building, just like the fossils found by an archaeologist. So the foundations become a palimpsest, able to resist to the different stratifications and modifications of time as bones always present in an indissoluble way. If linked to the ancient Vitruvian concept of firmitas, the foundations embody the most resistant component inside a building and for this reason they are able to support it but also to become its testimony. An element that is normally not visible can be the synthesis of a design process, like an invisible or encrypted and therefore hidden code. And of course foundations are the decisive element to understand the ways of touching the ground, identifying the need to carefully read the nature of the project site. (18)

A remarkable example of the relationship between visible and invisible in foundations is the project of the "Two houses - Ponte da Lima", Portugal designed by Eduardo Souto de Moura. Here the foundations in the first house (which follows the slope of the land) surprisingly occupy only a third of the living area of the house; while the second house seems to be suspended, with a small point of support on the ground. However, if we look at the invisible part, we realize how the foundation starts much lower, raising the house and suspending it on a large concrete pedestal (then covered by the ground). If we include the foundation in the perception of the building, it appears completely raised, detached from the ground, as if on a hidden trestle.
Another interesting example in the contrast between the visible and invisible component is given by the project Kanagawa Institute of Technology - Multi Purpose Plaza (Japan, 2008) by Junya Ishigami. Here there is an obvious contrast between the lightness of the very thin roof and the hidden foundation that is huge to balance the lack of pillars inside the large atrium. The main structure was conceived as a floating iron plane supported only by the four walls. The surface bends into a curve with the lack of internal support. In the foundation there are eighty-three piles and fifty-four ground anchors that were imposed on the huge concrete foundation beam, with the ground sloping at a 5m elevation difference. The resulting appearance of the structure thus resembles a suspension bridge.

5 Topography

The term topography literally means the writing of a place, identifying the importance of the ground in the idea of modification and transformation of the architectural project. It is shared the idea that the landscape is not only the background of the architecture, but the very object of the transformation. In other words, each connection on the soil is configured according to the strategy of modification of the place. Hence we can think of different approaches towards the place: building trying to alter as little as possible; or camouflaging within the terrain by building a new artificial topography; or again, detaching from the terrain or entering it completely. (21)

According to Diethelm, the contact between the building and the ground not only determines the transfer of loads, but also the interface with the topography of the place, in a relationship that is not only structural but also compositional. (22) Architecture in fact inscribes itself “in the geo - morphological structure of the landscape through its form”. (2) The relationship with the soil as a sense of belonging and appropriation of a place; not only a technical, geological, morphological aspect, but a real design strategy. (23) “It can appear as an adaptation, supplement or exaggeration, but also as a counterpart and negation of the ground”. (2) It should be specified that architecture comes into contact with the ground through certain specialized elements as for example foundations, plinths, supports, basements, seals, insulation. (2) These technical elements define the difficulty in the construction of relationship with the ground and have the duty to explain how and specifically where the relationship between building and ground is realized. (23)

Particularly in the case of embedding the building within the ground, this is not a mere synonym of underground space, or hypogean building, but a configuration designed so that the earth and the building, sharing a volumetrically defined space, are complementary. Even a building that simply lays on the ground, limiting points of contact to a minimum to a series of discontinuous elements, creates a relationship with the ground. The resulting interstitial space separates and at the same time connects the ground with the building and makes it possible to clearly read the supporting structure.

The real important point in the ground connection is the conflict between the aspiration to reach through the ground with the building and makes it possible to clearly read the supporting structure. The relationship with the soil as a sense of belonging and appropriation of a place; not only a technical, geological, morphological aspect, but a real design strategy. (23) “It can appear as an adaptation, supplement or exaggeration, but also as a counterpart and negation of the ground”. (2)

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The real important point in the ground connection is the conflict between the aspiration to reach through the architectural work the final moment as a process of transformation of a portion of the world, and the awareness of this nature. This is also important in the way we consider topography. When we consider this element, we must consider the whole corollary of parts of its materiality, deducing them from geology, geography, and history. Topography is about what architecture and landscape have in common. It incorporates the built and unbuilt terrain. In the essay "Anchoring" Steven Holl, highlights how the building is intertwined with the experience of a place. (16) The site of a building is more than just an element in its conception; it is its physical and metaphysical foundation. Architecture is an extension, a modification that establishes meanings relative to the place of which it appropriates.

At this point the research moves to understand possible alterations of the topography to show some hide components.

"That of fantastic hypotheses is a very simple technique. Its form is precisely that of the question: what would happen if...” (24) So what would happen if...to a building the ground line changed or if many layers of land disappeared? If the foundation was suddenly completely visible and no longer hidden? If elements were added or removed from a given structure, how would the design react?

This becomes an experimentation, through which an attempt is made to generate new knowledge. The motivation is driven by the intention to investigate the theme of the relationship between form and structure, in the connection to the ground. Besides studying different examples of ground connection, the intention is to evaluate how the same project reacts in the modification of some elements. In this sense is very interesting the competition proposal developed by Junya Ishigami for the recovery of the Polytechnic Museum, Moscow (2012). The project, to create a new level on the ground floor,
reveals and brings to light the underground level of the foundations, making it habitable, usable to the public. The old building is thus raised on the foundations, as in the picture of Semper during the restoration of the Polytechnikum, Zurich (1920). The foundations, previously invisible, in the project become part of the visible form, completely changing the perception and use of space.

6 Context of research

This thesis aims to focus and perimeter the research area within the contemporary Swiss cultural context. The reasons for this choice can be found in some characteristics that distinguish this place. In fact, the Swiss territory is characterized by the presence of difficult orography, which imposes in many projects an accurate study of connection with the ground, always looking for new solutions. This forces architects to think in three dimensions from the very beginning, understanding the different possibilities that the site offers and transforming a constraint into a design cue. (25) Thus it is recognized that the manipulation of the terrain is not only inseparable from the design of the building, but that it has the same importance as all the other components that contribute to defining the project. (22)

This cultural context also identifies a constant search in the project between the ideal as well as the real component, which is even more interesting due to the presence of design sites with difficult conditions, offering a wide range of interesting solutions. This attention to ideal aspects leads to a strong focus on aesthetic values even of technical elements. It can be traced at a historical level, in the construction of bridges, viaducts, tunnels, exhalation shafts - infrastructural elements that become works of architecture, as well as engineering. It should be noted that this aesthetic attention, leads to a study of every element of detail, finding different solutions, craftsmanship for each situation. The materials used are often natural or, in any case, with a great material presence, such as concrete, which reinforce the research around the structural components.

For this kind of analysis and experimentation on the relationship between form and structure, giving great attention to the role of ground connection and foundations, the building of the Leutschenbach School in Zurich, designed by Christian Kerez and Joseph Schwartz, has been selected. (Fig.1) In this building it is particularly interesting how on the one hand the building appears to be suspended, able to touch the ground on tiptoe, in small precise points; on the other hand observing the foundations, these (for various reasons related to the site - soil type and the project - philosophy of structure) are almost as deep as the visible part. In particular on the ground floor the metal structure rests on six central tripods. In this way, the distribution of forces in the subsoil is defined with massive walls in the basement, laid on a foundation with piles. These piles, are a requirement given by the lacustrine and not very dense nature of the construction ground. (26) For this reason, an analysis of extraction or erosion of the land around the building is carried out, making the different levels of the subsoil evident, verifying the change in perception of the building, as parts of it are developed. The next step for this part will be to define some possible design solution to inhabited the foundation part and image, show, design “what would happen if..?” (24)
Fig. 1 Arch. Christian Kerez, Eng. Dr. Joseph Schwartz, Leutschenbach School, Zurich, 2009 – Interpretative drawing. The intention is to show the relationship between visible and invisible, inserting the view of the foundations in a section with also the context surrounding the building. Assuming the underlying soil layers, until marking the resistant layer where the foundations are bound. So the building appears to float in the land.

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[3] Calvino, Italo. 1972. “Le città invisibili”, Torino: Einaudi - "From one year to the next, they say, The Eusapia of the dead is not recognized. And the living, not to be outdone, all that the hooded tell of the news of the dead, they want to do it too. So the Eusapia of the living has taken to copying its underground copy. They say that this is not just now happening: it would actually be the dead who built the Eusapia above in the likeness of their city. They say that in the two twin cities there is no longer any way of knowing which are alive and which are dead"
[6] Calvino, Italo. 1972. “Le città invisibili”, Torino: Einaudi - "Of Argia, from up here, one can't see anything; there are those who say: "It's down there" and one can only believe it; the places are deserted. At night, approaching the ear to the ground, sometimes you hear a door slamming"
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Structuring a common ground: vectors, notes & stories

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Abstract

This article proposes to open up the understanding of the notion of structure in order to widen its role as a crucial actor in the making of architectural and urban narratives. Merging explicit and potential dimensions, material and immaterial realities, this proposal goes beyond the dichotomy between structural exhibition and concealment.

Following Buckminster Fuller’s thoughts [1] inviting us to consider structure as a plural notion going beyond material concerns, the article investigates three levels of definition attached to structure. The first level is the most literal and material one, where structure reads as the built skeleton of the project. The second level focuses on structure as a strategy of (dis)assembly, allowing us to introduce transformation and the notion of time. The third level is an immaterial level fostering an interpretation of structure as a framework for multiple narrative developments. Behind this division, an additional and transversal level appears, situating the potential of structure as an organizational and relational support, active from the design stages to the construction site.

Through a selection of case studies, developed within the laboratory of ALICE at the EPFL, this article investigates the porosities and the mutual implications of such levels. Through a decade of ongoing pedagogical practice, ALICE has explored a wide spectrum of interactions between those levels and introduced the concept of protostructure defined as a structural primitive state. Five cases will serve as the main material to tackle a variety of design trajectories that differ in time and context, driving the construction of different narratives on architectural sustainability.

Introduction

For more than 10 years, the ALICE laboratory at EPFL has been teaching design studio to first-year students in the architecture section. The first year, as imagined by the Professor Dieter Dietz [2], is a real machine: 10 to 16 studios - depending on the year - led by the same number of studio directors participating in a common program, but refining it according to their culture and personal expertises. From this thinking of a common adaptable framework came the desire to stimulate a pedagogical dialogue.

At first anonymous, the matrix proposed to the students is a set of project rules allowing them to share a common scale, directions, and limits and thus to compare, assemble and join. Supported by the research carried out by Agathe Mignon in the context of her doctoral thesis [3], the matrix took on the name of protostructure. Both an imaginary reference system and a physical framework, the protostructure will soon be at the core of several experiments carried out on a 1:1 scale with all the first-year students. The last five projects form the corpus supporting this contribution: House 1 (EPFL Campus, Lausanne, 2016), House 2 (Toni Areal, Zürich, 2017), House 3 (Kanal, Brussels, Belgium, 2018), House Garden 1 (Evian, France, 2019), House Garden 2 (The (Real) Book, Geneva, 2020).

If a chronological evolution is evident with each realization bringing a new milestone of understanding, the purpose of this article is to explore other ways to link their trajectories. We propose to explore these case studies through the prism of three levels of definition attached to structure [3]. The first level is the most literal and material one, structure is read as the built skeleton of the project. The second level focuses on structure as a strategy of (dis)assembly, allowing us to introduce transformation and the notion of time. The third level is an immaterial level fostering an interpretation of structure as a framework for multiple narrative developments. Behind this division, an additional and transversal level appears, situating the potential of structure as an organizational and relational support, active from the design stages to the construction site.
1. **Structure as a Built Frame**

For the first year of the HOUSES pedagogical program, the Alice laboratory team proposes to the first-year students to occupy a timber structure built at scale 1:1.

![Fig. 1 Vectors. Construction site of House 1 (House 1, EPFL Campus, Lausanne, 2016)](image)

1.1. **Vectors: Repeating Structural Frames**

Measuring 11 meters wide by 11 meters long and 11 meters high, the *protostructure* of the House 1 project (EPFL Campus, 2016) has the format of a house and its construction is directly inspired by balloon-framing. This construction system, which appeared at the end of the 18th century in the United States, was based on the concomitant development of the wood industry and the small hardware industry: wood became available in small sections and nails could be manufactured easily and in large quantities. Unlike traditional framing, which works by assembling massive pieces of wood and working between them, the balloon-frame system is based on a logic of frames made up of nailed boards. It thus allows a fast construction which requires little labor and is thus particularly well adapted to the field of the individual housing [5]. The structure set up in the pedagogical context of the HOUSE program uses its codes in a construction made up of a succession of frames assembled by horizontal ledgers. The analogy with this system is pushed to the implementation of uncut sections assembled thanks to a logic of overlapping.

The repetition of the frames at regular intervals and their assembly by ledgers establishes a system of reference by describing an orthogonal reference. At the scale of the structure, all the frames are parallel to each other and the pieces for their assembly are perpendicular to them, creating three axes of reference. At the scale of the frame itself, the logic is reproduced. The joints between the pieces of wood use the geometric properties of the boards and are made at right angles. However, the intervals between the frames and between the assembly ledgers respond to different structural issues and are therefore not equivalent to each other. Without necessarily being orthonormal, the reference system drawn by the structure proposed as a basis of work offers, by the nature of its construction, a series of vectors on which a broad set of projects can be implemented.

1.2. **Nodes: Materializing a Reference System**

The design of the frames and the assembly of the parts between them therefore describes a particular coordinate system, each node of which constitutes a reference point. Unlike a theoretical mathematical reference point, the structure's reference point includes the thickness imposed by the construction. Each

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node is not simply the crossing of vectors but the assembly of several pieces of wood together. It therefore has an orientation, a meaning, that differs according to its location in the structure. Like the vectors described by the assembly of the frames, the network of nodes does not follow a regular system but has its own logic in response to the geometry of the structure.

The first phase of the design studio consists of appropriating the code of this landmark through the tool of drawing. Each of the students reproduces, by hand, the plan and section of the structure in order to grasp its dimensions and spatiality. The structure as it is composed proposes several spaces of different natures, according to the height or the position in the plan, which are thus locatable and characterized by this system of coordinates. In pairs, the students then invest this collective system by grafting their project onto it and developing a program. This phase allows each student to appropriate the code of this structure, which acts here as a support, just as the grid of a sheet of paper would have. Without constraining the nature, the dimensions or the appearance of the projects for which it is the starting point, the structure offers a common ground for experimentation.

1.3. Stories: Balancing Parts and Whole

If the structural character of this construction is ensured thanks to its role as a support for the design brief and thus its power to organize and contain the collective project, the same cannot be said of the statics. The construction of the structure does not have the capacity to carry the additions that the projects represent, it only carries its own weight. During its erection, many parts are added to reinforce the initial design in order to guarantee its stability. Both represented in a 1:5 scale model and built to scale 1:1, the basic structure is unable to support the weight and stresses due to the addition of material. By constituting a part of the whole, each of the interventions must then consider its own load and ensure the balance of the whole. This translates into the need to establish the structural scheme of each project and to confront it not only with the one established for the whole structure, but also with the ones from the other surrounding projects in order to compose a global scheme.

This choice has a great influence on the nature of the projects produced in the first HOUSES program. If they each respond to specific programs and deploy different spatialities, they are linked by their belonging to the collective construction. The same is true for the structure that constitutes the basis of the exercise. If some of the pieces added to ensure its balance are eventually replaced by certain parts of the projects, the structure persists in the midst of the students’ interventions and built of the same wood, it melts into the density of the construction. In the specific context of this exercise, the structure plays with the complexity of its definition to go beyond the role of purely static support. Without cancelling the need to understand and implement a stable construction, it accompanies the development of the project by giving the rules of the game.

2. Structure as a (dis)Assembly Strategy

Confronted with a specific context, the protostructure becomes an instrument for reading and interpreting it. Horizontal and vertical lines are the essential guides of the site, establishing a template whose order, levels, and limits are given by the surrounding elements. From a distinct support object, the protostructure acquires a potential for growth and transformation.

2.1. Vectors: Setting Expansion Capacities

In Zurich (House 2, 2017), the gantry of the first version House 1 (developed as if in vitro on the campus) was duplicated along a line to follow the motorway slip road that hosted it. This expansion movement gives a strong directionality to the whole. The protostructure is no longer a finished object, but a system seeking its limits within the context in which it is deployed. Within it, the projects develop in contact with their environment, adapting to it and responding to it. Like a fishbone, the installation is oriented - with a beginning and an end - but retains its modularity by section, which allowed it to later be dismantled and then partially reassembled (with a slightly different order) on the wasteland of Malley in Lausanne. This segmented reassembly allowed the project to adapt itself to a new context. Named Re-play, this new iteration became the site for new experiments.

Increasing its adaptability, the system will proliferate even more and in a multidirectional way in Brussels (House 3, 2018). Colonizing the ground floor of the showroom of the former Citroën factory, the protostructure found here its limits where it meets the building’s boundaries. It is a three-dimensional
grid in which the proportions of the resulting cell are reduced, increasing the density of the grid. It then became possible to subtract elements, resulting in a hollow core to give more air and height.

![Nodes diagram]

**Fig. 2 Nodes. Constructive catalog of the protostructure junctions through the years (House 1, EPFL campus+ENSA Versailles, 2016-2018; House 2, Zurich+Malley, 2017-2020 ; House 3, Brussels, 2018-2020 ; Houses/Gardens 1, Evian, 2019-ongoing ; Houses/Gardens 2, Geneva, 2020)**

### 2.2. Nodes: Integrating Demounting Strategy

This eminently urban project from Brussels explored the repetitiveness of a very simple knot, using a pinch strategy, which allowed reinforcements and additions to be developed on all sides. We are neighbors and, by immanence, we are transformed by vicinity (even if only visual). Dialogue is then not instituted, it is not obligatory, it is simply there *de facto*, with or without words and formalization.

In this inextricable situation, the *protostructure* is cut, knotted, reknotted, but nevertheless keeps its properties of total (dis)assembly. As for House 1, the knot, like the vectors, is a guide. It supports a vocabulary of the encounter that allows to safeguard one of the visceral potentials of the *protostructure*: the possibility of an elsewhere, of an otherwise...with the same elements. The respect of this common vocabulary also allows for a segmentation of the construction process. The *protostructure* was initially assembled on its own. It then housed the projects of the 11 studios, which were entirely prefabricated in Lausanne and transported by semi-trailer to finally be assembled in barely 10 days of intense activity.

The installation, although completely collapsible, is extremely well integrated into its context. Like its predecessor, it cannot be moved and segmented as it stands. Its potential for reuse belongs to the theme of mutation. Like an exquisite corpse, the support frame allows one to change the order of assembly of the pieces to suit another situation. Moreover, the predominantly bolted assemblies and the clamp of the node, added to its indoor location, make it entirely recoverable. Ironically, due to a lack of foresight and planning, the project was reduced to a pile of matches. Thoughtlessly cut up, the materials were transformed into waste. The sustainable logic of assembly (keeping full lengths of 5 meters joists) only met a theoretical relevance for this project.

We have investigated to understand if the dismantling strategy was too deeply linked to the design process, not allowing any other organization to take over the recycling process. A tender process has been organized by the organism managing the place and several answers has been received from specialists of the reusing question. The method was feasible but, due to a stress linked to the first COVID period, the process has been abandoned. Nevertheless, the dismantling - even disassembling - to the board, threaded rod, washer and nut, as proposed here, takes time [6], requiring arms to carry, eyes and hands to sort. Time and human energy consuming for the first part of the loop, reusing is still generally considered as economically unviable.
2.3. **Stories: Stimulating Unexpected Longevity**

In 2019, the grid extended from the mouth of the Venauge on Lake Geneva to Evian on the other side. There, the buvette completed in 1957 by Jean Prouvé for the Evian thermal baths became a generating element of departure with which the students entered into dialogue. The expansion of the grid, both in plan and in section, also made it possible to spread over the steep slopes of the thermal town towards the high woods and the Grange au Lac (a concert hall designed by Patrick Bouchain for the Evian Resort in 1993). The protostructure is then only partially materialized, but its complete deployment through the drawing allowed the Lake Geneva to be considered as a federating geographical entity and no longer as a border. Changing scale, the grid became territorial. This gesture of transcalarity profoundly transformed the protostructure as a pedagogical and design tool, marking the beginning of a new study cycle for the first year, turned towards the Lake Geneva basin.

Without inherent physicality, the matrix is initially regular and without limits. In this mathematical expansion, the choice of reference levels makes it possible to establish neighboring relationships. Then the encounter with elements of the context such as a tree, gives it edges to follow, an existence. It is stopped, cut, deformed, and thereby gains its thickness. The square, non-oriented cell is first a measuring instrument of the territory, but once materialized, takes on a structural role. The knot tightens, the horizontal and vertical elements intertwine to better support each other, the interaxis sometimes doubles to become directly load-bearing. The protostructure does not “land”, but melts into the ground, leans on the slope, settles down, becomes one with the place. So much so that some projects remain beyond their original timeframe. After two years in place, they are still standing, sound and well. No longer completely ephemeral, they have, by a gentle mutation, passed into the camp of the provisional that seeks to last. This is not “for lack of a better term”, but an opportunity seized, partially stimulated by the imaginary world conveyed by the Grange au Lac and Bouchain’s design attitudes. At ALICE, students and studio directors initiate projects without a program: the main aim of which is to play with the context in order to open it up to visitors with no other goal than, maybe, an impromptu stroll, an unexpected stop, a endless lunch break.

In Evian, the parts with the most fragile structural resolution, will be dismantled before winter 2019. Unbolted, unscrewed, sorted, they were not stored in pieces and will not come back to life elsewhere. The raw material was used the following year to start a new cycle on the banks of Lake Geneva. The spatial experience generated by the projects, so closely linked to the site, could not be carried over elsewhere this time. As for the remaining projects, they are surprising. In particular, their durability and their visitation patterns (weak but existing and benevolent) supported the alternative to maintain them, to follow their evolution.

From this patient observation, a practice of care took root. In spring 2021, a new teaching cycle will take place in Evian, on the site of the remaining projects [7]. A cycle designed on site to understand what becomes of a provisory that has lasted. The site of the installation - a gently sloping wooded plot of land, private but without fences - is gradually taking on the vocation of a public space. After having built the house there, we will try to cultivate the garden. The protostructure, at once a support, a tool, and a guide, still relies on a clear posture. One can then stick to it or transgress it, as it is the case with any set of rules.

3. **Structure as a narrative framework**

3.1. **Vectors: Tracing territorial datum**

In the fall 2021, the ALICE first year program restarted with the will to tackle the Great Geneva area, a cross-border agglomeration counting more than 200 municipalities. To open a dialogue with this vast territory, the shift in the framing already initiated in Evian is strongly reinforced. Instead of thinking of a localized and centralized main intervention, the Great Geneva House-Garden project is envisioned as a network of small interventions precisely sparse across the territory, a vast playground to host the reflections and interventions of the 10 design studios. To support the activation, by dissemination, of this large area, a common narrative is needed. Here, the design of the protostructure plays a crucial role. Its main principles remain open to modifications according to each site specificities: its dimensions, orientation, and subdivision are direct supports of negotiation. In particular, the different levels attached to the horizontal elements of the protostructure are carefully thought through. Each level, following the x and y axis, is not only related to the body and its potential of movement, but is also a response to its immediate surroundings. A specific datum can be created as a response to the local canopy height,
whereas another one can echo the average level of the Rhône river. In the words of the pedagogue Donald A. Schon, the protostructure then actively encourages a “reflective conversation with the materials of the situation” [8]. This conversation connects the student-architect and the site through the protostructure which is understood as a sum of vectors fostering an active dialog with an existing site and its lines of strength and potential.

Hence, the protostructure can help to open a dialog with one’s immediate surroundings, initiating stories of negotiation and installation. On a wider scale, it also helps connect a specific intervention to other sites, through the definition of transversal datum shared among a whole territory. To build a common ground linking projects that are not only physically far away from each other, but are also visually totally disconnected from each other, requires strong narrative lines. The proposal was to trace, using the vectors carried by the protostructure, common territorial datum. Altimetry of structuring site elements across the Great Geneva territory were identified [9], becoming levels of reference attached to the protostructure and forming a latent stratification shared among all the students. Those altimetry lines became immaterial strings connecting different sites, encouraging students to question the characteristics of their own location in relation to the experience of other students working at the other edge of the Great Geneva.

### 3.2. Nodes: Building a Shared Operative Language

The will to activate a large territory by a multiplication of small, precisely located interventions is somewhat reminiscent of the unbuilt project *Magnet* from the architect Cedric Price. This project proposed ten small urban structures to be placed in cities to stimulate public movement. Each structure was designed to make underused sites operate more beneficially by enhancing access and social experience for all. The magnets were understood as temporary catalysts of specific locations, for a specific timeframe. All the envisioned structures dealt with typical urban conditions and relayed on a paradigmatic urban vocabulary (platform, stairways, grand arch, arcade, promenade, pier, etc.); “[the magnets] all reference architectural elements that extend human circulation” [10]. This last point is of importance in the understanding of structure as a narrative framework. A *Magnet* is neither a finished object, nor an isolated statement. Each of the ten structures rather participates in the construction of an open-ended lexicon for intervention, precisely defined in terms of desired effect and intended impact but quite undetermined in terms of form and materiality. Those characteristics allow Price and his associates to think of implementing the Magnet series in very different urban contexts such as London or Tokyo.

Something similar is at play in the ALICE Year One pedagogy, not only for the Great Geneva House-Garden project but in all the iterations previously discussed. As newcomers into the ALICE laboratory, one of the first striking observations concerns the specific lexicon used. In the first place, the use of words and notions - unknown for outsiders - is disruptive. After few months though, the teaching team endorsed it and, more importantly, the use of the lexicon strongly spread among the students. “Protostructure”, “prototrend”, “cell”, “node” … a whole set of notions landed in the studios, now supporting the on-site investigation and the design process. A common new language? Yes, but one still opened to a plurality of exemplifications and interpretations. The notion of “node” gives us a good example. Attached to the DNA of the protostructure, it literally speaks of the constructive junction between the x, y and z orthogonal axis. It also relates to the junction between the protostructure’s vertical elements and the ground. For the launching of the second semester 2021, 4 of the 10 studios [11] ran an experiment around the versatility of the notion of node. In groups of 2, the students were asked to design their own node, at the basis of the cell and therefore of the protostructure development. This constructive choice was not meant to be carried out “in vitro”, it was meant to be linked to their site-specific conditions (such as “feet in the water”, “steep slope”, “dense forest”, etc.). The nodes were built by each pair at scale 1:1 and positioned on site. The variety of the nodes that resulted from this experiment was very important. This large spectrum led to discussions regarding the strength, directionality, and capacity of (dis)assembly of each node. At the end of the day, if we collectively agreed that several scenarios were relevant for a given situation, we also observed that each of those iterations was the bearer of a particular story. The way we choose to anchor to a site already conveys a specific narrative by grounding a structure deeply or not, by grasping existing elements, by supporting others, etc.
From datum to lexicon, from negotiation to constructive imaginary, the notion of structure shows a powerful capacity to support the development of a collective narrative. This capacity was further tested in March 2020, when the period of confinement due to the COVID-19 pandemic shook the entire world and impacted our working spaces and pedagogy. The different sites planned for construction (on the banks of the Arve and Rhône rivers in Geneva) were only visited twice by the students when they suddenly became physically out of reach, accessible only through memories and digital tools. Whereas the possibility of a construction on site was less and less likely, another construct was on its way: a soon-to-be book of 828 pages, collecting the work of the 10 studios [12]. The transition from a built project to a printed book was not hazardous: the protostructure, introduced as a collective framework enhancing dialog and collaboration, probably laid the groundwork for such an alternative response. The structure of the book is in itself telling: rather than chapters, The Real Book consists of islands forming an archipelago of thoughts and projects. Each island (the production of a studio or of a group of cooperating studios) can be looked at autonomously or in discussion with the others thanks to a shared template holding the heterogeneity of the proposals and mediums under few common rules. The table of contents begins with an initiative driven by students from different studios: “A1.1 Transversal narrative”. It consists of a series of short texts, punctuating the whole book, revisiting the last months spent in confinement and questioning the impact this context had on the construction of a common narrative. They observed: “What is certain is that a real difficulty of tuning the different imaginaries intervenes in time of distancing. It nevertheless seems judicious to go beyond this observation and to conceive this diversity as a real energy of the projects” [13]. Instead of seeking to align the interpretations, which would have probably led to a series of compromises and a search for the lowest common denominator, the students agreed that it was appropriate to work with divergent views and histories (including fictional narratives, myths, and speculations). Rather than juxtaposed, those narratives ended up actively cohabitating within The (Real) Book framework, very much like the students’ projects entered into dialog within the material protostructure.
Conclusion

Through the successive pedagogical experiences of the HOUSE 1 program, we had the opportunity to explore different aspects of the notion of structure and the implication it can have in the construction of the architectural narrative.

In the first part, we have seen how the mode of construction of the structure can frame the project exercise by creating a system of reference points. In a second part, we discussed the capacity of the structure to contribute to a strategy of modular assembly, to the expansion of the project, and perhaps to its dismantling and reuse of parts. Finally, in the last part, we introduced the importance of a common extended lexicon around the notion of structure, fueling the architectural project and narrative.

A fourth aspect appears in a transversal way. Beyond the final architectural production, working with a structural grid offers both organizational and relational potential. The catalog of parts and assemblies creates a common working base that allows for adaptation to very different project scales by facilitating logistics. This catalog also makes it possible to regulate the relationships between the various actors in the project by creating a specific vocabulary that is transversal to all the parties.

It is this last aspect that seems to us to be the most important in a pedagogical context. The protostructure that constitutes the basis of the work aims to highlight two major issues in architectural production. It is a tool that allows, through a set of simple rules, to understand and react to very diverse environments with the same rigor and it also constitutes a lexical field that leads to thinking of the project as a collective act.

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Purposeful transgressions: the role of the structure in the making of new space

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Abstract
The present research aims at outlining a conceptual framework for the creation of new spatial experiences in buildings, supported by a conscious utilization of their structural system. The proposed methodology is based on the introduction of purposeful transgressions within default structural solutions: “purposeful” because their critical intention precludes arbitrariness and “transgressions” because they involve unconventional interventions within the building’s load-bearing system. The methodology establishes a conceptual common ground for the collaborative relationship between the architect and the structural engineer by outlining a process of thought through which the structure becomes integral to the creation of new space. A selection of illustrative examples concretizes the principles of the proposed framework through analysis of built projects and by examining the formal characteristics of the spatial experience they materialize.

1 Introduction
1.1 Structure and spatial experience
The creation of a spatial experience is the aspect of building design most often allocated to the “purely architectural”; consequently, it is perceived as separate from the concrete requirements that govern decisions about the building’s structural system. Such division stems from the assumption that the experience of space is a largely subjective matter, and therefore not directly relatable to the quantitative logic pertinent to structural engineering [1]. As a result, the structure is bound to simply reconcile architectural intentions with the force of gravity and its contribution to space creation is limited to becoming exposed or concealed as best fitting to the architectural idea [2]. This efficiency-conducive division of tasks between architects and structural engineers dates back to the 18th century [3] and despite of the persistent promotion of interdisciplinarity, is largely in place today. The traditional hierarchical approach represents a missed opportunity in utilizing the space-defining power of the load-bearing structure [4], which as the only unequivocally permanent building system is always a silent protagonist in determining scale, proportions, room divisions, and relationship to the ground.

The goal of the current study is to outline a design approach that purposefully uses the structure in the process of conceiving and materializing new spatial experiences. The proposed approach draws conclusions from analysis of built works of architecture through the prism of recent neurological discoveries. It affirms that the structure has an inherent potential in the materialization of new spatial experiences due to the universality of the human perception of static equilibrium.

1.2 Universality of space perception and the relevance of new space
The way in which the brain perceives, processes, and records space determines the visceral, cognitive, and behavioural response of humans at the encounter with a spatial construct. The relevance of spatial perception to the field of architecture and construction relies on the answer to one question: Is the experience of space characterized by “universal validity” [5] or is it highly specific to each individual? Meaning, the experience of space can be purposefully created and sensibly explained only if the unity of “the nativistic (inborn) and the empirical (learned through experience) faculties for space perception” [6] are sufficiently similar in all human beings. Proving universality would mean that it is possible to consciously choreograph a spatial experience that universally affects inhabitants’ subjective perception.
The neuroscientific discoveries of the late 20th and the early 21st century have been helpful in identifying a new direction for our understanding of space perception, which points to a high level of perceptive universality among individuals. Not only our brains map and navigate spaces in the exact same way through a universal system of mental coordinates [7]; they also generate spatial maps and record them as a core attribute to memories to use as a source of knowledge at the encounter with spaces in the future [8]. Moreover, the discoveries suggest that perception and conceptualization of space are not processes that run simultaneously. This critically undermines the post-modernist culturally-dominant sensibility, where space is perceived through classifications and symbols.

Within the accumulated experience of space in the contemporary globalized society, humans’ perception of static equilibrium is attained and continuously confirmed. However, there are spaces that betray expectations fostered by past experience. In the context of this research, only such spaces are referred to as “new”. New spaces are constructs that have the ability to viscerally trigger the inhabitant’s ability for conceptualization by being unusual compared to the conventional. The creation of new space in architecture is essentially contained within the ability to materialize “sense-making unusualness” [9]. Its creation should be a central concern in the making of architecture, because space is the most direct and raw influence that buildings exert on human beings. Spaces have the power to affect the plasticity of the human brain [10], are crucial for the ability to record new memories [11], and play a critical role in the formation of learned behaviors.

1.3 Architect and engineer – on the making of new space through structure

The proposed framework outlines a conceptual common ground for the collaborative relationship between architect and structural engineer as it entails an indivisible correlation between the materialization of the architectural idea and the structural system. The creation of space, which purposefully challenges conventional solutions to trigger a perceptive response, warrants a relationship in which “neither the architect, nor the engineer are subordinate to the other” [12]. The structural system intentionally becomes one with the space-defining system.

However, it is important to note that this collaborative process can start only on the basis of a clearly defined architectural idea, which is both “sense-making and form-generative” [13]. This clarification is crucial because the very same architectural idea can be materialized through conventional or through new space. The only difference being that the latter, due to being unusual compared to the conventional, interrupts the “automatization of perception” and when encountered, provokes a degree of experiential “estrangement” [14]. When using the space-defining power of the structure, it is indispensable for the architect to involve the structural engineer from the very beginning of the design process. The role of the structural engineer in this case is not merely that of making the architectural project possible to build, but it is rather to collaborate with the architect to “develop the concept of how to build from the idea” [15].

2 Objective

The ultimate objective of the current paper is to affirm the existence and to concretize the nature of the relationship between the structural system and the spatial experience. The methodology positions the role of structural design in the overall spatial design process, clarifies the essence of the relationship between structure and space, and discusses the potentials for the purposeful use of this relationship. The paper outlines a set of consistent principles, unified in a conceptual framework, through which new space is conceived and materialized in architecture.

Without being formulaic, the methodology proposes conceptual grounding, which precludes arbitrariness in the decision-making process. This shows that the making of new space in contemporary architecture neither is the result of creative genius, nor is dependent on technological advancements or innovative materials. Ultimately, the framework affirms the role of the structure as a protagonist in space making and proposes a possible approach to include it as an integral part of the design process.

3 The “transgressive” approach to the design of new space

Unlike in purely scientific fields, “newness” in the realm of the spatial experience necessitates “little to no invention, much like the reassembling of an existing toolbox” [16]. Such reassembly, however, needs be rooted within the accumulated human experience of space, because neither a spatial constellation that is totally familiar, nor one that is totally bizarre is able to provide us with a meaningful spatial
experience. The first - because it has nothing new to tell us that we have not already experienced, and the second - because it uses totally unfamiliar architectonic language to communicate its ideas (“the merely fantastic soon dies”) [17].

The essence of the proposed methodology is contained in two words: purposeful transgression. Purposeful because its critical intention “precludes arbitrariness” [18] and transgression because it introduces a violation of the defaults within the constituent components of space (i.e. structure, ordering system, scenography, and morphology). The current paper focuses on transgressions within the structural system, described here through the term structural transgressions. The general logic of the transgressive approach with its conceptual steps are outlined in Fig. 1. A project’s starting point is always the idea about space that it strives to materialize. Once this architectural idea is clearly formulated, it becomes necessary to communicate it through coherent architectonic language. Here comes the most crucial moment in space creation, which determines whether the project will be successful in triggering the inhabitant’s ability for conceptualization. Such engagement can be instigated only through a contradiction with defaults, solidified through past experience of space. A contradiction in the context of space creation is more than just an opposition. In that sense, it is not the contradiction of opposites as described by Venturi through the examples of Villa Savoye, which is “simple outside yet complex inside” or in the Tudor plan of Barrington Court, which is “symmetrical yet asymmetrical” [19]. The contradiction of the transgressive approach is rather a purposeful incompatibility with the expected usualness. The method for achieving such contradiction in architecture is through purposeful transgressions.

The achievement of the abovementioned shift of perception aims at questioning the consciously or subconsciously accepted conventional way in which architecture looks, functions, and/or is generally built. The materialized product of the transgressive approach is a new spatial experience. The resulting architectural space is characterized as unusual compared to the conventional (i.e. is new), “builds close rapport with the conventional” [20] (i.e. is relevant) and invites to be adopted as the new conventional (i.e. is resilient).

4 The specific potential of the structure to materialize transgressions

The choice of an effective transgression to express a specific architectural idea is governed by the assessment of the status quo of perception. In a building, structural transgressions usually engage the inhabitants’ perception of static equilibrium. Such engagement can be achieved and intensified through different means. For example, by making the supporting structural elements seem disproportionately small to the mass that they carry, by obscuring, concealing, or dislocating load-bearing elements, or by choosing a material for the supported mass that makes it seem heavier than it actually is.

A simplified example of a structural transgression, which is liberated from the complex nature of building design, is the tensegrity table shown in Fig. 2. In a straightforward manner, it illustrates the power of a structural transgression on human perception. In a conventional table (Fig. 2c) the legs are
elements mainly designed to withstand compressive forces and this influences their size, material and proportions relative to the supported surface. However, in the tensegrity table (Fig. 2a,b), the vertical supporting elements are actually in tension, thus inverting the logic of the structure. One does not need to be an engineer, nor to be familiar with the concept of tensile and compressive forces in order to perceive the unusualness of such a table. The reason for this is the accumulated human knowledge attained through experience in the past. Having seen many tables, a person subconsciously has attained a general understanding of what is the structural role of the table legs, how many of them, and more or less how thick they should be. The abovementioned subconscious knowledge fosters an expectation about space, an expectation about the formal characteristics of the spatial construct known as a table.

Perception functions in the same way at the encounter with constructs of bigger scale and complexity - a room, an installation, a building. In the case when expectations (created and confirmed consistently through past experience) are re-confirmed, they often go unnoticed, a concept known as “automatism of perception” [14]. However, when those expectations are betrayed - the result is a new spatial experience - the ultimate goal of space making.

![Tensegrity table as a simplified example of a structural transgression. Transgressive solution-built (B) and conventional solution – speculative (C).](image)

In building design, despite the higher level of complexity, this transgressive structural thinking follows the exact same logic like in the tensegrity table. Structural transgressions in buildings can include, but are not limited to displacement or concealment of structural elements, switched or unconventional order of structural elements, structural organization which affects the perceived visual inertia, visual unification of tensile and compressive elements, integration of structural and non-structural elements, implementation of hybrid structural systems. The following section features a set of built projects that successfully materialize new spaces through structural transgressions.

5 Illustrative case studies

The current section shows a selection of contemporary buildings analyzed through the prism of the proposed conceptual framework. Each one of the following subsections is dedicated to a specific case study, which exemplifies a particular transgressive decision. This does not imply that there is no possibility for the application of more than one transgressive decision in the same project, nor the fact that all transgressions are equal in their space determinative force. The selected case studies include diverse building typologies coming from different countries, thus showing that the proposed framework is not dependent on context, program, scale, or culture.

It is worth mentioning here that the analysis of the illustrative examples is purely speculative, as it does not imply that transgressive thinking was consciously employed in the design process. The study uses examples only to show what spatial results can be produced through the use of the structure in a transgressive way.
5.1 Displacement and/or concealment of structural elements: Maison Bordeaux

In Maison Bordeaux (Fig. 3), the materialization of the architectural idea necessitates an uninterrupted open space to be created under the floating volume of the dwelling unit. This is achieved through a transgressive solution in which the main volume is partially supported from below and partially hanging from above. The solution relies on a vertical shift of the transversal beam, which is connected to the circular pillar, and on a lateral shift of the pillar itself. This results in the necessity to introduce a load-bearing tensile element to re-equilibrate the global structural system.

This transgressive decision challenges the logical aspect of perception concerning the static equilibrium. The shift and transformation of some of the structural elements challenges the human perception regarding the presence, location, and proportionality of the load-bearing elements to the building mass. The sense of weight is intensified through the secondary decisions for the material of the suspended mass and the minimal window openings in it.

Fig. 3 Example of displacement or concealment of structural elements. Maison à Bordeaux - arch. Rem Koolhaas (OMA), eng. Cecil Balmond.
5.2 Structural organization that affects the perceived visual inertia: Leutschenbach School

Leutschenbach School (Fig. 4) materializes space of uninterrupted continuity in the relationship between the inside and the outside. The transgressive solution consists in the shift of the structural supports on the ground level from the perimeter of the building towards the center of the building volume. The six massive tripod supports represent the one and only direct connection between the building and the ground. As a result, the entire façade surface on the entrance level is not interrupted by any structural element.

This transgressive decision challenges the logic for the anticipated level of visual inertia for a building of that size. It triggers a sense of levitation and unwarranted static equilibrium. Further intensification of the contradiction is achieved through the secondary design decisions for the fully glass-enclosed entrance level (accentuating the lack of structural elements in the façade), the minimal variation in the materiality, and the steel members being painted in the color of the concrete slab.

![Image of Leutschenbach School](image_url)

Fig. 4 Example of structural organization that affects the perceived visual inertia. Leutschenbach School – arch. Christian Kerez, eng. Joseph Schwartz.
5.3 Visual unification of tensile and compressive elements: KAIT Workshop

With the intention of blurring the boundaries between interior and exterior, the KAIT Workshop visually unifies the formal characteristics of the tensile and the compressive elements (Fig. 5). A default solution for a structural system of this type would generally entail the presence of two types of elements: vertical compressive elements (i.e. columns responsible for the transfer of vertical loads) and bracing elements (responsible for the transfer of horizontal loads). Due to their usual materiality and function, such elements are generally easily differentiable. However, this project visually materializes both of them in the same way. The building is supported by a composition of many thin vertical columns in combination with pre-stressed vertical ties. Despite serving a completely different structural function, these elements are visually very similar, thus challenging the perception of stability and active presence of a building of such size. The seemingly absent diversity of structural elements and the slenderness of their presence challenges once again the inhabitant’s instinctive notion of static equilibrium.

Fig. 5 Example of visual unification of tensile and compressive. KAIT Workshop – arch. Junya Ishigami, eng. Yasutaka Konishi.
6 Discussion and future work

The current study is part of a larger research project focused on the development of a conceptual framework for the creation of new spatial experiences in architecture through the introduction of purposeful transgressions. While the overall research project deals with all constituent components of space (structure, ordering system, scenography and morphology), the current paper focuses on structure only. Particularly, the analysis of a set of case studies has shown how structures and structural design can become active participants in the creation of new spatial experiences in buildings. Future work will include the application of the framework to personal projects, the analysis of additional case studies, and an exploration of a possible historical grounding for the proposed design approach.

Acknowledgements

We would like to extend gratitude to Prof. Kengo Kuma and Dr. Toshiki Hirano at the Kengo Kuma Laboratory at the Department of Architecture, Faculty of Engineering, Tokyo University. The research project was funded by the Japanese Government through the MEXT Scholarship for research.

References

To show and integrate instead of hiding – the supporting structure of the Plantahof Auditorium

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Abstract
The construction of the Auditorium for the Plantahof in Landquart was the result of a competition entry by architect Valerio Olgiati. However, without the close cooperation with the structural engineer Patrick Gartmann the lecture hall in fair-faced concrete would not have received the spatial tension and the obvious structural ambivalence with which experts and non-experts perceive the structure.

This project illustrates how each element needs to follow and complement the overall strategy.

1 Perception of the built environment and load transfer
We spend a lot of our time moving around in a built environment – we step out and perceive it. Most of the impressions we take in unconsciously: be it acoustically, visually or haptically. We move with the gravity and understand our environment through our own experiences. Buildings of the same type, which we are used to looking at, no longer catch our attention. Nevertheless, we sense if something feels right or is meant to be irritating. A play, explicitly or unintentionally created by the planner and the executor of the project.

Numerous examples show this. All have one in common: the structure stands independently and does not imitate architectural trends.

2 The Auditorium
2.1 The Intention – an own identity for a lecture hall
The agricultural education and consulting center "Plantahof" in Landquart is the competence center for the south-eastern part of Switzerland. The place comprises a farm, stables, a main building dating back to 1811, boarding school buildings and a gym from the 1970s as well as the Weber lecture hall from 2010.

The project of the lecture hall was selected in an architectural competition.

The auditorium consists of a lecture hall, technical rooms and a connecting structure to the existing buildings. The roof is inclined at 45° and gives the building a striking silhouette.
Fig. 1  Auditorium “Plantahof” facade on the south, on the right hand is the entrance to the main building through it one has to go to enter the auditorium, photo Javier Miguel Verme
We read the construction and try to understand it. Besides meeting the functional requirements the design intends to give the building and its structure an own identity. Every single element seems to be part of the supporting structure, part of the space and part of the construction. One has the feeling that every element needs to be there as otherwise the whole construction would collapse. Everything seems balanced and somehow at equilibrium. Thus, this gives the building its identity.

2.2 The play – to show and interact

In terms of urban planning, the 13-meter-high wall functions as a closure to the outdoor area formed by the existing buildings. Thus, this area becomes the actual square. The choice of the color of the concrete matches with the color of the cobblestones. Like this the wall surface seems perfectly integrated. The window at the bottom left and the angular support protruding from the wall is read graphically. It takes away the dominance of the high surface and even seems to support it.
The visitor enters the auditorium via the main building. A dark high room with several elements opens. One can see and understand that the before seen outer column continuous to the inside and this column primarily bears the roof by means of a supporting beam structure.

At this point the visitor has the first understanding of the connecting elements. Experts ask themselves: Are those elements really necessary? From a static point of view, one could have simply built a box, and the dimensions of the components would have been chosen accordingly. But that is not the goal. A building must offer more than the reduction to the static calculation.
With the support and the beam on the ceiling a spatial static play occurs that also solves the problem of the ventilation of the auditorium in a very elegant manner. The respective ducts were routed into the joist and the column and thus determine the dimensions of these components.

An L-shaped reinforced concrete element is perceived as a reinforcement of the entire structure. Once again, the question can be asked - could it have worked otherwise? Yes - the remaining components would then have had to bear a bigger load from a statical point of view. However, these elements belong to the interior and are responsible for the spatial experience of the viewer, who tries to follow the flow of forces of the building.
Outside two elements are arranged alongside the building; it seems as they need to prevent the roof from sliding away.

The wall foundations, the support and the beam are designed in a way to hold the technical installations in respective media channels. Therefore, the floor slab that is sanded as a shell, can simultaneously form the floor covering.

The architectonic concept is at the same time the concept of the bearing structure. The bearing structure includes at the same time spatial and technical requirements.

All elements that are visible serve various purposes at the same time. However not all of them are visible at first sight or they are not visible at all. Some secrets remain.
2.3 The Construction

The building shows the carful handwriting of the creation. Every joint and every panel was planned and has a certain use. The wall structure corresponds to a double-shell reinforced concrete construction. The inner and outer concrete wall measure 25 cm and 14 cm. The concrete was coloured with black pigments. The reinforcement was designed for crack width limitation with increased requirements. The stages and curing was carefully planned to improve the crack distribution.

Prior to the construction, all execution details were coordinated with the builder. The inner wall was build first and then the outer wall.

Fig. 1 Detail of the crossing between the beam and the column showing the ventilation tube, photo Emanuela Ferrari

Fig. 1 Detail of the inclined roof during construction, photo Emanuela Ferrari
3 Conceptual Design

A conceptual design as a methodology is a usual way for architects to approach the project development.

A building as a whole can only function or even excel in terms of its various components if each individual component is coordinated with one another. So have the individual planner the task to not only work together but to team up and reach a same goal.

As civil engineers we may and must develop a structural design that corresponds to the requirements way beyond the purely static functionality. This is not synonymous to a passive understanding of the role of the other discipline. It is rather an active collaboration to further develop the design to find the best solution together

This project illustrates how each element needs to follow and complement the overall strategy. Only then is the result not simply a stringing together of individual solutions, but an architectural masterpiece with its own identity.
The new retirement home in Giornico (CH)

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Link to the video: https://youtu.be/7iR97q7aYcl
Building for the Elysée and Mudac museums, Lausanne

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Abstract
The design of the new building housing the Elysée and the Mudac museums in Lausanne started back in 2015 after the design team led by architect Manuel Aires Mateus was selected through the architecture competition in connection with the second phase of the “pôle muséal”. The bold architecture concept implied strict cooperation between the architecture and structural design teams seeking to find the best solutions to implement the laureate idea, while bearing in mind the constraints encountered during the design process. Besides providing a general description of the different parts of the building, due to its higher complexity, this paper focuses on the aspects related to the design of the central volume, by presenting the different solutions that were studied at concept design phase, the criteria involved in the selection of the solution for detail design and its adaptations during construction.

1 Introduction
The building integrating the new arts district project (Plateforme 10) in Lausanne that will house the Elysée Museum and the Museum of Design and Contemporary Applied Arts (mudac) is at the final stages of construction.

Above ground, the building comprises a central and an annex volume, the latter being built around the former. Two underground levels are aligned with the central volume. In strict cooperation with the architecture team, fair-faced concrete is used throughout the project: grey concrete for the annex building and white concrete for the central building.

The most remarkable feature of the building is the central cube-shaped volume that emerges from the ground level. The MUDAC, dedicated to design where natural light is welcomed, will be housed by the upper volume of the cube at the first floor. The ELYSÉE, dedicated to photography where light shall be strictly controlled and artificial, is located at the basement. The ground floor is destined for services common to the two museums, like the library, cafeteria and ticket office. The annex building around the central volume will house the offices, workshop areas, deliveries and an auditorium. Additionally, this building extends further beyond the area around the central volume to include the complementary programme, the art info area and a restaurant.

Fig. 1 Full 3D view of the building structure.
This paper describes the structural solutions adopted for the central volume of the building and how they relate to architectural considerations, while also describing alternative solutions that were equated at conceptual design phase.

2 The idea for the building

The architecture concept for the building was well defined right from the start of the project at architecture competition phase back in 2015. The supporting and service facilities should be located in a detached body, which merges with the existing topography that is prolonged into its roof and is vertically cut by its façade. This allows the main central volume, where the two museums will be housed, to stand out at the centre of the site, fully detached from the peripheral building. It resembles a cube cut in two parts that touch only at three interior points, where the ground and ceiling topographies meet, symbolizing the two museums that coexist in one single building. The separation between the two parts of the cube also aims at the extension of the public space from the railway station through the recently built MCBA Museum (“Musée Cantonal des Beaux-Arts”) into a very transparent interior, where all public activities shall be concentrated. The four upper volume façades present a trapezoidal shape and are materially continuous with the lower ceiling, which is composed of several faceted panels with different slopes.

The east extension of the peripheral building for the complementary program also merges with the existing environment by replicating the existing stone arcades using a different, but also visually strong, material: concrete. In fact, material uniformity was pursued by the architects by favouring the use of apparent concrete throughout the project for all public spaces.

Fig. 2 Renders of the central volume from Aires Mateus for the architecture competition in 2015: general view of the cube (left) and interior view of the foyer (right).

3 MUDAC – Upper volume of the cube

Given the complex nature of this project and the impact the structure has on architecture and vice-versa, the strict cooperation between both disciplines started right at the competition phase, even before the project was formally initiated. Obviously, the main focus was the upper part of the cube.

The first relevant conceptual decision to be taken was the number of supports for the 43,8x43,8m cube-shaped elevated volume, which should be reduced to a minimum and placed at the interior of its square shape to achieve the intended visual separation from the lower part, while also assuring the necessary conditions for the stability and accessibility of the upper part. Three support points were proposed for the structure, positioned next to the façades, but slightly recessed in order not to expose them from the exterior, and equally spaced between each other as much as possible, so that the vertical load is similar at each one. This is also the minimum number of supports in a three-dimensional structure to avoid bending moments at the supports due to unbalanced loads. They were materialized as three rectangular shaped cores with minimum free distances between them of 23.5m 24.3m and 22.2m, which were also proven sufficient to incorporate the required staircases, elevators and ducts.

The choice of the structural solution and composition for the façades was probably the aspect that involved more reasoning from engineers and architects at conceptual design. Different solutions were discussed and tested as presented in the conceptual sketches of Fig. 3.
The preferred solution for the architects has always been the use of fair faced white concrete for the central cube. Three different solutions using “in-situ” fair faced white concrete with different exterior finishing were considered: a) no visible construction/formwork joints and no plug holes due to the use of form ties for formwork stability; (b) no visible construction/formwork joints but with plug holes due to form ties used for formwork stability; (c) visible construction/formwork joints and with plug holes due to form ties used for formwork stability. Each specific requirement has a relevant impact on the construction methodologies to be used:

- The absence of construction joints leads to one single pour per façade. Given the length and height (12.7m maximum) of each wall, an adequate vibration of the concrete at the bottom of the formwork is not plausible, suggesting the use of self-compacting concrete.
- The absence of visible formwork joints with the purpose of obtaining a smooth and uniform finishing requires lining the fully assembled exterior formwork surface with a continuous coating. A solution using a linoleum coating with welded invisible joints was studied. Its application involves assembling the formwork offset from its final position to allow enough space for the lining procedure. The full assembly must be ripped to its final position through jacking, before the pouring operation.
- The absence of plug holes requires the replacement of the typical form ties that self-restrain the high wet concrete pressures by a massive formwork retention structure capable of absorbing the horizontal loads at each side of the wall.

The requirements above were presented in order of increasing complexity. While a single pour per façade is attainable with the use of self-compacting concrete and a carefully prepared concreting operation, the lining of the formwork surface with a continuous linoleum sheet requires free space around the façades for the assembly and coating of the formwork outside its final position and a jacking operation which is not common in building construction. Finally, the elimination of plug holes due to the use form ties is clearly the most demanding requirement, as a dedicated steel retaining structure needs to be specifically designed for this purpose, considering high horizontal loads and strict deformation limits to assure an adequate performance of the formwork system and the required high quality surface finish for the walls.

Two additional requirements were given special attention when considering the solutions involving fair faced concrete for the façade: obtaining a high-quality thermal envelope by avoiding thermal bridges and the strict control of cracking which is deemed important to protect the concrete against freeze-thaw cycles. To address these requirements a double wall solution was envisaged: an interior non-apparent RC wall that provides the required resistance and stiffness for the peripheral distribution of forces and an exterior apparent wall that is suspended at the top by the inner wall using sliding bearings. This allows the wall to freely deform in its plane with the purpose to minimize longitudinal tensile stresses and thus, to minimize the risk of cracking due to concrete shrinkage and temperature gradients. The thermal insulation is placed continuously between the two walls avoiding any kind of thermal bridges, while also serving as formwork for the inner face of the exterior wall.

Besides the solutions using fair faced concrete, lightweight solutions were also equated, with the façade’s structure being composed by steel trusses. Two different options for the exterior finishing.

Fig. 3 Sketches of the different solutions equated at conceptual design for the façade composition.
were considered: (d) precast glass fibre reinforced concrete (GFRC) panels attached to the interior steel structure with thermal insulation in between; e) cement boards (AQUAPANEL) or external thermal insulation composite system (ETICS), both with painted smooth finish and with incorporated thermal insulation, attached to a secondary steel structure. While the first solution implies accepting the visual impact of the joints between the precast panels, the latter allows obtaining a fully continuous and smooth surface. The construction techniques for both solutions are much simpler than the solutions involving reinforced concrete. However, from an architectural perspective, much of the intended character for the cube is lost with these solutions: the expressiveness of the joints between GFRC panels overshadows the idea of one big cube cut in two pieces, while the very smooth surface obtained with the cement boards takes away much of the visual force and roughness that is obtained with the concrete solutions. Also structurally, the solutions with RC façade walls and suspended ceiling were proven much stiffer than the solutions with the steel structure coated externally with a non-structural finishing.

From the five initially considered solutions, two were retained for further studies, both structurally and architecturally:

- Solution “b” with fair faced concrete façades and faceted ceiling slabs having no visible construction/formwork joints, but constructed using traditional formwork systems with form ties; the structural system is based on continuous and monolithic RC elements (Fig. 4 (left)).
- Solution “e” with a continuous and smooth façade and faceted ceiling panels formed by cement boards or ETICS supported on a steel structure, mainly composed be trusses (Fig. 4 (right)).

Fig. 4 Architecture section and structure 3D view with concrete solution “b” (left) and lightweight solution “e” (right).

The main advantages and disadvantages associated with the concrete and lightweight solutions are summarized in Table 1.
Table 1  Comparison between concrete and lightweight solutions.

<table>
<thead>
<tr>
<th>Solution</th>
<th>Advantages</th>
<th>Disadvantages</th>
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| Concrete  | ▪ Aesthetics  
▪ Better integration with the lower volume, where some RC elements are kept  
▪ Higher stiffness  
▪ Better distribution of vertical and horizontal loads  
▪ Fire protection | ▪ Long duration shoring required  
▪ Reinforcement complexity for the faceted slabs |
| Lightweight | ▪ Simpler construction techniques  
▪ No long duration shoring required | ▪ Lower stiffness  
▪ Higher contribution from the roof structure required for global stabilization  
▪ Secondary steel structures required inside the technical false ceiling |

Even though being more complex from a construction point of view, the important advantages associated with the concrete solution and its higher identification with the architectural idea for the building played a major role in the selection of this solution at the end of conceptual design phase. The main structural elements supporting the elevated part of the cube are the inner RC walls, which distribute the peripheral loads to the locations where the floor structure is more rigid, i.e. near the supports materialized by the three RC core walls, and the floor structure itself. It is composed by the lower ceiling RC faceted slabs attached to the first floor slab above by Warren type steel trusses, allowing these elements to work together as a more rigid composite structure. The steel trusses are supported at the corners of the core walls by pot bearings. The concave down shape of the faceted slabs also allows these to transfer a portion of the vertical loads to the core walls by arch effect. The RC elements in contact with the exterior (façades and ceiling slab outside the glazed façade at the foyer level) are doubled with thermal insulation in between. The exterior façade walls are suspended at the top through corbels with free sliding bearings and shear connectors that are fixed in the transverse direction throughout the full perimeter, but are blocked longitudinally only at the centre parts of the walls. Additionally the outer walls are attached to the inner walls by articulated turnbuckles that are suited to take both tension and compression loads. The purpose of this system is to allow a continuous thermal insulation envelope, to preserve concrete as the only apparent material (from outside and from the foyer) and to minimize the cracking risk of the exterior apparent concrete elements.

Besides the typical structural plans ad details required for tender, a detailed description of the intended construction sequence and techniques was also deemed fundamental. The most relevant requirement for the construction sequence is to keep the shoring of the interior peripheral walls and faceted ceiling slab until completion of the roof steel structure, because the structure is stable only from this phase on (Fig. 5 (left)). For the assembly of the façade walls formwork and it’s lining with continuous linoleum, a temporary steel structure is proposed to be assembled alongside the building and scaffolding is put in place between the temporary structure and the building, making it possible to assemble the formwork panels and to coat them with a 4mm thick linoleum film, providing a uniform surface without joints as intended by the architects (Fig. 5 (right)). This film is glued to the formwork panels and the joints are welded and smoothed so as to be imperceptible. After the reinforcement, connecting rods and thermal insulation being assembled against the interior wall, the formwork panels are ripped on rails by means of hydraulic jacking to their final position. One pouring operation per façade with self-compacting concrete is envisaged, using either a chute from the top or pumping bottom up from a valve.
Fig. 5 Shored structure until completion of roof structure (left) and apparent façade wall construction with formwork assembly offset from final position (right).

The structure was successfully tendered, with the main competitors validating the proposed solutions and presenting preliminary drawings for the temporary steel structure with the jacking system and for the faceted slabs formwork, including the linoleum coating.

With the purpose to reduce the risk associated with the complexity of the construction techniques and to favour the construction schedule, during construction, the design team searched for simplifications of the defined methods and solutions, while always keeping in mind the requirements to maintain the initial architecture concept. The following changes to the initial solution were introduced:

- The double wall solution with insulation in-between was abandoned; only one wall at the position of the previous exterior wall was considered, which has both a structural and aesthetic function.
- All thermal bridges arising from the elimination of the double wall solution have been analysed by the building envelope specialist and corrected, as much as possible, from the inside.
- The linoleum coating was eliminated and the position of the formwork joints was carefully defined by the architects, considering 4.0x2.0m formwork panels; as a consequence, the temporary steel structure for the assembly of the formwork alongside the building is no longer required.
- A light prestress in the form of 4-strand flat ducts was added to the façade walls in order to minimize the increased risk of cracking arising from the higher restraint for deformation of the walls along its plane.
- The pot bearings for the support of the floor steel structure are eliminated to facilitate maintenance; instead, the trusses in the alignments of the supports are embedded inside the core walls.
- Removal of the shoring was anticipated by removing all the interior shores after the faceted slab, composite slab at first floor and floor steel structure were built; only the peripheral shoring was left until completion of the façades and roof steel structure.

A comparison between the initial and the as-built façade solutions is presented in Fig. 6.
A surveillance campaign has been put in place to evaluate the real vertical deformations of the elevated structure and to compare them with the expected theoretical values. Even though it is known that measured deformations in reinforced concrete structure may deviate substantially from the theoretical values, this comparison was considered relevant to assess the actual behaviour of this complex structure.

Six survey points were selected: one point at each façade corner and two inner points at the suspended ceiling. Five survey instants were defined:

- Survey 0: after completion of the structure and before starting the removal of the peripheral shoring
- Survey 1: after removal of the peripheral shoring at the north-west corner
- Survey 2: after removal of the peripheral shoring at the south-west corner
- Survey 3: after complete removal of the peripheral shoring
- Survey 4: 2 months after complete removal of the peripheral shoring

The total measured deformations after the complete removal of the peripheral shoring (survey 3) and 2 months after that instant (survey 4) together with the theoretical deformations for the same
loading scenarios – self-weight only and no long term effects for survey 3, and total dead loads and partial long-term effects for survey 4 – are presented in Fig. 8. Finally, the maximum theoretical serviceability deformations (quasi-permanent combination and long-term effects) are included for information.

![Fig. 8 Measured and theoretical vertical deformations.](image)

It is observed that the real deformations after removing the peripheral shoring are in line with the theoretical estimated deformations for some of the survey points and are lower for others. The point with maximum deformation has a 5mm actual deformation, while the expected theoretical deformation is 5.6mm. The deformations two months after the shoring removal are still in line with the theoretical deformations, with the maximum measured deformation being 7mm and the corresponding theoretical estimate being 7.8mm.

It should be pointed out that long term deformations have been estimated using a creep factor of \( \varphi(\infty,t_0) = 2.0 \). However, for comparison with the survey after two months of the shoring removal a lower creep factor must be considered. Besides 2 months not being enough to develop full creep, two other factors were observed during construction that are considered to have an effect on the creep factor: concrete resistance was proven higher than prescribed (C40/50 instead of C35/45) and the higher than usual age of concrete at the time of loading (around 120 days) due to the extended period the structure was shored. By applying the formulation in EN1992-1-1 [1], the creep coefficient at time of survey 4 is estimated at \( \varphi(t,t_0) = 0.58 \).

4 Final remarks

By analysing the as-built structure, both aesthetically and structurally, it is concluded that the strict cooperation between the architectural and structural design teams throughout the different design phases and during construction allowed to effectively materialize the initial idea for the building, while still adapting the technical solutions to the encountered constraints. Visually, concrete is the only apparent material, as initially intended. It was possible to introduce the formwork joints to the façade walls without offsetting the visual impact of the big cube cut in two parts, allowing however an important simplification to the construction techniques for the execution of the façades. Besides the modifications that were introduced throughout the project and the challenge that the reduced number of supports and complex geometry posed to structural engineers, a high structural performance was always pursued. The increased risk of cracking due to abandoning the double wall solution was effectively solved with the introduction of prestress. The deformations of the structure were kept within strict limits, with the theoretical values being confirmed through on-site survey. Special consideration was also given to the reduction of maintenance operations for the structural components by replacing the support solution for the floor steel structure through pot bearings by embedment of the steel trusses in the core walls.

References

The structural design of buildings as an articulation of bio-climatic principles and energy savings
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Abstract
This article details the multidisciplinary approach developed with the Renzo Piano Building Workshop office for the bioclimatic design of the new Ecole Normale Supérieure de Paris-Saclay located in the ZAC* du Moulon (*a joint development zone) on the Plateau de Saclay (91), France. The structure is located at the link between all the engineering disciplines involved and plays a key role in the development of this conceptual ambition, particularly around natural ventilation. The different stages of the process are presented below from the genesis and development of the concept, detailing the test phases carried out in design through to the execution phases by the companies.

1 Energy ambitions to match the excellence carried by ENS Paris-Saclay
The new Ecole Normale Supérieure Paris-Saclay (ENSPS) is a higher education establishment of excellence in Science and Technology characterized by a strong interdisciplinarity affecting the Fundamental sciences, Engineering, Human and Social sciences and Design. It has twelve teaching departments and sixteen research laboratories. To accommodate all of these entities, several blocks of buildings of 3 or 4 floors on a ground floor in double height of 8m, a performance hall "la Fabrica" (building n°8) and an amphitheater a 500-seat (building n°7), representing around 65 000 m², are arranged around an interior garden, called "jardin extraordinaire", in order to serve as a backdrop and protect it from the very windy environment of the Plateau de Saclay.
The energy objectives to reach were highly ambitious (regarding to the French regulations in force at the time the project was designed) and relate to:

- $\text{Cep project} < \text{Cep max RT 2012 -30\%}$
- $\text{Bbio} < \text{Biomax}$
- Renewable energy rate heat production $> 50\%$
- $\text{CO2 rejection heat production} < 100\text{g/kWh}$

To meet these objectives, the conceptual principles retained were as follows:

1- A bioclimatic design of buildings to reduce energy needs at source and promote passive pathways to ensure summer comfort
2- The privileged use of the energies contained in the air, the earth, the water and the sun
3- The implementation of efficient systems
4- The use of renewable energies
5- Performance monitoring

At first, it may seem that only the devices and systems relating to the MEP specialties are the main levers of these performances and that the others such as the structure play an auxiliary role in this process. However, this is not the case and this performance is on the contrary achieved and amplified by the involvement and contribution of all the devices and techniques from all the trades involved in the act of constructing a building. It is this state of mind that has guided all stakeholders throughout the various stages that have marked the completion of this project.

2 Energy efficient buildings that promote passive modes

As explained previously, the chosen conceptual approach aims to capture as much as possible the energies present in the air, the earth, the water and the sun. The Plateau de Saclay being particularly windy, it was therefore essential to be able to exploit this natural and strong characteristic for the project. This operation results in the installation of natural ventilation for, on the one hand, the atrium of building n°1 and, on the other hand, for the passive comfort of all the buildings of the project (excluding the performance hall and amphitheater).

2.1 Atrium of Building n°1

Building n°1 of the project is organized in the northern part with blocks of buildings facing each other separated by a central common space forming an atrium covered with a glass roof. This atrium is characterized by a very large volume whose energy treatment and summer comfort are based on natural ventilation.
Four low pressure draft stacks are thus distributed over its length; they allow the operation of "lung" whose operation switches to natural ventilation as soon as the outside temperature is above 15°C. A set of modeling and wind tunnel tests made it possible to design the height and all the characteristics of these chimneys and to fine-tune their control through the Technical Building Management in connection with the glass roof and its motorized concealment and the openings arranged in the upper and lower part of the atrium allowing air inlets.

Fig. 3        Wind tunnel tests and Modeling (left); view from inside the finalized atrium (right)

Fig. 4        Longitudinal section on the atrium

2.2 Structure designed to be an articulation of bio-climatic principles and energy savings

2.2.1 Vertical structures and bracing

The primary structure designed for the vertical elements necessary for the architectural, structural and bio-climatic challenges is simple. It is composed of south-north oriented concrete walls cast in place and left unworked as wished by the architects. These walls incorporate an insulator implemented during the formwork of the wall allowing the structural part and the cladding to be cast at the same time. The polyurethane insulation (lambda = 0.022W/m².K-1) is held in place by a patented dagger system (GBE system) thus making it possible to obtain a wall isolated from the outside with a raw facing both inside
the building and outside as a facade cladding. These sails have been the subject of precise research on the production methodology and specific formulation suitable for concrete. All the facings, both indoors and outdoors, have received a Hydroxi2000 water repellent treatment from Pieri. This solution made it possible to combine architectural ambition and thermal performance of the envelope.

Fig. 5  Schematic diagrams of the GBE system for vertical walls with exposed architectural concrete, cast in place and insulated from the outside

Fig. 6  Photos of the different stages of in-situ realization of walls made with GBE System
The primary structure in the east-west direction is made up of column-beam type elements on a 5.40m grid forming self-stabilizing frames over the entire height of the building. The bracing system, provided with the contribution of the floors as a diaphragm, is overall hybrid, but it should be remembered that the project is not subject to seismic loads.

Fig. 7 Layout diagrams of the vertical structures of the project building (left) and simplified structural axonometry of a standard block with wall and frames (right)

2.2.2 Design of floor structures

The program established by the client required the establishment of flexible and reconfigurable spaces over time in order to develop the spaces according to the needs of the school. In addition, the presence of an infrastructure car park needed to avoid in the upper level any intermediate support that would have required to create ‘recovery structures’ that still consume a great deal of structural height and are expensive. The principle adopted was then to design free platforms for the floors of all buildings with spans ranging from 10m to 16.5m depending on the block.

This type of span requires sufficient structural rigidity to aim the necessary resistance and also to sustain the frequency and vibration behaviors adapted to the comfort of users and to the level required by the research equipment set in the laboratories. From a mechanical point of view, to obtain this structural performance, it is necessary to distribute the material around the center of gravity of the section, in the manner of an I. By juxtaposing two I, we obtain a very resistant closed box with matter distributed all around a void. Rather than losing this vacuum, it was decided to use it as a channel for the needs of natural and mechanical ventilation. Indeed, the multitude of channels created allows ideal irrigation, and it then appeared possible to blow through them like a pan flute.

To shape these coffered floors, it was decided to use prefabricated prestressed concrete elements in the shape of a double U of the same width as the facade grid, namely 1.35m. The upper slab is poured on site on a permanent cement formwork installed at the head of each of the webs, thus making it possible to create the concrete slab for each floor as it progresses.

It should be noted that another family of elements has been designed for the floors, this time open in the shape of a Pi so that the flows remain in conventional ducts housed between the drop beams. These structural elements are intended in particular for laboratories using specific air treatments and subject to health control requirements that cannot flow directly into the boxes, or that require direct access to the roof. These Pi-shaped beams are also used for the floors of the first level of the entire project.
2.2.3 Functioning of coffered floors

As a reminder, in accordance with the program, the offices and teaching rooms are not air-conditioned. The design of the building should nevertheless make it possible to make the necessary arrangements to reduce the number of hours of discomfort to a minimum while ensuring energy savings. The first principle retained for this objective in summer comfort is to take advantage of the thermal inertia of concrete by capturing energy during the day and rejecting it at night.

The structure/ventilation hybridization was then also fully exploited to evacuate energy at a lower cost through natural ventilation passing through the floors. Thus, in parallel with the studies carried out on the natural ventilation of the atrium, specific studies have been developed to demonstrate the interest of using structural floors as ducts dedicated to this natural ventilation and the energy balance obtained by the installation of these principles. Indeed, the concrete being left exposed, it is then possible to exploit its full potential for thermal inertia. Thus, during the summer, the concrete gradually loads over the course of the day with the various heat sources external and internal to the building. This stored energy is released in particular during the night by the installation of free-cooling by natural ventilation circulating from the facades through the floor channels to the assisted natural draft chimneys. To allow this air path, the structure and the facade also had to be designed together in order to fully allow the desired hybridization. Thus, the exterior joinery elements are equipped with motorized openings slaved to the Building Management System (BMS) to the weather station. The concrete edge L-shaped beams are all provided with cavities (square against the facade and circular on the "return" side in the atrium).

This natural ventilation works by setting up a bypass system at the main air handling units. A very low consumption assistance fan has been installed in the chimneys to initiate the low-pressure phenomenon.
in the event of a slightly low wind speed, its energy consumption is negligible 0.1W/m³/h, and this is why the system is called "assisted natural ventilation". The principles of these chimneys are the result of research carried out for the atrium chimneys and were developed through wind tunnel tests.

The results of the studies carried out have shown that assisted natural ventilation provides:

- a reduction in the electrical consumption of auxiliaries by a factor of 4 compared to mechanical double-flow ventilation
- a real passive summer comfort by night unloading and increased flow rates
- an improvement in acoustic comfort, also obtained by the installation of sound traps.

Three operating modes were therefore retained, all operating on the principles of the implementation of this hybridization between structure and ventilation: Summer mode, Winter mode, Unoccupied mode

- Winter mode: double flow mechanical ventilation

  In winter, when the outside temperature is below 17°C, an automatic system for slaving the uprights triggers the closing of the facade valves. The fresh air is mechanically forced into the box beams which irrigate each room through independent cells; this fresh air is preheated free of charge by high efficiency energy recovery (recovery of the energy released by the occupants, lighting, computer equipment) on the extracted air collected by the 710mm diameter duct visible in the Atrium, connected to the double flow air handling unit which is finally rejected on the front of the technical rooms of the roof terrace.

- Summer mode (Outdoor temperature >15°C to 18°C): assisted natural ventilation

  In summer, when the outside temperature exceeds 17°C, an automatic control system triggers the opening of the front flaps. The fresh air is guided into the cells by a specially designed nozzle installed on the front. An insect and bird barrier screen is installed at the entrance to the box: a screen that can be cleaned and removed from the outer pathways.

- Unoccupied mode: natural ventilation

  During summer holiday, at night, if the outside temperature is lower than the building temperature, the doors are kept open. This makes it possible to unload the building of the heat accumulated during the day and to load the construction elements having a significant thermal inertia (beams, floor, structure…) in night freshness.

Fig. 10 Illustration of the operation of ventilation in winter mode (left); and of summer operation with ‘assisted natural ventilation’ (right)
Fig 11 Diagram of a typical distribution inside the floor: blue channels are the ones dedicated to fresh air (both mechanical and natural ventilation), the red ones are dedicated to extraction of air and the yellow ones to smoke extraction in case of fire in the atrium. However the partition is set underneath, there is always in the room one fresh air arrival and one for extraction.

3 Tests carried out during the design phase

Several series of tests, both numerical and physical on a 1:1 scale, in a wind tunnel or in laboratories, were carried out from the design phase to validate the concepts envisaged for the box floors. Thus, a prototype was produced in the "Centre Scientifique et Technique du Bâtiment" (CSTB) Laboratory in order to carry out acoustic and aeraulic tests, aiming to determine the choice of the type of sound traps to be installed in the ventilation channels to allow the reached acoustic attenuation while allowing to obtain the inlet and outlet air flow rates in accordance with those expected.

Fig. 12 Overview of the model and the aeraulic tests carried out in order to test the sound traps to be used in the channels and to find the most adapted and compatible one with natural ventilation

4 Phases of realization : from factory to the site

After consultation and designation of the companies in charge of the realization, the studies immediately started on the mode of realization of the floors. Once all the study stages had been validated, the industrial manufacture of the 940 box beams and 914 Pi beams, whole surface around 34 000 m², could be launched.
The weight and size of the elements were very decisive in the organization of the site, both in terms of lead times and in the choice of cranes capable of lifting the most important elements. To take into account the fact that the execution studies of the HVAC companies were not completed at the same rate, it was necessary to start manufacturing the prefabricated elements while leaving a possible adaptation thereafter by making the internal separating partitions of the box girders after installation on site (in red on the diagram shown below). These separating closions allow in the same floor channel to have both return and supply. One of the challenges for the company was to develop the most repetitive and efficient construction sequence possible, given the very large quantities of floors to be made using this principle. In this development, the company notably retained an inflatable formwork system to allow the keying between the prefabricated elements of the main L-shaped beams and the prefabricated box and Pi girders.

Fig. 13 View of the prefabrication device of the project element (top line); view underneath the coffered floor (bottom left); diagram of a box-beam (bottom right)

Fig. 14 View of the structure in progress with the use of inflatable forms (left and middle); interaction between facade and structure (right)
5 Feedback and conclusion

This exciting project was the opportunity to set up an innovative and hybrid system to fully exploit the strengths and intrinsic performance of each, namely the concrete material and natural ventilation. At the time of writing this article, the work is not completely finished and the experience feedback and the figures can not yet be fully analyzed. Nevertheless, it is already possible to confirm that multidisciplinary work is a key to the success of projects with a strong ambition for environmental innovation, contractual boundaries and technical expertise must be erased in favor of the project.

At this time, the delivered building is keeping its promise concerning its modularity, and the implementation of energy-saving systems, in particular natural ventilation. This success is due in particular to the work of all the players concerned from the outset, the choice of prefabrication which has made it possible to set up an efficient manufacturing cycle and to generally meet the deadlines for the completion of buildings in a context of strong innovation. The choice to work in Building Information Modeling (BIM) also allowed from the outset the design of hybrid ventilation while guaranteeing the modularity and scalability of future spaces designed. This choice also made it possible to ensure a construction process and effective monitoring of companies during the construction phase, both in the study phase, and for the management and monitoring of prefabrication and in-situ installation. However, it should be noted that to conceive of a conceptual approach mixing in such an important way the structure, the ventilation systems and the facade requires that the cooperation be total between all the actors both in the design phase and in the execution phase. The contractual links within a project management make it even more possible to advocate this approach, but this is no longer the case for companies in the construction phase, particularly in an allotment in separate lots or in macro-lots where in fact, operating in silos and defending contractual interests can take precedence over the interests of the project.

Going to the end of such a conceptual approach then requires a total investment, both technical and human, in order to animate all the players and ensure that all the specialties are moving in the same direction. After the final adjustments and tests, monitoring the project over the next few years will be instructive and will make it possible to verify that the building constructed behaves as expected.

References

Structural necessity as a change for architecture: the seismic retrofit of Rätia center in Davos (CH)

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Link to the video: https://youtu.be/XfGRX-UWZ48
Structure as a facade – a structure can more than bear

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Abstract

The standards for construction design are becoming more and more stringent. Therefore, today's answer to this ever-increasing and counteracting requirements are constructions that are conceptualized and solved in layers.

The Baloise Insurance Building was designed by architect Valerio Olgiati, the Swiss Life Arena, the new ZSC Lions icehockey stadium, by Caruso St Johns Architects. Irrespective of their different use, size, materiality and architecture both got one thing in common: Load-bearing facades. In this article we outline the approach of the civil engineers of Ferrari Gartmann in their collaboration with the architects based on two projects, that were realised together recently.

Both facades pursue the same strategy: reduce elaborate layers to make them more durable, avoid risky and heavy suspensions, add value on site as well as architectural expression.

To achieve this, it requires an extremely good and in every detail proven interaction of the individual elements on one hand and an excellent cooperation between the different planners and the executing contractors on the other hand.

1 Introduction

Basically, structures have always been built to protect people or overcome obstacles. These criteria fulfil for example the simple wooden beam across the river or the wooden shelter. It is a matter of course that the whole construction also withstands the forces from a statical point of view. Rural buildings testify to integral concept solutions: Whereas the stone wall is built against the weather side, the wooden structure will be built against the south. Such a solution makes it possible to optimally solve the needs with regards to insulation and weathering requirements with the material that is available.

Today, buildings do not perform better than in the past, however, the requirements to fulfil the individual criteria have increased drastically. By now they have reached such a high level that one single planner can no longer master them in its entirety. Therefore, several specialized planners are needed.

In den following every planner completes his tasks on his own. He completes his job and applies it to the structure. He does not care what happens next to it. In most cases this approach is identical to the one used by civil engineers.

Unfortunately, in this way it is not possible to satisfactory solve problems regarding the resource cycle or the sustainability.

2 Challenge: The facade

Basically, the supporting facade in the swiss building culture is prevailing. Only in the end oft the last century the layered facade outbalanced the new constructions. Equally, on-line with the insulation campaign, many load-bearing facades of the sixties and seventies were vested layer over layer.

Experience shows that applying material in layers solves the problems caused by different subject areas. However, building designs lack structural concepts that can be used again and again: in the first place for the project, they were initially designed for and in the second place in relation to other requirements. Hence, it is important that the civil engineer knows about the importance of the load-bearing structure and the requirements it must meet.

Therefore, the question for the civil engineer is addressed as follows: What must the load-bearing structure achieve and which requirements should it meet? Can it contribute to the building culture and
the sustainability, or does it only solve the problem of the flow of forces, sometimes better and sometimes worse?

Facades entail potential risks. Incorrectly selected and incorrectly installed exterior insulation can lead to devastating fires, as the burning of the Grenfell Tower has shown in the past.

To correspond to the high architectural demands with regards to the design of the walls, heavy elements are suspended by steel parts from the supporting structure, uncontrollably hidden in the insulation layer. The risk of rusting through and falling or the instability caused in an earthquake situation is underestimated from the professional community.

Facades bind resources. In most cases one did not think through, that all layers of the individual components of a facade must be brought back into the building cycle.

In the following, we outline two examples of load-bearing facades that have taken up these challenges.

3 Two examples

3.1 The Baloise office building

The new office building of the Baloise insurance company was built in the inner-city area of Basel. It is situated within walking distance to the train station. The building has a height of approx. 40 m and thus ranks as a high-rise building. It consists of four basement floors, the ground floor and nine upper floors. Due to the construction law the two top floors had to be built in a set back way on one side. The rectangular floor plan measures approximately 40 x 34 m. Almost in the center of the building we find the non-accessible atrium that lets a lot of light into the building.

The building itself has different entrances and access zones. Therefore, the office space can easily be used by various independent companies. The interior design of the premises is very flexible and offers a versatile use, whereas its serving rooms are located in cores. The technical rooms and the parking spaces can be found in the basement.

The office complex radiates an own identity. It is built in red colored fair faced concrete that is cast in a non-standard form. The building gets its unique and own expression by three main elements: Concrete slabs, concrete columns, and concrete walls, forming facade, interior design, fire sections, vertical load transfer and horizontal load transfer.

Fig. 1 Picture of the Façade of the Baloise Insurance office building, (left) Photo Sulzer Buzzi,(right) Photo Börje Müller
Fig. 2  Pictures of the Façade of the Baloise Insurance office Building, Photo Yohan Zerdoun

3.1.1 The support – a known element and various topics

The facade makes use of the static components of the ceiling and the column. With the conceptual association to the ancient building culture, the column was also sculptured symbolically as a human support. Repetitive elements such as the base, the shaft and the capital were typically used as creative elements. Optical gimmicks such as tapering or extensions at the column head symbolize slenderness or force transmission. The same themes are taken up by the facade of the Baloise building.

The columns transvers sections on the outside and on the side of the courtyard follow the static load. The pilars facing the courtyard provide at the same time the horizontal reinforcement of the building. Since the building is located in the highest earthquake zone in Switzerland, the horizontal load from the earthquake action is crucial.

Although the ceiling seems to float on the facade it forms an integral supporting element.

The tapering outer facade supports elude the normal range of vision whereas a change of material from concrete to steel allows a static force transfer.

The elaboration and development of this particular detail at the top could only be realized due to the broad experience in exposed concrete construction. For the construction of the facade an economical solution could be found because of the high number of repetitions as well as the prefabrication.

Fig. 3  Elevation Facade east (right) and north (left), Plan Valerio Olgiati
3.1.2 Functionality – technical solutions well thought through

The load-bearing structure which is visible on the facade primarily fulfils the load transfer up to the fundamentals.

The insulation layer runs over the window facade, which is set back. The floor structure and the ceiling substructure form the heat-insulating lining of the projecting ceilings.

The cable routing is entirely kept in the hollow floor. Like this the ceiling could be kept extremely slim in order to optimize the loads for the vertical and the horizontal ablation from the earthquake stress.

There is no sheet metal work or waterproofing layer on the visible parts of the slabs on the façade. Instead, the slab edges were prestressed to prevent that any rainwater can enter the interior space due to cracking in the ceiling. In addition, the concreting stages were designed in such a way that a normal force is generated by shrinkage forces on the already concreted edges.

Fig. 4 Picture and detail section through slab and facade, Photo and Plan Valerio Olgiati

Fig. 5 Pictures of the construction details of the outer columns where one can see the column detailing and the prestressing cables (left) and the concrete steps of the slabs (right), photo Ferrari Gartmann AG
3.1.3 Planning Process – there is no direct way

The construction project provided for a development plan with 3 construction fields. There was a competition for each of those building sites. The project we elaborated and submitted for the competition for one of the construction fields had two floors less than the final project; after winning the competition, the number of floors was increased.

The component dimensions needed to be increased following a check of the horizontal stabilization. However, this solution was rejected, and a new stabilization concept had to be developed. Instead of internal stair cores it was decided to arrange facade pillars symmetrically and centrally in the floor plan. Therefore, the forces due to torsion could be minimized. The cores which house installations, access and service rooms were thus decoupled from the elaborate coordination between the supporting structure and the access to the riser ducts. In consideration of the architecture it was finally possible to solve interior problems more elegantly.

The contractor prepared samples of the façade supports prior to the installation. Like this it was still possible to optimize the detailed project planning and adjust it to the construction process. At the same time the quality could be ensured thanks to the reference sample.

The planning process in this project was not straightforward, included various changes and was completed together with the contractor.

3.2 ZSC ice hockey stadion - Swiss Life Arena

The Zurich ice hockey team, the ZCS Lions, build a new stadium in Zurich Altstetten. It will seat 12’000 spectators. The arena has an area of about 170 m x 110 m. The structure is approx. 30 m heigh. The stadium is divided into several sections and consists of the parking area, the main ice hockey rink, the training ice areas and the stands. The areas of the stands are constructed with prefabricated stand components that are supported by prefabricated sawtooth girders. The main arena is spanned by a girder grillage made from steel framework that span approx. 84 m x 100 m. The other part of the structure is built with reinforced concrete.
The vertical load transfer for the entire structure over all floors and stands is provided by reinforced concrete slabs supported by reinforced concrete walls, interior columns and facade supports. It also includes a concrete framework spanning over 35 m. The horizontal stabilization against impacts such as wind and earthquakes is provided by reinforced concrete panels of the stair cores.

The facade is designed as a visible in-situ concrete structure. In the following this will be illustrated in more details.

### 3.2.1 The concreted curtain – a facade that acts as supporting structure.

From the competition up to the preliminary project phase it was planned to follow a standard facade concept. Originally it was planned to construct the cladding of the stadium in a layered way consisting of a load-bearing structure, an impermeable layer, an insulating layer and a curtain-wall with air space.

However, this structure confronted the planning architect and the civil engineer with a wide range of problems. Perpendicular joints pose a big challenge for an element façade. It is almost impossible to make all these joints completely tight. To guarantee the imperviousness of the joints for exactly as long as the service life of the supporting structure the maintenance would have been huge. Visually, the element facade does not correspond to the intended massive and heavy expression. The air space would further enhance the visual fragmentation.
The actual suspension poses another problem. It runs through the insulation and through the sealing layer up to the supporting structure. Like this, it is no longer visible and therefore cannot be visually inspected. Moreover, the contractor will also have difficulties to check whether all the suspensions have been properly installed. Generally speaking, the industry underestimates the risk that a heavy façade element could fall down.

In response to this challenge, the support structure simultaneously became the façade. The system change had further advantages. The maintenance and therefore the life cycle costs are lower. Likewise, the added value on site is ensured. Due to the simpler construction the elements no longer had to be manufactured somewhere in Europe and transportation routes could be eliminated. Hence the construction of the façade becomes much more sustainable.

![Fig. 9 Plans showing the connection from the slab to the façade, Plans Caruso St John Architects](image)

![Fig. 10 Façade at the actual state springtime 2021, photos Ferrari Gartmann](image)

### 3.2.2 The planning process – from the owner to the entrepreneur

The system change from a traditional structure to a load-bearing façade could not have been realized without the involvement of the client. Subsequently this was followed by site inspections throughout Switzerland to inspect numerous concrete facades. This ensured that the final decision was supported by the client.
In the following a small-scaled sample was produced to make sure that a realization was feasible. Equally, based on the experiences made here, the planning for the bidding phase and the execution could be adjusted and finalized.

The end result depends on the contractor. After awarding the contract, a large-scale sample was made together with the contractor. This procedure allowed to test visual influences from the molding and the concreting process.

The contractor and civil engineer recognized that the facade relief would in the end entail many transitions and construction joints if the facade with its storey-high stages would be constructed in a conventional manner. Inevitable overhangs of several millimeters and thus shadows would be visible, especially in the easily accessible areas. To minimize this risk, he decided to pour and concrete stages with a height of up to 12 m in one piece. However, as this free-standing wall would fall over in the event of a storm, temporary supports with steel girders were installed during the construction period.

As a result of this approach we see a kind of "Western"-style-facade, since the slabs are only connected afterwards.

Fig. 11 Large-scale sample for testing the pouring height for concrete with different phases (left) and mockup showing both façade forms (right), photos Ferrari Gartmann

4 Challenges and Potential

We are facing changes in terms of construction and planning. The traditional understanding of the architect as a master planner is being lost. The planning services are way too complex and multi-layered. Today the traditional civil engineer as planner for the supporting structure can hand over his services to technicians, who perform their work computer-assisted somewhere in the world. As a result, standard solutions and BIM processes are becoming more and more common. Due to the low fees paid to civil engineers, innovation and quality are slowed down and conceptual designs are not even developed.

A building as a whole can only function or even excel in terms of its various components if each individual component is coordinated with one another. Today, the building must and can accomplish so much that the planning performance is no longer the responsibility of the architect alone. The most diverse professional disciplines make their contribution. Therefore, the challenge lies not in the coordination, but in a holistic interface planning in terms of the project. If this challenge is met, the building becomes a work of art that represents more than just a series of functions.

The foundation is laid in the design. As civil engineers we may and must develop a structural design that corresponds to the requirements way beyond the purely static functionality. This is not synonymous to a passive understanding of the role of the other discipline. It is rather an active collaboration to further develop the design to find the best solution together on the construction site. To achieve this, it requires teaching that exemplifies interdisciplinarity on the one hand and a market environment that ensures the inclusion of the civil engineer in the initial design sketch on the other hand. This understanding is necessary to make sure, that the potential of conceptual design become rewarding. Only then can a building be a response to a wide variety of aspects: namely sustainability, social responsibility, and cultural contribution.
From exposed concrete vaults to concealed steel trusses: conceptual design as a creative design act rather than a phase

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Abstract
In this paper, it is argued that conceptual design in Model Code 2010 should be considered as a design act, rather than a design phase. This creative design act can be defined as an ill-defined problem and co-exists to some extent with the ‘other’ design acts the engineer performs during the design phase. This is illustrated by a case study. A part of the design process for the curved roof of a sports hall is traced. While the team had already decided on the architectural form and the structural concept during the competition, afterwards the roof gradually evolved from concrete vaults to a steel truss system. Several observations are made on the design process and an alternative scheme for the flowchart in Model Code 2010 is proposed.

1 Problem statement
This paragraph will zoom in on the concept of conceptual design, as described in Model Code 2010 [1]. In the Code, chapter 7 is dedicated to design, the first subchapter (7.1) is on conceptual design. Subsequent subchapters are on structural analysis and dimensioning, verification of ULS and SLS for several conditions, etc.

The first sentence of the part on conceptual design, defines conceptual design as a stage, a period preceding further design. During this stage, the identified needs shall be examined, requirements for potential solutions shall be defined, these potential solutions are to be evaluated and compared. The proposed methodology contains input, activities and the output – ‘the Basis for Design’. A flowchart illustrating this methodology ends with a full stop.

The subchapter on methodology starts by stating that conceptual design is a creative act, for which it is not easy to establish a methodology. This sentence contains two key elements that will be discussed in detail.

Firstly, it assumes that conceptual design is as well a specific type of design act. This implies that the further parts of the chapter ‘design’ treat another type of design act. For the sake of clarity in this paper, these two types will be referred to as ‘conceptual design’ and ‘detailed design’.

Emphasizing conceptual design as a type of design act rather than a phase, solves some inconsequence with the activities described in the proposed methodology. ‘Decision’ for example, is described as the activity in which potential solutions are evaluated and compared. To do this, the engineer has to make calculations, hence he has to make use of ‘detailed’ design procedures. Another example is ‘Modification’. It is described as the activity in which the engineer should re-design the solution if it is found to be insufficient. This again supposes the solution to be checked, in order to conclude whether it should be modified. Based on these descriptions, it can be discussed that no ‘conceptual’ design can be done without a certain degree of ‘detailed’ design.

Another argument to emphasize conceptual design as an act, rather than a preliminary design phase, is in the general approach of the design codes. Most ‘detailed’ design methods are by nature to verify. This supposes that the choices for shape, dimensions and material properties have to be made in advance. These choices ask the engineer to consider all kinds of boundary conditions such as executability or spatial restrictions. This applies even in the smallest of details in the final phases of the design task. It can thus be argued as well that no ‘detailed’ design can be done without a certain degree of ‘conceptual’ design.
The proposed alternative is to consider ‘conceptual’ as a type of design, part of the design task of the engineer, that co-exists to some extent with ‘detailed’ design tasks, throughout the whole design phase.

Secondly, the above mentioned first sentence in the chapter on methodology, defines conceptual design as a creative design act, for which it is difficult to develop a methodology. This brings to mind discussions on design methodologies that dominated the academic field of design studies for several decades [2]. In the beginning, design studies mostly focused on systematic methods of problem solving, often done by making use of flowcharts. Later, these methodologies were however largely contested [2], [3]. It was recognized that not all design problems could be approached systematically or solved scientifically, that several solutions were possible to solve the same problem and that sequential models were not in line with reality. The concept of ill-defined problems emerged, as opposed to well-defined problems. In short, these ill-defined problems are rather solved iteratively, the solutions are found by trial-and-error, the process depends on experience and intuition and as mentioned, many solutions are possible for the same design problem.

The difficulties to define a clear methodology for conceptual design, the fact that it is described as a creative design act, that experience and intuition are mentioned as tools for conceptual design, and the fact that iterative sequences are assumed in the description of the activities, advocate to consider conceptual design as an ill-defined problem. Considering conceptual design as such, would allow it to be studied by making use of the knowledge from the field of design studies. This facilitates to take valuable concepts such as the co-evolution of problem-solution [4] or the ‘opportunistic’ pursuit of structured design plans [5] in account.

The case study of the design process of a prominent roof structure will be used to illustrate the co-existence of conceptual design with ‘detailed’ design. Simultaneously, some additional observations will be made on the nature of this design process. Based on these observations, an alternative configuration is proposed for the flowchart as is included in Model Code 2010. In this alternative the emphasis of conceptual design is shifted from design phase to a type of design act.

As Corres-Peiretti stresses in [6], more knowledge and discussion on conceptual design is necessary, both for the quality of the engineers work and their education. Case studies of concrete design processes are valuable tools to discuss and explain them.

2 Case study

2.1 Context of the project

In July 2009 the competition for the renovation and extension of the municipal sports centre in Genk (BE) was organized. The existing building has a roof consisting of hyperbolic paraboloids. It was designed by Isia Isgour and André Paduart in the seventies and is one of the most prominent examples of a thin shell structure in Belgium.

A design team consisting of Bel Architects and the engineering office Ney& Partners was granted the project. The team decided to go against the competition brief and proposed not to extend the existing building, but to make a separate sports hall. They argued that it would be impossible not to harm the existing building by attaching a large extension to it, nor to give architectural value to these new sports facilities. The new building has a rectangular plan of 60 by 81m and is covered in longitudinal direction by three oversized arches, proposed as a shell structure in concrete. In this way the extension could become the counterpart of the existing structure.

Fig. 1 (left) The existing building with hypar roof, as subject to renovation and extension in the design competition. (right) Visualisation of the interior of the competition proposal.
2.2 Methodology

The design process is reconstructed based on files from the archives of both the architect and the structural engineer - further referred to as the engineer. Documents such as sketches, calculation models, presentations and mail conversation are organized in temporal sequence. This material will be used for a more elaborate discussion on the specific mechanisms in the design processes of both the architect and the engineer, though this is beyond the scope of this paper. Here, the phase after winning the competition is discussed, primarily focusing on the evolution of the roof structure.

Since both the engineer and the architect were thoroughly engaged in the conception of the structure, it was not opportune to study their work separately. Other specialists such as an acoustic engineer, engineers for special techniques were involved as well. They of course had a certain impact as well, but are not taken into account in this study. This, however, does not obstruct the observations made on the nature of the design process of the engineer.

2.3 Evolution of the structural concept of the roof

In what follows, the design process of the roof is summarized in a series of 27 images, to visually represent the evolution of the structure and to illustrate the text.

Fig. 2 Visualization of the design process (1-12). Framed numbers are the engineer’s documents.
Fig. 3 Visualization of the design process (13-27). Framed numbers are the engineer’s documents.

The numbers between brackets in the following description refer to the images in Fig. 2 and Fig. 3.

When the team resumes working on the project after winning the competition around the start of 201, the engineers send a draft of a first study report to the architects. It is a summary of the different design parameters for the structure, and their respective impact on the structural behaviour. The three
main parameters mentioned are shape, material and execution method. In this report, most attention is given to the study of the shape of the section, emphasizing the structural challenges of this shell structure. The concatenation of different arches for example, with different spans and supports on different heights, results in variable outward thrusts (1). It is proposed to optimize this imbalance, for example by varying in the thicknesses of the different shells, in order to minimize the thrust that will have to be compensated in the intermediate supports, or transferred by the outer shells to the outer supports. As was already mentioned in the competition proposal, it is repeated that because the vaults will only be supported on their end-points, they do not purely behave as shells, but as well as beams. The report concludes by stating that a solution is sought that will combine structural demands with architectural, economical and acoustic boundary conditions.

Several sketches, scattered around in the archives of both the architect and the engineer, show how different possibilities and ideas are explored. The possible position and size of perforations in the roof to give access to daylight is one of those central questions early in the design process (2).

The architects start by focusing on the exact outline of the arches. They define several “fixed” points, based on minimal heights for several sports fields, optimal surface of the hall, etc., and evaluate the look of several shapes (3). In parallel, the engineers explore other parameters, such as the consequences for formwork in a cast-in-place solution. For steep zones a double formwork is needed, which is more expensive than single sided formwork (4).

Following some remarks of the architects, the first study report of the engineers is elaborated and finalised by the end of March 2011. A table with eight options for execution is added, each with the materials in which these can be executed, the respective structural principles these imply and a clarifying sketch. (5). At this point, it is concluded that from a structural point of view, option ‘C’ - concrete with large prefabricated elements - is preferred. This option would be the most economic, leaves different options for execution open and has the advantages that the appearance is controllable and execution time can be kept short. The solutions in steel (options G, H) are mentioned as well as valuable and ’to be researched’, since they give similar advantages, but at this point estimated to be slightly more expensive.

The team continues and studies for example if it is possible to cope with the outward thrust by making use of lateral stiffening ribs, that can as well be used as canopies (6). Both the architect (7) and the engineer (8) as well continue to evaluate options for openings for daylight entrance.

By the beginning of June, several principles for the shape of the arches have emerged (9). It is decided that all curved lines geometrically should be circular arcs, to control the costs of formwork. These several options are as well verified structurally, and in mail conversation advantages and consequences are discussed. One of the shapes, that meets all the requirements for free heights for example resulted in large flat zones, which leads to large bending moments. It is as well examined whether diagonal tension cables might work to cope with thrust (10). Another issue is a nearly invisible bend at the springing of the outer arches. This as well produces significant bending moments, which again results in unforeseen thicknesses of the concrete shell (11). The importance of the funicular shape for a shell structure, becomes the leading argument for the engineers (12). On June 22nd, the client is persuaded to agree on a small reduction of the free height at the outer borders, since it will not hinder the sports activities in practice (13). To keep the structural principle pure and thus keep the thickness of the shells in control, a funicular shape for the arches is decided on.

In the meantime the engineers contacted a large contractor to discuss the project. In mail conversations it can be read how the contractor first proposes a solution in steel. The engineer answers that with a funicular shape, arches in concrete are the most logical solution. The contractor’s final response about the executability of a concrete shell structure (14), comes right before the deadline of the preliminary design phase. Their cost estimation for a cast-in-place concrete shell however, results in a price 20% higher than what was estimated before.

The team decides not to mention this offer to the client, since cost already is a critical issue. For the presentation of the design, the architects make a model in which the arches have prominent horizontal ribs on the inside (15). It is unclear why these were added. A few days later, a team meeting is organised where the input from the contractor is discussed and evaluated. The meeting agenda lists two options to discuss: cast-in-place concrete and prefab concrete. The team concludes to proceed with the option for prefabricated elements. At this point, the (implicit) decision for concrete seems to be final. The engineer proposes several options to stiffen prefabricated elements. This simplifies transport and reduces cost of temporary supports (16).
Right after summer break, on the 1st of August, the formal feedback of the client is received. It mentions to regret the loss of the calm simplicity of the vaults in comparison with the competition proposal (Fig. 1 (right)), in which several issues such as stability, light, acoustics and special techniques were aimed to be solved simultaneously.

The client’s feedback immediately results in a proposal of an ‘integrated’ detail for the elements by the architect (17). This feedback as well seems to urge the team to start fully investigating other aspects of the design. A closer look is taken at the acoustics. The parabolic shapes turn out to be very disadvantageous in terms of noise control, since they have a clear focal point (opposite to what was claimed in the competition). The acoustic engineer estimates that between 50 and 70% of the surface should be covered with sound absorbing material. Consequently, the visual appearance of different percentages of coverage is evaluated (18). Based on several technical and aesthetic arguments, the possibility to place this acoustic material behind the structural layer is discussed in a meeting.

On the 22nd of August the engineer tests this option and concludes that geometrically this implies very large openings (19). He communicates to the architect that structurally the roof would become a truss structure rather than a shell structure. This is studied further, since both quick calculations and a reference show that structurally it should be feasible (20). This configuration is tested in numerical models (21) and developed as prefabricated elements (22). Based on design remarks by the architect on these elements, the configuration of the net is shifted. A three dimensional model of this configuration is communicated by the engineer, without specifying the material (24). Timestamped on the same date however, a sketch to execute the structure in steel can be found in the archives of the engineer.

Around the same time, the team has a meeting with a company specialized in prefabricated concrete (23). They discuss different possibilities and the company prepares a cost estimation. Only the simplest option turns out to be in line with the estimated cost.

In the meantime, the engineer is testing simple models of three-hinged arches (25). By the end of September, he proposes a steel structure to the architect. This results in a lot of discussion about earlier arguments, but a few days later the team decides to proceed with this option.

The final clear, simple truss system of the executed structure only slowly emerges. In first proposals, the tops of the arches are kept clear of diagonals to make space for daylight openings, but the truss system is still statically indeterminate and complex (26). The final steel structure consists of a series of three-hinged arches in combination with isostatic steel trusses in the valleys of the roof (27).

Fig. 4 (left) Structure during construction. (middle) Finished interior. (right) Finished exterior of the building. Picture in the centre and on the right by Tim Van De Velde.

### 2.4 Observations

It can be concluded that three key elements finally resulted in the shift of the structural system. First was the cost. The available budget appeared to be insufficient to build the shells in concrete in the present local building context. This became clear throughout the process after a thorough study by possible executors of both cast-in-place and prefab solutions. Second was a remark of the client, made after the preliminary design phase. The remark slightly recalibrated the design rationale towards a calm, simple appearance of the whole. It emphasized the quality of a roof in which all elements are integrated. The roof being in concrete is not mentioned as essential. During the competition phase, the use of concrete and the shell structure seemed primordial. Although the option for a steel structure was mentioned early in the design process, the preference of both the architect and the engineer for a concrete shell structure keeps resonating throughout the design process. The latter may be – consciously or unconsciously – caused by the strong legacy of concrete shells and the proximity of Piauort’s building. This is further illustrated by the title of an article ‘steel roof in dialogue with an existing shell structure’
in a magazine on steel structures [4]. The third element is probably the most decisive in the process: a large portion of the internal roof surface had to be covered with acoustic material. If this would be placed on top of the inner face, the concrete would become largely invisible. Placing it behind the structural layer, implies large perforations.

3 Discussion

By studying the design process, it can be understood how the final structure got its form. It grew to be a concealed, steel truss system after a long design process. It is very important to note that it is not the result of an assessment in which two options are evaluated in the first phases of the design. Only by checking possibilities, sketching, discussing, researching and verifying all kinds of structural possibilities on different scales, the team gained progressive insight, necessary to accurately point the decisive design parameters to choose the type of structure. This work process is steered both by technical conditions and subjective influences.

In addition, it is clear that numerous valid outcomes were possible for this design problem. This particular design resulted from a specific and unpredictable combination of bias based on personal experience, intuition, et cetera and the particular sequence of decisions. Examples of bias are the first estimations for the decisive design parameters being thrust and openings for daylight, or the importance of the use of concrete. An example of the impact of the sequence of decisions is the discussion on the catenary shapes of the arches, taken on a moment that the structure was assumed to be a concrete shell structure. It can be argued that in the end, it is this structurally logical shape that keeps the conversation going with the neighbouring building. However, it can be seen in the case study, that if the team had chosen for a steel structure earlier in the process, it is less probable that the arches would have gotten their funicular shapes. Steel probably would not have urged the design team to discuss the boundary conditions of the sports fields with the client. This, as well as the iterative progress of the design and the trial-and-error methods used, advocates to consider this conceptual design process as an ill-defined problem.

In the studied case, both the architect and the engineer were largely engaged. Numerous references can be found to argue that technically, it is possible to make the building in concrete, both cast-in-place as prefabricated. However, progressive insights frequently recalibrated the importance of several parameters. The designers chose to keep revising decisions that were regarded as final when it became opportune. This engagement towards conceptual design, and the extent of this engagement, is a choice and an attitude. This seems more important than the team members function as architect or engineer.

The fact that decisions were reconsidered if necessary, implies that it was impossible for the team at the time to define when the phase of conceptual design was over. It can even be argued that until a certain degree, this design act continues until the last execution plan was finished, or even until the building was built. This asks to question the explicit full stop at the end of the flowchart in the Model Code. The final step in the flowchart before this full stop, suggests the formulation of ‘The Basis for Design’. The information this document should contain, such as an elaborate description of the structural concept, including information on geometry, details, construction materials, envisaged construction methods, as well as information on design working life, serviceability, et cetera, can in reality probably only be finished in detail after the whole design task has been finalised, but certainly not before the engineer begins the ‘detailed’ design stage.

At last, it can as well clearly be seen that the engineer is simultaneously involved with the activities (analysis, search, decision, modification,..) described in the methodology for conceptual design, as he is with his ‘detailed’ design tasks (calculation and verification). In the beginning the verifications are more rough and estimative, later they become more detailed design verifications.

Therefore, an alternative to the flowchart in the Model Code is proposed, based on a scheme proposed by Luyten [7]. In this alternative, conceptual design is not strictly defined as a separate stage in the design process, but as a design act that co-exists with ‘detailed’ design throughout the entire design phase. The diagonal line suggests the evolution of the ratio of conceptual design tasks and ‘detailed’ design tasks throughout the design phase. In the beginning, mostly conceptual design activities will have to be executed. This gradually evolves towards a predominance of ‘detailed’ design activities. It has to be emphasised however, that this proportional evolution does not have to be this linear. It only suggests a global evolution of the ratio throughout the design process.
4 Conclusions

In this paper, the description of conceptual design in Model Code 2010 is discussed. It is argued that conceptual design should be considered as a design act, rather than a design phase. Conceptual design cannot be done without a certain degree of ‘detailed design’, and the other way around. This creative design act can be defined as an ill-defined problem. As a result, an alternative scheme is proposed for the flowchart for conceptual design the Model Code. In this scheme, the co-existence of conceptual and ‘detailed’ design acts throughout the design phase is depicted, as well as the gradual evolution of their ratio.

This is illustrated by making use of a case study, in which a phase of an actual conceptual design process is studied in detail. Although the team had already decided on the architectural form and the structural concept during the competition, afterwards the roof evolved from concrete vaults to a steel truss system. Several issues, such as cost, execution methods, outward thrust and openings for daylight influenced this evolution. In the end, however, especially acoustic requirements turned out to be of decisive importance.

Based on this case study, some additional observations are made. The structure is the result of a design process, during which the designers chose to keep revising decisions that were regarded as final when it became opportune. This engagement towards conceptual design is a choice and an attitude.

In addition, it can be concluded that a case study is an effective tool to discuss conceptual design in engineering. Case studies can illustrate how small occurrences deviate the design process and show what a design processes consists of. This largely accommodates discussions on conceptual design, in order to further promote sound engineering.

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References

FPM41. Office building in Lisbon

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Abstract
The building is located within some of the busiest avenues in Lisbon, near the well-known Marques de Pombal Square, and has 17 upper levels with 1400 m² per level, for office use, and 6 underground levels for parking and technical areas. The different functional and architectural constraints led to the development of ingenious structural solutions and the need of creativity in the conceptual design. The different structural solutions explored are presented in detail in this paper, mentioning the analysis models and the main calculations, as well as the required studies of the different construction’s phases. During the construction, the critical regions were monitored and the observed displacements were compared with correspondent numerical values.

1 Introduction
The building, which is finalized since the end of 2019, is located between Av. 5 de Outubro and Av. Fontes Pereira de Melo in Lisbon (refer to Fig. 1) and nearby buildings with identical volumetry, such as Sheraton Hotel, Imaviz office building and Saldanha Residence building.

Fig. 1 Building location (google maps)

The architectural concept and the client constraints led to the development of special structural solutions. Some of the most important aspects were:

- The excavation of 6 underground levels, near to the metro tunnel and thus with strict displacements limits for the retaining wall.
- The large open spaces for office use with slabs panels of approximately 12m x 19m without columns in all the levels to improve flexibility and functionality of the internal area.
The large area at the main entrance of the building where, for several reasons, was not possible to provide columns, leading to a cantilever area of 12m x 19m in the fifteen upper floors. Each of the above-mentioned constraints became a very engaging challenge to solve. Several conceptual design studies were developed together with the full design team, which turned out to be a very inspiring activity, showing how structural engineering is a very creative profession. The authors worked deeply in every design stage, from the conceptual design to the reinforcement detailing phase and throughout the construction, providing support to the constructive plan and special scaffold design.

This paper describes the main structural solutions applied, mentioning, when relevant, the design models and structural calculations, and referring to the constructive process and the main difficulties along construction.

2 Main Inputs

The architectural design envisioned a building with a plan area obtained by the merge of two rectangles, the first aligned to Av. 5 de Outubro and the other aligned to Av. Fontes Pereira de Melo, with a gross area per level of 1400 m² and with 6 underground levels and 17 upper ground levels. The underground levels are mainly for parking and technical areas, a commercial zone and the lobby are located at the ground level, and the upper floors are for office uses. The lifts, stairs and restrooms are in the middle of the office area, with the intention to provide a large open space in the remaining area (see Fig. 2), leading to 12mx19m slab spans.

Fig. 2 Typical upper-level plan (left). Preview of the building entrance (right)

A second important constraint for the structural design was the building entrance in the South façade, where it was projected a large exterior area without any columns, leading to a 12m x 19m cantilever in 15 upper floors (see Fig. 2 - right). The main objective of this area was to provide a clear and public passage from the main avenue (Av. Fontes Pereira de Melo) to the new garden located at the West zone of the building. This was a private area that was given for public usage. The placement of structural columns in that zone would unavoidably lead to several constraints and would lay a private footprint in a public domain area.

Finally, a last important constraint was the excavation of the 6 underground levels at an urban site. The nearby metro tunnel led to a semi top-down solution in the East side of the building. Underground slab strips were cast to equilibrate the soil impulse in the retaining wall throughout the excavation until the slabs were built (Fig. 3). These slab strips are part of the final slab, were built every 2 underground levels and the self-weight being carried by provisional micro-piles located close to the final concrete columns. The required thickness of these slab strips (t=300mm) was then implemented in the remaining area of the underground floors and the spans were adjusted up to 9m. Regarding the mechanic characteristics of the soil foundation, silty clays, with good resistance leading to characteristic soil stresses from 450 kPa to 650 kPa, were found at level -6.
3 Structural conceptual design

The structural design was defined to fulfil all the imposed constraints and provide, as much as possible, an economic solution. Looking for a suitable, simple, and rational structural solution, requires plenty of time, engineering judgment, creativity, and pleasant brainstorming. A good answer usually appears only after long discussions over the advantages and disadvantages of each possible solution, with the exploration of structural systems in other known buildings, bridges, or any other type of structures. So, it is very common that structural solutions are inspired by other engineer’s structural forms or systems. A step forward is welcomed when the “original system” is improved, leading to an increase of knowledge in structural engineering. The solutions implemented in this building were also inspired in other known buildings with similar requests. The following section will summarize the main characteristics of these solutions.

The required open spaces in the upper levels, leading to approximately 12m x 19m spans, were obtained with a 350mm thickness concrete voided slab post-tensioned in both directions (refer to Fig. 4 and Fig. 5). The slab is supported by the central concrete core, where the vertical access and restrooms are located, and the peripheral columns, which are 1.35m spaced according to the folded shape of the slab border (triangular shaped border). The voids for the slab have 1.0mx1.0mx0.225m and were specifically produced in fiberglass for this building. This allowed to keep the metric of the façade columns with a ribbed slab with 350mm of rib, necessary to accommodate the prestressing cables. With this solution for the slab, it was possible to have an equivalent thickness of 250mm, leading to an equivalent slenderness of $L/h \approx 50$, lowering the global self-weight of the structure and mainly the compression stresses in the peripheral columns. In the section A (refer to Fig. 5) a prestressed concrete strip with 750mm thickness needed to control the slab deformation. The thickness variation was set to provide enough space for ducts and cables within the false ceiling.

The peripheral columns had three main constraints: they should follow the façade pleated metric, they should have the lowest width to maximize the net area of the offices, and the columns cross section should be the same at all levels and in all the periphery of the building. Following these constraints, steel rectangular hollow sections were adopted with 100mm x 250mm and variable wall thickness according to the applied stresses. It was not an easy decision because it is not very rational to provide steel columns under permanent high compression stresses. The use of concrete columns would be a more logical solution and a high-strength precast concrete column was proposed. However, for different reasons, mainly to simplify the construction process, the final solution of steel columns was adopted. As mentioned before, the cross section of the columns in the periphery of the building is equal in all the levels and the thickness is variable from 16mm to 8mm according to the compression stresses and fire protection. At the ground level, some of the columns have a double height (about 6m) and were filled with concrete to increase robustness for accidental actions. In fact, these columns are located at the East façade parallel to a very busy avenue (Av. Fontes Pereira de Melo) and therefore there is a strong possibility that a moving truck could crush into the façade and damage several columns. Also,
the columns were designed for an accidental action considering that if five columns are damaged the adjacent ones can support the weight of the building for the frequent actions.

Fig. 4 Structural plan of the typical upper levels and the prestress cable layout

Fig. 5 Cross sections.

The solution of having several columns with short spans all along the periphery of the building, led to the need of several transfer systems in the underground levels to allow the car circulation in the parking levels. These interruptions of the steel columns were mainly at the level -1 and several prestress
concrete deep beams (with one level height) were applied and designed (the red line in Fig. 6 is the projection of the upper levels columns, the green line is the projection of the cantilever zone and the blue zone are the prestressed deep beams at the level -1). The application of deep beams for these transitions are particularly useful, for these high loaded elements, due to its stiffness. In fact, if these load transfer systems had not enough stiffness, a redistribution of stresses would occur and the axial force in the adjacent steel columns would increase, leading to the need of different dimensions of the columns. Most of these structural elements have openings for people passage requiring an extra attention in the design and detailing.

![Fig. 6](image-url)

Fig. 6 – Plan view of the level -1 with the deep beams to transfer steel columns loads in the parking levels to allow car circulation.

Regarding the cantilever zone, the structural solution for the slabs is completely different. Truss beams with 12.25m span and spaced every 2.7m were adopted, supporting a 125mm lightweight concrete slab. The main objective was to minimize the self-weight of these floors since the eccentric loads produce a permanent bending moment in the main core. The loads from the truss beams are supported by the South façade truss and then transferred to the core by a spatial truss (refer to Fig. 7). The load-transfer system is very simple and clear, and the internal stresses may be easily checked by hand calculations, which is very useful for an adequate assessment of the results of more refined and complex calculations. Special attention was given to all the connections between diagonals, vertical and horizontal elements, providing joints geometry to guarantee a smooth and rational flow of stresses and avoiding stress concentration zones. The connection of the diagonal elements to the concrete columns is critical because it must transfer the vertical loads of the three levels in cantilever and a large tension force to the concrete zone, so it required special attention. A tooth plate with prestressed bars allows the transmission of the vertical and horizontal forces to the concrete in a rational and adequate way in which the connection’s structural behaviour and load path are perfectly clear (refer to Fig. 7).
The design included the detailed analysis of the construction stages and the monitoring plan of the displacements throughout the building process of the cantilever. The concrete zone was built following
the traditional methods. This area had to be built three levels above the spatial truss in cantilever to allow the connection of the main diagonals. The slabs in the cantilever zone were built with provisional steel columns that were aligned with the concrete columns at the underground levels, to avoid the need to extend the temporary elements down to the foundations. The provisional columns were removed when the 2nd spatial truss was built, i.e., when the cantilever was in the 8th level. Afterwards, the self-weight of the remaining levels was supported by the lower-level truss system already built. During the construction, it was measured the cantilever displacements were measured and compared with the numerical analysis values, allowing the validation of the numerical models and the confirmation of the structure expected behaviour. For example, the maximum displacement in the cantilever after removing the provisional columns was 2mm for a 12m span, showing the efficiency and stiffness of the structural system.

4 Structural Modelling

For the analysis of the structure and to check service and ultimate limit state several models were developed always seeking for the correct balance between the model refinement detail and the system complexity. Refined models were applied in critical or high stressed regions or complex load supporting systems, otherwise, more simple and safe equilibrium-based models were adopted.

The global finite element model of the overall structure was mainly used to analyse dynamic behaviour of the structure and to check stresses due to horizontal loads, namely seismic and wind actions. This model was also useful to evaluate the variation of axial forces in the steel columns due to long-term effects such as creep and shrinkage in the concrete core as well as different foundation stiffness between concrete core with a direct foundation and steel columns supported by the retaining wall.

For improved accuracy, different local models of several elements and regions were developed to properly evaluate structural behaviour and safety (e.g. stage construction analysis of the cantilever zone – Fig. 9b), steel connections – Fig. 9c) and many more…).

Fig. 9  a) Global model  b) Spatial steel structure of the cantilever stage construction model; c) Local 3D models of the connections.
5 Conclusions

This paper presents a summary of the structural concept, the design and the analysis of an office recently built in Lisbon with several complex structural systems to fulfil the owner’s will, the functional and the architectural needs. The goal along the design process was to provide rational and efficient structural systems and to choose the appropriate materials, regarding safety, lightweight, improved behaviour, simple load paths and economy. The large spans of the office levels, to improve utilization versatility and flexibility, the large cantilever of the fifteen upper levels to avoid a “private footprint” within a public area, the several deep beams in the underground levels to transfer the load of the peripheral columns and, consequently, to allow the car circulation in the parking levels, were some of the challenges of the design. Furthermore, the designers actively participated during the construction process in designing or supporting the design of provisional structures, in grasping the building difficulties of the design options, suggesting alternative solutions more adequate to the contractor equipment’s and man-work, to correct small construction faults “in real-time” and many others.

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When the constraints design the structure: ‘la Maison de l’Ordre des Avocats’ in Paris, France

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Abstract

This article itemizes the conceptual approach developed with the Renzo Piano Building Workshop agency for the design of the La Maison de l’Ordre des Avocats located in Paris (17th) right down of the new Tribunal de Grande Instance (Courthouse). The very strong constraints of the site have led to technical solutions both at the structural and environmental level, which ultimately constitute the architectural identity of the project. These different solutions have been developed in a multidisciplinary approach where each technical specialty must be able to find its place in interaction with the others in the style of a high-definition watchmaking mechanics. The structure plays a central role here both for the exposed elements and for those not visible.

1 Constraints, smallness and general context generating architecture

Housing the Ordre des Avocats de Paris, their pecuniary regulations fund (CARPA), an auditorium, a library and a business center dedicated to approximately 7,000m², the Maison des Avocats aspires to an architectural transparency, a democratic justice expression.

Fig. 1 View from the Palais de Justice forecourt

The architecture of the Maison des Avocats synthesizes various constraints. They are linked to the urban nature and the peculiarities of the site, to the functionalities of a dense and varied program, to the technical requirements arising from the limited influence of the foundations, to the challenges of a High-Quality Environmental standard (HQE) approach and to compliance with the Paris Climate Plan. All this combined with very clear architectural aspirations, expressing above all the idea of lightness. The Maison des Avocats is therefore an essentially glass building, light and "transparent" to store solar energy. Transparency is understood as the antithesis of opacity and even more as a principle which allows the depth of the building to be perceived and reveals its interior. The building opens a large facade onto the forecourt which remains open to the north-west. The south-west facade faces that of the court, and the south-east facade lines up with the André Suarès street. On the ground floor, where a
setback of the facades amplifies the suspension of the built volume, the floor of the square which extends into the hall underlines the relationship between the interior and the exterior. In gable, the tip of the building on the east side allows a set of double and triple heights which refines the bow.

By its lightness and the skilful balance of its structure adapting to a complex and cramped plot whose basement is impacted by works of art, it is reminiscent of a huge crystalline room on the scale of the city. However, constructing such a building on the trapezoidal plot allocated to the project on the forecourt of the new Palais de Justice in Paris (also signed by the architect Renzo Piano) was quite a challenge.

On the one hand, the location of access to the Line 14 metro station shaped the architecture of the Maison de l'Ordre des Avocats. Access is at the tip of the building, via a shaft allowing vertical circulation to Line 14, leaving little space available for the foundations and supports of the building (chap.2).

On the other hand, the basement is crossed in its transverse axis by the Line 13. The intrinsic characteristics of this metro line, as well as the geometry and the structure of the tunnel under the MODA had a strong impact on the architectural and technical choices that guided the design and construction of the structure throughout its development (chap.3).

Finally, the compactness in plan, linked to the plot allocated on the forecourt, as well as the compactness imposed in elevation (respect of a maximum height of 28m for the last floor) strongly influenced the structural and architectural design of the building (chap.4).

Fig. 2 Complexity and cramped nature of the context

The resulting architectural object is the product of these constraints and of the urban context into which it would fit. To resolve this complex architectural and technical equation and guarantee the building's visual lightness, steel is the structural material chosen for the project to overcome the large spans imposed by the context.

2 Impacts of the Line 14’s shaft

2.1 The apparent exoskeleton

To achieve the construction of the building by avoiding the well of Line 14 under the tip of the building, large cantilevers of up to 27m between supports were made on each side of the building. They made it possible to avoid any bearing of the main structure of the MODA on the closing slab of the well, which was not dimensioned to take strong vertical loads. This solution made it possible to “suspend” the tip of the building above the shaft of the RATP underground station. The space thus cleared on the ground gives great transparency to the ground floor on either side of the central core, while preserving the ancillary works.

To ensure the console crossing of the tip of the building, while excluding a heavy and invasive structure, steel was preferred for these slender characteristics. To reinforce its integration and this lightness, the supporting structure of the 27m cantilevers is made using an exoskeleton lattice (see also chap.4) taking advantage of the entire height of the facade, thus generating a very high inertial capacity and stiffness necessary to meet the very high demands of the double-skin glass facade.

The overall stability of the exoskeleton (non-tilting) is ensured on the one hand by the two double-steel columns D=2x406mm in line with the central concrete core and on the other hand by the two double-columns of the same diameter at the back. The double-columns close to the concrete core are...
directly integrated into the exoskeleton because they are placed in its plane. The others at the rear are set back on the underside of the building, offset from the plane of the exoskeleton. The transfer of loads, from the exoskeletons of the two main facades to the rear columns, is done through a mega-truss beam of about 400T crossing the building in its great width to connect all the elements.

The glass facades imposed maximum vertical deflections on the exoskeleton structure under harmful loads of about 6mm/module, or approximately 3mm/m. The double-skin of the Maison des Avocats is thus made up of a pendulum system, meaning that each glazing module is independent of its neighbours by being suspended from the upper floor at a fixed articulated point. This thus allows the free rotation of the facade plane under the vertical deformations of the exoskeleton. The joint widths between the blocks remain constant, despite the movements of the frame.

In addition, an initial precamber of about 120mm at the tip of the cantilever had to be applied to allow the structure under permanent loads to achieve near-perfect horizontality. Its loading (slabs, finishing work, facades, etc.) was carried out symmetrically, according to a precise order and regular three-dimensional surveys, so that the flatness of the floors is satisfactory. The technical false floors also made it possible to absorb small residual deformations.

2.2 Impacts on foundations

In addition, the hidden structures in the basement were also strongly impacted by Line 14. When building new structures, the RATP also requires them to move away at a certain distance from these infrastructures. A distance of 3m of void between the piles and the shaft of Line 14, a very thick concrete cylinder, had to be respected at all points in order not to generate structural interference between the two separate structures (no interference of the pressure bulbs of the piles of each structure). Of the 1,200m² of land allocated on the plot, only 400m² were actually available to build the foundations.
3 Passage of the Line 13

3.1 Impacts on the foundations and the raft

In the same way as with the shaft of Line 14, the pile foundations around Line 13 are arranged so as to always maintain a distance of 3m from the outer edge of the tunnel structure (see fig.4), thus ensuring that the frictional stresses of the piles at ground level do not influence the structural strength of the underground structure. A security approach was therefore adopted to overcome the uncertainties about the strength of the tunnel, the structure of which has not been the subject of a specific characterization campaign.

This distance adds an additional difficulty to the realization of the slab of the basement level, its free span above the metro being increased to about 13m maximum. A structural solution with a very thick reinforced concrete apron (1.50m thick) bridging the Line 13 was favoured, thus making it possible to maintain a sufficient distance between the underside of the basement and the tunnel vault. A solution using concrete beams, designed at first, was discarded during works stage, the latter generating too much fallout from the beams vis-à-vis the tunnel, and therefore the risk of increased interference between the two structures.

![Fig. 5 View of the concrete apron reinforcement bridging Line 13](image)

3.2 Vibration problem due to the Line 13

Line 13, by its nature and the curved geometry of the interface tunnel, is the source of significant vibratory phenomena, the impacts of which are very heavy on the sizing of the main structures of the building. These constraints have led to the absolute need to achieve an efficient vibratory and acoustic cut-off between the concrete basement and the emerging part of the building (steel structures and concrete central core). For this, a separation, in order to make them completely independent of each other, was carried out by means of high-capacity vertical spring boxes manufactured by the GERB company.
Fig. 6  Principle of vibratory cut-off by spring boxes at the Maison des Avocats
The principle is to have boxes crushed under permanent loads and a variable load share (G+0.3Q) of about 31mm, which corresponds to a natural frequency of 2Hz. Their vibratory efficiency is therefore due to this very fine adjustment of their rigidity in relation to the load to which they are subjected. For this, a global 3D structural calculation model – concrete-steel – had to be developed, from the design stages, in order to study the location and precise dimensioning of all the spring boxes under the building.

Fig. 7  Global structural design calculation model - Steel + Concrete (left) and Steel alone (right)
Knowing that the light superstructure of the work rests on only five main load-bearing points, which are the 4 double-steel columns and the central concrete core, very strong concentrations of forces are observed at the right of these 4 points of support formed by the steel columns. Each of the high-capacity spring boxes can only take up about 120 tons of vertical compression at most. These are therefore real “clusters” of spring boxes that have been implemented under the principal columns, going up to 15 boxes for the most stressed one, subjected to around 1500 tons under the most unfavourable ULS combination.

In addition, supplementary studies with a specialized partner, based on a global vibro-acoustic numerical model, were initiated from the design stage to make the vibratory cut-off more reliable and perfect. These studies thus confirmed that the natural frequency of the exoskeleton was very close to those of the spring boxes (around 2Hz). It was thus necessary to anchor the four columns of the exoskeleton on anti-vibration blocks, the rigidity and associated mass of which, concentrated as close as possible to the spring boxes, made it possible to improve the effectiveness of this vibratory cut-off.
Depending on the associated load reactions, the masses of the suspended beds range from 50 to 75 tons, the latter having been produced using large steel formwork filled with baryta concrete (high density concrete around 4T/m$^3$).

The overall stability of the building is thus ensured by the core and through the spring boxes on which it rests. It all comes down to a clever game of millimetre balances. Studies have shown that horizontal spring boxes also had to be added. Their role is to provide additional horizontal stiffness under asymmetric wind combinations (tending to subject the building to an overall torsion) while not disturbing the vibratory cut-off. The horizontal spring boxes are sensitive systems that only tolerate very few horizontal deformations.

Fig. 8 Spring boxes (left) and anti-vibration blocks in the basement (right)

Built on an anti-vibration mattress, the superstructure rests on these supports in a completely separate way at all points. The continuity of the cut-off joint had to be well auscultated to avoid the local formation of vibratory bridges liable to degrade the effectiveness of the cut-off.

At the same time, the slabs of the ground floor rest on smaller anti-vibration pads distributed more densely on the upper slab of the basement.

4 A compactness of the overall volume

4.1 Compactness in plan

The compactness of the plot in plan was the source of important questions to allow the best possible integration of the dense development program defined by the client. Associated building with structural constraints strong contributions to such a cantilever and after much discussion between architects and engineers, it was decided to reject the structure of the exoskeleton outside the interior. Inserting it in the centre of the double-skin of the facade made it possible to make better use of the surfaces while improving its architectural integration. The first studies naturally placed it inside the building, but this was to the detriment of the optimization and modularity of the interior spaces.

Fig. 9 Plan view of the courthouse of the TGI and the constrained plot of the Maison des Avocats
The exoskeleton structure then takes on a strong architectural presence, both from the exterior point of view, and from the point of view of the occupants inside the building. It enlivens the entire facade, like a belt on its three sides, while providing a subtle dynamic play of presence between day and night, depending on the natural light outside and the interior lighting.

The joint work of architects and engineers has established a strong link which unites the architecture of the Maison des Avocats with the understanding of its main structure which constitutes and defines it. The exoskeleton and its function of transmission of forces become readable thanks to a game of optimization of the material according to the stress levels in the members of the truss constituting it: the density of the elements close to the double-columns fades gradually as you advance through the cantilever to reach the tip. In this spirit of assuming the structure of the exoskeleton, which the architect and engineer share on this project, structural nodes have also been the subject of numerous discussions with the company. The project allowed the convergence of the architectural gesture with the complex manufacturing constraints linked to the visible welds of a structure that is both thin and highly stressed. A large number of these welds were carried out on site.

At the same time, the mega-truss (see chap.2.1.) and the internal secondary trusses (see fig.3) are integrated into the building in a hidden way by inserting themselves into the opaque walls formed by the thick partitions bordering the auditorium. These massive structures almost disappear inside the building, also contributing to the optimization of interior spaces.

4.2 Compactness in elevation

Limited to 28m to escape the constraints of French High-Rise Buildings legislation (IGH) the height of the building was another constraint by reducing the height of the floors from slab to slab to 3.20m, when it oscillates from 3.50m to 3.80m in the most office buildings. As the customer wanted to have a headroom of 2.70m, the overall residual floor height was therefore 50cm. The floors are thus made of custom-made steel honeycomb beams 30cm high in order to optimize space. The calculation of the duct sections for the passage of the horizontal networks led to the creation of very large openings in the steel beams requiring justification by finite element calculations with 2D plates for each of the beams, almost...
all different. The compactness of the floors and the associated large openings have generated a significant tonnage of floor beams, due to the need to use very thick steel plates to ensure their structural justification and in particular to verify the stress concentrations at the opening perimeters.

An active concrete slab visible on the underside, 14cm thick for thermal inertia, incorporates coils for heating and cooling. Once the metal frame was installed, the concrete slabs were poured on site. In view of the difficulty of modelling and understanding the overall path of the forces in such a structure, it was decided to separate the concrete slabs from the metal frame. This makes it possible to guarantee the control of the diffusion of the main forces only in the steel frame, without unforeseen captured forces in the concrete floors. For this purpose, the concrete slabs are simply suspended from the metal beams by means of steel connection pieces, developed for the project. These parts then only act as a suspension without transmitting parasitic horizontal shear forces. The concrete slabs are also cut by means of numerous transverse joints, further limiting their impact on the overall behaviour of the building. The implementation of the concrete slabs was a particularly delicate stage of construction, the goal being to follow the deformations of the frame according to the progressive loading of the building.

A 6cm perforated wooden false floor takes care of the acoustics and completes the floor.

Fig. 11 Typical section on one level (left) & view of cavities provided in the floor beams (right)

Fig. 12 Interaction between steel frame and the MEP systems
5 Conclusion: an architecture shaped by technique where all disciplines merge

The conceptual framework of the project was shaped by the constraints of the site. The solutions developed then completely shaped the architecture and the identity of the building. Even if the structure by its exoskeleton and the facade by its double-skin are the most visible elements of this concept, the technical challenges were even more important for the technical specialties made invisible and the subjects related to vibratory and acoustic comfort.

This project highlights the necessary proximity of the designer actors in the act of creation. The boundaries must be erased so that each one appropriates the constraints and the technical needs of the others to find the solution to his own problem, this interest and this crossed glance between the disciplines is the key of a strong and enriching conceptual approach as much for the project than for the expression of its own expertise. This project shows that this approach is fully possible. It should be noted that the relationship developed with companies in the construction stage must also allow the continuity of the work and the approach carried out during the design stage. In this case, it was of a very high quality, both human and technical, which made it possible to achieve a result at the level expected and make this project a very good reference.

![Fig. 12 View of the tip (left) and of the entrance facade (right) from the forecourt](image)

References


Smart Vierendeel: TRUMPF showroom, Chicago

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Abstract
In the ‘TRUMPF Smart Factory’ in Chicago, the German machine tool manufacturer, who specializes in laser technology, has combined a workshop and exhibition room into a technology center that showcases manufacturing within a digital network of machines. The showroom’s roof structures is conceived of eleven Vierendeel girders made from laser-cut sheet steel. The material-efficient structural system uses the possibilities of modern steel construction to re-interpret the 'truss without diagonals' developed by Arthur Vierendeel.

1 The Showroom
At the so-called Trumpf Smart Factory, the manufacturer of steel laser cutting machines presents the newest tools and industrial lasers in action. Individual processing steps from programming to production are shown. The installed high-tech production represents a consistent implementation of the “Industry 4.0” principle: the digitally supported interaction between man, machine and product can be experienced as “interconnected production in a realistic environment”.

The 45m x 55m large, column-free exhibition hall creates space for a variety of type of occupancies. In addition to the continuously reconfigured production line, there is enough space for temporary uses such as conferences and training courses for customers. Therefore, spatial qualities such as daylight lighting and flexibility of use are essential design parameters.

Fig. 1 View of the Vierendeel girder, with Observation Catwalk passing through the structure, and the machine assembly visible below

2 Conceptual Design Process: “Creating Structure as Space”

2.1 Design Constraints and Ambitions
The main design objective of the project was to make the main actors of the showroom – the machine tools and lasers – tangible for the viewer as directly as possible and from as many perspectives as possible. Ideas for various observation platforms and catwalks above the exhibition area were analysed in the early design phases.
However, the building height, which is limited in accordance with local building regulations, sets narrow limits to such an approach: for example, with a walkway system suspended below the 45 m spanning roof structure, the desired clear room heights cannot be achieved. In order to make this possible, the geometry and structural design of the roof framing must allow a flexible configuration of the access walkways, incorporated in the same elevation as the structure. Thus, it was quickly realized that conventional trusses cannot be used, due to the diagonals blocking the passage.

Another design goal was concerning the roof structure itself: As a visible element of the architecture, the roof truss should also serve as a demonstration object for the potential of modern sheet metal working tools. The exemplary use of optimized components manufactured with laser technology should reflect the technological ambitions of the client.

From a structural point of view, the girders have to transfer high local and asymmetrical loads from the walkways situated on different locations, in addition to the predominantly uniform vertical loads of the roof from its own weight and the comparatively high snow loads. The system must also be sufficiently robust and designed in such a way that no unpleasant vibrations occur when used by larger groups of people.

2.2 Concept development

The concept of an accessible roof space was developed in close dialogue between the architect and the engineer. Very early a Vierendeel girder solution was suggested as one of the most promising solutions. Different variants of the geometry of the truss were first sketched and discussed on a conceptual level and then studied in 3d models and physical architectural models to analyse how the structure is perceived from different angles and perspectives. Furthermore, the structural feasibility was tested with FEM, which informed the iterative design process. Whereas the structural system needed to adhere to the general principles of Vierendeel girders, the design team explored a cautious departure of characteristic shapes and geometries of Vierendeel girder. These studies concerned the spacing of the typically equidistant verticals, the spacing of the chords to each other, or the geometrical shape of the thickened connections from verticals to the chords, transferring the peak bending moments.

![Small scale models by the architectural team at Barkow-Leibinger explore the spatial effects created by the Vierendeel truss design (left and center), hereby making a decisive contribution to the form finding process and photograph of the final built design (right)](image)

While studying geometry options on a global scale and identifying advantages for the architectural design problem with some of the variants, an ideal fabrication, assembly and welding strategy was developed in parallel. Some of the geometrical studies forced the team to question traditional forms of framed steel connections. One of the concepts was to focus on using large steel plates that could be laser-cut using the client’s machinery and then assembled using simple fillet welds. This would reduce the complexity at the framed connections – which are traditionally realized using a high number of rivets, bolts or complex welding – by moving the joint to a location of the girder with comparatively low internal forces. At the vertical members for example, the bending moment diagram changes its magnitude along the member, from positive bending to negative bending, with zone of little to zero bending moment around mid-length of the member. A similar bending diagram also occurs at the individual segments of the upper and lower chords, which also suggests to position the welded joint not at the intersection with the vertical members, but halfway in between these connections. See a more detailed description of the segmentation of the structure in later sections.

The final geometric shape and fabrication methods were able to accommodate the design constraints mentioned above, while achieving the desired spatial qualities.
3 Arthur Vierendeel’s Design Principles

3.1 Conventional Truss Design at the Time and Theory of Secondary Stresses

Originally designed as timber trusses, iron and steel increasingly replaced timber as the dominating material for trusses over the course of the 19th century. In combination with the development of the graphic static method of analysing structures by Carl Culmann (1821–1881), as well as calculation methods developed at the same time by Johann Wilhelm Schwedler (1823–1894), it further increased the popularity of trusses as a long-span system for bridges and roofs. In parallel, iron construction underwent an evolutionary process that gradually departed from traditional timber carpentry jointing methods and mechanical construction towards riveted joints. The graphic static method, as well as Schwedler’s modelling abstraction or simplification of the joint as a frictionless hinge, however, implied that the connections between the diagonals and verticals to the chords were actually built as frictionless pins. Furthermore, riveted joints were increasingly favoured over pin-jointed connection, but the joints using rivets didn’t develop a frictionless pin (i.e. transferring no bending moments) connection, but instead they were showing some moment resistance [1].

The bending moments created in at the joints of the truss where analysed exhaustively at the time by leading engineers, including Friedrich Engesser and Emil Winkler who coined the term “secondary stresses” and with later contributions by renowned engineer Otto Mohr, providing one of the most accurate calculation models. Winkler stated that, according to his calculations, the secondary stresses can amount to up to 30 % of the primary stresses resulting from the pin-jointed trussed framework model [1].

3.2 Vierendeel’s “Poutre à Arcades”

With the controversy around the theoretical analysis of truss structures, the frame girder system (“Poutre a Arcades”) originally developed for bridge construction by the Belgian engineer Arthur Vierendeel's Design Principles

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With the controversy around the theoretical analysis of truss structures, the frame girder system (“Poutre a Arcades”) originally developed for bridge construction by the Belgian engineer Arthur
Vierendeel (1852-1940) offers the ideal solution for the design problem. In contrast to the conventional truss with diagonals, Vierendeel’s girders realize the shear-resistant coupling of the upper and lower chord layers only by means of rigidly (moment-) connected vertical struts. The vertical struts, which are thus loaded with bending moments and shear forces, are often designed with reinforced and widened cross-sections at the connection with the longitudinal chords in accordance with the flow of forces. Arthur Vierendeel originally intended the girders named after him as a more precisely analysed and dimensioned, and thus presenting a potentially more economical alternative to the riveted steel truss structures that were primarily used in the second half of the 19th century [2].

Despite some criticism on Vierendeel’s theoretical approach by Otto Mohr [1], the idea continued to gain momentum and the rigid frame analysis of importance not only for steel frame structures, but also for reinforced concrete structures as introduced by François Hennebique. Most notably, the pipe laboratory at Louis I. Kahn’s Salk Institute (1965) uses prestressed reinforced concrete Vierendeel girders [4], exploiting the unobstructed rectangular openings created between the members to fit mechanical ducts.

Vierendeel has thus responded to new steel construction technology and the insufficiently precise analytical methods available at the time for the structures manufactured in this way. 120 years after the premiere of the Vierendeel girder at the “Congrès International des Architects” in Brussels in 1897, further advantages of the load-bearing principle can be exploited with a parametric adjustment of a number of design variables that respond to the design problem, as well as precise design verification by computer-aided analysis processes and by re-imagining traditional shapes and geometries that respond and adapt to modern production methods and possibilities.

4 Re-imagining And Optimizing Vierendeel’s System

For the TRUMPF Vierendeel girder, on the one hand architectural reinterpretation, and on the other hand structural optimization was the motivation for reconfiguring the girder’s characteristic geometry, When experiencing a constant load on top of the girder, using simplified analysis, the Vierendeel system develops bending moments in the members that change linearly, with its extremes at the joints. As a design response to this condition, Arthur Vierendeel decided to thicken the members gradually towards the joint. At the Vierendeel girder for the TRUMPF project, the characteristic rounded transitioning of the verticals and chords at the joints was reimagined, towards more sharp and expressive design, in collaboration with the architect.

Corresponding to the gradually changing intensity of the shear force diagram across the span, the distance between the vertical elements increases towards the midspan, where little shear forces occur. Towards the supports, at the maximum shear force, the spacing between the verticals is the densest. See Diagrams “B” in Fig. 6.
The vertical distance between the two chords increases steadily towards the center of the span in accordance with the bending moment curve. This leads to a homogenization of the axial forces in the chords, which are controlled by the leverarm, the distance between the chords. The observation catwalk crosses the girders in the area where the maximum distance between the chords occurs. See Diagrams “C” in Fig. 6.

Fig. 6 Individual steps of geometry optimization with the simplified system (A), the optimization with regards to the shear force diagram (B) and the bending moment diagram (C)

Fig. 7 Section drawing of the final selected geometry

5 Fabrication

The Vierendeel girder is constructed from flat sheet steel. The segmentation of the plates is a result of the production-related geometric boundary conditions. Sheet metal with maximum dimensions of 3 × 1.5 m can be laser cut on the client’s machines. The resulting number of 98 web plates is joined together with the flange plates to form the girder. The structural optimization achieved a homogeneous stress utilization over the girder profile with sheet thicknesses of only 10 to a maximum of 20 mm. The girders have a weight of around 75 kilograms per square meter; an economical use of material for a walkable roof structure of this span. The comparatively small sheet thicknesses also enable a small weld seam volume for the steel hollow box construction. Due to the weld joint detail, which has the vertical plates projecting beyond the flange plates, most welded connections can be made as fillet welds, that are easy to produce. Since the vertical plates project beyond the flange plates, the fillet weld is not visible when looking at the girder’s elevation, resulting in a sharp laser-cut steel profile, and hiding the fillet weld from most direct view angles, which allowed more coarse welding techniques. Besides 2 defined bolted erection joints, discussed in the next section, about one third of the Vierendeel girder was preassembled using only welded joints, creating a monolithic appearance. Despite having theoretically the option of grinding the welds smooth and even strengthen the monolithic appearance further, the design team decided to keep the welded seams visible, and allow the visitor to clearly understand the segmentation of the truss when having a close up look at the structure.
Fig. 8  Logic developed for segmentation of the truss’ steel sheets in order to be able to fit them onto the client’s laser-cutting machines and simplify assembly and welding

For simplification of transport and erection, the Vierendeel girder was divided into three segments, each about fifteen meters long. The 1,200 km long transport of the prefabricated elements was carried out using eighteen extra-wide special transports. On the construction site, the girder segments were first joined using a bolted assembly joint and then lifted into place. The bolted joints are concealed within the welded box section of the girder, using access holes and pretensioned bolt connections.

Fig. 9  FEM Model developed for simulating the structure’s performance when exposed to dead loads and lived loads

References


All-in! – design and architecture with prefabricated concrete elements

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Abstract
All-in! describes the architectural symbiosis of program, function and design for the new Adidas building, named HalfTime. To accommodate the many internal and public functions, HalfTime is designed as a versatile multi-purpose building that brings as many of the company’s activities and functions as possible together under one roof. The exposed concrete structure becomes the binding link between form and function of the building. The building impressively shows how an unbiased design approach for precast concrete construction, with a strong collaboration between architects and engineers from the design competition to completion, allowed the limits of architecture to be redefined.

1 Introduction
As the result of an international architectural competition, the idea of the multifunctional HalfTime building was developed by knippershelbig together with the architects from Cobe in Copenhagen, CL Map and Transsolar. The multifunctional building, with a total area of 15,500m² and plan dimensions of 170m by 104m, blends into the surrounding landscape of Herzogenaurach as a rhombus-shaped solitaire.
As the central, only publicly accessible building, it connects the northern and southern parts of the World of Sports, the campus of the Adidas corporate headquarters in Herzogenaurach. Under the motto All-in!, its iconic roof covers the building's ground floor plan and unites the new building's conference and event areas, multi-purpose hall, staff restaurant, canteen and show kitchens as well as advertising areas (Fig. 1).

The design process began with the idea of creating an industrial pavilion in the park. A large roof structure floats above the landscape and creates a seamless transition between inside and out. The roof structure, which cantilevers up to 9m free spanning over the building's ground floor plan, appears monolithic and at the same time transparent with the folded façade surface, becoming the design's guiding theme. The shape of the roof construction (Fig. 2, 3) was thus not determined by purely logical structural requirements.

![Building view](image1.jpg) ![Interior view of the canteen](image2.jpg)  
**Fig. 2** Building view (left); Interior view of the canteen (right).

![Cantilevering roof](image3.jpg) ![Birds eye view](image4.jpg)  
**Fig. 3** Cantilevering roof (left); Birds eye view (right).

As in Vaccini's work [1], the roof structure is initially subordinate to its formal expression (Fig. 4, right) and is developed through the multi-layered integration of contemporary planning and construction demands of building.

The competition design already emphasised the use of concrete as a material that is appropriate for the intended industrial character of the building and has a formative influence on its design. A resulting central question for the buildability and thus also for the structural design was the choice of construction method, linked to the aforementioned requirements for the structure, form and function of the design-
defining elements. A short construction period was defined as a planning goal by the client Adidas at the beginning of the planning process, as were high demands on the quality of construction and design. As a building system made of prefabricated components, the interplay of the roof shape, its joints and detailing were the main challenges for the design of the roof structure. As Mangiarotti, Vaccinis, Mürlimatt et al. (2021) already showed, concrete construction allows a great variety of forms with material-appropriate joints. In the design, this was considered with an unprejudiced approach to the known, industrially influenced detailing of prefabricated concrete elements. Design criteria were the performance of the elements to be joined, practical construction considerations for the production and assembly of the prefabricated elements, as well as the architectural and functional integration of the technical requirements of the other design disciplines. Through the early involvement of the general contractor Max Bögl in the design process, the aspects of the construction could already be considered in depth in the design and the associated economic aspects of the design assumptions could be reliably evaluated in the comparison of alternative solutions.

2 Structural concept

The structure while serving its primary function of load transfer is at the same time connected to the architectural concept and expression of the building. Both the design language of the inner and outer load-bearing elements as well as the composition of the structural elements achieve the architectural demand for a robust, monolithic structural appearance.

The structural concept provides a separation of the inner and outer building areas (Fig. 5). The outer part of the building, the so-called “ring structure”, is formed from a lattice grid of inclined longitudinal beams with cross beams running perpendicular to them. The longitudinal beams are prefabricated concrete elements, the cross beams are made of in-situ concrete. The beams are supported on the ring walls within the building envelope as well as a few additional walls in the external area (Fig. 3 (left)). The ring walls were constructed as prefabricated concrete sandwich elements. The load-bearing element is on the outer face, while the attached shell faces inwards. The façades were designed as mullion-transom façades and fixed to the ring walls or inner roof beams.

The internal building area is formed from V-shaped prefabricated concrete elements. The roof beams, known as “V-beams”, are generally designed as single-span beams, while at recessed building corners and special areas they are designed as cantilevers. The V-beams are supported by the geometrically subordinate main beams of the building. The main beams run perpendicular to the longitudinal axis of the building and are supported by columns within the internal building area or on individual walls. The main beams and columns were designed as prefabricated concrete components, which significantly shortened the construction time.

A structural coupling of the inner and outer building areas takes place at a few isolated locations along the longitudinal axis of the building. On the one hand, this ensured a comprehensive thermal envelope...
without thermal bridges or expensive thermal separations, and on the other hand, it minimized differential deformations from wind and temperature stresses at the building transitions. The building is stabilised via the internal shear walls and core and as well as the walls of the ring structure. The foundation of the basement was built as a raft, the ring walls are supported on strip foundations at ground floor level. At the request of the contractor Max Bögl, the raft foundation was prestressed. The centric pre-stressing was introduced into the foundation via evenly distributed monoliths and served to reduce the amount of reinforcement due to constraining forces (crack width) [3].

3 Structure

3.1 Roof structure – Design and construction

A transparent roof for the use of the building was determined as the design goal. In addition to the structural and practical requirements for the construction of the roof structure, other design aspects had to be taken into account. Examples of these were the coordination of the proportion of windows with regard to the required and permissible lighting levels and the associated heat input, the integration of the cable routing through an installation beam, the architectural effect of the roof beams on the internal space and their integration with the ring structure. The result was a V-shaped, 2m high concrete cross...
Fig. 6 Rigid V-beam joint (left), reinforcement layout (right).

section. The leg widths are between 12cm and 20cm wide, the beam spacing is 2m. The resulting opening dimension between the beam legs of approx. 60cm was provided with a laterally draining glass covering.

About 1/3 of the roof area is thus translucent. The inclination of the support legs was chosen in such a way that, on the one hand, the window area can be achieved without additional shading and, on the other hand, the amount of indirect light into the building is maximised. The V-shaped beam geometry creates a uniformly bright ceiling soffit. This aspect is supported by the light colouring of the roof beams.

The beam height results from the design roof proportions and the structural requirements. A stiffening web was integrated inside the V-beams, which is used for roof drainage at the level of the crossing main beams. The V-beams are arranged as secondary beams between the main beams of the roof structure. The standard span of the secondary beams is 16m, beams at recessed building corners are designed as cantilevers and coupled with adjacent secondary beams by transverse bulkheads. The secondary beams are supported on the main beams by means of a notch at the ends of the beams (Fig. 6). Transversely to the notch, further openings are provided in the secondary beam, which allow for the intermittent crossing of services. The longitudinal distribution of the technical services is done within an installation.

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channel that becomes part of the architectural image. Cantilevered parts of the secondary beams are built rigidly around its support on the main beam (Fig. 6, left). The supporting moment is transferred via a pair of forces consisting of compression and tension struts. The beams are coupled in the tension area through a bolted connection. In the case of the single span beams, this beam closure is added to the construction purely for architectural purposes and is not load-bearing.

The geometry of the main beams is subordinate to the secondary beams, differing from the actual structural hierarchy. The longitudinal alignment of the roof memorable roof expression can thus be read by the user. The beam closure at the underside is an essential design element. The main beams have standard dimensions of 60/100 cm and are oriented transversely to the building's orientation. The dimensions of the main columns, which are rigidly supported at their base vary between 25/60 cm, 45/60 cm and 76/60 cm depending on the load, their location on plan and the geometric requirements due to a beam joint. At the transition of the roof structure to the ring structure, the edge supports of the roof structure are integrated into the ring walls through recesses. The length of the supports are up to 7 m.

Next to the large area of the restaurant, a multifunctional hall with dimensions of 24 m to 48 m is defined as a column-free space. V-shaped secondary beams distributed over three bays are supported on two main beams with spans of 24 m. The dimensions of the main beams here are 150/60 cm. The lower edge of the main beams are flush with that of the secondary beams. A mobile partition wall is suspended along the length of the main beams, which allows the hall to be used in three separate parts. The structural and functional demands on the main beams in the multi-hall require increased cross-sectional dimensions and prestressing of the prefabricated elements. In order to permanently guarantee the functionality of the mobile partition wall, the permissible deformations of the beams had to be minimised in the preconstruction state. In addition to the prestressing, the beams in the centre of the field were made 90 mm higher.

The load-bearing behaviour of the roof structure is differentiated into section-active and surface-active load-bearing systems [4]. For the investigations of the load-bearing behaviour, a 3D FE model was created with the FE software Sofistik. The V-beams were initially represented as beam elements (section-active) with the corresponding boundary conditions. Using a 3D FE model, the effects of horizontal loads from wind and imperfections on the stability elements were primarily determined. The coupling effect with the ring structure was also taken into account. For this purpose, the respective system stiffnesses of the substructures were determined at the coupling points and the load transfer from wind and temperature loads at the interface of the structures was iteratively determined using model variants. The
design of the stability elements and main beams was carried out based on the internal forces from the 3D FE model in accordance with the requirements of Eurocode 2 [3].

For the detailed investigation of the load-bearing behaviour of the surface-active load-bearing system of the V-shaped beam cross-sections, further detailed models were created. The calculation was carried out using area models of the composite V-shaped cross-section and the couplings through transverse bulkheads. The boundary conditions in the intersecting edges of the surface elements were taken into account with regard to the structural detail design. The bearing conditions were modelled via a spring stiffnesses in the FE models, component penetrations were represented in detail geometrically in the model and

Force redistributions due to the temporal and non-linear material behaviour of the concrete are investigated using limit value considerations. The concrete cross-sections were designed using the 3D FE detailed models based on [3], with a reinforcement distribution that is as uniform as possible for the geometrically identical beam cross-sections. A C50/60 concrete was used as the material for all V-beams.

3.2 Ring structure – Design and construction

The roof of the outer building area is a lattice grid consisting of half V-beams in the longitudinal direction of the building and vertical cross beams in the transverse direction of the building. The grid rests on the outer walls of the building and isolated outer walls in the delivery and spans up to 20m beyond the interior of the building and cantilevers up to 9m. The exterior walls are rigidly connected to their foundations via bolted connections. The inclined longitudinal beams are precast concrete elements with cross-sectional dimensions of 200/20cm with a concrete strength of C50/60. The longitudinal beams are fixed to the outer walls, which are at least 3.2m long. The coupling of the precast concrete components and the transfer of the tensile stresses from the longitudinal beams into the outer walls is ensured by a screw connection on the upper sides of the walls oriented towards the inside of the building. Compressive stresses from the bending force pair are concentrated and transferred into the walls via contact pressure, partly via recessed load plates at the wall ends. In the sense of the architectural hierarchy, the cross beams are designed with a lower height and have dimensions of 160/20cm. The cross beams and load-bearing ring walls are each arranged in such a way that the ring walls which are offset parallel to each other are connected by a cross beam. This allows the transverse bending of the grid to be introduced into the ring walls via a pair of forces in each case at the intersection points.

The constructive introduction of the compressive forces takes place via load plates embedded in the walls or the pure surface pressure in the mortar joint. The resulting tensile forces are absorbed via the wall bolts in the longitudinal direction of the building.

![3D FE Model of the ring structure.](image_url)
The load-bearing behaviour of the ring structure is a surface-active load-bearing system [4], which is characterised in particular by multi-axial bending and torsional loading of the slender beam cross-sections. The force curves and the design of the beam cross-sections were determined using a 3D FE surface model (Fig. 8). The modelling was carried out taking into account the geometric relationships of the connection areas as well as the construction-related joint and connection areas between the different beam types. Model studies of modelling variants by means of beam cross-sections showed an insufficient coverage of the stress conditions and processes in the nodes and detail areas, especially for the structural design of the joint details. Sensitivity analyses were therefore carried out to assess the influence of materially non-linear effects, while varying the stiffnesses in different areas using non-linear calculations [3]. These analyses were used to investigate both the system-related load redistributions between the components and connection areas and to determine the deformations in the service condition. The cantilevered beam cross-sections were designed up to 60 mm higher in some areas. The deformations of the ring structure were critical both visually and for the construction process. The façades had to be installed according to plan before the structural supports of the ring structure were loosened. The ring structure was only load bearing after it had been completely erected; no intermediate structural states were planned. The calculated deformations of the ring structure, especially in the horizontal direction and therefore perpendicular to the ring walls, were >10mm under dead load, within the limit range for the adjacent façade constructions. A monitoring of the deformations by the contractor was able to confirm a very close correlation between the vertical and horizontal deformations determined by calculation and those in reality in the critical areas.

During the design process, there was close coordination with the architects in order to achieve harmony between the wall positions, the structural effects on the geometry of the ring structure, the architectural compatibility with the internal building use and the façade design. For the creation of the monolithic roof appearance, the cross beams were constructed in in-situ concrete, for every separate bay between the longitudinal beams. A flexible formwork system was developed for the production of the geometrically identical beam cross-sections, which enabled rapid construction progress in combination with the prefabricated components (Fig. 9, right). The outer walls were constructed as concrete sandwich elements, with exposed visual concrete on both sides. The load-bearing shell of the ring structure is 25cm thick, the thickness of the core insulation is 14cm and the thickness of the inner load-bearing shell is 12cm.

References
Design experience
of a thin steel frame folded plate envelope
for the new railway station of Vasco de Quiroga

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Abstract
Vasco de Quiroga station is a new elevated station of México-Toluca railway, located in Mexico City. The lateral platforms are covered at the top by a roof and at the sides by a façade. The architectural concept for the façade should be a folded geometry and an auxiliary structure to support the roof and the rest of equipment. After a numerical feasibility analysis, the design was transformed into a single steel folded plate frame envelope, highlighting the structure explicitly in the architectural concept of the station. Because of the design experience, basic aspects are provided to guide the structural engineer to an adequate structural design for this type of structures in order to fulfill the requirements of the architects.

1 Basic description of the station
The Mexico-Toluca Interurban Railway is a railway line connecting the city of Toluca de Lerdo with Mexico City. It is a project of 58 Km long with 48,30 Km of railway viaducts due to the complex topographic and urban environment of the area. Initially, the project was planned with six stations: four stations in the State of Mexico (Zinacantepec, Pino Suárez, Tecnológico and Lerma) and two stations in Mexico City (Santa Fe and Observatorio).

As construction progressed, the government of Mexico City raised the need for a new station on Section III, in the interior of Mexico City, between the Santa Fe and Observatorio stations. This new intermediate station will serve residents of the Santa Fe neighbourhood, students of the Universidad de La Salud, as well as access for the Fourth Section of the Chapultepec Forest to be located on the grounds of the Military Industries of the Sedena.

Fig. 1 Vasco de Quiroga station general views. Platform layout (top) and front elevation (bottom) (p.c.: SENER Engineering)
Conceptually, the station will be added on the typical prefabricated viaduct by widening the cross-section and adding structural members to locate the lateral platforms on both sides of the viaduct. This allows the station to be designed without excessively modifying the main line cross-section maintaining the essence of the project but reducing the cost of the new station. The entire station is supported by reinforced concrete columns of rectangular hollow section founded on piles.

The station has a total length of 208 m divided into nine 26.00 m spans. The total width, including both 4.50 m wide lateral platforms, is 17.00 m. The platforms are designed as a 15 cm thick composite slab, consisting of profiled steel decking with an in-situ reinforced concrete topping. The platforms are supported on two parallel 30 cm thick reinforced concrete walls that transmit the platform loads to the viaduct deck. Passenger access to the platforms by stairs and elevators is directly from the street level, through the access buildings. The separation between the payment area and the free access area is inside the access buildings at street level. These buildings are located below the viaduct.

Fig. 2 Partial 3D view of the accesses to the lateral platforms of the Vasco de Quiroga station (p.c.: SENER Engineering)

The design of the Vasco de Quiroga elevated station consists of a single-level station built by viaducts in which the main prestressed concrete beam is shaped as a hollow box. The first initial design criteria are based on the intention to maintain the same mainline cross-section. However, the requirement of placing the lateral platforms of the new stations implies widening the cross-section. Since the magnitude of this width greatly exceeds a normal width for single-deck viaducts, it is essential to include additional diagonal brackets. Therefore, the structural section of these decks is defined by a resistant core or box and two side cantilevers supported on the diagonals brackets. The cross-section of this girder corresponds to the typical section at the mainline, which is made with precast concrete sections of 26.00 m length and a depth of 3.40 m.

2 Architectural conception of the protection roof and façade

The architectural concept for the protection elements is that the platforms are covered on the top by a roof and on both sides by a façade with openings for entrances from street level to the platforms. In addition, the roof of the station has to be open in its central part, where the vehicles circulate, protecting only the platforms.

The first sketches of the station demonstrated the architects’ idea for both protective surfaces: independent elements with independent structures. The initial design planned that the façade should be a folded sheet geometry, which required a structure to hold it to the viaduct deck. On the other hand, the roof consisted of cantilevered rectangular modules of variable depth supported on central columns connected to the main station structure.
After the review by the track infrastructure and railway electrical systems teams, the necessary facilities and equipment were added for the proper operation of the railway system and the station such as the overhead catenary system. This system requires to be hung at a certain height depending on the gauge of the vehicles. Since an opening was designed over the tracks, these systems could not be hung from the initially planned roof. Likewise, other elements and equipment necessary for the operation of the station, such as passenger information displays, also had to be suspended from the roof.

For these reasons, the architectural idea for the roof was modified. It was intended to design an auxiliary frame structure that would have a double function: to support the horizontal protection roof, formed by profiled steel decking with in-situ topping; and to provide support for hanging the catenary system and the rest of the elements suspended from the roof. The disadvantage of this solution is that the original idea of leaving a central opening in the station was truncated by this transverse frame structures.

Fig. 4  Reviewed architectural conception of the Vasco de Quiroga station. Exterior perspective with protection roof (p.c.: SENER Engineering)

Fig. 3  First architectural conception of the Vasco de Quiroga station. Exterior perspective of the whole station (top) and interior perspective (bottom) (p.c.: SENER Engineering)
Although the façade maintained the original conception of a folded geometry, the new feature introduced was that some of the façade panels were intended to be perforated or micro perforated, with different percentages of perforation depending on the location of the panels on the façade. In this way, sunlight would be allowed to pass through from the outside to inside of the station and could be adapted to the exterior landscape: placing the perforated panels in the areas that are attractive to the passenger, and opaque panels in those areas where the exterior landscape is uninteresting. However, these panels still required an auxiliary structure or skeleton on which they were to be placed because these panels could not be considered structural elements since the panels were HPL phenolics panels.

Fig. 5 Reviewed architectural conception of the Vasco de Quiroga station. Exterior perspective without protection roof (p.c.: SENER Engineering)

3 Envelope structure: conceptual design and feasibility analysis

After analyzing the architectural proposals, the station equipment requirements and the main structure of the viaduct, an integrative solution was proposed for the façade and roof structure: a single envelope with folded geometry to provide a solution to the architectural and functional requirements of the station. To successfully complete this design, proper coordination between all disciplines involved in the station design was required, especially coordination between structural engineers and architects.

The structure of the envelope with folded geometry allows to highlight the initial architectural conception of the façade, including the possibility to use perforated panels and provides weather protection with opaque panels. In addition, a folded geometry on the roof allows the suspension of station equipment, such as passenger information displays or the catenary system, integrating the same geometry both on the sides of the station and on its upper part. On the other hand, the folded geometry allows rainwater to be conveniently collected from the roof and led to drainage channels.

From a structural point of view, the main advantage of designing a frame folded plate envelope is to use its own shape as a resistant mechanism. In this way, the envelope can be designed with a reduced thickness of structural material such as a steel folded structure. The envelope is designed on the basis of L-rolled steel profiles at the corners of the folded plate with a double purpose: to reinforce the corners, as the main ribs of the structure; and to facilitate the construction of the structure, because the thin steel plates are expected to be welded to these steel profiles. Regarding the local stiffening of the steel plates, stiffeners orthogonal to the steel plates have been designed to ensure the local stability of the structure. Both the corner profiles and these stiffeners are intentionally placed on the inside of the station, hiding them from the exterior view of the station.

The envelope is composed of 8 spans of 26 m in length, that coincide with the spans of the viaducts. The approximate height of the envelope is 12m and the approximate total width is 19m. The roof of the envelope has a 6m wide central opening, so that the structure is interrupted in its central part to fulfill the architectural requirements. Each span is composed of 13 modules of 2m wide and 1m high, forming 90° angles at the corners where the L-rolled steel profiles are arranged. In order to allow access to the station platform level by stairs, openings are made in the lateral facades of the envelope.
Each module is designed with L203x203x12.7 steel profiles and the steel plates welded to these L-shaped profiles. The envelope is composed of opaque or perforated steel panels, depending on their location. The roof plates are always opaque to provide protection for the station. However, perforated panels are used on the facade in conjunction with opaque panels. The nominal thickness of the opaque steel panels is 5 mm while the nominal thickness of the perforated steel panels is 8 mm due to the loss of stiffness caused by the perforations. In any case, the maximum allowable percentage of hollows in panels is 50%.

To prove the structural feasibility of the conceptual design, an analysis has been carried out using SAP2000, a specific structural design and analysis software based on the Finite Element Method.

The modelling of the steel profiles has been realized by 1D elements (frames) and the steel panels of the frame folded plate envelope and the stiffeners by 2D elements (shell). Regarding the definition of boundary conditions of the structural model, it has been into account the real stiffness of the structure of the platform walls and platform slab on which the envelope is supported.
The loads applied to the structure can be summarized as follows:

- Permanent load of 1.50 kN/m² due to the weight of the station equipment and all suspended elements and 1.00 kN/m² due to the weight of the roof maintenance platform.
- Live roof load of 1.00 kN/m², due to the roof will be only accessible for maintenance.
- Wind vertical and horizontal load of 2.00 kN/m² and 1.16 kN/m², respectively, following a conservative approach.
- Uniform temperature load decrease of -10.6°C in winter, and uniform increase of +19.4°C in summer, for all steel elements.

The conceptual design of the envelope has been verified by a series of basic calculations as a result of the structural feasibility analysis. The structural feasibility analysis is based on three basic aspects:

- Boundary conditions
- Global stiffness mechanism
- Local stiffness mechanism

### 3.1 Boundary conditions

The envelope must be supported on the viaduct and platform structure. For a correct overall structural behavior, a linear boundary condition is required longitudinally on both sides. For this purpose, multiple point supports of the envelope are spaced every 2m, coinciding with the corners of the modules. In order to avoid vertical and horizontal displacements and rotation, it is necessary to design a fixed support. To materialize this support on the viaduct, two aligned point supports are arranged to simulate a fixed support by restraining the rotation by a torque.

![Fig. 8 Boundary condition analysis. Linear fixed support (left) and linear double support (right) (p.c.: SENER Engineering)](image)

### 3.2 Global stiffness mechanism

The global stiffness of the structure is reflected in the vertical and horizontal deflections of the envelope. In this structure, the central opening of the roof strongly influences the global behaviour. If the central opening were complete, the envelope would be divided into two independent pieces, working as two cantilevers. In this situation, the envelope is unstable in terms of its stability and it is necessary to design transverse elements that connect the two parts and provide the global stiffness.

Consequently, it is necessary to connect both sides with a transverse V-module at each end of the spans. In addition, a longitudinal V-module is designed on both sides of the central opening, connecting the transverse modules to each other. In this way, the central opening is surrounded by a V-module on each side, as a highly rigid ring, which provides the structure as a whole with the global stiffness required for adequate behaviour.

The verification of the vertical and horizontal displacements of the envelope is done taking into account the following limitations, according to Mexican standards:

- Horizontal deflection: \( H/240 = 28\text{mm} \)
- Vertical deflection: \( L/240 + 5\text{mm} = 80\text{mm} \)
The results obtained from the structural model for the vertical and horizontal deflections are lower than the limit values and the global behaviour of the envelope can be verified.

Fig. 9 Global stiffness analysis. Deflections calculation in SLS: horizontal transverse (top-left), horizontal longitudinal (top-right) and vertical (bottom) (p.c.: SENER Engineering)

3.3 Local stiffness mechanism

The local stability of the envelope structure can be controlled by designing a sufficient number of steel plate stiffeners and limiting the maximum spacing between them. For this purpose, the maximum design compressive stress of the element is calculated using the reduction factor for plate buckling according to section 4.4 of Eurocode 3 EN 1993-1-5. The following results are obtained for the maximum design stresses on the envelope panels:

Table 1 Maximum design compressive stresses for the steel panels of the envelope depending on their location and the percentage of perforations.

<table>
<thead>
<tr>
<th>Location</th>
<th>Dimensions [m]</th>
<th>Holes [%]</th>
<th>Thickness [mm]</th>
<th>Reduction factor for buckling</th>
<th>Maximum design compressive stress [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof</td>
<td>1.41x1.50</td>
<td>0</td>
<td>5.00</td>
<td>0.15</td>
<td>50.88</td>
</tr>
<tr>
<td>Façades</td>
<td>1.41x1.50</td>
<td>50</td>
<td>8.00</td>
<td>0.19</td>
<td>64.01</td>
</tr>
</tbody>
</table>

Once the limit stresses have been calculated, the von Mises stresses for Ultimate Limit States are obtained from the structural model and then can be compared. Thus, it can be verified that the maximum stresses of the plates produced by wind loads acting on the envelope are not higher than the limit stresses.
4 Conclusions

After analyzing the structural behaviour, the conceptual design of the envelope it has been considered technically valid according to the results obtained from the structural feasibility analysis.

From a conceptual point of view, the role of structures can support architectural ideas and even highlight them explicitly, although this requires great coordination between disciplines, open-mindedness to the architects' requirements while maintaining basic structural principles. A good collaboration between engineers and architects will promote synergies between teams resulting in projects of great value, as in this case, the use of structural form as an architectural attraction.

Regarding the aspects to be considered for the design of steel frame folded plate structures, the support conditions of the structure must be analyzed first. Once solved, it must be analyzed which are the necessary mechanisms to ensure the stability and the desired global behavior according to the boundary conditions and the actions acting on the structure. Finally, it must be verified that local stability is met by providing stiffening mechanisms to avoid instabilities such as buckling.

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References


New Uppsala’s town hall’s glass roof: a design challenge materialized through innovation

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Abstract
As a key part of the construction of the brand new Uppsala’s new Town Hall in Sweden, its rooftop presented itself as a challenge in which geometric and structural constraints would have to be overcome while preserving its functionality throughout all phases of the project, including not only its final use, but also fabrication and installation.

Thanks to an innovative solution, a substructure of tension rods was successfully integrated in the SLO structural system self-developed by Lanik Engineers, resulting in an optimal compliance of the structural parameters while facilitating its fabrication and assembly, achieving financial savings for the project.

1 Lanik’s SLO system
In order to be able to understand the implications of the several constraints of the project and how the solution was approached, we need first to form an idea of how the structural system used in the project works. We will not enter into the details of the design, the fabrication or the surface protection of the system, however, a general understanding of the system’s behaviour is essential for the purposes of this article

As the next section explains, from the very early phases of the City Hall’s project, a single-layer structure was proposed for resolving the glass roof. Lanik Engineers developed years ago its own system, the Single Layer Ortz system (from now on, SLO).

The single-layer structures, also known as laminar or membrane-type structures, place their knots on a surface (usually double curved) which is known as the generating surface. The structure takes shape with this surface as its base, and through a mesh of triangles or squares, the ensemble forms a polyhedron on the aforementioned surface. The axes of the rods on the real structure coincide with the edges of the aforementioned polyhedron. Except in the case of some structures with particularly simple and regular shapes, for example in cylindrical vaults (which have a simple curve) or in certain types of dome (providing they are symmetrical as far as rotation is concerned), the degree of diversity of the rods, and of the angles that are formed between them, is extraordinarily high, an aspect that has a strong influence on production processes.

In order to maximise these structures’ portability, so that they can be constructed far from the production centre, the SLO system breaks up the structural ensemble into two single types of elements: knots and rods. These elements are manufactured in very flexible and highly automated industrial facilities. The fact that they are pre-fabricated means that they can reach much higher levels of diversification, precision and finish than the levels that are normally obtained in the field of metal construction. Thanks to their excellent stackability, the elements that are finished in the workshop, including all the required layers of paintwork, are issued to the site in containers, obtaining high volume efficiency. They are assembled on site using screws only, which makes this stage of the construction process fast and safe.
In contrast to what happens in double-layer spatial structures, in which the rod-knot joints are considered to be swivel joints, the SLO system has been designed so that the rod-knot joint fits perfectly, particularly in a perpendicular direction to the generating surface. In this system, the knots are basically cylindrical parts whose main axis is arranged perpendicularly at the tangent plane to the aforementioned surface on the knot itself. On the side surface of the cylinder there are some planes cut perpendicular to the axes through which the rods in that knot pass. In turn, in each one of those planes, two threaded holes are drilled, which enables the end of each rod to connect with two screws in each side.

The relative position of a rod, as regards the knot’s local axes, is set by three coordinates:
- Orientation: this is the angle formed by the projection of the rod’s axis over the tangent plane to the surface on the corresponding knot, regarding a reference direction on that plane.
- Elevation: this is the angle formed by the rod’s axis with the aforementioned projection.
- Distortion (or twist): this is the angle formed by the knot’s axis with its own projection over the rod’s main plane of inertia.

Fig. 1  Lanik’s SLO system diagram

As opposed to what usually occurs in double layer spatial structures, and in general in most structural systems, with the SLO system there is no need for purlins or other auxiliary structures to fix the enclosure. The same rod profile (rectangular tube) has a flat surface that is considerably parallel to the generating surface and which is arranged in such a way as to facilitate the support and anchoring of the cladding elements. The structural slenderness awarded by the single layer, the neat section of its knots and profiles, and the absence of purlins, all go towards reducing the visual impact of these structures, giving them the highest possible degree of transparency. All this makes them extremely useful for supporting large glass surfaces where a good visual appearance is essential.

It is important to note that, even if the system allows the transmission of bending efforts through the knots, its main advantage, from the structural point of view, is that it allows to produce double curvature gridshells, which gain stiffness due to their geometry. This kind of membranes are very thin and flexible at its plane, but the dome shape gives them the needed stiffness in order to become self stable, by transforming the loads over the roof into compression and traction efforts along the shell. In order for this behaviour to be managed, the shape must have some minimum proportions of height and curvature. Otherwise, the single layer would behave more as a flexion plate.
2 Uppsala’s new Town Hall Project

This construction, designed by Henning Larsen architectural study, consists of a refurbishment and an addition to the existing town hall, with the intention of forming a closed town block. Along with this main and general idea, two specific parts were going to be the ones giving an identity of its own to the city Hall: a modern connection-building in the inside of the new block, and a glass roof with the purpose of opening the inner space and enlightening the whole construction, giving a sense of transparency and connection in line with the mindset of the political corporation towards its citizens.

This design concepts were embodied in the very early stages of the project, as the below image shows from the preliminary sketches by the architects:

Fig. 2 Components of Lanik’s SLO system including glazing and life line

Fig. 3 Sketch of the construction by Henng Larsen [1]. The importance of the rooftop is absolutely clear from the beginning of the designing.

Once the design was more developed, this was the intended result for the construction:
2.1 Requirements and constraints of the rooftop

Once the architectural idea was clearly set, it was time to evaluate the viability of the solutions. The glass roof would have to cover an area of 38,715m x 39,440m, which results in a 1526m$^2$ rooftop. As it was previously said, the geometry would have to provide the necessary stiffness so that the structure could be self stable, meaning that approximately it would have been needed a free height of about 8m to be certain of the self-stability of the single layer structure. However, due to local regulations, the maximum height would have to respect a limit of exactly 3m from the beams on which the structure would rest. In addition to this, we had to spare some height to allow ventilatory installations to pass through, and also consider the height of the supports, leaving the height available for the design of the structure at just 1.5m, a clearly insufficient one, generating a big problem regarding the stability of the structure.

In addition to this, some other requirements would only worsen the situation, as the structure was going to be stressed under remarkable gravitational actions. For starters, due to the location of the project, the snow load was set in 200 Kg/m$^2$, which is a large value for this kind of load.

Related to this snow value, and in order for the cladding to resist the actions, the glass package was calculated resulting in a very thick one (which also had to answer to a requirement of a $U$ value equal to 1.3 W/m$^2$K). Also, in this kind of structures is mandatory to use a tempered glass on the outside part of the structure, and a laminated double toughened glass on the interior side for security reasons. Considering all of these requirements, after the calculation, the glass package needed a double tempered unit on the outside and a double toughened unit on the inside, adding a very considerable self-weight load to the glass roof.

On top of this, there were more limitations to be bore in mind. As it was stated before, the construction consisted of the refurbishment of the two existing buildings plus the construction of another two new ones. These two existing buildings could not take any additional lateral loads, meaning that every horizontal reaction on those two supporting beams would have to be eliminated, probably by using sliding supports, which would only deteriorate the structural conditions, since they would allow the structure to ‘open’ itself while deforming.

So, in conclusion, due to geometric and structural requirements, it was sure to assert that the stability of the glass roof was not going to be given just by its geometry.

3 Approaching the challenge: definition of the solution

3.1 Putting together a team and structural effects needed

It was clear from the moment that the project arrived to Lanik Engineers that a conventional team to study the construction was not going to be enough, as we were facing a wide range of problems that needed different inputs to achieve the optimal solution. To give an appropriate response, a team was set consisting of: the head of the R&D department, two senior engineers, the project manager, and a designer. This multidisciplinary team was going to be able to contribute to the solution of the problem in very different ways such as:
- Modifications in the calculation software in order to introduce new elements
- Structural solutions which should help to solve the behaviour of the glass roof
- Financial limitations within the projected budget
- Fabrication and assembly considerations
- Drawing details and giving advice regarding components or possible interference between elements

Once the team started its work, it was certain that we needed to approach the problem of the stability of the structure. The constraint regarding the reactions on the existing buildings set us on the path, since we had previously used prestressed cables in other projects to eliminate horizontal reactions.

However, these cables had been used from one side of the perimeter to the opposed one because their only purpose was to help with the reactions, but in this case we needed a way to counter deform the whole structure, especially in the central area, and horizontal cables were not going to be enough. The team soon realized that a whole substructure was going to be needed, with vertical elements that could transmit the counter deformation in a homogeneous way and not only in the perimeter. It was then that we realised that this kind of structure had not been done before in the company, and it was going to be necessary to integrate all of these elements and counter effects into the already existing SLO system. In other words, the biggest problems were not going to be originated by difficulties in the calculation softwares or the geometric models, but rather by the integration of the solution in the fabrication of the structure and its assembly.

3.2 Preliminary calculations and form finding: the gravity as an ally

After some trials, the problem was not solved yet, since the deflection in the central area of the glass roof was just too much and the cables were not able to counter act in a sufficient magnitude. The idea of designing the substructure as a free form structure imitating the single layer then came up, as we thought it would have an even bigger effect in the central area.

In general, every roof structure and even more the dome shape roofs, have both gravitator and wind suction loads. In this case, the gravity had been a problem because of the very limited available height, but now that the solution included a full cable tension net, the gravity became an ally. The weight of the structure and the glass was enough to avoid an inversion of efforts due to wind suction, and this is how the gravity, the previous enemy, was the key for the cable solution to be feasible.

Self weight (cable net active):

Wind suction (cable net inactive):
Combination of dead loads and wind suction (cable net active):

Figs. 5, 6 and 7 preliminary models considering different types of loads

Structurally talking the concept worked, so the developed engineering was based in this solution.

3.3 A global solution: integration of all phases

At this point we had reached an agreement regarding the structural solution that we needed so the single layer could work properly. It was time now to look for a feasible way of taking it to reality.

We were soon facing problems with the cabled solution for various reasons:
- The assembly of the structure was going to be made by pieces
- We needed cables of more than 40 meters going through very exact points matching the vertical projection of the chosen single layer nodes to transmit the counter deformation
- The prestress of those cables forming some sort of catenaries could not be as homogeneous as calculated
- The cost of this solution was remarkably out of the budget

Our goal was now to look for a solution with the same effect but that could counter these identified disadvantages. We looked for some inspiration in glazed façades that use tension rods and we were instantly convinced that we had found the perfect match.

By using tension rods, we could form a modular substructure with a very easy and controlled assembly. The prestress could be given by just one of those modules in each line of the net and, with the use of some auxiliary tools, the exact calculated values to be achieved could be effortlessly monitored.

Also, we were able to integrate these elements very smoothly in the SLO system, since all joints in the system are bolted, and we had the option now of simply threading these tension rods into the substructure, instead of looking for ways for big cables to go continuously through it. Once again, this modular solution was presenting advantages.

In order to design the vertical elements transmitting the counter deformation, we tried to achieve synergies with the other self-developed systems of the company. The idea of taking advantage of the space frame system came into play, using the round bars as vertical elements, and the spheres of the system as the threaded nodes into which join the tension rods. The bolted vertical bar would be threaded into the lower part of the SLO node as shown in the following drawing:
Fig. 8  Detail of a tension rod joined in the perimeter and to one auxiliary sphere, which transmits through a space frame bar the counter deformation needed.

We had arrived to a solution that could be made in house and only needed the purchase of standard tension rods, which are significantly cheaper than >40m long thick cables. Also, this solution could be assembled following the same modules than the SLO structure, and the monitoring of the prestress was much easier to be done. A global solution integrating the whole life cycle of the project was now in place.

4  Construction of the glass roof

Since this article develops a whole designing process of an innovative solution taking into account every stage of the project, including its fabrication and assembly, it seems rather appropriate to illustrate the reality of the construction itself with the help of some images. As it was said, the structure was going to be assembled by pieces. These modules need some kind of shoring system before they become self-stable once the structure is finished, but in this case the temporary supports could not be withdrawn until the tension rods were working:
Fig. 9, 10, 11 and 12 Assembly of the structure. From left to right and from the top: view of the substructure of tension rods during their assembly; general view with pieces installed with their temporary supports; lifting of a piece; general view of the finished substructure

5 Conclusions

Many times, when a new solution is developed, we all tend to focus on solving specific problems without considering the different implications that the solution can bring with itself. In construction, it should always be kept in mind that the satisfaction of a client will not come only from the originality of a solution or its functionality, but also from the compliance with deadlines and budgets. This means that perfectly good and innovative ideas can be discarded because of their cost or the time needed for their development.

The case of Uppsala’s New Town Hall’s glassroof was a success not only because it solved a challenging structural problem, but also because it met all the geometrical and structural requirements while saving costs, cutting time limits, and easing fabrication and installation issues.

When thinking about innovations, we should always keep in mind that we live in a world in which ideas will be evaluated taking into account the difficulty of their application, and teamwork and collaboration between professionals with different backgrounds is the key to achieve global solutions that meet the problems caused in every part of a project’s life cycle, and not only in the designing phase.

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References


Layers of transparency and functionality: Academy Museum of Motion Pictures, L.A.

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Abstract
The 150 ft diameter dome of the new Academy Museum of Motion Pictures in Los Angeles is a steel grid shell with cable bracing and flat, shingled glass panels on a secondary layer. During the design process, the team negotiated between a semispherical form (architectural intent) and structurally more advantageous forms (e.g. catenary) and addressed the inherent challenges. The layering of the individual structural members, shading fabrics, conduits, sprinkler pipes, maintenance access, load attachment points, tie-offs for window washing and glass cladding is the design’s main characteristic and needed to be coordinated from the first sketches to the final installation, while meeting requirements of structural performance, functionality, and visual ‘lightness’ of the structure.

1 Introduction
The Academy Museum of Motion Pictures will celebrate the artistry and technology of film, becoming the world’s premier museum and event space devoted to the motion picture. Totaling 290,000 square feet, the project consists of a six-story tall renovated building, formerly known as May Co. department store, now “Saban Building”, and a dome-shaped new building housing a 1,000-seat theater. Both buildings are linked by several – partially suspended – bridges.

Above the theatre, the so-called Dolby Terrace, covered under a 45 m (150 ft.) wide spanning steel and glass dome and architectural centerpiece of the building, will be used for events and special exhibitions. The development of the structural concept for the dome, in collaboration with the Design Architect, Renzo Piano Building Workshop, will be the subject of this paper.

The canopy, hovering over the terrace created by the spherical concrete dome, not only needs to provide weather protection, but also adds light features, shading elements, fire protection, heavy duty tie-off anchors and many more features. The development of a well-performing and extremely lightweight structural concept on the one hand, but also the integration of all the additional layers of elements necessary to fully function as envisioned by the architect and client, was the main challenge in the design of the dome.

Fig. 1 View of the finished dome construction from Fairfax Ave.
“The Academy Museum gives us the opportunity to honor the past while creating a building for the future—in fact, for the possibility of many futures. The historic Saban Building is a wonderful example of Streamline Moderne style, which preserves the way people envisioned the future in 1939. The new structure, the Sphere Building, is a form that seems to lift off the ground into the perpetual, imaginary voyage through space and time that is moviegoing. By connecting these two experiences we create something that is itself like a movie. You go from sequence to sequence, from the exhibition galleries to the film theater and the terrace, with everything blending into one experience.” (Renzo Piano)

2 Primary Layer: Orthogonal Grid Shell Structure

2.1 Design Concept Development

When approaching the design task, and the structural and façade team was brought on board, the ambition was to lighten up the initially suggested space frame to a single layer grid shell, which responded well to the architects concerns regarding the visual heaviness and member sizes of the main structural components of the dome.

In order to make this change, the team had to substantially transform the dome’s load-carrying behaviour. Whereas in earlier versions of the dome design, the main structural members were running radially towards the apex of the dome, relying considerably on the bending stiffness of the space frame to be able to carry dead loads and wind loads, the new structural concept was to run the main structural members in form of parallel, east-west running arches across the dome, with north-south running members intersecting the dome’s main arches perpendicular. One of the design constraints leading to this decision was the newly created opening towards the south and north end of the dome, which would cut off the initially radially oriented arches and require massive transfer structures to guide the load towards the supports.

The generated single-layer structure was supplemented by cross-bracing, pre-stressed cables, which are essential for generating in-plane shear stiffness. Besides a significantly lighter construction, the thereby generated triangulated grid shell also shows a more favourable structural performance and redundancy (alternative load paths) than the originally suggested space frame construction. The triangulated grid shell system had been applied as early as in the late 19th century, such as for the roofs for the gas holders in Berlin developed by Johann Wilhelm Schwedler, that were structurally more efficient compared to contemporary domes, such as the ribbed dome at the Galleria Vittorio Emanuele II by Giuseppe Mengoni, built in the same era.
Fig. 3 Schwedler’s design concept for one of his domes for a gasometer in Berlin, Fichtestrasse, 1875 [1]; comparison between Schwedler’s concept (right, top) and non-braced systems (right, bottom) [2]

A structural challenge that needed to be addressed was the large openings towards the north and south end of the dome. The architectural ambition was to avoid a massive edge beam, as conventional for shell structures with free unsupported edges, but to maintain the lightweight appearance of the dome. An edge beam would disrupt the smooth visual lightness of the transition from landscape and sky to glass roof when viewing towards the north opening, which faces the nearby Hollywood Hills. Unsupported edges in shells however are prone to out-of-plane deflections e.g. caused by wind load. In order to address the low stiffness in this area, the structural engineering team tested several options for reinforcing the edge. After studying fan-shaped cable bracing and other systems, the design team decided to apply a bracing system first applied by Russian engineer Vladimir Shukhov (1853–1939) at the Petrovsky Passage (built 1903–1906), amongst other buildings.

Fig. 41 Shukhov’s system of cross bracing cylindric domes applied on the Academy Museum’s dome structure (left), first applied in the Petrovsky Passage in Moscow (right)

Spatial constraints at the dome required the bracing anchorage to be set slightly further up at the dome and not at the more advantageous position close to the dome support. This required to introduce a out of plane reinforcement of the east-west arches from the support connection to the area where the cables are anchoring to. This system adjustment slightly reduces the efficiency of the bracing. Furthermore, the location of the cross bracing needed to be inset in plan to the third row of the east-west arches, in order to let it visually disappear from the visitors perspective at the terrace. As a result the actual edge of the dome unsupported. The out-of-plane deflections at the shell edges however appeared to be manageable when studying the area in the analysis.
2.2 Final Design and Geometry

The overall geometry of the glass dome follows an exact 45 m (150 ft.) diameter sphere. The terrace is about 23 m (76 ft.) above ground, the glass dome’s apex is about 36 m (120 ft.) high. The openings at the south are about 3.4 m (11 ft.) high, in the north about 6.7 m (22 ft.). The bottom half of the sphere is cut off; however, it still overlaps partially with the supporting reinforced concrete structure of the Geffen theatre. At the area of the overlap, the steel/glass dome leaves a gap to the exposed precast panels, and several pins are employed to stabilize and provide support to the EW-arches.

The primary structure of the steel grid shell itself consists of 101.6 mm (4 in.) diameter round S355 HSS arches in east-west direction. The arches are oriented parallel to each other in plan and spaced at 1.2 m (4 ft.). In north-south direction, the radially oriented arches (north-south-arches) are made of custom-milled solid rectangular sections (about 50 by 60 mm) and intersect with the east-west arches perpendicular at every node. The resulting quadrilateral grid structure is braced (“cross bracing”) to provide in-plane stiffness. The 10 mm (0.4 in.) diameter twin cables are running diagonally and continuously over the entire dome and are clamped at every node of the grid.

Fig. 5 Explosion Diagram of the Dome’s main structural connection, illustrating the individual layers of structure, bracing, glazing support and glass (left) and the built construction, along with a bracket for conduits, shading support and sprinkler pipes mounted to the structural arch (right)

2.3 Structural Behavior

Oriented strictly in east-west direction, the round HSS arches are the main load carrying elements. They transfer the gravity loads of the dome all the way to the embed connections to the concrete dome. The north-south oriented transversal members are connecting the individual arches together. While the north-south struts are all sized equally throughout the structure, the wall thickness of the east-west arches was adjusted towards the north and south opening to account for required higher resistance caused by bending. Due to the spherical geometry of the dome, the arches were not following the ideal geometry of a catenary arch. The deviation from the catenary form leads to the characteristic deformation of domes, with the uppermost area sagging and the lower areas bulging. Some of these effects can be counteracted for closed spheres by introducing circumferential members. For the Academy Museum’s dome however, these circumferential lines were interrupted by the large openings. Therefore this counteracting load mechanism could not fully develop. Through the introduced cross-bracing however, the deflections were able to be sufficiently controlled within a range of a couple of centimetres under self-weight conditions, hence not causing a visually perceptible geometrical deviation from the ideal dome shape.
The key component for the structural integrity of the shell are the twin cables which run diagonally across the dome and avoid any in-plane rhombic distortion of the dome, hence creating an in-plane shear stiffness and a shell-like load-carrying mechanism. The structural design was developed, analyzed and optimized in an iterative manner using various cable pretension values to make sure the structure is perfectly tuned. One of the challenges of the pretension is to have sufficient pretension in all cables after completing all phases of construction, since the initial tensioning process of the cables needs to be performed while the structure is still on scaffolding and controlled while the dome is released off the temporary support.

Although the project is based in California which is located in the St. Andreas fault region and therefore prone to seismic activities, horizontal acceleration due to earthquake was less of a concern thanks to the base isolated construction of the supporting steel-reinforced structure of the Geffen theater. Nevertheless, seismic effects were thoroughly studied in for of a response spectrum analysis. The structural design of lightweight steel grid shell is usually most sensitive to asymmetrical loads such as wind load effects. Therefore, a specifically developed physical model (scale of model approx. d=15cm) of the dome was tested in a wind laboratory and the resulting wind pressure assumptions were used for the structural analysis. In terms of dead load, an analysis of the individual masses showed that the glass weight is significantly higher than the weight of the steel structure. This is mostly due to the comparatively thick glass panes. The glazing need to be designed to safely resist loads induced by maintenance personnel. When analyzing the design using 3rd order theory with the structural analysis software SOFiSTiK, the shell was identified as globally comparatively stiff, but the large openings towards the north and the south of the shell however result in large deflections and stresses.

3 Design and Integration of Secondary Layers

3.1 Glazing

Secondary T-shaped profiles are running approx. 25 cm (10 in.) above the primary east-west structure and serve as a support for the glazing panels. The secondary structure is supported on upstands that occur at each intersection between the EW-arches and the NS-arches. The upstands are oriented radially to the sphere surface. The glazing consists of flat, laminated glass made of two 12 mm glass panes. All glass panels have the same thickness, but almost every panel has a unique shape due to the changing dimension of the quadrilateral grid. The signature shingled appearance of the glazing is created by stepping the top side of the T section. The glass panels overlap each other slightly at every step.

The glazing system was designed with a frame bite of 15 mm what required careful investigation and limitation of all kind of movements, especially those caused by rhombic distortion effects. All laminated glass panes are stepped at the lower edge to allow for a hidden dead-load support. Due to the architectural desire of a highly transparent glazing, solar control coatings had to be avoided. In order to achieve a high comfort (temperatures) on the terrace under the glazing, roller shades as well as operable vents were integrated into the system.
3.2 Shading

The extreme transparency of the shell structure and its glazing required the design and coordination of a complex retractable shading system for which a traditional roller shade system was chosen. The integration of numerous tensioned wires, supporting brackets, and electric conduits required the coordination of several new elements to the structure. The structure also needed to develop the capacity to support local stresses introduced into the members by eccentrically positioned shading elements.

Fig. 7 Shading feature detail (left) and deployed shading elements (right)

3.3 Maintenance Features

The requirement for maintenance of the glazing, such as window washing, asked for a special solution in order to allow workers for access the entire dome surface. Whereas most of the inside surface of the dome can be easily cleaned using conventional maintenance platforms and man-lifts, the portion of the interior dome overlapping with the concrete dome on the east and west side and the entire external surface of the glazing required alternative approaches. The solution, which was developed in close coordination with the design team and maintenance consultants, was an interior catwalk, running between glazing and precast concrete of the dome, and an exterior maintenance stair, leading to the apex of the dome. The stair became one of the main design features of the dome. Workers can tie off from the stair’s uppermost platform with designated man rated anchors. With help of the additional features, the workers are able to reach every corner of the glass surface.

Fig. 8 Maintenance catwalk on the east side of the dome (left, similar on west side), and maintenance stair for a secured approach of the dome apex from the south side of the dome (right), both images are during construction

4 Analysis and Testing

A key aspect that was investigated in the structural analysis was the shear deformation of the dome’s grid. The analysed deformation of every node was evaluated using its three dimensional dependencies and mapped on the dome surface The team could then determine if the magnitude of rhombic distortion would present a concern for the glazing.
The maximum values seen in the analysis were tested in a racking mock up, where the deformations were applied to a 1:1 framing and glass specimen of the project. The design team could hereby determine if a problem would occur for the glass support, load resistance (the glass was tested with a local load in the same test) and for water tightness.

The diagonal twin cables of the dome structure are the essential element preventing such a rhombic distortion mechanism. The cables are clamped each time they pass a node of the primary dome structure. The individual segments of the cables will receive uniform pre-tension during installation, but as soon as the scaffolding is released, and especially in wind scenarios, parts of the dome geometry are deforming in a rhombic shape, explained in the section above. The diagonal twin cables, which counteract this rhombic distortion, will receive increased axial tension forces at areas of the dome where the structure deforms most. This results in different cable forces from one segment to the other. The differential value of tensile force is carried by the cable clamp.

Cable clamps work through the mechanical principle of friction. The pressure on the cable clamp is created by two pre-tensioned bolts. The bolt pretension and the friction coefficient are the main factors to achieve high performance clamps. Since the friction value cannot be exactly determined by theoretical analysis, tests are obligatory to confirm the required friction value in every specific case of application.

5 Fabrication and Installation

One of the main goals during manufacturing was to maintain the tight tolerance requirements of the global structure. The structure was assembled in the shop on a template representing a portion of the dome. This assured that the assembled pieces form exactly the final geometry when assembled on site. The pieces were manufactured using several technologies, mostly by conventional welding of flat steel plates, partially by CNC-milling, some of the pieces were fabricated using drop-forging, such as the cable clamp or the cable end fitting detail.

Fig. 9 Principle of rhombic distortion, differential movement in north-south and east-west direction (left); analysis of dome distortion effects (right).

Fig. 10 Fabricated cable clamp specimen with special zinc coating in the notched steel (left), and tested clamp assembly with visible traces of the cable wires (right).
Fig. 1 1 Template for shop “test”-assembly of the structure (left). The structure was then disassembled into smaller segments, so-called ‘ladders’ for shipping and installation on site. Shop inspection of fabrication and coating of upstand detail (right).

On site, the steel pieces arrived in so-called ladder frames, which means that 2 parallel arch segments were already pre-assembled, including the connecting north-south struts, and the secondary steel structure. This simplified shipping, handling and installation. After placing the primary structure (and attached secondary framing) on a scaffolding, the individual segments were joined using an internal pre-tensioned connection in the east-west arches, and the typical bolted connection for the north-south struts. The remaining element of the structural system, the diagonal twin cables, were installed and fully pre-tensioned. After interconnecting the ladder frames and connecting the structure to the installed embed connections, and final inspection of the cable pretension, the supporting posts of the scaffolding were carefully removed, so that the structure spans free.

Fig. 1 2 View of the structure from the terrace with the large northern opening, facing the Hollywood Hills (left) and a close up view of the structure with all elements installed (right): bracing, glass, electric conduits, sprinkler pipes, shading.

References
Textile reinforced concrete canopies

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Abstract
The Sächsische AufbauBank (SAB) is building its new headquarters building in Leipzig. The basic office use of the new building is supplemented by the key feature of a major new public space - the forum. The forum is formed from 22m high columns, acting as a transition between inside and out, that dissolves into a forest of supports carrying an overarching roof made of single canopies that cover the entirety of the site. Monolithic in appearance, yet lightly constructed textile reinforced concrete canopies, with a diameter of up to 5m, were designed. The innovative transfer of textile reinforced concrete elements into an industrial scale and the established partnership and development process with fabrication companies will be presented.

1 Introduction
The Sächsische AufbauBank (SAB), the federal development bank of the state of Saxony, is building its new headquarters building in Leipzig. The building accommodates 600 members of staff, a conference centre, canteen and underground parking, along with a major new public space - the forum. The forum is formed from columns, acting as a transition between inside and out, that dissolves into a forest of supports carrying an overarching roof made of single canopies that cover the entirety of the site. Beneath the canopies, a number of spaces and new public routes are created like clearings in a forest, inviting inhabitation and passage (Fig. 1 and Fig. 2).

The columns and roof provide a sense of visual and acoustic enclosure against the adjacent highway and integrate shading and passive cooling functions for the office spaces.
The hollow concrete columns vary in diameter of 40cm to 70cm with a length of 22m and are made of high strength spun concrete. The columns are fixed into the slab above the basement.

The canopies are fixed to the column heads and coupled (hinged) at their intersection points to create a horizontal roof plate, transferring the horizontal loads to the building structure (Fig. 2). Monolithic concrete canopies were architecturally intended. Considering the different diameter of up to 5m, lighter options for a monolithic concrete section were searched for. Though, challenging and innovative structural options for the canopies were planned. By using textile (carbon fiber) reinforced concrete, a durable and light monocoque structure was developed. A wall thickness of 30-60mm could be achieved, forming the monolithic, double curved surface of the exterior shape of the canopies.

2 Design and development

The development of textile reinforced concrete has opened up previously unfamiliar possibilities for the production and application of thin walled, geometrically freely formable concrete components in concrete construction. In addition to the readability of a material-appropriate construction, e.g. by its slenderness, hybrid and dissolved constructions are also a possible alternative. The design between the columns coupling points are load-bearing, its design and practical installation had a main significance for the planning. Different variants were tested for the material-appropriate form development of the canopies (Fig. 3). The 'status quo' describes the desired appearance of a volume-like design of the

Fig. 2 Roof plan with canopy arrangement (left); 3D-FE Sofistik model of the Forum (right).

Fig. 3 Geometrical variants for canopies. ©ACME
column head with a visible edge of 25 cm and an integrated, post-installed and therefore tolerance balancing connection.

By means of a hybrid solution of thin-walled textile-reinforced "skins" (Fig. 4, Pos. 1,4) with a thickness of 30-60mm and internally connecting reinforced concrete ribs (Fig. 4, Pos. 3), it was possible to achieve the external monolithic appearance as a concrete structure with a 40% reduction in dead weight (Fig. 5). Generated voids (Fig. 4, Pos. 2) were filled with insulation to simplify the fabrication and reduce weight. In order to minimise complexities in the fabrication, the load-bearing behaviour and the approval procedures ("ZiE"), the application of the textile reinforcement is limited to its material-specific arrangement in flat and thin-walled component areas. Thus, the punctual connection areas of the column heads, their connection to the columns and thus the formation of D-areas (areas of discontinuities) were carried out as classically steel-reinforced concrete constructions (Fig. 5).

The analysis and design of the forum was carried out with a 3D FE analysis (Fig. 2, right) and was based on recommendations for typical concrete design and 3D modelling approaches [1]. Dead loads, imperfections, snow-, static/dynamic wind loads and the supporting conditions for the forum were considered. The connection forces were then applied to local canopy FE models for a detailed analysis with shell elements for the Pos. 1 to 4 (Fig. 4).

3 Scale transfer of innovations

At the moment, the possible applications of textile-reinforced concrete components are still defined by the limits of production. In order to be able to fully exploit the advantages of the new material, new production methods in precast construction are required for the manufacture of the thin components. Especially where recurring elements are used in large quantities, the use of industrial production methods is necessary.
Prototypes can be used to test the limits and possibilities of new materials and selected designs from a practical construction point of view and to develop the requirements for production techniques. Questions about the type of production, the processes in the laboratory or precast plant and the practical construction methods can be evaluated under real conditions.

Practical optimisations can be incorporated into the design, costs can be evaluated and design ideas be tested on a real scale. Only by means of an experimental transfer from the laboratory scale, opportunities and potentials of textile concrete can be highlighted and risk potentials limited for practical applications.

The main challenges of the design are to understand the potential of the new construction methods. These include, for example, the production of thin-walled and large-area component dimensions in prefabricated construction as well as the production of components on an industrial scale. For the planned 186 canopies of the SAB Leipzig, the development of a manufacturing process was carried out within the framework of a three-stage negotiation procedure (tender competition, stepwise prototype development, final offer) together with the company Hentschke Bau GmbH in Bautzen. Using small- and large-scale prototypes, the findings were documented accordingly and the technical feasibility was demonstrated.

The material basis for the prototype was a fine-grain concrete, with a mixture developed by the Technical University (TU) Dresden. Together with an epoxy resin-impregnated flat carbon grid from Solidian GmbH in Albstadt, the concrete was developed and tested as part of the technical approval process "ZiE" for the canopies.
In addition to small-scale material tests of the concrete, the textile reinforcement and its composite behaviour were also tested. The "ZiE" also included the testing of the purely textile-reinforced shear connections at the transition between the "skins" and the ribs (Fig. 4 and Fig. 6). In addition to the structural load-bearing capacity, dynamic tests were also carried out on the basis of expected dynamic wind effects.

With the requirement to obtain an individual approval ("ZiE"), the practical application of textile concrete always appears to be associated with a certain risk and additional economic costs. In general, a "ZiE" is unavoidable for the realisation of innovative constructions in Germany.

As a basis for the approval process of components made of textile-reinforced concrete, there are comprehensive findings on test methods and empirical values from projects that have already been constructed [2, 3, 4].

The production of the prototypes took place in two stages. In the first stage, the material production and the manufacturing were tested using simplified geometries. The size of the area to be concreted, the shaping, the layering and also the colouring were tested. First tests on a larger scale showed a non-regressive blue colouration of the surfaces (Fig. 7). The reasons for this could be found in the concrete mix which was initially optimised for the manufacturing process and colouring. The fine-grain concrete of textile-reinforced components has a high hydration rate and a low water content.

Both result in the surfaces of the concrete having a very high impermeability to gases after only a few days and can effectively delay the entry of oxygen into the boundary layer. The speed of hydration depends on the temperature. High temperature leads to a disproportionately high rate of hydration. Hydration is an exothermic process. If heat dissipation is impeded, concrete components can heat up significantly in the course of hydration. The oxidation and thus the discoloration of the concrete surfaces depends on the temperature in addition to the gas permeability.
Increased temperatures favour the oxidation reactions and promote a blue discoloration. In order to remedy this partially occurring reaction for the type of concrete used in textile-reinforced building components, a concrete mix without granulated blast furnace slag was developed and the production process was slightly adapted.

The second stage involved the implementation of the final prototype. First, the lower concrete cover layer with a thickness of >30mm was placed in the formwork. In the second step, the reinforcement cage, consisting of steel-reinforced ribs, textile-reinforced cover layers (“skins”) and displacement bodies creating the hollow sections, was lifted as a prefabricated element into the formwork on the not yet hardened lower cover layer, secured against floating. Then the component areas were filled with concrete (Fig. 8). The textile reinforcement was already given the required shape, or formed in the required geometry during its fabrication, as the carbon fibres impregnated with epoxy resin can no longer be deformed in the hardened state.

The impressive result is shown in Fig. 9. The production and assembly of the canopies could be tested and proven under practical construction conditions. As a direct basis for comparison, another prototype was produced in monolithic reinforced concrete construction and connected to the textile-reinforced variant as an example. An external distinction of the different construction is hardly possible.

Both the technical feasibility of the developed construction and a marketable possibility to produce the canopies on an industrial scale could be technically proven. Aspects of the large-scale assembly of the still heavy concrete elements and the associated economic costs for the assembly ultimately led to the implementation of an alternative solution using a textile membrane-covered steel construction for the canopies (Fig. 10).

The transfer of scientific findings - from research to practice - initially requires a fundamental willingness to innovate and a pioneering spirit on the part of the clients, designers and construction companies involved. In times of economic boom, it is important to motivate and inspire building practice for new
developments and to provide a sufficient knowledge base of the theoretical connections and practice-relevant dependencies of the developments. The early integration of the fabrication procedure, such as limits, possibilities and its influences on the design process. The integration of executing companies in the planning process has proven to be target-oriented.

Particularly when it is a question of extending the previous limits of application, leaving the laboratory scale and verifying the knowledge gained in practice, a partnership-based approach is required between those involved in the design and the companies carrying out the work. A step-by-step transfer, such as through the construction of prototypes, as well as the partnership-based and interdisciplinary processing of topics at the interface between design and fabrication, always proved to be goal-oriented. In addition to questions of market motivation and execution, the degree of complexity should always keep the

![Fig. 9 Canopy prototypes: Textile reinforced variant A, right; Steel reinforced monolithic variant B, left. © Hentschke Bau GmbH, Bautzen](image)

![Fig. 10 Construction of the forum with membrane canopies on a steel sub-construction. ©Philipp Mecke](image)
requirements and tasks of the building regulations relevant to approval in mind as early in the design as possible.

4 Conclusion
The development of textile concrete opens up previously unfamiliar possibilities for the production and application of thin-walled concrete components and structures in concrete construction. The possible reduction of the dead weight and, under certain circumstances, of the resources used, requires the development of construction methods that are suitable for the material. The theoretical foundations for the production and load-bearing behaviour of textile-reinforced concrete components have been developed in initial steps within the framework of various research projects. The versatile application potential can be transferred into practice on the basis of the described partnership approach between design and fabrication companies. The current challenges identified are the transfer of findings from research to the practical building scale. This concerns the production of component sizes relevant for building practice, the development of manufacturing methods and processes on an industrial scale and the development/application of marketable tender strategies as a basis for a partnership-based implementation of innovations in the construction industry. As a basis for this, a corresponding degree of willingness to innovate and pioneering spirit on the part of the building owners, participating designers and construction companies is shown to be necessary.

References
Conceptual design of the structure of architectural pergolas for La Sagrera multimodal transfer center in Barcelona

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Abstract
The architectural project of the future multimodal transfer center of La Sagrera, located in Barcelona, contemplates the design of a pergola composed by nine rectangular modules with central supports. The previous reference design was planned using steel structures but the objective is to provide an elegant and sustainable solution. The new proposal for the conceptual design is a timber structure for the large-span roof formed by parabolic hyperboloids. With this solution, the proposed structure is lighter, more efficient and sustainable, has a unique aesthetic due to its curvature and the loads transmitted to the buried station structure are much lower than for preliminary reference design.

1 Description of the architectural concept
1.1 La Sagrera urban development project
The architectural project of La Sagrera is a large urban development operation whose center is the La Sagrera intermodal station that incorporates new residential areas and diverse economic activity, green areas and public facilities. The future multimodal transfer center of La Sagrera, located in Barcelona, will be an underground station for the exchange between different modes of transport. The intermodal station will be organized on various levels where the services of high speed, long and medium distance, suburban, metro, intercity buses and parking will converge.

Fig. 1 Conceptual cross-section of La Sagrera multimodal transfer center. Preliminary reference design (p.c.: BSAV)
The architectural proposal aims to integrate the station into the urban environment without discontinu-
ities and, to this end, a linear park is designed over the multimodal station. This large green corridor
integrates the intermodal station along its route and uses a large pergola as an identification strategy.
One of its functions is to protect the main entrance to the platforms from the weather and rain, which
is through a large open and underground concourse.

The objectives and strategies of the architectural narrative to resolve the integrated vision of the station
with the city are as follows:

- Highlight the visibility of the different accesses through a single theme to attract and create
  interest for pedestrians.
- Use of vegetation as an element found on the exterior, facade and interior.
- Design a light and versatile protection roof that integrates photovoltaic elements, allowing the
  protection of the entrance to the intermodal concourse from external agents such as rain and
  sun, and allowing the survival of plant species.

1.2 Conceptual architectural design of the pergolas

The roof to protect the entrance to the intermodal concourse consists of a series of architectural pergolas
located in close proximity to each other. To simulate an artificial forest, the shape of the pergolas evokes
tree shapes with a main trunk and a large crown at the top. This proposal aims to integrate with the
adjacent parks, the intermodal concourse and the city, without modifying the original idea of the project.

For this purpose, the use of timber as the main material of the pergola is the key point. This allows
the area to be assimilated to a forest, designing a sustainable solution in its construction and mainte-
nance, as well as providing a great versatility of use for new functions in the future and an innovative
and feasible structural solution.

The versatility of timber for the structure makes it possible to complete the layout with various
elements such as transparent, translucent, colored or opaque glass. Some of them will be photovoltaic
panels with solar concentrators capable of storing electricity without losing the optical properties of the
glass. On the other hand, the proposal allows the creation of secondary structures in which vegetation
can be entangled or simply create voids to allow the passage of sunlight, wind and rain. This is intended
to fill the access to the multimodal transfer center with color and life.

The pergolas, according to the preliminary reference design, have a total height of 16.00 m covering
part of the station and the intermodal building. In total, nine pergolas are grouped in three alignments
with three pergolas: one over the station and two more alignments over the intermodal building.

The dimensions of the pergolas and their support points are explained below:

- Pergolas over the station: modules of 48.00x39.50 m. Each one has four points of support on
  the roof, coinciding with the four corners of the skylights.
- Pergolas over the intermodal building: modules of 48.00x33.50 m. Each one has two support
  points. The pergolas on the central alignment are supported on the beams of the high-speed
  level structure, instead of resting on the roof. The pergolas located on the external alignment
  are supported on the diaphragm wall of the Metro Line 9 structure.
2 Structural feasibility analysis of the pergolas

2.1 Preliminary reference design

The preliminary reference design already had a steel structure designed for the pergolas and based on hot rolled steel tubes. Conceptually, the preliminary reference design consists of a set of steel-cantilevered lattices that support the horizontal roof of the pergola supported by the central columns of each module.

These steel lattices are of variable depth, starting from the central columns with a maximum depth of 3.50 m, and ending at the edges of each module with a minimum depth of 1.00 m. For each module, 12 steel lattices are required: eight lattices in transverse and longitudinal direction, and four of them from the center to the corners of each module. The both lattice chords are tubular profiles of 600x600x30 mm, while the diagonals are tubes of 300x300x16 mm.

The solution for the horizontal roof of each pergola consists of:

- A main structure, which is the perimeter of each module and two transversal and longitudinal beams coinciding with the lattices. The cross section corresponds to a rectangular tubular metal profile of 1000x600x20 mm.
- A secondary structure, composed by HEB 300 steel profiles located at thirds of the span of the main structure.
- An auxiliary structure, composed by HEB 140 steel profiles joined to the secondary structure, where the photovoltaic panels are supported.
- Connecting elements between the roof and trusses, by steel tubes of 240x240x12.5 mm.
- For the pergolas located on the skylights of the station roof, the columns are 600x600x30 mm. For the rest of the pergolas, the columns are 900x900x30 mm.
2.2 Description of the conceptual structural design

The new conceptual proposal for the structure of the pergolas is based on a substantial change in the approach to the proposed structure of the preliminary reference design. It is proposed to use wood as the structural material and the geometry of the structure as a resistant element.

To support the concept and architectural strategy of simulating tree structures, it is essential to expose clearly the timber structural elements, such as the main trunk and different branches where the leaves are located. Although there is not a large variety of hypars with a timber structure, the singular roof at the Hannover exhibition ground, built for the Universal Exhibition in 2000 in Germany, has been the most inspirational reference for the conceptual design of the pergolas.

There are many different structural solutions to solve that and an important chronological evolution and advancement of dendriform and arboreal structures in architecture has been produced throughout history [2]. However, in this case, the aesthetic criterion of the architects has been very relevant and thus the solution consists on a large-span roof as a modular structure composed of hyperbolic umbrella-shaped paraboloids with long cantilever structural members.

The new conceptual design is based on the following principles:

- **Efficiency:** It has been designed to provide a geometry to the structure that maximizes the use of the structural material, thus reducing the dimensions required for the different elements and reducing the number of structural elements required.
- **Economy:** Due to the efficiency of the structure, the total weight of the structure required and, therefore, the cost of the structure is considerably reduced.
- Elegance: The geometry of the pergolas in the form of a hyperbolic paraboloid provides an aesthetic solution of double curvature with a singular personality that, being located on central columns, tries to simulate an arboreal form.
- Sustainability: The use of wood as the structural material for the entire pergola reduces the carbon footprint and CO2 emissions derived from its construction.
- Versatility: The emptying of the non-structural part of the pergola generates a whole series of different possibilities for its function.
- Ease of construction and maintenance: The pergola can be built in parts in the workshop so that only the assembly is carried out on site.
- Utilization: Wood is a recyclable material and once its useful life in this function is over, it can be recycled for many other functions.
- Functionality: Protection against sun, wind and rain for pedestrians and the concourse level.

The new conceptual design of the timber structure maintains the same division into nine rectangular modules, as the preliminary reference design. The maximum elevation is also maintained at 16.00 m. The variation of the depth of the hypar is 6.00 m, with the lowest point located 10.00 m above ground level.

Conceptually, the new structure consists of dividing each pergola into four rectangular hypars. These hypars keep two of their sides in a horizontal position at the top of the pergola, while the other two sides have a variable height from the outer edges to the center of each module. The perimeter of each hypar conforms the main beams of the structure.

These main beams of the structure are of a rectangular section of variable depth, with a maximum of 2.00x0.60 m and a minimum depth of 1.00x0.60 m at the cantilevered edges. At the perimeter of each pergola, is designed a wooden edge beam with rectangular section of 1.00x0.60 m.

For the hypar, since it is a ruled surface, is designed a secondary structure with straight timber beams for supporting the photovoltaic panels. The rectangular cross-section of these elements is 0.30x0.20 m.

With respect to the columns, the section designed is a square timber section of 0.80x0.80 m. These columns are raised above the low point of the pergola to provide an additional mechanism to reduce the deflections produced by the structure’s permanent and variable load without requiring highly rigid elements. For this purpose, the main beams of each pergola are tied from the central columns with two cables: one from the center to the end of cantilever, and the other from the central columns to the mid-span.

For all structural elements, a homogeneous glued laminated timber material GL36h has been considered with a nominal density of 450 kg/m³.

### 2.3 Basic calculations and detailed structural model

A two-step process has been followed: basic calculations of the forces in the umbrella, followed by a detailed analysis by using a structural design and analysis software based on the Finite Element Method.

The first steps in the conceptual design of this type of structures should be made assuming that the umbrella hypar is a thin shell. The internal forces are derived without consideration of bending (i.e. membrane theory), meaning it is assumed that moments in the shell are zero, under a vertical load per unit area of ground projection [3].

![Fig. 5 Forces on an element in a hypar shell (p.c.: Princeton University) [3]](image_url)

The variable k of the equation of a hypar is the warping, or twisting of the hypar and is defined by the equation (1):
Note that the assumption of vertical projection is reasonable for relatively small warping (i.e. relatively flat hypars) [3]. In the case of Rios Warehouse roof designed by Candela, for example, using this simplified assumption yields stress results no more than 5% different compared to a more accurate solution [4]. Therefore, by calculating the roof warping of the Rios Warehouse and of La Sagrera conceptual design, it is concluded that the vertical projection assumption is acceptable, due to La Sagrera hypar is flatter than Rios Warehouse hypar:

\[
k_{\text{Sagrera}} = \frac{6 \text{ m}}{16.75 \text{ m} \cdot 21 \text{ m}} = 0.01705 \text{ m}^{-1} < k_{\text{Rios Warehouse}} = \frac{2 \text{ m}}{7.625 \text{ m} \cdot 5 \text{ m}} = 0.05245 \text{ m}^{-1}
\]

Once it has been verified that the simplified calculations are valid for this roof, it has been calculated the axial forces on the edges, valleys and the structure in the tympan, under the action of vertical permanent and variable loads \(p_z\). Considering these forces, it has been estimated the main dimensions of the timber structural members.

In contrast to the thin concrete slab, the conceptual design of La Sagrera consists of straight structural elements located in the generatrix. Hence, the axial loads on the structural members in the tympan can be estimated using the equation of the shearing force in a hypar shell [3]:

\[
N'_{xy} = \frac{p_z}{2k}
\]

Regarding the forces on edges and valleys, the tensile forces on the outer edges of the umbrella/tympan increase from zero at the corners and reach a maximum at the center. Similarly, compressive forces on the valleys increase from the edge and reach a maximum at the column. These maximum tensile and compressive forces \(T_{\text{max}}\) and \(C_{\text{max}}\), respectively can be calculated by one of two methods: membrane theory or the horizontal cantilever method [3]. The results of membrane theory and the horizontal cantilever method are the same.

The maximum forces in edges and valleys via membrane theory can be calculated with the equations (3) and (4). The membrane theory approach multiplies \(N'_{xy}\) by the length of the member. On the other hand, the equations of the horizontal cantilever method are (5) and (6). These equations are based on the equilibrium of the structural system composed by the horizontal edge and the valley and a concentrated resultant load \(F\) that represents the force acting over the shaded area of the Figure 6 located at the center of this area.

\[
k = \frac{c_0}{a_0 \cdot b_0}
\]
In order to verify the design by aforementioned equation, is necessary to perform an analytical structural model. The modelling of the timber structures, such as beams and columns, has been modelled by 1D elements (frames) and the photovoltaic panels by 2D elements (shell). Regarding the definition of boundary conditions of the structural model, it has been into account the real stiffness of the structure on which the pergolas are supported with elastic supports (springs).

The following two basic verifications that allow verifying the structural feasibility of the solution are based on the results of the detailed structural model:

- Limit the maximum vertical deflection along the cantilevered perimeter under permanent loads plus variable loads for the Serviceability Limit State of Deflection. The limit value adopted for the instantaneous deflection for cantilever members is \( L/150 \), according to clause 4.3.3.1 of CTE-DB-SE and Table 7.2 of Eurocode 5 EN 1995-1-1, where \( L \) is the cantilever length. [5], [6]

- Design the cross-sections of the main members of the structure for the Ultimate Limit State of Strength. The sections detailed analyzed are the columns for combined bending and axial compression; and the main beams of the hypars for combined bending and axial tension.

\[
T_{\text{max}} = N'_{xy} \cdot L_{\text{edge}} \tag{3}
\]
\[
C_{\text{max}} = N'_{xy} \cdot L_{\text{edge}} \tag{4}
\]
\[
T_{\text{max}} = \frac{F_x}{2 \cdot c_q} \tag{5}
\]
\[
C_{\text{max}} = 2 \cdot T_{\text{max}} \cdot \frac{L_{\text{valley}}}{L_{\text{edge}}} \tag{6}
\]
3 Conclusions

After analyzing the structural behaviour, the new proposal has been considered technically valid and constructible according to the results obtained from the structural analysis of this conceptual design.

The proposed structure is lighter, more efficient and has a singular aesthetic due to the shape of the hyperbolic paraboloids. In addition, it is more sustainable and environmentally friendly. On the other hand, the structure was designed with wood as the structural material, which is a great innovation in the design of a large-span roof with timber structure, such as this pergola located above the intermodal station of La Sagrera.

The loads transmitted to the roof are significantly lower than those of the preliminary reference design. The main reason is the huge difference in self-weight between wood and steel, reducing the amount of material required to resist the stresses by the efficiency of the adopted structural geometry.

The reduction of the vertical reactions due to the permanent loads of the structure's self-weight and the weight of the photovoltaic glass is approximately 75% for the pergolas over skylights and 65% for the pergolas over the high-speed station. It should be noted that the weight of the photovoltaic panels finally considered is 30.00 kg/m², which double the weight is considered in the preliminary reference design.

The reduction of the total vertical reactions for Ultimate Limit States is approximately 50% for the pergolas over skylights and 35-40% for the pergolas over the high-speed station. Besides increasing the weight of the photovoltaic glass, a uniform vertical wind pressure of 2.00 kN/m² has been considered for all the pergolas. In the preliminary reference design, a vertical wind pressure of 0.50 kN/m² was applied to more than 80% of the pergola surface.

The structural design has managed to meet the landscape requirements and the architectural concept, even providing great aesthetic value with the use of a double curvature geometry evoking an arboreal shape. The use of double curvature geometry for structural elements implies that the structural essence is explicitly exposed. This will reinforce the architectural objective of simulating a tree shape and will attract the attention of pedestrians and users of the multimodal transfer center of La Sagrera.

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References

F.E.W. structural prototypes: retrofitting residential buildings with ecological rooftop infrastructures

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Abstract
This contribution relates to the ongoing research “A Pilot Study of Integrated Resource Management on Rice University Campus: Towards Sustainable Urban Food, Energy, and Water (F.E.W.) Infrastructures” developed by the Architecture and the Civil and Environmental Engineering Departments at Rice University with external contribution of ETH Zurich and the Technical University of Munich. The objective of this research is to unfold structural design strategies to retrofit existing rooftop areas on residential buildings by embracing new technologies and generating active spaces for community use and social interaction. The main design strategy is the design of rooftop infrastructures for stormwater collection. These structures, in addition to creating new spaces for the community, will offer the opportunity to reuse collected water for agricultural irrigation systems. Indeed, this strategy is connected with the implementation of urban farming and agricultural landscapes to promote sustainable models of production and consumption. In this context, a modular system based on the combination of post-tensioned hollow ceramic components with a funnel-like bending-active membrane structure for water collection is currently under development. This paper highlights the context of the research project and presents the ongoing design of the rooftop prototype.

1 Introduction
As demand for food, energy, and water (F.E.W.) resources increases—along with the global population—ensuring long-term sustainable provision represents a massive challenge. This challenge is expected to play out more dramatically in urban areas, where 75% of the world’s population will be living by 2050 [1]. Existing urban F.E.W. systems are intricately linked with the natural environment and the existing models of consumption, and are currently under great physical, economical, and demographical stresses due to continuing population growth and urbanization, climate change, aging of the infrastructure, and extreme weather events. In the presented research project, we seek to assess the viability and effectiveness of rooftop infrastructures in providing more sustainable and resilient solutions for F.E.W. systems. The paper evaluates strategies for implementing structural systems and contextualizes them within a cultural, environmental, and architectural framework. Using Rice University’s Martel Residential College in Houston as a testbed, a pilot project is presented that proposes sustainable design strategies to retrofit existing rooftop areas on residential buildings, with the potential for further implementation across different urban and programmatic contexts. The project is currently under development at the
Department of Architecture and Civil and Environmental Engineering at Rice University in collaboration with ETH Zürich and the Technical University of Munich, along with the fabrication consultants Cerâmica Cumella.

2 Food, Energy and Water infrastructures

Individually, food, energy, and water (F.E.W.) systems can serve to increase resilience and mitigate the effects of scarcity and climate change. However, while considering food, energy, and water separately generates certain benefits, combining the three systems into an interconnected, circular system ensures that there is less or no loss of resources. For instance, the harvested rainwater could be used to irrigate crops on an urban farm, which in turn produces food for the local servery (reducing the energy needed for transportation), and eventually the food waste from that same servery becomes the compost for further planting, thus closing the cycle.

Our proposed F.E.W. rooftop infrastructure questions the prevailing paradigm based on the addition of multiple rooftop mechanical systems, energy systems, and water infrastructures such as solar panels, heating and cooling machines or water tanks (Fig.1). Alternately, our proposal integrates multiple systems following a holistic approach and aims to advance the conception of F.E.W. systems. Ultimately, the aim of the proposal is to develop a modular structural system with the potential of proliferating and being implemented on multiple rooftops.

![Rooftop water tanks in SoHo, New York (image by Frank Schulenburg)](image)

3 Rooftop infrastructures on residential buildings

In residential buildings, rooftops are most commonly dedicated to the placement of technical systems and mechanisms that make the building sound and operative from a functional perspective. However, solar panels, antennas, water tanks, elevators, electrical rooms, heating and cooling units or wind turbines collapse the rooftop’s area, generating, in turn, artificial landscapes with very low qualities from an architectural and urban perspective and with a poor result in terms of the integration of functional requirements and spatial qualities.

Considering the exponential growth of urban areas and the increasing lack of public spaces in city centres, the rooftop becomes an essential space to be reassessed and reformulated from engineering and architectural perspectives under a holistic and collaborative research model. In fact, rooftops on residential buildings could be regarded as privileged spaces with an enormous design potential for different reasons. On the one hand, as fundamental part of the building envelope, they have a prominent role in the design integration of environmental parameters such as solar radiation, rain and storm water collection, or wind circulation. But also, the rooftop is a common area in which residents and neighbours can interact and enjoy a space privileged with views, fresh air and natural light. In fact, there are some important references in architecture history that gave relevance to rooftop areas and
opened new visions and concepts to their redefinition. One of the most significant examples is the “Unité d’Habitation”, designed and built in Marseille (France) by the Swiss architect Le Corbusier in 1952. Motivated by an unprecedented need for housing after World War II, Le Corbusier conceived this high-rise multi-family residential housing project for the people of Marseille that were affected by the bombings in France. The “Unité d’Habitation” conveys an idea of communal living in what he described as a “vertical garden city” and one of the most striking concepts of the proposal was the innovative approach to accommodate communal spaces—the majority of them on the roof [2]. The rooftop design, conceived as a garden terrace, included a running track, a club, a kindergarten, a gym and a pool (Fig. 2). In Spain, Francisco Javier Sáenz de Oíza developed a similar concept in his project for “Torres Blancas” in Madrid (1969) where the impressive concrete structure is crowned with a communal area for the residents including a rooftop pool [3].

The precedents by Le Corbusier and Sáenz de Oíza, can be regarded as exemplary works that proposed an ambitious and harmonic integration of structural solutions and architectural questions that nowadays demand a thoughtful reassessment. Indeed, in recent years cities like Rotterdam, New York or Barcelona have been developing new urban policies promoting the activation of rooftops to maximize social, environmental and energy efficiency in existing and new buildings [4]. In addition to that, the emergence and availability of contemporary technologies and materials invite us to reformulate rooftop infrastructures through the lens of the potential implementation of integral building strategies and lightweight structural systems: more porous, more responsive, more ecological and more resilient.

Fig. 2 Unité d’habitation, Marseille. 1945-52. View of the model of the roof terrace. (source: Fondation Le Corbusier, Paris)

4 The potential of hollow structures in architecture and structural design

Within the domain of structural engineering, porous structures have been traditionally understood as the result of an operation of hollowing out of load-bearing elements in order to remove unnecessary material to produce lighter, and as such, more efficient structures. This position was effectively described by the structural engineer Pier Luigi Nervi as “the method of bringing dead and live loads down to the foundations … with the minimum use of materials” [5]. Similarly, the French engineer Robert le Ricolais asserted: “The art of structure is how and where to put holes: to build with holes, to use things which are hollow, things which have no weight” [6]. In addition, from an architectural perspective, a selection of interesting references that bear witness of this approach can be found in the figures of the Spanish architect Miguel Fisac and the Italian architect Marco Zanuso.
Miguel Fisac, in collaboration with the civil engineer Ricardo Barredo, explored innovative solutions based on the concept of structural hollowness using post-tensioned systems. Fisac regarded prestressed and post-tensioned concrete as “the only petreous material that works correctly in lintelled structures” [7]. Based on the analysis of the limitations of standard concrete structures, Fisac developed his investigations on light and hollow concrete beams inspired by the morphology of animal bones. In particular, Fisac designed several typologies of hollow concrete beams which form emerged out of the interaction of structural, environmental and architectural parameters (Fig. 3, top). Moreover, by hollowing out the sections, the aim was not only to reduce the self-weight of the structure but also to provide solar control and modulate thermal insulation properties, to collect and channel rain water and to incorporate natural light into the interior spaces [8]. In addition, the post-tensioned solution developed in collaboration with the civil engineer Ricardo Barredo was conceived to cover large structural spans, conferring great spatial freedom for the architectural design. The result of these multiple considerations materialized in his series of bone girders with complex sections and sophisticated profile is described by Carlos Asensio-Wandosell as “pure sublime form presented for our visual delight” [9].

Similarly, the Italian architect Marco Zanuso developed several projects working with prefabricated hollow profiles that, similarly to Fisac, were conceived to integrate architectural, structural and additional functional aspects. Zanuso advocated for the capacity to define a building “integrally”. According to him, the role of the structure should not be limited to its load-bearing function; in fact, structure should also serve additional functions such as incorporating lighting devices or housing mechanical systems [10]. One of the most interesting examples is found in his project for the Olivetti Factory in Buenos Aires (1954-1961). The structure of the factory was solved with prefabricated pre-stressed concrete hollow beams (48m long) supported by columns every 18 meters [11]. Additionally, the hollow girders were designed to integrate in their interior (93 cm diameter) duct-space for air conditioning distribution (Fig. 3, bottom) and to incorporate both natural and artificial lighting through their cantilevered fins [12].

In conclusion, the works of Fisac and Zanuso, among other prominent figures such as Alejandro de la Sota, João Filgueiras Lima (Lelé), and Jørn Utzon, hold significant value in the investigation into the integration of roof structures with water and energy systems, and provide relevant examples of a harmonic dialog between architectural, engineering, and ecological considerations. The design and conceptualization of F.E.W. rooftop infrastructures require this holistic approach and multidisciplinary model of collaboration. In the following chapter we will describe the proposal developed as a research collaboration. The proposal is presented at its conceptual phase. Material tests and structural calculations are currently under development and the results will be presented in future research publications.

5  F.E.W. Structural Prototypes

The objective of this proposal is to unfold sustainable design strategies to retrofit existing rooftop areas on residential buildings by embracing contemporary materials, tools and technologies. One of the main design strategies is based on rain and storm water collection to help with flood mitigation and, potentially, to reuse collected water for irrigation purposes. This operation is developed in parallel with the implementation of urban farming methods in order to generate agricultural landscapes that promote sustainable models of production and consumption in urban areas. In addition, these “urban shelters” represent an opportunity to rethink conventional rooftop infrastructures and create contemporary spaces that enhance social interaction and public health. Therefore, the aim of this proposal is to serve as a test bench to showcase ecological models of circular construction, consumption and social interaction.

The proposal is conceived with the potential of proliferating and being implemented on multiple rooftops, with Rice University Campus operating as a testbed. The selected site—the rooftop of Martel Residential College—is located at Rice University in Houston, Texas. Completed by Michael Graves in 2003, Martel College comprises an area of 107,032 square feet, serves a total of 232 beds for undergraduate students at Rice University, and stands at 4 floors tall. Structurally, the building has a syncopated concrete column grid that varies between 5.92 and 4.65 meters spans. The existing column grid operates as the basis for determining the structural rhythm of the proposal. While its rhythm is determined functionally (from the structural and infrastructural necessities of retrofitting the proposal to the roof), architecturally, it also begins to suggest an aesthetic relation to the Rice University campus.

The proposed modular system is composed of three main parts: a series of funnel-like bending-active membrane structures, post-tensioned hollow ceramic columns, and a floor system made of ceramic and pre-cast concrete blocks (Fig. 4, left). Each of these parts plays a different role in the overall system. Rainwater is captured through the expansive surface of the bending-active membrane structure. Working as a funnel, this structure channels the collected rainwater down to the post-tensioned hollow ceramic column and eventually to the blocks in the floor system. The use of bending-active structures allows for extremely lightweight, yet structurally sound solutions in which minimal amounts of material are necessary. The overall stiffness of the structure is generated by controlling the bending stresses within elastic beam-like elements, which in this case are materialized using bamboo rods. From the membrane structure, rainwater then flows through the column, before being diverted to additional pipes that bring it towards the different modules of the floor system.

The column acts as a water collection pipe built of a set of custom-made ceramic pieces. The cross-section of such pieces (Fig.4, right) was designed to facilitate an interlocking and watertight sealed joint between the pieces as well as the post-tensioning steel cables that guarantee the overall stability of the assembly. In fact, thanks to post-tensioning (Fig.5, left), it is possible to assemble each column using relatively small and light-weight hollow ceramic pieces (50 cm in height). These pieces can be easily produced, transported, and connected one to each other on site in order to reach the desired column height. Each column has two post-tensioning cables placed 16 cm apart one from each other. Despite the limited lever arm between the cables, by calibrating the post-tensioning forces it is possible to guarantee the desired stability against horizontal loads for the overall structure. To prevent cracks due to compressive stress peaks that originate at the interface between two pieces it is necessary to introduce a EPDM rubber expansion joint, as shown in Figure 5. Moreover, compressive forces coming from the post-tensioning cables have to be evenly distributed on the cross-section of the ceramic pieces where these forces are introduced within the column. For this purpose, a rigid steel cap is used at both extremes. In conclusion, the post-tensioned hollow ceramic pieces have a two-fold function acting both as structural columns and water pipes and, so, combining their static function with their potential activation as thermally-active surfaces.

The base of each column consists of a pre-cast concrete block, which is embedded within the floor system. Along with these foundation blocks, the floor system includes ceramic blocks, pre-cast concrete soil boxes and pre-cast concrete water tanks (Fig.4).

The foundation blocks fulfil several functions: on the one hand, their hollow sections facilitate an effective distribution of the collected water and, on the other hand, they serve as counterweight to balance out the above columns and the funnel-like membrane structures. In this way, the overall system becomes self-supporting and it does not require any substantial mechanical connection with the existing rooftop structure. In the proposed system, rainwater is conveyed inside the hollow ceramic blocks for cooling purposes, to the soil boxes for irrigation, and to the water tanks for storage (Fig.5, right). The fabrication of all the ceramic pieces and the design of the connecting joints is being developed in close...
collaboration with Ceràmica Cumella in Barcelona (Spain); the design and development of the light-weight membrane structures is part of a potential collaboration with the Swedish School of Textiles in Boras, Sweden.

The water management system generated by the proposal works to alleviate the effects of flooding and drought through storage and controlled release, and reduce runoff by capturing water and redirecting it into irrigation tanks. The materiality of the modular floor, with its high thermal inertia, also provides energy-saving cooling benefits to Martel College below. Finally, the system incorporates soil as an essential material that serves to absorb water and grow food for the student community. Regarding its potential implementation on different rooftops, this precast system provides substantial advantages in term of transportability, assembly and disassembly as well as strategies for reusing and adapting to different scenarios. The modularity of the structural elements, the proposed assembly method based on post-tensioning techniques, and the geometry of the ceramic pieces assembled through interlocking connections, facilitates a flexible construction combining sound and efficient structural solutions with construction principles based on circular construction.

The resulting hybrid prototype integrates food, energy, and water infrastructures into a minimum loss and minimum waste circular system, while helping to foster a sense of community, ecology and social interaction. Furthermore, while adapted to the specific irregular structural grid of Martel College (Fig.6), lends itself to operate as a prototypical flexible module that is able to be deployed onto other rooftops at Rice University and beyond. In fact, a prototypical adaptation of the structural concept presented in this paper is currently under development and has been selected to be built and exhibited as part of the forthcoming Seoul Biennale of Architecture and Urbanism 2021.

![Fig. 4 left: 1.modular floor system composed by ceramic blocks, 2.pre-cast concrete soil boxes, 3.pre-cast concrete water tanks and 4.pre-cast concrete column foundation blocks. 5. ceramic columns, right: Hollow ceramic column, section and geometry.](image)

### 6 Conclusions

The purpose of the presented research is to generate opportunities to boost interdisciplinary design approaches and bridge the gap between architecture and engineering. In doing so, we hope to explore new technologies and generate active and contemporary spaces for community use and social interaction. While the proposal presents an early stage within a long-term research process, as we continue forward, we will further develop the research on rain and storm water collection and increase the specificity of our understanding of its applications for flood mitigation and agricultural irrigation systems. This will include a quantitative evaluation regarding the capacity of the proposed modular structure to cover the varied demands for F.E.W. resources in specific urban contexts. In addition, we will continue to expand our findings on urban farming and agricultural landscapes—and the possibilities they provide as sustainable models of construction, production and consumption. Lastly, we will engage with the Rice community to further our understanding as to how F.E.W. infrastructures can enhance social interaction and public health.
Fig. 5  left: detail of the post-tensioned ceramic pieces assembly, right: visualization of the F.E.W. structural system proposed as a prototypical solution.

Fig. 6  Visualization of the implementation of the F.E.W. structural system proposed on the rooftop of Martel College at Rice University.
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References


Structure as source of syntactic ambiguity in contemporary architecture

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Abstract
In contemporary architecture, structure has become part of the explicit language of the building, often acquiring a somewhat-foreseen relevance yet conveying ambiguous visual information. This article discusses how structural elements participate in the syntax of contemporary architecture and whether the interface structure-space is becoming a source of syntactic ambiguity in contemporary architecture. Using selected case studies, the concepts of transparency, gravity-less, and cantilevering are evaluated first as sources of aesthetical, functional, and semantic value in architecture, and then as sources of innovation in structural design. Insights suggest that the aesthetical contribution of the structure to the architecture goes beyond the mere physicality of the structural solution.

Keywords: conceptual structural design, transparency, gravity-less, cantilever, contemporary architecture, structural morphology

1 Introduction
In linguistics, syntactic ambiguity occurs when one sentence can be interpreted as having more than one structure, resulting from the fact that different meanings can be attributed to the same sequence of words. Syntactic ambiguity arises, thus, from the interpretation of the underlying relationship between the words of a sentence. If architectural works are spatial compositions and as such, derived from a narrative materialized by physical elements, it can be assumed that those elements are organized with the ultimate intention to convey a certain meaning. Furthermore, it is valid to question what are the syntactic components of a contemporary architectural language and how clear is the meaning they provide. On the other hand, with the advent of new materials and advanced technology, the search for stronger, taller, or longer structures becomes more ambitious. Within this landscape of myriad challenging trends, contemporary architecture increasingly relies on technical solutions that are both defined and limited by the structure capacities. In doing so, structure becomes part of the explicit language of the building, often acquiring a somewhat-foreseen relevance yet conveying ambiguous visual information. This article discusses how structural elements participate in the syntax of contemporary architecture and whether the interface structure-space is becoming a source of syntactic ambiguity in contemporary architecture. By analysing the structural morphology of selected case studies, built in the last 20 years, common architectural concepts are examined in function of their design effects over the structural morphology. First, the concepts of transparency, gravity-less, and cantilevering are evaluated as sources of aesthetical, functional, and semantic value in architecture. Then, such design problems are revised as sources of innovation in structural design, i.e. how the addressed architectural concept challenges the structural design of the building. In this study, however, the focus is on the structure, which is understood as the array of all structural elements organized to transfer loads to the ground. This definition separates structure from construction, which is instead understood as how building components are organized to configure architectural space. Hence, in order to unveil the primarily role of the structure, the rationale behind the building shape must be exposed. In most cases, this entails divesting the building from its envelope and often a careful look at its construction process. Once the structure is exposed, one can clearly note that the shape of the building is not just a mere aesthetical play, but the optimal response to a demand of structural capacity when cantilevering or spanning over large free areas. Similar procedure is applied in this study, to evaluate the structural solutions according to their physical behaviour and aesthetical contribution to the architecture.
2 Transparency as source of structural innovation

2.1 Transparency in architecture

In their seminal essay about transparency [1], Rowe and Slutzky define literal transparency as the material quality of allowing the light passing through, i.e. of being see-through, and phenomenal transparency as ‘the perceptual quality that allows the mind to discern the underlying governing concept or spatial concept’ [2]. Transparency in architecture, thus, can be ‘understood as the dematerialization of the building envelope, the use of open form, or the confluence of form and meaning’ [2]. According to [3], transparency in architecture can be achieved by permeability or reflection. The former defined by boundaries acting as filters or openings, through which the light can pass or views can be framed. The later defined by surfaces or solids that, acting as mirrors, can create the sense of lightness. From a technical perspective, transparency in architecture is almost indivisible with the use of glass. Although already applied in gothic cathedrals, the glass acquired the relevance as architectural element when mass produced at the beginning of the 20th century. As spoken material, glass advanced the modernist’s social agenda by encouraging transparency as means to remove societal barriers, blurring the boundaries between private and public spaces [4]. In the post-modernism, transparency was considered as mere physical property and thus, architects sought to use it to explore less literal experiences of space or translucency [4]. Architectural transparency can also be a metaphor for democracy, achieved by large and open spaces without physical barriers, and characterized by visual continuity and connections [5].

2.2 Transparency and the structural ambivalence

The selected case study is the Suvarnabhumi Airport, built in 2006 in Bangkok, Thailand. The total area of the airport is 593,000 m² and it houses 45 million passengers per year. The design concept of the passenger terminal according to its architect Helmut Jahn, is to put passenger circulation before aircraft circulation [6]. To achieve this design goal, the main terminal building is located under a large steel roof designed by Werner Sobek Engineers. The central roof structure consists of eight super truss girders which are combined with two smaller truss and supported by 16 frame type steel columns (a.k.a. pylons), covering an area of 567 m length and 210 m width. Distance between the truss girders is 81 m, while central span is 126 m. Also, the roof has 42 m cantilevers on both sides. Each super truss is 210 m long and 9 m wide. In order to connect two super trusses, secondary trusses are used in grid form which carries the loads from the roof and the glass façade (Figure 1a). The shape of these main girders follows the distribution of bending moments, hence increasing its depth at the major moments and minimize it when the moments are reduced. The girders were designed with triple chords, where two chords are working in compression and the remainder working in tension (Figure 1a).

The roof appears to be floating while offering a large shading area. To avoid unobstructed circulations, few pylons were used to support the roof. The design philosophy of exposing the interior spaces to the eye of passengers when approaching from afar, led to a fully transparent façade. The 40 m high glass main façade of the main terminal is supported by cable-stayed cable posts and pre-tensioned cable trusses [7], guaranteeing few visual obstructions due to structural elements. Sun-shading elements were avoided thanks to ‘the shading that the roof provides’ [7]. Hence, transparency as a design concept in the Suvarnabhumi Airport is achieved by means of the building envelope alone. The glass façade, working as the urban face of the roof, symbolizes the ‘contemporarity’ of the large-scale building and connotes the openness of the space: inner activities are on display and there is no boundary between public and private areas. Accessibility is materialized by the transparency of this façade, whose operation resembles the fascination of Modern Movement ‘with making interior and exterior space continuous’ [2]. Whilst this transparency works at the urban scale, exposing the reflections of the airport’s inner constant movements towards the city, the vast magnitude of the space underneath the roof diminishes the effect of transparency as symbol of exposure and public continuity (Figure 1c). The users are in a constant flow, dealing with the overload of information in display and going through the sometime endless checking and security scans of the airports. The transparency of the façade is a mere object of literal light-passing-through and there is no room for phenomenological interpretations.

The airport structure also includes curved concourses, designed for the users to circulate towards and from the gates. These structures include a translucent upper portion and a transparent lower portion, where passengers, whether circulating or waiting, can observe the incessant activity of the aircraft gates, aprons, and runaways (Figure 2a). Here, transparency is offered for the user as sign of accountability and scrutiny over the actions performed in those areas [8]. Perhaps, as a counterbalancing effect of
being under constant surveillance through cameras, the curved concourses provide passengers with a sense of security and relief by showcasing the aircraft and staff activities.

Figure 1. (a) The central roof structure, (b) diagrams showing the girders depth defined the bending moment diagrams, and (c) passenger terminal check-in overview (Photo source: commons.wikimedia.org/wiki/User:Mattes)

The transparency from inside of the curved concourses finds a fortunate match in the conceptual design of these structures, whose repeating components are meant to resemble waves over the water. The structural design of the curved concourses comprises triple-chord lattice arches spanning 42 m (Figure 2b). The semi-elliptical structures are approx. 50 m wide and 25 m high [7]. Thanks to 5-pin arches, featuring an equivalent triangle-based array, the loadbearing system provides stability for both vertical and lateral loading (Figure 2c). The semi-elliptical section of the concourse building helps to emphasize a sense of watching-over, whilst fabric membranes are placed between the arches, covering the circulations areas [9].

Figure 2. (a) Interior view, (b) triple-chord lattice arches of the concourses, and (c) semi-elliptical section displaying distribution of vertical loading (Photo source: flickr.com/photos/koolgary/2462111376)
3 Gravity-less as source of structural design innovation

3.1 Gravity-less in architecture

According to [10], the noun *gravity* entails the sense of seriousness and importance, whereas as in physical terms, it defines gravitational attraction of an astronomical body for bodies at or near its surface. The adjective *less*, on the other hand, defines the condition of being destitute of or being unable to be acted on in certain manner. Hence, the concept of gravity-less in architecture can be understood as the quality of being destitute of a sense of seriousness, and / or the quality of remaining unaffected by the gravitational attraction of our planet. Although this latter quality is not yet physically possible to achieve, the sense of gravity-less can be achieved in buildings that challenge the perceptions and logic of a conventional structure. The perception of being unaffected by gravity can be triggered by structures challenging both the scale and the span of conventional buildings. In this regard, the use of structural elements such as cables and membranes can reinforce the perception a ‘floating’ structure, e.g. the oval tent-like glass roof over the Sony Plaza in Berlin; the use of strategically-located openings introducing daylight through the domes of mosques can create an ethereal atmosphere inside structures which would be otherwise rather defined as massive and heavy.

3.2 Gravity-less and the spectacle of the large ceilings

The selected case study is the Mercedes-Benz Stadium, completed in 2017 in Atlanta, USA. It was designed by HOK architects and BuroHappold Engineering. The most defining feature of the stadium is the large-span and retractable roof, which is used to cover football games and other major events in the city. Following the owner’s desire to create an architectural icon for the city and provide a remarkable experience to the local sport teams, the design team proposed a roof inspired in the form by the Pantheon’s oculus, but with the innovation of appearing ‘to open and close like a camera aperture’ [11]. The design team’s goal was to create an iconic, high-performance stadium and take a new approach to the movement of typical double-panel retractable roofs, as well as create a large scoreboard without obstructing the view of the sky. The fixed roof structure of the Mercedes Benz Stadium is a two-way steel box-truss system. Because the roof opening is not a circle, but rather a segmented oval to ‘better reflect the field below’ [12], the resulting nonorthogonal layout of fixed trusses was solved using the principle of reciprocal frames (Figure 3a). Primary trusses are 220 m long and 21.5 m deep. Four secondary trusses complete the framing of the opening and provide support to the additional operable structure, which consists of eight cantilevered petal-like structures and a LED video display as a screen structure. Trusses are fixed at one end to accommodate changing temperature, wind loads and petal movement. Due to the 17,000 tons self-weight, the load of the roof system is transferred through 19 reinforced concrete mega columns, each 54.5 m tall and up to 8.5 x 4 m wide [13]. In contrast, its predecessor, the Georgia Dome had a total self-weight of about 1,300 tons, whilst spanning 234 m x 186 m [14]. This world’s first hypar-tensegrity dome (Figure 3b), unfortunately demolished in 2017, was composed of a ring beam, 52 floating columns and a triangulated network of cables [14].

Figure 3. (a) The fixed roof structure of the stadium, designed according to the principle of reciprocal frames and (b) model of hypar-tensegrity roof of the Georgia Dome
The purpose of using a large-span roof is providing uninterrupted visibility and flexibility, so this multi-purpose stadium can be reconfigured according to the type of event it is hosting. From a structural perspective, the roof of the stadium is composed of two systems: the fixed and the operable structures. The operable system is both kinetic, for the purposes of opening itself to bring light in, and digital, for providing the spectators with the visual amusement of the digital screens too. However, in the attempt to bring the show up to the heights of the roof, the fixed structure invariantly remains defined by gravity, as if the ideas behind the design of this heavy roof were entangled with another gravity-less architecture; the one resulting from digital visualizations encouraging expressive contemporary skin-forms, which, nevertheless fail to embed the manufacture and fabrication aspects in the design process [15]. Amplified by the enormous scale of the stadium, the roof becomes a massive ceiling, whose sole purpose is supporting a parallel focus to the game (Figure 4, above). Spectators are saved, nonetheless, from spatial disorientation due to the strong presence of the game field, which defines the floor and its inherent cardinal direction. Afterall, ‘architecture cannot divorce itself from the effects of gravity’ and up is tied to the ceiling as down is tied up to floor [16].

Interestingly enough, the former tensegrity roof of the Georgia Dome displayed a sense of effortless whilst spanning the field (Figure 4, below). The membrane is translucid; thus, no additional kinetic system is need to provide light to the field below. The network of cables is almost imperceptible within the scale of the stadium, increasing the sense of a ‘floating’ roof. Hence, aiming at a gravity-less roof structure, the designers of the hypar-tensegrity dome responded by taking the weight out of the ceiling, releasing it from the effects of gravity and adding to the spectacle, by providing light. On the contrary, the design of the new roof structure attempts to hold complementary kinetic and digital focii, which demand the support of a massive and heavy ceiling. Although expensive and carefully designed, such structure is not relevant to the architectural space and only works for load-bearing purposes.

Figure 4. Above: interior view of the Mercedes-Benz stadium; Below: interior view of the Georgia dome (Photo Sources: es.wikipedia.org/wiki/Mercedes-Benz_Stadium, commons.wikimedia.org/wiki/File:Georgia_Dome_2008-08-30.jpg)
4 Cantilevering as source of structural design innovation

4.1 Cantilevering in architecture
A cantilever is a beam or member supported at only one end. It can be also applied as a transitive verb entailing the action to either support by or build as a cantilever [10]. Cantilevering in architecture refers to the design of buildings or components protruding from a surface, which often acts as its structural support. The use of cantilevers in architecture is often associated with the need of avoiding columns or vertical supports in a certain area, i.e. improved functionality, and with emphasizing a sense of a ‘flying-over’, due to visual connections or intended formal hierarchies in the overall building design.

4.2 Cantilevering and visual permeability
The first case study is the Villa Méditerranée, a multi-purpose building completed in 2013 in Marseille, France. Placed near the water’s edge, the most remarkable feature of this building is its unique cantilever, which, according to Stefano Boeri Architects, ‘in addition to overlooking the square seems to want to extend its projection over the breakwater. However, [it] is not trying to overcome the sea but rather to envelope it” [17]. Thanks to the space underneath, the sea and the seashore are visually included in the spatial experience of the building, simultaneously defining an area offers a useful public space for various activities and small pleasure boats. For AR&C Engineers, the technical challenge was the designing the 36 m long cantilever placed 14 m above the water level, maintaining the balance of the building, and simultaneously avoiding excessive deformation due to the 4,000 ton of the overhang. In addition, due to the design concept of being over the water, the foundations had to be built 15 meters below sea level. Thus, to decrease the weight of the cantilever, four steel trusses were supported in a massive frame structure (Figure 5a), following the logic of the bending moments distribution (Figure 5b). The low weight of the structure and the excessive flexibility of the structure, however, provoked resonance problems due to the oscillation of the building. The solution was using concrete cladding, which integrated with metal plates were anchored to the cantilever to increase its stiffness and building mass (Figure 5c). By making it heavier, the building oscillations were reduced.

Figure 5. (a) The cantilever truss structure of the Villa Méditerranée, conceptualized from (b) the cantilever optimal bending capacity distribution, and (c) the building envelope featuring strip windows.

The exhibition space inside the cantilever structure is enclosed by an envelope featuring long strip windows, which ‘offers a privileged viewpoint from which to observe the movements of the port’ [17]. The architects also claim that through walkable glasses on the floor, the presence of the water will be perceived from inside the cantilever space. The achievement of such interior-exterior visual communication is, nevertheless, arguable. The strip windows are overlapping the geometry of the braces, disrupting the aimed visual connection with the water and sea landscape (Figure 6a). Similarly, the triangulated nature of the trusses is affecting the functionality of the floor for exhibition purposes (Figure 5b).
The second case is the Busan Cinema Centre, completed in 2012 in Busan, South Korea. The building complex includes theatres, cinemas, offices, production studios, restaurants, and a conference centre. As a design idea, Coop Himmelb(l)au architects wanted to blur the boundaries between open and closed spaces and create an iconic ‘flying’ roof for the city [18]. B+G Ingenieure designed, thus, a 120 m long and 60 m wide roof supported by a single conic column, creating an 85 m cantilever. Given this size, the whole structure comprises separated sub-structures (Figure 7a). The roof itself consists of an orthogonal steel truss girder system with longitudinal and 5 m long transverse trusses. These members are connected to each other by rigid joints to minimize rotation for stability. The crown provides structural transition of high fixed-end moment from the roof to the double cone, through two double rings connected rigidly. The double cone acts as a vertical load-bearing element and transfers the load to the concrete shear walls by shear studs [19]. Unlike the Villa Méditerranée, in the cantilever structure of this centre only a small portion is habilitated as functional area. Thanks to Vierendeel trusses, an uninterrupted 3-story restaurant area was implemented in this portion of the roof (Figure 7b). Although the span of the cantilever in Busan Cinema Centre is 2.3 times larger than that of the Villa Méditerranée, in the former, the roof structure is massive and leads one to question whether this is the result of the formal challenge of supporting the roof on one column or a conscious design intention. In this regard, the clear structural logic behind the cantilever trusses in Villa Méditerranée appears as an advantage, compared against the design complexity of the irregular trusses in Busan Cinema Centre. Judging by the use of the lower face of the cantilever volume as digital display (Figure 7c), the use of freeform surfaces seems questionable as well, at least from an economical point of view.
5 Concluding remarks

Syntactic ambiguity arises from the interpretation of the structure in combination with other architectural elements. Whilst transparency is indeed achieved by means of architectural components, e.g. envelopes, these are possible thanks to the significant role of the structure. Gravity-less architecture added ambiguous significance to the studied roof structures, depending on whether they complement or become the spectacle themselves. Cantilevering structures, on the other hand, are syntactically ambiguous when it comes to the dialogue between the structure and visual intentions in architecture. Additionally, it should be noted that further critical analyses should question the use of thousands of tons of steel in some of these structures, with respect to environmental impact, embodied energy and carbon concerns.

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challenging gravity
The new Ponte Cabbiera Chapel in Val Malvaglia

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Link to the video: https://youtu.be/N5U-J7L-GPw
MAAT - Museum of Art, Architecture and Technology, Lisbon

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Abstract
The design of the building for the MAAT challenged the architecture and structural engineering teams to provide integrated and creative solutions to materialise the concept of a distinctive building blended with the landscape, while also providing flexible and fluid exhibition areas. This paper describes the approach to its structural design, namely the evolution of the solutions since conceptual design and the focus on proactively adapting them to allow the architects to freely implement the intended curved complex shapes without letting the structure take the spotlight.

1 Introduction
The new Museum of Art, Architecture and Technology (MAAT) is an outward-looking museum located on the banks of the Tagus river in Lisbon designed by British architect Amanda Levete and opened to the public in October 2016.

With a complex architecture based on curvilinear shapes, besides office and technical areas, there are three large and fluid exhibition spaces, which can work combined or totally independent: the Oval Gallery, the Main gallery and the Project Room. The main exhibition room, the Oval Gallery, consists of a double-height space that opens up from the main entrance through a ramped gallery along the winding curve that descends inside the building from the entrance hall to the ground floor.

Conceptually, the public pedestrian path along the riverfront is transformed, as it rises to the building roof from both extremities, into a large space that can serve as a multipurpose space with a 360º panoramic view that is merged with the surrounding landscape of the Electricity Museum and the riverside promenade.

The south-side cantilever and curved façade that distinguish this iconic building were clearly the highest challenge posed to the design team. This paper describes the various systems that were proposed for this part of the building and their evolution throughout the project as a result of a fruitful interaction between structural engineers and architects.

2 The initial concept
The initial architecture concept that established the base for starting the project is presented in Fig. 1. The building should allow visitors to walk over, under and through the building and is characterized by its curved shapes, especially the south façade that rises from the ground and advances in the direction of the river with an impressive cantilever. The exposition spaces should be open and flexible allowing the Museum to be filled with light, and thus, enhancing the visitors experience. To assure the intended fluidity with the pedestrian pathway along the river, the same granite cube pavement was extended to the roof. In contrast, the façade was proposed to have a bronze metal finishing.
The structural solution for the large south cantilever that was proposed by the structural engineers who advised the architects for the initial schemes is presented in Fig. 2. The cantilever is supported by several alignments of triangular structures that transfer the forces to the ground by a set of compression and tension struts crossing the Oval Gallery and the Main Gallery.

This solution allowed materializing the intended architecture concept for the exterior of the building, but imposed important restrictions the Oval Gallery.

### 3 Initial solution for the structure of the south cantilever and Oval Gallery

Even though the initial structural solution for the support of the south cantilever is effective in assuring the intended geometry for the building, the presence of the triangular shaped supports inside the Oval Gallery and the Main Gallery strongly affects the flexibility and fluidity of the exhibition areas. Also, from a structural standpoint two important issues arise:

- The high unbraced length of the compression struts implies using wide sections to provide an adequate buckling resistance.
- Some foundations are submitted to high tension forces; thus, a complex foundation solution shall be required.

At the initial stages of conceptual design an alternative structural solution was sought for this part of the building. The main pursued objectives were the following:

- The exterior shape of the building should be kept as it is the main feature that distinguishes it.
- The number of supports inside the exhibition areas should be minimized or ideally, fully eliminated.
- The structure for the roof should be adapted to its curved geometry to avoid a heavy filling with non-structural materials.
An alternative structural solution complying with these criteria was proposed. One single steel truss oriented along the longitudinal direction of the building positioned at the interior façade of the building, i.e., at the restaurant balcony (see Fig. 3 – left) is considered in replacement of the triangular shaped supports. It is divided in two spans of 40.6m and 25.1m with the intermediate column positioned at the separation between the restaurant and the main entrance. The structure in the perpendicular direction is composed by steel trusses evenly spaced at every 5.0m and positioned on the roof level following its shape. Their maximum spans are 24.6m between the back wall of the Oval Gallery and the main truss, and 17.1m as cantilever from main truss to the extremity of the roof at the south façade. Purlins at 2.5m centres span over the 5.0m between the steel trusses and serve as support for a composite slab on trapezoidal plate.

![Fig. 3 Alternative steel truss solution proposed for the south cantilever and Oval Gallery.](image)

Besides simplifying the structural concept for the building, the proposed solution is also able to eliminate all the visible supports inside the exhibition areas, while keeping the overall shape of the building. In contrast, the main steel truss could not be placed at the glazed separation between the inner space of the restaurant and its balcony, where it would be better integrated and less visible from the exterior. This would provide an important imbalance between the cantilever and inner span, making it difficult to control deformations at the junction with the façade. Even though the presence of the main truss in the restaurant balcony does not impair its intended use, it has an important visual impact from the exterior (Fig. 3 – right).

4 Final solution for the structure of the south cantilever and Oval Gallery

Before going forward with Detail Design, one additional attempt was made to fully hide the structure. Given the curved shape of the south façade and cantilever, the structural design team proposed a bold solution, where the main truss is replaced by an inclined arch. The arch could be fully hidden inside the curved shaped façade, while also allowing to use the wall separating the restaurant and the Oval Gallery as an additional interior support, which was not possible before due to the imbalance between the spans. The main steel truss could be removed and the spans of the perpendicular roof trusses reduced due to the introduction of the new interior support and the more favourable position of the support provided by the arch. The initial sketches with the proposed updates to the structural concept are presented in Fig. 4. The arch would span not more than 85m with a chord height of about 13.5m, resulting in a span to chord height between 6 and 7, which is considered adequate for a well-balanced structural performance.
As this proposal was considered to fit the intended architecture concept and to provide a significant improvement to the project, the study for the definition of the arch structure was continued. The following conditions to establish its geometry were considered:

- The arch shall be fully hidden inside the curved suspended south façade ceiling.
- Ideally, the arch shall be positioned within one single plane, even though inclined, to better control its structural behaviour. Also, its contour lines shall be perpendicular to the vertical planes defined by the secondary trusses.
- The shape of the arch shall follow the anti-funicular of the applied loads.
- The span of the arch shall be limited to the minimum required to avoid it to be visible.
- Due allowance shall be given to the requirement to provide the arch with adequate flexure resistance in its plane to sustain the live loads in an unbalanced configuration and to provide adequate buckling resistance.

The first step for the definition of the arch geometry was to identify the plane slope and its position along the direction of the secondary trusses, so that the arch can be fitted inside the suspended façade ceiling, while providing well balanced spans for the secondary trusses and an adequate chord height for the arch. The plane was set at an angle of 32.4º to the vertical and positioned as close as possible to the surface of the suspended façade ceiling without intersecting it. For the definition of the arch geometry within the inclined plane the starting points on the west and east side were fixed at levels +6.60 and +5.60 respectively (around the first-floor level), together with a free span of 73m. The highest point positioned roughly at midspan was set at +14.55 (right below the roof level).

The configuration of the arch is not circular or parabolic, but rather follows the configuration of the antifunicular of the applied loads. Thus, the arch is composed of straight segments with variations of slope occurring at the intersections with the secondary trusses. For the determination of the final geometry of the arch, the forces applied to the arch by each roof truss due to the permanent and the uniformly distributed live loads were first determined. By imposing the fixed coordinates and applying a graphic statics procedure [1], all the points at the intersections with the secondary trusses were determined so that the final configuration of the arch is the antifunicular of the applied loads (Fig. 5).

The estimated arch axial load obtained from the global finite element model was 35.1MN, which is close to the estimate obtained through the graphic statics method.

Due to space limitations, steel was favoured as the material for the arch. A hot finished (EN10210-1 [2]) circular hollow section with 711mm diameter and 60mm thickness (CHS711x60) in S355NH steel has been used. The option for a hot finished and tubular section is related to its higher resistance to buckling. It can be observed in Fig.6 – left that the arch fits well inside the geometry defined by the...
architecture team for the suspended façade. With the purpose to reduce the buckling length of the arch and to increase resistance to unbalanced loads, a truss was materialised in its plane by introducing diagonals, an upper chord at the level of the roof slab and a lower chord at the lower level of the suspended façade ceiling (Fig. 6 - right).

![Fig. 6 Final solution – arch inside the suspended façade volume (left) and complete trussed arch (right).](image1)

The arch supports are composed by 1.20m thickness RC walls which are fully disconnected from the surrounding structure and are oriented in plan according to the horizontal projection of the starter segments of the arch. Both walls are interconnected at the foundation level by a prestressed concrete beam with 2.10x2.10 cross-section that ties the outward-oriented horizontal forces introduced by the arch to the structure.

“Pratt” type steel trusses at 5.0m centres lean over the arch, having a maximum interior span of 14m between the RC separation wall and the arch, and a 12m maximum cantilever span south of the arch. Even though these trusses are on the same alignments of the Oval Gallery trusses, it was decided not to provide bending moment continuity over the RC wall so that the trusses remain isostatic, ensuring a more precise control over the vertical loads transferred to the arch. The structural configuration of the building roof and stability concept for the south cantilever are represented in the 3D model view from Fig. 7.

![Fig. 7 3D section from Revit model with structural concept for the roof.](image2)

As illustrated in Fig. 7, due to the insertion of the arch in an inclined plane, its stability relies on a horizontal force developing at each node which must be absorbed by the roof structure. This is achieved by directing these forces to the RC core walls at the back of the building through membrane forces in the roof composite structure (trusses and slab on trapezoidal plate). The connection of the roof structure to the RC walls is established by steel columns complemented by diagonal bracings (Fig. 8). This solution resulted from the decision not to extend the RC walls to the variable inclination roof slab, thus reducing the interaction between steel structure and reinforced concrete works.
Given the arch is laterally stabilized by the trusses that are supported by it, a temporary shoring in specific locations along its length was required until the completion of the roof. These locations were carefully studied during detail design, where the maximum reactions at each point were determined and dedicated foundation piles were included. The extremity of the south cantilever was surveyed during the removal of the shoring and 14 days after the end of the operation. The maximum deformation after the complete removal of the shoring was 18mm (lower than estimated), having stabilized at 23mm 14 days after with a portion of the roof revetments already in place.

IPE220 purlins at 2,5 centres span perpendicularly to the trusses and provide support to a 13cm thick composite slab on a trapezoidal plate. The purlins are inclined along their longitudinal axis and their cross-section is rotated to the vertical (Fig. 9 – right) so that the composite slab can follow the intended shape for the roof and, thus, minimizes the required filling over the slab. However, the upper chord of the trusses is not rotated to the vertical, because it would be impractical to assemble. Instead, it still follows the roof shape longitudinally, but the axis of its upper flange is lowered 2,5cm relative to the purlin level to avoid the slab thickness to be locally reduced above the trusses (Fig. 9 – left).

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5 The south façade

With the development of the project, the architecture team abandoned the initial idea of a metal finishing for the south façade for a solution inspired on the rich Portuguese tradition of handicrafts and ceramics. It consists of about 15000 pieces of three-dimensional ceramic mosaics producing a complex surface that offers different perspectives of light, water and shades. Its complex geometry required a highly adaptive steel structure to assure that each piece is positioned at the correct location and with the correct rotation. Three levels of structure were required:

- “Vertical” steel trusses at 5,0m centres which, depending on their location, are supported on the building concrete structure (east and west of the arch zone – Fig. 10 - right) or are suspended from the roof trusses (arch zone – Fig. 10 - left). To provide enough clearance, these elements are placed at a minimum of 30cm to the interior of the surface that defines the façade.
- SHS 100x5 purlins at 1,0m spacing connecting the vertical alignments and rotated according to the façade configuration (Fig. 10).
- Closely spaced aluminium profiles placed in vertical planes and supported by the purlins (Fig. 10 – left). Two aluminium profiles are provided per each alignment of ceramic pieces (one at the edge and one at the centre), which are fixed to the steel structure by specially designed adjustable clamps.

Fig. 10 South façade structure – “vertical” trusses and purlins suspended on the roof trusses (left) and supported on the concrete structure (right).

Fig. 11 South façade ceramic mosaics and adjustable aluminium profiles – details (left) and photo after construction (right)
6 Final remarks

The purpose was to build an iconic building in one of the most remarkable locations in Lisbon. The intended architectural concept was to integrate the building in the landscape, by gently extending the riverfront walkway into the building’s roof, allowing the public to freely walk over, under and through the building. In parallel, the curved shape of the building posed a significant challenge as it is complex not typical for buildings in Portugal.

The strict cooperation between the architecture and structural engineering teams and a correct understanding of each discipline’s needs since conceptual design phase and through the whole project was deemed fruitful. Even though the initial structural scheme considered by the architects was adequate to materialise the exterior shape intended for the building, the structural supports in the Oval Gallery and Main Gallery affected the flexibility and fluidity of the exhibition areas. To address this issue, an initial solution was proposed to include a steel truss along the main direction of the building that could replace those supports, liberating the exhibition areas from any visible structural elements. Even though this solution was able to keep the building’s shape and brought significant improvements to its functionality, the proposed truss positioned at the restaurant’s balcony had an important visual impact from the exterior, partially offsetting the fluidity of the curved façade. Finally, the truss was replaced by a large span arch fitted inside the façade false ceiling, thus optimizing the spans of the roof structure and most of all, being able to completely hide all structural elements inside and outside the building. This solution brought enormous advantages to the overall quality of the building, where at the end the structure is considered to effectively fulfil its main purpose of providing stability to the building without affecting the freedom of architecture to freely express its creativity.

Since it opened to public, MAAT is considered a landmark building featuring in various advertising campaigns, being awarded several architecture prizes and with the 2017 ECCS Award for Outstanding Steel Structures.

![Photo of the finished building.](image)

Fig. 12 Photo of the finished building.

References


Mohamed VI Tower in Rabat

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Abstract
With a total height of 250 meters, the Mohamed VI Tower in Rabat (Morocco), currently under construction, will be one of the tallest skyscrapers in Africa. The paper presents the different structural configurations considered at the conceptual design phase, that led to the solution finally adopted: a tube in tube configuration with an eccentric inner concrete core and an external Vierendeel frame, and composite structure for the floors. Besides, the paper addresses the main design requirements that had to face the detailed design phase: deep barrette pile foundation, serviceability criteria control that implied several strategies both in terms of design and analysis, and structure differential shortening control between the concrete core and the steel façade.

1 Project background
With a total height of 250 meters, the Mohamed VI Tower in Rabat (Morocco), will be one of the tallest skyscrapers in Africa. The building is currently under construction and will have a mixed use: residential, office and hotel [1], [2]. Architect Rafael de la Hoz, in collaboration with Hakim Benjelloun has undertaken the architectural design. Bernabeu Ingenieros, Ney & Partners and BESIX are responsible for the structural design and analysis. BESIX and Six Construct are also responsible for the construction works, in partnership with local contractor TGCC (Fig. 1).

Fig. 1 Exterior view (Rafael de la Hoz). The tower under construction.
The building is located at Salé, on the right bank of the Bouregreg River, in Rabat, and will be a key element in the Bouregreg Valley Development Project, that includes important urban initiatives, such as the Grand Theater and the House of Arts and Culture. The building includes the 250 meters tower, that has a spindle shape in elevation with an almost circular shape in plan with a varying diameter, and a podium, that makes around 200x120 meters in plan. Both structures have a common base-

2 Conceptual design and Structural configuration

The floorplan of the tower is organised by considering an eccentric disposal of the inner communications towards the South façade, generating a large open space at the North side. Indeed, the architectural intention was to concentrate the main uses of the tower (offices, residential and hotel rooms) in the North side, orientated towards the Bouregreg river, the Hassan Tower and the recently built grand theatre, while the South side, much smaller in surface due to the proximity of the core, houses all the technical rooms and services. The exterior façade shows this distinction, considering a transparent glazing facade at the North, and an opaque facade at the South that integrates solar collectors. This way, by considering a small technical area at every floor the existence of intermediate technical floors is therefore avoided, which constitutes a singular configuration that allows to maintain the continuity of the façade and enhances its verticality and slenderness.

Following the general layout and configuration of the floorplan, with the eccentric inner core and the spindle shape in elevation, an initial structural configuration was proposed, considering exclusively the contribution of the inner core to support wind and seismic horizontal loads. The floor decks were supported in the inner core and in the exterior façade, without intermediate support. The façade had therefore a structural role, but exclusively to support gravity loads, which gave great architectural freedom to its configuration, both in terms of façade composition and elevation in shape, since architecturally the intention was to reduce the diameter of the floorplan at the top, and also in the lower levels, according to the intended spindle shape. However, and although in terms of resistance and Ultimate Limit State analysis this solution was feasible, the inner concrete core stiffness was not enough to control lateral displacements and accelerations. The slenderness, height to width ratio, of the concrete core was around 15 in North-South direction and 12 in East-West direction, and it was proved to be insufficient in terms of Serviceability Limit State control. This was due in particular to poor soil conditions, as will be later explained, that reduced significantly the stiffness at the base of the tower, increasing the lateral displacements. Besides, the eccentric disposal of the inner core implied torsional efforts when submitted to transverse lateral loads (East-West direction).

A tube in tube configuration was therefore adopted, considering the contribution of the structural façade to support horizontal loads, and the design attention moved to the structural and architectural configuration and composition of the façade. Different alternatives were considered, in particular diagrid configurations and Vierendeel frame (Fig. 2). Other configurations as outrigger belt trusses were avoided due to the architectural intention of avoiding intermediate service floors and assuring a continuous configuration of the façade.

Fig. 2  Structural facade configurations. Diaerid and Vierendeel frame.
Many different diagrid configurations were tested, with varying sizes and angles of the diamond grid, considering both the architectural intention and the exterior and interior appearance of the façade, and structural requirements in terms of lateral behaviour and gravity loads transmission. Structurally, the diagrid configuration had several advantages related to the global rigidity of the structure and the ease of construction (no rigid connections), and architecturally it was also satisfying. However, the idea of highlighting the verticality of the tower, that will be one of the tallest in Africa, also supported by the client, moved the decision to the Vierendeel frame configuration, that was also both structurally and architecturally satisfying. A coordinated architectural/structural design process followed then to define the dimension and distance of the vertical columns, as well as configuration and height of the perimeter beams, together with the configuration of the façade, that include thin vertical slats that also contribute to highlight the verticality of the building. The final configuration considers a distance between columns of 1.7 to 3.1 meters, depending on the floors, and perimeter beams from 94 to 61cm high (bottom levels to top levels).

Regarding the floor structure, two alternatives were considered: post-tensioned concrete slab and composite structure. The post-tensioned solution had the advantages of reducing the total height of the floor structure and leaving a continuous free space below for ducts and installations but implied a higher self-weight. Reducing the total weight of the structure was a major constraint, because of seismic conditions and poor soil conditions that implied a complex and costly foundation solution, so decision was taken for the composite solution. The floor decks are therefore formed by steel beams, radially disposed every two columns, and concrete steel metal deck (Fig. 3).

![Fig. 3 Typical floor plan. Eccentric concrete inner core, external steel Vierendeel frame, steel beams radially disposed and concrete steel metal deck.](image)

### 3 Detailed design

According to the structural configuration and to the project and site conditions, the main design requirements that had to face the detailed design phase, and that determined several particularities and singularities in terms of analysis were: deep barrette pile foundation, serviceability criteria control that implied several strategies both in terms of design and analysis, and structure differential shortening control
between the concrete core and the steel façade, in particular in the areas were the two are closely located one to each other.

### 3.1 Barrette pile foundation

The site of the Mohamed VI tower is characterized by alluvial soil conditions with soft clay layers up to a depth of around -45m NGM, alternated with medium to very dense sand and gravel layers. A stiff marlstone substratum is found at around -85m NGM. The schematic representation of the soil stratigraphy and representative pressuremeter test results are shown in Fig. 4.

![Fig. 4 Schematic representation of the soil stratigraphy (left) and limit pressure $p_l^*$ as function of level (right). The blue arrows in the left figure indicate the bearing strata for the foundations of the tower (center) and the podium (sides).](image)

It is clear that due to the different loads of the tower and the podium, a different foundation system is required for both. The high loads and strict deformation criteria of the tower require a stiff foundation system which is provided by a so-called barrette foundation, i.e. single 1.2m x 2.7m diaphragm wall panels which are bored under the support of a bentonite suspension and cast-in-place up to a depth of -65m NGM. The foundation of podium consists of 600mm diameter Continuous Flight Auger (CFA) piles up to a depth of -10m NGM. Piles are required due to the possibility of uplift forces during a flood water level in the river and corresponding high groundwater levels.

The CFA piles of the podium are founded in the dense sand layer 1. This is the most optimal solution, since this layer has a higher capacity than sand layer 2. Also, the liquefaction potential of the lenses at the top of both sand layers implies that negative skin friction can occur after a seismic event, which would make longer piles even more inefficient in terms of bearing capacity.

Nevertheless, the thickness of this sand layer is limited, and limited variation of the pile length and capacity is therefore possible. Variations of the loads in the podium (e.g. caused by large differences in spans), are therefore leading to a variation of the number of piles per support. A total of 1737 CFA piles is finally required for the podium foundation.

The foundation of the tower consists of 104 barrettes, which are founded in the very dense sand layer 3. Different toe levels are defined to optimise the pile length and take into consideration the unequal distribution of the loads over the piles. The maximum depth does not exceed -65m NGM, however, based on practical limitations and to avoid execution risks related to the gravel layer at large depth (loss of support fluid).
The above description explains how the soil conditions lead to a complex foundation system that requires special attention. This is particularly the case for the soil-structure interaction: both foundation systems have significantly different toe levels and loading surfaces. In order to study the vertical interaction, a detailed study has been performed using 3D Finite Element structural and geotechnical models, created with SCIA ENGINEER and Plaxis 3D, respectively. The estimated settlement is shown in Fig. 5. The maximum settlement is around 20mm for the podium and around 40mm for the tower. The podium settlement is relatively small, due to the fact that the structure has a two-story basement, and the soil is consequently partly unloaded before construction. Therefore, the influence of the podium on the tower foundation is very limited, despite its large loading area. The tower settlement is slightly higher, but still limited for a high-rise building due to the presence of stiff layers below the tip of the barrette foundation.

Fig. 5 Estimated settlement of the tower in the centre and the surrounding podium.

3.2 Strategies developed to ensure serviceability criteria

With a total height of 250m and a limited width, the slenderness of the Mohamed VI tower is very high. As a result, the governing criteria of the tower is not the ultimate limit state but the serviceability limit state. More precisely, there are 3 criteria at the limit with regards to the lateral stiffness of the tower:

1. The story-drift that needs to be limited to h/300 to accommodate the façade design.
2. The total lateral displacement at the top of the tower that needs to be limited to H/450.
3. The acceleration at the top levels.

As it can sometimes be the case when designing a complex project, the number of constraints increases during the different design stages, and it becomes more and more difficult to the structural designers to find room for extra stiffeners in the structure (bracings, shear walls, columns, etc.). Hence, parallel methods were developed to comply with the lateral design criterion.

3.2.1 Young modulus

The central core lateral resisting system was designed with a high resistance C60/75 concrete grade. As per the EN 1992-1-1 code, the uncracked E modulus for a C60 is 39GPa. Design guidelines, and even codes, suggest to design shear walls with a reduced Young modulus value in order to take into account the concrete cracking effect under seismic or wind loading (50% as per EN 1998-1 for example). Early in the design process, it was checked that the tensile concrete stresses remain below the $f_{ck}$ values of 3.1MPa as per EN 1992-1-1. It was chosen, in order to reduce the design risk, to perform this check using $f_{ck}$ and not $f_{cmt}$. Furthermore, on-site concrete tests show an even greater tensile resistance of $f_{ck} = 4.7$MPa. Hence, concrete was considered as uncracked and the full value of 39GPa for the Young modulus is used in the SCIA ENGINEER FE structural model.
During execution on site, this value was optimized even further by performing a test campaign on the E modulus. Two sets of 10 samples were used, one for 28 days and one for 90 days [Table 1]. The 90 days characteristic value of 44.5GPa was considered in the final model.

Table 1  
Test campaign result on the core wall concrete E modulus

<table>
<thead>
<tr>
<th></th>
<th>28 days samples</th>
<th>90 days samples</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of samples</td>
<td>10</td>
<td>10</td>
</tr>
<tr>
<td>Average value for E</td>
<td>45.24 GPa</td>
<td>46.66 GPa</td>
</tr>
<tr>
<td>Standard deviation</td>
<td>1.45 GPa</td>
<td>1.14 GPa</td>
</tr>
<tr>
<td>Characteristic value</td>
<td>42.58 GPa</td>
<td>44.69 GPa</td>
</tr>
<tr>
<td>Design value retained</td>
<td>44.5 GPa</td>
<td></td>
</tr>
</tbody>
</table>

3.2.2 Steel coupling beams

The concrete central core is divided into 3 (4 for the lower levels) staircases/elevators blocks (Fig. 3). In order to increase the central core inertia by connecting them together, concrete coupling beams were foreseen at the very beginning of the design. Since the technical rooms are located on each floor, there is a high demand for MEP openings in those lintels. Furthermore, a minimum clear height sets another constraint when it comes to define the required height/cross section of those coupling beams. For them to be efficient, the lintels need to transfer shear forces, normal forces and bending moments.

After analysing all the constraints (openings and clear height) and results it was found that the tower stiffness was reduced by ±10% in the North-South direction and ±20% in the East-West direction, mainly due to the cracking of concrete. It was so decided during the detailed structural design to switch steel coupling beams from concrete to steel. This value engineering allowed to retrieve ±5% of the stiffness in the North-South direction and ±10% in the East-West direction (Fig 6).

Fig. 6  
Steel coupling beam: Bending moment and vertical shear are taken care of with contact to the concrete (first part with flanges and stiffeners) then normal forces is taken care with end plate, the depth of these end plates depending on the forces. Flange’s depth was limited to the required length to allow better pouring.

A special attention was given to the mix connection design between the core walls and the embedded steel coupling beam that required to transfer huge forces. To validate the design, both a local model with 3D finite elements was created with ABAQUS and a hand calculation was performed based on the theoretical model given by Kent & Park. With both methods, the validation was found by finding an equilibrium that balanced all the forces without exceeding the concrete local strength. The research program SMARTCOCO, together with EN 1998-1, was used to define the minimum technological-required reinforcement to confine the concrete around the connection [3, 4].

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3.2.3 Dynamic behaviour and implementation of TMD system

The control of lateral accelerations of the tower due to wind loads, that primarily governs the human comfort, depends on the dynamic behaviour. According to the ISO-10137:2007 [5] and the BLTWL and CTBUH criteria, originally proposed by Isyumov [6], lateral accelerations were to be limited at the upper residential level (R+49, +203m) to 5 to 7 milli-g for one-year return period and to 10 to 15 milli-g for ten years return period (Fig. 7). The frequencies of the structure for the main modes were: mode 1 (North-South): 0.17Hz; mode 2 (East-West): 0.20Hz; mode 3 (Torsional): 0.49Hz. Considering these frequency values, the ISO-10137 criteria were less restrictive than the BLTWL and CTBUH ones.

The wind tunnel analysis showed acceleration values of the tower in the East-West direction within the considered limits (12.1 to 11.5 milli-g for ten years return period), while in the North-South direction the acceleration values were clearly beyond the comfort limits (29.5 to 21.1 milli-g for ten years return period). The given values consider an inherent structural damping of 1% and scenarios considering both the existing and the future proposed surroundings. Indeed, the structure is stiffer in the East-West direction, due to the configuration of the inner concrete core, as shown in the frequency of the structure in modes 1 and 2, which explains the very different dynamic behaviour in both directions.

Considering the resulting acceleration values, the disposal of a Tuned Mass Damper (TMD) at the coronation levels was considered, a classical solution to provide supplementary damping and reduce lateral accelerations within the comfort threshold values [7]. Fig. 8 presents the main characteristics of the TMD, as well as the resulting acceleration North-South direction with TMD system.

<table>
<thead>
<tr>
<th>Requirements 1 and 2 should be satisfied</th>
<th>Acceptable Hourly Peak Values</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Once per year event</td>
</tr>
<tr>
<td>Residential</td>
<td>5 - 7</td>
</tr>
<tr>
<td>Hotels</td>
<td>7 - 9</td>
</tr>
<tr>
<td>Office</td>
<td>9 - 12</td>
</tr>
<tr>
<td>All occupancies</td>
<td>1.5</td>
</tr>
</tbody>
</table>

1) Peak resultant accelerations (milli-g) at the top occupied floor should be at or below the following:

2) The peak torsional velocity (milli-rads/sec) at the top occupied floor should be at or below the following:

Fig. 7 Peak acceleration limits for one-year return period (ISO-10137:2007). Peak acceleration limits for one-year and 10 years return periods (BLTWL and CTBUH).

Fig. 8 Tuned Mass Damper (TMD) system. 3D View & Main characteristics. Resulting accelerations North-South direction with TMD system, 10 year return period. Gerb, 2019.
3.3 Structure differential shortening

On every high-rise project, structural designers deal with differential shortening between the central core and the façade columns. Here, the concrete central core is totally vertical and the outer dimension/diameter along North-South axis of the façade Vierendeel steel structure varies between a minimum at level R+1 (31m), a maximum at level R+20 (37m) (Fig. 9) and a minimum at the last occupied floor level R+51 (19m).

Fig. 9 3D view on 2 stories of the SCIA ENGINEER model used to design the tower.

Thus, the particularity on this tower is that the eccentric central core is very close to the South façade, and it is therefore more challenging to comply with some restraints. Here are the three main challenges and the way they were handled in the design.

3.3.1 Levels: Theoretical vs Construction

Table 2 details the shortening values that are expected. It was calculated and tested that the central core, mainly due to the high expected creep values for a concrete grade C60/75, shortens more (maximum of 165mm) than the façade columns (maximum of 95mm). In order to target at the end of the execution on site the architectural theoretical levels, the shortening has to be compensated.

Without applying any specific rule on site to correct the levels, a standard execution will automatically compensate an important part (maximum 56mm) since a certain portion of the shortening has already happened at the moment when each story is executed.

It was decided, in order to compensate the remaining mid-term shortening, for both the central core and the columns at the facade, to pour concrete and erect the steel frame at higher levels than the theoretical ones (maximum 80mm).

Table 2 Long term maximum vertical shortening [mm]

<table>
<thead>
<tr>
<th></th>
<th>Concrete central core</th>
<th>Steel columns at the façade</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>R+14</td>
<td>R+29</td>
</tr>
<tr>
<td>Foundation settlement</td>
<td>45</td>
<td>45</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>19</td>
<td>46</td>
</tr>
<tr>
<td>Creep &amp; elastic shortening</td>
<td>15</td>
<td>25</td>
</tr>
<tr>
<td>TOTAL long term</td>
<td>79</td>
<td>116</td>
</tr>
<tr>
<td>Self-compensated</td>
<td>10</td>
<td>23</td>
</tr>
<tr>
<td>Overcompensated</td>
<td>65</td>
<td>80</td>
</tr>
<tr>
<td>Shortening to be expected after placement of finishes</td>
<td>33</td>
<td>40</td>
</tr>
</tbody>
</table>
3.3.2 Hinged diaphragm
The two lateral resisting systems that are the central core and the Vierendeel external tube behave independently without any outrigger effects. The floor system that acts like a diaphragm only transfers normal and shear forces from one system to the other. Hence, the connection between the lateral resisting systems and the floor elements needs to be hinged and remain as such in the long term after the shortening effects. Due to the differential shortening between the core and the façade, a rotation at those connections is expected. Fig. 10 shows the details for steel connections between floors beams and façade columns. Horizontal slotted holes in the steel connection plate will allow the required rotation due to the differential shortening. It is important to note that the rotational point is located at the mid-height of the diaphragm, that is the steel metal deck (mix Comflor).

![Fig. 10 Connection between hinged floor beams and the core wall (left) / the façade columns (right)](image)

3.3.3 Disorders in the finishes
As presented in Table 2, even with extra compensational measures, long-term differential shortening is inevitable between the central core and the steel frame at the facade after the placement of finishes (Fig. 11). This becomes more and more critical and sensitive when the height increases, and the facade comes closer to the core. It is expected that the differential shortening between the central core and the facade after placement of finishes can reach ±20mm. This displacement is to be added to all other deflections due to live loads, wind, and seismicity. It has to be noted that the most expensive and important living areas are located at the top of the tower (5-star hotel, observatory and the royal family’s apartments), where this effect is the most important.

Although this phenomenon is critical for finishes, it is very localized. Furthermore, it is precisely located near the radial floor beams hinges. Without any measures, disorders are also expected in the partitions because they would tend to take a parallelepipedal shape.

Therefore, it was decided to implement:
1) Dilatation joints in the floor finishes near the hinge location, that is along the facade and along the concrete core wall. This constraint leads to a necessary coordination process with the interior designer for the central core corridor locations.
2) Dilatation joints at some partitions walls connections with the central core and with the facade. Also, the connection between the partitions walls and the ceiling needs to allow a horizontal movement to avoid the parallelepipedal shape.

![Fig. 11 Differential shortening between central core (right) and façade (left)](image)
4 Conclusion

A high-rise always implies robust structural configurations and particular construction systems. The possible structural configurations are nowadays well established, and the design process to decide which system adapts better to the conditions of the project is somehow a straightforward process. However, every project has its own particularities, and the different structural configurations may adopt diverse configurations.

In the case of the Mohamed VI Tower in Rabat, these particularities were related to its location in a moderate seismic zone with very poor alluvial conditions, its singular architectural configuration with an eccentrical inner core, and the intention to enhance the verticality of the tower. These three conditions guided the conceptual design of the tower at the initial design phases and determined the main requirements and analysis conditions of the detailed design phase, that implied specific solutions to face them.

Finally, it is also important to highlight the implication in the design and structural configuration and construction of the project of many companies and teams from different countries, including Morocco, Spain, Belgium, Holland, England, Germany, France, United Arab Emirates, and India, in a very fruitful and satisfactory collaboration to develop what will be one of the tallest skyscrapers in Africa, and a milestone for Morocco and the city of Rabat.

Acknowledgements

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References


Conceptual design in high-rise buildings based on the competition submission “Hufelandstrasse” in Munich

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Abstract

Different usages in high-rise buildings typically require multiple structural grids in a building. Thus, structures for load transfers have to be considered. This paper presents a design of David Chipperfield Architects as part of the competition for a commercial building in Munich. For this project, a ribbed slab as exposed structure was developed on the ground floor. The aim was to propose a system with high innovation potential that also satisfies the demands of material minimization, sustainability and aesthetics. A detailed description of the form finding process of the ribs by means of the residual stress lines as well as the integration of the ribs in the overall appearance in the structure is provided.

Keywords: form finding in high-rise structures, residual stress lines in slabs, ribbed slab system, force-flow based structural design in high-rise structures

1 Introduction

Form finding and parametric design is already widely used in lightweight and artificial structures such as stadium roofs, pedestrian bridges or structures with very large spans. However, in high-rise construction different types of usages often need to be integrated into one building. With regard to the supporting structure, multiple load-bearing grids are required and constructions for load transfers have to be considered. As structural engineers active in high-rise construction, the aim and the expectation has to be to advance material-minimized structural design in this discipline by integrating structures following the force flow. Only in this way an economical and sustainable result will be obtained.

Within the scope of this paper, the competition submission of David Chipperfield Architects for the “Hochpunkt” in Munich will be presented, in which an effective and exposing structure has been elaborated. Section 2 explains the overall constraints and requirements for the design and, based on this, the conceptual design including variant studies. Section 3 provides a reference to similar historical constructions and discusses the potential for integrating digital fabrication methods. Section 4 illustrates in detail the optimization of the structure and describes the procedure for the design of the ribbed floor slab, while the last section summarizes the results.

2 Development of the structural design

2.1 Constraints and requirements

In August 2020, the HUF4 GmbH CO. KG in coordination with the City of Munich initiated a realization competition for the urban and open space development of a commercial area in the 11th district of Munich. The competition was organized as part of the BMW Group’s FIZ Future Project, which is intended to restructure, expand and strengthen the location of the BMW Research and Innovation Centre in a sustainable long-term perspective. The master plan specified that a future-oriented reconstruction shall be realized for both commercial and service purposes. In addition, the project should provide space for the public use, enable economic efficiency and flexibility of the working environment and create identification. The maximum height of the building amounts to 60 meters and needs to be integrated harmoniously into the city [1].

A significant aspect of the competition was the economic profitability and the sustainable quality of the design proposal. The specific requirements were an optimal ratio between usable space and the building’s exterior, a high degree of flexibility in terms of room layout and equipment standards as...
modular building system with regard to the specific requirements of the users, maximization of the leasable space with a high level of economy in terms of investment and utilization costs and a maximum functionality of the materials used to ensure the durability of the construction and its ease of handling and maintenance [1].

2.2 Concept development

The constraints mentioned in the previous section resulted in a major challenge for the design starting with the form of the building. The final proposal of David Chipperfields Architects presented in Fig. 1 has a tripartite shape in the floor plan for individual forecourts of the hotel, offices as well as stores. The optimally developed floor plan ensures an excellent orientation within the different usages and among the different parts of the building. In addition, ideal lightning is guaranteed by the wide surface of the façade [2].

However, due to the constraints, the structure is vertically subdivided into three layers as shown in Fig. 2. The office and hotel rooms are located in the upper floors. An ideal structural grid for this part of the building is 5.4m x 5.4m – partly also influenced by the chosen timer hybrid construction. In this way, a flexible interior design according to the demands of the occupants is ensured and the requirements for the office space and hotel rooms are satisfied equally. The parking belonging to the building is arranged directly in the basement. The proven and optimally suitable structural grid for an underground parking is 8.1m x 8.1m securing sufficient space for accesses, entrances and parking lots. In the “middle layer” is the ground floor primarily dedicated to the public. On this floor the vertical loads from the upper floors must be adapted to the structural grid in the basement. Consequently, a load transfer of significant forces is necessary.
For this particular load transfer, three different variants illustrated in Fig. 3 were discussed and evaluated in terms of its visual appearance, sustainability, economic efficiency, innovation potential and constructive complexity. The results shown in Table 1 have a direct correlation to one another and should not be interpreted as a general statement.

The first alternative was a slab with a beam structure in a rectangular grid, which would have distributed the load from the columns in the upper floors to the supports for the ground floor on the 8,1m x 8,1m grid. Based on the resulting stiffness distribution in the slab, the load actually would have followed the ribs and this solution was also the most feasible in terms of construction [3]. However, large bending moments caused by the deviation of the ideal force path would have been the consequence. The second variant was a slab with a beam structure as well. In this case however, the principle applied by Pier Luigi Nervi on several projects was adopted: The ribs are oriented to the residual stress lines ensuring the direct force flow. The third possibility was a slab combined with column heads (“mushroom columns”) representing the ideal form of a two-dimensional slab that would have reduced stress concentrations at critical positions [4].

Table 1  Comparison of the alternatives

<table>
<thead>
<tr>
<th>criterion</th>
<th>Beam structure</th>
<th>Ribbed slab</th>
<th>Mushroom columns</th>
</tr>
</thead>
<tbody>
<tr>
<td>Visual appearance</td>
<td>-</td>
<td>++</td>
<td>+</td>
</tr>
<tr>
<td>Sustainability</td>
<td>-</td>
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<td>Economic efficiency</td>
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<td>-</td>
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<tr>
<td>Innovation potential</td>
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<td>+</td>
</tr>
<tr>
<td>Constructive complexity</td>
<td>++</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

In a collaborative dialogue with the architects, it was decided to pursue the second variant with the aligned ribs, as on the one hand a visually very elegant design with innovation potential may be realized and on the other hand a lot of material is saved for a sustainable design.
Historical reference and innovation potential

The structure of ripped ceilings is not a new concept and has been realized several times so far, in primarily projects of Pier Luigi Nervi must be highlighted. Subsequently, two references shall be presented followed by a summary of the recent investigations at the Block Research Group and the Digital Building Technologies Group led by Benjamin Dillenburger. In particular the combination of digital fabrication and computational design applied to the production of material-minimized slabs is especially interesting and attractive for the design proposal.

The first reference is the “bone ceiling” shown in Fig. 4 (left) by the architect Hans-Dieter Heckeraus in Freiburg. The name “Bone Ceiling” was given due to its design inspired by the structure of bones as highly developed lightweight structure highlighted by Stoller [5]: “Bones must be strong, because they carry the entire body weight. Nevertheless, bones must be light, because every additional gram of weight requires unnecessary energy for movement. In addition, bones must be designed to save as much material as possible since all bone cells must be formed and need to be constantly supplied with nutrients.” All of the mentioned aspects should be equally considered for an optimal result. High stability with minimal dead weight and material savings for a economical, sustainable construction are the basis prerequisites for a successful design. For this reason, Heckeraus used material only along the residual stress lines for a slab which is as light as possible and has an adequate load-bearing capacity. Particularly remarkable is the positive performance of the ceiling in the sustainability evaluation compared with lightweight ceilings used today such as hollow core or prestressed concrete slabs representing the state of the art. Especially the material savings contributed to a great result, since large amounts

Fig. 3 variant study for the load transfer in the ground floor

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of CO₂ are already emitted in the manufacturing process of concrete. Though, high personnel costs due to the elaborate construction process were the main disadvantage [5].

The second reference is the ribbed slab by Pier Luigi Nervi in the Gatti Wool Mill in Rome. Nervi’s basic concept of designing was not only to achieve a synthesis of form and structure within the framework of the static laws. It was his aim to consider the demands of aesthetics and economy equally. Specifically, this means for the slab shown in Fig. 4 (right) that the ribs follow the principal moments for enabling an arrangement of the reinforcement in their statically optimal direction. In addition, Nervi recognized the high construction effort. Therefore, he used a special, continuously displaceable formwork made of reinforced concrete [6].

Fig. 4 “Bone ceiling” in Freiburg by Heckeraus [5] – Gatti Wool Mill in Rome by Nervi [4, 6]

Due to high construction costs, this ceiling system has not been imitated regularly and has been adopted only marginally in the recent past. Nonetheless, this design is being investigated increasingly in research, especially in the Block Research Group and the Digital Building Technologies Group at the ETH Zurich. Special focus is on exploring how digital fabrication methods can contribute to improve the sustainability of concrete slabs. For this purpose, a ribbed ceiling has been designed with no reinforcement needed. This prevents corrosion damage and increases the durability [7]. The prototypes were produced by two methods: 3D printing and – of special interest for the design proposed in the competition – prefabrication of the moulds (e.g. from EPS) using CNC-techniques [8]. Through the repeated application of the same moulds, the ribbed slab could be built economically on a larger scale and costly formwork processes may be minimized. In addition, the necessary reinforcement can be placed more conveniently compared to the usage of 3D printing. Fused deposition modelling (FDM) would be another promising option, as it allows to produce large formwork elements with a minimum amount of material consisting in particular of bio-based recyclable plastic such as PLA. Therefore, transport, handling and recyclability of the individual formwork elements are the main advantages of the ultra-light system [9].

4 Form finding process of the beam structure for optimizing the force flow

As mentioned in section 2, it was decided to transfer the loads from the first layer by a slab with ribs aligned to the force flow. Due to the total of 16 upper floors, the loads had to be minimized as far as technically possible. For this reason, a timber hybrid construction was chosen since, on the one hand, about 50% of concrete can be avoided resulting in considerable weight and CO₂ savings and, on the other hand, all requirements for fire protection and sound insulation can be satisfied. In addition, a high degree of prefabrication was intended enabling a modular construction process.

After determining the point loads on the slab on the ground floor, the form finding of the ribs was performed. For this purpose, Karamba3D was chosen which is a parametric engineering tool embedded in Grashopper, a plug-in of the CAD software platform Rhinoceros [10].

First, the floor plan of the first floor was modelled as a flat ceiling supported on columns in the structural grid of 8,1m x 8,1m of the underground parking. Second, the slab was stressed and calculated with the determined point loads from the upper floors and the area loads on the slab itself. Based on this result, the residual stress lines were analysed. Thereby, the values were not of particular interest,
but rather their direction for the comprehension of the load-bearing behaviour of the slab. By repeatedly reducing the mesh density by half, the force flow in the slab became increasingly visible as illustrated in Fig. 5.

![Fig. 5 termination of the force flow in the slab](image)

Subsequently, the precise shape of the ribs and their density have been discussed in detail with the architects. In this way, an efficient and effective transfer of the point loads to the supports was ensured as well as a construction satisfying all aesthetic demands, since it would have been an exposing structure. The final design was obtained by means of circular, elliptical, regular curved but also straight ribs (Fig. 6). A further consideration was to modify the cross-section of the ribs according to their internal forces. Due to the significant additional effort in the formwork as well as in placing the reinforcement compared with only marginal reductions in concrete, this possibility has not been investigated in detail.

![Fig. 6 final design of the ribbed slab](image)
Fig. 7 presents the result in form of a visualization. A design has been achieved which has both an external and internal identification character as required by the competition. In addition, a building was designed that has great innovative potential in the construction method and that could have provided an example of modern structural engineering in high-rise construction.

5 Conclusion

Due to the rapid growth of the urban population, the construction sector has expanded significantly. As a result, a major challenge and responsibility has arisen to build in a sustainable way. This can only be achieved by reducing emissions, minimizing the amount of material used and through the ethical and conscientious behaviour of structural engineers. The most important tool available is the form of the design representing the interests of all involved parties. Nowadays, technical progress made it possible to incorporate form finding by computational design and digital fabrication methods [11].

Within the scope of the competition for the Hufelandstrasse, the main objective was to propose a design in which these aspects should be implemented as well as possible and simultaneously consider economic interest through an efficient concept. Thereby, efficiency was not only interpreted as lightweight structure but also as a construction with low CO₂ emissions. The result is a design that combines identity-creating elegance and sustainability with a limited construction budget. This was accomplished on the one hand by minimizing the use of materials, clear load paths with comprehensible, efficient structures and on the other hand by a modular construction with a high degree of prefabrication leading to a significant reduction in construction time.

Despite the material savings and the planned prefabrication of the formwork and the structural elements in the upper floors, the formwork effort in the ceiling above ground floor would have been very elaborate. The formwork approaches mentioned in section 4 are promising, but the adaptability to a
much larger scale would have been a major engineering and construction challenge, impacting especially on the economic efficiency of the system. Alternatively, another material would have been conducive, whereby steel would not necessarily have been more sustainable and cheaper. Also, the forces were simply too large for timber.

Nevertheless, the final design can be regarded as a project of excellent interdisciplinary collaboration. It was demonstrated that aesthetic aspects are compatible with the static requirements. In the ribbed slab, the structural characteristics are part of the architectural interior design. By minimizing the concrete, a physical presence of the structure was achieved, which creates distinctive spaces making them perceivable at the same time. Indeed, this is the fundamental idea of structural design, as evidenced by the quote by Jörg Schlaich in 1996 (cited by [4], S. 614): "Structural design means to combine knowledge with intuition, experience with fantasy, and aims at inventing an efficient structure including a unique form. Architects and engineers can learn a lot from each other: The architects can learn from the engineers about the interrelation of manufacture, form and load-bearing behaviour; about the importance of structure detailing, the aesthetics of a pure, efficient structural form [...] The engineers can learn from the architects, first of all how to see, the view for the visual; about the social and ecological aspects of a building [...]”

Material-minimized and force flow-based design has been and is being done – many renowned structural engineers such as Pier Luigi Nervi, Frei Otto, Christian Menn or Jörg Schlaich built like this. Nevertheless, this approach to high-rise structures engineering has rather fallen into oblivion in recent years. However, it must be the overall aim to use the existing knowledge again and to re-integrate it into classical building constructions. Tools such as digital fabrication possibilities can and should be used in the future to build in an environmentally conscious and resource-optimized way.

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References

Supporting the giant 23,000 tonne ITER Tokamak nuclear fusion reactor

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Abstract

ITER is the most ambitious energy project in the world today, whose main objective is demonstrating the scientific and technical feasibility of nuclear fusion as a safe, clean and unlimited energy source. The ITER experiment builds on the concept of the tokamak, a torus continuous tube surrounded by coils that produce a magnetic cage to confine the high-energy plasma. The ITER Tokamak machine currently under assembly is 24 metres high, 30 metres wide and weights around 23000 tonnes.

Supporting this vast and totally unique machine has also been a challenge in itself, due to the highly demanding environment, the limited space available, the complexity and huge magnitude of the loads involved -both under operational and accidental conditions-, the safety requirements arising from ITER being a nuclear facility, and, last but not least, the management and contractual obstacles associated to this multi-national and multi-disciplinary endeavour.

This paper provides a global overview of the main principles driving the design of this very singular supporting structure within the ITER reactor building, including the very complex loading induced by the Tokamak machine. Some historical background about major design evolutions that significantly affected the original structural concept is also given, focusing on the increasing relevance that reinforced concrete took during the design process in order to address many of the challenges this once purely mechanically-oriented engineering task had to face over the years.

1 Introduction

Supporting things is a key role for most structures. Bridges support roads and railways, buildings support people’s homes and offices, and dams support water (laterally). Cooperation between different agents is a key aspect for the successful completion of these projects, especially when diverse disciplines are involved. For the structural engineer this starts from knowing and understanding the loads that the supporting structure must withstand.

The supporting role is also behind most concrete structures in the industrial and nuclear sectors, although it sometimes seems these developments have gained less attention from the structural design community. In some cases, the design and construction of these supporting structures in more industrial contexts also face significant challenges.

Then, the usual and convenient, but also oversimplifying, total splitting between mechanical design and civil engineering design activities is simply not possible. A fully integrated approach is then required where ‘having the loads’ is clearly not enough, especially when the loads are heavily dependent on the supporting structure itself. The supporting structure for the ITER Tokamak machine is a unique example that required the development of a bespoke design solution.

2 The ITER Tokamak machine – A brief mechanical overview

2.1 Main Tokamak machine components

The ITER experiment builds on the concept of the tokamak, a torus continuous tube surrounded by coils that produce a magnetic cage to confine the high-energy plasma. The ITER Tokamak machine
currently under construction and assembly (Fig. 1) is 24 metres high, 30 metres wide and weights around 23000 tons. The most relevant components of the Tokamak machine are:

- The 10000 tonnes superconducting magnets in charge of producing the magnetic fields to initiate, confine, shape and control the plasma (Fig. 2 left)
- The Vacuum Vessel, an 8000 tonnes stainless steel chamber that houses the fusion reaction and acts as a first safety containment barrier (Fig. 2 right). It has penetrations called ports to provide access to the plasma at three levels (lower, equatorial and upper): 18 ports at the upper and equatorial levels and 9 at the lower level.
- The 3800 tonnes cryostat, a stainless-steel chamber (29 x 29 m) surrounding the vacuum vessel and magnets to ensure an ultra-cool, vacuum environment (Fig. 2 middle).

Fig. 1  The ITER Tokamak machine [1]

Fig. 2  Main ITER Tokamak machine components [2]
2.2 Some specificities driving the design of the supporting structure

The design of both the Tokamak machine and its supporting scheme is determined by some operational and accidental regimes that the system must be able to cope with. These constraints mainly come from the following sources: (i) thermal conditions corresponding to different machine states (section 2.2.1), (ii) gravity loads induced by the extremely large mass of the system (section 2.2.2), (iii) seismic threats to be faced (section 2.2.3) and, last but not least, (iv) the high congestion in the very limited space (in comparison with the magnitude of the loads involved) available for the implementation of the supporting scheme (section 2.2.4).

2.2.1 Tokamak machine thermal conditions

The main Tokamak machine components operate in a variety of thermal conditions that are behind many of the design constraints that need to be accounted for from the very beginning of the design process.

Under normal ITER plasma operation conditions, the superconducting magnets are at 4K, whereas the vacuum vessel and the cryostat are at 100°C and room temperature (around 30°C), respectively. During some maintenance and conditioning operations, the vacuum vessel temperature increases up to 200°C. Some postulated accidents inside the cryostat (cryostat ingress of coolant events -Cr ICEs-) foresee the rupture of pipes carrying cryogenic helium coolant for the feeding of the superconducting magnets, with the resulting leaks making the cryostat temperature drop to around -93°C (180K). This extremely quick summary reflects that maximum temperature ranges up to 500°C (not simultaneously) have to be accommodated by the Tokamak machine design. For such stiff and heavy components this has important implications. These thermal gradients also affect any conceptual approach for the design of the supporting structure.

2.2.2 Gravity loads

The 23000 tonnes of the Tokamak machine clearly limit the options available for the design of a feasible supporting scheme, especially when taking into account constraints imposed by the other factors.

2.2.3 Seismic aspects

As a nuclear facility, the seismic design of ITER must ensure the corresponding seismic safety requirements are met according to the nuclear regulation of the host state (France).

The design earthquake for ITER (Safe Shutdown Earthquake -SSE-) is generated as the envelope of two seismic events: the “Seisme Majore de Securite” (SMS) and the paleoseis. The Zero Period Acceleration (ZPA) is equal to 0.315g. The vertical motion is derived by multiplying the horizontal motion by 2/3.

2.2.4 Highly congested space

Despite the magnitude of the loads involved, the availability of space to devise a robust supporting scheme is very much reduced due, not only to the global size of the machine, but to the vast amount of penetrations and services that must access the machine at various levels.

2.3 In-machine supporting scheme

The first mechanical design choice at a Tokamak machine level is to completely decouple the superconducting magnets system and the vacuum vessel from a structural point of view. These two large and heavy parts only share a common supporting element: the pedestal ring (Fig. 4), which is an integral part of the cryostat. The pedestal ring is a stainless steel, very stiff circular beam of rectangular hollow cross-section, with wall thicknesses up to almost 200mm.

The whole magnet system (Fig. 2 left) is supported mechanically speaking by 18 Toroidal Field Coils (TFC), which rest on the pedestal ring by means of the so-called TFC Gravity Supports (TFCGSs). Each of these supports induces a vertical load around 5.7MN only due to the mass of the magnet system and is designed to be very flexible for radial motions, in order to accommodate the thermal contractions resulting from the large temperature variations, but stiff in the circumferential and vertical directions.

The vacuum vessel (Fig. 2 right) has penetrations called ports to provide access to the plasma at three levels (lower, equatorial and upper). It has 18 ports at the upper and equatorial levels, but only 9 at the lower level. The vacuum vessel rests on 9 gravity supports, each transferring a vertical load around 9.7 MN to the pedestal ring due to self-weight. These supports are also designed not to constrain the radial expansion of the vacuum vessel when its temperature increases to 100°C or 200°C.
3 The ITER Tokamak machine supporting structure

3.1 Historical background and design rationale

There is a key decision, inherited from the internal arrangement selected for the main machine components, that determines many other factors downstream the design process: where to place the supports, both in the vertical (supporting function) and horizontal (seismically restraining function) directions.

As already anticipated, only one supporting level (Fig. 3) is adopted for vertical loads: the pedestal ring upper flange. This is a very sensible choice in view of the thermal gradients associated to normal operating conditions of the Tokamak machine. Providing supports at supplementary levels would make the transfer of gravity loads not statically determined and would require a very complex way to make this supporting condition compatible with absorbing the thermal expansions/contractions involved. Note that both radial and vertical thermal deformations of the vacuum vessel when supported on the pedestal ring reach some centimetres for equatorial and upper ports under usual operating or maintenance conditions.

For horizontal loads, the same (and only) supporting level as for vertical loads is kept. Implementing no additional levels for the transfer of horizontal loads is also a natural choice in view of the thermal deformations expected, which are absorbed by means of (i) large bellows to accommodate differential thermal expansions between the vacuum vessel and the cryostat and (ii) mechanical independence of the magnet system with respect to the rest of the Tokamak machine. This mechanical independence is actually compulsory from a thermal point of view if the magnets are to operate at 4K.

Fig. 3 Internal supporting scheme – Tokamak machine arrangement

However, having the pedestal ring as the only interface to transfer horizontal loads has significant implications when seismic responses are considered. This design choice implies that horizontal seismic loads and, especially, induced rocking moments due to the vertical eccentricity between the Tokamak machine centre of gravity and the elevation of the supporting level, are to be transferred only through that interface. This would involve huge seismic loads and an-almost-impossible-to-design-support in practical engineering terms, unless seismic motions are filtered before reaching the Tokamak machine. This is, at least partially, behind the high level ITER project decision to introduce a seismic isolation system for the whole reactor building, the so-called Tokamak Complex, which is seismically isolated from the seismic pit basemat by 493 anti-seismic bearings (ABSs) that support the bottom basemat of the entire building. The ABSs are 900x900 mm2 pads mounted on top short columns (“plinths”) that come out the seismic pit basemat. Fig. 4 shows a comparison between seismic floor response spectra at the support level of the Tokamak machine for the design earthquake of 10000 years return period with and without seismic isolation. It can be clearly seen that seismic loads increase by a factor of 3–4 without
seismic base isolation, a loading increment that would not be affordable for the project, since it was difficult enough already to design the support with the help of the Tokamak Complex base isolation. In addition to this, it must be recalled that the support of the Tokamak machine is only one (though a very relevant one) of the hundreds if not thousands of ITER systems affected by this seismic isolation.

Fig. 4 Seismic FRS at Tokamak machine support – Isolated vs. Non-isolated building

From a thermal point of view, differential expansions/contractions at the vacuum vessel and magnet system supports are addressed by a very specific design of the latter. Since neither the vacuum vessel nor the magnet system have any other structural connection with the building, these motions are then absorbed by the corresponding supports. However, when it comes to the transfer of thermally imposed displacements to the building, the fact that during cryostat ingress of coolant events the temperature of the pedestal ring drops to around -100°C with respect to that of the building drives the design of the whole supporting structure.

Fig. 5 shows the conceptual design of the supporting scheme the authors first saw when started working on the design of this complex interface, more than a decade ago. The pre-conceptual design was based on 18 steel columns (one below each TFCGS) welded to the pedestal ring and anchored to the reinforced concrete basemat slab below. The red arrows indicate the direction of the pedestal ring contraction under Cr ICEs and the resulting deformed shape of the columns is sketched also in red. Over-constraining imposed deformations of very stiff structural systems is always to be avoided and this solution was discarded not long after this deficiency was identified.

Fig. 5 Pre-conceptual design of the Tokamak machine support (discarded, 2010)
In addition to the main contribution from the vertical columns, two complementary load transfer mechanisms were foreseen as reflected in Fig. 6: 18 radial plates located at the lower periphery of the cryostat that prevent the relative motion of the machine with respect to the building in the toroidal (i.e. circumferential) direction while allowing for thermal expansion in the radial direction (Toroidal Skirt Support -TSS-), and a continuous support at the periphery skirt of the lower cryostat preventing the vertical motion of the cryostat while allowing for relative motions between the base of the cryostat and the bioshield wall in the horizontal plane (Vertical Skirt Support -VSS-).

Fig. 6 Complementary load transfer mechanisms for Tokamak machine support (VSS & TSS) [2]

3.2 Implementation

3.2.1 Introduction

A complete re-design of the Tokamak machine supporting structure was then addressed substituting the pre-conceptual solution based on 18 stiff steel columns by a reinforced concrete structure, the so-called ‘reinforced concrete crown’ (RC crown), while keeping the two additional load transfer mechanisms described previously (VSS and TSS). This implied non-negligible changes in many of the most important ITER systems, which were tackled in a very efficient and coordinated manner by the project.

This new supporting concept relies on two additional and key supporting elements that served as the interface between the Tokamak machine loads coming from the pedestal ring and the reinforced concrete structure, the so-called cryostat support bearings (CSB) and steel transition piece (STP).

3.2.2 The Cryostat Support Bearings (CSBs)

18 CSBs are in charge of transferring the complex loading from the pedestal ring to the RC crown while allowing for the required (radial) thermal contraction of the Tokamak machine with respect to its supporting structure (a contraction that can reach values in excess of 20 mm during Cr ICEs). The CSBs are low friction spherical bearings with a primary (flat, mainly for relative machine-building horizontal sliding) and a secondary (spherical, mainly for assembly) sliding interfaces. A sketch with the position of the CSBs between the cryostat pedestal ring and the RC crown is shown in Fig. 8, complemented with a picture of an actual CSB already put in place [2]. Extensive design and advanced FE analysis works were undertaken for the definition of these key elements, including an experimental campaign with both scaled and full-scale models, before the CSBs were manufactured and assembled on site.
3.2.3 The Steel Transition Piece (STP)

The vertical loads transferred by the CSBs are very high due to the large mass of the Tokamak machine. Typically, the dead load of the equipment alone at each bearing point is of the order of 1100 tonnes and this increases to more than 2500 tonnes under certain extreme accidental conditions. Thus, stresses under the bearing are quite significant. In addition, there are relatively high horizontal loads and moments applied, whose magnitude is limited by the frictional capacity of the primary and secondary sliding surfaces of the CSB. The highly concentrated loads coming from the CSBs could not be transferred directly to the RC crown, since this would exceed the local capacity of concrete to withstand such large compressive stress levels. In addition, strict positioning tolerances for the CSBs and a correct connection with the dense pattern of radial (HB50) and circumferential (HB40) reinforcement bars of the RC crown, made it necessary to design a transition element, the so-called STP. The STP supports the CSB and transfers vertical and horizontal loads and moments into the RC crown, which in turn takes this loading to the basemat below and the bio-shield walls.

Due to the large horizontal loads and moments applied to the STP by the CSB, a standard anchorage plate is not sufficient as the loads are too large to be accommodated by anchor bars welded to the Top Plate. Large shear keys and vertical surfaces are required for transferring the horizontal loads and moments, respectively. This, along with the highly demanding construction and assembly tolerances to be met, made it necessary to split the STP in two main components, the Embedded Frame (Fig. 9 right) and the Top plate (Fig. 9 left). Intensive integration work based on 3D CAD models retrofitted by on-site surveys was undertaken for the construction and execution designs (Fig. 10 left), whereas the final structural design and substantiation required advanced non-linear FE analyses (Fig. 10 right) to account for the non-linear (i.e. contact) transfer of loads between steel and concrete parts, as well as for non-linear material properties, mainly under accidental load cases (concrete cracking and crushing and local steel yielding).

Fig. 8 Cryostat Support Bearings

Fig. 9 STP on site: view of Top plate (left) and Embedded Frame (right) [2]
3.2.4 The reinforced concrete crown

The design of the RC crown structure (Fig. 11) is integral with the bio shield wall at its outer edge and with the bottom slab, directly supported by the ASBs. The main structural elements of the RC crown are the 18 radial walls that carry self-weight, vertical dynamic loads and a certain fraction of the horizontal dynamic acting on the Tokamak machine. As already mentioned, a CSB is placed on top of each of these 18 radial walls in order to provide the required interface with the pedestal ring. The radial walls are connected toroidally by a circumferential wall with openings of various sizes at different sectors, which provides a significant stiffness and capacity to the global structural system in the radial walls out-of-plane direction. High strength concrete class C90/105 has been used for the RC crown.

The highly demanding loads and reduced space to accommodate them, the construction and integration constraints (note that the RC crown was executed long after the basemat and bioshield had been erected, with the main reinforcement of the radial walls already put in place), and the interface with the STP which, apart from the steel-to-concrete interfaces, involved mechanical couplers to connect radial and circumferential reinforcing bars to the STP Embedded Frame, made the design and construction of this supporting element a complex challenge.

References

[1] www.fusionforenergy.eu

Acknowledgements

The authors of this paper have witnessed, contributed in various forms and, only occasionally, led the works described in this article, but do not at all claim to be only participants in this long and challenging process. Many other people and a variety of companies, under the leadership of ITER Organisation and Fusion for Energy, have participated and been absolutely key in the design and construction of this complex and multidisciplinary supporting structure throughout the different phases involved. This paper just intends to provide a very quick overview of this process from the personal point of view of the engineers that have written these lines.
Examples of how solutions for difficult problems were found

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Abstract
Applying some Principles of Conceptual Design, as discussed in Madrid, like Stiffness, Shortest Path or even Constructability, some examples are described, divided in 3 groups:

1. New design of a section of the S. Paulo Metro, line 3. The idea is to describe the difficulties of the interaction of a small viaduct, over the ground and a thick soft soil layer, and the continuous welded rail required by the S. Paulo Metro Company.

2. Existing structure with problems, like the Italy Tower, a reinforced concrete frame structure, founded over Franki piles, that collapsed in S. Jose do Rio Preto. The idea is to describe the difficulties to find the collapse reason, the diagnostic, as the structure verification shows no problem and the building inclined more than 30 degrees before collapse.

3. Execution problems along the Cut and Cover section of the S. Paulo Metro, line 3. The idea is to describe how the construction technics need to change due to changes in the cement industry, in this case related to the speed and heat generation of the cement hydration reaction. The wall, concreted some weeks after the bottom slab, is heated during the cement hydration but, when the reaction declines, needs to cool down and contract, but the slab does not allow and the wall cracks. Usually one crack each 5 m, 1 mm open. This can also happen in a long box girder bridge. This behaviour begins in the 70ths due to cement changes in Brazil, but how to understand it, create and dimension the solution?

1 Introduction
This paper discusses the difficulties to understand clearly the 3 problems defined in the abstract and to define the solution. The 3 problems were uncommon, at least at the time they appeared, and to be able to overpass the difficulties it was necessary to be obstinate and not to give up the search. In a certain sense it is necessary to have faith in our ability, what is not always true, but is always useful. In all 3 cases the problems were unexpected, complex and of course difficult to understand. When a possible explanation appeared, it was necessary to look for some kind of confirmation, through facts or even experiments. The use of long continuous welded rails is really a problem when the viaducts are made of not continuous spans, around 30 m, that suffered strongly in winter due to low temperature and the rail can fail, presenting a dangerous opening in the failure section. This already happen in S. Paulo, in a winter and rainy day when the D. Pedro II station, line 3 red, in rush hour, trembled strongly making all the people afraid to run out of the station. This problem would be worse in the small viaduct, 10m span, like in the line 3 between Bresser and Belem stations, mostly because of soft soil thick layer. To have an idea it is some 8m with around zero SPT.
In the case of Italy Tower, after verification shows that structure was ok, confirmed by the fact that it was able to stay some hours inclined more than 30 degrees, it was necessary to find a reason for the collapse. A good idea appeared remembering that piles do not respect columns requirements and could present brittle failure. Looking for Franki piles load tests it was discovered that, in a cap with many piles, the critical one had 6, there could be an important stiffness difference and the hypothesis of uniforme vertical load distribution should be abandoned. In the 70ths the Brazilian Cement Industry began to change the cement composition trying to attend the people needs for better compression strength (fck) also in early ages. This was obtained by using more fine cement, with the problem of increasing the speed of the hydration reaction and generated heat. This decision creates problems in thick walls like in Metro Cut and Cover and sometime later in pavements over bridges and viaducts.

2 Bresser-Belem Elevated Railway Track – S. Paulo Metro, line 3.

In the end of 70ths, Maubertec, where the author worked at that time, begun to design the line between the stations Brás, Bresser and Belem. As there were a thick soft soil layer engineers of Metro Company were afraid of the settlement due to densification and suggested a small viaduct. Pre-loading landfill as well soil exchange was also discussed, but time and cost of construction conducted to the viaduct solution. The pre-dimensioning defined a structure of two beams for each line, 10 m span, simply supports by caps over 2 Franki pile Φ 52 cm, but as the Metro required continuous welded rails it was necessary to calculate the structure for low rail temperature and rail failure. At that time there were already a good text defining the criteria for calculation of the interaction between the rails and the viaduct, proposed by the consortium [1] HMD – Hochtief, Montreal and Deconsult hired by the São Paulo Metro to write a manual for designing Metro Lines. The structure should be verified for a decrease of rail temperature of 35 degrees end for the case of rail failure, when a force of 640 kN should be applied to each face of the failure of each one of the 2 rails. The force of 640 kN corresponds to the force needed to prevent any deformation of the rail TR57 (73 cm²) under 35 degrees of temperature change. As at the time there were already programs able to calculate the fall of temperature, it was decided to use it. After a good discussion it was decided to limit the rail tension stress to 2 tf/m² and, in the case of rail failure, one should consider 2 rails failure and an opening limitation to 3 cm. The structural model is represented in figure 1.

![Figure 1 – First idea – simply supported 10m span beams over 2 Franki piles](image)

The model shows that in the assumed position of the possible failure the opening would be 15 cm, a big danger for the metro operation. What would be a solution for this problem? After some discussion appeared the idea of create some stiffer supports. A pre-dimensioning shows that a cap over 8 inclined steel piles each 50 m would be enough. The connection of the cap to the deck should be strong and stiff enough to support the horizontal force with small displacements. Figure 2 shows the modified model and figure 3 the connection of the deck and the stiff cap. In fact, the opening was limited to 3 cm, and Metro Company decided to use the solution all along the section with soft soil, what includes some of the stations, like Bresser.
Figure 2 – Final Idea – Undisplaceable supports with 8 inclined steel piles each 50 m to limit the rail opening at rail failure section

Figure 3 – Additionally to the bearings for vertical load (red) the beams were connected to the cap by bearings for horizontal ones (blue). Note the rail passing over the joint.

The rail (TR57) fixation on the deck was made with Landis type as showed in figure 4. The Landis fixation has an elasto-plastic behaviour with stiffness of $K_l = 16,000 \, \text{kN/m/m}$ and strength of 120 kN/m.

Figure 4 – The fixation of the rails on the deck is of Landis type, with an elastomeric plate.

It is important to call the attention of the reader for the fact that for long bridges or viaducts usually built for High-Speed Train, till 1km for example, with one joint in the rail each joint in the deck, the temperature changes are not important for the design of the structure, only to evaluate the forces on the fixations. This is clearly stated in the Spanish code – [2] IAPF – 2010. The Eurocode [3] EN 1991-2, generally speaking, states that the structures should be designed considering the temperature change in the rails of 35 degrees, as it was considered here.
3 Collapse of Italy Tower in S. José do Rio Preto, S. Paulo, Brasil.

Italy was a 16-floor tower and attic built above a common use floor (PUC) with other two towers Portugal and Spain. Below PUC floor there are 2 parking floors and the ground floor for commercial use. The typical height floor to floor is 3.15 m, growing in the lower floors up to a maximum of 5.2 m at the ground floor. The structure of the typical floor features a skew angle of approximately 38° in relation to the lateral street Luiz de Camões. This rotation made the structure quite complex and Luiz de Camões facade jagged (see Figure 5 for shape of typical floor).

Demanded also a transition at the PUC level, part parallel to Front Av. Bady Bassit and part parallel to Lateral Street Luiz de Camões (see Figure 6 for shape of the PUC floor). It should be added that this is not a conventional structure, both by the transition and by the complexity of the typical floor. This complexity led on the one hand to the creation of unfilled frames and to the increasing responsibility of the 2 elevator towers. Below the ground floor are the foundations (no basements) built up of caps on Franki piles φ52 cm. These piles were performed with lengths around 10 m. See Figure 7. The profile of the subsoil consists of a thick layer of loose clayey sand, based on a compact sand layer on top of the sandstone. The piles were practically supported on sandstone, soon after crossing from 1,5 m of compact sand. The water level is 3 m deep.
3.1 Description of the collapse

The collapse is described in detail in section 3 of the [5] IPT report. This description is supplemented by the testimony of the Manager of the Portugal-Spain-Italy Condominium, Mr. Valter Lazarus, and by observation of the photos. From these data gathered about the collapse, it is considered especially important the following aspects:

a) Before the collapse, the building showed no signs of anomalies, such as cracks or differential settlements.

b) Collapse began with a bang similar to a transformer overflow at 2:00 in the morning.

c) The glass and the facade frame broke and twisted due to significant differential settlements, for a few hours (2:00 to 6:00am).

d) Around 5:30 Mr. Valter went upstairs, beside column 62, to call families who were still in the building. Rising noted that the plastering of walls and ceilings fell down.

e) In the collapse Italy Tower would have rotated a bit around its vertical axe, opening the joint with Spain Tower in the back and started settle slowly as a whole, more in the back than in the front. In a certain moment the building stops to settle as a whole and would continue to rotate till an inclination of more than 30 degrees. Suddenly and quickly the structure collapsed. The collapse occurred day 16th/Oct/97. See Photo1.

f) Observing the pictures of the debris 3 additional important aspects are noticeable:

- The piles of the columns of Av. Bady Basset façade show tensile failure of the rebars.
- These same columns show tensile failure in splices at the PUC level. See Photo2.
- The upper floors, when falling on the lateral street Luis de Camões, where positioned in inverse order to the natural, i.e., the 9th floor is over the 10th, which rests on the 11th, and so on, all upside down. This fact is observed from the 6th floor to the roof. See Photo 3.
It is necessary to point out that this behaviour is unusual. In a structural collapse, usually the structure falls vertically. When the collapse is linked to the Foundation, sometimes the structure tilts and falls laterally, but not upside down and not too far.

3.2 Search for the diagnostic
In these circumstances, the verification began 8 months after collapse, it is more difficult to determine the cause of the collapse, as the verificators couldn’t visit the collapsed building. Data collected by the IPT helped a lot. Structural defects, or even other, in a properly reinforced structure should generate a collapse more ductile, with more warning than the observed. So, it is more appropriate to seek the cause in very rigid elements and at the same time very brittle and also to interpret how the kinematics of collapse occurs and find out what are the parameters that could explain it. Initially it is important to consider the largest number of collapse hypotheses. The creation of these hypotheses, regardless of them can be proven or not, must come from the analysis of the available data and the results of a calculation model. The available data, especially those gathered in the IPT report, pointing to considerable deviations of construction, emphasizing the following ones;
- overlay thickness for slabs and walls far greater than those provided for in project
- deviation of the columns in relation to the transition beam
- columns with reduced dimensions (P64 - 65 x 80 instead of 80 x 80 as designed)

These deviations were at least partially compensated by a concrete better than specified. Concrete fck 25 MPa for the columns and 20 MPa for the beams, instead fck 18 MPa. So, two verifications are fundamental: the designed structure, from the design drawings and the structure as built, from IPT inspections and measurements. Secondly because the models used at the time of the design (1982 about 40 years ago) does not meet the requirements of [6] CREA, that contracted the verification. In fact, in 1998, a verification of this kind should consider at least a frame elastically supported by the Foundation, which was not possible or usual at the time of the design.

3.3 Calculation Model
The choice of model is also a difficult and delicate task in a verification like this. As can be seen from the calculation memory, the model used in the design of the Tower includes, for the study of wind, a set of 2D frames chosen by the designer, fixed on the foundation. As the building has a complex structure, function of architecture, and has an important transition in the PUC level, it is convenient to use a 3D frame, including the foundations, in order to examine further the soil-structure interaction and to show if the decisions of the designer were enough good. In order to not deviate too much from the usual practice it was decided to make a linear analysis. To evaluate the geometric non-linearity, i.e., the second order effects, it was decided to use the coefficient \( \gamma_c \), as proposed in the revision of [7] NBR6118/78 in discussion at that time. The final decision was to use the TQS, Brazilian linear elastic software that models, calculates, scales and details automatically the structure. This software is very convenient because it allows to quickly verify a large number of different collapses or each of the deviations of construction found. If the nonlinearity of an element needs to be considered, it can be done approximately through equivalent secant stiffness. One important problem that should be remembered is the
stiffness of the columns for normal forces. As columns are submitted to really different stresses, one need to correct its stiffness, in order to avoid significant unreal differential settlements. In fact, during the construction process, floor by floor, the model should simulate the construction, correcting the column length floor by floor, neutralizing significantly that kind of errors.

3.4 Situations to be verified

The goal of this item is to discuss in general terms the verification to be done and the sequence in which they must be performed.

A - Designed Structure, considering only the data contained in the design drawings for foundations, structures and architecture

B - Structure as Built, considering the design data and only the construction deviation explicitly identified in the rubble.

As already mentioned above considerable construction deviations have been observed specifically in Italy Tower, noting the following:

- overlay thickness of slabs and walls far greater than those provided by the design drawings, as identified by IPT, causing considerable increase of load.
- columns with reduced dimensions (P64 designed 80 x 80 and built 65 x 80 cm).

As already said, one should seek brittle causes for the collapse. The points that initiated this collapse should be sought on the basis of the two previous verifications A and B. The designed structure A did not indicate potentially critical points. Verifying the structure as built B showed that it was still acceptably safe, but at least two potentially critical points that could be the source of the collapse: the Transition-Beam and the Foundation, mostly column P62. Looking for brittle elements it was found the piles, with reinforcement smaller than the minimum for columns, they may present a structural brittle failure that occurs without warning, as the collapse in question. If its failure is geotechnical, no problem, but if it is structural, it may be the problem, and this is the case as the ultimate structural resistance is far smaller than the geotechnical one.

Trying to identify cases that could create a brittle failure 4 hypotheses of collapse were proposed:

- Eccentricities on startup of the columns from the transition beam at the PUC level.
- Defect in a pile
- Construction geometrical errors when beginning the driving operation.
- Piles with different stiffness in the same cap

3.5 Safety criteria

3.5.1 - Foundation safety

As usual it was adopted the verification of the piles by working loads. So, for design with only vertical loads, the load limit for the average in 1 cap was 1300 kN per pile. When the vertical loads are composed with the wind, according to Foundation Design and Execution – [8] NBR 6122/86, this value was increased by 30%, reaching 1690 kN per pile (1300 x 1.3). As in the last 20 years the load limit for these piles increases from 1300 to 1500 kN, due to load tests, it was decided to accept this new limit; 1500 kN for vertical loads and 1500 x 1.3 = 1950 kN when wind forces are considered. This ultimate structural capacity varies from 2400 kN for concrete of fck 14 MPa (minimum obtained from boring of specimens by IPT) to 2800 kN for fck 17 MPa.

3.5.2. Structural safety

In the verification of the structure, the safety criteria are based on the Method of Limit States according to Design and Execution of Reinforced Concrete Works – NBR 6118/78 and its review under preparation at the time (98) approved in 2003. According to these criteria, one should treat the safety of the Designed Structure differently than Structure as Built.
A. Designed Structure
The structure was designed for the following actions: weight, masonry and overlays, live loads, wind, constructive eccentricities, second order effects, global and local. The partial safety factors in Brazilian codes were at the time – 1998.

$$S_d = \gamma_{fg1} S_{g1} + \gamma_{fg2} S_{g2} + \gamma_{fl} (S_q + 0.8 S_w)$$

with $\gamma_{fg1} = \gamma_{fg2} = \gamma_{fl} = 1.4$

Resistances were: concrete $f_{ck} = 18$ MPa, $\gamma_c = 1.4$ and steel $f_y = 500$ MPa, $\gamma_s = 1.15$.

B. Structure as Built
The structure as built was verified for the same combination of actions, by changing some values depending on the knowledge generated by IPT surveys. This additional knowledge allowed us to reduce the partial safety factors:
The new safety factors were: $\gamma_{fg1} = 1.3$, $\gamma_{fg2} = 1.25$, $\gamma_c = 1.26$.

C. Collapse Hypotheses
When verifying each of the collapse hypotheses the following actions were considered:

a) vertical loads for the structure as built, that is, according to the IPT surveys.
b) wind Loads, equal to 30% of standard loads (30% in pressure), since at the day of the accident the wind was moderate. (basic speed of 19 m/sec).
c) A collapse hypothesis [8] NBR 8681/80

To verify that a structural element had not contributed to the collapse or failed, it should show for the same combination of actions previously set, plus one collapse hypotheses, the following safety factors:

$$\gamma_{fg1} = \gamma_{fg2} = \gamma_{fl} = 1.1; \gamma_c = 1.1 \text{ and } \gamma_s = 1.0$$

3.6. Verification results

3.6.1-Designed Structure
Analysing the foundations for the case of vertical actions only or the one considering also wind, all results were acceptable. It is important to note also that the adopted calculation model showed that the caps of the columns P61 and P62, linked by a beam works as a single cap. See Figure 7. The stability parameter $\gamma_z$ reaches a maximum of 1.13 which indicates the need for consideration of global second-order effect, what has been done. Only to have an idea, $\gamma_z$ is defined by the equation below (from NBR6118):

$$\gamma_z = \frac{1}{1 - \frac{\Delta M_{tot,d}}{M_{tot,d}}}$$

is the second order effect

$$\Delta M_{tot,d}$$

is the first order moment

Analysing the structure, it was concluded that the reinforcement of columns and beams determined from all calculated forces resulted deficient in some columns and beams. Considering only vertical actions, no deficiency has been found. The differences, however, show consistency, that is, there are pieces with more than needed and pieces with lack of resistance. In fact, as the designer chose some columns to resist wind, these were the ones with safety margin, the others with lack of resistance. In order to improve the significance of the results from the point of view of non-linearities, the elastic stiffness of some elements was replaced by its secant value. This is not usual but could and should be made here to better evaluate the ultimate resistance of the whole building. These corrections changed the distribution of forces, increase the displacements and the stability parameter $\gamma_z$ reaches 1.24 also acceptable. The deficiencies were not high and as the sum of resistances in each level was greater than the sum of forces, it was decided that plastically the designed structure was OK.
3.6.2. Structure as Built
Checking of the structure as built, as defined previously, considered the conditions for the designed structure with 4 amendments: loads of floor overlay as IPT survey, loads of masonry overlay as IPT survey, column P64 dimensions of 65 x 80 as built, instead 80 x 80 as designed, concrete resistance from specimens taken by IPT - fck 25 MPa for columns and 20 MPa for the beams. For the case considering only vertical actions, the pile load varies fairly, and are critical in the P61/62. The mean pile loads were higher than 1500 kN and not acceptable. Combining vertical loads and wind the maximum pile load was higher than 1950 kN and also not acceptable. In conclusion, the structure as built has safety below the required standard, requiring reinforcements. The deficiencies, however, are far from explaining collapse.

3.6.3 - Collapse Hypotheses
The Collapse Hypotheses studied were defined in 3.3. The results will be described in this section, grouped according to those guidelines.

3.6.3.1 to 3 - Eccentricity of the columns born from the transition beam, pile defect, pile positioning errors
These Hypotheses do not identify a brittle failure able to explain the collapse. Piles present always good redistribution and the transition beam, despite of loss of safety were far from collapse and were also ductile.

3.6.3.4. Piles with different stiffness supporting the same cap
A research on φ52 Franki pile tests, concentrated in short ones (from 6 to 10 m long) with point in rock, shows that their stiffness could change a lot, between 1 to 3.5. Within these Hypotheses and this variability, at least 6 cases were verified, all related to the cap P61/62. See Figure 7. The idea of these cases was to find a possible failure sequence that could start the collapse. The results show in first place a great sensitivity to the variation of stiffness. Secondly, they show that it is not difficult to obtain pile loads overpassing the limit established between 2400 and 2800 kN. The maximum pile load found were 3570 kN, applied to the pile E108. These results also show that the failure of one of these very stiff piles could detonate the collapse, with a pile failure sequence. It is important to add that, in each of these cases the columns were verified just above the cap. This verification showed that these elements had enough strength to impose these additional loads to the piles.

3.7 - Conclusions
The findings lead to the conclusion that: The design, both structural and foundations, are primarily suitable for the time they were developed. The differences detected are acceptable and certainly have nothing to do with the collapse of the building. The execution of the work introduced, as IPT survey, additional loads and geometric imperfections that should not be executed without the designer’s verification. Additional loads (+25% in total) exceeded the boundaries of acceptability, but were not so dangerous because it was compensated by a better concrete than specified. The increased load on these piles reduces the safety required and Foundation strengthening would be necessary. The piles should be loaded at least till 2400 kN to justify the collapse, which was only reached through the hypothesis that some piles were stiffer than the nearby supporting exceptional loads (up to 3570 kN in the cases studied) and displaying brittle fracture. It was considered, therefore, that the decisive factor of the collapse was the application of the conventional criterion of uniformly stiff piles to a case of short piles with point in rock where the geotechnical load capacity is much higher than the structural strength. It was suggested to open a revision process of the foundation’s standard, discussing this problem and proposing specific criteria for cases like the present.

4. Cut and cover execution problems along the S. Paulo Metro, line 3
The usual solution for buried galleries in São Paulo Metro was Cut and Cover made in an excavation supported by steel piles and wood planks. To have an idea of this gallery it is represented in figure 8.
In the 70ths the Brazilian Cement Industry began to change the cement composition trying to attend the people needs for better compression strength (fck) also in early ages. This was obtained by using more fine cement, with the problem of increasing the speed of the hydration reaction and generated heat. This decision creates problems in thick walls like in Metro Cut and Cover and sometime later in pavements over bridges and viaducts.

The wall, concreted some weeks after the bottom slab, is heated during the cement hydration but, when the reaction declines, needs to cool down and contract, but the slab does not allow and the wall cracks. Usually one crack each 5m, 1mm open. See figure 9.

As people was used to build this kind of structure without problems, after the appearance of the wall cracking, the 1st idea was to reduce the temperature using cold water and, where possible, using steel formwork. The result was not good, more cracks appear due to differential temperature along the thickness of the wall. The second idea was to avoid trying to cool the wall and using wood formwork. The cracks were still there. The solution came when trying to study the thermodynamic problem and people discovered that the cement had changed and had now a more rapid hydration reaction that allowed a more rapid gain of strength, but also produced a higher increase of temperature that causes the cracking problem. The theory confirmed that the cracks were due to the increase heat produced by the hydration reaction and the only solution was to reduce this heating using cold water, cold aggregate or even ice. The experience shows that one should avoid temperatures greater than 50 degrees and calculate the amount of cold water or ice necessary for that. This problem is not only important in very wide galleries like described in the Shanghai fib Symposium by Gu, Kunpeng, but also in usual galleries like the one presented here, and probably these problems will depend on the local climate of each country or work.

5. Conclusion
The examples described above clearly justify the importance of a good understanding, not only of the structure behaviour, but also of the structure construction, to obtain a good conception of a new one or a good diagnosis of bad response of an existing one. Of course, we need to add the concern about the execution where, not only the codes of practice, but mostly what is described by the design, must be clearly understood and respected in terms of materials, geometry and construction sequence, to avoid an unsafe final result.
References

[2] - IAPF – Actions to be considered to design railway bridges – 2010
[6] - CREA-SP Process of Italy Tower - SF/97 40255 from 12/30/97
[8] - NBR 6122/86 - Foundation design and execution
[9] - NBR 8681/80 – Actions and safety of structures
Challenging gravity: the beauty of buoyancy

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Abstract
Buoyancy acts upwards and counteracts gravity. This may be used favourably. For obvious reasons you need to construct the marine concrete structure elsewhere from its destination. Weight is important for structures that shall float, clumsy structures do not float. This calls for proper design, including choice of structural shapes. Shell structures are often used as they are efficient in carrying distributed loads. At the same time design must be tailor-made to the construction facilities and capabilities. Some elements of conceptual design are not so difficult; the purpose of the structure is one of these. It is important to understand and have passion for the purpose of the structure. And you must know what to optimize on. These and other elements of conceptual design are outlined, through principles and examples.

1 Introduction
Concrete bridges and quays must sometimes resist sea water. They are often, but not always, built in situ. This paper concentrates on marine concrete structures that need to float, during construction, towage, installation, and operation. These structures benefit from buoyancy. Reference [1] gives an overview of floating concrete structures and indicate why they will be more important in the future. Reference [2] describes submerged floating bridge tunnels, an important future structure, but not described in detail here, although it is a perfect example of where buoyancy helps carrying gravity loads, Fig. 1.

Fig. 1  The submerged floating bridge tunnel

Floating concrete structures are not a novel concept. Lambotte built his concrete boat in 1848, Fig. 2. The British built Mulberry Harbour during World War II to aid invasion of Normandie, Fig. 3. The floating bridge in Fig. 4, in Washington State close to Seattle, carries heavy traffic. Fig. 5 shows the harbour extension in Monaco.

Fig. 2  Lambotte’s boat from 1848  Fig. 3  Mulberry Harbour
These four examples illustrate the different purposes the structures have, leisure, warfare, car infrastructure and ship infrastructure. It is worth noticing that the harbour extension also solves a space issue in Monaco, which Monaco needs. These purposes must be understood to create a proper design. For bridges most nations have detailed descriptions of loads and other criteria, for other structures there is a job to be done to establish the basis for design.

2 Basis for design

In the early phases of conceptual design, there are frequently little money and time available. The client wishes to confirm that his or her idea is technically feasible and that costs are within the budget of the business plan. It is then crucial to concentrate on the most important aspects, and not be misled into details of minor importance. This takes practice, but the practice can be trained.

Typical most important basis for design may be:

- Purpose of the structure, indicating size and shape
- What to optimize on
- Loading

Loading includes waves, wind and current, and requires knowledge about where the structure shall operate, and how to get there. The latter requires knowledge about construction place and method, not always known initially.

Typically, the basis for design becomes more detailed as design proceeds. If it proceeds, it does not always do, and that may be a good solution.

An advice for efficient conceptual design; be brave and daring:
Fig. 7 Be brave and daring in your conceptual design

If too daring, you will get the experience and you will have to moderate the design.

3 Weight and floating stability

It may seem obvious, but weight is very important and given some attention here. Not only the magnitude, but also its location. This is illustrated in Fig. 8.

Fig. 8 The beauty of buoyancy and the importance of the location of weight and buoyancy. To the left buoyancy and weight balances, next the floating structure will tilt and maybe capsize, and next the floating structure will sink, and lastly the floating structure will rize and maybe tilt.

Many of us have experienced this practically, for example in a canoe or a small boat. A danger is the internal water that may eliminate the positive effect of the stiffness of the external swim area, this has caused many car ferries to capsize. Real structures may have many compartments with water in it, Fig.9. The compartments will reduce the negative effect of the internal water and improve the hydrostatic stability. The compartments will also reduce the consequence of a possible leakage or failure.

Fig. 9 Floating stability.
Fig. 10 The oil platform Gullfaks C, to the left, weighed 1.5 mill. tons when transported several hundred kilometres to installation in the rough North Sea. The Troll A gas platform, to the right, during tow out from a fiord, carrying 22000 tons 150m above sea level

4 Construction and cost

The construction is a very important part of the conceptual design of marine concrete structures that shall float. The importance of weight requires accurate weight control.

Small structures may be built on a dock and lifted out, either by a land-based crane or a seagoing crane. Long and narrow structures may be built in a ship dry-dock. Large and more circular structures may need a large graving dock, alternatively built on a barge. Creative ways of coupling structural parts may be fruitful.

Fig. 11 Salmon Home #1 built on a dock and lifted into the sea by a seagoing crane. Weight 250 tons

Fig. 12 Shore approach protecting and bridging two gas pipes, built in a ship dry-dock
Sometimes it is possible to improve the buoyancy-weight relationship, by providing voids in the concrete where the concrete is less effective, e.g. bubble-deck, or by using Styrofoam. Fig. 14 shows two examples of the latter. In both cases water pressure is allowed in to eliminate unwanted hydrostatic pressure on the walls. This may make it possible to suffice with central reinforcement only, effective for weight saving as the cover requirement for concrete structures in the sea is large.

Again, for weight reasons, LWA concrete is frequently used. Reference [3] describes an environmentally friendly all LWA concrete with an unreinforced unit weight of 1.6 t/m³. A bit more expensive than ordinary concrete, but often worth it.
Another cost element is the reinforcement. Slender sections need more reinforcement per unit \(m^3\). A simple one-parametric example illustrates this, by means of a floating box of outside dimensions 8mx8m, 20m high, freeboard 1m, two thicknesses 0.3 and 0.7m. Reinforcement 20mm bars center 200mm plus as required to resist external water pressure, Fig. 15.

The thin-walled box is less expensive and has a better GM (better floating stability) as compared to the thick-walled box. This is not surprising; it simply demonstrates the follow-on effect of increased hydrostatic pressure due to increased weight. This very simple example illustrates how weight and hydrostatic pressure interrelates, with consequences for the cost.

Another simple parametric exercise is that of concrete strength and unit weight, [4], first published in 1988 when preparing for platforms for great depths, and guiding Norwegian research on the importance of high strength and light weight aggregate concrete (the Troll A platform, installed in 1995, sits in 303m water depth). The exercise considers a unit cylinder that shall float, carrying its own weight plus a payload.

Fig. 15 One parametric (wall thickness) sensitivity evaluation for a floating box

Fig. 16 The effect of strength and unit weight of concrete, on concrete volume required for a unit cylinder
These simple parametric evaluations turn out differently for a floating body, as compare to a land-based structure.

5  Contemporary floating concrete structures

Marine concrete structures have been proposed and used for floating solar energy, tidal energy, wave energy and wind energy. A floating wind energy structure will be shortly described here, for its conceptual values.

Fig. 17 shows a concept for a floating wind turbine, differing from deep draft floating wind turbines.

Fig. 17  The OO-Star floating wind generator

An obvious benefit is that the concept can work where the water depth is not large. Other benefits are favourable motion characteristics in waves, the possibility to lay dockside and be equipped, and being able to be built on a barge or in a shallow graving dock.

Salmon farming is traditionally performed in open nets, having challenges with lice, algae, escape, garbage to the sea, and overfeeding. An obvious solution is a closed cage, collecting water from clean depths and collecting garbage, dead fish, and feed remnants.

Salmon is an animal and deserves respect, and a happy salmon is also more valuable than an unhappy fish. The Salmon Home #1 has been mentioned, it works perfectly well, another much larger cage is the Stadionbasseng, Fig. 18.

Fig. 18  The Stadionbasseng

To design a fish cage, knowledge of the fish welfare is essential. It needs fresh water, right temperature, right amount of oxygen, and right feeding, to name a few, and the cage needs to be clean and debris cleaned away.
The internal water volume of the Stadionbasseng is 37000m³. Changing the water every 40 min, for fish welfare reasons, means pumping in 16m³ water per second and directing the flow in a favourable way for the fish, and for the cleaning of the inside.

The submerged floating tube bridge is another type of structure that will emerge as requirements to the environment and infrastructure become stricter. fib Bulletin 96 describes this in detail [2].

fib Bulletin 91 [1] describes more examples of floating concrete structures, it identifies 12 different sectors of applications, there may be more for the creative designer. Also included are explanations of pontoons and anchoring, tools often required.

6 Conclusions

Conceptional design is rewarding because it is important for the result. It is rewarding to work with for that reason, and because you need to be involved in the overall aspects of the purpose of the structure. And it is rewarding because it can be learned, you do not have to be super intelligent, but you will need to be a bit patient.

As mentioned, passion is important. My first conceptual design, as a 4-5-year-old child, learning from my father that concrete does not burn. The solution, in my head, was a concrete box for my beloved teddy bear, fig. 19.

Fig. 19 The teddy bear protection box

If you are not experienced yet, best of luck, be passionate, think totality, train, and be patient.

References

Infrastructure design: the benefits of contextual and conceptual considerations

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Abstract

Infrastructure design is a task that primarily falls under the remit of civil engineers. In order to improve the design process, these civil engineers must employ a broader perspective that takes account of the contextual and conceptual considerations inherent to the field of architecture. To demonstrate this, a number of infrastructure projects drawing on contextual and conceptual considerations are presented and discussed, and the benefits of these considerations are highlighted.

1 Introduction

Built structures can be divided into two primary sub-groups: buildings, defined as “usually roofed and walled structures built for permanent use (as for a dwelling)”, and infrastructure, defined as “the resources (such as personnel, buildings, or equipment) required for an activity”.

This article concerns infrastructure in the generic sense, which covers the construction of transportation routes as well as factories and plants. Here, the term is not restricted to underground constructions as is typically the case in the field of civil engineering.

The following discussion stems from the observation that existing infrastructure design methods are not especially advanced and focus primarily on technical and functional aspects. However, infrastructure projects require technical and architectural planning of a high calibre and achieving this largely depends on the underlying design work. This is all the more true in light of recent developments in digital tools, which have resulted in a range of new solutions, possibilities, and risks.

2 Traditional Infrastructure Design

2.1 Design – A Subjective Act of Creation

Design is a creative activity, as defined by Huntley [13]: “The act of creation is when an original thought is born, and it is the same act in original science and original art”. This also applies to the design of infrastructure since each infrastructure is a unique creation, a prototype.

Another noteworthy definition is that provided by Clement Greenberg [11], which defines the “area of competence” specific to each of the arts as “coinciding with all that was unique to the nature of its medium”. One art cannot borrow its medium and its effects from another. Each art must strive for a sense of purity that provides assurance both of its quality standards and its independence. As such, infrastructure design must be considered as a specific area with its own medium.

As recalled by J. Lucan [17], Nicolas Durand specifies that: “For a project to be well designed, it must be the result of a single act; this is only possible if one is thoroughly familiar with all of the parts that must be incorporated into its composition, otherwise there is a risk that too great a focus on detail will detract from the project as a whole”.

As such, the design of infrastructure can be considered an act of creation with its own medium; an act that is better performed by experienced individuals.

2.2 Differences in the Design Processes of Architects and Engineers

Infrastructure is usually designed by civil engineers, and it is interesting to look at where the line is actually drawn between civil engineers and architects.

During the Renaissance period, the difference between architects and engineers was based on the work they did, as described by Andrew Saint [28]: “On the whole, the terminology of the time applied not to the person but to the job he was doing. If you designed secular or religious buildings, and their adornment, you were likely to be called an architect; if you designed forts, walls, towns, ports, canals or machines for war and peace you were likely to be called an engineer”.

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He also points out the following [28]: “Bridges were the structural type first affected. Until about 1830 major bridges were often built by men classified as architects. Thereafter, they passed into the hands of engineers”.

It was during the Industrial Revolution in the 19th century that the increasing complexity of construction techniques culminated in a specialized branch of scientific and technical expertise. This led to the separation of the professions of architect and civil engineer: “It is plausible, then, to ascribe the widening gulf between architectural and engineering skills to more complex materials and structures, and the need for specialized calculations.” [28].

2.3 Infrastructure Design Methods

One design method applied in the field of infrastructure is the use of the golden ratio in the construction of stone arch bridges, as recalled by Huntley [13], and by Leonhardt [16].

One of the first publications on bridges is a book written by Andrea Palladio [25]. In 1570. In his third book, he sets out a number of principles to follow in order to construct attractive and durable bridges. In reality, the book concerns the application of a particular type of bridge design based on observations of other structures, rather than the design process itself.

It was in 1714 that Henri Gautier [10], one of the first inspectors of bridges and carriageways, made mention of the fact that bridges could be built in different ways depending on their location and the available materials. However, in order to compare different types of structures, one must be able to apply the technique of “structural analysis” as referred to by Kurrer [15]. The first theories on vault calculation date back to the 17th century. Beam theory was then developed by Euler, Bernouilli, and Navier in the 18th and 19th centuries. Subsequently, a method was developed by Cross in the 20th century for analysing hyperstatic structures. This confirms that, bridge analysis leading to bridge design is a new science.

One of the first conferences on infrastructure design was held in 1996 in Stuttgart under the auspices of Professor Jörg Schlaich. He defined design in [29] as: “the birth of the structure, at which point all of its characteristics and qualities are defined”. This notion of “birth” is linked to that of creation mentioned above.

At this conference, it was noted by Christian Menn [23] that “the objective of design is to produce an optimal blueprint in terms of economy of elegance that takes account of the location, the functional requirements, and the dimensional constraints”. He explicitly mentions the importance of elegance, aesthetics, and the site or surroundings; but does not provide a method for incorporating those aspects into the design process. In a similar manner, Michel Virlogeux [36] states in his book: “The chosen structure must express the desire of the designer [...]. it is necessary to consider both rhythm and harmony”. However, no methodology is described.

In order to help engineers in their design work, a number of guidelines have been published. This includes the Swiss Federal Railways manual published in 1992 and co-signed by Uli Huber [12] regarding the aesthetics of the company's structures. However, these guidelines tend to offer examples rather than methods.

Finally, the publications by Christian Menn [24], Jürg Conzett [6], and Philippe Menétrey [21] all conclude that design is a process of synthesis. However, a bridge design guide was recently published by the International Federation for Structural Concrete (fib) [9], which talks about the design process being based on the principle of breaking a project down into sub-problems for which designs must be drawn up and subsequently incorporated into a holistic solution. This design methodology, according to which a structure is the sum of its individual parts, can be problematic if those parts cannot be aligned. It differs from the synthesis approach referred to above, thereby adding to the confusion and illustrating the lack of consensus in infrastructure design.

It can thus be concluded that a design methodology for infrastructure does not yet exist.

2.4 Constructive Logic in the Design of Infrastructure

Constructive logic is an essential component of infrastructure design that cannot be dismissed: “structural safety, serviceability, and durability must all be incorporated into any design for a bridge, without compromise” [23]. When it comes to infrastructure, constructive logic must take precedence. Given that infrastructure is subject to particularly heavy use, it is important to mention environmental factors and service life.
The Tamis Bridge shown in Fig. 1, designed by Christian Menn, a structure of great finesse, is a good illustration of the power of constructive logic.

![Tamis Bridge](image)

Fig. 1 Tamis Bridge, Christian Menn, 1963

Moreover, constructive logic makes it possible to design harmonious structures, as pointed out by Le Corbusier [7]: “Engineers are also architects, as they use calculations based on the laws of nature and their works provide us with a sense of harmony. There is therefore such thing as an engineering aesthetic, since performing calculations necessarily requires certain factors of the equation to be defined, which is ultimately a matter of taste. However, the manipulation of these calculations is approached with a pure state of mind, and it is this state of mind that steers those tastes along the surest paths.”

Nonetheless, a trend has emerged in recent years towards structures with special forms that do not comply with constructive logic. The power of structural analysis programs enables this kind of bias. In addition to their questionable aesthetic, these structures are also found wanting in terms of fitness for use and durability as mentioned by Walter Kaufmann and Beat Meier [14], who state that the recent developments in structural calculation tools must be accompanied by a thorough design process in order to avoid the construction of inadequate structures.

2.5 Aesthetics in the Design of Infrastructure

According to the definition given by Carole Talon-Hugon [32]: “Aesthetics is a reflection on a certain set of objects dominated by the notions of beauty, sensibility, and art.”

Aesthetics has always been a matter of interest in the field of architecture and construction. As early as 25 BC, Vitruvius discussed the notion of beauty in his treatise *De architectura* [34], dedicated to Emperor Augustus: “All these [types of building] must be built with due reference to strength, convenience, and beauty. Strength will be assured when foundations are carried down to the solid ground and materials wisely and liberally selected; convenience, when the arrangement is faultless and presents no hindrance to use […] and beauty, when the appearance of the work is pleasing and in good taste, and when its members are in due proportion according to correct principles of symmetry”. According to Vitruvius, beauty is one of the three attributes of any construction.

Viollet-le-Duc [35], a theorist of the rational school who made a great contribution to the adoption of a form of architecture inspired by engineering construction principles and who indirectly sparked interest in such constructions, said in 1863: “There are two ways of expressing truth in architecture: we must be true according to the program of requirements, and true according to the method and means of construction. Being true according to the program of requirements means meeting, in a precise and simple manner, the conditions imposed by need; being true according to the method and means of construction means using materials according to their qualities and properties. Questions considered as purely a matter of art, namely symmetry and outward form, are but secondary complements to our defining principles”. In his view, therefore, aesthetics is merely of secondary importance.

Likewise, J. Chaix [4] writes in 1890: “The same rarely applies to bridges, viaducts, and canal bridges, the principal dimensions of which are not left to the discretion of the architect, but are almost always governed by considerations wholly foreign to matters of art”.

As regards bridges, David Billington [2] states that: “The designer must think aesthetically in a way that structural form becomes structural art”. He actually encourages engineers to view aesthetic criteria in a different light during the design process.
F. Leonhardt [16] also emphasizes the question of aesthetics: “It is not possible to perform an exhaustive analysis of aesthetic matters through critical thinking alone; they are too deeply anchored in our emotions, where reason and logic are absent”. He adds: “The expression of aesthetic qualities is not born solely of the form, colour, light, and shade of the object, but also of the immediate environment in which it is located”. In saying this, he highlights the importance not only of the aesthetic of an object, but also of its context.

Aesthetics is therefore an aspect that must be considered when designing infrastructure, though this has not always been the case.

### 2.6 Summary of the Limitations of Traditional Infrastructure Design

The demarcation between engineers and architects leads to differences in the kinds of projects they work on: architects deal with buildings, and civil engineers with infrastructure. This in turn leads to a difference in their methodologies.

Architects consider projects as a whole, taking account of the surroundings, in which they are located as well as the space, light, image, and meaning.

Engineers, on the other hand, focus on technical and functional aspects. In abandoning the history of construction, civil engineers have also abandoned part of the culture of construction. Moreover, by concentrating purely on technical aspects, they neglect those aspects linked to the project's surroundings, image, and symbolic value. As such, engineers have used in practice what can be referred to as “traditional” infrastructure design, namely that which is taught in most universities. The resulting structures have sometimes turned out to be a disappointment, which has caused developers to introduce obligations for engineers to collaborate with architects.

The consequence of this, as pointed out by Kaufmann and Meier [14], is that two opposing trends have emerged in the field of bridge construction. The first entails the construction of “signature” objects, designed primarily to have symbolic value. Many bridges of this type are gargantuan sculptures that are both inefficient and expensive and run up extremely high maintenance costs.

The Hans-Wilsdorf bridge over the Arve in Geneva, which was built in 2012 and is shown in Figure 2, is indicative of the problem posed by “signature” objects. The bridge was intended to take the form of an open deck tunnel that provides a sense of protection, with a truss structure passing over the top of the road. The initial drawings veered towards a random truss design. As Émilie Veillon recalls, citing the architect responsible for the project [33]: “Comparing the two models (random and regular) allowed us to define a rational approach for the project whilst still maintaining the conscious image of a complex, random structure. In order to achieve that we further developed the model, and ultimately reduced it to two different truss elements that comprise the main body of the structure. Most of the work was devoted to discouraging a structural interpretation of the bridge in favour of a holistic understanding of the object”. At a structural level, it is the arches that link the two riverbanks and bear most of the load; the transverse elliptical sections hardly play any load-carrying role at all. Consequently, the quantity of steel and the thickness of the sheets required is no longer justifiable, warranted purely by formalistic considerations.

Fig. 2 Hans-Wilsdorf Bridge over the Arve, Carouge, AB ing. and Brodbeck Roulet, 2012

The second trend identified by Kaufmann and Meier [14] is the appearance of low-cost structures that neglect the aspects of image and context. Prefabricated beam bridges supported by hammerhead piers are a tragic example of this type of structure.

However, there is one more avenue to pursue when it comes to infrastructure design, which this article aims to reveal.
3 Contextual and Conceptual Considerations

3.1 Contextual Considerations

The contextual considerations that go into a project, especially an architectural project, are well summarized by Vincent Mangeat [19] in the following quotes:

- Constructing an area/constructing a town/constructing infrastructure/constructing a house are one and the same operation.

- The area surrounding a structure must set the tone for the project. It is important to know how to read a structure’s surroundings, as this is how the theme of the project becomes localized, becomes contextualized once the surrounding area itself is taken into account. If the surroundings change, the project changes, even if the theme remains the same.

- The construction of a house also incorporates its surrounding area in the sense that to build means to work with the land, and in that the project is rooted in its location. A project should improve the location in which it finds itself and reenvision its surroundings.

- The construction of a work of infrastructure or an edifice of any kind should be considered tantamount to the construction of the entire surrounding area; you cannot simply “erect” a particular object in a location without taking account of the aspect of shaping that location, since it is integral to the structure itself.

The area surrounding a structure comprises a strong cultural dimension, and forms part of the lived experience of the inhabitants of a particular place, thus it must be considered and incorporated into a project as part of a contextual approach.

3.2 Conceptual Considerations

It is worth recalling the notion of emotion highlighted by Le Corbusier [7]: “Architecture is an act of art, an emotional phenomenon that defies and transcends questions of construction. Construction is about getting from A to B; architecture is about emotions.”

Mangeat [19] notes that every project has a meaning: “In the same way that words mean something, architecture represents something: as with all things, but especially in architecture, there is [...] that which is meant and that which carries meaning. That which is meant is the object of our speech; that which provides it with meaning is exposed and developed by means of scientific processes”.

These notions of emotion and meaning must therefore also be incorporated into any project.

A building must be the fruit of an idea, as recalled by Jacques Lucan [18] in reference to Valerio Olgiati: “A building must be the fruit of an idea, whereby that idea defines its intrinsic set of rules, renders it an organic whole, and endows it with a sense of unity; an architecture that is born of itself”.

It is the concept that comprises the idea behind a project: “‘concept’ is an abstract and general idea that constitutes the principle behind the project, the position of its creator with respect to the theme; the concept also includes the principle according to which the project takes form or inhabits a space [...]” [19].

The objective of a concept is therefore to put the idea behind a project into words in order to flesh out the different notions, emotions, and meanings. A conceptual approach allows these notions to be incorporated into a more holistic vision.

3.3 Contextual and Conceptual Approach

Contextual and conceptual approaches can be combined within a project, as demonstrated by Olivier Boissière [3] in his summary of Jean Nouvel’s technique: “At the dawn of the 1980s, Nouvel laid the foundations for an approach underpinned by two beliefs: firstly, that architecture can no longer regard itself as an autonomous and self-contained discipline that is detached from the world around it; and secondly that a project is by nature specific, due to matters of location, climate, topography, destination, economy, and culture. ‘If I am asked the same question twice, I will produce the same project twice’ he is quoted as saying. He adds: ‘Ultimately, a concept is composed of a select set of ideas and defining it involves delving into the finest of details.’”

In this sense, contextual and conceptual considerations form part of any project, and more specifically of infrastructure projects as shown in the following section.
4 Projects Based on Contextual and Conceptual Considerations

4.1 Transjurane Highway Infrastructure

The perception of highway infrastructure differs depending on the location, the individual, and its use. When planning such infrastructure, attention must be paid to all aspects in order to create a coherent structure. It is likely for this reason that conceptual considerations have now been initiated with respect to this type of project.

One of the first approaches is that taken by Rino Tami in designing the section of the A2 highway between Chiasso and Airolo. His work is described by Flora Ruchat [26] as follows: “The retaining walls, bridgeheads, and rest stops form spatial sequences that are perceived both by users of the motorway and by external observers as a series of structural elements, resulting from a particular function.” She goes on to describe the manner in which this occurs: “As such, the detail, born of a clearly defined idea, becomes the standard, the vocabulary, the model, and incorporates the singularity of different episodes into one uniform whole.” Hence, creating a concept leads to the development of a language that imbues the highway with its own image and identity. That concept involves defining a formal and structural guiding principle.

It was from Tami’s work that Flora Ruchat and Renato Salvi drew inspiration when designing a section of the Transjurane highway. As described by C. Dionne [8], “Flora Ruchat-Roncati and Renato Salvi drew their inspiration from the work of the Ticinese architect Rino Tami [...], borrowing certain principles pertaining to the incorporation of the landscape such as the application of a regular geometry based on recurrent use of 30° angles.”

As referred to in [31]: “the portals of the Transjurane highway thus evoke a sense of movement,” and “the main method used by Salvi to give the motorway the appearance of a dynamic whole [...] is the off-vertical form of these portals, which lean forwards, backwards, or sideways and change gradient more or less quickly and more or less severely.” Moreover: “The idea was to combine the elements – the portal and the control centre – so as to obtain a simple and economical solution whilst minimizing the impact on the landscape.”

These conceptual approaches, which were developed to invoke a feeling of movement through form using unitary, sloping structures, are clearly illustrated by the portal of the Mont Terri Nord tunnel shown in Fig. 3.

Fig. 3 Portal and ventilation system, Mont Terri Nord, Renato Salvi, 1998

As stated by Renato Salvi [31], “The Transjurane highway was an exceptional and singular experience, since it did not necessarily involve spatial planning, but rather the art of placing objects in a landscape, thereby imbuing these objects with a clear sculptural identity.”

Thus, the conceptual choice to invoke a feeling of movement using sloping structures lends a sculptural and dynamic dimension to a range of projects, resulting in designs that form a cohesive unit across an entire motorway section.

4.2 Pünt da Suransuns

The Suransuns footbridge is located on the Viamalia path in the Grison Alps. It was designed by Jürg Conzett, and the project along with its accompanying conceptual considerations is described in [5]. Two conceptual approaches were pursued. The first was to link the two riverbanks using a stressed ribbon bridge that can adapt easily to the change in ground level. The second was to use local stone for the structure, namely Gneiss stone from Andeer.
The light, transparent stressed ribbon structure with a heavy solid stone deck creates a sense of contrast that pervades the footbridge as a whole, as shown in Fig. 4. This contrast is further highlighted by the particularly transparent guard rails, which are made of stainless-steel tubes inserted directly into the stone.

Fig. 4 Pùnt da Suransuns, Jürg Conzett, 1999

It is interesting to note that Jürg Conzett has worked with Peter Zumthor, who once said [18]: “Materials and constructions must have a connection to their location and must sometimes be derived directly from it. Otherwise, it seems to me that the landscape will not accept the new structure.” Such a connection is certainly apparent in the case of this footbridge, which uses stone originating from the local area. In becoming specific to its location in this way, the project is an example of a contextual structure.

4.3 Hagneck Hydroelectric Power Plant

The hydroelectric dam in Hagneck, where the Hagneck canal feeds into Lake Biel, was built in 1900 and had to be replaced in order to afford better protection against flooding.

The task of designing the new hydroelectric plant was the subject of a design competition that was won by the engineer Martin Valier and the architect Christian Penzel, associates at the same firm. As described in [26]: “It blends harmoniously into the landscape, laying its story out for all to see along an entire stretch of path which, with an almost film-like sense of theatre, constantly offers new perspectives.”

The concept is intended to mimic a theatrical experience; an experience of a journey, all at high speed. Thus, as stated by Seitz in [30], “The plant is set at a lower level so that the bridge does not pass over the dam itself, but across its flank, thereby intensifying the experience of the structure as the user finds themselves only just above the surface of the water. In addition, the openings into the interior of the plant make it possible to see the turbines, further enhancing the user’s perception of the structure.”

One contextual decision was to integrate the concrete structure of the dam into its environment by using aggregate taken from the surrounding land in the concrete to ensure its colour matches the landscape as shown in Fig. 5. In terms of morphology, the structures are rounded in shape, a concept that is intended to be reminiscent of and symbolize hydraulic constructions.

Fig. 5 Hagneck Hydroelectric Power Plant, Penzel Valier, 2015
The contextual and conceptual considerations thus resulted in a project that embraces the surrounding landscape as a whole, as part of a global, unifying, and scenography.

4.4 Viaduct over the A9
The viaduct over the A9 at the junction with the H144 main road passes over the highway and the cantonal road, following a partially curved trajectory and having a hump-shaped profile.

The concept developed in connection with the competition was to create a unitary viaduct with a steady and simple curvature. A concept that is all about unity and continuity, as summarized by Menétrey et al. [20] and illustrated in Fig. 6.

Thus, the deck remains constant across the whole length of the structure, the edges of the bridge extend over the abutments, and the piles are similar but for a slight variation in incline to compensate for the difference in height. In an extreme show of dedication to the quest for unity and attention to detail, the conduits were integrated between the ribs of the deck so that they are not visible.

Fig. 6 Viaduct over the A9, INGPHI and B+W Architects, 2012

The rationale behind this was to consider the deck section as a single unit, as described by Lucan [20]: “One of the things the design process was focused on was developing each specific section taken as a whole. As such, it was a question of refraining from fixating on relationships and links, and instead avoiding any fragmentation without losing the sense of monolithism.” Consequently, the deck section was designed in the shape of an airplane wing.

Further developments followed in the wake of this project, and Menétrey developed a method [21] that involves creating designs that incorporate all requirements into a single structure. The aim is thus not to limit oneself to creating a set of distinct structural elements, but rather to incorporate all of the requirements set out in the program, the constraints of the site, aesthetic considerations, and sustainable development criteria into a single, unitary structure.

4.5 Strengthening of the Bridges over the Paudèze
The bridges over the Paudèze are prestressed concrete structures built in the 1970s using the free cantilever method. After more than 40 years of service, the structures were in poor condition and required reinforcement, corrections to their geometry, and strengthening as described by Menétrey and al. [22]. The size of the structures, the deterioration in their condition, and certain limiting factors meant that contextual and conceptual aspects had to be considered.

The resulting concept involved reinforcing the bridges without modifying their image as free cantilever structures yet modernizing them at the same time. Inclined struts were inserted between the road deck and the box-girders to reinforce the structures, with all other reinforcements placed inside the box so as not to be visible.

The struts are designed to look like a piece of netting or lace, as shown in Fig. 7, and are made of UHPFC in order to maintain consistency with the concrete of the existing structure. They were precast, and only the connection points with the girders and road deck were cast in situ to form rounded, fluid joints.
The struts serve as punctuating structures that allow an orderly organization of the space, as introduced by Max Bill [1].

Finally, during the strengthening work, several damaged sections of concrete were reprofiled using mortar. Despite several attempts in situ, it was not possible to reproduce the colour of the existing concrete. As a result, the reprofiled sections were left unmodified, thus fulfilling the brief of modernizing the structure without changing its image. This reprofiling serves as a reminder that structures slowly deteriorate over time and symbolizes their limited lifespan, highlighting the impermanence of such constructions.

5 Summary of the Benefits of Contextual and Conceptual Considerations

The benefits of contextual and conceptual considerations have been discussed in relation to different yet complementary projects, and the following serves to summarize this discussion.

It appears that contextual and conceptual considerations allow for a more holistic approach to projects and represent more than just a technical solution. The Transjurane project demonstrates that a concept can give rise to a language, in this case a language that imbues the highway with its own image and identity along the entire length of the section in question. Similarly, the scenography and material considerations that went into the Hagneck power plant project have resulted in a structure that embraces all of its surrounding environment as part of a more holistic vision. In general, such contextual and conceptual considerations lend a sense of cohesion to a project.

The projects discussed also help to illustrate that contextual considerations in the broadest sense – namely incorporation of the local environment, the landscape, heritage, and users – allow these elements to feed into the project. This is clearly demonstrated by the Suransuns Bridge project and by the Hagneck power plant, both of which drew their materials from the local area in order to forge a link between the structure and its context. Moreover, in the case of the Hagneck power plant, scenography considerations provide different viewpoints that allow users to feel like part of the structure. As regards the bridges over the Paudèze, it was a desire to preserve the heritage value of the structures that resulted in the reinforcements taking the form of concrete struts. These considerations thus serve to contextualize a project, making it more specific and better tailored to its surroundings.

By incorporating these considerations into a concept, a project can acquire meaning and the concept itself can serve as a guiding principle running through the entire project. Equally, in the case of the Transjurane project, the desire to incite movement as symbolized by the sloping structures creates an image that is both dynamic and sculptural in nature. Finally, as regards the Hagneck power plant project, it is the scenography that allow users to comprehend the structure by opening up views of the water accumulated in the plant, views that both symbolize and justify the project.

Infrastructure should no longer simply be planned to act as a transit hub or production site but should be designed on the basis of contextual and conceptual considerations, whereby the project as a whole is approached as an architectural endeavour.
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Principles, structures, and processes: engineering & architecture collaboration in footbridge design

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Abstract
The paper presents the creative process and collaboration between Bernabeu Ingenieros and Burgos & Garrido Arquitectos in the design of several footbridge projects: Lent-Tabor footbridge in Maribor, Slovenia; three stress-ribbon footbridges over the Tajo river in Toledo, Spain; Goián – Vila Nova de Cerveira footbridge over the Miño River, connecting Spain and Portugal. Together, they present a coherence in principles and strategies: adequacy between form and type; consistency and coherence among the different scales of the structure; relation and integration of the footbridge within the existing pedestrian and cyclist network, and to its connection with the riverbanks. Very different structural types and solutions are considered in each project, according to its requirements, in a growing challenge in terms of difficulty and maximum span that stimulates structural innovation.

1 Introduction
Footbridges are a very particular kind of projects. They have the large scale of civil engineering infrastructures, which usually implies major structural requirements, but also a minor scale related to the pedestrian user, which makes the attention paid to the connection and constructive details and to secondary elements as handrails especially relevant. Besides, the presence of the footbridge in the landscape, and its integration within the existing pedestrian and cyclist network and its connection with the riverbanks, add a very significant environmental and urban condition to the project.

This multiplicity of factors, scales and conditions requires the combination in its design of different skills and sensibilities and makes specially interesting and fruitful the collaboration between structural engineers and architects.

Bernabeu Ingenieros and Burgos & Garrido arquitectos have collaborated in the design of several footbridge projects, developing together a common understanding and strategy. The paper presents the creative process and collaborative design of three projects:

- The Lent-Tabor footbridge in Maribor, Slovenia (Fig. 1). Formed by two longitudinal steel girders on either side of the deck, the footbridge crosses the Drava river in three approximately equal spans that mirror the spans of the existing old bridge located nearby, establishing a dialogue with it and enhancing its symbolic value.
- Three stress-ribbon footbridges over the Tajo river in Toledo, Spain (Fig. 2). The consideration of stress-ribbon structures allowed to save spans from 106.2 to 132.5 meters with a very restricted height of the deck and minimum intrusion in the environment, and to maintain the same structural dimensions and sizing for the three footbridges, by simply adapting the catenary layout in each case.
- The Goián – Vila Nova de Cerveira footbridge over the Miño River connecting Spain and Portugal (Fig. 3). With minimal presence in the landscape, it considers a hybrid typology, a self-tensioned spatial structure that combines a suspended bridge in elevation with the arch effect of the deck in plan.

The paper presents the main characteristics of these three projects and extracts its similarities and differences. Considering the specific requirement of each project very different structural types and solutions were adopted, although they all have in common similar design principles and processes.
Fig. 1  Lent-Drava footbridge over the Drava River, Maribor, Slovenia.

Fig. 2  Stress-ribbon footbridges over the Tajo River, Toledo, Spain.

Fig. 3  Goián – Vila Nova de Cerveira footbridge over the Miño River, Spain, Portugal.
2 Lent-Tabor footbridge over the Drava River. Maribor, Slovenia

The Lent-Tabor footbridge over the Drava River in Maribor, Slovenia, results of an international competition held in 2011. It is located near the city centre and crosses the Drava river just beside the Old Bridge of the city, the Stari Most, a three-span steel arch bridge from the beginning of the 20th Century.

The proximity to the existing Old Bridge determines the configuration of the new footbridge, that is located at a lower level, and seek to establish a dialogue with the historic bridge and to enhance its symbolic value. Indeed, the dialogue between the two structures, existing and new, is very intense. It is established both in the upper view of the footbridge from the existing bridge, considered in the leaf drawings of the footbridge pavement and the riverbanks, that refers to the oldest vine in Europa that grows near the footbridge, and in the lower view of the Stari Most steel structure from the footbridge. This relationship between the two structures also affects the structural design of the footbridge, which replicates the positions of the intermediate piers in the river of the Old Bridge, crossing the river in three approximately equal spans of around 50 meters; span distribution: 48-42-42m (Fig. 4, Fig. 5).

The structure is formed by two longitudinal steel girders located on either side of the deck. While the internal longitudinal steel plate of the web is continuous, the external vertical stiffeners adjust their geometry to configure an attractive curved cross-section in which the whole structure, covered in wood, integrates the deck and lateral railings in a single piece [1, 2]. A set of transversal steel girders, with hollows to allow the passage of installations, support the deck. They are rigidly connected to the longitudinal beams, stabilizing them against lateral buckling, and building a Vierendeel beam that guarantees the transmission of horizontal forces to the abutments and intermediate piers without the need for additional bracing elements.

Fig. 4 Plan. Lent-Tabor footbridge over the Drava River, Maribor

Fig. 5 Elevation. Lent-Tabor footbridge over the Drava River, Maribor

Longitudinally, the constant depth of the deck appears to be like a continuous slim band. The depth has been adjusted to the minimum, enhancing the slenderness and elegance of the footbridge. This configuration, however, implies low vibration frequencies, close to the frequency of pedestrian passage, which can lead to a synchronisation of both, that may cause resonance and an excessive level of movements. The proposal includes therefore the disposal of Tuned Mass Dampers (TMD) to improve the dynamic response, reducing the vertical accelerations to meet the required comfort criteria [3].

The deck rests directly on the abutments located on both banks, and on two intermediate piers in the river. The materialisation of these intermediate piers required a delicate and transparent solution that would emerge naturally from the river and respect the continuity of the deck, while guaranteeing...
the adequate transmission of vertical and horizontal loads. At this purpose, a set of slender inclined steel tubes was considered for each pier. It offers the image of a random distribution of junco plants that emerge from the river and support the deck. However, this apparently random arrangement of the steel tubes derives from a structural logic: for each pier there are four slightly inclined steel tubes located on both sides of the deck, which directly transfer the vertical loads of the main girders, while three additional tubes, with a greater inclination, are arranged in the central area of the section, guaranteeing the horizontal stability of the footbridge (Fig. 6).

The result is a simple and delicate footbridge that crosses naturally the river as a continuous wood pathway, that preserves the view of the existing Old Bridge and enhance its value.

Fig. 6 Cross section. Lent-Tabor footbridge over the Drava River, Maribor

3    Stress-ribbon footbridges over the Tajo river. Toledo, Spain

The three footbridges over the Tajo river in Toledo, Spain, takes part of the project of integration of the Tajo river in Toledo, that covers a large extension following the riverbanks into the city. The purpose of the footbridges is to connect the two riverbanks, one of them accessible from the city centre of Toledo and the other much more rustic and abrupt, with an important difference of levels between the two.

A very important restraint of the footbridges was to respect the 500 years flood level, that is located very high from the actual flood level, and to avoid intermediate piles in the river, which forces to spans longer than 100 meters. Considering these requirements, as well as the design intention to maintain the structure as close as possible to the flood level and to avoid big or grandiose structures that would acquire excessive presence in the landscape, led to consider stress ribbon structures. Stress ribbon structures adopt a catenary shape, which means gravitational load transfer is done by axial forces, allowing to adopt a minimum thickness of the deck.

The main starting design issue of the footbridges was to choose an adequate location and outline, both in plan and in elevation. The location had to respect the 500 years flood level, to suit with the required levels at both banks, and to reduce the footbridge length as much as possible.

The three footbridges cross the whole length in a single span of 132.5, 113 and 106.2 meters, with a span to sag ratio of nearly 65 [4] (Fig. 8). As a reference, the longest span stress-ribbon built structure is the Yumetsuri bridge, built in Japan in 1996, that covers a maximum span of 148 meters, with a span to sag ratio of 42.

Stress ribbon footbridges are formed by a couple of catenary cables, and a very thin concrete deck, formed by 27cm thick precast elements, that are afterwards post-tensioned, to control the appearance of stress forces and to assure structural continuity, improving dynamic performance of the structure (Fig. 9). Only near the supports important bending moments appears, and a bigger deck thickness is required, in a length of approximately 6.0 to 7.5 meters in each side. Those areas are built with in situ concrete, with a varying thickness up to 90cm.
The span to sag ratio of the catenary layout was adjusted in each one of the three footbridges, depending on its length, to make the axial forces similar, with the purpose of adopting identical solutions for the prefabricated elements of the deck and for the abutments, that will have similar horizontal forces, regardless of its span.

Since the transfer of loads is done through axial forces, and the catenary slope is very soft, the resulting horizontal forces at both supports are extremely important and require particular foundation systems to be transferred to the ground. This was solved differently on both sides, since the massive
granite foundation level was also differently located. On one margin, left riverbank, the granite level was very shallow, and the horizontal forces could be directly transferred to the ground by a system of anchored cables, that were post-tensioned, in order to control lateral displacement of the abutment. On the other margin, however, the granite level was much deeper, and a system of inclined micropiles had to be set, transferring both vertical and especially horizontal loads. This system entails a certain lateral displacement of the abutment, that must be considered in the analysis, since it modifies the catenary geometry.

The three stress-ribbon structures adapt naturally to each location and crosses the Tajo river with minimum presence in the environment, preserving and enhancing the natural landscape.

4 Goián – Vila Nova de Cerveira footbridge over the Miño river. Spain – Portugal

The Goián - Vila Nova de Cerveira footbridge over the Miño River connects Spain and Portugal. With minimal presence in the landscape, it proposes a hybrid typology, a self- tensioned spatial structure that combines a suspended bridge in elevation with the arch effect of the deck in plan. The concept design approach is based on classic structural types, in particular light cable supported bridges [5, 6, 7], but with an innovative variation on the standard that explores the possibilities of the curved plan [8].

The footbridge spans 265m, and is a spatial suspended structure, with two pylons located on the riverbanks, avoiding intermediate supports on the riverbed, and only one suspension cable. The towers are located eccentric with the axis of the footbridge deck, that adopts a curved layout both in plan and in elevation. The curved layout in plan fits better to the footbridge arrival in both riverbanks and improves its structural behaviour. Indeed, the eccentric location of the suspension cable within the deck generates important horizontal transverse forces, that are supported by the curved deck by behaving as an arch. This configuration is also very convenient for supporting and controlling wind loads. It is not a classic suspended bridge because of its singular configuration due to the curved layout of the deck and its arch-like behaviour (Fig. 10, Fig. 11).

The complexity of the considered geometry and its work by form with a spatial decomposition of forces for both the suspension cable and the footbridge, related one to each other, required the use of form-finding algorithms to develop its form. The Combinatorial Equilibrium Modelling (CEM) algorithm, developed by Patrick Ole Ohlbrock and Pierluigi D’Acunto at the Chair of Structural Design at the ETH Zurich was applied [9, 10 and 11]. The CEM algorithm is a form-finding tool based in 3D graphic static and the graph theory, that enables interactive generation of spatial mixed tension-compression equilibrium forms. A form-finding process was developed together with PO Ohlbrock and P D’Acunto to define the geometrical configuration of the funicular structure, considering different static equilibrium configurations to control and to optimize the structural behaviour of the footbridge. This process considered possible variations of the main parameters that define the geometrical and structural configuration of the system and in particular the curvature of the deck in plan, the height of the pylons, and the curvature of the main cables.

A key design element in this configuration and optimization process is the tensile force of the main suspension cable. However, to increase the height of the pylons has also important implications in relation to its presence in the riverbanks and in the sizing and structural configuration of the pylons, that are very conditioned by buckling, that increases with height. After several tests, a total height of the pylons of 55 meters was adopted, which allows reducing tension force in the main cable so it can be formed by two standard steel wire Full Locked Coils Strand (FLC) Cables, while controlling the size and the presence of the pylons in the riverbanks.

The proposed construction process maintains the concept of hybrid spatial arch-suspension structure approaching in its successive stages the final structural and geometric configuration. Suspension bridges start the construction by stretching the suspension cable. In this case, the suspension cable supports the vertical forces while the horizontal forces must be stabilized with temporary cables anchored at the riverbanks. A combination of the construction processes of suspension bridges in elevation and cantilevered arches construction in plan.

The result is an innovative structure, a subtle and slender footbridge that both offers a new path of research in structural concept design and preserves the environmental values of the river and the landscape.
Fig. 10  Elevation and plan. Goian-Cervera footbridge over the Miño River, Spain, Portugal

Fig. 11  Cross section. Goian-Cervera footbridge over the Miño River, Spain, Portugal
5 Design, principles and processes.

From the analysis of the three projects several similarities and differences may be extracted, in order to highlight the main design principles and processes (Figs. 1, 2 and 3).

First, the three footbridges presented have in common that they all cross relevant rivers, with important symbolic, historical, and cultural significance: the Drava, the Tajo and the Miño rivers. They are all located in areas with a certain urban character, but mainly in natural environments of special relevance and beauty. The landscape is therefore an essential element in the character of the three projects.

However, the location of each project has its own qualities and particularities. In the case of Maribor, probably the most urban and central of the three footbridges, the presence of the Old Bridge, the Stari Most, is a determining factor. The footbridge is inevitably associated and conceived in its relationship with the historic bridge, enhancing its presence and value, and establishing an intense dialogue with it. Thus, the new footbridge locates very close to the existing bridge, but at a lower level, and considers two intermediate supports in the river, just as the existing bridge does, and in the same position, mirroring its three approximately equal spans.

Neither the Toledo nor the Goián-Cerveira footbridges have existing structures located nearby. There are no visual obstacles close, and the depth and the nature of the riverbed, as well as the riverbanks and the surrounding landscape, suggest crossing the river without intermediate supports. In Toledo, the Tajo River separates the city centre area from a more rural and rugged side. The footbridge connects both areas, allowing from the left riverbank, more rustic, to access directly to the city centre, and from the right riverbank, on the city side, to have access to a more rural and wilder pathway along the river. In the case of the Goián-Cerveira footbridge, it connects two cities, one Spanish and one Portuguese, with the aim of creating a common park, the largest cross-border park in Europe. The Portuguese side is more urbanised and integrated into the city, while in the Spanish side the presence of an historical building, the San Lorenzo fortress, centres the attention, and there is a strong interest in its further development and enhancement. The footbridge will promote the development of both sides, complementing each other in character and services, and will enhance the relation between the countries.

Considering the above, although with different contexts and conditions, the three project locations suggest considering non-intrusive structures, respectful with the landscape and the existing surroundings. Footbridges that do not seek to be the centre of attention, but to put in value the existing environment and locations.

Nevertheless, the very different river width of the three rivers, and the disposal of intermediate supports in the case of the Maribor footbridge, results in very different span lengths: 48m in Maribor, 106.2 to 132.5m in Toledo, and 265m in Goián-Cerveira. As a result, each project considers a very different structural type, in a growing challenge both in terms of structural requirements and innovation: double steel girders in Maribor, stress-ribbon structures in Toledo, and a hybrid suspended self-tensioned structure in Goián-Cerveira.

Physical and conceptual models were made during the development of the three projects to visualise the problem, to study the relation with the site and to help understanding its structural configuration (Figs. 12, 13 and 14). In the case of the Goián-Cerveira footbridge a Mola Structural Model was used [12]. Mola is an interactive physical model that simulates the behaviour of structures. The model consists of a set of modular pieces connected through magnetism, allowing the visualization of movements and deformations of its elements (Figs. 14). Physical models contribute and improve the understanding of structures, actively assisting the design process.

Despite the very different structural solutions the three footbridges manifest a common strategy and design principles.

First, adequacy between form and type. Each structural type has its own characteristics, which implies very diverse structural conditions and forces on the different elements that constitute the footbridge (deck, piers, pylons, abutments…). According to this, the design strategy is to respond and to adapt to these structural conditions, that suggest in each case different considerations in terms of geometry, configuration, and materiality, searching the adequacy of the structure between form and type. There is not an “a priori” design shape nor form, but it naturally emerges from the considered structural type.
Besides, the treatment and configuration of the different elements is considered being also conscious of the minor sale of the project that the pedestrian user implies. There is a seek of consistency and coherence among the different scales of the project, from the general layout to the transverse section and the connection and constructive details, that requires special care and attention.

Fig. 12 The conceptual model of the Lent-Tabor footbridge shows the continuity of the deck over the light and transparent piles in the Drava River.

Fig. 13 Stress-ribbon footbridges model over the Tajo River. The model explains the slenderness of the structure and its relation to the environment.

Fig. 14 The Mola Structural Model of the Goian-Cervera footbridge allows to understand the spatial balanced geometry and to test different configurations.
Finally, special importance is given to the landings and accesses, to the relation and integration of the footbridge within the existing pedestrian and cyclist network, and to its connection with the riverbanks. As has being previously exposed, footbridges serve to connect and to communicate different areas, cities or even countries, and they seek to put into value the existing environment and locations. For this purpose, the integration of the new structures within the existing communication networks and its relation to the urban or natural environment are key factors, specially considered in the design process in the accurate choice of the location and layout of the footbridge, and in the treatment of its landing and accesses.

As a conclusion, the collaboration between structural engineers and architects has proven to be very fruitful and satisfactory in these design processes, sharing principles and strategies and combining skills and sensibilities.

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References

Balance between design and construction

Jan L. Vítek

Abstract
The design of structures is influenced by many factors; the construction process belongs to the most important one. Especially in prestressed concrete structures, the construction process can influence the costs and the speed of construction. Recently a new bridge was built in Prague, where architectural requirements were governing for the design. The costs were very high. The paper shows some possibilities how the design could have been influenced, which would have resulted in the reduction of costs with a minimum impact to the appearance of the bridge.

1 Introduction
A structural design and a construction process of the structure are two phenomena, which should be carefully combined. This principle is applicable in all structures; in the following the bridge design and the construction will be discussed. Bridges are primarily built as structures which should provide a transport from one side of the valley to the other side. During the history the bridge structures developed. Initially, the crossing of larger span was a pure technical problem, and the finding of an appropriate structural system was a major challenge. Bridges were designed by specialists in building art and the load carrying function was the prevailing criterion for the bridge design. Structural systems were limited by materials and construction processes available at that time. For longer spans, usually two structural systems were feasible almost until the middle of the 20th century: either arches or suspension bridges. Architectural supplementary elements were added to bridges mainly in cities in a form of various decorations or sculptures. Excellent bridges were built using these principles [1], [2], [3].

During the 20th century, the bridge technology significantly developed. Stone bridges were replaced by concrete bridges and steel bridges became a subject of extensive development achieving the maximum span almost of 2 km [4]. Concrete developed in the first half of the 20th century when large concrete arches were built at roads and railways. At the second half of the 20th century, the development of prestressed concrete made it possible to extend the span of bridges with a concrete deck up to 300 m in frame structural systems [5] and more than 500 m in cable stayed systems [6]. Building of larger spans became usual and major structural issues of the bridge design were solved. At the same time, construction technologies were developed for individual structural systems. The application of prestressed concrete made it possible to develop methods of the sequential construction of large bridges which reduced the costs and accelerated construction processes.

During the time, there were attempts to design structures which were not determined only by a mechanical efficiency, leading to the clear and transparent structural performance. The systems which appeared interesting from the architectural point of view, but not entirely efficient in terms of the transfer of forces, were sometimes built. The costs were usually significantly higher than those of structures with statically more efficient systems. It is questionable if the extravagant appearance of these structures is worth for the increased costs. The Alamillo bridge in Sevilla [7] may serve as an example (Fig.1). Now these trends may be observed more often. The second phenomenon, which can be seen sometimes in the bridge construction, is an inadequate relation of costs spent for the structure itself and costs spent for the construction process. There are structures, which are easy to build and the construction costs may be minimised. On the other hand, there are sophisticated structural systems which may be rather efficient in terms of the consumption of the material, but the costs for their construction are enormous. The efficiency of such structures should be evaluated as a one unit, including costs of the structure and construction process, and even including the costs during the service life. The other issue which should be considered is a long service life of bridges. They are designed for the service life of 100 years or sometimes even more. The actual design codes are used for design and for loadings. Experience from last periods clearly showed that the load was increasing during the time. Bridges which were designed...
in a more conservative way, could have been strengthened, and used for some years more, while those bridges which were designed just on the limit of the code requirements had to be replaced or their rehabilitation was extremely costly. The conceptual approach to the design is therefore very important and it should consider all the mentioned phenomena and to find a reasonable balance, so that the efficient structural system would be designed having a perspective to serve to the client for many years without extraordinary expenditures.

2 Troja bridge in Prague

The Troja bridge in Prague is an example of a complex bridge structure which can be used for illustration of alternative solutions. It is upon the reader to assess if the mentioned alternatives (see the section 3) would be worth to be applied or not. The bridge was built according to the winning project in the design competition. R. Koucký and L. Kábrt (architects) and J. Petrák and L. Šašek (civil engineers) are the authors of the design. The author of the paper took part in the construction as an expert of the contractor. The bridge was built recently and now it serves to the traffic without any difficulties. However, the initial costs (~ 55 mil. EUR) were higher than the usual costs of similar bridges in Czechia, the operational costs will be evaluated during the time.

2.1 Description of the bridge

The Troja bridge crosses the Vltava River in Prague (Fig. 2). It transfers a two-lane tram track, four lanes of road traffic, pedestrians, and cyclists. A more detailed information on the bridge can be found in [9], [10] and [11]. The bridge has two separate spans supported by bearings on the pier and abutments. The main span is 204.4 m long, and the side span is 40.4 m long (Fig. 3). The lanes for pedestrians and cyclists are located on steel cantilevers fixed to the edge beams of the concrete deck in the main span as well as in the side span. The total width of the bridge is 36 m. The main span has a structural system called a network arch. The efficiency of the structural system and low consumption of materials in the structure itself were major arguments for choosing a network arch.
The concrete bridge deck is carried by the steel arch using a network composed of inclined hangers made of prestressed steel bars. In total 200 hangers are used and they are located in four planes. The rise of the steel arch is only 20 m; the rise/span ratio is 1/10; the arch is very flat. A single box cross-section of the arch in the central part of the span splits into two sections towards to supports, where the arch is connected to the tie. The tie of the arch is located above the bridge deck. The tie is composed of two longitudinal beams with a steel-concrete composite section. Each beam is prestressed by 6 cables composed of 37 strands 15.7 mm in diameter. The deck is formed by a grid and a thin slab. The grid is made of the prestressed steel-concrete composite tie of the arch and precast prestressed transversal beams, which are suspended on the tie. The transversal beams carry the concrete slab. The bridge deck is prestressed longitudinally and transversally. The cross-section of the main span is plotted in Fig. 4. The prestressed tie is the only stiffening element of the bridge deck in the longitudinal direction. The position of the tie and its small dimensions were designed on requirements of the architect. The purpose of this arrangement is elimination of any longitudinal element under the bridge deck. The curved shape of transversal beams should remind the waves in the river and a longitudinal beam would appear as a disturbing element from the aesthetical point of view.

The side span is a classical cast in situ structure. Two longitudinal beams are the main load carrying elements, which support transversal beams and a slab. Transversal beams are cast in situ and their shape is identical to that of transversal beams at the main span (Fig. 7).

2.2 Construction of the bridge
Network arch bridges are usually built on the fixed platform directly in the final position or in a different position and then they are transported to the final position. They are often made completely of steel, so that their weight was minimised. A typical span of network arch bridges varies in the range from 60 to 120 m, which allows for the efficient assembly. The span of the Troja bridge is rather large and the bridge deck is in spite of small dimensions of the cross-section rather heavy. If the conditions allowed, it would be convenient to assemble the arch and the deck on the river bank and float it into the final
position. Such construction was not possible due to the local conditions on the banks and in the river. It was necessary to build the bridge in the final position. After evaluation of many alternatives, it was decided to build five temporary supports in the river and to build the bridge deck first. Then the assembly of the arch followed, and finally the hangers were installed. When the structure was completed, the temporary supports were released. Finally, the cantilevers for pedestrian strips were assembled and the construction of the bridge could be finalized.

2.2.1 Construction of the bridge deck
The incremental launching technology was used for the assembly of the bridge deck. In order to reduce the weight of the launched structure, only a basic grid of the deck was launched. It was composed of the steel part of the composite arch tie and precast transversal beams. The slab and the concrete part of the arch tie were cast after launching of the basic grid. However, the longitudinal stiffness of the longitudinal elements (steel parts of the tie) was very low. The load carrying truss was produced, which was composed of a definitive steel part of the arch tie, forming a top chord, and temporary diagonals and a temporary bottom chord (Fig. 5). This truss had a sufficient stiffness in longitudinal direction and made the launching of the basic grid possible. The temporary elements (diagonals and a bottom chord) were rather expensive, but significantly cheaper than a fixed scaffolding, which could represent another possibility of the construction of the bridge deck. After finishing of launching, the slab was cast and a platform for assembly of the arch was prepared. A temporary truss stiffened the bridge deck which enabled a transport of heavy parts of the arch to the bridge deck. The temporary truss represented a stiff longitudinal element which is a missing part of the bridge deck.

2.2.2 Construction of the arch
The individual parts of the steel arch of the weight 60 – 80 t were delivered to the site and welded into larger elements. These elements were subsequently transported to the bridge deck and welded into three large pieces (each of about 700 t), which were lifted using temporary towers and welded together (Fig. 6). After dismantling of the temporary towers, the hangers were installed. The hangers were made of Mac Alloy bars of the diameter 76-105 mm. The hangers were initially stressed only up to about 10% of their tensile strength. During their installation, deformations of the arch and deck were small and also the deflections of the hangers remained small; the linear performance of hangers could be assumed. A sequence of assembly of hangers was chosen so that the additional adjustment of the stress in hangers was not necessary. Then the temporary truss was deactivated by interruption of the lower chord at several places. Finally, the temporary supports in the river were released which resulted in the final prestressing of hangers. The deflection of the deck at the midspan was about 150 mm and the measured values corresponded to the predicted deflections very well.

2.2.3 Construction of the side span
The side span is a classical cast in situ structure, which is located outside of the river. It was cast on the fixed scaffolding. The total length of the side span was divided into three parts which were subsequently cast and then the entire structure was prestressed longitudinally and transversally. The side span is illustrated in Fig. 7.
3 Possible modifications of the design

The design of the Troja bridge was strongly influenced by requirements of the architect. Some of them resulted in too sophisticated details, in more expensive structure, in more complex construction or in more demanding maintenance. One can doubt whether it was worth to follow these requirements completely, whether their contribution for the appearance of the bridge was really so significant. In the following, slight modifications of the design are discussed, which would improve the design and at the same time their effects on the appearance of the bridge were minimised.

3.1 Longitudinal beams

One of the major issues was the absence of stiff longitudinal beams in the main span. It was purely architectural requirement for a nice appearance of the bridge from the bottom. Fig. 8 shows the view on the soffit of the main span of the bridge. A requirement on the nice view on the soffit resulted in significant difficulties. First, the connection of transversal beams and the tie of the arch had to be solved. The load from the bridge deck is transferred to precast transversal beams and using their suspension on the arch tie, it is transferred to hangers and to the arch. The transversal beams have a mutual distance of 4 m, i.e., the total load from 4 m of the length of the bridge is carried by two connections between the transversal beam and the tie. The connection is a significant detail, which must exhibit a long-term reliable function for an entire length of the service life of the bridge. Finally, a solution using a stiff steel element was designed.
The connecting element is shown in Fig. 9. The drawing of the transversal beam can be seen in Fig. 10. The connector was embedded into the precast transversal beam and then it was connected using the screws to the steel part of the tie before the launching. If the longitudinal beams were located under the bridge deck, the connection with transversal beams would become a standard detail of crossing of two concrete beams, which was used at the side span. Such solution would be significantly simpler, completely reliable, and cheaper.

Fig. 9 Steel connector Fig. 10 Precast transversal beam (main span)

The construction of the main span is another issue closely related to the absence of longitudinal beams. If no longitudinal beams were used, the temporary truss had to be produced, assembled, and dismantled to make the launching possible. If the longitudinal beams were located under the slab of the bridge, two possibilities could be considered:

a) The longitudinal beams would be deeper than transversal beams (similarly as it is in the side span (Fig. 7)). It was proven that such beams would be stiff enough for launching over the span of 32 m (the span between temporary supports), and no temporary truss would be necessary. The savings would cover the production, assembly and dismantling of the temporary truss. The appearance of the main span would be similar to that of the side span which would stress a unity of individual spans. However, deep longitudinal beams would change the appearance of the soffit of the main span rather significantly (compare the side span – Fig. 7), which might have been unacceptable for the architect and/or the client.

b) Transversal beams would be deeper than the longitudinal beam. The appearance of the soffit of the main span would be influenced only very little. The contour of the soffit would be very similar to that shown in Fig. 8, the longitudinal beams would be mostly hidden behind individual transversal beams. In order to allow for the launching, rather small steel elements would be necessary for creating a launching track at the bottom surface of the main span. A simple scheme of a possible solution is plotted in Fig. 11.

If the tie of the arch is located above the bridge slab, it is exposed to the direct attack of de-icing salts during the traffic in the winter. Special attention was paid to the design of the concrete mix so that the durability of concrete was guaranteed. If the tie was under the slab, it would be much better protected against environmental impacts.

Fig. 11 Scheme of the launching track
3.2 The rise of the arch
The rise of the arch is very low (1/10 of the span length). It results in a rather high compression force in the arch and also in the high tension in the tie. A question can be raised if the low rise of the arch is really necessary. It would be possible to increase it and to make the structural system more efficient. The more acceptable ratio of the rise and span is about 1/8. It would require to increase the rise from 20 to 25 m. The impact to the appearance of the bridge can be seen in Fig. 12. Both figures are drawings into the photograph. The left figure shows the arch with the rise of 20 m and the right one shows the arch with the rise of 25 m. Of course, if the figures are beside each other; the difference can be immediately seen. However, if only one figure was observed, it would not be so easy to decide which rise is shown in the figure. The appearance of the arch would be at least comparable. Increasing the rise of the arch would bring savings of about 15% of the weight of steel. It would make the arch lighter and cheaper.

![Illustration of the arch, left: with the rise of 20 m, right: with the rise of 25 m](image1)

3.3 Hangers
The number of hangers was also a big issue during a detailed design of the bridge. In the original design (in the competition), 264 hangers were planned. In the detailed design, there was a proposal to reduce the number of hangers to 160. Finally, a compromising number of 200 hangers was used in the final design of the bridge. The number of hangers is dependent on the stiffness of the arch and of the bridge deck. If a longitudinal stiffening beam was used as well as the arch with higher rise, the number of hangers could be reduced, which would make the bridge visually lighter, less anchorages would be necessary which would contribute to the reduction of maintenance and to savings.

3.4 Cantilevers
Steel cantilevers carry the pedestrian lane on each side of the bridge. They are welded to the steel elements embedded in the edge beam of the bridge deck (Fig. 13). Alternatively, the cantilevers could be made of concrete and they could be cast together with the slab of the bridge. They would be significantly cheaper and easier to build. The number of joints would be reduced. The steel cantilevers require
a maintenance and especially, the longitudinal joint between the concrete edge beam and the steel walking board suffers from leakage. A slightly heavier concrete structure of cantilevers would not make any difficulty in the design.

4 Conclusions

The Troja bridge in Prague is a nice bridge structure. It serves to the traffic successfully. However, the costs were much higher than expected in the competition. The reasons may be found partly in a sophisticated structural system and in a complex construction process and partly in strict requirements of the architect. The paper shows some phenomena which could lead to the reduction of the costs and to improvement of the durability due to the more favourable structural performance and due to the simplification of the construction process. The design and the construction process should be balanced. The optimal solution should respect the eligible requirements of the architectural design and also the feasibility and functional requirements of the construction process. The longitudinal stiffening beams in the bridge deck is a major issue in the design of the Troja bridge. This almost invisible part of the structure (compare Fig. 2) would significantly contribute to the simplification of the structural system as well as of the construction process. It would be better to invest the costs to the stiffening of the structure than to temporary structures which were dismantled. The extremely low rise of the arch is also unfavourable for the costs and its contribution to the architectural impression of the bridge is at least questionable. Finally, the steel cantilevers are demanding for the maintenance. The concrete alternative would be easier to build, cheaper and almost maintenance free.

The example of the Troja bridge shows that it would be convenient to find an optimal solution which would require a closer co-operation among the architects, designers, and contractors. The construction process must be considered in an architectural and structural design. A balance between structural design and construction process is the only way how to build a functional, durable, sustainable and economical structure.

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References

E pluribus unum: hybrid structural solutions for next-generation bridge designs

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Abstract
This paper highlights key features of some recent structural designs from the authors’ practice in the field of bridge engineering, all of which can be classified as “hybrid structures”, in various manifestations and interpretations of the term. Thereby, the paper discussed the key decision processes and design iterations that led to the chosen forms and material combinations, the advantages and disadvantages compared to more traditional solutions, and the challenges that are presented to the task of structural verification of these structures. From the point of view of structural forms, examples of bridges that combine the load-carrying behaviour of girders, arches and shells, in various assemblies, are presented. In the shown examples, it will be illustrated how variations of symmetry and asymmetry can lead to challenging, yet particularly interesting structural behaviours. Regarding materialization, the examples will highlight means of using steel, concrete and timber in combination, yet otherwise in their “barest” possible form, largely avoiding sealing layers, bearings, expansion joints and other devices.

1 Introduction
The next generation of architecturally and technologically ambitious bridges needs to take into account technical as well as societal requirements and boundary conditions that go significantly beyond the mere need for a transportation link that is aesthetically pleasing. Environmental sustainability, structural robustness, control of life cycle costs, low maintenance needs and early planning for end-of-life provisions are among the more often cited requirements for the bridge of the future. In structural design and engineering for bridges, this new set of demands is increasingly leading to a paradigm shift - away from more classic, archetypal forms and uniform material choices towards new, hybrid solutions for the structural form and materialization. This paradigm shift is a key driver of innovation and renewed expressiveness in bridge engineering and architecture.

This article summarizes the implementation of some of these concepts and new paradigms in structural designs for bridges on which the authors recently worked in their practice. The chosen examples are a mix of completed works, works currently in the implementation phase, and not (yet) materialized conceptual work for structural design competitions. The focus of the article will be placed on pedestrian and bicycle bridges, as well as on a bridge with a broader city-planning scope, as these types of bridge are most suited and more often considered for novel structural concepts and solutions.

2 Hybrid steel girders for pedestrian and bicycle bridges
The following two examples will show some completed work on pedestrian and bicycle bridges for medium spans, which made use of hybrid structural systems and steel as a construction material in order to obtain structural efficiency, minimal maintenance efforts and the desired, innovative structural form.
2.1 Museion Bridge, Bolzano (IT)

The Museion Bridge in Bozen/Bolzano (IT) was built in 2008 as a result of a design competition for the city’s new museum for modern and contemporary art, for which the bridge represents a principal means of access as well as an architectural highlight at its main entrance. The bridge’s architectural design by Berlin-based architects KSV Krüger Schubert Vandreike, who also designed the museum building itself, envisaged a twin bridge with a complex, doubly-curved shape in elevation, in the plan view and in the cross-section, see Fig. 1. Bergmeister GmbH was responsible for the structural engineering and design in the pre-construction and construction phases of the project.

The structure is composed of two steel box girders with nearly triangular cross-section of continuously varying depth. In the elevation, each girder forms an asymmetric arch along the span of approximately 52m. The steel girders are rigidly connected to the abutments through large steel inlays cast into the concrete, to which in turn the bridge superstructure was welded during erection. The abutments are grounded through heavy bored piles to provide sufficient rigidity against rotation. The resulting integral structure has well-known ([1], [2]) advantages with respect to rigidity and maintenance needs. For the girders, the given system leads to a mixed arch-beam behaviour, which leads to significant compression in the thin-walled doubly-curved steel elements. In order to achieve sufficient local buckling stabilization, the box girders are longitudinally stiffened with narrow spacing; the corresponding structural analysis of the doubly-curved panels required the use of non-linear FEM analyses to account for the “plate” buckling behaviour, which lies in between the one of (planar) plates and (curved) shells.

The chosen materialization and structural solution allowed for a rapid construction on site, with relatively light equipment suitable for the restricted space at the location, and provided sufficient strength and rigidity (for both bending and torsion) to compensate for the challenges presented by the desired architectural vision and form. The main challenges during the fabrication of the structure were presented by the plate forming needed for the complex lower surfaces of the bridge. A sufficiently narrow spacing of internal diaphragms helped overcome this constraint.

The final structure fulfilled the high expectations of the developers and the public, and is currently an appreciated addition to Bolzano’s city landscape at a focal point, at the Talvera river between the medieval and the modern part of town.

Fig. 1: Museion Bridge, Bolzano (IT); Architectural Design: KSV Krüger Schubert Vandreike, Berlin; Structural Engineering: Bergmeister GmbH (W. Weiss, C. Ferraro); photos: S. Drube.
2.2 Isarsteg Nord, Freising (DE)

The Isarsteg Nord in Freising (DE) provides a second example of a medium-span pedestrian bridge for which a strong, sculptural architectural vision could be materialized through the use of steel box girders, the integralization of columns and other support points with the superstructure and the deployment of advanced structural analysis techniques. The structure was built as a result of a design competition, with a project duration of three years and completion in 2015. After completion, the design of the bridge by J2M architects in Munich, Bergmeister GmbH and &structures was awarded with the German Steel Innovation Prize 2018 and the “Preis des deutschen Stahlbaues 2016”, [3].

This new crossing of the river Isar in Freising, near Munich, serves as a pedestrian and bicycle connection between newer residential districts and the old town of Freising, while at the same time improving access to existing and planned recreational areas in and along the Isar meadows. In order to achieve the highest possible level of integration into the natural environment of the meadows, the supporting structure and the pathway are largely “of one piece”: the footbridge, ramps and staircases form a structural unit and seamlessly merge into each other. The resulting structure gives the impression of a “path sculpture” and evokes the structure of a branch, see Fig. 2. The impression of a branch or tree trunk exposed to the forces of nature is reinforced by the choice of weathering steel as main construction material, which forms its long-term corrosion inhibition through a layer of reddish-brown oxides.

The integralization of support points and superstructure presented a number of advantages, but also challenges for the structural design and verification: on one hand, the rigidity presented by the fully welded connections and the continuity of the girders allowed for a very slender main span, with a length of approx. 58m and a girder depth of only 1.2m, without incurring in critical vibration or deformation problems. At the same time, the asymmetry of the support positions caused a complex mixed torsional-flexural behaviour and points of localized stresses, which required special attention in detailing and welding. The choice of welded steel box girders, in this case acting in combination with the concrete layer of the deck to form a composite section, was again crucial to overcome these challenges.

The result of the chosen materialization and structural concept is a pedestrian bridge that achieves the desired architectural and landscaping effect while being virtually maintenance-free, owing to the absence of bearings or expansion joints and of the need to apply and maintain corrosion protection paints.

Fig. 2: Isarsteg Nord, Freising (DE); Architectural Design: J2M Architekten / Ch. Mayr, Munich (DE); Structural Engineering: Bergmeister GmbH (J. Taferner, M. Gander) and &structures (O. Engelhardt), photos: O. Jaist.

3 New Tegetthoff Bridge Graz – bridge, traffic junction, city square

The current Tegetthoff Bridge over the river Mur in the centre of Graz (AT) is a typical steel-concrete composite structure as they were built in the 1970s, able to accommodate three lanes of road traffic and two narrow walkways but designed with pure functionality and material economy in mind. Driven by its desire to increase the use of light railway public transportation (trams) and of further enriching the already-remarkable modern architectural landscape in the proximity of Graz’s medieval town centre, the City of Graz decided to carry out a design competition for a replacement structure for the Tegetthoff bridge, which should carry tram traffic in addition to road vehicles, as well as a much wider band of pedestrian and bicycle traffic. The design team composed by Arch. Prof. W. Tschapeller ZT GmbH (Vienna) and Bergmeister GmbH won this competition with the design shown in Fig. 3 and is currently
carrying out the construction design. The chosen structural design once again showcases the use of hybrid structural forms to achieve architectural or, in this case, city- and traffic-planning objectives with innovative approaches and solutions.

The new Tegetthoff Bridge crosses the Mur with a single span of approx. 65m. While the western side of the river shore has been built-up and is currently represented by a steep supporting wall reaching from the river up to the road running parallel to the river, the eastern side features a much-used walkway closer to the water level. At the same time, while the western bridge end leads to a narrow street framed by two taller buildings, the eastern end opens towards a wide square. Taking these boundary conditions into account, it was chosen to give the bridge a wider cross-section on the east side, thus helping to visually channel the (pedestrian) traffic from the square towards the bridge, and simultaneously to create a direct passage, on the bridge, between the upper road level and the lower level of the river walkway, via the means of stairs on both sides of the bridge centreline. The latter should also serve as a new space to meet, rest and enjoy the city and river landscape.

Fig. 3: New Tegetthoff Bridge, Graz (AT), winner of the 1st prize in the design competition. Architectural Design and graphics / renderings: W. Tschapeller ZT GmbH; Structural Engineering: Bergmeister GmbH (J. Taferner, A. Taras); a) conceptual sketches; b) renderings; c) sections.
This architectural and urbanistic concept was put into structural form through the use of composite steel-concrete girders, with three main box girders and large cantilever arms made of fine-grain steels (with steel grades up to S460) and a cast in-situ concrete slab of varying thickness. In the walkway and bicycle lanes, the latter is directly walked on, without sealing or surface covering. Only the central part, which needs to accommodate the rails for the new tram lines, has a 17cm thick asphalt layer and a corresponding sealing. The need for the allocation of three girders was given by the significant width of the bridge, which at nearly 28m at the widest point nears half the length of the bridge, as well as the significant traffic loads to be carried. This extra width, combined with the stairs and low position of parts of the lateral walkways, is what gives the bridge users the experience of a “square on the water”.

For the architectural concept, the view of the bridge from the river walkway, which passes below the bridge, was also of great importance. The wide, continuously changing cantilever arms on the side of the bridge, combined with the nearly-triangular shape of the outer box girders, create a large degree of visual “motion” in the view of the underside, reminding one of the structure of a large barge or ship. Considering the namesake of the bridge, an admiral of the Imperial and Royal Austro-Hungarian Navy, this appears suitable.

For the structural engineering of the bridge, the chosen hybrid solution and cross-sectional shape and materialization presented a number of interesting features and challenges. For one, the near-triangular shape of the main girders implied that the points of maximum stress (at the lower extremities) occurred in regions with very low sectional width, of only 400mm. This type of shape was found to be ideally suited for the deployment of a hybrid steel section, with thick steel plates made of steel grade S460 in the flanges and more conventional S355 steel in the thinner webs and wide, upper flange plates. The large eccentricities that could be present in the traffic loading from trams required the transverse diaphragms to be designed with particular care for buckling. Most importantly, the desire to avoid sealing and an additional non-load-carrying layer on the lateral parts of the cross-section required special attention to the concrete detailing, particularly in the parts of the bridge where the concrete is partially placed in the tension part of the section. In these areas, it was finally chosen to foresee rebars made of stainless steel and to use a concrete grade of at least C40/50, with higher tensile strength.

The project is currently awaiting the final budget approval by the Graz city council and the co-funding stakeholders. The authors believe that, pending final approval, the bridge will add significantly to the city landscape of Graz and provide an example for a new type of (and role for) a bridge, which manages to combine functions of a crossing, a traffic junction and of a city square, while providing a very appealing architectural experience.

4 Further conceptual works – multi-span bridges with hybrid structural systems

The final parts of the paper are dedicated to two recent designs by the authors which, while not proceeding to realization, still serve to highlight the potential for structural and architectural innovation in bridge construction that can be found through hybrid structural solutions.

4.1 Design for a pedestrian and bicycle bridge in Heidelberg (DE)

The first example of the above-mentioned type of hybrid solution is the authors’ contribution to a design competition for a large network of pedestrian and cyclist crossings in Heidelberg (DE), with the main span crossing the river Neckar at a width of approx. 240m. The bridge design, developed by the authors together with Arch. Ch. Mayr (J2M architects, Munich) features a network of continuous steel box girders with alternating and varying girder heights on each side of the bridge deck, functioning as both parapets and structural elements. Welded, open-section cross-beams span between these, with stringers then spanning between them and prefabricated concrete elements forming the actual bridge deck.

The alternation of the side with the highest / lowest depth on the left or right side of the cross-section leads to an equivalently alternating viewing perspective for the bridge users, which will have the wider panoramic opening on either the left or the right side of their direction of travel. At the same time, reprising the concept explained for the Isarsteg Nord, all stairways and ramps are used as integral bridge piers, this time for a much larger network of paths. One such pier/stairway is placed in the middle of the river Neckar and leads to a viewing platform immediately by the water.

The alternating sides of the parapet-girders lead to a hybrid structural behaviour for the bridge cross-section: while some parts of the bridge are characterized by a heavy torsional action, given by the large cantilever of the bridge deck with respect to the lateral position of the girder (Fig. 4b), other parts can be described as having a trough section (Fig. 4c), with two equally-sized girders on both sides.
The resulting structural behaviour, while complex, is actually quite advantageous: while bending continuity is achieved, the strong reduction of the cross-sectional area on either of the sides at the end of every second span leads to a reduction of constraints (due e.g. to temperature) and thus the possibility to reduce the number of required expansion joints. At the same time, the presence of a trough-type section near the mid-span points leads to advantages with respect to the vibration shapes and frequencies when compared to bridges with more uniform cross-sectional shapes.

For the materialization of the superstructure, weathering steel was once again proposed, as this type of corrosion-inhibiting steel, if correctly deployed, presents significant advantages from the point of view of maintenance and durability. In order to achieve sufficient torsional rigidity in all cross-sections, box girders were foreseen. These are welded air-tight to avoid the inclusion of water and air, which might cause uncontrolled corrosion in weathering steel if permanently enclosed within the structure. Over the Neckar, the lateral girder needed to have a larger structural depth; in this region, the girder features large triangular openings, thus taking on many characteristics of a truss girder. These openings are also used to present seating opportunities towards the historic city centre and castle, while also preserving the option to look westwards.

In the mentioned international design competition, the presented design was awarded the 2nd prize.
4.2 Design for a pedestrian and bicycle bridge over the Rhine (CH/AT)

A second, even more recent example for a contribution by the authors to a design competition that highlights some key features of advantageous hybrid structural solutions is shown in Fig. 5. This contribution to an international competition for the design of a new bicycle and pedestrian bridge over the Rhine between Au (CH) and Lustenau (AT) is a “hybrid structure” in a variety of ways, and represented an attempt to implement a truly novel shape with an innovative combination of materials and structural typologies: timber, concrete and some structural steel, used in a structure that acts as a combination of arch and beam, girder and shell.

The competition asked for a solution to span the approx. 280m of the alpine Rhine that, at the site, forms the border between Austria and Switzerland. An eco- and hydrological renaturation project has been approved for this stretch of the Rhine, and this was supposed to be taken into account in the proposed designs. Preliminary hydraulic studies showed that up to five piers could be placed within the river without endangering the flood protection management in this stretch of river, which has been particularly prone to such events in the past. This number of piers, with resulting shorter spans, awoke the design team’s interest in a covered timber bridge solution, which has a very significant tradition in the region and which is currently experiencing an international revival [4]. At the same time, and counter to what would happen in more traditional covered timber bridges, it was decided early on to attempt to achieve the widest possible visual spans for the users, which from the bridge should be encouraged to admire the impressive alpine landscape of this stretch of the Rhine valley.

Keeping these boundary conditions in mind and reprising (in different form) some ideas from previous common competitions described above, the authors and Arch. Ch. Mayr (J2M architects, Munich) designed a covered timber structure with 5 spans of 56m each and 4 intermediate supports, of which however only every second is used as support for one of the sides of the load-carrying bridge structure.

Fig. 5: Structural Design for a pedestrian and bicycle bridge over the Rhine between Au (CH) and Lustenau (AT); Architectural Design: J2M Architekten / Ch. Mayr, Munich (DE); Structural Engineering: Bergmeister GmbH (J. Taferner, A. Taras); a) conceptual sketches; b) plan and side view; c) structural analysis model of the hybrid timber-concrete structure.
The structure itself is composed of a timber “roof”, made from glulam elements (lateral girders/arches, longitudinal ribs and transversal arches), that acts both as scaffolding for an additional concrete layer on top and as main structural element. Equivalently, the concrete shell is both the layer protecting the timber from atmospheric agents and a structural component in and of itself. The final structure is thus a hybrid, composite timber-concrete shell structure with discrete ribs and a concrete shell. The bridge deck is suspended from this hybrid roof structure.

As can be seen in Fig. 5, this rather unique structural solution leads to a very large opening, of 2*56=112m, on either side of the bridge, i.e. on either the left or right side of the direction of travel of the bridge users, while still taking advantage of the presence of intermediate supports at half this span.

As can be expected, however, the “skipping” of a support for either of the sides leads to significant torsional effects in the structure, which need to be absorbed. For this purpose, heavy transverse stiffening ribs, made from structural steel and shown in blue colour in Fig. 5c, were devised in the design at each of the supports. These stiffening ribs are rigidly connected to the concrete piers and supported each of the long-span lateral arches at their apex, reducing and withstanding the mentioned global torsional effects. The lateral, main arches above the large openings thus acted as a hybrid between true arch (with mainly compression forces) and girder (with mainly bending moments).

An additional challenge presented by this type of design, which concerned the envisaged concrete shell, is the fact that rather steep, curved areas are present at the meeting points of the shell with the piers. In order to overcome this challenge, concrete gunning (torcreting) with special mixtures was proposed, in combination with the use of carbon fibre reinforcement mats, which are easy to bend and deploy and particularly durable.

For the suspended deck of the bridge, it was proposed to use smaller steel beams and angles made from weathering steel as main load-carrying and bracing components, with wooden floorboards as the pedestrian deck. The covering of the roof made sure that this – aesthetically appealing – solution was also durable.

The authors believe that the proposed solution, while certainly complex and “experimental”, nicely highlights the possibilities presented to bridge engineering and architecture by an undogmatic mixing of structural materials, functions and forms.

5 Summary and conclusions

This contribution to the symposium highlighted some recent structural design work by the authors and their colleagues, which has one common denominator: the undogmatic mixing of structural archetypes and solutions, as well as materials, into novel “hybrid” designs, with the aim of achieving a desired shape, functionality and/or structural behaviour. The authors welcome the wider acceptance of such hybrid solutions that they could observe both in their own practice and in other successful projects. The new impulses towards hybrid solutions can clearly be attributed to two main developments in bridge design and engineering: a welcome and overdue expansion of the cooperative dimension in the work of architects and engineers in the discipline of bridge design, and an increased openness of project developers and clients towards solutions that bring the promise of longer durability, low maintenance efforts and life-cycle costs, and the potential for the creation of a distinctive landmark through a bridge construction project. At the moment, both these developments can be mainly observed in the field of pedestrian and bicycle bridges, with some forays into bridges for motorized vehicles within sensitive urban contexts. It is to be hoped, though, that this positive trend will further solidify and expand to all aspects and fields of bridge and infrastructure engineering.

References

Conceptual design of HPFRC and UHPFRC road girder bridges

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Abstract
This paper presents some critical considerations and thoughts about the use of High-Performance Fibre Reinforced Concrete (HPFRC) and Ultra High-Performance Fibre Reinforced concrete (UHPFRC) for the construction of road girder bridges. The ideas here described are based on an on-going research which combines material development, specifically a 100 MPa-strength self-compacting tailor-made concrete, with design considerations built on international code and provisions. Following a design-oriented perspective, different typologies of girder bridges have been studied, both at ULS and SLS. As main finding, the HPFRC may represent an optimal alternative for the construction of road girder bridges; in fact, on one hand, it allows the development of precast structures which overcome the current limitations for this structural typology; on the other hand, a 50% self-weight reduction and relevant savings both in passive and prestressing reinforcement are achieved.

1 Introduction
An international effort to embrace novel Ultra High-Performance Fibre Reinforced Concrete (UHPFRC) solutions into the bridge sector is observed due to the unique opportunities provided by its strength and durability [1]. In the USA the Precast/Prestressing Concrete Institute (PCI), together with the Federal Highway Administration (FHWA), are guiding precasters for developing their own UHPFRC mixes with local available materials [2] and designers to carry out an optimal dimensioning of these structures [3]. Another example is represented by the firm DURA Technology that developed its own UHPFRC, resulting in a catalogue of standardized girder solutions. Since this company was founded a decade ago almost 150 bridges located worldwide endorse its success [1]. These novel materials also proliferate in Europe, and as a pioneer example, the firm RDC has designed and built the first UHPFRC road girder bridge in Spain [4]. Figure 1 shows design examples of the cited companies.

Fig. 1 International application of UHPFRC bridge girders: (a) decked I-beam [5], (b) U-girder [6], and (c) bridge over the Vernissa River (picture courtesy of RDC).

Towards this direction, this paper aims at discussing some critical considerations about the use of UHPFRC and HPFRC for the construction of road girder bridges. First, this paper illustrates differences between these materials in term of general properties and applicable standards. Next, the mechanical advantages of designing with fibre reinforced concrete are conceptually described and applied to case studies, carrying both bending and shear verifications. Finally, a typological analysis of I- and U-girder road bridges making use of HPFRC and UHPFRC is critically presented and discussed.
A general overview of the material properties

2.1 Concrete classification

The concrete can be categorized according to its compressive strength in conventional concrete (CC), high-performance concrete (HPC) and ultra-high performance concrete (UHPC). The definition of the concrete compressive class is very important to choose the standards that cover its application.

Specifically, next Figure 2, top, illustrates the compressive strength ranges corresponding to CC, HPC, and UHPC according to several international codes. There is not a common agreement among the different standards, in fact the limits between concrete classes are not perfectly defined.

Furthermore, steel fibres with different length, cross-sectional shape, and tensile strength can be added to the concrete, resulting in, respectively, FRC, HPFRC and UHPFRC. Therefore, in these cases, the compressive concrete class is not enough to completely define a fibre reinforced concrete, since also the post-cracking tensile class is required. Figure 2, bottom, illustrates the most popular trends of fibre content (in percentage by volume) found in the technical literature for FRC, HPFRC, and UHPFRC. The Figure 2, bottom proves that there is a common range for the three types of concretes. However, UHPC matrix can sustained higher fibre content than 2% thanks to its self-compacting fresh behaviour, while FRC and HPFRC present workability problems with this high fibre contents.

The main advantages of this material came with its post-cracking tensile strength, which can be obtained with the procedure detailed in the Model Code 2010 (MC2010) [7]. This standard proposes a 3-point flexural test, from which a graph plotting applied force versus crack-mouth opening displacements (CMOD) is obtained. Then, residual flexural strengths (1) are calculated supposing an elastic behaviour. This hypothesis is theoretically incorrect due to the cracked state of the sample during the test; however, di Prisco et al. [8] validated it experimentally.

\[ f_{R,j} = \frac{M_{R,j}}{W_{el}} = \frac{3F_j}{2bh_{sp}} \]  

(1)

The residual flexural strengths at CMOD of 0.5 and 2.5 mm, are designated as \( f_{R1} \) and \( f_{R3} \). The \( f_{R1} \) value and the ratio \( f_{R1}/f_{R3} \) defines the Post-Cracking Tensile Class (PCTC). The minimum ratio considered by MC2010 is 0.5 which indicates the most softening post-cracking behaviour. On the contrary, \( f_{R1}/f_{R3} = 1.3 \) defines the maximum hardening material according to this standard. Based on these limit values, five different ductility classes (from a to e) are defined. The rigid tensile law provided by MC2010 is applied in this work. It relates the \( f_{R3} \) with the direct tensile strength \( f_{Ftu} \) as \( f_{R3} = f_{Ftu} / 3 \). This coefficient is widely recognized in the technical literature as illustrated in Fehling et al. [9].

2.2 Mix design

A collection of concrete mixes found in the literature proves that to a compressive strength increases corresponds a: (i) higher cement content, (ii) smaller maximum aggregate size, (iii) lower water-to-cement ratio, and (iv) higher content of reactive admixtures as silica fume or slag.
Figure 3, left, illustrates the relation between the maximum aggregate size $D_{\text{max}}$ and the mean compressive strength of the mixes ($f_{\text{cm}}$). Furthermore, the different colours gather them into three groups according to their reactive admixture content (expressed in kg/m$^3$). A segregation into two groups is clearly identified. The UHPFRCs are clustered together into a small area limited by a maximum $D_{\text{max}}$ of 2 mm. Furthermore, they present the highest content of reactive admixtures. When a compressive strength of 100 MPa is required, a significant cheaper material can be achieved by maintaining the mix design into the right group where $D_{\text{max}}$ is higher than 10 mm.

Concerning fibres, the right graph of Figure 3 describes the mean average residual flexural strengths $f_{\text{Rm}}$, calculated as $f_{\text{Rm}} = (f_{\text{R1m}} + f_{\text{R3m}})/2$, versus the fibre content ($V_{\text{f}}$, expressed in percentage by volume). In addition, fibre tensile strength (in MPa) is showed using different colours. It is interesting to observe that with the same fibre content a higher $f_{\text{Rm}}$ is generally achieved with more resistant fibres. As a general rule, UHPFRC contains fibres with a length varying between 10-20 mm and a tensile strength higher than 2000 MPa [10]. On the contrary, HPFRC tends to be fabricated with longer fibres (between 35 to 60 mm). Shorter and stronger UHPFRC fibres respect to HPFRC can be as 5 times more expensive in term of fibre cost.

### 2.3 International standards

Decades of research upon UHPFRC and HPFRC result on the development of several standards, which can be simplify split into two main groups:

- **International standards**: MC2010 [7], new draft version of MC2020 [11], and new draft version of EC2 [12]. These documents introduce fibre-reinforced and high-performance concrete as a variety of conventional concrete, but there is no consideration of UHPFRC.
- **Specific national standards for UHPFRC**: French code NF P18 [13], Swiss code SIA 2052 [14], American code ASTM C1856 [15], Japanese code [16]; these national standards have been specifically conceived for UHPFRC; they focus on fine-grained cementitious materials with high compressive strength and a predominant hardening response at the post-cracking stage under tensile stresses.

The international standards are based on well-accepted and rooted documents, but as a drawback, they require long time to incorporate recent advanced in concrete development. As an example, the current version of EC2 [17] consider a maximum compressive class of C90, while the new version of the same document extends it only to C100 [12]. On the contrary, the second group defines UHPFRC as a different material. Due to this fact they seize the material outstanding properties more than the first ones. As a matter of fact, the shear strength model of the French code provides three times higher strength than the new draft version of the EC2. As a detrimental aspect of specific national standards for UHPFRC, they are not so well diffused among the practitioners.

As final remark, the Canadian code [18] merge both approaches, introducing into its national concrete standard a specific annex for UHPFRC. According to authors, this example should be followed as it would carry a better acceptance and diffusion of this new material.
3 Design of I- and U-girders road bridges

3.1 Introduction

This Section 3 focuses on how conventional design of girder bridges is modified when adopting these new materials. In the following, after describing the general aspects of conventional structural verifications (Section 3.2), applications to examples of I- and U-girders are treated separately, respectively, in Section 3.3 and 3.4.

3.2 General aspects of structural design

3.2.1 Flexural analysis

Fibre reinforced concrete shows a significantly high tensile strength, and this effect can be accounted for in the design. In fact, according to MC2010 [7] at SLS, a tensile stress equal to \( f_{Ftsk} = 0.45 f_{R1k} \) can be allowed. Savings in prestressing steel can be achieved because of this more permissive restriction when compared with conventional concrete. The standard compressive limits of \( 0.60 f_{ck} \) (under the characteristic load combination) and \( 0.45 f_{ck} \) (under the permanent load combination) should also not be exceeded, respectively, to avoid longitudinal cracks and assume a linear creep. The high compressive strength of these materials can bring an area reduction of the compressive flange.

Concerning the ULS verifications, in the calculation of the ultimate bending moment, the concrete can withstand an ultimate tensile stress of \( f_{Ftuk} = k_0 \cdot 0.33 f_{R3k} / 1.5 \), where \( k_0 \) represents the favourable or detrimental distribution of the fibres with respect the analysed actions. In this work it is assumed \( k_0=1 \). The constitutive tensile law is considered as elastic until reaching \( \varepsilon_{el} \) and then a plateau is encountered until \( \varepsilon_{fu} \). The ultimate strain \( \varepsilon_{fu} \) varies from 4 to 10 according to the technical literature.

Figure 4, top, compares three different ultimate bending moments (\( M_u \)) of a prestressed I-girder bridge made of HPFRC, corresponding to different values of the ultimate fibre strain \( \varepsilon_{fu} \). Concrete compressive and post-cracking tensile classes, are, respectively, 100 and 12c. Comparing the \( M_u \) obtained, it can be stated that fibres present a negligible effect on the ultimate bending moment for this kind of cross-section; in fact, the \( M_u \) increases only 0.8% if the fibres are considered in the calculation.

On the contrary, Figure 4, bottom, shows the same comparison, applied to the negative bending moment calculation of a ribbed deck. In this case, as the area under tension is larger, the ultimate bending moment \( M_u \) increases 62% when considering fibre contribution. As main finding, the fibre contribution to the ultimate bending moment is strongly dependent on the cross-section and the applied bending moment.

![Stress distribution](image)

**Fig. 4** Ultimate bending moment calculation varying the ultimate fibre strain: (top) 3 m-height girder bridge with a 0.25 m-thick slab, (bottom) 0.25 m-height ribbed deck with a 0.60 m-width slab.

3.2.2 Shear analysis

The shear strength provided by fibres may allow to replace the passive shear reinforcement, which represents around 40% of the passive steel in conventional girder bridges. To ensure enough ductility of the members when stirrups are eliminated, MC2010 Eq. 7.7-15, expressed in (2), limits the minimum characteristic value of the ultimate tensile strength \( f_{Ftuk} \). For the example of HPFRC100, the limit is \( f_{Ftuk} \geq 0.08 \cdot 10 = 0.8 \) MPa, considering a 12c, with \( f_{R1k} = f_{R3k} \), \( f_{Ftuk} = f_{R3k}/3 = 4 \) MPa. Therefore, this criterion is largely fulfilled with a fibre content of around 80 kg/m³ (\( V_f \approx 1\% \)).
Whereas there are different mechanical models to account for the fibre contribution into the ultimate shear calculation, the basic concept is illustrated in Figure 5. \( V_f \) stands for the vertical component of the tensile stress supported by the fibres along the inclined shear crack area \( (A_{cv}) \). Theoretically, the main difference between shear models lays in the consideration of the fibre contribution to shear strength together with the concrete contribution, or as a separate term. In the developed examples, substantial differences are observed when applying different standards.

![Fig. 5 Shear strength component resisted by fibres.](image)

Regarding shear verification at SLS, cracking must be avoided by limiting the principal tensile stresses to the same limit as bending SLS verification, \( f_{Rk} = 0.45f_{Rk} \). Furthermore, a high compressive level is possible with HPFRC, which benefits both SLS and ULS verifications. Even though the positive effect of the compressive level on the calculation of the ultimate shear strength is accepted in several standards, the new draft of the EC2 [12] (Annex L for fibres) does not consider it.

### 3.3 Case of study 1: I-girder

A 42 m-span I-girder bridge made with six NU2000 girders [19] spaced 3.70 m was selected as the reference project. Girders and deck were made of CC50 \( (f_{ck} = 50 \text{ MPa}) \). The 4 different girder cases presented as alternatives use UHPFRC of CC125 and PCC21; the deck employs CC50 in the case study 1 and CC60 in the longitudinal joint of the case study 3. Case studies 2 and 4 emulate the decked I-beam system proposed by Tadros et al. [5]. Figure 6 shows the cross-sections of the reference project girder (on the left) and 4 different case studies considered as alternatives.

![Fig. 6 I-Girder scheme alternatives.](image)

For every case of study, SLS and ULS conditions were verified for both shear and bending. The four design cases could bear shear loads without any passive reinforcement, which was 1016 kg (42% of the girder passive reinforcement) for the reference project.

The prestressing of the reference girder consisted of four unbonded tendons. Two of them presented a parabolical longitudinal profile whereas the others were straight. The web width was defined to accommodate passive reinforcement, ducts, and concrete covers. Hence, in the different cases the prestressed has been kept away from the webs to minimize its thickness. When pre-tensioned strands are used, some of them must be debonded near the supports. In addition, strands may be required also at the top flange. Instead, when designing with post-tensioning, blisters above the bottom flanges should be designed to anchorage the prestressing tendons.

Case study 1 and 2 present a higher span-to-depth ratio (slenderness) of 30 while case study 3 and 4 maintain the value of 18.9 as the reference design. The reduction of the mechanical arm in the first
two cases entails an increase in prestressing steel. This effect is counteracted by the reduction of self-weight, as illustrated Table 1 where the material consumption of the different alternatives is compared with the reference project. The case study 4 represents the optimal solution regarding material savings, both in concrete and prestressing steel, but, as a drawback, its complex formwork requires higher investments from precasters. An intermediate alternative which combines simpler formwork with interesting material savings is represented by case study 3.

Table 1  Material consumption and comparison of I-girder bridge alternatives with reference.

<table>
<thead>
<tr>
<th>Case of study</th>
<th>$V_{CC50}$</th>
<th>$V_{CC60}$</th>
<th>$V_{UHPPFC125}$</th>
<th>Prestressing steel</th>
<th>Concrete savings</th>
<th>Prestressing savings</th>
</tr>
</thead>
<tbody>
<tr>
<td>NU2000-Ref.</td>
<td>64</td>
<td>-</td>
<td>-</td>
<td>60</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Case study 1</td>
<td>35</td>
<td>18</td>
<td>-</td>
<td>88.5</td>
<td>↓18</td>
<td>↑50</td>
</tr>
<tr>
<td>Case study 2</td>
<td>-</td>
<td>27</td>
<td>-</td>
<td>54</td>
<td>↓44</td>
<td>↓10</td>
</tr>
<tr>
<td>Case study 3</td>
<td>-</td>
<td>37</td>
<td>6</td>
<td>49.5</td>
<td>↓32</td>
<td>↓18</td>
</tr>
<tr>
<td>Case study 4</td>
<td>-</td>
<td>33</td>
<td>-</td>
<td>40.5</td>
<td>↓48</td>
<td>↓32</td>
</tr>
</tbody>
</table>

Next, a cost comparison of the alternatives with respect to the reference project was performed. The UHPFRC cost was considered as 5 times the cost of a standard concrete. After discussing the results with a construction company, to achieve a clear economic benefit using UHPFRC for I-girder bridges, the main conclusion was that this material has not a clear advantage in economic savings for conventional spans. In view of this meeting, new goals were defined: (i) to achieve longer spans to reduce the number of bridge substructures, (ii) to reduce the cost of the concrete mix, (iii) to minimize modifications to the existing formworks, and (iv) to maintain a conventional deck.

Transportation requirements limit the maximum girder length to 40-45 m [20]. Furthermore, from the craftsman perspective, longer prestressed beds are unusual, and the arrangement of blisters at the top of the bottom flanges would require a modification of current formworks. In addition, the calculation of a new alternative with HPFRC of CC100 and PCC12c revealed that a polygonal prestressing profile was required to avoid passive shear reinforcement.

It has been highlighted the significant material savings and weight reductions when designing I-girder bridges with UHPFRC. However, professionals are yet reluctant to deviations from their current practices. For them to become familiar with high performance enhanced fibre reinforced concretes, the construction of HPFRC structures which overcome conventional span ranges is set as an initial milestone to accomplish.

### 3.4 Case of study 2: U-girder

It is a common practice for precasters to arrange tendon deviators and anchorages on the free space inside U-girders, avoiding any interference between ducts and webs/bottom slab. Hence, U-girders could be a better typology to be designed with HPFRC. A 12.50 m-width deck with a 0.25 m-thick cast-in-place CC30 slab is considered as a case of study. Two girders spaced 6.50 m bear the deck. Figure 7 shows, on the left, a reference cross-section made with CC. If the typical span-to-depth ratio (slenderness) of U-girders is 20 [21], the corresponding conventional span is 45 m. The goal was to reach a 60 m-span and to eliminate the passive shear reinforcement with the use of a HPFRC mix of CC100 and PCC12c. The proposed HPFRC cross-section is shown in Figure 7, right.

![Fig. 7 U-Girder scheme alternatives.](image-url)
To ease segments transportation and to avoid any undesired cold joint, Voo [6] designs short segments, 6-8 m long, to be able to pour them with one concrete batch. However, a scaffolding system must temporarily give support to the girder before the prestressing release. From the authors experience, the use of longer segments with the minimum number of temporary supports is a reasonable alternative. To experimentally ensure the avoidance of cold joints, a mock-up segment can be concreting and search for any planes of weakness without continuity of fibres.

As joints between segments must be under a compressive state, it is convenient to locate them far from sections of maximum bending moments. Therefore, two temporary supports must be arranged to bear three 20 m-length segments. This decision saves 14% of prestressing respect to the 2-segment alternative with a joint at midspan. Prestressing steel was designed as 6 unbonded tendons of which 2 deviates at L/5 from the supports for a better approximation to the bending moment distribution. This configuration results in the elimination of the conventional shear reinforcement.

Different shear formulations according to several standards and publications have been applied to the critical cross section. Figure 8 indicates the ultimate shear strength $\tau_{rd}$ from the different applicable models. There is a huge scatter in the results; as an example, the new version of the EC2 predicts a 40% less ultimate shear strength than the ones obtained from MC2010 and MC2020. This evidence points out a relevant lack of common agreement derived from the use of very different shear models. Three out of the four shear models account for the beneficial effect of the compressive level upon the ultimate shear strength. However, the new draft version of the EC2 (Annex L for FRC) differs from its own analogous conventional concrete formulation since it ignores the beneficial effect generated by the compressive stress. In addition, the Figure 8 also indicates, with vertical discontinued lines, the minimum shear strength required for using a web thickness of 0.125 and 0.150 m. Finally, a thickness of 0.150 m was used since the Model Code formulations are considered for the HPFRC girder developed.

![Fig. 8 Shear formulations applied to HPFRC U-girder case.](image)

As main finding, this second case of study proved that HPFRC allows to overcome conventional span limits for U-girders by 25% (from 45 to 60 m) avoiding any passive shear reinforcement in the webs. Furthermore, common construction techniques are proposed for the conceived example.

4 Conclusions

This paper presented a critical overview on the application of UHPFRC and HPFRC for the conceptual design of road girder bridges. The main findings of this research are:

- There is a strong difference between UHPFRC and HPFRC mix designs and mechanical properties, which causes the UHPFRC to be around five times more expensive than HPFRC.
- As a general rule, fibre contribution can be neglected at ultimate bending moment calculation for prestressed girders. This contribution should be accounted for only specific cases. Furthermore, fibres can reduce prestressing steel due to less restrictive SLS stress limits.
- On the contrary, fibres can account for a relevant part of the ultimate shear strength. As a matter of fact, all the girder alternatives have been designed to replace passive reinforcement by fibres. In addition, principal tensile stresses at the SLS can be higher.
- Ultimate shear strengths predicted by the national standards for these structural elements indicate a very high scatter among results. A stronger agreement should be pursued.
- The case of study 1 depicts that concrete and prestressing consumption can be reduced, respectively, by 50% and 30% when adopting an optimized UHPFRC decked I-girder solution for a conventional span.
- The case of study 2 proves that conventional U-girder spans can be extended to 25% with the use of HPFRC maintaining the same cross-sectional area of CC 45 m-span solution.
Acknowledgements

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References

The first pearl-chain arch bridge

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Abstract

The Pearl-chain method is to post tension pre-fabricated straight concrete segments together in a desired shape – for instance an arch shape. The method can be utilized when building arch bridges, and it is especially advantageous when erecting bridges above highways, canyons or rivers. In those cases, the segments can consist of so called SL-decks, which are made of a combination of light aggregate concrete and normal concrete. Using these unique decks, the joined Pearl-chain arch is light, and can be lifted further by crane during the erection. By using the Pearl-chain method, the closure time of the below highway during erection can be reduced to hours instead of months, because the time consuming work with preparation of foundations and post-tensioning of the concrete segments is done at a site next to the road.

In Denmark, the first ever Pearl-chain arch bridge has built on a challenging soft soil spanning over a river. The bridge had a main span of 13 m (full arch), and two side spans of 6.5 m (half arches). A tension tie was integrated in the roadway above the arches as a top slab, and it was connected to the half arches at the bridge abutments. By that approach, transfer of the horizontal reaction force to the soft soil could be avoided. In between the arches and the top slab, a specially developed pervious concrete was applied.

The lifting procedure of the Pearl-chain arches for the bridge took half a working day, and a mobile crane could lift all the Pearl-chain arches from one side of the river. The reason for the low weight was that SL-decks were used as the concrete segments in the arches.

1 Introduction

At present time, most short or medium span concrete bridges are built as slabs or frames, but 100+ years ago, it was also common to see closed-spandrel arch bridges of stone, masonry or concrete [1]-[2]. Arch bridges are optimal structures when using concrete, and by reintroducing the arch shape, there may be a potential to lower the material consume and the emission of CO₂.

In many industrialized parts of the world, it is no longer a valid economical case to build concrete arch bridges due to high cost from labour to set up scaffolding and formwork on site. A problem is also that roads must be closed for long periods during the erection of new bridges – a challenge that is also relevant for concrete slab-, and frame bridges.

The pearl-chain method presented in this paper is a promising solution of how to reintroduce arch bridges in present concrete bridge design without increasing the cost, and with a reduced road closure time during erection. The technological solutions are already investigated in other publications by the author [3]-[6], and in this paper, the focus is on the use of the technology for the first time in an actual road bridge in Denmark.

2 The Pearl-chain method and SL-decks

The Pearl-Chain (PC) reinforcement technology is basically to tension concrete elements together in a desired shape. Each segment consists of pre-fabricated concrete with a cast-in curved post-tensioning duct. Post-tensioning cables are pulled through the ducts in all segments, and the cables are then tensioned so that the segments are in compression, and become one large curved structure. Figure 1 shows an example of the PC principle with a circular arch consisting of eight flat precast elements. Here, the ducts are curved in a circular arch shape with a radius corresponding to the radius of the joined circle shape. α is the angle between two elements.
The post-tensioning cable follows the shape of the arch, while the visible appearance shows the non-continuous shape of the joined flat concrete segments. The segments can for instance be made by SL-decks, which is further explained in the next paragraph. When the post-tensioning cable follows the desired arch shape, when pulled inside the curved ducts, the force from post-tensioning only introduces normal stresses in the arch and no bending moments.

The PC arches can be lifted in place by a crane, and several PC arches can be positioned adjacent to each other to achieve the preferred total width of the bridge. On top of the arches, a filling layer is created to level out the road surface.

SL-Decks can be used as segments in the PC-arches: The SL-Deck is a commercialized pre-fabricated deck element (primarily for buildings) and it consists of a combination of light aggregate concrete (LAC) blocks (3MPa characteristic cylinder compressive strength) in the bottom and regular concrete (55MPa characteristic cylinder compressive strength) above. The production method of these decks is flexible, and the end surfaces of each SL-Deck can be provided with a slight inclination enabling several elements to be joined together in a piece wise arch shape. In the joints, the direction of the post-tensioning cable is always perpendicular to the end surface, and there is no force component acting transversely. The SL-Deck is a composite of light- and normal concrete, but closest to the joint to the next SL-Deck it consists of at least 200 mm massive regular concrete. This ensures an even transfer of forces between the elements, and a neutral axis in the middle of the cross-section, at the level of the cable. When the cable duct forms a 90 degree angle with the end surfaces of the SL-Deck elements, the cable goes from one SL-Deck to the next without sudden bending.

In Figure 2, half of the regular concrete in an SL-Deck is cleared away in order to obtain a view of the geometry of the LAC blocks and reinforcement solution. In this case, the PC-arch has four LAC blocks across and a total width of 1.65 m, but elements can be cast as wide as 3.6 m.

Pre-tensioning strands can be positioned in the grooves of normal concrete between the LAC blocks as shown in the figure, and the duct with the post-tensioning cables is placed centered in the deck at a height that ensures that the post-tensioning will not create a significant bending moment – only a normal force. This is possible since the neutral axis changes depending on the cross-section in the element: At the massive concrete zones at each end, the neutral axis is in the middle, but in cross-sections with the LAC blocks, the neutral axis is above the middle because of the LAC blocks.

Reinforcement bars are positioned between the blocks in the transverse direction.
The SL-deck have excellent fire and acoustical properties due to the LAC blocks in the bottom, and this makes this type of deck especially useful in PC arch bridges with long widths (four hours of fire proofing in tunnels), and with pedestrian or cyclist passing underneath (damping loud sounds from trucks as good as wood wool).

3 The first PC bridge

The first PC arch bridge was erected in a rural site in Jutland in Denmark above Vorgod Creek. Figure 3 shows the finished PC bridge at Vorgod Creek.

An existing old three-span bridge was torn down, and the new bridge was erected in the same location. The critical traffic loads on such bridge would come from either special vehicles with eights axels up to a total of 80 tonnes, or from a so called “tandem system” with only two close axle loads of a total of 600 kN (loads without safety coefficients). For small span arch bridges, the tandem system (positioned in the ¼ point of the span) is most critical.

3.1 Geometry and structural system

The new bridge had a main span of 13 m (and a rise of 1 m), and two side spans of 6.5 m. The Pearl-chain segments were made by SL-decks, and four adjacent PC arches were used in each span. The adjacent PC arches were joined by reinforcement. Each main span PC arch were constructed by eight elements of equal length, hereby achieving a circular shape of the joined structure.
The circular shape was chosen due to the simplicity with equal elements. As long as the arch has a relatively low rise, the difference between using circular shape, a parabola shape, or a catenary shape is insignificantly small in regards to additional bending moments. Therefore, the best and most economical solution is the circle shape. The SL deck elements for the PC arches are seen in Figure 4.

![Figure 4](image1)

**Fig. 4** Cross-section and longitudinal section of SL-decks for PC arch bridge

At the site, the soil conditions were poor, and it would be problematic to resist the horizontal reactions from loads on the arches by the possible pile foundations. Therefore, it was chosen to create a more complicated closed static system in regards to horizontal forces, see Figure 5. This was done by attaching the ends of the side span arches to a concrete topslab above all the arch spans. The purpose of the topslab was to resist the horizontal forces from the arches, and therefore, the top slab was post-tensioned to avoid cracking.

![Figure 5](image2)

**Fig. 5** Longitudinal section of Vorgod Creek PC bridge. By courtesy of Sweco Denmark.

Between the arch and the top slab was put a state-of-the-art draining filling layer consisting of a pervious concrete specially developed for the Danish climate. A number of pre-fabricated side elements
closed off the sides of the bridge. The elements were applied, but they really were not necessary since the filling layer of pervious concrete was strong enough to transfer loads from the top slab to the arches, and since the filling layer was designed to be perfectly draining and capable of resisting chlorides. A cross section of the bridge is seen in Figure 6.

For the chosen setup, gravel could have been used as filling to reduce the cost and CO2-emission, but a future potential, which should be investigated, is to use the pervious concrete as part of the load carrying system in combination with the arch and the top slab – a “sandwich arch bridge”. This would have a potential to increase the capacity for such arch geometry significantly compared to only resisting loads via the arches. Initial scaled tests have shown a capacity increase of at least a factor 10.

Fig. 6   Cross section of Vorgod Creek at crown of main span. By courtesy of Sweco Denmark.

3.1.1 A critical detail

The connection between side span arches and the top slab was critical, since the horizontal reactions from the arches are transferred here. Figure 7 shows the final solution for the joint detail as a side cut through the detail.

To make sure to avoid lifting of the top slab at the connection point, there was established post tensioning vertically through the concrete beam/block connecting the arch and top slab. The connecting block also had to resist the bending moment coming from the distance between top slab and arch end.
3.2 Erection of the bridge

The PC arches can be assembled on site by positioning the SL-decks on their side edge in the desired shape in a cleared flat area next to the bridge location. Then, mortar joints are poured, and after hardening for a day, the post tensioning can be pulled and grouted. In the specific bridge, due to the small spans, it was easier to do the assembly of the arches at the SL-deck factory, and then transport the arches to the bridge site.

At the site, a mobile crane would lift the arches and position them on prepared foundations. Conservatively, there was a preliminary system of cables put in between the abutments to resist the horizontal reactions from the arches, see Figure 8. Nevertheless, after positioning the arches, it was clear that the preliminary ties were not necessary.

The PC arches were relatively lightweight (could all be lifted in place from one side of the creek) due to the LAC in the SL Decks, and only 270 mm cross sectional height. Only the final two elements, where the post-tensioning was anchored, were more heavily reinforced massive normal concrete elements to resist the local forces from the anchorage, and the connection to the foundations.

The lifting procedure of all arches took only half a working day, which indicate that the method has potential to reduce the closing time significantly if used above roads compared to other bridge types.
After the lifting of the arches, the longitudinal joints between arches, and the transverse joints between arch and foundation were poured. Subsequently, the side elements were lifted into position.

The filling layer of pervious concrete was cast in steps of small heights to avoid a load case with full load from filling in only one side of the main span, which is unfavourable. Furthermore, it was to make sure that the pervious concrete was properly vibrated.

Finally, the top slab was cast in-situ and post-tensioned, before the temporary post-tensioning between the supports were removed. The last details were the construction of water membrane, the driving lane, and the crash barriers.

4 Conclusion
The first ever PC bridge was erected in Denmark. The method of joining pre-fabricated concrete elements together in an arch shape, and the lifting procedure with a mobile crane was successful and lasted only half a working day. This proved that PC bridges can reduce the closing time, when erecting concrete bridges above roads from month to hours.

Due to the poor soil conditions at the bridge site the bridge was planned with a main span of 13 m and two side spans of 6.5 m. The side spans would consist of half arches, and a post-tensioned top slab, spanning above the whole bridge, resisted the horizontal reactions from the arches, so that no horizontal load should be transferred to the foundations.

The setup of this type of static system gave complex details such as the connection between arch and top slab, but it was necessary for an arch bridge under these conditions.

In between the arches and the top slab was a perfectly draining filling material of pervious concrete. There is a potential in using pervious concrete in combination with PC arches and a top slab in the future as a “sandwich arch bridge” construction. For the current PC bridge, the load carrying part was only the PC arches, and instead of pervious concrete, gravel could have been applied to reduce the CO₂-emission. Pervious concrete was only used for a research purpose in the bridge.

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Conceptual design of the concrete railway bridges on the Púchov Žilina corridor

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Abstract
The paper deals with the conceptual design of two prestressed railway bridges that were put into service in the year 2020. The first bridge is a multi-span structure with 12 spans and a maximum span length of 51.5 m. The bridge is prestressed by bonded, external and EDS tendons. The bridge was originally designed with the assumption of using construction technology based on the cantilever balanced method. The second bridge is a five-span structure with a maximum span length of 65 m and prestressed with classical bonded and external tendons. The bridge was built by the cantilever balanced method. The paper presents bridges from both structural and architectural point of view.

1 Introduction
The presented railway concrete bridges were built within the project “Modernisation of the Puchov – Žilina Corridor for track speed 160 km/h. The bridges are located on the Puchov – Považska Bystrica track section. The length of the section is 8.3 km and includes two tunnels, Diel 1080 m and Milochov 1891 m and three railway bridges. The new railway bridge over Nosice canal is a steel Langer girder with a span length of 124 m and two others are from prestressed concrete.

Fig.1 Location of the bridges on the Púchov – Považska Bystrica section

2 Prestressed bridges on the section
Bridges built from prestressed concrete are crossing river Vah and Nosice dam reservoir respectively, see Fig.1. The character of the obstacles, span lengths over 50 m and much greater traffic load in comparison with road bridges (heavier structure) lead to the decision to use the cantilever balanced...
method as the only construction technology from the very beginning design process. The proposed solutions, e.g., number of spans, span length, longitudinal profile, were further formed by local conditions and mainly by architectural requirements.

The new railway bridge over the river Vah is a five-span double-cell box girder bridge with a span length of 47.5 m + 3 × 65.0 m + 47.5 m. The new railway bridge over Nosice dam is an extradosed bridge with 12 spans and a span length of 39.0 m + 10 × 51.5 m + 37.59 m.

The design of both bridges was affected by the required track profile. A small distance of 4.2 m between track axes is the reason why only a single bridge with two tracks was designed, see Fig. 2. Why double-cell cross-sections, it is given by the width of the structure. Both bridges have a width of cross-section approximately 16 m including side footways, besides extradosed bridge had to have vertical outer webs due to anchoring of the stays. Stays could not be arranged in one plane because of missing space between tracks. To avoid oversized deck slab in the transverse direction with a span length of approx. 10 m, an inner web was used in cross-section. The inner web decreases the effect of the shear lag and creates more space for an arrangement of bonded tendons.

Bridges were designed according to Eurocodes for UIC-71 train, Load Model 71 with \( \alpha = 1.25 \) and Load Models SW/0 and SW/2 [1], [2], [3].

![Fig. 2 Railway profile used for design of the bridges – cross-section of Extradosed bridge](image)

3 **New railway bridge over Nosice dam**

The new railway bridge over Nosice dam is extradosed structure with 12 spans and a length of 596 m, see Fig. 3. The design of the bridge was affected by several constraints. The first is a type of obstacle, requirements for navigation channels and with this connected architectural aspects. The obstacle is the reservoir of the dam Nosice. The obstacle is very flat, so the bridge is well visible from the reservoir banks. The reservoir is also a well-known recreation area with Spa Nimnica.

3.1 **Superstructure**

Two navigation channels were required with a clearance of 31.5 m in width and 7 m in height. Track alignment was approx. 10 m above the navigation channel and therefore this constraint has not affected the depth of the bridge deck. However, from an architectural point of view, it was decided to have a constant depth of the superstructure. To avoid standard and a bit boring superstructure an extradosed tendons were proposed. Four stays are deviated through the short pylons and anchored in 2 m
long concrete blocks attached to outer webs, see Fig.4. The cantilever balanced method was chosen for bridge construction from the beginning and extradosed tendons also helped to avoid hunches over the intermediate supports. The depth of the deck was proposed as $l_{eff}/16$. Conservative slenderness was assumed because permanent action due to weight of ballast, sleepers and rails is much greater than carriage pavement on road bridges and as well as rail traffic action is greater than load due to vehicles. Besides, two tracks are set on the bridge deck. The span length of 51.5 m was proposed with the intention to minimize the depth of the bridge deck and the required capacity of the bridge builder. The depth of the cross-section is 3.2 m, the sectional area of the heaviest segment is 17 $m^2$ and the length of 3.5 m, so the maximum weight of fresh concrete during the casting was 150 tons. The length of side spans was proposed $\approx 75\%$ of the main span.

Prestressing consists of 12 and 19 strand bonded tendons, 19 strand external tendons and 19 strand EDS tendons, see Fig.5. All tendons were protected from stray current. As mentioned above cantilever balanced method was selected for bridge construction with 13 m long starter, 5 segments with a length of $3\times3.5\ m + 3.75\ m + 4.0\ m$ and closure 2 m long. Starter and each segment were prestressed by four 12 and 19 strand cantilever bonded tendons, area of a strand was $1.5\ cm^2$, and strength $f_{pk} = 1860\ MPa$. Continuity tendons were embedded not only in the bottom slab but also in the top slab at the midspan area because high negative bending moments due to train load were assessed. EDS tendons were anchored in concrete blocks attached to the outer web. They penetrate the cantilever deck slabs through rectangular openings. The lifting forces from EDS tendons were partly transmitted to the inner web by internal concrete frames. The concrete struts could not be used here due to the installation of external...
tendons. The track alignment of the bridge is not straight in a horizontal plane but has a circular shape with a radius of 2000 m (track 1). The EDS tendons could be bent in a horizontal plane to follow track direction. Therefore, the geometry of the deck is polygonal, each hammer is straight and internal angles are located at the midspan sections.

Fig.5  Prestressing arrangement - bonded and EDS tendons

3.2 Substructure

The deep foundation was designed with the concern of the lower quality of the ground below the dam reservoir bed. Concrete piles with a diameter of 900 mm and variable length ranging from 10 m to 13 m were used. Each pier was supported by 30 piles. Mutual interaction of the piles was ensured by RC slab with a thickness of 3.2 m and 3.5 m (piers 6,7) and dimensions 13 x 15 m. The bridge has two fixes, piers 6 and 7, to withstand great horizontal forces that include braking and traction forces in combination with centrifugal forces and wind action. The horizontal longitudinal forces used in the design had a value of 8.75 MN due to train load and 3.11 MN due to wind load (characteristic values).

The piers have a variable length ranging from 10.4 m to 18.2 m and they are fixed to foundation slabs. Due to unknown current directions, a circular cross-section was proposed. The diameter of the cross-section in the case of ordinary piers is 3.0 m and piers with fixes 4.0 m. As shown in Fig.3, each pier ends with a hammerhead supporting three bearings.

4  New railway bridge over river Vah

The new railway bridge over river Vah crosses river Vah and adjacent inundation of the river, see Fig.1. The river is not navigable here.
4.1 Superstructure

The bridge arrangement was influenced by the following conditions. The river has a width of more than 120 m here in direction of the track and the side span ends near the portal of the new tunnel Diel, see Fig.6. Therefore, one pier would be always situated in the riverbed. Moreover, designers were asked to create an illusion of an arch bridge, because the adjacent bridge structure was designed as a steel Langer girder. These conditions led to the decision to build a five-span box girder bridge with a length of the main spans 65 m and side spans 47.5 m which is 73% of the main span length. The illusion of arch bridge has been reached by hunches and oversized cross-section depth of 8.0 m at intermediate supports and 3 m at midspan section, see Fig.7. The slenderness is $l_{\text{eff}}/8$ and $l_{\text{eff}}/21.67$ respectively.

Fig.6 View on the railway bridge over the river Vah

Fig.7 Midspan cross-section of the bridge over the river Vah

A balanced cantilever method was proposed for bridge construction from the very beginning. The starters have a length of 13 m and segments $2 \times 3.0 \text{ m} + 2 \times 4.0 \text{ m} + 2 \times 5.0 \text{ m}$ and closure 4.0 m. The weight of the heaviest segment was 190 tons, see Fig.8.
Prestressing is typical for construction technology used. Bonded cantilever and continuity tendons and as well as 8 external tendons in each span were used, see Fig.9. Due to the very high variable load, the continuity tendons were embedded also in the top slab at the midspan area (8 tendons).

Fig.8  Construction of the bridge – starter with stabilisation

Fig.9  Prestressing arrangement - bonded and external tendons
4.2 Substructure

A deep foundation was designed also for this bridge. Concrete piles with a diameter of 900 mm and variable length of 9 m and 10 m were used. Each pier was supported by 30 piles. Mutual interaction of the piles was ensured by RC slab with a thickness of 3.2 m and dimensions $13 \times 15$ m. Foundation pits for piers in the riverbed (pier 4,5) were designed with sheet piling as protection against water around the whole perimeter. Sealing of the pits was carried out with a jet grouting plug of the riverbed. Riverbed was protected here with rip rap backfill having a weight ranging from 200 kg to 500 kg.

The bridge has two fixes, piers 3 and 4. The piers have a variable length, the shortest has a length of 7.8 m and the tallest 14.3 m. A circular cross-section was proposed also for this bridge. The diameter of the pier cross-section is 5.0 m and each pier end with a hammerhead supporting three bearings.

5 Conclusions

Introduced bridges represent a further step in the application of prestressed concrete on Slovak railways, where currently dominates as the main structural material steel.

Originally proposed sequence of steel bow-string arch bridge structures over the dam Nosice had been refused due to architectural deficiency and replaced by concrete structure prestressed by conventional and EDS tendons. Besides structural reasons, the EDS tendons have very important architectural significance.

Different terrain conditions in river Vah inundation in comparison with dam Nosice and demand of investor to create an illusion of arch bridge along with the application of relatively cheap technology of cantilever balanced method lead to the design of continuous five spans prestressed concrete structure with oversized hunches at intermediate supports.

Prestressed concrete confirmed its ability to replace structural steel in the area of railway bridge constructions not only by competitive costs but also by better and more diverse architectural solutions, better noise response and as well as lower maintenance costs.

Acknowledgements

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References

Conceptual design of small footbridges

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Abstract
Although the conceptual design is mostly the domain of large structures, it is also very important for the smallest bridges. For people living in a local region, even the smallest bridge could play an important role in their lives. They walk or ride across it to work, to meet friends, to enjoy free time. Therefore, it is necessary to pay attention also to the smallest pedestrian bridges and to keep in mind, that within their conceptual design the criteria and restrictions are often significantly different from those of the large or multi-span bridges.

1 Introduction
Although the conceptual design of small footbridges is similar in many respects to the design of multi-span and larger bridges, it has its specifics, especially in the urban areas. In our opinion, the characteristics of the conceptual design of small footbridges could be summarized in the following points:

Factors, which have the same weight for conceptual design of small footbridges as of multi-span and large bridges:

▪ Nature of the obstacle and approaching roads (considering the possible height of embankments at the abutments, maximal water level during construction and during floods, gaborites on the bridge and under the bridge, possibilities of construction options in the given local area);
▪ Local characteristics of the area (surrounding architecture; important elements or activities in the immediate vicinity of the bridge; development of new activities that have not been there before due to of the missing interconnection of these areas; considering the impact of the bridge construction on the landscape - should the bridge dominate or blend into the environment?).

Factors, which have less weight on small footbridges than on multi-span and large bridges:

▪ Speed of construction – mostly it is not decisive for small footbridges.
▪ If the small footbridge is a part of a new, relatively long cycle route, the optimization of the structure, with intention to reduce the amount of structural materials as much as possible, will bring only negligible financial savings. And on the other hand, the complexity of the structure will rise and will create a larger possibility for construction errors made by unskilled workers.

Factors, which have more weight on small footbridges than on multi-span and large bridges:

▪ During conceptual design, the author should keep in mind that a small construction site will not have the same background as the large bridges have. Each activity will be unique, it will not be repeated. Construction methods, that can be performed with a relatively simple technological equipment, should be preferred.
▪ It should be remembered that some parts of the structure are likely to be built by less skilled workers and that site supervision will be less rigorous. It is therefore necessary to design these parts of the structure as simple as possible to minimize the possibility of construction errors. It is better to slightly oversize the structural parts. The details should be simple, but at the same time sufficiently robust and functional.
Consider the relationship of the local community to the bridge maintenance. The necessary needed maintenance should be minimized by designing a maintenance-free drainage; preferring an integrated or semi-integrated solution; designing the not visible and obviously not maintained parts to be made of concrete.

Trying to create a relationship of the local community to the bridge. Paying the increased attention to the architectural design of the railing and the colors of the bridge, to the composition of some local characteristics (coat of arms of the city, symbols of the regions, folklore features, something that the local community will be proud of, or will be able to tell something interesting about the bridge to the visitors, etc.).

In some cases, let the local community decide about the color of the bridge or some other small architectural feature (railing design, lightning of the bridge, etc). They will appreciate to be involved in the design process and by this means it is more likely that they create at least some relationship to the bridge, to “their bridge”.

For better explanation of the meaning of the above-mentioned points, some examples from Slovakia are presented in the next part of the contribution.

2 The footbridge in Senica

The city's requirement was to create a footbridge in the city park, which would be somehow exceptional, even if it will have a relatively small span of 16 meters.

Fig. 1 Footbridge in Senica - under construction.

The conceptual design was based on these basic limitations that defined possible proposals:

- A small footbridge in the park must reflect the environment in which it will be built (it must not disturb the peaceful character of the city park).
- Construction must avoid tree felling.
- Ensure the longest possible service life with minimum maintenance.
- Maintain the maximum flow profile of the riverbed.
- Flat terrain - limits for max. height of embankments.

Ensuring the longest possible service life with minimal maintenance has reduced the possibilities of the construction material for the load-bearing structure almost exclusively to concrete, as both steel and wooden structures require more frequent maintenance.

After inspection of the existing footbridges in the area, it was clear that their maintenance is almost zero. The relationship of the community to their bridges in each region depends on its cultural level as well as on the chosen self-government, which continues its predecessor activities.

Therefore, when designing a footbridge in a region, it is appropriate to look at existing footbridges and, based on that, estimate the probable treatment of new bridges in the future. There are regions that
take better care of their footbridges and bridges (mostly larger or historical towns) and there are regions where they do not pay any attention to them at all. In such regions, it is justified to use a slightly more expensive solution, which is, however, significantly more durable even with zero maintenance.

The existing bridge located at the nearby area has already been in a state of disrepair after 40 years of operation, even though its railing was beautifully painted (the visible parts are in some cases well maintained, but the rest of the structure is not).

The request to avoid tree felling meant that any precast concrete alternative will be limited by the possibility of using only a small crane.

To make the footbridge unique, even with such a small span, we decided to use a GFRP reinforcement in the upper part of the deck. This means that it will be the first bridge in Slovakia where this reinforcement type was used with the aim to extend the service life of the bridge. Expansion joints were eliminated (the bridge is semi-integrated) and maintenance-free elastomeric plates were used as bearings. Drainage was solved with a stainless-steel drip edge. For the load-bearing structure, the post-tensioned slab with the thickness of 750 mm with a curved shape in the longitudinal direction and the span of 16 meters was chosen. The curved shape of footbridges in longitudinal direction is always of practical importance (drainage, higher height of the structure above the water during floods, smaller embankments at the abutments) and it also has an architectural significance - it enhances its appearance and subconscious feeling of "crossing the bridge". For small footbridges, the main architectural element is always the railing. In this case, a wooden railing was chosen, so that the bridge fits well into the environment. On the other hand, this type of railing will need some maintenance, or replacement over time, but it is not a load-bearing element and it will not have a significant impact on the service life of the main structure. Furthermore, according to our observations, the visible parts of the footbridges usually have a better chance to be maintained, at least a little.

3 The footbridge in Ožďany

As a part of the conversion of many unused railway lines into cycle paths, it is often necessary to deal with the installation of new structures on existing supports. Such a case occurred at the footbridge in Ožďany.

![Footbridge in Ožďany – tender for construction.](image)

Based on the inspection of the related region and requirements of the investor, the following boundaries limiting, respectively defining, the design options were inspected:

- Existing abutments and gabarit requirements under the bridge exclude the use of structures with upper deck.
There is a large space right next to the bridge, which can be used to assemble the structure (minimal restrictions to the traffic during construction, the possibility of assembly with large cranes).

The bridge crosses the road known as “windy roads of Ožďany” (motorcycle spot).

There is a requirement to minimalize the construction costs with respect to the project budget.

Based on these boundaries it was clear, that the most suitable alternative would be a steel structure with a lower deck (since the whole structure is visible also from below, its maintenance should be better). Since the bridge will cross a well-known motorcycle spot, the design was adapted to this fact, with intention to emphasize this place even more. The truss system with tubular elements and a red-black colour, resembling the frame of the famous Ducati motorcycles, was designed. With the use of non-conventional joints of the truss elements the risk of building the footbridge by an unqualified company has been eliminated. The designed solution has higher technological requirements for the production and assembly of steel structure, but its impact on the additional costs is negligible. As for the social aspects, the motorcycle posters, which are installed on the existing abutments, will be removed, and reinstalled after the structure is completed. Certainly, this gesture should underline the respect for this community. Other thing to consider was the fact, that after the bridge completion, the best view to the motorcyclists riding on the serpentines will be from the top of the abutment. This would cause a negative impact on the smooth bicycle ride by means of photographers and curious people standing on the bikeway. Instead of creating barriers for them to enter the bike path, which would be sooner or later destroyed by vandalism, a viewing platform was designed behind the abutment. The final solution has considered several aspects related to the construction of the footbridge. This should be a standard procedure for a conceptual design of each footbridge and its related surroundings. Surveying local relations is one of the most important parts in the design process of a small bridges and it is therefore necessary to devote enough time to do it. Designer should spend some time at the future location of a bridge and consider all aspects of the region and all construction options.

4 The footbridge in Zborov with occasional possibility of passing heavy lorries

Within the bikeway near the Zborov castle, it was necessary to bridge a local stream outside the village, while the following requirements, restrictions and characteristics of the area were identified:

- The footbridge must be as cheap as possible with minimal maintenance required.
- The footbridge must be able to withstand the occasional passage of heavy vehicles (access to the nearby castle ruins - repairs, construction of the bikeway, etc.).
- The footbridge should blend into the historical character of the locality and deduce a feeling that the bridge is an entrance to the approach route to the castle ruins (crossing the stream under the castle hill).
- The footbridge must respect the nature of the stream bed and fulfil the requirements for 100-year flows.

The requirement for minimal maintenance, sufficient load-bearing capacity for trucks and a minimum price led almost immediately to the idea of using prefabricated bridge girders. The only problem with this solution was the giving of a historical feeling to a modern concrete bridge. Since there are no buildings or other bikeways leading along the banks of the stream and the bridge is not visible from its sides, it was decided to adapt only the bridge deck and accessories to fulfil the “historical feeling” requirement.
From an architectural point of view, it is not appropriate to combine too many materials. By this means the solution with all visible concrete parts, except for two lanes designed for smooth passing of cyclists, covered with stone cladding was designed. The historical character was complemented by a wooden railing, which is very similar in shape to a railing of the bridge across the castle moat, to which the cycle road leads.

Thus, the cycle road leading to the castle was "bounded" between two bridges, the new and the historical one, which are very similar to each other when crossing them. Thanks to the combination of a modern concrete load-bearing structure with a bridge deck and railing that have a historical appearance, it was possible to create a relatively simple and inexpensive structure that fits into its surroundings. GFRP reinforcement is used in the deck slab to extend the durability of the bridge and it will be the first bridge in Slovakia, with a GFRP reinforced load-bearing structure, on which the passage of heavy vehicles is also permitted. This kind of a hybrid bridge, modern on one hand, with a historical look on the other hand, can only be designed when its modern parts are perfectly hidden or are not visible from standard view angles. Otherwise, it would be very disruptive and therefore such a solution should be well considered regarding the site. However, it is one of the ways how to solve a bridging with so many limiting factors.

5 The footbridge near Topoľčany

As a part of the cycle route near the town of Topoľčany, the biggest challenge was to find solution for bridging the Nitra River with an elegant structure. Even through, this footbridge is not very small, its conceptual design has gone through several phases and its design has considered all the aspects that apply to the smallest bridges.
Site restrictions and characteristics:

- Relatively wide, regulated riverbed (approx. 100 meters wide) with side flood banks.
- Limited possibility for embankment height near abutments due to limited maximal longitudinal slopes of access roads.
- An initial, pointless requirement for a design solution, that does not, in any way, interfere with riverbed in between the side flood banks.
- Strict requirement that no interventions are allowed into the flood banks during construction.
- Requirement to create an iconic design for a limited budget.
- Requirement for the minimum necessary maintenance of the structure during its entire planned service life of 100 years.

The initial requirement of the Slovak Water Managements Enterprise (no piers are allowed in the riverbank), would result in a span of more than 100 meters. Such a solution would be iconic, but it also would make the construction significantly more expensive and the resulting structure should most likely have to be made of steel, which was inconsistent with the requirement for minimum maintenance. Therefore, the first task to do was to convince authorities, that the piers with a small cross-section in the inundation area of the river (not in the main riverbed) will not change the flow capacity of the riverbed at all. A hydrotechnical calculation was performed and paving of the riverbed in the floodplain of the river was proposed, 5 meters in front of and 5 meters behind the bridge. This solution made it possible to reduce the theoretical span to only 38 meters, which led to a possibility of designing a concrete structure within a given budget. At the same time, the possibility to use the cast-in-place construction method showed up, using standard available scaffoldings, which can span distances up to 40 meters. The limiting heights of the embankments at the abutments required the thinnest possible bridge deck (800 mm), which was achieved by choosing an extradosed post-tensioned solution. To achieve as
much space between the maximum possible water level and bottom of the structure a curved longitudinal shape of the structure was chosen.

In the first design phase, Macalloy bars were designed for extradossed post-tensioning. Macalloy bars were designed to be anchored to the bottom surface of the bridge deck and into steel plates at the pylons, which are partially embedded in concrete.

The shaping of the pylon was chosen not only with regard to the anchoring of the Macalloy bars, but also with regard to the architectural point of view. At their upper part, the symbols from the coat of arms of the town of Topoľčany were placed, made of thick steel plates. The ribbing of the bridge deck at the bottom surface was designed to give the footbridge a higher aesthetic value, especially when viewed from the sidewalks and the access roads leading to the footbridge along the river. This shape of the bridge deck complicated the reinforcement and also the formwork, but the resulting effect for only a small increase in costs was definitely worth it. In our opinion and our philosophy, which was followed when designing bridges, increasing the price up to 15% is acceptable, when designing a significant architectural structure. Great attention was also payed to the design of the railing, not only from an architectural point of view, but also from the point of view of simplicity of production and assembly. Triangular parts connected by screws through rubber compressible washers allowed easy mounting on a significantly curved surface. Maintenance free drainage was designed in form of galvanized rainwater spouts passing through the kerbs.

During construction phase, aesthetically not very pleasing anchors of the Macalloy bars at the bottom surface were replaced by hidden steel anchoring elements. New solution consisted of steel tubes embedded in concrete, with steel anchor plate at the bottom and separation layer along their surface to avoid bond with concrete, and by this means without any change of the point of load transfer of the Macalloy bars (the eccentricity considered in the original design was maintained).

This alternative solution, proposed by the designer, also make it easy to replace the bars in the future if needed.

The bridge deck is connected to the piers by means of concrete hinges and supported on elastomeric bearings on the abutments. As the entire new cycle route will be lit during night, the lighting of the footbridge has also been carefully designed. High poles for lamps would completely disrupt its silhouette, so a hidden lighting with LED strips and illumination of the pylons and symbols from the city's coat of arms with spot lamps was designed. The LED strips embedded in the middle kerb are protected against vandalism, but since their possible damage cannot be completely ruled out, the footbridge is also equipped with the possibility of installing classical public lighting poles, but we firmly hope that they will never have to be installed. The final solution is a very elegant structure with modern elements, which is also the first extradosed footbridge in Slovakia (extradossed road bridges are already a common practice in Slovakia).

6 The footbridge in the Zborov village centre

Conceptual design of this footbridge was based on the following restrictions and characteristics of the site:

- No embankments at the abutments are allowed, due to the connection of existing sidewalks along the riverbanks and the immediate vicinity of road (it is not possible to rebuild a road because of a footbridge).
- Requirement to ensure that the structure will be min. 0.5 meters above the water level during a 100-year flows.
- Requirement to respect the historical character of the site and the promenade (lots of greenery).
- Ensure the longest possible service life with minimal maintenance.

From the above restrictions, it was clear, that the bridge deck must be as slim as possible, with load-bearing elements placed above it. The spatial constraints on both riverbanks and the nature of the obstacle automatically ruled out any cable-stayed and suspension systems. Thus, the only logical and aesthetically pleasing possible alternative was to use a load-bearing arch located above the deck (langer type beam). At the same time, the arch fits well into the context of the historical character of the area. Also such a solution does not obstruct the view of the park when crossing the footbridge and does not disturb the side views through the tangle of stays, which would arise in the alternative with two arches.
(using only one arch is also a significantly cheaper solution). The historical character of the site is respected by means of chosen decent black colour of the main structure and the use of the lattice railing.

Drainage is solved by maintenance-free rainwater spouts. The concrete bridge deck requires no maintenance and the steel arch is visible, so there is a greater chance that it will be properly maintained. To enhance the historical character of the bridge even more, the coats of arms of the village are set into the concrete at the both sides of arch springing. Low water flows in the summer months will allow easy construction of the bridge on a scaffolding. The bridge deck is post-tensioned in the longitudinal direction. The bridge will be called the "dragon bridge", as there is a dragon in the coat of arms of the village and with a little imagination the arch shape of the bridge resembles a black dragon. Span of the bridge is 25 meters.

7 Final words

Even the smallest bridge must always be designed to meet all criteria, whether structural or functional. Even the smallest bridge requires full attention in the design of all its details and its interaction with the environment. As the famous Othman Amman said "it's a crime to build an ugly bridge" and that is true also for small bridges, because they are a part of a small community for which they are as important as the largest ones. For someone, they are even more important, because through these small footbridges they go to work, to the store, to see friends. Nearby them, one may meet his love on a first date, from them children may throw stones into the water, and to them one may nostalgically return from distant parts of the world.

Therefore, to every bridge, even to the smallest one, its designer should “breathe in” a soul and not just take it as a technical work, although of course, the technical aspect should dominate. But if you don't give a "soul" to the bridge, ordinary people will never appreciate it enough, and that's a great pity. It's a missed chance to make the world a better place.

We, the engineers, create a world that subsequently shapes humanity, and such a constant interaction of people with the environment shapes society. By this means, every technical work should have its own soul, otherwise we will gradually lose ours, too.

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Durable and sustainable conception and refurbishment of road bridges

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Abstract
In bridge design and refurbishment, main efforts are often devoted to calculating the strength according to current codes and traffic loads. However, durability issues are responsible for most of maintenance costs and determine the extent of repair works and traffic impact. These topics become ever more crucial in an environment of ageing infrastructure parks, increasing road congestion and limited budgets.

In this paper, based on practical experiences and guidelines of the Swiss Federal Road Office, the main drivers of deterioration and the possibilities to control them are discussed and practical recommendations are given. Robustness, strategic considerations and aesthetics are also addressed. The aim of this paper is to share experiences to promote building bridges that are more robust, need less maintenance and last longer.

1 Introduction
The study of deficient bridges that have been built in the past has several goals: to help finding these deficiencies in bridges, to better understand their causes, to refurbish them correctly and to enable engineers to avoid the errors of the past in conceptual design. According to the author’s personal experience of nearly 9 years in maintenance planning at the Federal Road Office of Switzerland (FEDRO), life expectancy and Life Cycle Costs of road bridges are a function of the parameters listed in Table 1:

Table 1 – Main parameters governing life expectancy and Life Cycle Costs of road bridges

<table>
<thead>
<tr>
<th>parameter</th>
<th>governing aspect</th>
</tr>
</thead>
<tbody>
<tr>
<td>corrosion and degradation due to environmental influence</td>
<td>conception, details and durability of the waterproof sealing</td>
</tr>
<tr>
<td></td>
<td>water drainage and its redundancy / consequences in case of leakage</td>
</tr>
<tr>
<td></td>
<td>conception, details and durability of lateral bridge edges</td>
</tr>
<tr>
<td></td>
<td>structural elements above the pavement level and below the pavement level</td>
</tr>
<tr>
<td></td>
<td>quality and execution of the injection of prestressing ducts</td>
</tr>
<tr>
<td></td>
<td>strength reduction due to corrosion-induced sectional losses of reinforcement</td>
</tr>
<tr>
<td>degradation due to exploitation</td>
<td>conception, details and durability of pavement joints</td>
</tr>
<tr>
<td></td>
<td>deflection of cantilever elements</td>
</tr>
<tr>
<td>loading</td>
<td>robustness regarding unforeseen load cases or arrangements</td>
</tr>
<tr>
<td></td>
<td>increases in traffic load / requirements for special truck crossings</td>
</tr>
<tr>
<td>strategic reasons</td>
<td>road enlargements beneath passages over highways</td>
</tr>
<tr>
<td></td>
<td>possibility to enlarge the cross-section and river corrections</td>
</tr>
<tr>
<td>aesthetics</td>
<td>conception in an aesthetically accepted and durable way</td>
</tr>
</tbody>
</table>

One sees that loading, on which design engineers normally concentrate, is only one aspect amongst many others, and an increase in loading only a subchapter of it. Corrosion and degradation make about 70-80% of the cost for bridge repair works within a large refurbishment project of FEDRO that typically
covers 15 km of highway length, while only the remaining part goes to strengthening. This paper contains a collection of short case studies, which are quite representative for the entire set of bridges the FEDRO manages. A further chapter is dedicated to experiences in maritime (Mediterranean) climate. Finally, recommendations and a summary are included.

2 Case studies and recommendations
This chapter reuses the entries of Table 1.

2.1 Conception, details and durability of the waterproof sealing
In Switzerland, highway road bridges have been built for a large part during the sixties and the seventies of the 20th century. At that time, waterproof sealing of bridge decks was not implemented or inefficient. Chlorides for de-icing have however been used from the beginning of the seventies, and it generally took approximately one decade for the chlorides to reach the top reinforcement layer. Therefore, in the beginning of the eighties, reinforcement corrosion on bridge decks appeared. In this period, waterproof sealing became widespread, but details were not sufficiently well-conceived, and materials lacked sophistication. In the beginning of the nineties, with the upcoming of the potential field measurement method, it became possible to quantify corrosion for entire bridge decks, and waterproof sealing and its details became technically sufficient. However, making all elements on the bridge deck watertight, e.g. the drainage elements, remains a difficult and important task.

A modern waterproof sealing may be made of polymer bitumen tightening or liquid plastics (polymethyl methacrylate), the latter in case of complicatedly shaped bridge decks, and details have to be strictly controlled [1]. There are also developments to use Ultra High Performance Fibre Reinforced Concrete (UHPFRC) not only as a structural strengthening top layer for bridge decks, but also as a waterproof sealing. As strains tend to concentrate in joints, this new material may still be too brittle to ensure waterproofing in details.

Figure 1 shows a pass road bridge built in 1984, which today still has the original, floating waterproof sealing. The problem here are not the steam evaporation tubes, since inside the box girder (where they are also placed), corrosion is very limited. Instead, the edge joints of the bridge edges are not tight, and chloride-contaminated water runs through them and on the outer surface of the bridge’s box webs, leading to significant damage.

Figure 1 – Bridge with floating waterproof sealing on a pass road in Valais / Switzerland [2]

The bridge shown in Figure 2 is also a box girder (built in 1981). Here, the bridge edges are not tight either and the chloride water finds its way along the steam evaporation tubes to the lower surface of the bridge deck, causing damage to the reinforcement.

Figure 2 – Bridge with floating waterproof sealing on a highway road in Valais / Switzerland; chloride water contamination around steam evaporation tubes, left: outside the box girder, right: inside the box girder [3]
The functionality of a waterproof sealing is visible only in an indirect and delayed way. However, damage caused to the bridge decks becomes significant with time, and refurbishment projects should anticipate the ageing and the replacement of this critical element.

A first way to access the overall functionality is the georadar method that detects corrosion in the top reinforcement layer. It has to be calibrated with several samples (drilled cores). This allows determining the extent of refurbishment work for call for tenders and execution projects. While these drilled cores may be water tightened afterwards, bigger, local windows cut in the pavement and the watertight sealing in order to control the bridge deck are very difficult to be sealed correctly afterwards and give only a local answer. Since it normally takes years between thus accessing the bridge deck and repairing it, such an opening accelerates degradation locally.

When repairing the bridge, a finer and more precise method is needed and provided in the form of the potential field method. Generally, the chloride-contaminated concrete is removed in the critical places and reinforcement replaced if necessary. If contamination is widespread, an additional top layer of new concrete, often UHPFRC, may be applied. In this context, this material has major advantages: its very high strength allows using it in thin layers and it has excellent bond strength, between reinforcement and the matrix, but also between the matrix and the existing concrete. It should be considered that removing it in future refurbishment projects is extremely difficult.

2.2 Water drainage and its redundancy / consequences in case of leakage

The drainage of rain water, be it chloride-contaminated or not, is crucial to the durability of road bridges. Box girders are quite common, and in the past the drainage pipes have generally been led through them, for reasons of enhanced accessibility. However, when these pipes present a leak, important amounts of water may pour into the box girder and stagnate there, causing corrosion, as has happened in many cases (e.g. see Figure 3).

Figure 3 - Leakage in the box girder of a pass road bridge (built in 1989) in Valais / Switzerland [4]

Structural repair inside a box girder is very difficult and costly, due to difficult access and limited space. Placing drainage pipes outside a box girder is much safer, and the cost of a repair intervention from above the pavement level is significantly lower. Figure 4 shows an elegant way to hide these pipes while still keeping them outside the structure. It should be kept in mind that even in that case, the water collectors on the pavement levels should best be connected with double-layered tubes to the drainage pipes to ensure protection from leakage since they still pass through the concrete section.

Figure 4 - Bridge in Lausanne (built in 2017), engineers: Muttoni & Fernández Ingénieurs Conseils, architects: UAS ag, left: view from below, right: with drainage pipes hidden yet outside the structure [5]
2.3 Conception, details and durability of lateral bridge edges

Lateral bridge edges are directly exposed to chloride-contaminated water and fog. In the past, they had therefore a much shorter life span (25 years and less) than the bearing structure itself. In some cases, a protective layer has been placed on the concrete. These protective layers are subjected to degradation, too, start to peel partly when not replaced regularly and do not only offer no protection any more, but also hide the concrete’s state to the eye (Figure 5).

Figure 5 – Highway bridge built in 1983: coating peels (1999 repaired for the last time) on its lateral bridge edge, Valais / Switzerland [6]

Incorrectly conceived lateral bridge edges may not only represent a weak point concerning waterproof sealing and have consequences for the bridge deck, but their own integrity may also be compromised. An insufficient reinforcement connection to the bridge deck may cause severe crack opening and strong corrosion of the bridge edge’s reinforcement (Figure 6, bridge built in 1983). This is even more dangerous when transverse prestressing heads are placed in this region, as shows Figure 7 (bridge built in 1963).

Figure 6 - Separation of a lateral bridge edge of a pass road bridge in Berne / Switzerland, left: cracking, right: insufficient reinforcement connection with no crack control [7]

Figure 7 – Bridge on a pass road in Berne / Switzerland, lateral bridge edge, left: severe cracking and water infiltration, right: ill-conceived connection reinforcement and badly-placed prestressing heads [8]

Due to the short life span, interventions on lateral bridge edges are quite common and represent a disturbance for traffic when replaced. As they are subjected to increased shrinkage since placed on the rather old and rigid, existing bridge deck, they are heavily reinforced. In order to extend their life spans and thus reducing interventions, better control cracking and better protect their reinforcement, they may be executed with UHPFRC.
2.4 Structural elements above and below the pavement level

Several bridge types have major structural elements above the pavement level and therefore exposed to chlorides, such as bridges with a U-shaped girder (trough bridges), cable-stayed bridges, suspender bridges, but also extra-dosed bridges such as the Ganter Bridge on the Simplon Road in Valais / Switzerland (Figure 8, an over 170 m tall and over 600 m long national landmark conceived by Christian Menn). The prestressing cables that lead from the bridge deck to the top of the bridge piles are covered in concrete triangular walls, which had been subjected to alkali-aggregate reaction and have been protected against humidity and chlorides in the past. The coating has been subjected to periodical testing, is still functional and will be replaced in due time.

Figure 8 – Ganter bridge (built in 1980, designed by Christian Menn), left: central span and bridge piles, right: concrete triangular walls (source: Google Maps)

Even structural elements below the pavement level may be exposed to corrosion, be it from chloride-contaminated fog that descends to edge beams of multiple-beam cross sections (Figure 9 left) or humid, badly ventilated environment that may cause severe corrosion to normally weather-resistant Corten steel members (Figure 9 right).

Figure 9 – left: Chloride-induced corrosion on the edge beam of a multiple beam cross section of a pass road bridge (built in 1970) in Berne / Switzerland [9], right: cross section losses in the flange of the steel member of a composite pass road bridge laterally supported by rocks (built in 1977) above and along a mountainous stream in Berne / Switzerland [10]

2.5 Quality and execution of the injection of prestressing ducts

Although post-tensioned, injected tendons are generally sufficiently covered with concrete, they may still be subjected to corrosion if the injection is insufficient and contains voids or the injection grout lacks cement and does therefore not provide a sufficiently protective alkaline environment for the steel strands. The injection process of post-tensioned cables has been demanding in the past and still is, since cables often have lengths of 100 m or more, and voids may persist in high points of the parabolic ducts, or water segregation might occur there. It is at the high points where a deficient waterproof sealing of the bridge deck makes things worse (see Figure 10), although prestressing strands may also be at risk in edge beams of multiple beam cross sections.

The auscultation of prestressing cables is technically difficult. Methods like impact echo or ultrasonic sound produce many candidates for insufficient injection that prove to be correctly injected instead when destructively opened for corroboration. The potential field method allows to determine corrosion of the outer layer of reinforcement, but not of the prestressing strands. The georadar method is not conclusive either. Following this statement, it would need to destructively open the prestressing ducts at a statistically sufficient and reasonably located number of places, which would severely harm or destroy the bridge.
A promising method to access post-tensioned cables in a non-destructive way could be the magnetic flux leakage method that has initially been introduced to detect wire breaks in cable-stayed or suspender bridges. It has shown in a field test that section losses of wire strands inside prestressing ducts in concrete sections are probably detectable [12]. It could especially work well in heavily prestressed cross-sections with few transverse reinforcement (stirrups), as is the case with a large part of the road bridge population that encounters problematic prestressing duct injection.

2.6 Strength reduction due to corrosion-induced sectional losses of reinforcement

When verifying and investigating an existing structure, quite often the engineer determines the structural strength on an unimpaired bridge and regards separately the deterioration due to degradation, which are mainly sectional losses of reinforcement (mild and prestressing), but may also concern alkali aggregate reactions weakening the concrete matrix. However, structural verification and degradation go together, and if the design engineer does not want to draw a too conservative picture of strength losses, complete, quick and cheap overviews (by the potential field method) may be combined by well-chosen intrusive investigations, sometimes stochastically sufficient.

To the experience of the author, structural-related cracks in existing concrete bridges become generally visible if the bridges are ill-conceived, e.g. skew slab bridges of the 1970s that are not sufficiently reinforced to distribute loads transversely, bridges with shear deficits or in case of impaired bridge movements, i.e. frame corners or abutments. Otherwise, in well-conceived bridges, limiting steel stresses in serviceability state seems to be efficient for avoiding cracks. For prestressed bridges that dominate road bridges (apart from underpasses), this goal is normally achieved anyway. However, durability seems – to the author’s view and as exposed in this article – an issue of many other parameters as well whose occurrence is not related to limit states, and has to be addressed by correct conception rather than by ultimate limit design.

2.7 Conception, details and durability of pavement joints

Pavement joints are local bridge equipment that is exposed to heavy and very frequent use and therefore fatigue. Joints may be heavy steel elements in case of long bridges and high vehicle turnover, or just be a stripe of elastic and sometimes reinforced bitumen, although nowadays an alternative to the latter exists, in the form of high performance polymer mass on polyurethane basis, which is of constant hardness in summer and winter, as opposed to the bitumen that becomes very soft in strong sunlight.

In the case of steel pavement joints, durability depends on the massiveness, the correct grouting beneath when implementing and sufficient and correctly anchored reinforcement connecting the steel elements with the bridge / abutment. Elastic mass joints were off more quickly and may show lane grooves after some time; this problem seems to be improved with the use of high performance polymer joints, but long term data beyond 10 years of use is still missing. Steel joints also exist without reinforcement that normally connects them to the bridge / abutment (Figure 11); the connection is ensured in this case by glue bonding of polymer concrete between the steel joint and the bridge’s concrete surface. Long term experience is missing here also. Questions arise whether glue bonding is able to cope with the intensive fatigue loading that pavement joints are exposed to.

Before and after pavement joints, the pavement has to be removed in order to bond the watertight sealing to the joint. As watertight mastic asphalt is applied onto it, this acts a barrier for water drainage, and therefore the transverse water drainage of the pavement must be restored after the replacement of...
a pavement joint. As these works are complicated, for small bridges it may be easier and worth in terms of Life Cycle Costs to replace the pavement and watertight sealing on the whole bridge in the same time.

Except for the case of bitumen joints, pavement joints are normally harder than the pavement before and after them, and support ribs orientated diagonally to avoid lane grooves or pools may be put into the pavement (Figure 11). As they are generally just placed inside the pavement, their efficiency is at least questionable.

![Figure 11 – Diagonally orientated support ribs before and after a steel joint (put in place recently) glued with polymer concrete on a pass road bridge (built in 1974) in Valais / Switzerland](image)

The rubber joint below the pavement joint might become untight, letting chloride-contaminated water pass beneath, which is even more critical in case of Gerber joints. These rubber joints must therefore be controlled regularly.

The life span of a steel joint is approximately 25-30 years, of a bitumen joint 15 years. Polyurethane joints are quite recent and long term data is not yet at disposal, but they are promising. They exhibited problems with excessive bonding between the elastic mass and the corns of pavement’s rolled asphalt. It has been observed to lead to a better performance to place a polymer concrete or mastic asphalt stripe between the elastic mass and the pavement.

If pavement joints are not correctly implemented, their lifetime is heavily impaired and may not even reach 10 years, even in the case of steel joints. Figure 12 shows examples for these premature failures. The reasons were the following:

- problems with quality of bitumen available at the time of construction period
- bituminous joints were too weak for amount of traffic flow
- too high shear forces on bituminous joints on curved ramp bridges
- up to 8 lanes of traffic made many construction phases necessary, which impaired durability
- too weak steel joint elements
- too weak or wrong connection reinforcement
- badly grouted steel plates of steel joints with voids below and
- loose or broken prestressing bolts as a consequence
- implementation during winter (since less traffic), and therefore
  - wrong initial position of joint for longitudinal bridge lengthening and shortening
  - unfavourable weather conditions for material casting
- Inefficient local supervision of construction works
Replacing these joints is a difficult intervention, requiring traffic deviation with high costs as a consequence. This is ever more painful when joints have to be replaced well before their intended lifetime due to inappropriate or too cheap choices. Integral bridges are largely in favour of FEDRO, avoiding joints completely for short bridges (up to approximately 30 m length) or at least at one abutment (60 m length; these values depend of course on the temperature range due to weather conditions and the material the bridge is made of). The pavement of bridge ends with no pavement joint may be equipped with glass fibre grids to distribute cracking. Measuring the true bridge movements in joints allows reducing the size of the joint elements, making it less cantilevering and therefore more durable. Always dimension connection reinforcement in a correct way. The choice and conception of a pavement joint should be executed by an experimented engineer, and its implementation tightly controlled.

2.8 Deflection of cantilever elements

Structural parts of a road bridge that are subjected to local bending may be sharp corners of skew slabs (see Figure 13), cantilever end spans (Figure 12 top right) or, to a lesser degree, slender transverse cantilever edges of large box girders, since they are normally transversely prestressed. They represent a disturbance for axle loads, giving rise to noise disturbances, but also for the blade of snowploughs, which is especially true for pass roads. It might become necessary to locally support structural elements.
2.9 Robustness regarding to unforeseen load cases or positions

Road bridges may comply well with the loading placed in the intended way, but they can exhibit problems when encountering unforeseen situations. Road traffic may access the footpath that is not conceived to bear it (Figure 7 right), or an accidental vehicle may damage or cut the cable of a cable-stayed bridge (Figure 14 left). Special trucks with higher axle loads and total weight than normal traffic may be prevented from accessing the hard shoulder (Figure 14 right).

Figure 14 – left: cable-stayed bridges are vulnerable to aberrant vehicles (bridge pictured in Valais / Switzerland, built in 1987; source: Google Maps), right: prevention of special trucks to access the hard shoulder reduces number of traffic lanes on a short highway bridge (built in 1969; internal source)

Bridges that are optimized for the design situations may however be vulnerable and lack robustness regarding to changes in use. In the past, high material costs often led to very slender structures that may exhibit structural weaknesses when verified with modern codes. As codes still evolve, the conceptual engineer should take robustness into account when designing a new bridge e.g. for impact of too high vehicles on passages over highways.

2.10 Increases in traffic load / requirements for special truck crossings

Road traffic has considerably changed since the beginning of the last century, and so have the Swiss codes’ load models that have to be considered when designing a bridge. In general, an increasing intensity of traffic and weight of vehicles led to ever heavier load models in these codes, see Figure 15 left.

When refurbishing a bridge, strengthening it is complicated and costly, and therefore the load model that was provisioned for design in Switzerland in 2003 was represented in the Swiss refurbishment code SIA 269/1 in 2011, accompanied with reduced coefficients that took actual traffic measurements and simulations on highways into account. The immediate gain was that when verifying existing bridges against state-of-the-art knowledge and highlighting structural weaknesses, a lot of them did not need strengthening any more since load levels to be resisted had been considerably lowered.

It became clear later that bridges that were only capable of bearing such reduced loads, well below the level of codes for new bridge design (Figure 15 left, “2003” curve), could not bear the actions from special trucks (Figure 15 right) that are heavier and have higher axle loads than normal traffic and initially had not been allowed to use highways in Switzerland, but became commonplace later on.

Figure 15 – left: comparison of sum of traffic loads for varying bridge span length, according to the different traffic codes (the year they entered into force is mentioned; “Meystre” is a synonym for the Swiss code SIA 269/1) [16], right: a special truck that may drive on a Swiss highway (source: Schwertransportmagazin).
This is a major issue where interests are strongly opposed between transportation agencies that want to maintain driving permits for special trucks and institutional bridge owners that cannot bear the costs of maintaining strength levels of their bridges, let alone increase them. The load coefficients of SIA 269/1 are currently under revision and will even be increased, although there seems to be potential that the load model underlying it may be conservative since it is an envelope for many static systems.

### 2.11 Strategic reasons

In Switzerland, motor traffic grows by more than 1 per cent each year. More than 40 per cent of this traffic is absorbed by the highways. Especially the ones around big agglomerations such as Zurich, Basle, Lausanne and Berne are very much congested on peak hours. Hence, this encourages enlarging the highways, in several cases up to 8 lanes. Passages over the highways are often too narrow, and their abutments stand in the way of such enlargements. Figure 16 shows one such passage which is still too narrow, even with some reserve space available, since it is simply not possible to push over all existent lanes to one side in order to have enough space on the other side for 2 additional ones.

![Figure 16 – Passage (built in 1974) over a highway with 6 existing lanes in Berne / Switzerland (internal source)](image)

Increasing the number of lanes does of course also affects the bridges where the highways run over. Enlarging a bridge implies shifting a significant fraction of the weight on the outer lanes (where most of the trucks would circulate). As mentioned before, many of the bridges managed by the Federal Road Office of Switzerland have been built in the sixties and seventies of the last century. When verifying them in order to enlarge them, structural weaknesses inherent to the code provisions of that time may arise, making it very difficult to enlarge the cross section. This is a major problem in key bridge works that are virtually impossible to replace, such as the Felsenauviadukt in the north of the city of Berne (Figure 17). When conceiving a new bridge, FEDRO guidelines [17] ask the conceptual engineer to check for a possible future enlargement of the bridge deck.

In Valais / Switzerland, a large number of highway bridges or passages over it cross the Rhone river. A huge cantonal project to correct it and refurbish the riverbed makes the protection of bridge piles and abutments against water-driven erosion necessary and sometimes endangers a whole bridge.

![Figure 17 – Cross section of the Felsenauviadukt (built in 1975, designed by Christian Menn) with very slender transverse cantilever edges (internal source)](image)
2.12 Aesthetic aspects

Bridge design follows mainly its function and topographical constraints and has generally been a civil engineer’s task. Architects have expressed their ideas on some infrastructure works such as tunnels or also bridges, but as the form of structural elements was given by static considerations, they mainly selected elements of secondary importance such as bridge abutments (Figure 18) or tunnel portals (highway A16 Transjurane / Switzerland or by Rino Tami in Ticino / Switzerland). While a cooperation between architects and engineers may give interesting results, regarding to bridges, its expression should not result in ornamentalism that might even impair accessibility of structural elements for later maintenance works.

Figure 18 – Replacement project (2021) of an existent passage over a highway in Vaud / Switzerland with a multiple beam girder of post-tensioned UHPFRC and a structurally relevant shape of abutments that reminds of the typical passages over highways very much widespread in Switzerland; leading engineers: IUB Engineering SA, architects: Atelier Jordan & Comamala Ismail architectes

3 Exposure of concrete road bridges to maritime climate

In maritime climate, humidity stemming from the sea contains not only chlorides attacking the reinforcement, but also sulphates producing gypsum that cracks the concrete matrix. While in Switzerland, chloride attacks are generally limited to the bridge deck and the elements exposed to chloride fog and constricted to the short periods that correspond to the use of deicing salts, maritime chlorides and sulphates attack the whole structure, and the winds with which they are transported often last for weeks. For these reasons, bridge refurbishment may involve the whole structure and can be extremely costly, as it was the case for the refurbishment of the arch bridge Krk (built in 1980) in Croatia.

Techniques to address these issues range from increased concrete coverage for reinforcement, coating or applying a hydrophobic layer and the use of adapted mixtures for new concrete (air entraining agent, increased sulphate resistance). The use of UHPFRC for its increased density and protection for reinforcement is not very common yet since its high material costs weigh heavily in countries where salaries are lower than in Switzerland. A way to circumvent this is to produce this new material directly in the countries involved. When it comes to budgets, a rich country such as Switzerland is in a comparatively comfortable situation. Mediterranean countries resort to road tolls in order to maintain their highway road bridges. For the secondary road net where this is not possible, maintenance is often very hard to keep up.

4 Summary and conclusions

This paper has attracted attention to topics that are of upmost important in bridge refurbishment projects. The main driver of degradation is the use of de-icing chlorides that contaminates the concrete and attacks the reinforcement. While executing research in order to find alternatives to the use of chlorides is highly desirable, existing bridges may be refurbished and new ones correctly conceived in order to protect them to a maximum from such contamination. This is of particular concern for constructional elements such as watertight sealing, drainage elements and lateral bridge edges. Special attention has to be addressed on details.

Conceiving a bridge type with structural elements above the pavement level should be weighed against a possibly excessive use of chlorides, and therefore be carefully reflected before applied to pass roads.
Sectional losses of reinforcement have to be quantified reliably (“loss maps”, locally validated by intrusive investigations) in order to correctly determine the bridge’s remaining strength. However, ultimate limit design will often fail to address durability issues that have to be solved in conception.

Pavement joints are critical elements the longevity may be severely compromised if not conceived and implemented correctly. Dysfunctional pavement joints may represent heavy noise disturbances. Replacement is costly and complicated. Design and implementation demands skill and experience and should not be delegated.

Extraordinary design situations such as accidents or water-driven erosion may represent a greater danger to bridges than the ones normally considered. The intensity of the level of traffic for which the bridge has to be designed / verified must be in accordance with the requirements for special trucks.

Even a well-conceived and durable bridge may have to be replaced prematurely for strategic reasons, which have to be found out about in due time when conceiving a new bridge.

Bridges are constructions that shape our environment. Passages over highways are visible to hundreds of thousands of persons every day, and should therefore respond to aesthetic requirements as well. However, functionality and intelligent design should confer the shape to structural elements in an overall concept, instead of expressing artistic ideas on isolated parts of the bridge.

Maritime climate may be more aggressive and attacks more enduring and covering the whole concrete structure, compared to the short periods and rather limited localisation corresponding to the use of deicing salts in Switzerland. Techniques to protect bridges aim at preventing the agents from entering the concrete surface. The combination of severe attacks and far lower budgets make it very difficult for Mediterranean road authorities to maintain their infrastructure.

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References

The widening & retrofitting investigation for a reinforced concrete voided slab bridge deck

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Abstract
In bridge widenings, compatibility between the new and existing deck is crucial to determine moment transfer ability and the resultant load-related stress distribution characteristics, considering the condition of the existing structure and future construction stages, alongside its associated creep and shrinkage effects. Common best practices involve methods of substrate preparation, reinforcement anchorage and stitching. With industry initiative moving toward the use of integral bridges, widenings encasing expansion joints are often declared unfeasible, both from a cost and safety point of view. The objectives of this study are to determine the internal forces caused by creep and shrinkage of the widened bridge; investigate common design and/or construction practices in bridge widenings; and to calculate the effect of the abovementioned variables on the deformation before and after connection to the new structure. The new and existing bridge deck was modelled using SOFiSTiK 3-D FEA. The existing structure displayed degree 4 (> 0.7mm) bending cracks [1] on one of the main spans which raised the capacity concerns during the conceptual design, preliminary design and assessment stages. The structural solution for the project also investigated the use composite materials for the retrofitting in order to increase the capacity of the existing structure whilst still ensuring compatibility through the bridge widening.

1 Introduction
The oThongathi Dual Interchange Bridge (B292) is located at km 21.61 on section 26 where the N2 crosses the M43 Watson Highway. This structure is one of two major concrete bridges that span across the N2 between Tongaat and Ballito, set to be widened soon whilst accommodating two-way traffic during the duration of construction. The latter consists of the uThongathi River Bridge (B294) which is located at km 24.20. The location of both structures can be gauged from Figure 1. The Annual Average Daily Traffic (AADT) on the N2 over this section of the route, for the projected year 2044, is between 120 000 and 150 000 in both directions. During the preliminary design, site and structural surveys were produced. Geometric information pertaining to the design options were produced by geometric specialists; geotechnical information was obtained from the as-built drawings and further input were provided from geotechnical specialists. Supplementary analysis during the detailed design was necessary to address the extent of the project. Widening B292 on both sides were investigated but based on the results of an economic analysis, it was decided that the outside widening was most feasible which is the current and proposed lane configuration. The former option is regarded as the future lane configuration whilst the latter option is most desirable for access reasons during construction and from a safety point of view for traffic management. B292 (Figure 2) is a double carriageway dual interchange located near the oThongathi Toll Plaza carrying the N2 over the M43, that will be widened on only one side of the bridge. The deck is a continuous reinforced concrete (RC) voided slab deck, cast integrally into piers, however the existing girder type dictates the arrangement for the widened bridge consisting of conventional cast-in-situ methodology. The total length of the structure is 50.1m with a depth of 800mm corresponding to a span/depth ratio of 20. The abutments are perched solid wall-type which simply support the deck and multiple column piers cast monolithically into the deck are founded on pile caps using precast piles. The arrangement for the voided slab bridge consists of 9.05m jack spans and 16m main spans with a newly widened width of ± 14m that accommodates 3 traffic lanes in each direction. Shear requirements consist of solid diaphragms of 1.7m at the piers. The deck depth is 800mm and 500mm diameter voids are used at 750mm spacing, therefore the internal webs are 250mm wide.
1.1 Capacity of the Existing Structure
Cracking results from longitudinal rotation of the main span, despite the previous attempts to repair the cracks, it is evident that the cause of cracking had not been eliminated. Excessive stress resulting in cracks for both bridges have continued to propagate due to movement and strain at the deck soffit level. The main concern is that the existing structure displays severe transverse cracks at both outer sides that will inevitably be widened. Both bridges that make up the Tongaat Dual road-over-road Interchange constructed as a continuous structure in 1989 displayed similar defects after a principal inspection conducted in 2011. Cracks on the wing wall (B292 B) and abutment (B292 A) with resulting spalls on the respective bridge parapets were attributed to longitudinal movements. This phenomenon can be due to forward movement of the abutments or thermal movement of the deck, or a combination of the two. This is evident since in some locations the gap between the parapet handrail of the deck and the parapet handrail of the abutment has closed, which has resulted in the minor spalling of parapet corners. However, in some locations, after spalling has occurred, there remains a gap between the two, which indicates that the movement is reversible and most probably due to the latter i.e., temperature gradient differences or thermal expansion of the deck. B292 had a substantial number of cracks on the deck soffit that had prior to the principal inspection been repaired, using a crack injection. In 2011, further cracking on the deck soffits and bridge cantilevers for both structures totalled 28 metres (14m each for B292 A and B292 B). Upon further examination, crack propagation identified from either side of the existing repair crack indicates that the secondary distress mechanism is transverse related. When the bridge contracts, it is expected that the new cracks will close, whether partially or completely, due to the temperature gradient differences or thermal expansion of the deck as described above and when the bridge expands, it is further expected that the new cracks will open, apart from the shrinkage effects or...
curling of the integral slab edges when this was cast, which is also considered as a possible cause of the existing cracks of the bridge.

1.2 Conceptual Design Phase

The new structure (widening) must be “stitched” integrally to the existing structure which will require some demolition and exposing of existing reinforcement in the transverse direction. Thereafter, a casting gap arrangement must be created such that the new deck can shrink and elastically shorten prior to being linked to the old structure. The chosen option for detailed design was kept consistent with the existing deck type. Solid deck sections and separate spine girders were considered during the preliminary design stage; however, these options were declared unfeasible for obvious reasons. The latter permits the use of a longitudinal joint to cater for movement and stresses along the span length whilst the former results in a non-homogenous cross section and resultantly, differential, or excessive moment distribution between the two deck sections, despite the monolithic connection. In bridge widening’s, compatibility of the new and existing structure is critical to minimise differential moment transfer and distribution, considering the condition of the existing structure, construction stages along with the associated creep and shrinkage effects. It is a common design principle to reduce any excessive strain placed on the existing structure when the stitch is cast. The size of the stitch is also proportional to the amount of thermal shrinkage cracking subjected to the section therefore temporary supports and the preloading regimes are necessary to reduce any excessive strain placed on the existing structure when the stitch is cast. The structural solution involved retaining the existing bridge and widening towards the outside since the existing piled foundations had already been constructed in 1994 (with stub walls at the abutment and stub columns at the pier).

2 Actual Practice: Widening Bridges

The cross section of these deck types has a very deformable nature which effects the transverse and longitudinal load distribution and resulting design moments. Compared to standard bridge deck types which exhibit a parabolic transverse stress distribution, the addition of voids contributes to large variations and peak transverse stresses in the top and bottom flanges of the cross section. This effect can be seen when modelling the existing voided bridge deck with a series of I-beams, as opposed to a single cross-sectional element under the influence of its’ self-weight. It is also evident from the diagrams how this phenomenon contributes to a stress raising effect in the longitudinal direction for a constant cross section. Optimal void diameter ratios are therefore often selected in the order of between 0.6 to 0.8 to increase efficiency without creating excessive stresses from cellular distortion due to the presence of thin and flexible flanges. The current configuration results in a void diameter ratio of 500 / 800 = 0.625.

2.1 Voided Slab Bridge Decks

The use of voids in bridge deck slabs are for obvious reasons, such as, reduced material uses in ineffective areas which promote structural efficiency and subsequently result in lower overall construction cost. In present time, the chosen example adds value because voided structures have their revival in practice. However, the voids provide concerns during construction and complicate the analysis of the structure whereby a different flexural stiffness in the longitudinal and transverse direction needs to be specified to account for the orthotropic slab behaviour. This reduction is accounted for through the application of equivalent plate parameters for the relevant elements. It has been proven that the spacing of the voids has minimal effect on the transverse stress distributions, orthotropic behaviour, and cross section deformation, therefore the void diameter ratio should form the basis of the equivalent plate parameters for orthotropic plate theory [2]. As such, it is a common design principle that the incorporation of voids begins to alter the structural performance of the slab in terms of orthotropic behaviour when the void diameter ratio exceeds 0.6. The averaging of the transverse stresses when using 2-D models prevent the identification of the peak stresses around the voids, which occur due to many different loading conditions such as shear and bending in both longitudinal and transverse directions. Therefore, to idealise the voided bridge deck slab, orthotropic plates were used. Considering the analytical complexity of the numerical model, different elements are specified to cater for the beams and respective flanges (plates) of the voided deck bridge resulting in the following mesh generation (Figure 3). The analytical model of the bridge therefore consists of beam elements to cater for the voids with top and bottom shell/area elements that represent the respective flanges of the voided deck. Two types of symmetrical cross sections were used for simplicity to differentiate between the existing and new
deck. The stitch was modelled using shell/area elements of 800mm depth. The deformable nature of the deck cross sections impacts on the transverse and longitudinal load distribution and resulting design moments.

2.2 Widening based design standards
Existing bridge widenings are an important but difficult aspect and rapid growth leads to traffic congestion on national expressways and as such, an effective solution to the traffic congestion will inevitably result in increased highway and bridge widening costs [3]. This study further examined the feasibility of using pre-stressed concrete (PC) precast girders to strengthen existing RC girders of the same nature. The girder type in question was that of a voided slab bridge deck and the paper proved that this different arrangement of deck types could in fact be spliced successfully, to provide a monolithic and even more robust connection as in the case of RC only. The structural analysis involved a grid beam finite element method and due to the higher stiffness of the PC precast girder, as expected, the lateral splice promoted the mechanical characteristics whilst reducing both the internal forces and deformation of the existing structure, therefore the arrangement was feasible. The paper further went on the provide a practical design proposal for widening existing RC voided slabs which is applicable in the case of many RC voided slab bridge decks, from small to medium span ranges. In the case of [3], the first method of splicing consisted of RC + RC, whilst the second, evaluated RC + PC, whereby the existing structure consisted of 10 girders and the new structure contained 6 girders. The former used identical beam cross sections for the new structure, whilst the latter utilised 50mm deeper sections with a single box void. Therefore, as such the existing RC beams contained three webs and the new PC beams only contained two webs. For this reason and due to the increase in depth, the slender concrete cross section of the new PC structure, without accounting for the pre-stress, still has better torsional rigidity than the former.

3 Modelling Widened Bridges
Lateral splicing, load application, internal forces, and deformation of the existing bridge before and after connection are critical for beam bridge widenings. The widening is modelled by creating a lateral splice in the longitudinal direction using the new beam dimensions to ensure design loads can be applied monolithically to the entire structure. In the longitudinal direction, the existing and new voided slab bridge deck were transformed into an equivalent I-sections. In the transverse direction, the webs of the voided slab behave similar to that of line supports. The transverse analysis depends on the slab thickness and rotational stiffness of the connection between the slab and continuous wall or web. However, the system stiffness was simplified since the depths of the top and bottom flanges are constant for B292 in both directions, thereby accounting for orthotropic slab behaviour.
3.1 Initial results
Large variations and peak transverse stresses contributed to a stress raising effect in the longitudinal direction for a constant cross section, therefore solid diaphragms over the supports were specified in the analysis. Therefore, in these areas, the transverse stiffness of the voided slab was increased accordingly to match the existing bridge cross section. The transfer of the top flange stresses due to live loading and other action effects down the interior webs toward the bottom flanges into the substructure is crucial for the system analytical model calculations.

3.2 Model runs
The transverse stress distribution was monitored by creating a cut at the central pier where the highest tensile stress is expected over this support. Hereafter, the structure was analysed before (Figure 4) and after (Figure 5) the stitch was cast in place. Without accounting for the relieving effect from the reinforcement on the stresses induced within the concrete, calculations reveal the tensile stress at the top of the newly constructed pier will exceed that found within the existing structure. In addition, at the interface of the stitch and the existing deck, the tensile stress is ± 7MPa. This necessitates the use of temporary supports in the vicinity of the stitch along the bridge span length and preloading regimes in areas found above the newly constructed piers.

4 ACTION EFFECTS
As mentioned previously, a hybrid model was adopted to generate the cross section of the deck. The behaviour of this model was also verified with hand calculations. The load that acts on the upper structure needs to be resolved and introduced as the Dead Load + Superimposed Dead Load + Live Loading (NA, NB, NC Loading + Earth retaining pressure at abutments) as well as early and long-term creep and shrinkage.

4.1 Dead Load + Superimposed Dead Load
In the case of RC or PC precast beams, much of the self-weight or dead load does not affect the entire superstructure design. When cast-in-situ slabs, connections and additional dead load are used, then this is often the most critical design case in both transverse and longitudinal directions. Therefore, in the case of cast-in-situ construction, the self-weight of the new structure plays a more important role in bridge widenings. The widening is cast continuously with the existing deck to create the structural interaction between the two bridge decks. For instance, if the new deck was cast directly against the old...
deck, some of the deflections of the new deck under its’ self-weight will be transferred to the existing deck. Therefore, to avoid this, a 400 – 500mm stitch section is cast. Common best practice favours stitches to be cast at least 3 months after removal of the falsework to allow initial deflection under self-weight and early shrinkage takes place with minimal effect to the existing structure. Preloading regimes also aim to combat early settlement of the new structure and balance the new deck in the temporary conditions before the stitch.

4.2 Early and long-term creep and shrinkage
For precast structures, minimal creep and shrinkage effects are expected, therefore the calculations are simplified by only considering the effects of dead load, live load, and settlement difference [3]. However, in some cases [4], it has been proven that after connection of the existing and new structure, tensile stresses which developed in some girders attributed to concrete shrinkage and creep exceeded the tensile strength of the concrete use in the bridge. The latter paper primarily investigated the effect of concrete shrinkage and creep on the widening of highway bridges using PC girders through grillage analysis using a finite element program. However so, B292 is a RC cast-in-situ structure and needs to be analysed accordingly. An effective measure to minimise the internal forces and differential deformation in the widened bridge is to offset the connection time between the new and existing bridge by as far as possible [4]. Further knowledge supports that a large portion of the ultimate shrinkage (RC & PC) and creep (PC only) has already occurred throughout the service life of the structure and therefore the residual time-dependent longitudinal shortening of the existing deck is minimal. However so, the newly widened and constructed deck, will not be at the same pace, and therefore large differential deformations can result in undesirable internal forces and stresses. For the case of the RC Tongaat Dual Interchange Bridge, we are only interested in the internal forces due to shrinkage since there is no pre-stress found or envisaged for the existing and new structure. However, even though common practice favours a wider stitch than what was maintained between the existing girders, the size of the stitch used is proportional to the amount of thermal shrinkage cracking subjected to the section [4], with the additional problem of catering for vibrations and movement due to uninterrupted traffic. For the longitudinal analysis, since the magnitude of NA & NB loading greatly depend on the loaded length, the widened part of the structure was used as notional lane 1. The action effects of both analyses are critical with emphasis on the internal forces and deformations.

5 ANALYSIS AND RESULTS
The deck soffit cracking is primarily transverse which puts the main longitudinal reinforcement in question, however the shear and distribution or secondary reinforcement was also checked at various locations in the structure.

5.1 Bending requirements
The largest voided slab span is 16 metres, and a depth of 800mm corresponds to a span/depth ratio of 20, in the common range (≤ 25m) for these types of bridge decks [5], although still relatively slender in comparison to similar bridges successfully constructed. This is also considering the fact, that all piers are built in, with the diaphragm soffit coinciding with the deck soffit, without any haunching at supports. Another voided slab bridge within 20km of B292 which is approximately the same width as the unwidened structure with a main of 24m contains a 25 % increase in section depth at integral piers and corresponds to a span/depth ratio of 16.

5.1.1 Longitudinal bending
The critical load case occurs at mid-span from the inner to the outer pier. The minimum area of main reinforcement in RC members is calculated by assuming \( b_0 \) is equal to the average breadth of concrete or section below the top of the voids (i.e., excluding the compression flange) for non-rectangular voids (cl 3.8.4.1). The main reinforcement provided in the existing structure is 20Y25 + 22Y25 ALT bars with 2Y32 bars found at each outer side of the deck for the bottom flange which results in the design crack width being exceeded at SLS conditions.

5.1.2 Transverse bending
The critical load case also occurs at mid-span from the inner to the outer pier. Transverse reinforcement provided in the flanges should satisfy the lesser of the following criteria for the top (principal
compression) and bottom (principal tension) flanges of either 1000 or 0.70% (cl 3.8.4.2 b) and 1500 and 1.0% (cl 3.8.4.2 a) of the minimum flange section, respectively. In voided slabs, the global transverse bending effects (My) are significant, even so in determining the transverse shear calculations and it is expected for this reason to be the purpose of the specification. Minimum flange sections for transverse secondary reinforcement are to be determined in directions parallel to the webs, however additional reinforcement may be required in RC members to control early shrinkage and thermal cracking (cl 3.8.9). The main reinforcement provided in the existing structure is 400Y12 bars with ABR, constant along the span length of the entire existing structure.

5.2 Shear requirements

Voided slab bridge deck types are the subject of major critique [6], such that two precast PC voided slab structures exhibited severe longitudinal shear transfer failure, which led to increased chloride ion penetrability and corrosion of beam pre-stressing strands. Longitudinal shear can also be established as the cause of cracking more so than the phenomenon of transverse shear, expanded on in the previous sections due to the orientation and location of cracking. However, a transverse analysis is also warranted since the structure will be connected in all means, as mentioned before. The localised cracking results transversely and is concentrated in the mid-span of the main span. With both outer piers being hinged, as described before, and therefore designed to withstand movement with minimal moment and the pier base, excessive mid-span loading conditions could also result in the increased longitudinal rotation of the deck and subsequent deformations and cracking. However so, even though the main aim of voided deck slabs is to reduce the bending moments, which has been studied extensively in single span structures of 20m, 30m and 40m [7], resultantly the shear resistance drops when comparing solid to voided slabs, as expected.

5.2.1 Longitudinal shear

The critical load case occurs at both the inner to the outer pier where the existing structure, provided with Y12 links @ 500mm c/c for the inner and outer beams is found to be under-reinforced. For shear in voided slabs (cl 3.4.4.3), the longitudinal ribs between the voids should be designed as beams (cl 3.3.5 and cl 5.4.2.3) to allow for shear forces in the longitudinal direction, including any shear due to torsional effects (cl 3.3.4).

6 DISCUSSION OF WIDENING POSSIBILITIES

Since RC is designed to withstand cracks under service load, the corresponding section stiffness is a cracking stiffness, whereas in the case of PC, this would be the full section stiffness. The voided slab relative stiffness for RC and PC, concludes that the cross-section stiffness of the latter was at least a third greater than that of the former, even despite the shallower PC section [3]. PC is also designed to certain stress limits that prohibit tension in critical areas of the structure whereas with RC, the voided slab works under cracked conditions. Inevitably, a higher stiffness implies increased load bearing capacity in comparison to a lower stiffness, which promotes the use of PC sections. The mechanical characteristics of the existing structure was determined to demonstrate that an RC + RC bridge widening is indeed feasible, and the best option considering the current distress mechanisms. It should be noted that since the existing bridge has longitudinal voids, another possible solution is to post-tension the existing slab. The structure exhibited severe transverse cracking, a distress mechanism common amongst voided slab bridge decks, which will often crack due to the stress raising effect in the longitudinal direction as earlier discussed. However, so, both the transverse and longitudinal structural analysis was performed and at various locations in the structure, the internal forces and deformations were determined as described above.

7 CONCLUSION

At SLS, the design crack widths are exceeded for the longitudinal bending calculations. Even though shear transfer failure is expected in precast structures, inevitably, the new structure (widening) with different concrete strength and elastic moduli will perform differently in early ages with regard to stresses and deformations in comparison to the existing load bearing structure. With the possibility of another addition to the widening in the future, ensuring shear transfer becomes more pivotal between the existing structure to the adjacent widened ones. Furthermore, excessive deformation of void formers has been reported, with the additional problem of void formers containing water. The later results
in excessive pore water pressure in the structure and hence an item has been created to drill into void formers and release the water which subsequently reduces the pressure build up. From the transverse analysis at ULS, there is great advantage by increasing the section depth of the existing structure for the rebar provided since in some cases the ultimate design moment exceeds the moment of resistance of the steel. Aesthetically speaking, increasing the section depth is viable using steel or carbon fibre plates for the existing structure and matching this with the depth of the new deck. Preliminary calculations indicated that the existing structure may not be sufficient to meet code requirements and as such, strengthening the structure using composite plates was investigated. However, the cracks are monitored on continuous basis and despite minimal or negligible movement, for the new structure (widening), this phenomenon (when the ultimate design moment exceeds the moment of resistance of the steel) has been alleviated through provision of couplers and adequate reinforcement. Hydro demolition should be used to break into the existing structure to avoid delamination on areas of microcracking when other alternative means are used such as jack hammers and drills which damage the concrete interface. This is especially important when attaching the new structure to the existing structure. The lapped reinforcement in the critical areas of the bridge deck is crucial to ensure transverse live load distribution, as well as an in-situ concrete stitch that is also warranted. Since the existing structure displays severe transverse cracking in the bottom flanges of the voided slab bridge, which occurs from the mid-span to the outer support in some cases, using PC cast-in-situ girders (post-tensioned) will inevitably induce excessive creep in the adjacent zones of cracking and may have further negative impacts on the flanges due to shear lag effects. However, the new structure (widening) has been chosen to be designed as RC and certain pre-camber levels for the deck have been defined to alleviate some of the internal forces and deformations of the entire structure. Further analysis on the effect of cracking on slab behaviour and transverse shear on cellular voided slabs is recommended to be conducted.

Acknowledgements

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References

Abstract
The protection of the environment and the mitigation of human-induced climate change has become a global objective and a pledge by the majority of the developed nations. The major steppingstone for the legislative directions of the participating countries was the 2015 Paris Agreement. For the engineering industry, this direction has several repercussions that need to be explored through the diverse nature of engineering works. Carbon footprint reduction in designs is the main objective, which will be achieved through management and design tools and approaches. A set of guides have been and are being produced to support these objectives and national standards are beginning to develop.

This study focused on different types of tools for quantifying the amount of carbon in a bridge. Three case studies were reviewed, each with a different design for the same footbridge. The variations of calculated carbon content for each case study were assessed, as well as the impact that different tools have on the outcomes. Behaviours and design trends driven by the characteristics of carbon tool are discussed. Finally, the study also addresses the uncertainty during the stages of preliminary design and detail design and comments on the accuracy of existing tools.

1 Introduction
In the 21st Century the world is facing the impact of climate change and the need for an effective and progressive response has been recognised internationally in the Paris Agreement [1]. In 2019 the UK became the first major economy to set a target of being net zero carbon emissions by 2050 [2]. Such a goal entails searching for the balance between produced greenhouse gas emissions and the amount removed from the atmosphere.

1.1 Net Zero Strategy in the United Kingdom and the Legal Framework
In 2008 the Climate Change Act [3] was passed by the parliament, committing the government and the devolved administrations of the United Kingdom to an 80% reduction in carbon emissions by 2050, compared to the 1990 greenhouse gas emissions. In 2019, this Climate Change Act was extended to aim for 100% net zero by 2050 [4]. However, even though the act and its extension have legal status in the United Kingdom, it remains unclear how this will be imposed and implemented.

The feasibility of this goal is subject to debate, but the consensus is that it is difficult to reach [5], and it will require sustained policy amendment and implementation in all economic activities to achieve it. Several guidance documents have been published and discussions have been taking place across the industry to identify the gaps in the existing approach, the needs to be fulfilled to create a net zero culture, the tools to achieve such a goal, as well as the methods of implementation.

Codes and standards are facing a rapid change to offer a platform for infrastructure projects to manage and reduce whole carbon in the delivery of projects [6], to provide requirements and guidelines for quantification [7], and calculation methods [8]. These codes are fundamental for understanding and putting carbon reduction into practice.

1.2 Concepts
In order to obtain the most appropriate solution, we need to know and understand objective indicators and parameters which facilitate the decision process. The life cycle assessment (LCA) includes the extraction of raw materials, transport, refining, production, further processing, assembly, in-use and finally its end of life phase, see Fig. 1.

The embodied carbon of a product is often measured from cradle to factory gate or from cradle to site. The embodied carbon in the library of a LCA tool is measured from cradle to “grave”, which is the most complete boundary condition.
1.3 Carbon reduction phases

Measurement is key in the carbon reduction process. It allows us to optimise our design as well as understand the effectiveness of the actions and decisions taken.

First, a representative group of existing projects is assessed to establish a baseline carbon footprint. Then, this data is analysed, and target areas are identified. The results can be split by elements (for example, earthworks, structures, ancillaries) or by construction stages. This information is used during the decision-making of new designs. In each stage of the project life, the carbon footprint is recalculated to control and measure the carbon reductions of the new designs. Afterward, this new information is used to recalibrate our approaches and measures.

There are multiple available tools to calculate the carbon emissions of a design; therefore, understanding the differences and their impact on our decision-making is critical for an effective carbon reduction process.

2 Case studies – Stanhope bridge

We present three case studies to explain how the choice of carbon tool affects the measurement of carbon in a design and how this could affect decisions at the preliminary design stage. To achieve comparable results the following methodology was undertaken. First, the design of a single structure was considered. Then, three structures options were proposed for the single structure. Afterward, three tools were used to measure carbon on all three options.

Stanhope bridge is located in Highgate, North London, and it comprises a steel foot/cycle bridge supported on masonry abutments suffering from cracks on one of the abutments and deterioration of the bearing supports. The assessment and feasibility study carried out by WSP recommended the demolition of the bridge and abutments and the creation of an at grade crossing. This would minimise any ongoing risk and future maintenance cost. As part of the optioneering study, Haringey Council requested the inclusion of carbon emissions for each option within the comparison table.

The Options Comparison report developed five design options. This paper focuses on only three: simply supported steel Warren truss; integral concrete precast arch, and simply supported timber deck, see Fig. 2.
As part of the optioneering study, a preliminary sizing was carried out to determine the approximate material quantities required, whilst providing the client with an idea of the foot/cycle bridge’s aesthetics. At this stage, the purpose of the exercise is to obtain a ‘high-level’ understanding of the carbon footprint associated with each superstructure type. In this specific project, all the options needed piled foundations due to the high soil retention loads allowing us to focus the study on the deck.

Fig. 2 Integral concrete solution and Steel Warren Truss solution with headroom and vertical alignment constraints.

2.1 Calculation assumptions

The carbon quantities were calculated for the deck structure only. To isolate the deck, any difference in the earthworks, mass hauling, or abutments were not considered. This is reasonable because all the options required piles and therefore, we can assume that carbon emissions from the foundations will not vary significantly. Only in the case where two or more solutions resulted in similar carbon emissions would an in-depth comparison of the foundation requirements have been undertaken. Surface / pavement layers, and handrails were not included either.

The service life of the footbridge is 120 years as stated in the project scope. When an element is not found in the library of the carbon calculation tool it is replaced by the amount of material it comprises.

2.2 Materials

The assumed material quantities based on the preliminary sizing of the structure are as follows:

The precast concrete integral arch option would be made of 148 tonnes of precast concrete; 4.5 m³ of in situ concrete. It was assumed that approximately 0.9 m³ of concrete would be required for the repairs over the 120 years design life.

The Steel Warren Truss option would comprise 15 tonnes of structural steel, including connections, and 31 kg of paint. Four elastomeric bearings are assumed to be made of 6kg elastomer rubber and stainless steel. Their replacement would be every 25 years.

The Timber truss solution would comprise 9.4 tonnes of hardwood timber. The steel connections (bolts, etc) would be made of 140 kg stainless steel. Their replacement would be every 50 years. The elastomeric bearings were assumed to be similar to the Steel Warren trust.

3 Carbon tools

This study calculated the carbon emissions of the three case studies using three of the commonly used tools in the UK industry when the calculations were carried out: One Click LCA [11]; Highways England (HE) Carbon tool(v2.3) [12]; BCSA & Tata Steel / BCSA / Atkins Carbon Footprint for steel / composite bridges. These three tools are PAS 2080 compliant.
For the optioneering report, carbon quantities were calculated using One Click LCA, an automated life cycle assessment software widely used by the industry. It uses more than 90k data points in the world including ICE and Tata Steel libraries. The calculations are split into earthworks and mass hauling; construction materials; construction operations and use phase.

The HE carbon tool provides the carbon footprint of bridges by dividing the civil engineering elements of the bridge into separate carbon calculations including earthworks, fencing, drainage, road pavements, structures and retaining walls, business and employee transport and others. Each individual element is then summarised to provide the total carbon footprint. This way, it is easier to identify the most onerous case that needs to be targeted for carbon reduction.

The BCSA & Tata Steel bridges carbon footprint calculator is focused on steel and steel-concrete composite bridges. Construction, maintenance, and traffic delays are considered for the measurement of the carbon footprint. One of its most important points is the subdivision of the structure into its core elements (deck, abutments, foundations etc.) that help identify the elements that carry most of the weight of the carbon production.

The carbon emissions results are summarised in Table 1.

<table>
<thead>
<tr>
<th>Option</th>
<th>One Click LCA</th>
<th>Highways England</th>
<th>BCSA/Tata Steel/Atkins</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Materials + Maintenance</td>
<td>Site impact</td>
<td>Materials + Maintenance</td>
</tr>
<tr>
<td>Integral Concrete</td>
<td>39</td>
<td>27</td>
<td>34</td>
</tr>
<tr>
<td>Steel Warren Truss</td>
<td>36</td>
<td></td>
<td>33</td>
</tr>
<tr>
<td>Timber Truss</td>
<td>14</td>
<td></td>
<td>14</td>
</tr>
</tbody>
</table>

3.1 Tools comparison

Based on the results from the three tools, one of the main differences is the consideration that is given to Life Cycle Assessment by One Click LCA. The embodied carbon in the libraries of Highways England and the Tata Steel tools do not consider LCA, these tools are referred to as non-LCA tools.

The data referring to the carbon footprint of the bearings is lacking, because of the unavailability of the Environmental Product Declaration (EPD) that is associated with bearings and their manufacturer. One Click LCA and Tata Steel tools provide the option to calculate the footprint of the bearings, while the HE tool does not.

A relevant difference among libraries is the ability to import materials from more detailed sources. The non-LCA tools are not open-sourced and therefore it is impossible to import data from outside sources. The embedded material library of the HE tool is very limited and may not represent the realistic options available to the designer.

Another major difference between non-LCA tools is the maintenance factor. In the HE tool, no maintenance consideration is made, while the Tata Steel tool provides sufficient options to put a comprehensive maintenance plan in place and calculate the associated carbon production.

3.1.1 General comments to all options

One Click LCA, as explained in section 1.2 above, automatically calculates the emissions for replacement during the required service life of the structure, in our case study, 120 years. It also offers the option of inputting your construction quantities, energy use on site, materials use on the site that do not constitute part of the asset, water use on the site, waste generated on the site, and additional trips for the transport to the construction site. Supplier research indicates that reliable figures for these quantities are not possible as they are not currently monitored or tracked. Because of this, an average site scenario
was selected with an estimated on-site area of 768 m². The characteristics of this scenario are shown in Fig. 3 and the result is shown under Site impact for One Click LCA in Table 1.

![Table 2](image)

**Table 2**  
Input comparison among tools for Integral concrete option.

<table>
<thead>
<tr>
<th>Materials &amp; Maintenance</th>
<th>One Click LCA</th>
<th>Highways England</th>
<th>BCSA/Tata Steel/Atkins</th>
</tr>
</thead>
<tbody>
<tr>
<td>Precast concrete</td>
<td>ICE library</td>
<td>General concrete</td>
<td>C40/50</td>
</tr>
<tr>
<td>In situ concrete</td>
<td>C40/50, 40% recycled binders in cement (400 kg/m³ / 24.97 lbs/ft³)</td>
<td>ICE Library C40/50 CEM II/B-S - 28% GGBS</td>
<td>C40/50</td>
</tr>
<tr>
<td>Repairs (0.3 m² every 40 years)</td>
<td>Automatically calculated</td>
<td>100% CEM I - C40/50 Business travel</td>
<td>Automatically calculated</td>
</tr>
<tr>
<td>Others</td>
<td>1000 km Private Vehicle for Maintenance</td>
<td>Traffic Delays 1 Principal Inspection + 2 Interim Inspection</td>
<td></td>
</tr>
</tbody>
</table>

Table 2 summarises the key elements of each calculation, the material picked from each library and how the carbon emissions from the factory gate to “grave” were calculated for the non-LCA tools.

The lack of maintenance option in the HE tool was overcome with the usage of bulk materials. Although this refers to construction materials, the fact that the tool offers the option of different cycles in the life of the structure can be utilised to simulate the maintenance of concrete over a period.

For site impact, the following rough estimated values were used in the non-LCA tools to obtain an approximate value for comparison with One Click LCA results described in section 3.1.1.

The HE tool allowed to input of fuel, energy (35000kWh) and water (20000l); business and employee transport (30000km); and waste treatment. In the other hand, the Tata Steel tool treats waste treatment and traffic data separately, and includes all of the relevant information such as business and employee transport (30000km); as well as fuel and energy consumption for transportation (speeds, distances etc. as per default values) and usage of site equipment.

These quantities were assumed considering that concrete requires curing periods for various elements that may extend time on site. Also, the greater volume of in-situ elements, the more workers will
be required on-site, and this translates to a larger compound for parking and welfare. Temporary works will be higher for in situ construction and where safety cannot be built into the construction stages.

4 Discussion

4.1 Geometry, topology and materials
During the conceptual design stage quantities of materials are determined from simplified calculations and designers’ knowledge and experience. Geometry and topology influence the total amount of material required. To optimise the design, it should be performed to solve the existing constraints problem and the structure should be shaped according to the material used, avoiding artificial solutions. To quote Eduardo Torroja, “Each material has a different specific personality and each shape imposes a different stress state. The natural solution of a problem -art without artifice-, optimal compared to the set of previous constrains that originated it, impresses with its message, satisfying, at the same time, the technical and artist demands”. Also “One should become so familiar with the structure as to have the feeling of being, in full vitality and sentiment, part of it and of all its elements. It is necessary to achieve a sincere understanding of the process of resistance, through the deformation that is always essentially united with the process of stressing. The comprehension of a structure requires intuitive knowledge of the ethology of its resistance and of its constituent materials” [13].

The specification of these materials is assumed at conceptual design stage. Decision can be taken during detailed design to reduce the carbon emissions of the chosen solution [14]. Fig. 4 shows the potential impact of the material specifications for reinforced concrete.

<table>
<thead>
<tr>
<th>Measure</th>
<th>Average</th>
<th>Climate Impact</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>No action</td>
<td>Max</td>
</tr>
<tr>
<td>Cement - supplier</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>lowest</td>
<td>-7%</td>
<td>-6%</td>
</tr>
<tr>
<td>highest</td>
<td>13%</td>
<td>14%</td>
</tr>
<tr>
<td>Cement - additives</td>
<td></td>
<td></td>
</tr>
<tr>
<td>slag 20%</td>
<td>-10%</td>
<td>-8%</td>
</tr>
<tr>
<td>fly-ash 20%</td>
<td>-10%</td>
<td>-8%</td>
</tr>
<tr>
<td>fly-ash 5% (fly or partly)</td>
<td>-18%</td>
<td>-12%</td>
</tr>
<tr>
<td>fly-ash 15% (fully or partly)</td>
<td>-18%</td>
<td>-13%</td>
</tr>
<tr>
<td>slag 25% (fully, according to TGD)</td>
<td>-19%</td>
<td>-15%</td>
</tr>
<tr>
<td>Reinforcement - supplier</td>
<td></td>
<td></td>
</tr>
<tr>
<td>low</td>
<td>-19%</td>
<td>-19%</td>
</tr>
<tr>
<td>European average</td>
<td>-6%</td>
<td>-5%</td>
</tr>
<tr>
<td>unknown origin</td>
<td>31%</td>
<td>46%</td>
</tr>
</tbody>
</table>

Fig. 4 Calculated performance of individual measures to reduce climate impact [15].

4.2 How the results of the carbon tools would influence the design process
Comparing concrete and steel alternatives in Table 1, the climate impact reductions in Fig. 4 could result in the concrete option being more carbon efficient than the prefabricated steel footbridge once the quantities and specifications are refined at Detail Design.

In addition, new technologies and materials are being developed and their use could significantly reduce the carbon emissions. For example, the use of reinforced fiberglass is currently being study for more sustainable maritime constructions [16]. However, there are not public Environmental Product Declarations (EPDs) for these new materials. Tools should balance the flexibility to incorporate new materials and carbon factors from supplier’s information with the desire for a consistent approach.

As good practice, the comparison of solutions should be carried out among LCA results. For example, integral solutions require more concrete at the abutments while bearings require more maintenance. Also, after demolition Fiber Reinforced Polymer wastes are currently deposited in landfills while in the case of concrete, once demolished, can be recycled for later use as soil aggregates [17].

Clients are starting to require that contractors compile the data of on-site work but until we have tracked information records along two or three years the estimation of the construction at the preliminary stage will not be reliable. However, construction efforts and transport to the site have a very small proportion of the CO2e of the whole solution, and the savings in this stage will not significantly affect the difference in carbon between one option or another.

This study has highlighted the associated difficulties and the tedious thinking process required to consider all the elements involved during construction and maintenance when using non-LCA tools.
The results in Table 1 are proportional for each tool. Therefore, the used tool would not impact the rank of the options if used correctly.

The number of available tools to assist in the calculation is increasing. Each client tends to have its preference which results in the development of multiples options. For example, RSSB Rail Carbon Tool has been developed only for Network Rail and the UK Rail Industry in general [18]. Depending on the characteristics of the selected tool some factors could be underestimated or overlooked. They also would focus the attention on identifying the most onerous element (foundations, substructure, structure, ancillaries) or phase (materials, construction, maintenance) that needs to be targeted for carbon reduction. Designers should be conscious of the characteristic of the selected tool.

During the Conceptual Design stage, carbon calculations must be used to inform targeted design decisions. The efforts should be centred on the LCA material embodied carbon, Fig. 1, including its demolition. It would be fruitful to engage with the contractor during the preliminary design; however, the current uncertainty of the real quantities on site and the relatively small potential variations put in a second level the carbon emissions during construction. The fact remains that increased prefabricated materials and increased buildability will help to reduce waste.

Also, designers have the responsibility to overcome the existing barriers to implement the most efficient solution, whether engaging with the client or upskill ourselves.

The next step would be to track the quantities of carbon emissions during the lifespan of a project (preliminary design, detail design, construction, use and maintenance) to be able to compare initial estimations and final results. The authors of this paper propose to treat carbon emissions in the same way as risks and to follow a similar management based on Eliminate / Reduce / Inform / Control (ERIC) [19]. Design logs should inform and evaluate the impacts of the changes from the initial solution.

### 4.3 Further discussions

Another topic of discussion could be how far as designers we should go and if it is our moral duty to reduce the in-use carbon by, for example, reducing carriageways in our proposals, which could adversely affect our fees. If so, this can be achieved through engagement with our clients to make the environmental aspects of the scheme a success. Also, we need to understand the in-use carbon associated with an alternative. For instance, the option “doing nothing” might appear to be a zero-carbon approach and result in extra congestion or longer journey distances.

### 5 Conclusions

There are multiple available tools in the industry to calculate the embodied carbon in our designs. In order to obtain reliable results, carbon calculators should be compliant with existing codes. These tools should also allow the inclusion of new materials to catch up with innovations in the field. We need to be aware of which are the characteristics (e.g. stage or phase focused, LCA or non-LCA) to avoid any kind of overlook.
At the Conceptual design stage, the focus should be on the product (A1-A3), its maintenance (B2-B3), and the end of life of the structure (C1-C4), as those are the detected areas where we can be more effective.

Designs should be performed to solve the existing constraints problem and shaping the structure according to the material used, avoiding artificial solutions.

The use of tools to compare options and identify where the largest carbon reductions could be made may be of more value than the absolute carbon number that comes out at the end.

Analysing the estimated values at preliminary design and detail design stages and comparing them with the actual values obtained at the construction stage would improve our designs in the future by detecting errors in the system or underestimations.

To reduce obstacles to achieve the most feasible carbon-efficient design we must engage with all the involved parties at this early stage to prioritise carbon reduction in the decision-making process and facilitate the buildability of the design.

Acknowledgements

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References

Trimming the structural ‘fat’: the carbon cost of overdesign in bridges

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Abstract
Two hypothetical bridge designs are presented, one 'lean' and one with significant 'fat'. They are both highway bridges with the same overall geometry. Both are formed from pre-cast beams and in-situ fill using the same concrete mixes and beam types. For illustration, the 'fat' solution incorporates piles and lightweight fill, whereas the 'lean' design uses spread footings and a regular granular fill. Embodied carbon (CO\textsubscript{2}e) for the two bridges is calculated using How to calculate embodied carbon published by the IStructE. The 'lean' solution is shown to contain 38.9% less carbon than the 'fat' solution (885 tCO\textsubscript{2}e vs 1449 tCO\textsubscript{2}e). Commentary on the solutions is provided. Reasons for overdesign and options for discouraging overdesign are discussed.

1 Background
In 2019 the UK Government enshrined in law the requirement to bring all greenhouse emissions to net zero by 2050. Concrete construction is a key contributor to carbon emissions - cement alone is responsible for 8% of global CO\textsubscript{2} emissions [1].

Infrastructure accounts for 13% of cement use [2]. While small compared to buildings (83%) [2], it is also considered one of the most difficult industrial sectors to fully decarbonise. While use of structural alternatives such as timber grows in building design, such practice is difficult to replicate in bridges. Bridges are designed for a 120 year design life and require - and will continue to require - durable materials like concrete. A high proportion of process emissions from concrete can only be offset through carbon capture, a technology which remains unproven. This means the civil engineer's role in reducing emissions through design efficiency is vital.

For the UK to meet the targets it has set itself, it must re-think concrete construction. While whole-sale cradle-to-cradle changes are required, the 21st Century civil engineer has a key role in kick-starting the process. Concrete is a versatile and durable material and surely has a role to play in future construction, but must be used wisely. Efficient design and prudent specification can both contribute significantly to reductions in structural embodied carbon (CO\textsubscript{2}e).

2 Scope
This paper presents two hypothetical concrete bridge designs – one building in significant ‘fat’, representative of overly conservative design, and one with the design made ‘lean’ via better design decisions. Both bridges are simple highway bridges with one lane of traffic in either direction and footpaths. Both bridges are formed of pre-cast concrete beams connected with in-situ fill. Embodied carbon for the two designs is estimated using the principles laid out in the document How to calculate embodied carbon published by the IStructE in 2020 [3].

The paper focusses on some of the decisions made towards the latter part of the concept design stage, during the development of the Approval in Principle (AIP) document. At this time the designer has certain decisions – and items to agree with the client – which may have a significant impact on overall embodied carbon in the structure.

3 Problem statement
An existing single carriageway with 4m lanes and 3.5m footways runs in a cutting. A new bridge is required to carry a single carriageway road spanning over the cutting. It will have 3.5m lanes and 2.5m footways. Thus the overall bridge width parapet-parapet will be 12m.
No ground investigation has been carried out near the site. A desk study of geotechnical information in the local area suggests generally good quality granular fill with high SPTs (approx. 40), however some poor quality made ground is known to have been used 100m along the route during construction of the road cutting. It is agreed to progress the design in the absence of ground investigation (GI), with 'design validation' occurring once GI has taken place in a year's time.

A feasibility study is carried out where various solutions are considered. It is decided to progress a solution with pre-cast pre-stressed beams with in-situ concrete infill supported on in-situ concrete abutments. It is decided to use cantilever concrete wingwalls.

4 Solutions

4.1 Baseline

Both solutions have the same geometry (clear span = 18.9m / structural depth = 1.0m). Both solutions use TY8 pretensioned pre-cast beams. TY beams are standardised beams in the UK and Ireland, produced by multiple different manufacturers. They have been used commonly on road projects since the 1990s [4]. TY8 beams are 750mm wide and 800mm deep. Full geometries are available from manufacturer literature. Concrete grades are as follows: beams C50/60; in-situ deck C40/50; abutments and wingwalls C40/50; piles C32/40. All concrete is assumed to use cement type IIB/S (25% GGBS).

4.2 Scenario 1 (Fat)

Scenario 1 prioritises ease of construction, avoidance of risk and design simplicity.

The TY8 beams are arranged as a sold slab one against another. This is considered easier to construct as no formwork is required between the beams, only a backing rod or similar. The whole beam besides the soffit is cast into the concrete making cover requirements less onerous. It also reduces hard to inspect areas and reduces the number of potential roosting ledges for pigeons. Pre-cast parapets are selected so that they can be sourced from the same supplier as the beams, and so that a patterned concrete finish can be achieved.

The beams rest on elastomeric bearings. The bearings are supported on RC 'inverted T' abutments with back walls to prevent water ingress to the bearing shelf. The wingwalls are RC cantilever walls. Because of the concerns of the possible presence of poor quality made ground, piles are specified to support the foundations of the abutments, wingwall 1, and wingwall 2. Fig. 3 shows the structure in plan. Longitudinally, one abutment is free and one abutment is restrained. Both abutments are designed ignoring any propping action from the deck. The use of bearings simplifies the analysis as the deck becomes essentially simply supported. It also makes the division of responsibilities between the bridge engineer and geotechnical engineer clear and reduces the amount of interface required between the different parties. This is considered beneficial because it minimises the iterations between teams that eat into the design programme.
Fig. 3  ‘Fat’ scenario – plan showing deck outline; abutment outlines; wingwall outlines; piles; parapets (left). 3d visualisation (right).

The abutments and wingwalls are constructed by excavating out the existing fill – which then is disposed of. Lightweight fill (expanded clay) is specified to manage the eccentric load on the piles. Fig.4 provides a long section and elevation of the fat scenario. The use of both lightweight fill and piles is an extreme case, but reflects a highly risk-adverse approach taken due to uncertain ground conditions.

Fig. 4 Fat scenario – long section showing main bridge section and extents of backfilling, elevation showing formed finish above ground level and piles beneath abutment and wingwall 1 and wingwall 2.

4.3 Scenario 2 (Lean)

Scenario 2 adopts a similar construction typology but with a greater emphasis on material efficiency. The TY8 beams are spaced in a beam and slab configuration, making the deck more efficient than a solid slab configuration, albeit with greater formwork requirements than Scenario 1. Steel parapets are selected to minimise weight on the deck. Fig. 5 provides a section through the lean solution. GRC formwork is provided spanning between the beams.

Fig. 5 Lean scenario – deck cross section (pre-cast dark grey; in-situ light grey; surfacing hatched)

The bridge is detailed as integral with full fixity at the abutments. A diaphragm is cast between the beams. Large bars are required to control cracking in the hogging zone but sagging effects are reduced at mid-span, reducing pre-stressing requirements.

The designer assumes good quality granular fill, which will be transported from a source less than 50km away. The poor quality made ground 100m away is acknowledged and its potential presence on site is identified in the project risk register and geotechnical risk register. Spread foundations are specified instead of piles, utilising the bearing capacity of the soil. The deck connection means that the overturning due to soil loading is not critical and granular fill can be specified instead of lightweight fill. However, the connection also means thermal actions become significant and the abutment foundations are designed accordingly. Fig.6 shows the plan view of the lean scenario. Fig 7 provides sections through the lean wingwalls.
Fig. 6 Lean scenario – plan deck outline; abutment and wingwall spread foundations outlines (no piles); parapets.

Fig. 7 Lean scenario – wingwalls. Spread foundations are used instead of piled foundations. Steel parapets are specified. Conventional granular fill is used instead of lightweight fill.

5 Carbon analysis

Concept level calculations were produced to justify the 'fat' and 'lean' configurations. A schedule of principal quantities was then produced. Pavement materials, drainage apparatus, and utilities apparatus were all excluded from the schedule, being assumed to be similar in both options. *How to calculate embodied carbon* published by the IStructE was then used to determine the embodied CO2e for each structure, using a cradle-to-completion scope (Modules A1-5). Table 1 shows the embodied carbon factors assumed for each material.

Table 1 – Cradle-to-completion embodied carbon factors for each material (to three significant figures)

<table>
<thead>
<tr>
<th>Material</th>
<th>Density [kg/m³]</th>
<th>Waste rate</th>
<th>Travel distance [km]</th>
<th>A1-3 [kg CO₂e/kg]</th>
<th>A4 [kg CO₂e/kg]</th>
<th>A5w [kg CO₂e/kg]</th>
<th>Σ A1-5 [kg CO₂e/kg]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete C32/40</td>
<td>2400</td>
<td>5%</td>
<td>50</td>
<td>0.138</td>
<td>0.00533</td>
<td>0.00849</td>
<td>0.152</td>
</tr>
<tr>
<td>Concrete C40/50</td>
<td>2400</td>
<td>5%</td>
<td>50</td>
<td>0.159</td>
<td>0.00533</td>
<td>0.00960</td>
<td>0.174</td>
</tr>
<tr>
<td>Concrete C50/60</td>
<td>2400</td>
<td>5%</td>
<td>50</td>
<td>0.180</td>
<td>0.00533</td>
<td>0.0107</td>
<td>0.196</td>
</tr>
<tr>
<td>GRC formwork</td>
<td>2400</td>
<td>5%</td>
<td>50</td>
<td>0.235</td>
<td>0.00533</td>
<td>0.0136</td>
<td>0.254</td>
</tr>
<tr>
<td>Reinforcement</td>
<td>7850</td>
<td>10%</td>
<td>300</td>
<td>0.760</td>
<td>0.0320</td>
<td>0.0900</td>
<td>0.882</td>
</tr>
<tr>
<td>Prestress wire</td>
<td>7850</td>
<td>10%</td>
<td>300</td>
<td>0.760</td>
<td>0.0320</td>
<td>0.0900</td>
<td>0.882</td>
</tr>
<tr>
<td>Galvanised steel</td>
<td>7850</td>
<td>1%</td>
<td>300</td>
<td>2.76</td>
<td>0.0320</td>
<td>0.0284</td>
<td>2.82</td>
</tr>
<tr>
<td>Expanded clay</td>
<td>500</td>
<td>5%</td>
<td>50</td>
<td>0.393</td>
<td>0.00533</td>
<td>0.0219</td>
<td>0.420</td>
</tr>
<tr>
<td>Granular fill</td>
<td>1800</td>
<td>5%</td>
<td>50</td>
<td>0.00747</td>
<td>0.00533</td>
<td>0.00162</td>
<td>0.0144</td>
</tr>
<tr>
<td>Elastomer</td>
<td>1500</td>
<td>5%</td>
<td>300</td>
<td>2.85</td>
<td>0.0320</td>
<td>0.153</td>
<td>3.03</td>
</tr>
</tbody>
</table>
Notes:

- Modules A1–A3: kgCO2e released during extraction, processing, manufacture (including prefabrication of components or elements) and transportation of materials between these processes, until the product leaves the factory gates to be taken to site. [3]
- Modules A4 and A5: kgCO2e released during transport of materials/products to site, energy usage due to activities on site (site huts, machinery use etc.) and the kgCO2e associated with the production, transportation and end of life processing of materials wasted on site. [3]
- Concrete mixes based on UK average cement mixes
- GRC formwork assumes C50/60 concrete with 2% glass fibre
- Reinforcement (B500) and pre-stressing wire (Y1860) assume UK average recycled content
- Galvanised steel is assumed hot-dip galvanised
- Expanded clay assumed for use as aggregate
- Granular fill assumes UK average aggregate
- Elastomer assumes natural rubber

Fig. 8 provides a graphical visualisation of CO2e for each structure by component.

Cumulative CO2e for the 'fat' solution was determined to be 1449 tCO2e. Cumulative CO2e for the 'lean' solution was determined to be 885 tCO2e, a saving of 564 tCO2e (38.9%). The overall percentage reduction is clearly skewed by the saving from the backfill which is very large (389 tCO2e). Nonetheless, substantial savings were also achieved in the deck, abutment and wingwalls, as highlighted in Table 2.

Table 2 CO2e savings by component

<table>
<thead>
<tr>
<th>Components</th>
<th>Fat (t CO2e)</th>
<th>Lean (t CO2e)</th>
<th>Δ (t CO2e)</th>
<th>% saving</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fill</td>
<td>444</td>
<td>55</td>
<td>389</td>
<td>87.7%</td>
</tr>
<tr>
<td>Deck</td>
<td>130</td>
<td>97</td>
<td>33</td>
<td>25.5%</td>
</tr>
<tr>
<td>Abutment</td>
<td>328</td>
<td>258</td>
<td>70</td>
<td>21.3%</td>
</tr>
<tr>
<td>Bearings</td>
<td>1</td>
<td>0</td>
<td>1</td>
<td>100.0%</td>
</tr>
<tr>
<td>Wingwall 1</td>
<td>294</td>
<td>247</td>
<td>47</td>
<td>16.1%</td>
</tr>
<tr>
<td>Wingwall 2</td>
<td>154</td>
<td>132</td>
<td>22</td>
<td>14.5%</td>
</tr>
<tr>
<td>Wingwall 3</td>
<td>27</td>
<td>26</td>
<td>1</td>
<td>2.9%</td>
</tr>
<tr>
<td>Construction</td>
<td>70</td>
<td>70</td>
<td>0</td>
<td>0.0%</td>
</tr>
<tr>
<td>Total</td>
<td>1449</td>
<td>885</td>
<td>564</td>
<td>38.9%</td>
</tr>
</tbody>
</table>
Fig. 8 provides a breakdown by *material* of CO$_2$e for each structure. The reduction in C32/40 was achieved by avoiding piles in the 'lean' solution abutments and wingwalls (-103 tCO$_2$e). Fig. 9 again clearly illustrates the profound difference between specifying expanded clay and ordinary granular material for a large fill volume.

As designers it's important we recognise the significance of design decisions and their relevance to real life. Some popular metrics of embodied carbon saving are [5]:

- 1 tCO$_2$e = one-way flight London-NYC
- 2 tCO$_2$e = going vegan for a year
- 3 tCO$_2$e = ditching the family car in favour of walking and cycling

Thus, the savings in the deck CO$_2$e alone is equivalent to going vegan for 26 years!

6 Commentary on solutions

In the 'fat' solution lightweight fill (expanded clay) was specified by the abutments. This was with good intentions – lightweight fill significantly reduces lateral loads potentially unlocking structural savings – but it has very high embodied carbon compared to conventional fill due to the high-temperature firing processes involved in its manufacture (1100-1300°C) [7]. Lightweight fill can be useful, for example for filling next to an existing structure and avoiding strengthening, but it shouldn't be the default backfilling material, especially when used to make possible a bad retaining structure concept. The enormous disparity between the CO$_2$e for the two fill materials is testament to that.

Solid slabs do have benefits, as discussed in Section 4.2, but the beam and slab option in the lean solution is clearly more efficient structurally. Unsurprisingly that results in direct material savings and the associated reduction in CO$_2$e. Using lighter steel parapets instead of concrete parapets also helped to save carbon in the deck. Use of bearings in the 'fat' solution simplified the design. The designer felt 'safer' with a solution which could be easily analysed. The 'stitch' on fully integral bridges was also considered difficult to fix. However, because the beams in the 'fat' design were simply supported this meant that the beams worked less efficiently and contributed to the higher figure for the 'fat' deck CO$_2$e.

Lack of GI triggered a kneejerk risk-averse reaction in the 'fat' solution. The designer defaulted immediately to piled foundations in the knowledge they would 'work' even if the ground were poor in some locations. In the lean solution spread foundations were specified assuming the ground to be good quality. It is assumed the contractor would then have a ‘risk pot’ if the ground were worse and redesign were required. The first option may be lower risk, but results in higher CO$_2$e. It also underlines the need for good GI. Clearly there are occasions where use of piles will be unavoidable e.g. for poor soil conditions or longer spans, but they shouldn't be the default when other options may be viable such as spread foundations or bank seats on reinforced earth.

Interestingly, the wingwalls were an area where CO$_2$e reduction was more difficult. Not using piles made a good contribution, but ultimately making savings on the cantilever walls was difficult. It was a surprise that the wingwalls contributed such a high proportion of the overall CO$_2$e: 33% for the fat
scenario and 46% for the lean scenario. To realise savings in the wingwalls, wholesale changes in form would be required, for example via using materials with lower embodied carbon such as gabions or reinforced earth. This underlines that the area in which we as designers may normally most focus is not necessarily the area in which we can make the greatest positive impact. For example, spending time refining the number of pre-stressing strands in the beams is likely to have a much lower CO2e impact than making a small adjustment to the foundations or wingwalls. This highlights the importance of conducting an embodied carbon assessment, however simplified, as early in the design as possible.

7 Overdesign

7.1 Programme and budget constraints

Pressured programmes and design fees feed into a general culture of risk aversion, encouraging tried and tested techniques, and discouraging innovation. The AIP – which is the culmination of the concept design - can often be developed under time pressure and a solution hurried out the door, with a view that efficiencies will be realised at detailed design stage. The reality is that the detailed design will often be equally time pressured and the efficiencies will not be realised. In fact, peace of mind factors may be added throughout the process, leaving ghost redundant capacity in the system. When the AIP is written the constraints for the final design get finalised. As succinctly highlighted in the MEICON report [6]; 'The role of the structural engineer must be viewed in the context of a design process. The greatest potential for influencing material efficiency is held at concept design stage. Once designs are fixed (in)efficient is locked and the role of the engineer becomes making it work, rather than making it work well.' 'Innovation' may take the form of materials or techniques normally outside the scope of the client specification, which would require a departure from standard. Departures generally need to be agreed at the time of the AIP production, thus it's important to make sure they are included at concept stage and are not an afterthought. Early engagement of contractors and suppliers are other obvious ways to help develop good concepts but consideration needs to be given that their preferred solution may not be the lowest CO2e approach.

7.2 Simplification / rationalisation

The relatively low price of steel and concrete when compared to the design and construction time is a factor which leads to waste. Low prices may be used to justify increases in material to 'simplify' or 'rationalise' the design. This may be to homogenise elements, for example choosing larger pre-cast members, or larger pile diameters throughout or to make pile spacing or gridlines equal. The design may be simplified to facilitate simplified analysis and expedite the design process. Sometimes the low carbon option may not be the easiest option to design. Sometimes the low carbon option may not be the easiest to construct. It's important overly conservative simplification does not take place simply to minimise the risk of site error - competence of the contractor's workforce should be expected and be scrutinised by the client.

7.3 Absence of good GI / SI

Naturally, uncertainty stimulates risk averse behaviour in designers. This is often compounded by the typical procurement routes and contractual frameworks on projects. As in the design example in Section 4, piles were specified instead of spread foundations because of the absence of good GI. Absence of good SI, for example boreholes through existing masonry abutments or testing of existing concrete may lead to 'belt and braces' solutions, 'eliminating' risk. This may be via making an existing structure 'redundant' which could be partially re-used. As per Section 7.1, the solutions are unlikely to change, even if better GI / SI becomes available later. Clients must recognise that absence of good GI / SI early in the design process has downstream implications, and make sure that good procurement and access strategies are in place to allow this to happen. As early as possible in the design process we should stress the benefits of good GI / SI - which may include reduced CO2e - to our clients.

7.4 Embodied carbon metrics

Clients are starting to embrace low carbon approaches, however there are still many issues with how this is managed. The importance of minimising carbon is often moved to a parallel workstream of those with specialist sustainability expertise. The ambition of minimising carbon should be embedded into the main design team. Designers should be competent in calculating CO2e and doing this as early in the
design process as possible. Many clients stipulate the use of the CG300 [8] AIP template on their projects. The template doesn’t make direct reference to embodied carbon at all. Section 3.10 'Environment and sustainability' is a generic expression which could refer to the need to manage great crested newt habitats or minimise noise pollution. Those are important considerations but shouldn't be lumped together under one banner.

How clients seek to quantify carbon savings is an area which could undoubtably be improved. Some schemes have incentives or requirements to make carbon savings stage by stage, sometimes of an unrealistic magnitude. This can encourage bad practice during the concept and 'cheating' between phases. A common example is using loose definitions of concrete mixes at feasibility stage to justify the use of GGBS as a 'saving' later. Such practice actually discourages the development of good concept development because it encourages the designer to build in fat knowing that cutting it back later will provide benefits (financial, reputational etc.). The latter stages of a project are not when CO2e benefits can be realised. Instead, clients should have a figure in mind for the embodied carbon they expect for a project. When tenders are made this information should be available to designers so it can inform on their proposals. At the optioneering stage CO2e should be at the forefront of considerations. Going forward, well thought out targets also help to orientate the contractor towards supply chains which best manage CO2e.

8 Conclusions
The following conclusions are made:

- The 'fat' design had 1449 tCO2e and the 'lean' design had 885 tCO2e.
- Regular granular fill instead of lightweight fill resulted in a very large saving (389tCO2e).
- An integral beam-slab deck with steel parapets instead of a simply supported solid slab deck with concrete parapets resulted in a significant saving (33tCO2e).
- An integral abutment with spread footing instead of a piled free standing abutment resulted in a significant saving (70tCO2e)
- Wingwall spread footings instead of piled footings resulted in a significant saving (71tCO2e).
- There are many reasons for overdesign but most stem back to the selection of a poor concept.

9 Recommendations
The following recommendations are made:

- Designers should focus efforts to minimise CO2e at the concept design stage. This is the stage at which most difference can be made.
- Designers should work with clients to set meaningful targets for CO2e on projects.
- Designers should stress to clients the importance of GI/SI in developing the concept.
- Bridge designers should not over-focus on the superstructure when trying to minimise CO2e. The substructure, approach structures, and fill are as – if not more – important.
- Lightweight fill (particularly expanded clay) should be specified judiciously – not to make bad concepts work.

References
rediscovering the past
Beyond the spherical solution: the contractor’s contribution to the roof of the Sydney Opera House

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Abstract
The history of the design decisions directly related to the construction of the Sydney Opera House remains largely anecdotal. A rich group of items recently discovered in Australia may now start filling this gap, as documents brought to light include the drawings issued by the general contractor to build the concrete formwork for the shells, drawings of the temporary structures and falsework, site images, and contractor's notes. All in all, the drawings display sophisticated combinatory solutions for attaining the structural form required whilst introducing repetition and flexibility in the making of the discrete pieces. While suggesting a remarkable combination of manufacturing and structural shrewdness, these blueprints call into question the canonical history of the building roof’s famous ‘sails’ and the rhetoric of the ‘spherical solution’ used to arrive at them.

1 Introduction
The Sydney Opera House is the object of a prodigious hagiography of the personalities involved in its realization and their legendary querelles. Publications on the building rely on the memoirs and actions of three major actors: Jørn Utzon (the architect of Stage 1 and 2, 1958-1966), Ove Arup (the structural engineer and project manager of Stage 2, 1959-1973), and Peter Hall (the architect and project manager of Stage 3, 1966-1973) [1]–[5].

By contrast, and somewhat surprisingly for a building of such legendary renown, the history of the design decisions directly related to its construction on site remains largely anecdotal, if not utterly obscured by the so-called ‘spherical solution’, namely the breakdown (or approximation) of the original roof shells into triangular sectors, eventually labelled ‘sails’, belonging to the same sphere. By contrast, the contribution of the contracting side to the engineering of the assembly process and, by extension, the detailed design of the components involved, has thus far frustrated scholarly attention, also due to the scattering of the original documentation. This is particularly so for the work produced by the general contractor for Construction Stage 2, the Australian company Hornibrook, which played a significant role in developing the construction solutions and the casting procedures for the roof shells.

A rich group of items recently discovered by the authors in several locations across the Australian state of New South Wales may now be set to shed important documentary light on the details of such role, and the collaboration it entailed particularly with the structural engineer.

The documents include site notes, new original site images, and a massive corpus of 5,300 drawings issued by the contractor, including construction layouts of the site as well as calculations and execution instructions of the temporary structures used for the erection of the building roof's famous 'sails' (fig. 1). Though technically classifiable as shop drawings, i.e., non-contract production documents describing the manufacturing process leading to the realization of the building [6], Hornibrook's sets betray strong degrees of design integration with the drawings produced by the structural engineer ARUP, which, in several cases, contain explicit references to 'Hornibrook solutions'. If this type of notational citations suggests at least an accredited combination of efforts in the project, scope and magnitude of the shop drawing series reveal the considerable endeavour of the construction company in the conceptual ordering of the physical tasks.
Fig. 1 Hornibrook Ltd, Sydney Opera House, Stage 2. On the left: construction layout indicating the location of the casting yard for the shells’ components and, highlighted in red, the different storage areas. On the right: northern precast segment storage layout used for organising the rib segments before their erection (NSW State Archive and Records).

2 The formwork system

Among the collection of drawings, 85 of them describe the formwork system developed to build the concrete shells. Examining this lot is interesting because it reveals not only the length of the engineering work on the contractor's side but also the integration of industrial fabrication thinking and ad-hoc construction concerns.

To understand the achievements, one must first introduce the problem, which in this case concerns the roof and the form of its main components, also known as the 'sails'. These have been described as a combination of 'side' and 'main' shells, with the latter taking the shape of an ogive vault formed by a series of arches labelled 'ribs'. The dominant geometry describing each half vault is a slice of a sphere with a radius of 75 meters, which gives all the ribs the same curvature and allows them to be notionally divided in concentric and repetitive segments. Each rib is formed by a variable number of segments with a Y-shaped cross section. All ribs start with three solid concrete segments, followed by a fourth one featuring a cylindrical void to reduce its weight, and the remaining ones designed as an open Y, closed at its top by means of precast cross bracings (fig. 2). Moreover, as each half vault feature a fan-like spherical shape all the ribs have a tapering section that increases from bottom (pedestal) to top (ridge). Mutual connection between segments of the same rib was assured by a stabilizing compression force obtained with prestressing cables running along the rib’s radius. As such the result post-tensioning forces were ‘nearly centroidal’ [7] with each rib being self-supporting upon its completion.

Each ogive arch or rib is completed with a special last segment acting as a connector between the rib and the ridge. Like the ribs, also the ridge of the vault is formed by a series of concentric precast elements.

Leaving instrumentally aside all the cast-in situ elements of the system (i.e., pedestal and tripods footings) and its special pieces (i.e., ridges, crowns and warped segments) allows one to focus on the formwork designed and tested by Hornibrook for the production of the rib segments.

As explained in the drawings, each formwork accommodated five contiguous segments. Each segment was separated by a precast bulkhead which, besides working as a formwork diaphragm, also acted as a matrix for the positioning of the spigots and the anchor plates that had to be embedded in each segment. Moreover, to assure the necessary geometrical continuity between segments, the segment last poured in the previous formwork was positioned as first in the following one (fig. 3).
Fig. 2  Explanatory diagrams showing the different types of shells and their components. On the left the typical cross-section of a main shell with the axonometric view of a rib-block with an open Y-shape section.

Formworks were made out of two moulds: an exterior one and an inner one (fig. 4). The form of the exterior one was shaped against the Y cross-section of the ribs. It was divided in two shells that could be closed and opened via a rail sliding system actioned by hydraulic cylinders placed at the base of the shells. The shells, built with a light frame structure in steel studs and plywood lining, were completed with a cast-in-situ curved spine, running along the centre-line of the rib that realized the base form of the Y stem. Once the formwork was stripped from the segment, the central spine acted as temporary support for the piece itself before it got lifted by the crane. For the fabrication of the steel inner forms, Hornibrook designed a special timber jig with two adjustable horizontal arms through which it was possible to set out the interior tapering geometry of each segment, necessary to follow the varying cross section resulting from the discretisation of the sphere in slices (fig. 5). Once realized, the inner form needed to be adjusted and modified so as to allow the insertion of pockets and corbels for stressing anchors, bolts and other permanent connections. Original shop drawings show a series of so-called “modification to inside formwork” alternatives, which illustrate and detail the numerous construction variations required or imagined.

Fig. 3  Construction sequence showing the Hornibrook formwork system used to manufacture rib segments with Y-shape cross-section (photographer: Max Dupain and Associates. Records and negative archive: un-commissioned Sydney Opera House construction photographs, 1965-1972. Courtesy: NSW State Library).
Fig. 4  Hornibrook Ltd, Sydney Opera House, Stage 2. Rib Segment Formwork, Section, Frames, Detail, Segments from 1 to 5 (NSW Archive and Records).

Fig. 5  Hornibrook Ltd, Sydney Opera House, Stage 2. Special timber jig with two adjustable arms for moulding the interior formwork according to the tapering of the rib cross-section (NSW Archive and Records).
In synthesis, the drawings for the formwork articulate sets of sophisticated combinatory solutions for attaining the structural form required whilst introducing repetition and flexibility in the manufacturing of the discrete pieces. In order to do so, their producers had to consider the vertical layering of segment sub-pieces across the Y section of the rib as well as the tapered progression of the segments along the curve of the half arch, which was made possible by the introduction of sliding registers into the idea of the form. All this without losing sight of the limited, narrow space available to organise a casting yard around the footprint of the building, in itself demanding a high rate of reuse of the moulds, as well as stockage locations for the segments awaiting erection (fig. 6). Shape, length and functioning of the formwork, in other words, had to respond to architectural ambitions, structural engineering requirements, manufacturing precision and speed, site logistics, and economy of materials.

Fig. 6 The Sydney Opera House construction site (photographer: Max Dupain and Associates. Records and negative archive: un-commissioned Sydney Opera House construction photographs, 1965-1972. Courtesy: NSW State Library).

Such challenges acquire significance against the celebrated 'spherical solution' for the roof, which is by now part of architecture's modern history. On the one hand, the 'spherical solution' allowed for a conceptual macro-discretization of the sails into ribs, and for envisioning the production of the latter through a nearly industrial process. Yet, on the other hand, at the 'segment' scale, it could not foresee and solve all the engineering issues embedded in the very solution, which remained open for the construction of a roof constituted by over 2,400 precast segments, the majority of which required an ad hoc precast bulkhead, precast cross bracing, and specific adjustments to accommodate all the necessary post-tensioning apparatuses.

Such degree of detailing required the structural engineer ARUP to issue 30,000 dimensions to Hornibrook – dimensions that were promptly translated by the contractor into detail drawings often supplemented with data tables indicating variables dimensions and locations of single details. Those dimensions were generated by a system of coordinates based on the spherical configuration which was also at the base of the surveying criteria adopted for controlling both the casting yard (including the formworks) and the erection of the roof [8].

3 Contractor’s agency in the project

Even such a short analysis of the construction of the formworks for the structural segments of the sails enables a series of considerations on the work conducted as well as the process that led to it. Firstly, it
shows the enormous amount of product engineering and operational planning that went into the definition of the catalogue of components and their casting procedures. Whilst responding to the building performance requirements set by the architect and the engineer, the general contractor made strategic decisions concerning sub-component geometries and combinations, moulding systems and fabrication sequences, element re-use patterns and bespoke requirements. Type and extent of the documentation produced, together with the photographic records of the operations on site, betray the significant degree of autonomy enjoyed and exploited to this end. While the geometry of the precast components of the arches suggests differences with the streamlined aesthetics of the architectural surface of the sails, it does respond very well to both the production-related needs for modular yet flexible casting on a difficult site and the extreme complexity embedded in the task of recomposing all the pieces of the three-dimensional structural puzzle.

Hornibrook's successful search for manufacturing efficiency and assembly viability suggests that the elements of interest in the construction of the sails go beyond the definition of their overall form and the methods employed to extrude its surface in layers. Indeed, they include the composition of its discrete precast pieces and the process of manufacturing them. This because it was the set of decisions underpinning such a process that determined not only the layout and the organization of the site but also structured construction operations and quality assurance methods for critical parts of the project and important portions of its duration.

The casting of the formworks thus bears testimony to the existence of 'agency' functions on the general contractor's side, requiring vision and the ability to enforce it. As an inevitable aside, due to space limitations of this paper, it could be important to reflect on the fact that, if Hornibrook's experience and track record to this point of its history had produced a kind of manufacturing shrewdness capable to respond to the challenges thrown at them by the official professional design team, the company's actual ability to do so on the Sydney Opera House was determined contractually, by the provisions explicitly regulating work boundaries and expectations of the builder during Stage 2.

4 Design or translation of intent?

Irrespective of the importance of contracts in enabling critical contributions to project developments, did the work of Hornibrook as described amount to 'design', or did it embody the mere translation of design intent into instructions for production, as per the conventionally accepted nature of shop drawings?

If one looked at the image of the building and the compositional logics of its structural system as a whole, then the answer to the design question would be negative. By servicing a higher order concept - that of the form and the structure of the sails - the contractor's documentation and the work instructed within it would be subordinate to these main ends; as such, they would not constitute design per se. Yet, if one considered design almost etymologically - as "a problem-defining, problem-solving, information-structuring activity that, on the basis of understood conditions and rules, defined specific courses of action" - then the casting of the formworks would attain full design status.

In fact, when sketched in these terms, design activity would not be limited to what definable solely under architecture prescriptions or structural engineering work, but rather enter all the specific dimensions of the building procurement process - including at least site layout, building components production, building erection, and building use and maintenance. Such scenario would shape the idea of both 'building' and 'project' in scholarly useful ways, with 'building' becoming understood as the combined result of the implementation of multiple scope-specific designs; and 'project' indicating the social space where the gradual integration of these designs would occur, following a process of negotiation between objectives internal to each design dimension and objectives related to their integration - very much the case with the work carried out by Hornibrook on the Sydney Opera House [9].

5 Design as a broad construct

Opening the notion of design up in the way just outlined makes it plausible to turn established mental images of construction around and think of the building process, with all its ramifications, as a system of design production independent of corporative schemata - a cycle, that is, within which all the information necessary for the implementation of the building would have to be conceived and either produced or assembled. How this system organized to deliver its product, what logics it followed in doing it, what it would be constrained by, and how many units of production it would consist of would then
become the object of the discussion. Such a conceptual framework would add critical dimensions to the analysis of the design process and its dynamics, certainly by positing the importance of socio-technical diversity within the project team, and with it the relevance of sophisticated actor-networks descriptions across the history of the project [10].

Analytically, the design system of sorts determined through this exercise would be helpful for two reasons. Firstly, because it would provide a proper index of the design challenges that exist within the building process, and a measure of the substantive breadth the design task must gain to respond to them. Secondly, because it would help form a view of the building project not tied a priori to specific actors but rather open to the recording of direct or indirect design contributions, to qualify in relation to the areas of impact. By creating the conditions for isolating and then bringing together the work conducted on disparate design domains by clusters of contributors, such a multi-dimensional view of design could be used as a tool to interrogate project challenges and results, eventually to intervene on the dynamics that led to them.

The authors have a research funding application pending almost exactly on this topic in Sydney. Hopefully, it will be possible to make more than informed guesses on the efficacy of such analytical methods before too long.

Yet, the importance of the difference just articulated, essentially between ‘design as product’ and ‘design as process’, can be gauged effectively by returning to the sails of the building and the rhetoric surrounding their creation. While their canonical history celebrates the so-called ‘spherical solution’ as a stroke of genius on the side of the architect and the engineer, the story of the works put together by the contractor for their fabrication on-site tells a tale of work planning and ingenuity that counterbalances the myth of the ‘eureka’ moment by highlighting the amount of labour – intellectual as well as physical – required to make Utzon’s great idea materialize (fig. 7).

Fig. 7 On the right: The rib formwork system located on the eastern side of the casting yard (photographer: Max Dupain and Associates. Records and negative archive: un-commissioned Sydney Opera House construction photographs, 1965-1972. Courtesy: NSW State Library). On the left: Hornibrook Ltd, loading diagrams for the member of the erection arch (NSW State Archive and Records).

Without taking anything away from the leap of imagination that led to the solution eventually employed, the actual construction of the sails owes a huge debt to the preparatory design and engineering work by the general contractor. Indeed, the spherical solution generated a series of significant construction chain challenges, from task identification to site planning, system engineering to visualization of decisions, work monitoring to quality control, which were all tackled by the main party ‘on the ground’. Hornibrook did overcome the technical issues posed by the fabrication of all the parts required through the production of copious, detailed documentation based on and refined via a long period of prototyping work, which would be difficult to liquidate as mere, although remarkable, construction management. If such documentation will necessarily remain a critical object of analysis and reflection in future studies on the building and the meanders of the technical design process, a provisional conclusion can be attempted on fairly safe grounds: for a building justly considered unique and out of time – and as such worthy of world heritage status [11] – the mundane aspects of its realization and the design challenges
these raised for the industry at the time may well constitute the true gauge of its ‘concrete’ achieve-
ments.

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An ambiguous order: the structural concept and design of Juha Leiviskä

Fangjie Xie, Toni Kotnik

Abstract
Together with Alvar Aalto and Reima Pietilä, Juha Leiviskä is considered as one of the main protagonists of Finnish Modern Architecture. What characterizes the work of Leiviskä and makes it unique is the specific use of daylight. His light is an oblique light that both hits vertical surfaces directly and is reflected to create experiences of layered light with a distinct feeling of depth. It does not only illuminate surfaces, it appears to originate and exist vibrantly in the architectural space itself. And the building structure plays a significant role in this use of daylight. It makes it possible, the structural order is woven together with light and shadow of space. This paper explores the use of structure in the work of Juha Leiviskä in more detail. According to Leiviskä "architecture is closer to music then to visual arts" and it is argued that specific techniques from piano playing have been transferred into compositional principles of structural elements resulting in an exclusive two-layer beam system. This way of working and designing and the holistic logic of structural design is illustrated by a series of case studies from various phases of the career of Leiviskä.

1 Introduction
In 1917 Finland gained its independence. Architecture in Finland became a part of the large-scale national project of “the search for Finnish self-identity” [1]. Finnish architects, like the pioneers of the European Modernist movement, studied with great enthusiasm the technologies of the first machine age. The distinctiveness of Finnish Modernism lies in the relationship of parity between the architect and nature. In an article on Finnish architecture, Siegfried Gideon remarked that “Finland is with Aalto wherever he goes” [2]. For Gideon, the celebrated pavilion designed by the architect for the international exhibition in New York, symbolized the duality that was destined to mark the development of contemporary architecture in Finland in the 20th century: from humanism to materialism, from a functional to a picturesque approach, from judiciousness to emotionality.

The Golden Generation of figures [1] like Alvar Aalto or Reima Pietilä adopted from the Modernist movement and the International Style primarily those methods that accorded with their national mentality and tradition. Perhaps it is due to this latter circumstance that the phenomenon of Finnish Modernism arose: it is a fairly pragmatic and introverted school, always following home-grown traditions, oriented on local materials and climatic peculiarities.

Surely this is the case for the Finnish architect Juha Leiviskä (1936-), within the Finnish discourse widely considered the successor of Alvar Aalto and Reima Pietilä [4]. Leiviskä is especially known for his church architecture and other sacral buildings, all characterized by a specific use of daylight. In modern architecture, the use of daylight is usually based on variations of zenithally light or narrow roof or wall slits to guide light along the surfaces of the architectural space. In contrast, Leiviskä’s light is an oblique light that both hits vertical surfaces directly and is reflected to create experiences of layered light with a distinct feeling of depth. It does not only illuminate surfaces; it appears to originate and exist vibrantly in the architectural space itself.

A similar use of oblique light can be seen with other Nordic masters like Jørn Utzon or Sverre Fehn. Utzon and Fehn, however, used indirect lightning along surfaces primarily for dematerialization and atmospheric rendering. Leiviskä, on the other hand, considered light as a material on its own that is overlayed into space resulting, from judiciousness to emotionality.

For the architecture theorist Malcom Quantrill, the specific quality of light in Leiviskä’s work has its origin in a conversion of de Stijl’s “perspectival compositions and focal planes into shuddering, shimmering surfaces that translate the abstractly static elements of early Modernism into a lively and
truly ethereal syncopation of formal relationships” [4]. Leiviskä has admitted the influence of the Dutch de Stijl movement on his design thinking. But the de Stijl-movement had no constructional imperative, no *techne*. “Without the benefit of material weight, de Stijl drawings and ‘constructions’ appear to float in space. Not so the architectural constructs of Leiviskä, which seem to occupy a no-man’s land in modern architecture, a zone poised between the *material* and the *immaterial*” [4].

This paper explores in more depth the “constructional imperative” in the work of Leiviskä that is the role of structure within the design thinking of Juha Leiviskä. The purpose of this exploration is to illustrate the ‘structural mechanism’ that governs the weaving of light, shadow and space in a two-step approach: first, illustrating the establishing of the ‘structural mechanism’ at the beginning of his career, second, the variation of this mechanism as basic theme within subsequent projects.

2 Dual-Layer System

In 1963, Juha Leiviskä graduated from Helsinki University of Technology, and soon after started collaborating with Bertel Saarnio on a number of architecture competitions. Already one of their first competition entries for the Kouvola Town Hall was successful and helped to establish the own office in 1967 (Fig. 1). As breakthrough project, the Kouvola Town Hall is listed in every retrospective since then. At the same time, it has never attracted special attention and detailed reflection. The reason for this might be that the project clearly relates to modern Finnish architecture represented by Alvar Aalto like the Stora Enso Headquarter in Helsinki, completed only short before the competition for the town hall (Fig. 2).

Fig. 1 Juha Leiviskä, Town Hall, Kouvola, Finland, 1964-1968

Similar to Aalto’s design, the overall layout of the town hall of Leiviskä is based on a rigid grid. However, in contrast to the Stora Enso, Leiviskä’s design is not a monolithic block but the main volume of the town hall subdivided into two functional units and an additional administrative wing rotated away in order to adjust to the site and define a semi-public outdoor space in-between [3]. The plan, thus, is already illustrating the use of basic operations: the overcoming of the rigidity of the grid by means of splitting and layering of multiple grids. Although Leiviskä’s hallmark poetic use of light is still absent, his basic design method of a *dual-layer system* is emerging already and with it the fundamental principle for organization of structural systems evident in his later projects. A system already distinct from the design principles of Aalto and other Finnish contemporaries.
This difference is especially visible in the detailing of the façade. At first glance, the gridded façade system of the town hall resembles Alvar Aalto’s organization of the façade of the Stora Enso. But Aalto breaks the abstract rationality of the grid by encasing it with slanted plates out of Carrara marble. This results in a deep, almost corporal façade with a rich play of light and shadow. The window frames disappear behind this thickened grid and the glassing seem to be attached at the frame. The abstractness of the grid is further reduced by Aalto by framing of the grid at the first floor and the roof edge thereby mirroring a classical ternary partition of the façade.

In contrast to the plasticity of Aalto’s design, Leiviskä does not attempt to smooth out the abstractness of the organizational grid. Instead, he keeps the purity of the grid, the repetitiveness is not broken by any partitioning of the façade like in Aalto’s case. Instead, the depth of the floors and the framing of the windows is used as visible second layer that provides additional rhythm and depth to the façade. The same layering approach is also visible on the ground level. Aalto underlines his ternary partition of the façade by a deep recess of the entrance resulting in ground level disappearing in the shadow. Leiviskä, on the other hand, articulates the ground level shifting the second layer inwards, changing its materiality and the subdivision of the windows.

The treatment of the façade illustrates how Leiviskä relates and learns from existing precedence and is able to express and articulate the architecture through a design language of layering and rhythmic variation, a dual-layer system of interacting elements.

3 Musical Order

This thinking in layers and rhythms has its origin in music. In young age, Leiviskä wanted to be a musician. Playing piano and studying harmonic systems have impassioned him since childhood and continued to this day. This passion has fuelled Leiviskä’s believe that music and architecture share certain principles: "To me, architecture and music are the arts which are closest to each other. They are the same thing spoken in different tongues. The aim in both is to create from human dimensions space to be experienced by people… in the boundless universe that is way beyond our comprehension." [4]

This interplay of music and architecture is especially visible in the southern German Baroque to which Leiviskä was introduced during his study time by Nils Erik Wickberg [3]. During late Baroque, church architecture devised internal shapes so distinctive as to intensify the sound. At the same time, the concurrent renewal of piano technology made multi-layered harmonic experimentation possible enabling musicians like Johann Sebastian Bach, a composer repeatedly saluted by the Leiviskä, to explore new and complex harmonic structures in counterpoints that originated from medieval times.

According to Leiviskä "counterpoint is … ‘a point at the point of a point’. It aims at harmonizing individual sounds at every point; that is, they are together, but independent. The result intended is a rich texture full of tension" [5]. A tension created by "interactions, small and large, light and shade, soft and loud, intimate and monumental … There are ruling climaxes and pauses, thematic elements and their development. There is an immense number of delicate, ever changing nuances" [4].

For a musician, the experience of the counterpoint as an interplay of the left and the right hand in the piano performance is a spatial one: "… on the piano, I realised how to achieve some kind of continuity, how to join things together and not just produce separate sounds… Independent of the instrument, music is the construction and experience of spatial continuums. It’s the same with architecture "[5]. It is in this experience of the counterpoint that Leiviskä can claim that music and architecture are the same thing spoken in different tongues. And it is here where the lesson of the counterpoint transforms into architectural language of the dual-layer system, a system of harmonizing individual rhythms that brings them “together, but independent” like in the façade of the town hall in Kouvol
down into several volumes without questioning the unity of the building. Like multiple voices in a musical score, the family of volumes forms a unity coordinated by the underlying grid. At the same time, each volume has its own identity, defined by shifts in the horizontal and vertical by one-quarter or one-eights of a unit resulting in a complex denticulation of the volumes with plenty of opportunities for natural light to enter into the depth of space.

Fig. 3  Juha Leiviskä, Parish Center, Nakkila, Finland, 1968-70

In the basic layout of the parish centre, the concept of the counterpoint transfers from music into space with light as medium of articulation. The design, thus, offers Leiviskä an opportunity to incorporate his appreciation for German Baroque church architecture. Repeatedly he recalled the deep impression he received when visiting the Pilgrimage Church of Wies in southern Bavaria for the first time. According to Nils Erik Wickberg “never before or since has architecture, sculpture and painting merged into such an integral whole as in the Bavarian Rococo… it can be called a Gesamtkunstwerk, a total work of art… But all this inexhaustible wealth is ultimately just an instrument for the play of light. Everything has been designed by an infinitely inexhaustible ingenuity, such that it captures, scatters and composes light” [3].

This experience started a lifelong pursuit to represent a similar quality of light and space in his own designs ever since: "The firm conviction was born in on me then that I was going to have to achieve the same kind of intensity in my own work, if I ever got any" [1]. In studying the construction of several Baroque churches in more detail, Leiviskä realized that "the church is comprised of two nested oval shells…an infinite number of indirect reflections of light of different degrees has been created, generating a joyous state of being that constantly changes in accordance with the time of the day" [4]. Leiviskä took full advantage of this learning, as he expressed it late in his career: " I have some idea of how light behaves, of how it can be made to vibrate…The spaces are designed to produce reflections and reflections of reflections, or after-reflection" [5].

Fig. 4  Juha Leiviskä, Parish Center, Nakkila, Finland, 1968-70, structural model

In the design of the perish hall this learning is visible in an activation of the structural system for the modulation of light and its reflection especially in the opening of the fifth façade. Similar to the doubling of shells in the church, the structural system is doubled up and the double-layer system of the
Kouvola Town Hall re-emerges as double-layer structural grid in order to control light into the clerestory direction. Leiviskä completely sliced the structural system into the lower, primary and the upper, secondary parts ((Table 1, 5-0), respectively adopting concrete and slim wooden frames, while sharing one modular control system. The main gridded concrete frame functions as unifying system, binding together the various parts of the building and providing a basic support. The secondary gridded timber frame enables flexible modulation and differentiation of the ceiling and skylight. Due to the separation of the two in height, Leiviskä gained a certain degree of freedom to deal with the amount of light without losing control of the whole due to the presence of the main grid system.

In the parish centre, this double-layered structure enables the adjusting of the instrument of light. Light undergoes multiple layers of reflections amid the white matte surfaces of the walls, frames and louvers and distribution in space. The resulting difference in the luminosity causes a continuous change in the atmosphere of the space (Fig. 5) and a flow between them. A "continuous changing, shimmering veil of light" [5] is woven. With the double-layered structure, Leiviskä also has established his main method of design motivated by the principle of counterpoint in music and his interest in the control of light in Baroque churches.

5 Variations

Leiviskä’s dual-layer system cannot be considered as being complex, neither from a structural perspective nor from a design methodological. But it is within this non-complexity that allows for a high degree of freedom and enables extraordinary spatial potentials. Being able to move components in all directions offers possibilities to the architect for the simultaneous control of light, sound, and movement of people.

Over a period of three decades after the completion of Nakkila Parish Centre, Leiviskä continued the use of the dual-layer structural system in the design of a series of churches. Like a musician he was exploring variations of this basic theme and resulting expression of light in space. Although specific with respect to site and context, the churches share the basic design principle of the dual-layer structural system as generator of the design that shapes the space and the atmosphere. The Leivikä’s serial explorations resembles a switching on and off of tones and variation of rhythms to activate various spaces and atmospheres within the same order resulting in different light conditions and spatial themes.

5.1 Variation a: Offset

Referring to the two main directionalities within the context of St Thomas’s church in Oulu (1971-75), Leiviskä worked with two layers (Table 1a): one large-scale, monumental layer that forms a cross-shaped spine directed towards the public areas and the city (northeast side, facing the altar on the left); the other gridded layer scaled more intimate, small-scale as an open ‘front’ onto the fenced yards by the park (southwest side, facing the altar on the right). [3]

Structurally, these two gridded layers of various scale are organized like the dual-layer system of the Nakkila Parish Centre. The monumental layer consists of two intersecting portal frames by 90 degrees as the main frame at the lower level, then joining the northwestern high wall to bear the upper secondary frames on the right hand. By offsetting the intersection closer to the left, he endows the latter with the freedom of variation while reducing its span so as to enable cantilevers to opens up the left elevation to the natural light and the birch woods outside. While the secondary frames meet the load-
### Table 1  Four variations of the dual-layer system by Juha Leiviskä

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<th>Basic principle: Parish Centre, Nakkila, Finland, 1968-70</th>
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- **a) Offset**: St. Thomas’s Church, Oulu, Finland, 1971-75
  - ![Diagram](image) ![Image](image) ![Image](image)

- **b) Vibration**: Parish Church & Centre, Myyrmäki, Finland, 1980-84
  - ![Diagram](image) ![Image](image) ![Image](image)

- **c) Jump**: Parish Center, Kirkkonumm, Finland, 1980-84
  - ![Diagram](image) ![Image](image) ![Image](image)

- **d) ???: Church of Good Shepherd, Pakila, Finland, 1997-2003**
  - ![Diagram](image) ![Image](image) ![Image](image)
bearing walls on the right hand, the continuous skylights between the frames illuminate the wall as well as the altar.

By entering the left high side windows, the strong south-facing sunlight first softened by the birch trees and the frosted glass, then diffuses into the space and leaves vibrant tree-like shadows on the glass, contrasting with the directly projected sunshine on the wall from the skylight on the right. Two different temperatures and different luminance complement each other at different heights in the main congregation space. With the sun changing throughout the day and the seasons, a continuous change of light is achieved that results in a vibrant indoor space.

5.2 Variation b: Vibration

Like St Thomas’s church in Oulu, the Parish Church of Myyrmäki (1980-84) also has a two-sided context. Leiviskä hoped to preserve more of the site's original birch trees on the northeast and screen the southwest elevated train station and high-rise offices by adopting a sequential back-belly layout [3]. And to keep in line with the research on sound effect at the time, a challenging short-axis pattern of hall is adopted. The back is an entire high wall set up in the southwest side (the alter side) to isolate sound and images, and the belly a dynamically changing series of combined small volumes that create a fragile and sensitive human scale and atmosphere towards the birch woods.

To deal with the challenge, Leiviskä tried to vary the dual-layer beam system’s lower part to enrich the spatial experience by creating more layers along the short axis (Table 1b). Then for the higher secondary part, he applied a regular system of parallel-rowed beams so homogeneous as to endow the lower main frames with freedom of variation, which are embodied a series of T-beam. Reminiscent of the fugue composition, the T-beam arrangement brings out direction and dynamism to interact with the birch woods.

Due to the presence of the upper beam, the main beam could be set back from the exterior wall, thus adding a cavity both flavouring the layers in the short axis and providing a reflective and reverberant space for the interior sound transmission to a satisfactory musical effect. A free veil-like curtain wall with multi-layered louvers is introduced behind the T-beam order, letting the trees-through sunlight gently illuminate the entire hall. A continuous skylight is opened between the frames above the altar to give another layer. Through the curtain, the light flows upward to the skylight overhead the altar, converging the fluidity along the axis and even extending further outside. While guided along the axis by the gentle light, the visitor’s spatial experience is a multiple one.

5.3 Variation c: Jump

The Kirkkonumm Parish Center(1980-84) is an extension of the original church completed in 1970. The overall layout is a long articulated “ashlar” that frames the hill. The room arrangement evokes the ancient monasteries in Southern and Eastern Europe, where a compact long horizontal block is rhythmically divided by vertical towers, belfries and many a cupola [3].

In the design of the hall, Leiviskä needed to consider a ternary division in different sizes for temporal use. In order to give the hall, the right scale and space in both separation and combination, the architect designed a changing ceiling and introduced a series of discrete windows that features a jumping dynamic along the axis.

In the parish centre, Leiviskä continues to adopt the dual-layer system with east-west parallel beams below and north-south ones above (Table 1c). Jumping side windows made strong light-dark contrast to render the long wall more dynamic not only on its side. At the transition of the height differences, the small cylinders used in Myyrmäki and Oulu are reinforced to an effect of complete detachment, so as to complement the ‘clerestory’ for the jumping lighting dynamics under.

5.4 Variation d: Veil

In the Church of Good Shepherd in Pakila (1997-03), Leiviskä didn’t try to vibrate the whole space by light as before. The dual-layer beam system remains there, but it shrinks to a corner of the building to focus on the altar, which is decorated only with a modest "veil of light" [5].

On the one hand, the primary beam set in the east-west direction frees up the space on the south side, making the free arrangement of short walls and shading panels possible; on the other, it visually obscures the intersection of the secondary beams with the vertical components (Table 1d). Jointly effected under the side high windows in the southeast and the opening to the south, the solidity of some of the altar components almost disappears, giving way to the domain of light and shadow.
In this building, Leiviskä no longer pursues complex spatial levels and directional changes; instead, he focuses on the use of light to make a no man’s land by breaking down the boundaries between inside and outside, between reality and illusion. Common altar decorations, such as fabrics and paintings, are removed. Instead, Leiviskä firstly "dyed" the light in collaboration with an artist, Markku Pääkkönen, by colouring some of the slab backs that reflect the sunlight colourfully, and applying special glass strips that tend to form light spots inside. And without the natural light, such ensemble wouldn’t be present. Under the ever-changing natural light of Finland, the entire south façade acts like a "veil" to enclose the quiet and ever-flowing space and atmosphere of the interior. Here, all that matters are time. The architect rang the coda of his church design with this highly ambiguous approach.

6 Conclusion

As Juha Leiviskä has summarized his work himself ones: “I have the feeling I keep on drawing the same building.” This is surely true with respect to the series of church buildings that he was able to design over a period of three decades. In this series he has been experimenting with variations of a structural principal or organization, the dual-layer beam system inspired by his passion for music and his fascination with German Baroque churches. From the start of his career this principal is present, first in plan but soon as primary, structural order.

Although embodied in the form of structure, Leiviskä’s order shows little quest for structural efficiency in a common understanding. Rather, his structures are conceptual frames for light to be guided into space. It’s the attraction of space, not performance of structure, that called out Leiviskä’s conception and practice of his order. His structure is not a means to support or complete the space, but part of the interwoven light and shadow of spaces. One could even say, the structure here is space. There is no doubt that Leivikä’s thinking about structure is idiosyncratic: though not exactly in the sense of a master of structural engineering, he is well-deservedly a master of tectonic design.

With this tectonic utilization of structure, Leiviskä also provides the constructional imperative, the teche that the architecture theorist Malcom Quantrill was trying to detect in his comparison of Leiviskä’s work with de Stijl. But this study illustrates that the continuation of Finnish modern architecture is not so much grounded in direct translation of a modernistic manifesto into the Nordic context but rather personal amalgamate of Modernism and Baroque, of architecture and music, of space and light. With the help of the structural system, Leivikä was able to explore in depth the effects of light. In his design, the light not only illuminates the surfaces, it appears to originate and exist vibrantly in the architectural space itself.

With his masterly use of light, Juha Leiviskä has to be seen in a row with other Nordic master who loosened their building from the formal rigidity of Modernism, and sought instead to naturalize simple volumes by suffusing them with a light distinct to the North [9]. With his non-structural conception of structure, Leiviskä modulates the distinctive light of the North, and in doing so directs its appeal to the poetic imagination rather than the rational mind. A conception of structure beyond the rationality of performance.

References

Form and forces of dry stone trulli and its influence on contemporary structural design

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Abstract
Against the background of scarce resources, this research is motivated by the understanding of structural forces inherent in corbelled constructions which have survived centuries. Using the dry stone Trulli of Alberobello as a case study, the goal will be to establish criteria which can be used in the design of contemporary structures. The criteria are based on the methods of construction, material knowledge and the understanding of structural forces which are built on tacit knowledge and are intuitively derived. This can directly influence contemporary structural design practices and workflows where this knowledge is integrated in early design phases.

1 Introduction
The use of modular spatial structures in the construction industry has the potential to shape the design and fabrication process, due to efficient assembly and disassembly against the background of scarce resources. Dry joint connections in the construction industry have gained acceptance for their versatility and reduced labor costs in comparison to traditional brick and mortar methods. The ease of assembly and disassembly make using connecting blocks for spatial assemblies appropriate for temporary and unstable environments. This research investigates past and present practitioners and academics in the field of architectural and structural design as a base to establish connections between structural design, material efficiency and history of construction knowledge. Utilizing modular methods for the design and construction of dome and cantilevering structures.

Using the ‘Trulli’ structures of Alberobello as a case study demonstrates the existence of dry-stone construction methods which have survived centuries. Here a specific method of corbelling combined with an intuitive understanding of structural forces was adopted as a construction method where the economic conditions of the time required temporary solutions for housing. The question arises as to how much intuitive knowledge of forces was gained through trial and error techniques of working with the material, as opposed to knowledge which is passed down through the generations relating to corbelled construction techniques. Perhaps this tacit knowledge is inherent in the community and influenced by the cultural and economic circumstances of the time. How much of this knowledge is attained and developed for more advanced structural conditions with cantilevers? Contemporary researchers have used computational design and structural analysis tools to assist in understanding these forces. Specifically in relation to the Trulli structures, researchers including Todisco et al, 2017 [1] 2016 [2], Foti et al, 2016 [3], Stigliano, Fallacara, 2012 [4], Fradosio et al, 2019 [5] and Cardinale et al, 2011[6] have discussed aspects related to the structural design, analysis, context and even the experience and thermal qualities of the structure. The following will describe the dry stone Trulli, the structural understanding, the experience of working with the material and how this understanding of forces influences contemporary design workflows.

2 Trulli Structures of Alberobello
The ‘Trulli’ is a vernacular building typology which continues to be of great interest to contemporary researchers. The case study of the ‘Trulli’ houses of Alberobello demonstrates the existence of dry-
stone construction methods which have survived centuries and still being used. The ‘trulli’ of Alberobello (Fig. 1) built in the 16th century are originally mortar free structures located in the Puglia region in the province of Bari, South East Italy. During a study visit the district of Rioni Monti was investigated which comprise entirely of trulli. Within the grouping of individual cells, three possible types of aggregation were identified: the housing unit developed parallel to the road, orthogonally, or in both directions. “The Rione Monti district of Alberobello, separated from the rest of the village, is the largest settlement of the city of trulli (1,030 trulli); built on a considerable slope, it opens fan-like towards the hill and extends for 15 hectares. The Rione Aja Piccola district, with its 590 buildings, is the second largest settlement of trulli. Its houses converge towards a common farmyard, where up until the end of the feudal age farmers were required to thresh grain publicly.”[7]

According to Dipasquale and Silva (2009) the trulli can be classified depending on the shape of the dome. Each architectural typology “presents many variations in size, number of steps, presence and shape of the outer stairs, shape of the internal plan (circular, pseudocircular, quadrangular), and shape of the base profile (circular, oval, quadrangular with rounded corners or square corners).”

The simplest type is the primitive dome with an ogival profile which operates as a shelter. The walls are habitually low and thick, as well as a circular plan devoid of openings, with the exception of a small and very low entrance with a trilithic arch. To improve internal use, restructuring the space and the area-volume ratio, over time, the shape of the ground plan, both internal and external, advanced toward the square base. To protect from the cold winter winds coming from the north, the entrance is generally oriented towards the southeast. The only furniture elements are niches cut into the wall to store objects, within these rural buildings. Dipasquale and Silva (2009) further describe that, “In the Itria Valley we can see the evolution from the rural type to a newer, less rustic type and one more suitable to permanent housing: to the central core of an almost square shape smaller rooms are added, the alcove and the small trulli that generally contain the niches for beds and fireplaces, connected to the main trullo by semicircular or segmental arches.”

2.1.1 Method of Construction

This is a unique form of architecture named ‘Trullo’ derived from the Greek word ‘Thulos’ meaning dome shaped construction. It is believed to be influenced by the Harran, an ancient city in Mesopotamia (approx. 3rd millennium BC). Other indirect influences come from Mycenean architecture such as the ‘Treasury of Atreus’. Located in Mycenae, Greece, the ‘Treasury of Atreus’ [8] built in approximately 1250 BC where corbelled arches are developed as part of the interior; the beginnings of advanced dry-stone construction methods. Part of this underground structure is also in the shape of a ‘beehive tomb’ or ‘thulos’ dome which bear resemblance to the dry stone ‘trulli’ houses of Alberobello, that were built much later around 1500 AD. A ‘thulus’ is a circular chamber tomb covered by a masonry dome realised through “the successive placement of cantilevered stone elements covered by a mound of earth and
rubble stones at the extrados.” [9] The chamber room is obtained through excavation of a circular pit along a hill slope.

The construction is built ‘a secco,’ meaning mortar-less construction using the abundant cheap limestone in the area. The uniqueness of the Apulian Trulli is due to various economic, cultural and social reasons which have survived the ages to become one of the most advanced dry-stone building techniques. In 1996 it became a UNESCO world heritage site.[10] It is believed that because of the complexities of building approvals with the King of Naples who ruled the area; the local landlord Gian Girolamo Acquaviva d’Aragona ordered the local farmers of the houses to build in such a way that it can be dismantled easily in the case of a royal inspection. As shown in Fig. 2 a vertical section of a dome and spatial structure comprises of three main parts. According to Todisco et al, (2016); stones with a structural function (candela), an insulating filler and the masonry cladding (chiancarelle) also known as stone shingles. “The latter layer is composed of small stones required for rainwater defence and structural protection against weathering. Only the first layer (candela) is responsible for sustaining the structure; the other two layers can be considered non-structural.”

The space between the two walls as shown in Fig. 2 is filled with waste and rubble. During the construction of the wall, the possible windows and the alcoves with vaults are cut into the thickness of the wall. The small openings for windows are mounted by a stone lintel and the door (Fig. 3) is made with a double lintel, finished partially of stone and wood, and often matched by an overlying semicircular discharge arch.

The construction procedure of the dome is determined by the size of the stone available. When the site has initially been chosen, the master builder or farmer draws the plan directly on the ground. This technique not only has reference to the ancient Greek temple construction, but also modern 21st Century building practices. The subsequent process involves creating the ground plane and the floor, removing the layer of soil covering the limestone then building the wall. This consists of two parallel rows of
large stones, which support the adjacent stones, creating a ‘double’ wall. The wall is the compacted by laying the individual stones, to form a level slat side, where the joints are then offset, then the gaps are filled with slivers of stone. Based on the availability of stone sizes, the space between the two walls is complete of irregular stones. Generally, the foundation becomes wider with a solid base of 40cm high which then becomes a seat at the entrance and a stair at the side of the building.

The process of setting the stones involves a ring formation, where each stone slightly overhangs in relation to the original ring. The stones in the same layer are in lateral tension with each other, forming an almost rigid annular system. Corbelled structural elements will tend to topple inwards, contrasting to the internal stresses of the arches that create each horizontal ring, and by the friction between the rings. The round shape of the dome is achieved with no tools or methods, but only by visual control and the skills of the craftsman. When the internal structure is complete, then the outer shell is formed with limestone slabs (chiancarelle) as shown in Fig. 3. The chianchiarelle are tilted towards the outside and placed with joints staggered over the underling stone blocks for the facilitation of water outflow.

2.1.2 Material Knowledge

According to Todisco et al (2017), “trulli are built using a very simple technique that ensures a minimum level of workmanship on the material and, at same time, eliminates the need for temporary scaffolding for the construction of the dome. The result is a structure composed of thick double-leaf walls, which support a corbelled dome.” As shown in Fig. 2 and 3, layers of stone are used to shape and build up the dome shape. Upon inspecting a few of these Trullo structures during a field visit in 2016, it was noticed that some of them utilised mortar for additional structural integrity. As these structures are still being built to the present day, many of the modern trulli houses use additional mortar for not only waterproofing issues but also structural stability.

The type of stone used in this region of Apulia is linked to the abundance of limestone (Fig. 4); its physical characteristics such as high compressive strength, workability, low coefficient of thermal expansion and low freezing, make it especially suitable as a construction material. The tools used for cutting and sketching out the stone often includes of hammers (martellina), chisels (stambe) and serrated tips (martidde). Where there are small and unformed holding stones, it is common to use a mortar of lime and earth to mason the stone and plaster inside the building. Where available, an easily workable sandstone (tufo) is used for the jambs and lintels of the openings. In some cases the interior walls are painted with lime and some parts are plastered with mortar of lime and earth. Fig. 3 reveals the partial plaster and paint work on the exterior and interior of the structures. It was observed that the modern inhabited trulli was more commonly finished with plasterwork for both aesthetic and functional reasons. The addition of skylights and openings are facilitated through the arrangement of stones and cut stone within the (candela), insulating filler and masonry cladding (chiancarelle).

Contemporary practitioners including Archistrati founded by Amanda Roelle have established workshops [11] working towards the education and restoration of the trulli structures in the Puglia region. The As shown in Fig. 5, older stone shingles (chianche) are re-used and re-shaped manually by
the stone masons, craftsman and workshop participants, then assembled on site. Knowledge of the size and shape of each piece to be placed in the corbelled construction typology was transferred between the trullaro (master builders) and the participants.

2.1.3 Structural forces

As mentioned above, the shape of the dome is achieved with only the visual control and the skills of the craftsman. The participants in the 2014 trulli restoration workshop gathered information about the structural systems of the trullo. As shown in Fig. 6 the main structural system is accompanied by the water shedding (drainage) system.

![Fig. 6](Image source: Roelle, A. 2014)

The structural forces inherent within the trulli have been analysed by contemporary researchers using computational structural analysis tools to assist in understanding these forces. An example is the work discussed by Todisco et al, (2016, 2017) part of a project called Apuliabase. The project is aimed at providing a preliminary structural assessment of the trulli in Apulia. This structural assessment pursues to create a database for efficiently acting for the conservation of the structures under consideration and for identifying those which are in the worst conditions. The Apuliabase Project has in part developed a simple and efficient methodology for ranking trulli with quick, cheap and non-invasive surveys on the structures. It provides a preliminary study that gives an idea about the relative ‘safety’ of the case studies considered, and a limited feedback on the absolute safety of the structures. They have presented in the research paper the geometrical data and proportions of a set of 30 trulli located in Alberobello. Specifically related to structural forces, they estimate the difference, in terms of internal force distributions, between an actual trullo and a trullo with an idealised conical candela. Here the candela is identified as the most structurally vulnerable element of a trullo.

Using graphic statics the internal force distribution (Fig. 7) for the actual geometry (on the left) and the conical dome (on the right) is revealed. The geometric idealization of the candela’s shape is based on the same height, span, thickness and total dead load of the real one. Todisco et al (2017) further explains that “because of the simplifying assumptions, this analysis cannot represent a complete study for evaluating the effects of adopting an idealised dome profile instead of the actual one. Nevertheless, it clearly illustrates that assuming a conical shape of the candela is not sufficient for a preliminary assessment of its structural behaviour and can lead to incorrect results.”

![Fig. 7](Image source: Todisco et al, 2017)

In another study by Fradioso et al (2019) titled ‘Further refinement of the Corbelling Theory for the equilibrium analysis of corbelled domes’; it was concluded that the “typical conical shape of the outer shell ‘chiancarelle’ layer of the domes of Trulli does not correspond to any static principle but probably
it is only a simpler and more economical technical solution to the problem of drainage of the meteoric waters.” This reveals that the construction method is highly influenced by the judgement of the master builder and the inherent knowledge acquired through experience more than being based on fundamental engineering science. The comparison studies mentioned above however give a useful analysis in which to base further research in relation to the safety and reliability of the trulli structures through time.

Structural design through rules of proportion developed from experience and practical training were used to design and build ancient structures from Greek temples to the dry stone trulli which survive to this day. The ‘masterbuilder’ acquired the knowledge to work the material as well as how to give the building an ‘architectural’ design. “It is only after the Renaissance that the aesthetic and structural aspects began to diverge and two distinct professional figures emerged: the architect and the engineer. The first concentrated on the rules of proportions and on aesthetics while the second explored the scientific rules embedded in the practice of building.” [12] Conceptual knowledge of structural performance is a qualitative method, in contrast to a quantitative approach. It involves the ability to visualize and sketch the deformations and the section force distributions in a structure, to develop a sense for material properties, section dimensions and to understand how a parameter’s change influences structural behaviour. As stated by Ji. T et al, (2018), “a physical understanding of the problem and an intuitive interpretation of the theoretical calculations are rarely present. Yet many of our great engineers have emphasised the importance of such understanding.” [13] The acquired and accumulated experience then requires synthesising to reach a sense of intuition. Knowledge of the material and working methods of specific construction typologies such as the dry stone trulli structures suggest the need for this intuitive understanding in contemporary structural design practice.

3 Contemporary Design Workflows

The above example of structural analysis described by Todisco et al, (2016) to determine the safety of the trulli structures is based solely on equilibrium. Recent structural design in relation to the prototyping and manufacturing of dry stone vaults stems from this equilibrium approach based on TNA. Thrust Network Analysis is a graphical form finding tool for exploring three-dimensional compression-only shapes. Substantial research on Thrust Network Analysis was initially developed by Block [14], and further developed by Rippmann [15]. It was successfully implemented as the main design, analysis and fabrication tool for the development of the ‘Amadillo Vault’ 2016, a compression only self-supporting mortar-less sandstone vault structure. The Block Research Group has since developed strategies to generate vaulted structures based on tessellated iterations. This development provides a high degree of control and automation of the resultant surface geometry. Developments in mechanistic operations due to industrialisation have amplified and refined this process of subtraction and assembly. A recent example ‘Autonomous Dry Stone’ [16] by the ETH Zurich investigates on-site planning and assembly of stone walls with a robotic excavator further advancing the assembly process of structural design. Another example is ‘Sequential modular assembly Robotic Assembly of Cantilevering Structures through Differentiated Load Modules’ [17] by the DDU, TU Darmstadt which explores the possibilities of automated assembly of cantilevering structures shown in Fig. 8. Here, the implementation of computational design workflows of self-supporting assemblies can be used to adapt past methods of modeling, simulation, manufacturing and assembly techniques.

Fig. 8 Sequential modular assembly Robotic Assembly of Cantilevering Structures through Differentiated Load Modules’ (Image source: Wibranek, B et al, 2020)
The use of modular structures in the construction industry has the potential to shape the design and fabrication process due to efficient assembly and disassembly techniques. This is, however, variant on the geometry and framework of the method. The issue of tolerances still apply and have to be accounted for in relation to the material and methods used. The work of Matter Design studio explores the relationship of stone megalithic structures and dry joint connections. [18] Here computational analysis and shape grammars are used to sort and compose the geometry of stones for a wall structure.

The geometry of each stone placed in the trulli structures however were not specifically designed to be interlocked, nevertheless present a method of corbelling and compression which have survived centuries. According to a study by Rafailidis, G. (2016), in a paper titled Overhang: Corbelled Structural Systems, suggests that there is a broad wealth of new typologies of corbelled structures in architecture which are, until now, not yet exhaustively researched or applied. “There are two main challenges that corbelled structures pose. Firstly the high mass-to-void ratio requires a large amount of construction material. Secondly, stacking vast amount of blocks in a complex pattern requires significant effort. These issues need to be explored further in both their constraints but also in light of emerging technical advancements in automated fabrication.” [19] Drawing on work established through the design and production of contemporary modular prototypes discussed, the integration of structural design methods into computational design and automated fabrication workflows contributes towards efficiency in the early design phases of structures. This can lead to future working methods in relation to the design, fabrication and automated assembly of complex cantilevered structures.

4 Conclusion
This research with the specific case of the Trulli structures has shown the impact in investigation into historical corbelled construction methods which have influenced contemporary structural design practices. The criteria established in this research include the methods of construction, material knowledge and the understanding of structural forces which are built on tacit knowledge of the craftsman and master-builders of the time. Dry-joint connections are used due to the ease of assembly and disassembly required by spatial constructs on unstable ground as well as being material efficient. Extending the knowledge of structural design inherent in the Trulli structures will be of benefit in the background of scarce resources.

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References


Focus on the very first steel bridge in France, its reuse, and its designer Emile Cheysson

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Abstract
The first steel bridge in France was erected on Champ-de-Mars in Paris for the Universal Exhibition in 1867. Still in use in Langon, Brittany, this bridge is nowadays unknown but remains a milestone in the history of French civil engineering, because of its new material but also its conceptual reuse.

This bridge can inspire and guide us through the challenges we (will) all have to deal with, including the scarcity of material resources. Instead of aspiring desperation, this challenge (and fact) is about celebrating life in all its dimensions, including its vulnerability and desire of joy among all.

The paper will describe all the implications that such a specific example around the 1st steel bridge in France and its designer can have in the life of modern-day engineers. A brand-new paradigm can emerge from it.

1 The story of the bridge

1.1 From 1st April to 3rd November 1867: At the Universal Exhibition Paris 1867

1.1.1 Geometry and location

Both useful and innovative, this bridge had its place perfectly at the Universal Exhibition 1867 and Emile Cheysson asked the “Henri Joret et Cie” company to build it under his responsibility. Mainly known for its bridges and railway roads, in France but also in Algeria, the Henri Joret et Cie had built several similar bridges before the Universal Exhibition [8] but never with this new Bessemer steel material [12].

Fig. 1 Extract of a lithography of the Universal Exhibition Paris 1867, by Deroy [7]. The bridge is located inside the yellow circle.
The bridge has a span of 25 meters and a width of 21 meters between its handrails [8]. The central main roadway is 9 meter large and is flanked on both sides by 6m wide sidewalks. The deck is made of 12 cm thick oak logs, tarred on 3 sides, and covered by a 25 cm thick stonework.

Besides the self-weight of the bridge, traffic loads allowed are 400 kg/m² [8] and then 500 kg/m² due to the large number of visitors on the Quai d’Orsay [10]. A test was also performed with 11 tons on one axle [10]. Interesting hand calculations in Reference [8], pages 13-17, establish a maximum stress of 12 kg/mm² in steel sections and 6 kg/mm² in iron sections.

1.1.2 Dismantling of the bridge

On the recommendation of Frédéric Le Play [11], Emile Cheysson is put in charge of the dismantling of the entire Exhibition. The Ille-et-Vilaine Department acquired the Parisian steel bridge [10] to make out of it the main steel structure of the bridge [10], [15] and [17], that crosses the Vilaine River in Langon and thus has replaced the old ferry that was before navigating between the two banks of the river.

No documents have been found about how the bridge was first disassembled, then transported and built again in Langon. Nevertheless, some ideas can be given here based on:
- observations on site: connecting steel plates are visible on the main structural members every 4 or 5 unitary elements of 1,14 m long, i.e. approximately every 5 m.
- comparison with another project that was both built around that time and that was much more documented: The Eiffel Tower, built in 1889, for the next Universal Exhibition. Most of the elements of the Tower were indeed prefabricated in workshop with a 5-meter length and had to be transported with trucks pulled by horses [6].

Although it is confirmed the main steel structure in Langon is the one that was in Paris, it is difficult to say if the elements were transported by boat, truck, or train. The Henri Joret et Cie had its factory in Montataire, north of Paris, was connected with the French northern railway network and owned a port on the canalized Oise River [9]. Steel was fabricated in the forges of Terrenoire (Loire) [10].

1.2 From 1868 until now: Langon, Ille-et-Vilaine, Brittany, France

1.2.1 Location

The bridge is in Langon, a town between Rennes and Redon in Brittany, and crosses the Vilaine River to reach Sainte-Anne-sur-Vilaine on the other side. With its length of 200 km, the Vilaine river goes through 4 French Departments (Mayenne, Ille-et-Vilaine, Loire Atlantique and Morbihan) and flows into the Atlantic Ocean around Arzal and Pénestin.

![Fig. 2 Panoramic view of the bridge deck, with the Vilaine River flowing below.](image)

1.2.2 Geometry

Measurements were made directly on the bridge on 09 March 2021 by the author, assisted by Maxime Amieux.

The bridge consists of three 25-metre sections, placed side by side in Paris and end-to-end in Langon. The Parisian bridge of about 25m x 18m thus becomes here a bridge of about 75m x 6m. The main roadway is 4,2 m wide and there is a 1,15 m sidewalk on each side. The height of the arch is approximately 3,20 m.

Each 25-meter section in Langon is made with 4 longitudinal arches. The deck is transversally supported with continuous beams on 4 supports distant of 1,8 m from each other, and two cantilevers of 55 cm. Each 25-m-long span is divided into 22 unitary elements longitudinally, clearly recognizable.
through the layout of the transversal beams of the bridge and the vertical posts of the handrails on the deck (every 1.14 m approximately).

Abutments on earth and both piles in the river are made of granite. Stone parapets between steel spans on the bridge deck are decorated with schist plates having the inscriptions “N” for Napoléon III, Emperor of the French, on one side, and “E” for Eugénie, his wife, on the other side.

Steel sections were also measured. The main arches are I-profiles with the following dimensions: height of 25 cm, width of 15 cm and thickness of 1 cm. A quick check can be performed by comparing those dimensions with the section surface given in [8], i.e. 57.04 cm². The measurements on site give approximately the same result, with a surface of 53 cm².

![General view of the bridge across the Vilaine River](image1)

**Fig. 3** General view of the bridge across the Vilaine River (left), Structure of the bridge from one bank of the Vilaine River, showing the 4 main longitudinal arches, the transversal lower trusses and upper beams, the bracings - X and V shapes - (right).

1.2.3 Present use

This bridge is unique in France and is astonishing for two more reasons: it is unknown, and it is used as a regular and contemporary bridge. In March 2021, road traffic signs installed at both sides of the bridge indicate that vehicles above 16 tons are not allowed to cross it. Starting from 17 May 2021, the Ille-et-Vilaine Department decided to lower the admissible loading on the bridge to 12 tons, based on a recent inspection by a checking engineering office [16].

Moreover, informal discussions with locals in March 2021 confirm that some heavier vehicles cross it and if not, have to go backwards in an unsafe manner, even leading sometimes to stops directly on the nearby railroad crossing.

A quick survey of the bridge also shows the loosening of some stones and the corrosion of some supports. The relative displacements between stones are recorded with gauges, which implies the bridge is nonetheless monitored and checked to determine its serviceability over time. Regardless of the difference between the load assumptions and the reality of the loads effectively applied (on a daily basis or accidentally), the bridge behaviour under fatigue and the maintenance operations play a role in its suitability for use.

![General view of a stone parapet on the deck](image2)

**Fig. 4** General view of a stone parapet on the deck (left), with 2 gauges measuring the evolution of the loosening of the stones (right), March 2021.
1.3 Comparison between the 2 versions of the bridge

Different periods, places, shapes and uses: with the same overall amount of steel for the main structure, this bridge is versatile and requires a summary before patterns or interpretations can be sketched. Main characteristics are given in the table below and give an overview of the differences and similarities between the 2 configurations.

Table 1 Comparative information taken from both versions of the bridge.

<table>
<thead>
<tr>
<th>Description</th>
<th>Unit</th>
<th>On Quai d’Orsay, Paris</th>
<th>On the Vilaine River, Langon</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Span</td>
<td>[m]</td>
<td>1 x 25</td>
<td>3 x 25.0 (+ parapets: 2x1.8)</td>
<td>The bridge is 3 times longer in Langon than in Paris</td>
</tr>
<tr>
<td>Width (Roadway – Sidewalks)</td>
<td>[m]</td>
<td>21 (1x9.0 – 2x6.0)</td>
<td>6.5 (1x4.2 – 2x1.15)</td>
<td>The bridge is 3 times narrower in Langon than in Paris</td>
</tr>
<tr>
<td>Deck surface</td>
<td>[m²]</td>
<td>525</td>
<td>511</td>
<td>Approximately the same surface</td>
</tr>
<tr>
<td>Materials for the deck</td>
<td>-</td>
<td>Oak, tar, stones</td>
<td>Concrete and asphalt</td>
<td></td>
</tr>
<tr>
<td>Loading</td>
<td>-</td>
<td>400 - 500 kg/m²</td>
<td>16 tons</td>
<td>Lowered to 12 tons on 17 May 2021</td>
</tr>
</tbody>
</table>

Fig. 7 Both configurations of the bridge, before and after transportation to Langon: sketches and schematics in red (Artistic conception of the author, 2021)

Although their deck surface is approximately the same, sidewalks occupy much more space in Paris than in Langon (respectively 57% [=12/21] against 35% [2,3/6,5]).

Many sources such as [10] and [17] confirm that the original Parisian Bessemer steel structure of the bridge was fully re-used for the bridge in Langon. Besides M. de Lagarde, chief engineer at the department of Ille-et-Vilaine only asked Joret et Cie to change the wood and stone deck into a full metallic one and used the entire original Parisian Bessemer steel main structure for the bridge in Langon [10], and this is confirmed to be the first use of the Bessemer steel in France in [13] and [14]. Cast-iron handrails of the Parisian version have also been changed into thin iron ones to go along the 2x75 meters of the bridge [10]. The full bridge cost 65,300 francs, which is considered as an extremely low price for that time and let wonder how much profit would have been possible if other structures from the Exhibition had been re-used instead of being demolished [10].

In a more general and concise way, the bridge has evolved from a short and wide version to a long and narrow one, with the same original Bessemer steel main structure: Same material and elements but with a different overall shape.
2 Its designer Emile Cheysson and his life philosophy

2.1 Biography (1836 – 1910)

Emile Cheysson was born on 18 May 1836 in Nîmes. His mother was protestant but raised him as a catholic, the religion of his father. “And nobody would be able to say that I did not keep my word”, as the journalist Frantz Funck-Brentano once heard from her own mouth [1].

In 1854, he started studying at the French Ecole Polytechnique and 2 years later in Ecole des Ponts et Chaussées, where he performed a training ([1], page 51) on the foundations of a lock for the transatlantic ships in Le Havre.

His first job in 1859 as a public officer was based in Reims and he there performed numerous works: the 30 km long railway line between this city and the military base of Châlons, achievement of the Canal between the Aisne and the Marne Rivers, construction of sewers, etc [1], page 78.

In 1864, he was nominated Engineer and then Director of the Machine Department for the Universal Exhibition of Paris 1867, after having been introduced by the General Inspector of Ponts et Chaussées M. Bommart. He met with Frédéric Le Play in 1865, the general commissioner of the event. This moment in his career decided the rest of his life. He became the friend, the secretary, the associate and always remained the disciple ([1], page 57) of Frédéric Le Play, a man considered as a founder of the French sociology. Once the Universal Exhibition ended, Emile Cheysson oversaw the dismantling of the Exhibition, which included the steel bridge that was sold to the department of Ille-et-Vilaine [10].

Right after his work for the Universal Exhibition, from 1868 to 1870, he became Professor of Administrative literature in Ecole des Ponts et Chaussées. He also started in 1868 to take care of the vineyard he had acquired in Chiroubles, that was before belonging to the monks of the Abbey of Cluny. It is still producing wine today with its 26 hectares: Le Domaine Cheysson [3]. He was spending time there, cultivating the vine, studying and being with his loved ones ([1], pages 61-63).

The 1870-1871 is a period both important for France and Emile Cheysson: as the Director of the Mills of Paris, he improvised milling facilities to supply Paris with flour during the Siege of Paris, one episode of the Franco-Prussian war. French writer Victor Hugo described on 30 December 1870 the daily life at that time with these words: “It’s not even more horse that we eat. Is this maybe some dog? Or maybe a rat? I start having stomach aches. We eat the unknown. [2]” Emile Cheysson thus helped as much as he could the Parisians not to die from hunger.

From 1872 to 1874, he is the Director of the Creusot factories, with its 15,000 workers [1]. From now on, he will forever link mathematics with social issues, highlighting the necessity to insure workers against all kinds of accidents and use for this purpose, insurance: “the only science with mathematics as a basis and ethics as a crowning achievement” [4].

He is sent back to Paris in 1875 to improve the navigability of the Seine River and then becomes Chief Engineer in the Department of Maps and Plans and graphic statistics of the Ministry of Public Works. Starting from 1878, he “understands how an exact determination of the ground topography can be of great interest for science, agriculture, civil engineering and national defense” ([1], pages 36-37) and thus continues the work of Paul-Adrien Bourdaloué, who defined the first levelling of France in 1857 [5]. He also publishes numerous works of graphic statistics during that period, and monographs and comparative legislation [1], embracing many various topics such as agriculture, transportation, and daily life of workers.

Fig. 7 A historic marker found on the bridge to help determine the ground topography (“nivellement général”) of France. General view (left), Detail (right).
Starting from 1882, he is Professor of Political and then Social economics at Ecole libre des Sciences Politiques, and from 1884, Professor of Industrial economics at Ecole des Mines.

He spends the rest of his career, that ends in 1906, to strengthen his contributions to improve the “material and social conditions of the workers” in all the possible ways (creation of a Social Museum, generalization of individual savings and pension plans through mutual insurance, etc.).

He dies during a stay in Switzerland, on 7 February 1910, after being violently hit by a sled in the town of Leysin [1].

2.2 His life philosophy

During all his life, and more radically after meeting Frédéric Le Play during the Universal Exhibition of Paris 1867, Emile Cheysson tried to combine science with love and act accordingly, seeing it as a “social duty” that is “way too much unknown or at least neglected by a large number of the lucky ones in life” [1]. He said he had three principles: “Love, Peace, Optimism!” [1], page 77.

Be curious, learn, do not hesitate to invest time, understand, go deep, and hit the obstacles, then define and reduce them, finally suggest proposals to bypass and finally forget the obstacles and win the support of all the parties, towards a sustainable and holistic development. This could sum up the following paragraphs, with the emphasis on the fact that this is a continuous cycle and not a chronology, like an endless discussion between two old friends or a winemaker with his vine, just like Emile Cheysson liked it and applied it to the reuse of the bridge in Langon.

2.2.1 Touch and be touched – Education – “Listen”.

“To study with good will the institutions of other people, it is the way to understand each other better and to penetrate each other, to dissipate misunderstandings and prejudices, to move with a common impulse towards justice” [1], page 71.

▪ Elaborate a state of the world on your own.

Universal Exhibitions have always tended to establish an encyclopaedic state of the world, theme by theme, nation by nation.

With his books of statistic graphics, Emile Cheysson invented a way to sum up in a simple and easy way information that were before presented with long lists of numbers and often arbitrarily separated. He made the wealth of France tangible with these publications [1]. The way things are shown often guide the way we will answer to them. We should expand our scope of knowledge, make associations between ideas that at first sight seem unrelated or even contradictory. Far from being needless, it is the story of Emile Cheysson who thanks to his various and many commitments was able to forge and strengthen a strong answer to the challenges he was passionately facing, with ingenious connections and solutions unnoticed by others [1], page 84.

It would be a mistake to think that such a work must necessarily be done with data from all over the world and focused on only one topic. This bridge shows that unknown wonders of the past are reachable within a few kilometres and be valuable in many ways. I am convinced our knowledge depends more on our curiosity than on the material resources we have in hand (including a state-of-the-art smartphone or any other digital device). We should build local maps with all the singularities worth to be known, depending on the interests raised, and irrigate the schools, offices, warehouses, city halls, libraries, and our own neighbours with these witnesses from the past: they are the beginning of a long ball of wool that was just waiting to be unwound.

▪ Contact the reality of the other.

Frédéric Le Play in 1867 had the merit of considering social issues as part of the world that must be known during the Universal Exhibition. Indeed, beyond the factual state of the material world, spending time in the company of different persons help give us a clear view of the world we live in, and thus, after that, a better knowledge of where we want to stand inside.

It can be through books: let us not forget that Emile Cheysson for example did literature studies and knew Latin very well. It can also be through punctual meetings or gatherings, national or international, in the context of organizations or less official networks; or more permanent with setting up all kinds of peer-to-peer mentoring. Emile Cheysson had for example long and regular talks with Frédéric Le Play outside, on the Saint-Sulpice square in Paris.

This is about giving time without purpose, except the highest interest on the quality and honesty of the relation. This has to do with freedom and should be encouraged, widespread and deepened in all

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spheres of life (if it is really necessary to divide the life in spheres): sharing of technical books between engineers of the same district or city, mentoring between an established engineer and a young teenager eager to learn structural design, partnership between a few engineers from different companies to share their thoughts, problems and wishes for improvement, organizing meetings with people using the structures you designed, or the ones who decided to build it. Emile Cheysson had maybe an opportunity that many did not have at that time: a strong and loving family around him, a work as a public officer that made him discover various worlds according to his multiple assignments and with a financial stability. But he never denied it and even claimed his luck as the root of its commitment.

2.2.2 Discuss the positive and the negative – Love – “Answer”.

“[…] it is beautiful to act on the existing miseries […], it is better and more human, going straight to the causes of the evil, to work so that the evil disappears.” [1], page 23.

- Define what is at stake and its enemies.

Emile Cheysson considered alcohol and more generally diseases as an enemy of the family and thus tried everything to prevent workers not to go in “cabarets”: healthy housing, sufficient food, education for children, rest on Sundays.

When thinking of civil engineering, is it enough to fight against climate change by using carbon neutral materials? Are these materials used for useful purposes? Can’t it be a question that we, as engineers, ask ourselves before starting any project or job? And what about ugliness? How to define it and make it simple enough so we can both “act like a primitive and think like a strategist”, as the French poet René Char wrote in 1946? Emile Cheysson had one conviction and was nonetheless member of the Institut and 107 other organizations [1], all dealing with topics related to his fight against the poor life conditions of the workers, and his wish to preserve and strengthen all the necessary links to preserve the life in common of men [1], page 91.

Let us not be afraid to name the mistakes, describe and circumscribe them, and then give and share our thoughts with other parties to discuss their veracity and the strategies to make them disappear.

- Create new stories, share, and pass on.

When Emile Cheysson takes part to the “nivellement général de la France”, he contributes to another vision of our geography. Nowadays, each civil engineering project is still based on the “NGF” topography (Nivellement Général de la France). We should develop tools that can be shared by the most of us, engineers but also citizens, to create new behaviours, aiming at even better life and working conditions.

Emile Cheysson strongly believed that values like family and religion or habits would save our society from all the dangers. Sadly, or not, family and religion are nowadays transformed, reduced, or replaced by other values that can take different names depending on the political points of views (Money as a new religion for some, Individualism instead of Socialism, Nation against Globalization, etc). We should think about it, take time, and discuss the nature and the size our values should be nowadays, as a worker, as a citizen, and as a human being on Earth and in the Universe. We should play with the scales and create new stories that generate new visions, and new ideals towards which we can peacefully all go. I realize the difficulty of the task but want to remember myself what Emile Cheysson was saying: “Always do what you are afraid of”. He saw too many causes working separately, and almost against each other. His work allowed many of them to get unified, and to realize they had common goals and gather their forces to face challenges together.

His vineyard is still producing wine today. He loved nature and this old untouched ground can be a great source of joy for his family and all the wine consumers. We should create a new link with the old, find the universal in the multitude, whether cultural, spatial or chronological, and be humble in finding the way to be useful.

Acknowledgements

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Structure and architecture in dialogue: design micronarratives of the N2 motorway (1961-86)

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Abstract
In 1969, the architect Rino Tami (1908–94) signed the essay Problemi estetici dell’autostrada and, in 1984, the more explicitly titled one: L’autostrada come opera d’arte. Both texts discussed how an infrastructural work, such as a motorway, could not only participate but innovate in the architectural narrative through its structural design. Tami’s essays bear witness to the unique design of the Canton of Ticino N2 motorway (1961–86) that clearly integrates structural and construction functions with ‘artistic directives’, representing a known case in architectural history. This paper focuses on the design and construction history of the N2, discussing how the motorway’s formal unity has been achieved through embedding the dialogue between structure and architecture in the adopted design processes and highlighting the operational context, the strategic design tools, and the ‘standard’ design solutions used.

1 Introduction
The 300 km of the Swiss N2 motorway links the cities of Chiasso and Basel, crossing seven cantons. The construction of the Ticino Canton sector, which is more than 140 km long, began in 1961 and was finished and inaugurated on 23 October 1986. Built over 25 years, in segments made entirely of reinforced concrete, the highway is an extraordinarily unified work characterised by evident rigour and formal coordination among the structural works located along its route.

In 1961, the famous Italian critic Bruno Zevi (1918–2000), speaking about the Italian motorway Autostrada del Sole, attributed the realisation of an infrastructure that was non-uniform and inconsistent from an aesthetic point of view to a ‘fragmentation of the works’. He complained about the involvement of 18 companies and 27 engineers in the construction of the various viaducts in the motorway’s 84 km Apennines section [1]. In that same year, in Ticino, 25 engineering consultants and 25 construction firms (in addition to associations, committees, communities, consultants, and consortia) were involved in the construction of the first 10 km of the N2 highway [2]. Furthermore, along the entire route of the N2 and including its bridges, 78 structures were built between 1961 and 1968, and 115 between 1980 and 1986, 20 years after its initial construction. New structures incorporated technological advances that had occurred during the intervening period.

It is clear that the N2’s formal, structured design is due to a design strategy that differed from the centralised one called for by Zevi, an efficient ‘coordination’ already personified by the architect Rino Tami (1908–1994) [3,4], who in 1963 was designated the N2’s ‘consultant on aesthetics’.

In the following pages, the operational context, the strategic tools, and the ‘standard’ solutions used in the ‘N2 design strategy’ are reconstructed with the general aim of highlighting the dialogue between architecture and structure in shaping a specific Ticinese culture in infrastructure design that has persisted in recent works [5,6].

The study was conducted, within the FSN project ‘Architecture in Canton Ticino, 1945–1980’ promoted by the Archivio del Moderno – Università della Svizzera Italiana, by material analysis of primary sources in the canton’s archives, which have been poorly investigated to date: the National Roads Office (USTRA) Archives, Bellinzona; the Renato Colombi Archives, Archivio del Moderno, Balerna; and the Rino Tami Archives, Archivio del Moderno (AdM), Balerna.

2 Operational context
In the 1940s, it was decided to complete the main road network in Switzerland by adding two major transit routes, one from Sankt Margereten to Geneva and one from Basel to Chiasso. In 1954, the Federal Council appointed a commission to develop this road network. The commission, which was responsible for defining the legal basis for and clarifying the financial aspects of the operation, remained in charge until 1958, when the commission published its first report. For Ticino, the proposal
foresaw a predominantly two-lane road with no motorway tunnel through Saint Gothard. On 5 April 1959, the politician Franco Zorzi was elected councillor of the Department of Construction. Confident that infrastructure would play a crucial role in modernising the canton, he firmly opposed the federal programme, foreseeing the construction of a road that had four lanes and was complemented by a Gothard tunnel. Following the 8 March 1960 ‘Federal Law on National Roads’, connecting roads of major importance and general interest for Switzerland were declared to be national roads and were divided into three classes. The Ticino section of the N2 national road was included in the first class, defining it typologically as a true motorway.

Two terms, chosen by Zorzi, were key to the creation of a clear operational strategy for the N2 project: ‘reinforced concrete’ and ‘design coordination’. The choice of reinforced concrete was strictly linked to the canton’s economic and productive development. The choice of design coordination was undertaken in accordance with a vision of infrastructure as a tool for modernising the territory, avoiding its aesthetic spoiling [7].

### 2.1 Design coordination and workflow

In 1959, Zorzi decided to set up an autonomous section of the Department of Construction that would coordinate the N2 project from its layout to construction on site. Thus, on 5 May 1959, one month after his election as head of the Department of Construction, he met with civil engineer Renato Colombi (1922–2015) [8], a graduate of the Federal Polytechnic. Colombi was employed in the hydroelectric sector of the company Blenio SA and was an expert on large reinforced-concrete constructions. Zorzi explained to Colombi the intention to form a new team of technicians within the department that, under Colombi’s leadership, would be entirely dedicated to the motorway project.

This team constituted the Ufficio Strade Nazionali (National Roads Office) and was officially established on 7 July of that year [9]. As the section’s ‘chief engineer’, Colombi began recruiting technicians. On 6 August, he met Zorzi with engineer Francesco Balli (1925–2015), a graduate of the Federal Polytechnic who was also employed by Blenio SA and was a specialist in reinforced-concrete construction [8].

The National Roads Office began working unofficially on 1 October, when Colombi’s first collaborators settled in a small villa in Bellinzona. On 7 October, examinations were held for drafters and to recruit eight civil engineering graduates of the Federal Polytechnic, who were employed by the office under private companies’ fees [9]. The Office’s activities officially began on 1 January 1960. The team numbered eight engineers, one technician, six draftsmen, and one secretary in charge of expropriation procedures. Despite its small staff, the Office was divided into three main services: ‘design’ (directed by Balli), ‘geotechnical laboratory and materials testing’ (entrusted to engineer Marco von Krannichfeldt), and ‘administrative services’ (directed by Renzo Sailer). In April, the ‘works management’ service was added, coordinated by engineer Glauco Nolli. Between 1961 and 1963, three technical consultants were involved in supporting the Office tasks: the traffic engineer Jacques Richter, the geotechnical engineer Ezio Dal Vesco, and the architect Tami [10].

From its foundation, the Office represented the operational hub for the various actors in the project, which ranged from the Federal Council (Swiss government) itself and the canton’s municipalities to engineering firms and contractors. The Federal Council established the annual loans, the general time planning, and the route layout. In accordance with the law of 8 March 1960, the office design section’s first tasks involved design services for the route layout, which was proposed on a scale of 1:25000 by the Federal Council and was detailed in ‘general plans’ on a scale of 1:5000.

During this phase, the office design section drew up the ‘general plans’ of the individual sections according to a list of priorities and then shared them with the municipal administrations so that they could be examined and commented on by them. After being modified, the plans were submitted, once, to the Federal Council for approval. While the ‘general plans’ were under review by the municipal administrations, the design section was working on the final execution drawings of the sections for which the approval process had already been completed. Given the design section’s limited staff, the network expanded during this phase to involve professional engineering firms that were already active in the Ticino region. Between 1961 and 1968, while about 25% of the motorway was being built, the Office’s staff increased from 14 in 1960 to 168 in 1968, and the number of consulting engineering firms involved in the execution phase increased from 3 in 1961 to 38 in 1968 [11].
3 Design strategic tools

3.1 Private bidding

Consistent with its general objective of ‘coordination’, the Office used an effective operational tool that was valid both in the design and tendering phases: private bidding competitions. In particular, extending the practice of private bidding competitions to the design execution strengthened the relationship between the Office and the other professionals involved and enabled the Office to formally control the individual products in accordance with the overall architectural vision for the motorway.

The first private bidding competition for design was announced on 4 January 1961, for the Melide viaduct. Analysis of the jury and its participants clarifies the operational aims of this tool. Five engineering firms (Hans Eichenberger, Gellera and Lombardi, Losinger and Cie, A. Casanova, and Conrad Zschokke), all Swiss and mostly from Ticino, were invited. Among the jury members were Zorzi himself, representing the Department of Construction; Balli and Colombi, representing the Office; Luigi Pini and Amedeo Marrazzi, representing Ticino’s professional engineering firms; and architect Tami. While the inclusion of professional firms on the jury ensured that the Ticino Canton’s professional community was committed to the N2 project, Tami’s inclusion was the key element in the public’s perception of the motorway as a project that would enhance the region. Indeed, Tami was already the director of the motorways section of the Swiss Association for Territorial Planning (AS-PAN), of which Zorzi himself was president. The jury’s reports indicate that the projects were judged from two perspectives: ‘construction’ and ‘aesthetics’. Although unusual for infrastructural designs, the aesthetics perspective called for the projects to be verified using photomontages explicitly requested by participants to enable them to check architectural aspects of the designed structure in relation to the landscapes in which they were being built. Thus, in the Melide competition, Eichenberger’s viaduct was awarded the contract because it was considered ‘valuable’ for its ‘lightness and unity of rhythm of the structural elements’ [12]. Even before the solution was presented, it had also been judged ‘excellent’ in terms of its construction concept.

Following the successful outcome of the Melide competition, a continuous dialogue was established between Tami and the Office, aimed at the ‘harmonious’ integration of the new road with the canton’s landscape. In 1963, when ‘the freeway works entered the execution plan’ [13], Tami was officially designated by the Council of State as an ‘aesthetic consultant for the motorway works’, a role he held until 1983.

3.2 Corrections on ‘road profiles’ and ‘ground connections’

By participating in the jury, Tami directly influenced the individual viaducts within the overall view of the sequence of structures. Aesthetic coordination was implemented, thus, in the ‘general plans’ drawn up by the Office as the basis for the design of the private bidding and through the study of architectural details.

From the very first construction sites of the N2, Tami charged his young collaborator, architect Aurelio Galfetti (1936–), to compile some ‘albums of errors’. These albums used photographs and annotations to document the main formal errors made in the design of the various structures and their relationships to the landscape. These errors were identified, classified, and transformed into a series of ‘standard solutions’ to be adopted moving forward. Tami elaborated these standard solutions in a series of dense drawings of ‘standard bridge abutments’ (Fig. 1) and ‘standard overpasses’. In the early 1970s, the Office translated Tami’s bridge abutments drawings into a series of ‘standard plans’ [15], which were execution drawings that included details of the reinforcements for various dimensional hypotheses. They described, for example, inclined walls featuring standard bridge abutments or profiles of the retaining walls to be used on the entire route (Fig. 1).

Furthermore, the dialogue between Tami and the ‘design service’ technicians of the Office focused on the road profiles. In this way, unique structural figures took shape, both for the substructure viaducts and the road overpasses. In accordance with the global use of reinforced concrete, the substructure viaducts conformed to the image of high-pier girder bridges featuring ‘sliding and profile’ decks, while the overpasses conformed to the image of ‘overpasses with sliding decks and inclined piers’. During this phase, Tami elaborated the novel figure of the ‘tunnel portal’, featuring the whole route as a series of architectural artefacts overplayed to the road, designed, one by one, by Tami himself [14].
3.3 On-site corrections

In special cases, Tami’s influence also extended to the construction phase, engaging in the architectural dialogue with the engineering firms and contractors.

One example is the case of the Capolago viaduct, built in 1964 by the Zschokke Company. The project involved building a structure that had spans of 20 metres and slender rectangular piers on which rested, with transverse lintels, the longitudinal beams of a prestressed reinforced-concrete deck, completed with slabs cast in situ. The beams, with T or double-T sections, were composed of 2 prefabricated elements, each 10 metres long, coupled on site with longitudinal prestressing cables, which provided both a productive and constructive advantage. However, the appearance did not comply with Tami’s ‘shaped’ and ‘profiled’ vision set out for the N2 viaducts. Therefore, during the execution phase, Tami corrected the viaduct by designing a new deck cross-section and shape for the connections between it and the piers. By inserting a continuous lateral ‘edging’ in the form of an inclined wall composed of small, prefabricated reinforced-concrete elements (Fig. 2), Tami reconstructed the deck profile to create a continuous longitudinal band that inclined outward. While the viaduct was under construction, the execution drawings, developed between 1964 and 1965, called for the 495 prefabricated elements [16]. They were one metre long and different through the valley from on the mountain, to be assembled on site into the new reinforced-concrete enclosure.

In 1968, Zevi appreciatively noted ‘the profile of the prefabricated viaduct of Capolago’, which had been completed slightly more than a year before, for its ‘value of lightening the figurative weight of the roadside’ [17]. He thereby culturally validated the design strategy developed in Ticino, in contrast to the much-criticised Italian project.
4 'Standard' design solution

4.1 High-pier girder bridges featuring ‘sliding and profile’ decks

During this phase of the project, the most interesting collective effects of this project strategy were suggested in the design of the standard structural bridge type—the high-pier viaduct with a continuous box-section girder—developed by the canton’s engineering studios following Tami’s indications. On this topic, the architect’s conception had affected the engineering project, stimulating those professionals to reflect on the overlap between Tami’s formal themes and the structural design.

For example, the Bisio viaduct (1962–65) and Ruina viaduct (1977–84) projects indicated, on one hand, the continuation of Tami’s ‘coordination’ and, on the other hand, the establishment of a specific competence acquired by Ticino engineering firms regarding this approach to the project.

The Bisio viaduct’s design was developed by the Ticino engineering firm Bernardi–Gerosa for a 1962 competition. Five engineering firms (four of which were from Ticino) were invited to bid privately: Bernardi and Gerosa from Mendrisio, Marazzi and Pini from Lugano, Lombardi and Gellera from Locarno, Augusto and Alessandro Rima from Locarno (consortium with Elektrowatt) and Emil Schubiger from Zurich. In addition to Tami, the jury included Zorzi, Colombi, Balli, and engineers Hermann St’sssi, Edmond Rey, and Eichenberger. The Bernardi–Gerosa firm won the contract, thanks to its design of a viaduct that was considered of ‘excellent formal appearance’. After the competition, at Tami’s suggestion, the viaduct’s appearance was further improved by the addition of inclined abutments designed to connect the deck line geometrically with the terrain’s slope.

In the execution phase, the invitation to focus attention on the viaduct’s architectural lines was further reflected in the elegant solutions developed independently by Bernardi and Gerosa. The inclined sections of the deck’s girder box and the strong overhang of the carriageway platform contributed to the elimination of the traditional structures that supported the motorway’s twin roads. Furthermore, the head of the two-cantilevered carriageway was emphasised to create a ‘light band’, which contrasted with the ‘corresponding shadow effect’ created by the overhang. Finally, the design was completed by reinforced-concrete supporting devices placed at the foots of the piers and between them and the deck (Fig. 2).

The Ruina viaduct was built in the Biaschina gorges between 1976 and 1984, designed by the engineers Balmelli and Filippini. After a private bidding competition [18] that involved five engineering firms from Ticino, Balmelli and Filippini’s proposal won the second prize. Then, their bid was com-

Fig. 3 Bisio viaduct, cross section and sketch of the deck (ASTRA Archive), picture of the viaduct (AdM, Colombi)
bined with Kessel and Blaser’s and thereby won preference in the execution phase. According to the jury’s report, the competition was characterised by a variety of proposals, including those based ‘on solutions that have been widely tested’ and those characterised as ‘new and original’, confronting the jury with a ‘real choice’ (Fig. 4). Kessel and Blaser’s project, which presented a mixed solution in concrete and steel and was characterised by special Y-shaped supports, was judged the best in terms of aesthetics for ‘the degree of lightness and transparency it offers as a whole’.

Balmelli and Filippini’s viaduct was also judged well for ‘aesthetics’ because it was ‘formally valid’ in accordance with ‘tradition’. In fact, the latter proposal’s deck ‘flowed harmoniously’, and the rhythm of its high, slender piers was described as ‘almost regular’. While its appearance conformed typologically to the vision developed by Tami in the 1960s, in terms of structural design, the viaduct presented a particularly innovative solution. According to the jury, ‘the parts’ sizes responded well to the forces’ flows’, and the viaduct differed from all other structures on the route because of the design of a unique box deck made of pre-stressed reinforced concrete that included both of the road’s pathways, thanks to two cantilevered side decks of 7.5 metres (Fig. 4).

Fig. 4 Ruina viaduct, photoediting of the viaduct design, top-down and from left to right: Kessel + Blaser, Balmelli+ Filippini, Simona-Tarchini Tunesi, Bernardi [18], execution drawing by Balmelli+Filippini (ASTRA)

4.2 Overpasses ‘with sliding decks and inclined piers’

The codified structural elements of the viaducts were also applied to the overpass design, affecting the standard structural typology of overpasses with ‘inclined piers’.

For example, the two overpasses at Soresina and Riale Zarigo [19], built between 1968 and 1970, demonstrated the construction of the architectural image of the ‘N2 overpasses’, through the combination of ‘inclined abutments’ with the overhangs ‘shadow lines’. The architectural design of both structures was by Tami, as part of the study of the series of type bridges, and focuses on the shadow effect of the deck, the geometry of the connections to the ground of the structure, and the variable section of the inclined piers (Fig. 5). The engineering of both structures was done by Ervinio Kessel.

Kessel worked in collaboration with Tami, supporting the architectural design by exploring different static solutions: a two-hinged rigid frame scheme with inclined piers (Sprengwerk scheme), that of a continuous beam with three spans, and, finally, a rigid frame with V-shaped piers. The solution chosen for the Soresina overpass was an asymmetrical frame, with 35-metre spans, V-shaped piers (hidden in the embankment), and an inclined box-beam with a constant section, 1.5 metres high, in pre-stressed reinforced concrete (Fig. 5). The structure was characterised by some construction ‘tricks’. While the use of pre-stressed reinforced concrete enabled the reduction of the deck’s thickness, the ‘heavy’ ends of the beam were built as hollow elements with a Sagex filling (a kind of polystyrene).

The variable section of the inclined piers, originally proposed by Tami and also present in Kessel’s first static solution, disappears in the execution design of the structure, following the adoption of the second scheme. For the Zarigo overpass, the first hypothesis, proposed by Kessel as the one aesthetically closest to Tami’s architectural proposal, was discarded for economic reasons; the second, although economically advantageous, was set aside because of the complexity of the arrangement of
the reinforcement. The third solution was finally chosen, and the structure was, thus, built as an asymmetrical frame with a constant-section channel-beam and V-piers embedded in the embankment.

Fig. 5 Tami’s studies for Lamone-Rivera overpasses, 1968 (AdM, Tami); Kessel’s drawings for the Soresina overpass: study for the adoption of the Sprengwerk scheme (AdM, Tami) and the execution cross-section drawings, 1970 (ASTRA)

5 The N2 design legacy: the rules and variants for site-specific structures

This paper has discussed the design workflow and methods of design of the Swiss N2 motorway (1959–86). The operational context, the design tools used, and the standard design solutions adopted were analysed through the narrative of the individual works.

In conclusion, it is thus possible to report some findings of the N2 design approach through history-based analysis.

On the one hand, the operational context analysis highlights how the N2’s formal unity was achieved through the design coordination strategy established by Zorzi. As a consequence of Zorzi’s choices, the Office—as the operational hub—ensured that the success of the established design workflow was maintained over the years of the N2’s construction. In this operational framework, the adoption of the private bidding tool played a lead role in both selecting the actors involved in the project and extending the architect’s formal coordination, even to the execution projects.

On the other hand, the design analysis reveals as the integration of architect’s ‘correction’ in some ‘standard’ structural topics allowed shaping, in a sense, a common design language among the engineers of the canton involved in the 25 years of the N2’s construction. The enrichment of the engi-
neers’ bridge design vocabulary—through the terms ‘connection to the ground’, ‘decks’ shadows effects’, and ‘rhythm of the columns’— is the most inspiring legacy of the N2: demonstrating an efficient conceptual design approach in order to integrate structural functions with ‘artistic directives’ regarding the overall architectural vision of the motorway and site-specific structural solutions, the direct reference point of recent reinforced-concrete infrastructural works of the Ticino canton (such as the Transjurane).

Acknowledgements
The studies were conducted within the FSN project ‘Architecture in Canton Ticino, 1945–1980’ (www.ticino4580.ch) promoted by the Archivio del Moderno. The author would like to express her gratitude to engineer Marco Fioroni and archivist Davide Campana for their collaboration in research conducted in ASTRA’s historical archives in Bellinzona, and to anonymous reviewers of this article for their suggestions.

References
Overview of the historical reinforced concrete bridges in Slovakia – inspirations for modern structures

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Abstract
In Slovakia, there are several amazing reinforced concrete bridges around the age of 100 years that are still in a good condition. Even today, some of them would be considered as an outstanding structure. The most interesting old reinforced concrete bridges in Slovakia, which have survived until today, were selected to be presented in this paper. Their common features, which have helped them to withstand environmental loads for 100 years are pointed out and described in more detail.

1 Introduction
When designing bridges, the design should be based on the latest knowledge and modern bridge construction methods. However, sometimes it is necessary to look back in the history and learn from the past, which solutions proved to be good and those, which did not. Otherwise, the structural engineers risk repeating the mistakes of their ancestors or developing a solution, that has already been invented and passed the test of time. Slovakia is not a big country, but it has several interesting bridges older than 80 years, of which the most interesting ones were selected to be introduced in this paper. These are the bridges, from which designers can learn or which could be inspirational even today.

2 The oldest preserved reinforced concrete bridge in Slovakia – Krásno nad Kysucou
The bridge (Fig. 1) was built in 1891 according to Monier's patent bought by the Wayss company, by which several bridges of this type were built on the territory of the former Austro-Hungarian monarchy. It has two arches that reach a clear span of 16.8 m, while their thickness varies from 400 mm at the arch springing to 150 mm in the middle of the span.

Above the reinforced concrete arch there is a thick layer of plain concrete protecting the reinforced concrete at the arch springing, where most of the water flowing through the bridge pavement accumulates. The thickness of the plain concrete above the main arch reached a height of 1 meter at vault springing and its thickness gradually decreased, until it diminished approximately at one third of the vault. The rise of the arch is 2.40 m. The free width on the bridge is 6.1 m and the total length of the bridge is 36.2 m. This unique bridge, on which the statue of St. John of Nepomuk is placed, was still in a good technical condition (before reconstruction) even after 120 years of service life, even though it is located on one of the busiest roads in the region. It is also one of the oldest still functioning reinforced concrete bridges in Central Europe. In the load test performed during the reconstruction of the bridge in 2014, the bridge showed only negligible deformations under a truck weighing 25 tons. The maximal deformation in the horizontal direction was at the limit of measurement accuracy and in the vertical direction the deformation had a value lower than 1 mm. The bridge showed almost no reinforcement corrosion. There were identified several factors that helped the bridge to stay in such a good condition even without any maintenance. The leakages were minimal at the bridge, even at the arch springings, while this part of the reinforced concrete arch was well protected by a plain concrete overfill capturing chlorides from deicing salts. The minimum thickness of concrete cover to reinforcement was of 30 mm. The entire surface of the arch was coated with cement render, and it was found out that this 2mm thin layer was able to stop carbonation and it is currently the part of the further research. After 120 years of service, carbonation depth reached only 4 mm! After the reconstruction of the bridge a repeated core
drilling was performed with a very interesting result. After one year the carbonation of the new applied layer of shotcrete reached the same depth as the original concrete protected with cement render after 120 years (Fig. 2).

![Fig. 1](image1.jpg)

**Fig. 1** The oldest preserved reinforced concrete bridge in Slovakia built in 1891.

![Fig. 2](image2.jpg)

**Fig. 2** Carbonation depth of the new concrete layer after 1 year reached the same depth as the original concrete after 120 years of service (original concrete was protected by a simple, 2 mm thick, cement render).

### 3 Peter’s Bridge in Liptovský Hrádok

This reinforced concrete bridge built in 1941 has a relatively large main span, which reaches 55 m (bridge is not prestressed). The height of the girders in the middle of the span is only 1.9 m (approx. 1/30 of the span). From structural point of view, the 3-span bridge is a gerber-girder structure. Its shape was fully adapted to the structural requirements: box girder has a height of 4.0 m above the piers, and it changes into a T-beam in cross-section towards the middle of the span as well as towards the abutments. The free width of the bridge is 6.0 m (Fig. 3).
The bridge was designed by Michal Piasecký, a Russian emigre with Slovak citizenship, and built by prisoners of war. At the end of the World War II, 11 partisans were killed during its defence, and they have a memorial nearby the entrance to the bridge. Despite its age, the bridge is in a relatively good condition, even when compared to some much younger prestressed bridges of similar span. The bridge has a very well-resolved, almost maintenance-free drainage, which significantly reduced the leakage of the bridge. One drain gully is located every 7 meters on both sides of the road. Thickness of concrete cover to reinforcement is more than 30 mm. The bridge is located on the main road in a severe cold environment, with a significant road salting during winter time, the negative effect of which is evident by the complete disintegration of the reinforced concrete columns of the railing.

Nevertheless, the load bearing structure was still in a satisfactory condition even after almost 80 years of service. Only those parts of the bridge were suffering from reinforcement corrosion, where more severe leakage occurred (nearby damaged expansion joints at the abutments). The first reconstruction of the bridge took place in 2018, when the pavement and railing of the bridge were replaced - the load bearing structure needed only small repairs (Fig. 4).
4 The concrete arch bridge in Komárno

One of the most impressive reinforced concrete bridges in Slovakia was built directly on the foundations of a former steel truss bridge (1913 – 1945) with a span of 115 m. The reinforced concrete arch bridge was built between the years 1951 and 1955. At the time of its construction the bridge was one of the largest bridges of its type in Europe. The span of the arch is 112.5 m, with a very small rise of the arch, which is only 8.5 m, (Fig. 5).

With this “courage of the arch” (span^2/rise), which is 1480 m, the bridge broke the previous record set in 1911 by the Italian bridge Risorgimento. In the middle part the arch of the bridge is only 250 mm thick (Fig. 6).

The old supports of the previous bridge were very cleverly used during construction of the recent one by placing the entire new structure on them by means of sliding plates. This temporary sliding support made it possible to introduce a horizontal force into the structural system, which activated the passive earth resistance at the springing of the arch. However, the calculations differed from the real long-term deformations of the soil that resulted in excessive deflections of the central part of the arch. These deformations stabilized few years later. The bridge is currently being reconstructed.

![Fig. 5](image)

**Fig. 5** View of the reinforced concrete arch bridge in Komárno.

![Scheme](image)

**Fig. 6** Scheme of the concrete arch bridge in Komárno.

5 Visintini

The investigated bridge, which was built in a period between years 1910 and 1919 (the exact year is not known), was situated on the 3rd class road in Hungary, in the Nőiregyházá district, and a very similar bridge is still in service in Slovakia. The bridge in Hungary was a very rare bridge structure consisting of ten reinforced concrete truss girders with a span of 5 meters. The bridge was in service until 2012, (Fig. 7).
After the bridge was disassembled in 2013, one of its main truss girders was transported to Bratislava, to the Technical Building and Research Institute (TSÚS). Later, it was subjected to non-destructive tests followed by load-bearing capacity test and destructive tests on drilled core samples. The results provided valuable information about the structural state of this rare, approximately 100-years-old, reinforced concrete truss girder, as well as about the properties of concrete and reinforcement used for its construction.

Visintini type girders were commonly used in bridge construction industry between years 1910 to 1920 [3], [4]. The girders of this type, manufactured at the beginning of the 20th century, are described in detail in the article written by Ing. Špaček in 1908 [5]. In the article [5], the method of their production and their calculations, as well as the results of load-bearing tests carried out in Zurich, Krakow, Wroclaw and Paris, are described in detail. A period photograph of the transport of such a prefabricated bridge girder (Fig. 8) has been preserved in the book "Massivbrücken gestern und heute" [2] and captures the construction of a bridge in Erdmannsdorf (Germany) in 1910.

Fig. 7 View of the truss girders during bridge demolition in 2012 – after more than 90 years of service without any repair.

Fig. 8 Transport of the Visintini girder during the bridge construction in Erdmannsdorf (Germany) in 1910 [2].
Back in the days of the construction of these bridges, saving the amount of material and the weight of the prefabricated elements played a key role. The prefabricated product was light enough for manual transport by workers and did not require the use of a heavy-duty crane during bridge assembly. The reinforcement of the diagonals corresponded to the load they had to carry; thus, each diagonal had a different amount of reinforcement. Anchoring of the diagonals was performed by making hooks passing through the bottom reinforcing steel plate (this connection solution has slightly weakened the cross-section of the steel plate at the point where diagonals passed through it), (Fig. 9).

![Cleaned reinforcement of the truss girder (left) and detail of the connection of diagonals to bottom flange (right).](image)

During the load-bearing test with a uniformly distributed load, these joints behaved almost like perfect hinges (based on deformations). The failure occurred by rupture of the bottom steel plate in tension at the anchorage point of the diagonal bar, nearby the middle of the span. After load-bearing test, the reinforcement was exposed by removing the concrete layer and the state of the reinforcement nearby the joints was visually examined. No broken joints (diagonal – steel plate) were observed and the load was reliably transferred by them. The thickness of the concrete cover to reinforcement was 25 mm. The bridge did not leak during operation and even after more than 90 years of service was still in a good condition.

6 Krajinsky bridge

The Krajinsky bridge in Piešťany, built in the years 1930 – 1931, is one of the cultural monuments of Slovakia. It is a structurally efficient structure, which fits well into the surrounding environment. From the structural point of view, the Krajinsky Bridge is a reinforced concrete arch bridge with the biggest span reaching 52 m. The 29 meters long side spans are formed by arches supporting the bridge deck. Since the arches of the shorter side spans have a smaller rise to span ratio than the central arch, they produce almost the same horizontal load. By this means, the horizontal arch forces in between the largest span and the side spans are almost balanced. To cover the difference between these horizontal forces, the piers supporting the main span are more massive and have a slightly offset footing in the direction of the resultant of forces. The bridge appearance is very elegant in the surrounding environment and it has become one of the iconic structures of the Piešťany town, (Fig. 10).
This is a good example of a bridge, that has a structurally logical shape and at the same time it is aesthetically pleasing. The main span of the bridge served without major repairs for more than 80 years, with the reinforced concrete hangers in between the arch and the deck being the only weak point. These hangers are in the immediate vicinity of the road and have been splashed with water from passing cars. Such parts of structures are always severely affected by degradation mainly caused by deicing salts and freeze-thaw effects. For this reason, some hangers have been strengthened with additional steel rods.

7 Conclusions

Within the research, which was carried out in Slovakia, with a focus on the oldest reinforced concrete bridges [7], several bridges older than 90 years were examined (only bridges which were not reconstructed in the past were selected). From this research, it was possible to draw the following conclusions regarding the key factors that ensured their long service life:

- The 25 mm thick concrete cover to reinforcement was sufficient even in such cases, where carbonation of concrete was already a few centimeters behind the reinforcement. If there was no leakage at the bridge, the reinforcement corroded very slowly (without delamination of the concrete cover), even through it was many years in carbonated layer of concrete.
- All bridges older than 90 years had good functional drainage (with only a small or no leakage).
- Laboratory research has demonstrated, that in some cases a thin cement render (2 mm thick) was able to completely prevent carbonation of the underlying low quality concrete [6] (research on this topic is still going on).

Even though Slovakia is a very small country, it has many very interesting and structurally exceptional bridges, that have been forgotten about. Many of them can serve as an inspiration during conceptual design of new structures, and it is therefore necessary for young engineers to learn about them.

Acknowledgements

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Railway bridge engineering: lessons learned over the past 100 years

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Abstract
Hundreds of historic railways bridges in Austria, which are still in operation today, are remarkable examples of durable infrastructure. They withstood technical progress, environmental impacts and man-made catastrophes like world wars during their long service life. So, what can we learn from historic railway bridges? Which principles of the past can be transferred into the context of modern infrastructure? Which failures should be avoided in future? The following article describes diverse aspects and fundamentals of railway bridge engineering in the course of time. It covers a broad spectrum from the beginning of railway engineering up to the current activities in realization of high-performance railways.

1 Introduction
The origins of public railways in Austria go back to the 19th century. Within short time, large and challenging railway tracks were built like the Southern line or the Tauern Line. Historic bridges like the Viaduct Semmering or the Danube Bridge in Linz are remains of those pioneer days (Figure 1). It is remarkable that despite of increased demands regarding wheel load and velocity, these bridges are still in safe operation today. [1]

Design and construction of railway bridges has always been done with a high degree of quality awareness and smart generosity. Railway bridges are built for a service life of 100 years and therefore, high initial quality is crucial – regarding design, calculation, and fabrication. In the following chapters, various aspects are introduced, which influence the realisation of durable and long-living railway infrastructure.

Fig. 1 Left: Kalte Rinne Viadukt Semmering, Austria, built 1857 by Carl Ritter von Ghega. Right: Steyregger Donaubrücke Linz, Austria, built 1926 by company Waagner Biró & Kurz. Photo: Deopito.

2 Type
Bridges are necessary, where transport routes like rivers, roads or rail lines intersect. Railway operators differentiate between railway bridges and bridges over railway lines. Railway bridges lead tracks over barriers and have railway-specific design and equipment. Bridges over rail lines are related to other transport routes, but also have a strong influence on railway safety and reliability (Figure 2).
Currently, the Austrian railway network includes 6,580 bridges, which are classified into railway, road, pedestrian, and pipework bridges. By definition according to the rulebook, bridges have a minimum span of 2 m. Constructions with a smaller span are defined as culverts and have different design and maintenance rules. [2], [3]

Fig. 2  Left: Protected historic pedestrian bridge in Kritzendorf, Austria, built in 1889 and renovated in 2017. Right: Bridge chain over the central switchyard Kledering, Austria, built in 2013. Photo: Deopito.

3 Material

The usage of robust and durable materials is crucial for long-living infrastructure. In addition, the requirement of sustainability gains in significance today. Bridge materials require high strength, longevity, and resistance against aging and unannounced fracture. Whereas at the beginning of railway engineering bridges were mainly made of stone and steel, the dominating material is reinforced concrete today (Figure 3). In situ concrete can be formed to various shapes like straight girders, curved arches, or edged trusses, which allows high freedom of design. Another reason for the dominance of cement-based materials is the lower maintenance expenditure compared to corrosible metals. [4]

In modern railway bridge engineering, steel is mainly used for specific applications like wide-spanned framework or composite bridges. Since steel structures are relatively slender and lightweight, they also provide benefits during construction phase with shorter rail track blockings. Furthermore, steel has advantages over concrete in the context of reutilisation. Considering sustainability in the material decisions might cause a shift from concrete to steel again. [5]

While timber was used at the beginning of railway engineering especially in the dry regions of the USA, it is not widespread anymore today. The main reason is the material’s fast loss of strength in case of moisture penetration. However, the ongoing greening of civil engineering might intensify timber constructions in infrastructure again.

Fig. 3  Distribution of bridge surface (in square metres) by material (dome / concrete / steel) and construction year in the rail network of ÖBB-Infrastruktur AG.
4 Structure

The determination of structural system is usually done in early project phase. This definition has a strong influence on all following design issues. Therefore, later changes of the structural system are complex. A distinction can be drawn between bridges with the load-bearing structure arranged above or below the railway (Figure 4). Whereas the first type has advantages regarding clearance gauge below the bridge, the second type is more robust and cheaper in realization.

Since the beginning of railway engineering, details and principals of design and maintenance were standardized in technical codes to build and operate efficiently. Therefore, the freedom in conceptual design of railway bridges is smaller compared to pedestrian or road bridges. The structural system is closely connected with material and manufacturing process. A high degree of prefabrication increases the design efforts but improves execution quality and promotes efficient material usage.

5 Functionality

The combination of road and railway bridges is unusual. Few examples of combined road and railway bridges demonstrate that their combination is rather inefficient. The reason is that road bridges have higher frost and icing salt exposure, which leads to lower service life and different maintenance intervals. In addition, the extraordinary high traffic loads of railway bridges require a much higher degree of structural stiffness than road bridges.

Whereas overlapping functionalities are common in road engineering with combined infrastructure for walkers, bicycles, and automobiles, it is unusual in railway engineering. The main reason for functional separation are the high requirements, which are connected with railway operation, concerning track system, overhead cables or safety devices. An exception of functional separation are modern railway stations, where issues like passenger safety and satisfaction must be implemented in bridge design (Figure 5).

Fig. 4 Left: Trisanna bridge on Arlberg, Austria, built 1884 and renovated in 1964. Right: Protected historic viaduct over Dimbach in Upper Austria, built in 1909 and renovated in 2020. Photo: Wett & Windhager.

Fig. 5 Left: Shopping City Vienna Main Station opened in 2014. Right: Conversion of the railway station Braunau finished in 2020. Photo: Kaiser & Deopito.
6 Aesthetics

Another finding of historic railway bridges is that striving for elegance is essential in civil engineering. Bridge engineers should always keep in mind that infrastructural constructions are representative parts of built environment. Therefore, aesthetic standards are mandatory (Figure 6). Fineness starts with small details like the design of joints and continues on bigger scale like the shape of an arch. [6]

In addition, material surface, texture and edge formations require detailed design to obtain appealing appearance for many years. There is a strong relation between building condition and aesthetics. Rusty steel constructions are not only structural deficits, but also create an unattractive appearance.

![Fig. 6](image1)

**Fig. 6** Left: Hammelbachbrücke Bernhardsthal (Austria) built in 1838 by Carl Ritter von Ghega. Right: Rheinbrücke Lustenau-Sankt Margrethen (Austria / Switzerland) designed by Bernhard Ingenieure in 2013 (right). Photo: Deopito & Wett.

7 Analysis

A principal in structural engineering is that the more detailed a calculation is performed, the higher utilization is acceptable. Therefore, sophisticated calculations go hand in hand with economical use of resources in railway bridge engineering. Whereas consistent Eurocode standards are obligatory for the design of new railway bridges, the assessment of existing structures is still performed by national standards and guidelines.

Due to intensive railway activities in Austria at the beginning of the 20th century, a significant number of railway bridges has reached a service life of 100 years currently. This is a respectable age and requires accurate recalculations if they should be kept in service without restrictions in maximum load or velocity. Therefore, the residual life assessment via smart monitoring and structural mechanics is a current research focus in railway bridge engineering. [7], [8]

The structural assessment of historic steel bridges requires a deep understanding of rivet constructions. Actual research results reveal that riveted bridges have remarkable good fatigue behaviour, which strongly depends on the rivet’s pressing forces (Figure 7). [9], [10], [11]

![Fig. 7](image2)

**Fig. 7** Left: Cut through a historic rivet joint. Right: Simulation of a rivet joint via FE-analysis: the cyan areas of compression have positive influence on the fatigue behaviour (University of Technology Graz – Institute for Structural Steel).
8 Quality

Quality awareness has always been an important aspect of railway engineering. Building constructions with 100 years of service life need highest initial quality. Current studies of historic steel bridges show surprisingly good material characteristics, which indicates that already 100 years ago, railway engineers only used the best materials available (Table 1 and 2).

Also, in concrete engineering the design standards intensify the focus on quality assurance currently. In dependence of the damage consequence class, the latest Eurocodes demand a minimum level of quality control in design and execution. Third part audits and a continuous 4-eye principle become obligatory for critical infrastructure.

Already in early phase of design, questions regarding inspection and maintenance must be considered. Supervision is mandatory including external audits and self-monitoring. Dual testing with monitoring concepts and continuous documentation become binding.

Table 1 Mild steel: Mean value of chemical components (mass %) depending on the production process. [12]

<table>
<thead>
<tr>
<th>Element</th>
<th>Puddle Steel</th>
<th>Bessemer Steel</th>
<th>Thomas Steel</th>
<th>Siemens-Martinent Steel</th>
</tr>
</thead>
<tbody>
<tr>
<td>Carbon C</td>
<td>0,025</td>
<td>0,094</td>
<td>0,048</td>
<td>0,090</td>
</tr>
<tr>
<td>Silicon Si</td>
<td>0,147</td>
<td>0,101</td>
<td>0,009</td>
<td>0,008</td>
</tr>
<tr>
<td>Manganese Mn</td>
<td>0,213</td>
<td>0,491</td>
<td>0,462</td>
<td>0,477</td>
</tr>
<tr>
<td>Phosphor P</td>
<td>0,396</td>
<td>0,047</td>
<td>0,051</td>
<td>0,035</td>
</tr>
<tr>
<td>Sulfur S</td>
<td>0,061</td>
<td>0,047</td>
<td>0,044</td>
<td>0,038</td>
</tr>
<tr>
<td>Nitrogen N</td>
<td>0,009</td>
<td>0,014</td>
<td>0,014</td>
<td>0,005</td>
</tr>
</tbody>
</table>

Table 2 Material analyses (mass %) of historic steel railway bridges in Austria (year of construction 1906 – 1914) indicate the usage of Siemens Martin Steel. [12]

<table>
<thead>
<tr>
<th>Element</th>
<th>Sample AIT 1</th>
<th>Sample AIT 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Carbon C</td>
<td>0,047</td>
<td>0,028</td>
</tr>
<tr>
<td>Silicon Si</td>
<td>0,013</td>
<td>0,002</td>
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<tr>
<td>Manganese Mn</td>
<td>0,363</td>
<td>0,301</td>
</tr>
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<td>0,009</td>
</tr>
<tr>
<td>Sulfur S</td>
<td>0,018</td>
<td>0,020</td>
</tr>
<tr>
<td>Nitrogen N</td>
<td>0,005</td>
<td>0,005</td>
</tr>
</tbody>
</table>

9 Inspection

Efficient maintenance is crucial for critical infrastructure, and life cycle approaches must be considered in early phase of design. How can vulnerable parts of a construction be assessed, and areas monitored, which are not accessible during operation? Deterioration of steel bridges is mainly cause by corrosion and fatigue and depends on traffic load and servicing quality of corrosion protection systems. Deterioration of concrete bridges is mainly caused by carbonation and chloride penetration. Both effects interact and have a non-linear progress. [13]

Historic bridges teach us that efficient and handy accessibility correlates with inspection diligence and quality. Especially historic steel bridges are characterized by a high degree of transparency. Structural health monitoring via automatized systems is an ongoing research focus for several years as a useful addition to human inspections. Prototypes of drones and acoustic or optics monitoring systems show promising results. However, the implementation into a reliable maintenance process is complex and requires further investigations. [14]
10 Rail Operation

Building railway bridges under operation is demanding. Punctuality and reliability have highest priority in modern infrastructure and rail track blockings must be avoided as much as possible. Support measures and construction phases need to be considered in detail (Figure 8). Railway projects in the open countryside have less rigorous operational boundary conditions than modifications of existing railway systems.

![Fig. 8](image)

**Fig. 8** Left: Hoisting of a historic steel bridge during rail operation. Right: Shifting of a pedestrian tunnel made of concrete under rail operation. Photo: Deopito.

11 Innovation

Bridge engineers orientate towards the latest state of technology and desire the highest initial quality. Research and development are realized by long-term collaboration with universities and industry. Therefore, bridge builders and operators need expert knowledge to ask smart questions and define appropriate requirements. Sometimes, the unconditional desire to create sustainable solutions is also connected with the courage to rework or reject ideas.

Railway engineering has always been a driver of innovation. For example, August Wöhler (1819 – 1914) was an outstanding railway engineer, who developed the mechanical basics of steel fatigue more than 100 years ago. The investigation became necessary after several railway accidents due to broken train wheels. In his work, he identified that steel strength under cyclic load is much lower than under static. [15]

Today, the Austrian railways also encourage and support new technologies in civil engineering. For example, this regards the implementation of new materials like Ultra-High Performance Concrete (UHPC) for light-weight concrete trough bridges, or the experimentation of innovative construction methods like concrete shells made of pneumatic formwork (Figure 9).

![Fig. 9](image)

**Fig. 9** Left: Provisional bridge made of Ultra-High-Performance Concrete by University of Technology Graz – Institute for Structural Concrete [16]. Right: Realization of a wild crossing via pneumatic formwork by University of Technology Vienna – Institute for Structural Concrete [17].

12 Conclusion

Safety, reliability, and punctuality are the fundamentals of railway engineering for more than hundred years. Railway bridge design is influenced by various aspects, which have not changed principally in
the last 100 years. Railway bridges are characterized by remarkably high service load and major consequences in case of damage. Therefore, high structural stiffness is crucial in connection with precise and straight load flow.

Although the design freedom is limited in railway bridge engineering compared to other building tasks, the conceptual design phase is decisive. Inaccurate decisions become difficult to fix in later stages of design. Close attention to detail is precondition to ensure durable and low-maintenance assets. Efficient maintenance requests reliable data to ensure system availability. Therefore, the connection of local monitoring with smart structural mechanical models seems to be a promising application.

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behind the curtain
A new tool for the conceptual design of structures in equilibrium based on graphic statics

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Link to the video: https://youtu.be/9_3iyEHmEy0
An interactive implementation of algebraic graphic statics for geometry-based teaching and design of structures

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Abstract
This paper presents an interactive implementation of graphic statics, which can be integrated into a CAD environment. Graphic statics is a well-known design and analysis method for two-dimensional discrete structures that relies on geometrical rather than analytical representation of the relation between the structure's geometry and the equilibrium of its internal forces. The method was formalised in the 19th century, but slowly disappeared from structural engineering practice over the 20th century. Recent developments have introduced Algebraic Graph Statics (AGS), which formulates the geometrical relationship between the graph representations of the reciprocal form and force diagrams in graphic statics using linear algebra. AGS and its extensions enable automatic construction of force diagrams from given form diagrams, and allow a few basic modifications of the force diagram from which the form diagram is updated. This paper builds on the previous work of AGS by implementing a real-time, bi-directional workflow allowing users to impose various constraints, and perform geometrical modifications in either the form or force diagram from which the other is automatically updated by using an iterative geometric solver. The presented implementation of interactive AGS provides a robust computational back-end to harness the advantages of traditional graphic statics for geometry-based teaching and design of structures.

1 Introduction
Recent research has demonstrated how the principles of graphic statics can be combined with computational tools to create interactive drawings that provide visual feedback to the user in real time [1-3]. Such interactive implementations of graphic statics have not only introduced new and effective methods of teaching structural design [4], but also enabled advanced research [5, 6]. Despite its numerous benefits, interactive graphic statics drawings still have some major drawbacks. The tedious and time-consuming process of constructing drawings in a procedural manner requires previous knowledge and experience with graphic statics [7, 8]. More importantly, each drawing is representative of just one instance of a structure, meaning that topological changes to the design require a complete redraw of the form and force diagrams. Algebraic Graph Statics (AGS) introduced an algebraic method of formulating the reciprocal relationship between the form and force diagrams, which enables automatic construction of force diagrams from graph representations of form diagrams given by the user [9]. “Bi-directional” AGS extended the method, allowing geometric transformations of a force diagram that result in an automatic reconfiguration of the corresponding form diagram [10]. Other methods for generating reciprocal diagrams have been presented using Airy stress functions [11] and projective geometry [12], but these limit to self-stressed structures and add 3D polyhedral geometries into the workflow.

This paper presents a computational implementation and extension of previous research in AGS in an interactive design workflow. In order to create a fluid user experience while maintaining a robust back-end of solvers, various rules and constraints of graphic statics construction are explicitly defined, formulated and incorporated in an integrated computational pipeline using the COMPAS framework [13]. The examples presented in this paper demonstrate how the proposed implementation can be used to maximise the inherent benefits of graphic-statics-based structural-design explorations in a smooth and intuitive manner through controlled modifications, while minimising the need for manual construction of form and force diagrams.
2 State of the art

The fundamentals of AGS are clearly laid out in [9]. The main procedure in AGS identifies the degrees-of-freedom (DOF) of a system $k$, which is defined by the number of independent edges, which reflect the degree of indeterminacy of the form diagram. The loads in the independent edges can be chosen freely by the user such that the forces in all other edges of the form diagram are back-calculated, from which the geometry of the reciprocal force diagram can be computed (Fig. 1a). However, as stated in [9], the inverse problem of computing the form diagram from a given force diagram was not introduced. This problem requires a constrained optimisation procedure that is not simply the reverse implementation of form-to-force construction.

Vedad et al. [10] presented a pipeline to address this inverse problem for some cases (Fig. 1b). It allows controlled manipulations of the force diagram, from which a new form diagram is iteratively computed with a root-finding procedure based on the Newton method. Although this method addressed the inverse problem for some cases, fully bi-directional modifications of both form and force diagrams are still not possible. The method in [10] requires the user to provide the exact geometry of a valid force diagram, which is not straightforward or apparent in most cases since uninformed movements of vertices may result in an invalid equilibrium configuration. Moreover, constraints can only be applied to the form diagram, which inhibits force-driven explorations.

The goal of this paper is to develop a workflow that enables a fully bi-directional interaction between form and force diagrams. This workflow enables not only controlled modifications to both diagrams, but it also allows for automatic modifications inherited from constraints intuitively applied to them (Fig.1c).

Fig. 1 Overview of the a) AGS procedure [9], b) “Bi-directional” AGS procedure [10]; and c) Interactive AGS, presented in this paper.

3 An interactive implementation of algebraic graphic statics

This section provides an overview of the workflow and computational setup of the interactive implementation of AGS.

3.1 Workflow

The workflow of interactive AGS (Fig. 2) can be summarised in the following steps.

a) Input pattern for the form diagram, including externally applied loads and reaction forces, as a network of line segments.
b) Identify the supports.
c) Check the DOF $k$ and assign loads to at least $k$ (independent) edges in the diagram. If more than $k$ edges are selected, the additional loads are treated as “target loads” (see Section 3.2); as a convention, positive values indicate forces in tension, and negative in compression.
d) Create the form diagram $G$. 

Fig. 2 Interactive AGS workflow.

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e) Compute reciprocal force diagram $G^*$ and inherit default constraints from form diagram boundary conditions.

f) Interactively modify and add constraints to the form diagram.

g) Interactively modify and add constraints to the force diagram.

h) Solve equilibrium by parallelisation.

Fig. 2 Overview of the interactive AGS workflow.

3.2 Constraints

During procedural constructions of form and force diagrams, various graphic statics constraints are intrinsically built into the drawings (i.e., definition of load lines, placement of the diagrams, etc). However, when the initial form diagram is generated from a network of line segments (Fig. 2d), these constraints need to be explicitly identified, defined and imposed onto the diagrams. Therefore, when the force diagram is obtained (Fig. 2e), the following constraints are automatically imposed (Fig. 3):

1. vertices selected as supports in the form diagram will have their x and y coordinates fixed;
2. the vertices in the form diagram with an externally applied load are constrained to remain on the line of action of the load;
3. edges representing the reaction forces have their orientations fixed in both diagrams; and,
4. edges representing the externally applied loads have their orientations fixed in both diagrams, and their lengths fixed in the force diagram.

Fig. 3 Imposing default constraints from boundary conditions.
In addition to the above-listed default constraints, the following constraints can be applied to or removed from the form or force diagrams at any point during the workflow (Fig. 2f, 2g):

- vertex partial fixity in the x-direction, which restraints the vertex horizontally;
- vertex partial fixity in the y-direction, which restraints the vertex vertically;
- edge target orientation, which constrains the corresponding edges of both diagrams to align with such orientation;
- edge target force in the form diagram, which constrains the force of the edge in the form diagram and the length of the corresponding edge in the force diagram.

3.3 Solver

This section describes the solving strategies to compute the initial force diagram (Fig. 2e), and the parallelisation of form and force diagrams respecting their constraints (Fig. 2h).

3.3.1 Algebraic computation of force diagram

The coordinates of the initial force diagram \((x^*, y^*)\) can be computed linearly from the force densities \((q)\) of the form diagram's edges and the geometry of the form diagram as in [9] with the following equations:

\[
\begin{align*}
L'x' &= C'Qu, \\
L'y' &= C'Qv \\
\end{align*}
\]

where, \(C\) and \(C^*\), are the connectivity matrices for the form and force diagrams, respectively. \(Q=\text{diag}(q)\) represents the matrix of force densities, and \(u, v\) are the coordinate difference vectors of the form diagram's edges in \(x\)- and \(y\)-direction, respectively. The vector of force densities is computed from the geometry of the form diagram and the value of force densities in the independent edges. Although the force diagram can be computed with this procedure (Fig. 2e), only the forces assigned to the set of independent edges are considered, and additional constraints such as target forces to other edges cannot be imposed. The geometry of the form diagram is fixed from the start and no interactive modifications can be performed once the force diagram has been created.

3.3.2 Parallelisation of form and force diagrams

A parallelisation algorithm is developed to allow interactive modifications of form and force diagrams respecting their constraints (Fig. 2h). The process can be divided in two consecutive steps: (i) the force diagram is updated iteratively to reflect the constraints imposed; and (ii) form and force diagrams are updated by solving the least-squares intersection of lines. These two steps are executed interatively until form and force diagrams are reciprocal and the constraints are respected, or until the process hits a maximum number of iterations. The two steps are described in detail below:

i. Updating constrained force diagrams

To update the vertex coordinates of the force diagram, the following algorithm based on target lengths and orientation vectors is adapted from [14]. The algorithm will update the vertex coordinates of the force diagram taking into account only the edges that have constraints assigned to them. Let \(E_i^{constr}\) represent the group of edges \(e_{ij}\) connected to a vertex \(v_i\) of the force diagram with constraints assigned to them. Edge \(e_{ij} \in E_i^{constr}\) can be constrained to a target length \(l_{ij}^t\), and/or to a target orientation vector \(\hat{t}_{ij}\). For each iteration, the coordinates of \(v_i\) are updated with barycentre \(p_i\), which is computed by the following equation:

\[
p_i = \frac{\sum_{j \in E_i^{constr}} (v_i^* + l_{ij}^t \cdot \hat{t}_{ij}^*)}{n(E_i^{constr})},
\]

where if edge \(e_{ij}\) is constrained exclusively to a target length \(l_{ij}^t\), the target vector \(\hat{t}_{ij}\) is the unit vector pointing from \(v_i^*\) to \(v_j^*\), and, if \(e_{ij}\) is constrained only to a target vector \((\hat{t}_{ij})\), \(l_{ij}^t\) is the edge's length.

Once the force diagram is updated, respecting the constraints, form and force diagrams are not guaranteed to be reciprocal anymore (i.e., corresponding edges may not be parallel). An additional procedure is needed to parallelise the corresponding edges of the two diagrams by updating their vertex coordinates.
ii. Least-squares parallelisation of form and force diagrams

Form and force diagrams are updated by solving a least-squares problem for the intersection of lines [15]. This procedure is applied sequentially first to the form and then to the force diagram. The procedure applies similarly to both diagrams. When it is applied to the form diagram, the orientation of the edges of the force diagram are considered; and, conversely, when it is applied to the force diagram, the directions of the edges of the form are used. It is thus sufficient to explain the iterative process by only considering, for example, the form diagram, which is done next:

In that case, the ideal position \( x_i, y_i \) of vertex \( v_i \) of the form diagram connected by \( m_i \) edges to each neighbour vertex \( v_j \) with coordinates \( x_j, y_j \), can be found by solving the following system:

\[
A_i \begin{bmatrix} x_i \\ y_i \end{bmatrix} = b_i,
\]

with:

\[
A_i = \sum_{j=1}^{m_i} \left( (1 - \hat{t}_{ij}^* \hat{t}_{ij}^T) \right), \quad b_i = \sum_{j=1}^{m_i} \left( (1 - \hat{t}_{ij}^* \hat{t}_{ij}^T) \right) \begin{bmatrix} x_j \\ y_j \end{bmatrix},
\]

in which \( \hat{t}_{ij}^* \) is the orientation of the edge’s corresponding dual edge in the force diagram.

4 Applications

This section presents graphic-statics applications of the proposed implementation of interactive AGS through a series of examples.

4.1 A “bad” input: form finding of an arch

The first example deals with the well known problem of form finding of an arch subjected to an equally distributed vertical load. When constructed procedurally, the geometry of the arch can be found step by step. In an AGS-based workflow, the user needs to provide the geometry of the “correct” arch from the beginning, which is not always easy or convenient to construct without the force diagram. This example shows how given a “bad” starting form diagram geometry, imposing proper constraints can result in correct form and force diagrams with the desired boundary conditions.

The initial form diagram \( G \) represents a semi-circular arch (Fig. 4a). Following the convention of AGS [9], the external forces are input as edges to the form diagram (applied loads in green, reaction forces in cyan). The two internal vertices in the extremity of the arch are defined as supports (shown in black). This system has DOF of only one, which means that a desired force magnitude can be assigned to only one independent edge. A force magnitude of -1.0 is assigned to the independent edge (highlighted in bold in Fig. 4a) resulting in the initial force diagram \( G^* \). As expected, the magnitudes of the other applied loads are not 1.0, which signifies that the semi-circular arch does not have the correct geometry for an equally distributed loading case.

![Fig. 4](image-url)

Fig. 4 a) Initial form \( G \) and force \( G^* \) diagrams of a semi-circular arch, where a force magnitude of -1.0 is assigned to the highlighted independent edge; b) updated form \( G \) and force \( G^* \) diagrams after imposing target force magnitudes to the applied loads, resulting in a parabolic arch subjected to equally distributed vertical load.
As introduced in Section 3.2, target force magnitudes can be imposed on the form diagram, which, as consequence, will be reflected as target lengths in the force diagram. Therefore, in order to assign equally distributed vertical loads to the arch, a target force -1.0 is applied to the loaded edges in the form diagram, or equivalently, target lengths of 1.0 to the loaded edges in the force diagram (Fig. 4b). With the default constraints from the boundary conditions already imposed, the dual equilibrium is performed updating both form and force diagrams. The resulting form and force diagrams in Fig. 4b now shows the “correct,” parabolic arch subjected to equally distributed vertical loads. Fig. 4b corresponds to one of the possible parabolic arch solutions, which depend on the magnitude of the unconstrained horizontal reactions, which in this case is equal 3.81 after the interactive parallelisation. Controlling this horizontal magnitude to alter the arch height will be discussed in the next example.

### 4.2 Form and force diagram modifications

The second example shows how the geometry of the arch from Fig. 4b can be modified through controlled translations of the vertices of both diagrams. After the transformations, the new diagrams are then parallelised (Section 3.3.2). Fig. 5 shows two possible manipulations on the force diagram. In Fig. 5a, the three vertices on the left side of the force diagram are dragged to decrease the magnitude of the internal forces, resulting in a taller arch. In Fig. 5b, the vertices are moved further to the right, such that the form diagram results in a geometry that corresponds to a funicular cable in tension.

![Fig. 5](image_url)

a) Moving the three vertices on the left side of the force diagram $G^*$ to the right, which results in reduced internal forces and therefore a taller arch in form diagram $G$; b) further movement of three vertices until the forces flip from compression to tension, with form diagram $G$ becoming a funicular cable.

Fig. 6 shows two examples of modifications in the vertices of the form diagram. In Fig. 6a, an internal vertex of the arch is moved up and its $y$ coordinate is fixed, which constrains the arch to pass through this point. The target force magnitude constraints still apply, i.e., the loading case is equally distributed. After the translation of the internal vertex in the form diagram, both diagrams are updated (Fig. 6a). Similarly, in Fig. 6b, the right support of the arch is moved up. After this modification, both diagrams are updated while respecting all imposed constraints, resulting in an arch with uniform loads applied to it, but with uneven vertical force reactions due to the different support heights.

![Fig. 6](image_url)

a) Update in form $G$ and force $G^*$ diagrams generated by moving, and constraining an internal vertex of the arch, controlling its structural height; b) update in form $G$ and force diagrams $G^*$ generated by moving one of the supports.
4.3 Constraint-based form finding: Constant edge forces

The last example discusses the case of the truss depicted in Figure 7a. Supports are assigned to the two extremes of the truss, and the load is equally distributed to the bottom chord of the truss, having an individual force magnitude of +1.0. The initial form and force diagrams are computed in Figure 7a. In this example, a new geometry of the truss will be computed such that the compression forces in its arching edges become constant. A target force magnitude of -2.5 is imposed to these edges as well as the vertex fixities along the bottom chord of the truss, constraining them to remain horizontal. The parallelisation algorithm results in the truss depicted in Figure 7b, which has constant compression force along the arching edges of the truss and non-constant tensile forces in the horizontal bottom chord. As a consequence, the struts connecting the arching top edges and bottom horizontal chords of the truss are no longer vertical.

![Fig. 7](image)

The constant-force truss problem can be discretised further, as depicted in Figure 8a. Further modification is applied to the vertices along the bottom chords of the truss, which are constrained to follow a sinusoidal curve. The new geometry of the truss can be found with the presented implementation while always maintaining the constant-force constraint in the top arching edges of the truss and the same loading case. Different variations of the constant-force truss obtaining by accentuating the curve on the bottom chord are depicted in Figures 8b-d.

![Fig. 8](image)
5 Conclusions

This paper presented a fully bi-directional implementation of AGS by enabling users to impose various constraints to both form and force diagrams in an intuitive and interactive manner. With the introduction of an iterative geometric solver that simultaneously updates form and force diagrams, a flexible and robust application of computational graphic statics is made possible in a design-oriented workflow.

This implementation is a step forward in establishing an interactive tool incorporated in a CAD environment with a graphical user interface that can leverage the power of graphic statics for geometry-based teaching and designing of structures. This open-source tool is currently in development and will be made available soon to the scientific community. Future work will focus on incorporating various geometric objects as constraints, as well as enforcing adaptive target force magnitudes.

References

Component reuse in structural design: emerging practices and tools for the circular economy

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Link to the video: https://youtu.be/VCyD1yqXH8w
Algorithmic circular design with reused structural elements: method and tool

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Abstract
Structural systems are responsible for a significant portion of embodied carbon emissions in buildings. A potential path to increase sustainability is to integrate circular economy principles in structural design, which advocate for prioritizing the reuse of structural materials to extend their service life, limiting their physical transformation to locational and functional changes. In this way, structural projects of the past can not only serve as an inspiration for the future, but the material itself can also be reappropriated. Recently, computational approaches for material reuse have gained traction. This paper extends previous work by comparing several algorithmic formulations for reuse-driven design, introducing a new Grasshopper-based tool that implements them, and demonstrating their application on a case study.

1 Introduction
The Intergovernmental Panel on Climate Change (IPCC) states that the building sector needs to be “zero-carbon” by 2050 to meet the targets set by the Climate Agreement and avoid extreme climate catastrophes. The construction sector, accounting for 13% of the world’s GDP [1], uses 50% of all materials [2], [3], generates 36% of the waste [4], and emits up to 12% of global greenhouse gas emissions for building material extraction, manufacturing, and construction, counting only Europe [5]. This is due to the sector’s linear model, which extracts, produces, uses, and disposes of building materials and resources. To remediate this detrimental condition, worldwide, a transition to a circular repair-reuse-recycling model is urgently needed in today’s construction sector [6], [7]. A circular model would extract maximum value from building materials by extending their service life or reusing them at the end of their service life as new resources, while minimizing their environmental impact. Due to rapid urbanization, it has become more attractive to demolish buildings and rebuild new ones rather than deconstructing them and reusing their materials. Global implementation on a large scale of a circular model in the construction sector has not yet been successful. The fragmented supply chain in the Architecture, Engineering and Construction (AEC) sector prevents both the broad application of circular strategies in construction practice. This could be addressed through the uptake of digital tools such as computational design tools and databanks of materials. This paper proposes new algorithms and design methods that enable the use of buildings instead of the earth as material mines and depots.

Building on recent developments in computational approaches for helping designers reuse materials [8], [9], this paper considers the design of structures built with reused materials. The proposed method assesses the capacity of a newly generated design to use materials from a stock of available materials from reuse. To test and validate the algorithm and method, the linear timber structure elements of a conventional house are inventoried and reused in the design of geodesic domes (Fig. 1), which are clad and used as greenhouses. This illustrates how algorithmic matching of reused materials can be integrated into the design workflow of architects and engineers through quick computational feedback. The case study in this paper uses timber elements as working with this material is accessible to most constructors with relatively simple tools.
2 Material reuse approaches

Many inspiring projects have illustrated the feasibility of material reuse in various contexts: the reuse of 180 pieces of bent glass from the Centre Pompidou's façade in Maximum's architectural project in Paris, France [10]; the use of waste material as lost formwork in filler slabs such as the 2000 Wall House project by Anupama Kundoo in India [11]; to name a few.

![A geodesic dome example built with renewable materials (Credit: N. Petit-Barreau, Anku).](image)

Fig. 1 A geodesic dome example built with renewable materials (Credit: N. Petit-Barreau, Anku).

2.1 Computational methods for material reuse

Despite these contemporary examples, material reuse is not the norm in today’s design and construction practices. This is partly due to the added challenges that working with existing, often irregular material resources bring. Instead of designing in an unconstrained manner with an assumption of infinite material supply, architects and engineers who wish to reuse existing material must devote time, creativity, and flexibility to devising form and space with a geometrically and structurally constrained kit of parts of limited size. These challenges may be surmountable in boutique projects, but are hard to address at scale in everyday construction.

One response to these challenges is the use of computational methods, which can use automation to assist in designing with a fixed material inventory. Already common in architectural design for non-reuse cases, computational design methods such as parametric design space exploration and rule- or grammar-based design approaches can be productive generative tools for material reuse. While there are a variety of methods used in previous literature and discussed in this section, there are two fundamental design philosophies.

The first, bottom-up design, starts with available material objects and algorithmically aggregates them into architectural assemblies. Computationally, it has its origins in shape grammars [12] and more recent work in making grammars [13], and uses predefined rules to automate and control the process of aggregation. This approach has the advantage of a guaranteed geometric fit of the existing material into the new design, and more naturally follows a non-computational physical workflow, e.g. building with blocks. The challenge of this approach is that it can be very hard to control the resulting design, and to meet any additional design intentions, such as overall formal, spatial, and structural goals. This approach is also more easily adapted to inventories of self-similar parts, e.g. dimensional lumber, than truly diverse material stocks that are of interest in reuse. Examples of this method include [14]–[16].

The second philosophy, followed in this paper and others before it, is top-down design. This approach has its roots in conventional parametric design and optimization, and starts with a “target” design concept model. An inventory of available construction elements is algorithmically searched, and parts are selected and matched to the target, ideally in terms of both geometrical fit and structural capacity. Typically, the matching is not perfect, and the inventory elements must be processed in some way to be used in the final construction. Various algorithms have been used to conduct and optimize this matching process (and minimize processing and waste), as shown in Table 1. Heuristic algorithms such as Greedy Search are simpler to implement but not guaranteed to result in the best match. More rigorously formulated optimization algorithms can be slow.

If the matching process is fast enough, it can be used within or in addition to other design considerations, such as overall form. Then, the overall design can be modified or optimized to most closely fit the available material inventory. This has both practical and conceptual appeal as it has a similar philosophy to traditional form-finding, which attempts to minimize material mass; rather than imposing
their abstract formal ideas on architectural problems, designers can discover geometries that meet important performance goals. In previous work, this has been demonstrated in [9], [17], [18].

Table 1 Summary of previous work in computational assignment optimization of reclaimed structural elements. *N/A means that structural capacities of the inventory elements are not considered in the matching.

<table>
<thead>
<tr>
<th>Reference</th>
<th>Structural application</th>
<th>Problem</th>
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<th>Algorithm</th>
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<tbody>
<tr>
<td>Mollica and Self 2017 [20]</td>
<td>Arched truss with tree fork connectors</td>
<td>Inventory matching</td>
<td>*N/A</td>
<td>Greedy Search</td>
</tr>
<tr>
<td>Bukauskas et al. 2017 [22]</td>
<td>Trusses with linear elements</td>
<td>Inventory matching with structural mechanics and cutting stock optimization</td>
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</tr>
<tr>
<td>Brütting et al. 2018[17]</td>
<td>Trusses with linear elements</td>
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<td>Lokhandwala et al. 2018 [24]</td>
<td>Funicular shells with planar polygonal panels</td>
<td>Inventory matching (2D polygonal packing)</td>
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<tr>
<td>Allner et al. 2020 [14]</td>
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<td>Brütting et al. 2020 [8], [23]</td>
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<td>Inventory matching with structural mechanics and cutting stock optimization</td>
<td>Simultaneous</td>
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<tr>
<td>Amtsberg et al. 2021 [9]</td>
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<td>this paper</td>
<td>Grid shells with linear elements</td>
<td>Inventory matching with structural mechanics</td>
<td>Separated</td>
<td>Hungarian Algorithm</td>
</tr>
</tbody>
</table>

2.2 Research gap

This paper extends previous work by introducing a new computational approach to design for material reuse in a flexible, interactive, designer-driven workflow. Central to this approach is the use of the classical Hungarian Algorithm, first introduced in 1955 [25], which has only been applied to the material reuse problem twice before in previous literature [21], [9]. This paper is the first to demonstrate the Hungarian Algorithm as implemented in a free, open-source tool for Grasshopper, which is used together with recently introduced tools for design space exploration, including sampling and single- and multi-objective optimization. Compared to previous work, which has formulated the reuse problem in mathematically rigorous but rigid ways, this paper proposes a modular approach that can be adjusted.
iteratively and quickly based on the evolving priorities of the design team. This workflow is tested and analyzed in a case study design problem to further understanding of algorithmic material reuse for sustainable architecture.

3 Methodology and case study

The reuse approach proposed by this paper is summarized in Fig. 2. Source material is identified and digitally processed into an inventory, and a parameterized design model is created and linked to structural analysis to generate possible target designs with computed axial forces. A cost matrix is assembled that includes all possible pairwise matches between the inventory and target, considering both geometry (member length) and structural mechanics (tensile and compressive demand and capacity). The Hungarian Algorithm takes the cost matrix as an input and returns the optimal global match and a matching cost, which can be used with the parameterized design model in shape optimization and related approaches. More details of each of these steps are given below.

![Material reuse workflow conceptual overview](image)

3.1 Inventory processing and cost matrix computation

The first input is a BIM-like 3D model of existing material to be reused. This paper focuses on reuse of linear timber elements, and assumes the stock material comes from a timber-framed house (discussed specifically in Section 4.1). Elements in the house model are digitally catalogued and processed so that they can be matched to a target design, using the process discussed in the following section.

A cost matrix $D$ of dimension $a$ and $b$ is computed by comparing each of the $a$ elements in the target design to each of the $b$ elements in the inventory. Each pairwise cost in the matrix is computed in 2D Euclidean space using the $L_2$ norm; the two dimensions of this space correspond to member length and axial load capacity/demand. As shown in Fig. 3, the axial load capacity is computed specifically for each pair, dependent on the target element’s length and axial load sign (tension or compression). The load demands are computed by performing a 3D frame finite element analysis on the dome, with a fixed, pre-specified cross section and material property. A large penalty value is added to the cost measure when the inventory element is insufficient in length or capacity, as a way to ensure that matched elements are geometrically and structurally feasible. Other types of distance function, e.g. a weighted $L_2$ norm can also be used here to address specific preference.
3.2 Optimal matching algorithm

Given a cost matrix $D$ defined in the previous section, with a target elements and b inventory elements, the optimal matching problem can be formulated as follows:

$$
\min_{T} c(T) = \sum_{i,j=1}^{a,b} D_{ij} T_{ij}
$$

s.t. $\sum_{i=1}^{a} T_{ij} \leq 1, \forall j \in \{1, ..., b\}$

$\sum_{j=1}^{b} T_{ij} = 1, \forall i \in \{1, ..., a\}$

$T_{ij} \in \{0, 1\}, \forall i \in \{1, ..., a\}, j \in \{1, ..., b\}$

where the design variables are the entries $T_{ij}$ of the assignment matrix $T$, where $T_{ij} = 1$ means inventory element $j$ is assigned to target element $i$, and 0 otherwise. The first inequality constraint ensures that each inventory element $j$ can be assigned to at most one target element. The second equality constraint enforces that exactly one inventory element is used at target element $i$. The third constraint enforces that the assignment matrix is binary. The cost matrix $D$ encodes the cost of assigning element $j$ to $i$, computed as described in Section 3.1. An optimal matching $T^*$ is the assignment that minimizes the total matching cost $c(T)$. In this paper, the matching cost of a given design is defined to be $c(T^*)$.

The problem described in equation (1) is an unbalanced linear assignment problem, a well-studied combinatorial optimization problem in the literature and widely used in various practical contexts [26]. Because of the discrete nature of the problem, Greedy Search algorithms (also called best-first search algorithms) have been shown to be practically effective [22]. However, despite their simplicity of implementation, the assignments obtained from the greedy search algorithms are generally not globally optimal. In this paper, the Hungarian Algorithm, a combinatorial optimization algorithm specifically developed for solving the linear assignment problem, is used to solve it to the global optimality in polynomial time [25].

The formulation in eq. (1) is an instance of integer linear programming (ILP) problems, and one can use more generic ILP machinery (e.g. the branch-and-bound algorithm [27]) to solve it. Moreover, eq. (1)'s constraint matrix is totally unimodular, a mathematical property that guarantees that the optimal solution is integral even if the binary constraint is relaxed [27]. Thus, linear programming algorithms designed for continuous problems (e.g. the simplex algorithm [26]) can be directly applied, relaxing the binary constraint, but still leads to integral solutions. In summary, the linear assignment problem is an integral linear programming problem with a very special mathematical structure, and algorithms with very different inner workings can be applied to solve it. In Table 2, empirical runtime data for these algorithmic options is presented. The Hungarian algorithm is about 60 times faster than the other two options.

Table 2  Runtime for solving a linear assignment problem with randomly generated 100*500 cost matrix. ILP and LP solved using Gurobi [28] via JuMP.jl [29], Hungarian using Hungarian.jl [30] with the Julia programming language [31]. All algorithms converge to the same optimal solution.

<table>
<thead>
<tr>
<th>Integer Linear Programming (ILP)</th>
<th>Linear Programming (LP)</th>
<th>Hungarian Algorithm</th>
</tr>
</thead>
</table>

Fig. 3 Assembly of cost matrix for all pairwise combinations of inventory and target elements.
Prior work in assignment optimization for material reuse uses a mixed-integer linear programming (MILP) formulation, additionally introducing continuous design variables for the structural nodal displacements and member forces so that structural analysis and assignment are treated simultaneously [17], [23].

In contrast, the formulation presented in this paper relaxes the MILP formulation by removing the structural analysis and stock length constraint. This relaxation has its advantages and disadvantages. The relaxation turns a MILP problem into a linear assignment problem and thus enables very efficient algorithmic treatment (see Table 2). However, since the structural capacity and stock length constraints are encoded as penalties in the objective function, instead of enforced as hard constraints, global optimality in terms of structural capacity is not guaranteed. Since the FEA is performed before the material matching with pre-assigned values, the load demand calculation is conservative, compared to the accurate load demand calculation used in the simultaneous design and analysis framework enabled by the MILP formulation [23]. However, the lower-bound theory of plasticity (e.g. as discussed in [32]) guarantees that the results are structurally safe. This approach also trades in hard constraint enforcement. This is a classic trade-off in constrained optimization formulation, and its unconstrained counterpart with penalty. Furthermore, while it is easy to extend the MILP formulation to solve the cutting-stock problem where one inventory element can be partitioned into more than one structure members, the Hungarian algorithm can only compute a one-to-one assignment. Such constraints can limit the solution of equation (1) to be sub-optimal compared to the cutting-stock MILP formulation when reducing cut-off waste [23]. Despite these disadvantages, this paper values the flexibility and independence of proprietary MILP solvers to enable a rapid and interactive computational design experience.

In order to facilitate the use of this research in both academia and industry, a Grasshopper script is made freely accessible online [33], released under the MIT license. For the matching part, a C# backend is provided that works out-of-the-box, using an existing open-source implementation [34]. To further improve the computational efficiency, an alternative matching backend is provided, written in the Julia programming language [31] using the Hungarian.jl package [30], which requires slightly more installation overhead but is at least an order of magnitude faster.

3.3 Parametric design model and optimization

In order to explore the full potential of the material inventory, a dome collection is parameterized in a flexible way that allows the number of domes, the radius of each dome, and the subdivision (called density in this paper) of each dome to vary, illustrated in Fig. 4. Geodesic dome designs are generated individually using the RhinoPolyhedra [35] Grasshopper plugin. Two NURBS curves are parameterized by a set of control points; whose vertical positions become the main design variables. The curves are sampled \( n \) times along their lengths, where \( n \) is the total number of domes. The vertical coordinate of the sampled point is used to define the radius or density parameter for each dome. The total number of variables is \( 9 \) (4 control points on 2 curves, plus \( n \)).

Several objective functions are possible to create a performance design space to be explored. The total matching cost, introduced in 3.2, is one important performance metric; a minimal matching cost means that surplus material in offcuts and structural capacity are minimized. As noted in 3.1, a one-sided penalty term is used to enforce length and capacity constraints. A second related objective function is inventory utilization, which computes the amount of structural material used from the inventory compared to the total available. A maximal inventory utilization indicates that the inventory is being used as fully as possible, and offcuts and extra stock is minimized. If an invalid match is returned by the Hungarian Algorithm, the inventory utilization is 0. Finally, the total floor area of the domes is an important objective to be maximized, which reflects the functional value of the reused materials’ configuration. Notice that while the first objective, the total matching cost, is computed by solving an inner optimization problem (equation (1)) using the Hungarian Algorithm, minimizing the matching cost or finding its trade-off with the other two objectives involves running an outer optimization loop using single-objective or multi-objective optimization machineries.
These objectives are used together and separately in a number of design space exploration experiments, using the free Design Space Exploration (DSE) plugin for Grasshopper [36], [37]. These include design space sampling, single-objective optimization, and multi-objective optimization.

4 Results

Using the methods described above, a series of design space exploration experiments can reveal a variety of opportunities for material reuse, discussed briefly in this section.

4.1 Inventory processing

First, the inventory of available stock material is digitally processed from a BIM model (in this case, from a publicly available model of a wood-framed house [38]). Individual linear framing elements are catalogued and digitally “cut” lengthwise to create a stock of elements with square cross sections, based on the construction and connector logic of the geodesic dome greenhouses that inspire this research (Fig. 1). In total, the processed inventory contains 2371 elements, organized into 13 types (roof rafter, wall stud, etc.). The minimum, maximum, median, and mean lengths are 0.2 m, 10.6 m, 3 m, and 3.62 m respectively. There are four different cross sections: 25 mm$^2$, 50 mm$^2$, 60 mm$^2$, and 100 mm$^2$. In this paper, all timber elements are assumed to have the same allowable strength and elastic modulus, 6.5 MPa and 10500 MPa respectively. The inventory is summarized visually in Fig. 5.

The large size of the inventory was found to make fast execution of the Hungarian algorithm challenging. Therefore, a decomposition strategy is implemented in which the original inventory is binned into two inventories of similar size and character in an automated process. The following results demonstrate possible constructions with one-half of the total inventory from the original house; the other half of the inventory could be used to construct similar arrays of domes.

4.2 Multi-objective optimization

The digital inventory is linked to the parameterized geodesic dome models described in 3.3, with optimal assignment to elements in the domes executed by the Hungarian Algorithm and tool described in 3.2. This creates a design space with multiple objective functions: matching cost (minimize), inventory...
utilization (maximize), and floor area (maximize). Using the MOO tool from DSE, the bi-objective plots in Figs. 6 and 7 are created; each takes about one hour on a standard laptop. Intuitively, there are trade-offs between some of objectives: a minimal matching cost will produce fewer domes with less total inventory material, to reduce waste. Inventory utilization and floor area are intuitively correlated, but not necessarily perfectly so. Fig. 6 reveals the first trade-off, with Pareto optimal and near-Pareto optimal designs highlighted. There is an increased total matching cost when material coverage is maximized, and the best matching option uses only 2% of the inventory (Fig. 6, bottom-left). The best option to choose depends on how much the design team wants to avoid offcuts and oversizing, and whether the unused inventory can be easily stored for future use. In this paper, offcuts are not considered for additional matching, but could be re-added to the inventory to make them easier to reuse.

In Fig. 7, a similar trend is observed: more than 10x more floor area is achievable compared to the optimal matching cost design, with an increase in matching cost of approximately 5x. If the design team has a preferred number of domes, that could also be used to choose among the Pareto-optimal options. Interestingly, similar amounts of floors area can be achieved with a wide variety of dome counts by using fewer large domes or more small domes.
4.3 Design space sampling and feasible region analysis

Design space sampling techniques can also be used to assess the effects of the length and strength constraints on the matching process. Fig. 8 shows the results of a Latin hypercube sample across multiple numbers of domes of varying radii with fixed density (=1), in terms of matching cost and inventory utilization (achieved using the Sampler and Capture tools in DSE in about one hour). Infeasible designs have very high matching costs and inventory utilizations of 0. The sampling results show that 42% of the generated designs are infeasible due to elements with no possible matches; these designs typically have long or highly stressed members. This type of analysis can be used to reveal the overlap between a parametric design space and a material inventory, and the design space can be adjusted to contain more feasible options. In this case, the feasible rate is considered acceptable and still allows for many possible designs.

![Fig. 8](image)

**Fig. 8** Visualization of feasible regions in design subspaces for the geodesic dome example. As the number of domes increases, there is less flexibility on radius (scale).

4.4 Single objective optimization

Finally, conventional single-objective optimization can be used to find a preferred configuration of domes, using Improved Stochastic Ranking Evolution Strategy (ISRES) as the global optimizer and Constrained Optimization BY Linear Approximations (COBYLA) as the local optimizer via the RadiCal tool in the DSE suite. The stochastic, global optimization method is used to find a region of interest in the non-convex design space, the local optimization is used to fine-tune the final result. The total runtime is about 10 minutes. The objective function is a combination of two of the previous metrics discussed: matching cost divided by floor area, which balances the need for waste minimization with the desire to generate as much functional space as possible. The number of domes is held constant in this case to reduce the challenges of discrete variables; Fig. 9 shows the result for a single dome, and Fig. 10 shows the results for 15 domes. The results show that high-quality and flexible results can be generated quickly within a design workflow.

5 Conclusions

This paper reviews and compares algorithmic formulations for reuse-driven design in existing computational approaches, and introduces a new Grasshopper tool to implement them. Both flexible design space exploration and efficient optimization are obtained by the use of the Hungarian Algorithm in a nested loop workflow. For small problems, the material reuse efficiency is computed in real time; for larger problems in a few seconds. The potential of this approach and tool are demonstrated on a real world case study that will be explored through physical experiments in future work.

Acknowledgements

Authors acknowledge assistance from Nicolas Petit-Barreau on constructing geodesic domes.
Fig. 9  Left: single dome matching; number of design elements: 65, matching cost: 197, floor area: 29.6, material coverage: 0.04, objective: 6.67. Right: Close-up view of the matching result and the FEA analysis.

Fig. 10  Number of design elements: 975, matching cost: 4316, floor area: 317.3, material coverage: 0.39, objective: 13.71

References


Connecting lines: investigating the potential of ruled surface structures for circular construction

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Abstract
The capacity of ruled surface structures for merging load-bearing and stabilizing functionalities with architectural qualities becomes evident in numerous built examples. Depending on the construction method and the material used, they can consist of discrete, similar, and sometimes even identical elements. This could contribute to the reusability of their basic elements. As modular entities or units, ruled surface structures could become an alternative to conventional prefabricated building components. This paper identifies and discusses design criteria that influence the reusability of ruled surface structures and their constituent elements. Moreover, it suggests new application scenarios for reusable and modular ruled surface components.

1 Introduction
According to [1], a ruled surface is defined as “a surface generated by a moving straight line with the result that through every point on the surface a line can be drawn lying wholly in the surface.” A similar definition is provided in [2]: “Cylinders, cones, one-sheet hyperboloids, and hyperbolic paraboloids are surfaces that carry families of straight lines. Thus, they could also be generated by moving a straight line.” Numerous iconic buildings of the twentieth century feature wall or roof elements based on singly or doubly ruled surfaces, such as conoids, hyperboloids of revolution, and hyperbolic paraboloids. Some of the most renowned examples are Antoni Gaudí’s Escuelas de la Sagrada Familia, Eladio Dieste’s Church of Christ the Worker, and Félix Candela’s Los Manantiales Restaurant. Other notable instances are the Parrocchia di Longuelo by Pino Pizzigoni and the now-demolished Laboratorios Jorba by Miguel Fisac. In all of these examples, ruled surface elements do not only have a structural function, but also make an important contribution to the buildings’ architectural expression and distinctiveness. They could hence be considered as tectonic constructs [3], [4].

In addition to that, the aforementioned examples highlight yet another quality of ruled surface elements. At the time of their construction, advanced digital design and fabrication tools were not available. Their realization was enabled by the fact that ruled surface structures, respectively the formwork for such structures, can be designed and built with relatively simple means. The reason for this is the geometric simplicity of straight lines, which are the generators of ruled surfaces, and which are present in many basic construction materials and aids. Due to the qualities of ruled surfaces and their potential for architectural applications, some scholars have started wondering why they are not used more often today [5]. Moreover, they have started questioning the ways in which computational tools have been used for the design and realization of geometrically complex structures over the past decade [6], [7].

These considerations become even more relevant in the light of an incipient climate change, which makes the reduction of CO2 emissions one of the biggest challenges facing our society today. One third of the global emissions originate in the building construction industry [8]. Increasing reuse and circularity has been identified as one possible way of addressing this issue. Last year, for example, some foundations and associations in Denmark allocated 40 million DKK to research and development projects that contribute to the construction sector’s conversion towards a circular economy [9]. Recent and pioneering research on this topic has been contributed by Certain Measures [10], an office for design science, as well as the Structural Xploration Lab at EPFL [11]–[13].

Ruled surface structures could make a relevant contribution to this development. Besides being relatively easy to construct, they can consist of discrete, similar, and sometimes even identical elements, which should facilitate the reusability of their basic elements. Moreover, as modular entities or units, ruled surface structures could become an alternative to conventional prefabricated building components. This research aims to verify the assumed suitability of ruled surface structures for reuse and
circular construction. The research is still in an early stage. Most of the here presented studies were embedded in the research-based MSc course “Tectonics in Engineering and Architectural Design,” carried out at Aarhus University’s Section of Civil and Architectural Engineering in the fall semester of 2020. As stated in its description, this course aims to provide students with a better understanding of the mutual relationships between structural engineering, building construction and architecture, focusing on design criteria that relate to and connect these fields.

2 Study of precedents

The first phase of the research focused on the study of precedents by means of a tectonic analysis and physical models. In addition to two of the previously mentioned buildings, Antoni Gaudi’s Escuelas de la Sagrada Familia in Barcelona and Eladio Dieste’s Church of Christ the Worker in Atlántida, the selected precedents also included Vladimir Shukhov’s lattice tower structures in Russia, and le Corbusier’s and Iannis Xenakis’ Philips Pavilion for the Expo 58 in Brussels. Moreover, the study also looked into more recent examples, namely the Antwerp Law Courts, designed by Rogers Stirk Harbour + Partners in collaboration with Arup and Bureau Van Kerckhove, as well as a low-tech space truss concept developed by Oliver Baverel and colleagues [14], [15].

Table 1 The selected precedents feature different types of ruled surface elements with varying functions and made of different materials.

<table>
<thead>
<tr>
<th>Project</th>
<th>Element</th>
<th>Material</th>
<th>Geometry</th>
</tr>
</thead>
<tbody>
<tr>
<td>Escuelas de la Sagrada Familia</td>
<td>roof</td>
<td>brick</td>
<td>conoid</td>
</tr>
<tr>
<td>Shabolovka Tower</td>
<td>wall</td>
<td>steel</td>
<td>hyperboloid</td>
</tr>
<tr>
<td>Church of Christ the Worker</td>
<td>wall</td>
<td>brick</td>
<td>conoid</td>
</tr>
<tr>
<td>Philips Pavilion</td>
<td>roof, wall</td>
<td>concrete</td>
<td>hypar</td>
</tr>
<tr>
<td>Antwerp Law Courts</td>
<td>roof</td>
<td>wood, steel</td>
<td>hypar</td>
</tr>
<tr>
<td>Space Truss</td>
<td>truss webs</td>
<td>wood</td>
<td>hypar</td>
</tr>
</tbody>
</table>

2.1 Tectonic analysis

Taking inspiration from Chad Schwartz’s book “Introducing Architectural Tectonics: Exploring the Intersection of Design and Construction” [16], the tectonic analysis was based on seven core criteria. The first two criteria addressed the existence and relevance of precedents, and the importance and impact of the place, respectively the site, including topography as well as social, cultural and climatic conditions. Another criterion was the anatomy of the studied buildings, whereas anatomy refers to the four elements of architecture – earthwork, framework, cladding and hearth – as defined by Gottfried Semper. Other criteria were the type of construction, differentiating between tectonic and stereotomic, respectively lightweight and heavy construction, the presence and relevance of representation, including ornamentation, as well as the role of the detail on different scales. Last but not least, the study also considered space, respectively the spatial quality and complexity of the precedents, as a criterion. The results of the study were visualized by means of radar charts.

2.2 Physical model studies

In addition to being subjected to a tectonic analysis, the precedents were also studied by means of physical models. The aim here was not to create exact reproductions, but rather to investigate the geometrical logic and functionality of the buildings’ ruled surface elements. The model studies of Antoni Gaudi’s Escuelas de la Sagrada Familia and Eladio Dieste’s Church of Christ the Worker went even further than that.

Among the selected precedents, Dieste’s Church of Christ the Worker, characterized by its distinctive conoid-shaped brick walls, is the only example in which the generating lines, or rulings, of the ruled surface remain invisible. Their manifestation in a physical form was only required during the construction process, as illustrated in [17]. In order to make the rulings visible in the physical model,
wooden sticks with a rectangular cross-section were chosen as a basic material, not only for the walls, but also for the roof. This also led to the idea of adapting the roof geometry to a conoid-based logic. In addition to highlighting the ruled-surface geometry of the brick walls, the edge-glued wooden sticks of the physical model could also be read as a reinterpretation of Dieste’s iconic building, using nail-laminated timber construction, a method known as Brettstapelbauweise in German, instead of brick.

A similar approach was pursued in the study of Gaudí’s Escuelas de la Sagrada Familia. As is the case with the Church of Christ the Worker, the dominant material in this building is brick. However, the conoid-shaped brick roof is built upon wooden beams that manifest some of the rulings of the roof surface, and that served as formwork during the construction process. Along the central axis of the building, the wooden beams are supported by a steel profile, which in turn is supported by several steel columns. With the focus of the physical model study being on the roof, the walls were omitted. Instead, profiled acrylic glass elements were placed in their position, which served as supports and facilitated the model building process. The main surface of the roof was built analogously to the before described study, using wooden sticks with rectangular cross-sections. Again, this could be read as a reinterpretation of the precedent, applying nail-laminated timber construction as an alternative to brick.

Fig. 1 The physical model study of Antoni Gaudí’s Escuelas de la Sagrada Familia (left) was carried out by Anna Olesen and Mads Emil Bisgaard. The physical model study of Eladio Dieste’s Church of Christ the Worker (top and bottom right) was conducted by Signe Skytte Madsen, Henriette Jensen Matthiesen, and Cecilie Skov Potempa. Photos: Markus Hudert. Editing: Ruven Wiegert.

The results of the two physical model studies are shown in Fig. 1. Both cases could be seen as speculative studies on what these iconic buildings would look like if they were built with linear wooden elements, using a technique similar to nail-laminated timber construction. In order to make them work in reality, an additional layer of cladding would be required, similar to the external façade layer of the “Timber Prototype House,” which was recently showcased as part of the IBA Thüringen [18].

3 Towards modular and reconfigurable construction systems

In order to verify the assumed suitability and potential of ruled surface structures for reuse and circular construction, a research through design approach was applied in the second part of the study. The design task was to develop a reusable and reconfigurable construction system, based on the principles of ruled surfaces, and to be used as a charging station for electric cars on a site in Aarhus. An inspiration for this was a recent project developed by the architecture office Cobe [19]. One of the boundary
conditions of this design task were size-related constraints, enabling the transportability of the modules. Regarding the materiality of the load-bearing components, the focus was on timber and engineered timber products. The iterative design studies were carried out in groups of two to three students. In the following, two of the five resulting proposals shall be discussed in more detail.

3.1 Conoid roof and wall elements

One of these two proposals was developed by Signe Skytte Madsen, Henriette Jensen Matthiesen, and Cecilie Skov Potempa. Two key sources of inspiration were Dieste’s Church of Christ the Worker and the Kohta shelter, a project designed and built by the Aalto University Wood Program [20], [21]. The proposal features roof and wall modules based on a conoid geometry. In response to the above mentioned size constraints, the roof modules are trimmed and have a triangular contour. Both the roof and the wall modules consist of linear wooden elements with rectangular cross-sections. They are connected by spacer elements, equally made of wood. Future studies will have to verify if this is a viable solution, or if continuous rib elements need to be added. The latter would need to be fabricated with a curvature that fits to that of the ruled surface component. Potentially, this could turn out to be a disadvantage of conoid geometry, as elements with a specific curvature might have a limited reusability.

While fulfilling virtually all of the design constraints, the main quality of this proposal is the many ways in which it can be configured. Eight triangular roof elements and four wall elements can be combined into laterally open higher-order units with a square footprint. Rectangular areas of various proportions and size can be covered by putting together several of these higher-order units.

Fig. 2 One of the developed construction systems employs conoid-based ruled surface components that are reconfigurable and transportable. Illustration: Signe Skytte Madsen, Henriette Jensen Matthiesen, and Cecilie Skov Potempa

Fig. 3 Eight triangular roof elements and four wall elements can be combined into a laterally open higher-order unit with a square footprint (left). The system equally allows for the creation of enclosed spaces (right). Illustrations: Signe Skytte Madsen, Henriette Jensen Matthiesen, and Cecilie Skov Potempa
Moreover, the system can generate both closed and open spaces. Fig. 2 provides an overview of the system’s different components. Fig. 3 depicts two types of higher-order units, and Fig. 4 shows the configuration chosen for the construction site.

Fig. 4  The configuration shown here is comprised of two laterally open higher-order units. It is tailored toward the size and context of the chosen site, the northeastern part of Harald Jensens Plads in Aarhus. Illustration: Signe Skytte Madsen, Henriette Jensen Matthiesen, and Cecilie Skov Potempa

3.2  A multi-material kit of parts
The second proposal, developed by Leo Gamborg Heinzl, Mattias Brun Jakobsen, and Morten Juhl Stistrup Nielsen, is a kit-of-parts for a charging station unit consisting of timber, bio-composite, and metal elements with linear and plate-shaped geometries. In plan, the unit is reminiscent of the shape of a paper coffee filter. The four corner points of its perimeter correspond to the endpoints of two concentric circle segments. The lower part of the unit consists of cross-braced steel elements. The roof is composed of two faceted and oppositely oriented conoid-shaped parts, whereas the substructure of the roof is composed of metal elements and timber beams. Bio-composite panels are used as roof cladding. Because of its shape in plan, arrays of such modules can have various forms, ranging between a closed circle and curvilinear or quasi-linear shapes. Fig 5 shows an exploded view of the kit-of-parts as well as a schematic top-view of three possible multi-unit configurations.

3.3  Summary and limitations
Altogether, the five resulting proposals provide a good indication of the versatility and potential of modular and reusable ruled surface structures in building construction. Although the initial goal was to develop a system that can serve as a charging station for electric cars, it is clear that the proposed systems could be used for a wide range of applications, including bus stops, bicycle shelters, and other types of urban infrastructure and furniture. One should keep in mind that the studies mainly provide insights on the potential of combining and reconfiguring ruled surface components. Due to time constraints, structural and construction related aspects have only been studied in part and need to be investigated in more detail. Nevertheless, the results are considered as an incentive for further research.

4  Outlook
The above presented studies and considerations constitute the foundation for forthcoming research on reusable timber components with hyperbolic paraboloid shapes. Some possible applications are prefabricated wall and roof elements as well as tower structures for wind turbines. Disassembly and reuse will be addressed at the scale of the basic element and that of the component. In both cases, the construction method and the connection types will play a crucial role. The research will look into different types of construction, such as gridshells made of lamellas, reciprocal configurations, as well as digital
Fig. 5 This kit-of-parts combines timber, bio-composite, and metal elements with linear and plate-shaped geometries (left). The top view of three multi-unit configurations (top right) is shown in a schematic manner. Illustrations: Leo Gamborg Heinzl, Mattias Brun Jakobsen, and Morten Juhl Stistrup Nielsen

design and fabrication workflows for ruled surface components made of Cross-, Dowel-, or Nail-Laminated Timber.

As shown in the example of the Antwerp Law Courts, gridshell construction is a suitable approach for prefabricated and self-contained hypar components. One remaining question would be how to connect several of such components with each other while at the same time enabling disassembly and reuse. One option would be to use the logic of construction, in this case that of layered lamellas, to connect components with each other. To some degree, the planks of the cladding could also contribute to that.

A similar challenge is present with hypar CLT components. Reimagining CLT components as semi-finished products with build-ups similar to board systems could lead to new ways of connecting them on site, for example by adding planks that bridge over the boundaries of the individual components.

In reciprocal build-ups, the basic element is the strut. Reciprocal hypar structures can be built and, potentially, extended piece by piece in a continuous manner. A recent example that illustrates the potential of reciprocal configurations with interlocking joints is the “Gradational Reciprocality” pavilion, designed and built at Harvard GSD [22].

The type of connections between elements and components is equally important, as joints play a critical role as enablers of disassembly and reuse. Form-locked connections, and here especially snap-fit joints between panels, enable wood connections without steel and adhesives, as demonstrated by researchers at IBOIS [23]. However, while potentially reducing the environmental and ecological impact in the initial construction phase, this type of connection is difficult to release without making the direct reuse of the elements impossible.

Force-locked connections with mechanical fasteners such as screws and bolts can be released relatively easy. Nonetheless, reusing screwed joints with same positions has its limitations. Even when using screws with increasingly large diameters, the same holes can only be used for a limited number of times.

Elements joint by means of adhesive bonding, as is the case with Glulam and CLT, are virtually impossible to disjoin without damaging them. Hence, such components should be reused as often and
long as possible. Here, it could make sense to use glued-in threaded bars, in order to increase the number of times these elements can be connected and disconnected. Using mechanically laminated timber could enable the reuse of the individual boards. However, the deformation of the boards due to creep could make this challenging.

Relevant references for forthcoming studies are the ongoing research on hypar CLT components at Technische Universität München [24], recent work on hypars at ETH Zurich [25]–[27], as well as the work of other specialists in the field of ruled surfaces [28] and circular construction [29].

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References


Sustainable seismic design of bridges inspired by ancient temples

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Abstract
In earthquake prone regions bridges are often designed with huge pile foundations that can comprise up to 50% of the total volume of reinforced concrete of the whole structure. The reason for this maybe irrational ratio is the design dogma that the columns should be firmly fixed to the ground. The survival of ancient temples supported on uplifting columns suggests that the above dogma is not a necessity. This paper presents a design concept for bridges that is based on rocking: Precast elements are connected dryly using a restraining tendon in series with a spring, to form a flexible and resilient system that is expected to sustain zero damage under the design earthquake. The design moment at the bottom of the column can be close to zero, something that could potentially lead to avoidance of the pile foundations, thus saving material and reducing the cost and environmental footprint of the bridge. Numerical findings as well as recent large scale tests are discussed.

1 Introduction
Since the 1970s the seismic design philosophy prohibits collapse under the design level earthquake but allows for significant damage; this is why structures are designed to be ductile. The Christchurch 2011 earthquake demonstrated that the public had tacitly agreed with this concept, only because it had not realized the consequences: Plastic design means that the structure will need to be demolished after the design earthquake. So, during the last decade there is a societal demand for structures that not only avoid collapse, but also sustain no damage, that is they are resilient.

In parallel, the above goal needs to be achieved at a low cost, a constraint which is highly correlated to efficient use of construction materials – a long standing and fundamental constraint of structural engineering that has been lately branded as “sustainable construction”.

In seismic design of bridges in low seismicity areas like Switzerland, resilience can be achieved by designing a fixed base bridge that behaves elastically (Fig. 1a). In high seismicity areas this is not cost-effective and the state of the practice is to use seismic isolation (Fig. 1b). A recent technology that competes with seismic isolation is restrained rocking (Fig. 1c). The concept is based on the early work of Priestley and Tao [1], Stone et al. [2] and the PRESSS project [3-5]: Precast elements are connected via unbonded tendons. In such systems, the lateral drifts required to yield the tendons are much larger than conventional structures because the tendon strain is equal to its deformation divided by its whole length. The concept has been applied to bridges by multiple researchers [6-31]. Two bridges based on this concept have been recently constructed in New Zealand and China [32-33]. Restrained rocking solutions are compatible with Accelerated Bridge Construction, as prefabricated elements can be used and they minimize the construction site work, because of the dry connections.

Both seismic isolation and restrained rocking effectively protect the superstructure and allow for its elegant and economical design. However, they result to base moments that often require large pile foundations that would not have been needed to take the axial loads only. Such pile foundations can comprise 50% of the total Reinforced Concrete used for the whole structure, essentially doubling the required material. They are based on the design dogma that structures should be firmly connected to
circular construction. The research is still in an early stage. Most of the here presented studies were embedded in the research-based MSc course “Tectonics in Engineering and Architectural Design,” carried out at Aarhus University’s Section of Civil and Architectural Engineering in the fall semester of 2020. As stated in its description, this course aims to provide students with a better understanding of the mutual relationships between structural engineering, building construction and architecture, focusing on design criteria that relate to and connect these fields.

2 Study of precedents

The first phase of the research focused on the study of precedents by means of a tectonic analysis and physical models. In addition to two of the previously mentioned buildings, Antoni Gaudi’s Escuelas de la Sagrada Familia in Barcelona and Eladio Dieste’s Church of Christ the Worker in Atlántida, the selected precedents also included Vladimir Shukhov’s lattice tower structures in Russia, and le Corbusier’s and Iannis Xenakis’ Philips Pavilion for the Expo 58 in Brussels. Moreover, the study also looked into more recent examples, namely the Antwerp Law Courts, designed by Rogers Stirk Harbour + Partners in collaboration with Arup and Bureau Van Kerckhove, as well as a low-tech space truss concept developed by Oliver Baverel and colleagues [14], [15].

Table 1 The selected precedents feature different types of ruled surface elements with varying functions and made of different materials.

<table>
<thead>
<tr>
<th>Project</th>
<th>Element</th>
<th>Material</th>
<th>Geometry</th>
</tr>
</thead>
<tbody>
<tr>
<td>Escuelas de la Sagrada Familia</td>
<td>roof</td>
<td>brick</td>
<td>conoid</td>
</tr>
<tr>
<td>Shabolovka Tower</td>
<td>wall</td>
<td>steel</td>
<td>hyperboloid</td>
</tr>
<tr>
<td>Church of Christ the Worker</td>
<td>wall</td>
<td>brick</td>
<td>conoid</td>
</tr>
<tr>
<td>Philips Pavilion</td>
<td>roof, wall</td>
<td>concrete</td>
<td>hypar</td>
</tr>
<tr>
<td>Antwerp Law Courts</td>
<td>roof</td>
<td>wood, steel</td>
<td>hypar</td>
</tr>
<tr>
<td>Space Truss</td>
<td>truss webs</td>
<td>wood</td>
<td>hypar</td>
</tr>
</tbody>
</table>

2.1 Tectonic analysis

Taking inspiration from Chad Schwartz’s book “Introducing Architectural Tectonics: Exploring the Intersection of Design and Construction” [16], the tectonic analysis was based on seven core criteria. The first two criteria addressed the existence and relevance of precedents, and the importance and impact of the place, respectively the site, including topography as well as social, cultural and climatic conditions. Another criterion was the anatomy of the studied buildings, whereas anatomy refers to the four elements of architecture – earthwork, framework, cladding and hearth – as defined by Gottfried Semper. Other criteria were the type of construction, differentiating between tectonic and stereotomic, respectively lightweight and heavy construction, the presence and relevance of representation, including ornamentation, as well as the role of the detail on different scales. Last but not least, the study also considered space, respectively the spatial quality and complexity of the precedents, as a criterion. The results of the study were visualized by means of radar charts.

2.2 Physical model studies

In addition to being subjected to a tectonic analysis, the precedents were also studied by means of physical models. The aim here was not to create exact reproductions, but rather to investigate the geometrical logic and functionality of the buildings’ ruled surface elements. The model studies of Antoni Gaudi’s Escuelas de la Sagrada Familia and Eladio Dieste’s Church of Christ the Worker went even further than that.

Among the selected precedents, Dieste’s Church of Christ the Worker, characterized by its distinctive conoid-shaped brick walls, is the only example in which the generating lines, or rulings, of the ruled surface remain invisible. Their manifestation in a physical form was only required during the construction process, as illustrated in [17]. In order to make the rulings visible in the physical model,
Fig. 3 Restrained rocking frame with the tendon anchored at: (a) the foundation; and (b) the bottom end of the column.

Fig. 4. (a) $F-u$ relationship for cases in Fig. 3a,b; (b) $M_{top}-u$ and $M_{bot}-u$ relationships for case in Fig. 3a, and $M_{top}-u$ relationship for case in Fig. 3b; (c) $M_{bot}-u$ relationship for case in Fig. 3b.

Assuming a horizontal force $F$ applied at the beam, the lateral force – deformation relation (“pushover curve”) when the frame is anchored at the foundation is:

$$F = \left(1 + \gamma\right) N_m g \tan(\alpha - \theta) + \frac{2Nk_{res} h}{\cos(\alpha - \theta)} \sin\alpha \tan\alpha \sin\theta$$

As $\alpha$ and $\theta$ are small, equation (1) can be linearized to:

$$F = \left(1 + \gamma\right) N_m g \alpha \text{sgn} u + \frac{2Nk_{res} h}{\cos(\alpha - \theta)} \left(\frac{1}{2} + \gamma\right) N_m g \frac{u}{2h}$$

The linearized equation when the tendon is anchored at the bottom of the column is:

$$F = \left(1 + \gamma\right) N_m g \alpha \text{sgn} u + \frac{Nk_{res} h}{2} - \left(\frac{1}{2} + \gamma\right) N_m g \frac{u}{2h}$$

Fig. 4a plots equations (2) and (3) for different values of $k$.

For all values of $k$, the system presents a bilinear elastic behavior. There is no hysteresis and unloading follows the same branch.

When $k = 0$ (i.e. no tendon), the post uplift stiffness of the system is negative. Collapse is reached not because of material failure, but when the restoring force becomes zero, i.e. when the columns reach the point of neutral equilibrium. This defines the displacement capacity.
Adding a non-prestressed tendon algebraically increases the post uplift stiffness. When the stiffness remains below $k_{\text{crit}}$, where $k_{\text{crit}} = \frac{(1+2\gamma)m_g}{4b\alpha}$ or $k_{\text{crit}} = \frac{(1+2\gamma)m_g}{b\alpha}$ depending on whether the tendon is anchored within the foundation or at the column, the post uplift stiffness of the system remains negative, the slope of the second branch is milder and the displacement capacity increases.

When the stiffness of the tendon becomes larger than $k_{\text{crit}}$, the post uplift stiffness becomes positive and the system never becomes unstable, given the assumption of no sliding.

By choosing appropriate restraining systems the pushover curves of the two variations of the rocking frame can be made identical. This is not the case for the base moment—top displacement curve: When the tendon is anchored in the foundation, the base moment increases with displacement. However, if the tendon is anchored within the column, the base moment is capped and equal to the load carried by the column times its halfwidth. As the goal is to reduce the base moment, this paper will focus only on the latter case.

3 Dynamic behavior of the restrained rocking frame

The equation of motion of the restrained rocking frame of Fig. 3b is

$$\ddot{\theta} = -\frac{1+2\gamma}{1+3\gamma} p^2 \left( \sin(\alpha \text{sgn} \theta - \theta) + \frac{\dot{u}_t}{g} \cos(\alpha \text{sgn} \theta - \theta) \right) - \frac{1}{1+3\gamma} p^2 \sin \alpha \frac{kb}{m_g} \sin \theta$$

(4)

Linearizing equation (4) gives:

$$\ddot{u} + \frac{1+2\gamma}{1+3\gamma} \frac{3g}{2} \alpha \text{sgn} u + \frac{3}{2} \left[ \frac{1}{1+3\gamma} \frac{akb}{m_e} - \frac{3g}{2} \frac{1+2\gamma}{1+3\gamma} \frac{1}{P^2-A} \right] u = -\frac{3}{2} \frac{1+2\gamma}{1+3\gamma} \frac{\dot{u}_t}{g}$$

(5)

Unless extra yielding reinforcement is provided internally or externally, energy is only dissipated during impact and it is usually taken into account via a coefficient of restitution [36] defined as

$$r = \frac{\dot{\theta}_{\text{after}}}{\dot{\theta}_{\text{before}}}$$

(6)

When the stiffness of the restraining system is positive extra damping can be provided to the system in the form of extra yielding bars and the force deformation loop takes a characteristic flag shape [49]. Then the dynamic response of the system can be approximated by an equivalent elastic oscillator and design follows the standard elastic spectrum based approach.

However, when the stiffness of the bilinear oscillator is negative, there is no equivalent elastic system and the elastic spectrum cannot be used [50]. To avoid time history analysis, Reggiani Manzo and Vassiliou [51] have observed that all negative stiffness oscillators (NSBE oscillators) of the same uplift force, will exhibit roughly equal displacement, no matter what their post uplift stiffness is, as long as they are not close to failure. Therefore, the displacement of the class of rocking structures of uplift force $F_{\text{up}}$, can be computed using as a proxy the displacement of a bilinear oscillator of uplift force $F_{\text{up}}$ and infinite displacement capacity (ZSBE oscillators) (Fig. 5). This allows for the construction of $u_{\text{max}} - F_{\text{up}}$ spectra (not to be confused with elastic $u_{\text{max}} - T$ spectra) that can be used for the design of negative stiffness restrained rocking bridges.

![Fig. 5 Schematic of NSBE and ZSBE oscillators.](image-url)
4 Restrained rocking system using disc springs and an unbonded tendon

The above analysis is highly idealized because:

a) Usually a restraining system comprises solely a tendon that has a finite yield strain. For a tendon strength of 1800MPa, the yield strain is 9×10⁻³. Assuming a bridge column of 9.6×1.6m, rigid body analysis shows that the tendon will yield at a drift ratio of 11%, if the tendon is anchored in the bottom of the column and 5.5% if it is anchored in the foundation.

b) The column is not rigid, but will sustain flexural deformation along its length and local deformation at the column - foundation and column - cap beam contact zones. If a relatively stiff tendon is used and the column ends are not protected the deformation will cause concrete spalling.

c) There might be sliding at the interfaces between the column and the cap beam or foundation.

To test the validity of the static analysis of section 2 this section very briefly presents the results of cyclic tests on a 1:5 scaled precast RC column that were performed in the laboratory of the Institute of Structural Engineering of the ETH. To avoid damage to the column, its ends were protected with steel jackets. The restraining system comprised unbonded tendons in series with disc springs (Fig. 6).

The column was compressed to a normalized axial load of 5% and then laterally loaded in cycles up to 15% drift. No damage was observed and the force deformation – loops (Fig. 7) showed no deterioration. The energy dissipation implied from the loops is due to friction of the setup, not damage to the column.

Fig. 6 Left: Schematic illustration of the tested specimen. Right: Specimen tested at ETH Zürich.

Fig. 7 Force-drift relation for: (a) low drift ratio, (b) medium drift ratio, and (c) high drift ratio.

5 Case study – proof of concept

Within the context of a proof of concept, this study presents the planar rocking response of a bridge supported on two bents comprising two-columns each (i.e. a total of 4 columns), when seismically isolated or when allowed to rock restrained with a flexible rocking system. In this numerical application, the cylindrical piers of the free-standing bridge bent are 9.6-m tall with a diameter d = 2b = 1.6 m. We assume that the deck has a mass of 1500t. These are typical dimensions for highway overpasses and other bridges in Europe and USA. The FEMA P695 set of 28 near field pulse-like ground motions are
used as excitation (14 ground motions × 2 horizontal components each). Two scenarios are examined: all excitations scaled to a) PGA=0.40g and b) PGA=1g. This proof of concept study focuses on median (along the 14 ground motions) response parameters of each scenario.

5.1 Seismic Isolation

Two cases are studied: One with an isolation period of 2s and one with 3s. The damping ratio is 10% in both cases, which is a typical value for seismically isolated bridges. Table 1 provides the median displacement demand, shear at the top of the bents and moment at their bases (including the P-Δ effects). The analysis was performed with direct integration of the equation of motion.

5.2 Restrained rocking

A restraining system with \( k_{res} = 21'676.6 \text{ kN/m (per column)} \) is chosen. This system increases the displacement capacity (i.e. the displacement that causes overturn) to 2.5m. Then, via Equation 4 one can compute the response of the restrained system. Table 1 provides the median displacement demand and shear at the top of the bents, as well as the moment at their base for each scenario. In the PGA=0.4g scenario, the restrained rocking system developed a design moment of the foundation which is 30-40% of the isolated systems, while the displacement demand is also smaller. In the PGA=1.0g, the base moment of the rocking system is 13-17% of the isolated systems – at the expense of a much larger displacement demand. The residual displacement of the rocking system is expected to be close to zero.

Table 1  Median displacement demand, shear at the top of the bents and base moment of the seismic isolated system and restrained rocking frame.

<table>
<thead>
<tr>
<th>System</th>
<th>PGA=0.4g</th>
<th></th>
<th>PGA=1.0g</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>u (cm)</td>
<td>V (kN)</td>
<td>M (kN.m)</td>
<td>u (cm)</td>
</tr>
<tr>
<td>Isolation T = 2s</td>
<td>25</td>
<td>935</td>
<td>9'908</td>
<td>63</td>
</tr>
<tr>
<td>Isolation T = 3s</td>
<td>39</td>
<td>647</td>
<td>7'662</td>
<td>98</td>
</tr>
<tr>
<td>Rocking</td>
<td>19</td>
<td>652</td>
<td>3'379</td>
<td>191</td>
</tr>
</tbody>
</table>

6  Conclusions

Allowing a bridge pier to uplift during an earthquake reduces the design moment of the foundation and can possibly drastically reduce its size. To enhance the stability of such a system, without increasing the design moment of the foundation, a restraining system comprising an ungrouted tendon in series with a spring system has been proposed and tested up to 15% of drift without any damage. As a proof of concept, a planar model of a rocking bridge has been compared to bridge using seismic isolation of 2 or 3s isolation period. The rocking bridge developed a base moment ranging 13%-40% of the seismically isolated cases, depending on the intensity of the design ground motions.

Acknowledgements

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References


Algorithm-aided structural-optimization strategies for the design of variable cross-section beams

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Abstract

The optimization of important structural components - such as beams - has always represented a crucial challenge in architectural and structural design, especially considering how the optimization of certain components often involves the loss of control of structural shapes.

In this scientific contribution, an analytical model for shape optimization is presented. The shape optimization of the test-case, a variable-section beam, is modelled and optimized using two different approaches and solvers: i) MATLAB-GA®, a stochastic, population-based algorithm that randomly searches the optimal solution among population members, by mutation and crossover operators; ii) Gh-Octopus®, a Multi-Objective Evolutionary Optimization solver, which allows the production of optimized trade-off solutions between the extremes of each goal, able to support designers in decision making. The methods combine Computational Geometry and Parametric Design and allow control of the shapes of structural elements while increasing structural performance.

The good accordance between the results retrieved by the numerical model implemented on MATLAB-GA® with the numerical model implemented using Gh-Octopus® allowed the validation of the analytical method presented in this contribution.

1 Introduction

In the latest developments regarding structural optimization, the scientific community is increasingly directed towards the interest in shape optimization [1]. Despite the complexity of the problem linked to the shape of structures, it appears to be more efficient and effective in improving structural performance considering the influence of the shape on the structural behaviour [2] [3] and for this reason, the demands of performing shape optimization methods are increasing. The attention in the design of structural/architectural shape and its optimization has led the designers to look for a method able to communicate the architectural language searched and/or needed for each specific architecture by modifying the structural performance.

This paper introduces two methods for minimising the volume of a non-prismatic beam element by altering its shape along the base through an emptying function, studied and obtained applying the principles of computational geometry joined with computational design, so allowing the perfect control over the shape of the structural element.

Two approaches are presented and compared: firstly, a Timoshenko beam element using a metaheuristic algorithm (MATLAB - ga) and secondly, a parametric model with integrated FEA, which utilises a multi-objective approach (Hype/Octopus in Grasshopper environment).

In Section 2 the analytical model based on the standard Timoshenko kinematics hypothesis is presented. The analytical model of the beam is then used to obtain deformations and stresses of the beam,
under different constraints, while load is assumed as the sum of a generic external variable vertical action and the self-weight. The solution is obtained by numerical integration of the beam's equation and constraints are posed both on the maximum stress and on the vertical deflection. The section variability is described assuming a rectangular cross-section with constant base and variable height according to a trigonometric series. Optimization is thus performed by minimizing the beam volume, so considering the effects of non-prismatic geometry on the beam behaviour.

In Section 3 two different numerical solutions are developed and compared, evaluating the solutions obtained from the different methods.

2 Analytical beam model

To solve the problem of variable section beam, the equation of elastic line is introduced, that can be written as follows [4]:

\[
\frac{d^2}{dx^2} \left[ EJ(x) \frac{d^2}{dx^2} y(x) \right] = q(x)
\]

(1)

Eq. (1), in abbreviated form, simplifies as:

\[
\frac{d^2}{dx^2} [Jy''] = \frac{q(x)}{E}
\]

(1.1)

By developing the second order derivative, we obtain:

\[
d[J'y'' + Jy'''] = \frac{q(x)}{E}
\]

(1.2)

and deriving one more time:

\[
J''y'' + 2J'y''' + Jy'''' = \frac{q(x)}{E}
\]

(1.3)

Reorganizing Eq. (1.3) from the maximum order derivative to the minimum one, it yields to:

\[
y''''(x) \cdot J(x) + 2y'''(x)J'(x) + y''(x)J''(x) = \frac{q(x)}{E}
\]

(2)

where:

\[
J(x) = \frac{1}{12}bh^3(x); \quad J'(x) = \frac{1}{4}bh^2(x) \cdot h'(x); \\
J''(x) = \frac{1}{4}b[2h(x)h''(x) + h^2(x)h'''(x)]
\]

(2.1)

are respectively the moment of inertia and its high order derivatives.

The Beam section Depth Function is then introduced, which varies with depth beam-section:

\[
h(x) = h_0 - \psi(x)
\]

(3)

where \(h_0\) represents the initial beam section depth (first boundary condition) and \(\psi\) identifies the emptying function which is the sum of many harmonics as the following one:

\[
A_0 \cos x \frac{2 \pi}{L} \cdot \frac{\pi}{2}
\]

(3.1)

The function \(\psi\) can be written as the sum of \(N\) harmonics:

\[
\psi(x) = \sum_{i=1}^{N} A_i \sin \left( \frac{i \pi x}{L} \right), \text{ where } i \rightarrow \text{odd number}
\]

(3.2)

The following relation between the emptying function \(\psi\) and the Beam section depth Function \(h(x)\) is so derived:

\[
h(x) = h_0 - \sum_{i=1}^{N} A_i \sin \left( i \frac{\pi x}{2} \right)
\]

(4)

By considering the Equations of Moment of Inertia:

\[
J''(x) = \frac{1}{4}b[2h \cdot h'' + h^2h''']; \quad J'(x) = \frac{1}{4}bh^2h'; \quad J(x) = \frac{b}{12}h^3
\]

(2.1.a)

and the equation of beam section depth function and its high-order derivatives, we obtain:
\[ h(x) = h_0 - \psi(x); \quad h'(x) = -\psi'; \quad h''(x) = -\psi'' \quad (3.1) \]

Substituting Eqs. (3.1) and (2.1.a) in Eq. (2), the following expressions are obtained:
\[ y^{IV}(x) \cdot f(x) + 2y'''(x)f'(x) + y''(x)f''(x) = \frac{q(x)}{E} \quad (5) \]

with:
\[ f(x) = \frac{1}{12} b \left[ h_0 - A \sin \left( \frac{\pi}{L} x \right) \right]^3 \]
\[ f'(x) = \frac{1}{4} b \left[ h_0 - A \sin \left( \frac{\pi}{L} x \right) \right] \left[ A \frac{\pi}{L} \cos \left( \frac{\pi}{L} x \right) \right] \]
\[ f''(x) = \frac{1}{4} b \left[ 2 \left( h_0 - A \sin \left( \frac{\pi}{L} x \right) \right) \left[ -A \frac{\pi}{L} \cos \left( \frac{\pi}{L} x \right) \right]^2 + \left[ h_0 - A \sin \left( \frac{\pi}{L} x \right) \right] \left[ A \frac{\pi}{L} \cos \left( \frac{\pi}{L} x \right) \right] \right] \]

The term \( q(x) \) in Eq. (5) is the sum of two contributions: the dead load \( A(x) \gamma \) - which can be written as a function of the material density and the beam section – and the variable load \( q_o \) which represents the external forces; hence, Eq. (2) is written as follows:
\[ y^{(4)}(x)f(x) + 2y^{(3)}(x)f'(x) + y^{(2)}(x)f''(x) = \frac{q_0 + A(x)\gamma}{E} \quad (7) \]

To develop the Equation System in a vector form we introduce vector \( z \):
\[ z = \begin{pmatrix} y(x) \\ y'(x) \\ y''(x) \\ y'''(x) \end{pmatrix} \quad (8) \]

by imposing:
\[ z' = f(z, x) \quad (9) \]

where vector \( f \) can be written as follows:
\[ f(z, x) = \begin{pmatrix} y'(x) \\ y''(x) \\ y'''(x) \\ -2f'(1)y'(3)(x) - f'(2)y''(2)(x) + \frac{q_0 + A(x)\gamma}{E} \end{pmatrix} f(x) \quad (9.1) \]

It, in a simplified form, becomes:
\[ f(z, x) = \begin{pmatrix} y'(x) \\ y''(x) \\ y'''(x) \\ -2f'(1)y'(3)(x) - f'(2)y''(2)(x) + \frac{q_0 + A(x)\gamma}{E} \end{pmatrix} f(x) \quad (9.1.a) \]

In the hypothesis of a beam section fully defined by the Equation \( f(x) = \xi^2 A^2(x) \) and by deriving two times the ratios \( \frac{f'(1)}{f} \) and \( \frac{f'(2)}{f} \), at the aim to parameterize the System Problem, it follows:
\[ \frac{f'(1)}{f} = \frac{3 h'}{h}; \quad \frac{f'(2)}{f} = 3 \left[ 2 \left( \frac{h'}{h} \right)^2 + \frac{h''}{h} \right] \quad (9.2) \]

Finally, we obtain:
\[ f(z, x) = \begin{pmatrix} y'(1)(x) \\ y''(2)(x) \\ y'''(3)(x) \\ -6 \frac{h'}{h} y'(3)(x) - 3 \left[ 2 \left( \frac{h'}{h} \right)^2 + \frac{h''}{h} \right] y''(2)(x) + \left( \frac{q_0}{E} \right) \frac{1}{f(x)} + \left( \frac{\gamma}{\xi \sqrt{f(x)}} \right) \frac{1}{E} \end{pmatrix} \quad (9.3) \]
which can be solved for the only parameter $h$, making the parameter $f(x)$ explicit:

$$f(z,x) = \begin{pmatrix}
-6 \frac{h'}{k} y'''(x) - 3 \left[ 2 \left( \frac{h'}{k} \right)^2 + \frac{h''}{k} \right] y''(x) + \left( \frac{d_0}{E} \right)^3 + \left( \frac{y}{E \xi} \right)^2 \frac{12}{b}(x) \\
y'(x) \\
y''(x) \\
y'''(x)
\end{pmatrix}$$  \hspace{1cm} (9.4)

3 The Optimization Problem and Numerical Developments

The study of the geometry of the variable cross-section beam was implemented starting from the evaluation of a parallelepiped-shaped beam fully supported on both sides (Fig.1(a)).

The thickness ($b_0$), the total height ($h_0$) and the span ($L$) are constant parameters, and for this case, the dimensions of the examined beam will be as following: $b_0: 0.9 \text{ (m)}$; $h_0: 2.7 \text{ (m)}$; $L: 21 \text{ (m)}$.

Considering the starting geometry of the beam (Fig.1 (a)), the optimization problem related to the test case will be implemented to obtain the minimum necessary volume, ensuring high structural performance; as mentioned, one of the necessary constraints to control the shape of the cross-section, will concern the geometry that will be used as an emptying function as shown in the Fig. 1(b). The applied load to the fixed ended beam, was assumed uniformly distributed, equal to 100 kN/m, which adds up to the self-weight.

By considering the relations between the emptying function $\psi$ and the Beam section depth Function $h(x)$ derived in Eq. (4), the following expressions are obtained:

$$\psi(x) = \sum_{i=1}^{n} A_i \sin \left( i \pi \frac{x}{L} \right) ; \quad Vol = b_0 \left[ \int_0^L h(x) \, dx \right] = \left( b_0 \, L \, h_0 - \sum A_i \frac{2\pi}{L} \right)$$  \hspace{1cm} (10)

and consequently, the Objective Function of the Optimization Problem assumes the following form:

$$\text{O.F.:} \quad Vol = b_0 L \left( h_0 - \sum A_i \frac{2\pi}{\pi} \right) = b_0 L \left( h_0 - A \frac{2\pi}{\pi} \right)$$  \hspace{1cm} (10.1)

where $A$ is the only parameter to be optimized, while $b_0$, $h_0$ and $L$ are fixed variables.

From the Von Mises criterion [5], it is necessary to consider the simultaneous presence of normal and tangential stresses present in the same section:

$$\sqrt{\sigma_z^2 + 3\tau^2} \leq \sigma_{id}$$  \hspace{1cm} (10.2)

and, according to the boundary conditions corresponding to a fixed ended beam, the maximum stresses $\sigma_{\text{max}}$ and $\tau_{\text{max}}$ are simultaneously reached at the extremity sections, allowing so to impose a single constraint regarding stresses (Eq. (10.2)). Finally, the optimization problem can be formulated as follows:

\begin{align*}
\text{Find the:} & \quad \min V ol_{(1)} \\
\text{by imposing the constraints:} & \quad g_1 \leq \sigma_{id}; \quad g_2 \leq \frac{1}{250} L; \quad \text{with} \quad \sigma_{id} = f_y / 1.4
\end{align*}  \hspace{1cm} (11)
where \( g_1 \) is described by the Eq. (10.2) and \( g_2 \) represents the maximum allowable displacement (\( \delta_{\text{max}} \)).

### 3.1 Numerical Model Using Matlab

The above optimization problem was solved by implementing a suitable MATLAB® code, by assuming the beam properties summarized in Table 1.

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Value</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>( h_0 ) [m]</td>
<td>2.7</td>
<td>section height at the extremes, constant</td>
</tr>
<tr>
<td>( b_0 ) [m]</td>
<td>0.9</td>
<td>section width, constant</td>
</tr>
<tr>
<td>Span (L) [m]</td>
<td>21</td>
<td>Beam span</td>
</tr>
<tr>
<td>( \gamma ) [N/m^3]</td>
<td>2.5*10^3</td>
<td>material weight</td>
</tr>
<tr>
<td>( q_0 ) [N/m]</td>
<td>100*10^3</td>
<td>uniform live loads</td>
</tr>
<tr>
<td>( E_{\text{mod}} ) [N/m^2]</td>
<td>3.6*10^8</td>
<td>Young’s modulus</td>
</tr>
<tr>
<td>Data beam ( \sigma_{\text{id}} ) [N/m^2]</td>
<td>21.4*10^6</td>
<td>Von Mises Stress</td>
</tr>
</tbody>
</table>

By introducing the boundary conditions (fixed ended beam), the optimization problem was solved through MATLAB®-ga (Find the minimum of the function using genetic algorithm).

The use of a Genetic Algorithm made it possible the comparison, reported in the following sections, with the results obtained by implementing the optimization problem in Grasshopper® environment.

A Genetic Algorithm initially creates a random initial population and successively a sequence of new populations. At each step, the algorithm uses the individuals in the current generation to create the next population. The algorithm stops when one of the stopping criteria is met. In particular, the \( ga \) solver was set as follows: Constraint Tolerance: \( \varepsilon \); Max Stall Generations: 100; Population Size = 10; Function Tolerance = \( \varepsilon \). The Constraint Tolerance is used to determine the feasibility with respect to nonlinear constraints. The algorithm stops when the average relative change in the fitness function value over Max Stall Generations is less than the Function Tolerance.

#### 3.1.1 Matlab-ga Results

After 100 generations, the optimal one lobe sine emptying function solution is \( \Delta h = 1.7766 \text{ m} \); the initial volume was \( V = 51.0300 \text{ m}^3 \). The minimum volume is \( V = 29.6539 \text{ m}^3 \). The imposed constraints are not violated: \( g_1 = 20.98 \text{ MPa} < \sigma_{\text{id}} \) with \( \sigma_{\text{id}} = 21.40 \text{ MPa} \). The Deflection \( \delta \) (\( g_2 \), in mid-span) is equal to 11.65 mm < \( L/250 \) (84.00 mm).

The following images show the graphs related to the post-optimization data:

![Graphs](image)

Fig. 2: a) displacement and material graph; b) M, V, and stresses trend; c) optimized shape.
3.2 Numerical Model Using Grasshopper

The optimization process used in this section is a self-automated process held in Grasshopper plug-in canvas. In the entire process different plug-in are involved: i) Grasshopper®, adopted for the parametric model; ii) Karamba3D® used to obtain the output for FEA Results; Octopus® plug-in adopted as optimization solver. In the proposed optimization problem, the only parameter considered as variable design vector is the amplitude $\Delta h$ (Figure 3), while the displacement $\delta$ and the Von Mises stresses represent the constraints, as formulated in Eq. (11).

After fixing the span length and the height of the beam - described by the constant parameters $L$ and $h_0$ - it was necessary to discretize the entire length of the span to derive the curve described by the sine function. The following parameters have therefore been defined:

\[ c = \log(0.5, \alpha)^n \]  
\[ \alpha = \Delta h \in [0; h_0] \]  
\[ f(x) = \alpha \cdot 0.5 \cdot (\cos(x) + 1) \]

where: $c$ represents the crest factor of the sine-function (moving on x axis); $n$ is the number of steps on curve (total points of the discretized curve); $\alpha$ describes the range of the amplitude of the sine-function.

By adopting Karamba3D it was possible to reconstruct the entire beam geometry by assigning the characteristics of a shell to the element described in Table 1. The results obtained by Karamba3D (FEA), considering the parameter $\alpha = \Delta h = 0$ and imposing the variable load $q_0 = 100$ kN/m, are the following: Mass: 127575.002253 (kg); Displacement ($\delta$): 0.177063 (cm).

To correctly calculate the main stresses in the beam for each point of the previously triangulated shell - by the "MeshtoShell" component in Karamba3D - a Python script was connected (Fig.4) by means of Gh-Python component; the result regarding the Von Mises Stress is: $\sigma_{\text{max}} = 8.1$ Mpa $\leq 21.4$ (MPa). So, by the Python Script it was possible to extrapolate, in a specific portion of the mesh, the normal stress values in X and Y direction and the tangent stress value in direction XY (with the List Item command) and then to achieve the corresponding Von Mises 3D stress value.

![Fig. 3 Shell element and assigned geometry using Grasshopper.](image)

![Fig. 4 Gh-Python Component to calculate Von Mises Stress for each point in the developed Shell.](image)

3.2.1 Hypervolume Estimation Algorithm for Multi-Objective Optimization

The evolutionary algorithm named Hype, integrated into the Grasshopper environment through Octopus plug-in, is based on the fitness assignment [6] [7]. It allows to search for many objective functions at once introducing the Pareto-Principle for Multiple Goals; to formulate the problem summarized in Eq. (11), the Boolean Hard Inequality Constraints were imposed. This procedure allows to obtain different solutions summarized on the resulting Pareto – Optimal Front. Considering the optimization problem in Eq. (11), after 100 generations, the solver returned the graphs in Fig. 5 (on the right), where different solutions are shown (Pareto-Optimal Front). Each solution, marked in red (10 non-dominated solutions), represents a feasible solution in which no constraint is violated. Among all, three different solutions were analysed (Fig. 5) and the respective results are summarized in the Table 2.
Table 2  Results Comparison of Three different solutions (Fig.5(b)) obtained using Grasshopper.

<table>
<thead>
<tr>
<th>Test Case and Results</th>
<th>Δh [m]</th>
<th>Volume [m3]</th>
<th>( \delta = \frac{1}{250} L ) [mm]</th>
<th>( g_1 \leq \sigma_{id} ) [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Before Opt.</td>
<td>0.0</td>
<td>50.04</td>
<td>1.8</td>
<td>8.15</td>
</tr>
<tr>
<td>a</td>
<td>0.46</td>
<td>46.68</td>
<td>6.3</td>
<td>8.51</td>
</tr>
<tr>
<td>b</td>
<td>2.51</td>
<td>27.31</td>
<td>15.3</td>
<td>7.63</td>
</tr>
<tr>
<td>c</td>
<td>1.56</td>
<td>36.29</td>
<td>8.7</td>
<td>8.61</td>
</tr>
</tbody>
</table>

Fig. 5  Three different solutions (a, b, c) on Pareto-Optimal Front (schematized on the right)

3.3  Results Comparison between MATLAB-ga and Grasshopper-Octopus

By adopting two different methodologies for the numerical solution of the optimization problem in Equation (11) - one integrating the fourth-order ordinary elastic line differential equation including the effects of a variable inertia (MATLAB+ga) and the other evaluating a shell element through finite element analysis (Gh+Karamba3d+Octopus) – the corresponding results are comparable, as shown in Fig. 6. The solution "c" was chosen from the Pareto - Optimal Front (Fig. 5), that is mainly in accordance with the MATLAB-ga resulting shape. Moreover, solution "c" can be considered the safest one (unlike solutions “a” and “b”). Table 3 summarizes the comparison between the two numerical models.

Fig. 6  Results Comparison
Table 3  MATLAB-ga and Gh-Octopus Results Comparison

<table>
<thead>
<tr>
<th>Solvers</th>
<th>Parameters</th>
<th>Before Opt.</th>
<th>Optimized</th>
<th>Ratio [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Matlab+ga</td>
<td>Volume [m³]</td>
<td>51.03</td>
<td>29.65</td>
<td>-41.9</td>
</tr>
<tr>
<td></td>
<td>δ [mm] (mid-span)</td>
<td>1.5</td>
<td>11.6</td>
<td>+673.3</td>
</tr>
<tr>
<td></td>
<td>Δh [m]</td>
<td>0.0</td>
<td>1.77</td>
<td>Inf.%</td>
</tr>
<tr>
<td>Gh+Karamba3D+Octopus Result (c)</td>
<td>Volume [m³]</td>
<td>51.03</td>
<td>36.29</td>
<td>-29.26</td>
</tr>
<tr>
<td></td>
<td>δ [mm] (mid-span)</td>
<td>1.8</td>
<td>8.7</td>
<td>+383.3</td>
</tr>
<tr>
<td></td>
<td>Δh [m]</td>
<td>0.0</td>
<td>1.56</td>
<td>Inf.%</td>
</tr>
</tbody>
</table>

4 Conclusions

In this paper, a practical solution has been provided to reduce the self-weight of an important structural element such as a beam, combining the needs of shape-control to merge efficient solution and architectural narrative. Two different methods have been proposed. The first one integrates a scheme based on the fourth-order ordinary elastic line differential equation, considering the effects of variable inertia. To solve this problem, a suitable optimization algorithm was developed using MATLAB+ga.

The second method is based on Algorithm Aided Design (employing Grasshopper + Karamba 3D+Octopus) and allowed to validate the analytical method proposed in Section 2. The slight discrepancy between the results generated by the two strategies is due to the different algorithms used to obtain the solution. Furthermore, a small variation of the FEM results was observed by varying the resolution of the mesh: refining it, the data of the Finite Element Analysis gradually approach to the solution obtained by the model based on the Timoshenko kinematics hypothesis.

Finally, good accordance between the two methods was obtained, especially in terms of shape, so validating the proposed analytical method.

Acknowledgements

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References

Robustness-oriented conceptual design of precast concrete frame structures

Simone Ravasini, Beatrice Belletti, Emanuele Brunesi, Roberto Nascimbene, Fulvio Parisi

Abstract
Ongoing research on structural robustness of precast concrete (PC) structures is a source of debate in the structural engineering community because few studies are currently available in the literature exploring it thoroughly. The importance of preventing progressive collapse of such structures relies upon redistribution capacity of structural members through connections and tying systems. A recent study has demonstrated that continuous ties along members can substantially improve the progressive collapse performance of PC buildings. In this paper, quantitative data from nonlinear dynamic analyses of a frame structure under peripheral column removal at different floors is processed to identify the most demanding scenarios. Furthermore, the dependency of the robustness assessment on the ties’ mechanical properties has been investigated both in terms of resistance and ductility. Finally, some remarks concerning the design of key elements, such as columns placed at the corner of the building, are given. The study is aimed at driving the conceptual design of PC frame structures towards robust solutions able to mitigate the risk of progressive collapse under extreme events.

1 Introduction
After the terroristic attack on the World Trade Center in 2001, the engineering research community has focused on progressive collapse of steel and reinforced concrete (RC) buildings [1]. Accidental actions or unforeseen events may cause loss of vertical load-bearing elements, after which the structure survivability relies upon the redistribution of gravity loads to the adjacent undamaged elements. Indeed, the structural robustness is defined in Eurocode 1 [2] as the “ability of a structure to withstand events like fire, explosions, impact or the consequences of human error, without being damaged to an extent disproportionate to the original cause”. The main resisting mechanisms to resist progressive collapse in cast-in-place RC structures involve compressive arch/membrane action at moderate deformations, and tensile catenary/membrane action at large deformations.

Few experimental tests were performed on reduced-scale specimens to investigate the progressive collapse performance of PC planar frames [3]–[5] and beam-slab sub-assemblies [6]. Recent works have shown that the contribution from concrete topping mesh reinforcement is crucial to mobilise diaphragm action in hollow-core slabs [7]. Tohidi et al. also investigated the role of longitudinal joints in hollow-core slabs [8]. Very few researchers investigated special joints and detailing in order to improve both seismic and progressive collapse performances [9]. In general, PC buildings could withstand actions due to vertical element loss through connection detailing, diaphragm resistance and tying systems [10], whose role is crucial, but lack of exhaustive indications exist from experimental, numerical and normative points of view [11]. European PC buildings commonly use dry connections and mechanical devices, such as threaded dowels, to connect beam and column precast members [12].

In a recent study carried out by authors [15], an easy-to-use reliable numerical modelling approach was developed and applied to a PC frame building to assess its progressive collapse performance. It has been demonstrated that typical European-designed PC systems are more prone to progressive collapse compared to counterpart monolithic RC systems. Nevertheless, it was observed that a fundamental role is played by continuous tying reinforcement placed along beam members and through beam-to-column connections, which were able to provide load paths to resist gravity loads.

This paper presents conceptual design considerations based on the aforementioned previous work [15]. The progressive collapse performance of a frame structure under peripheral column removal at different floors is analysed and most demanding scenarios are identified. The dependency of the robustness assessment on the ties’ mechanical properties both in terms of resistance and ductility is also investigated. The findings of the present study aim to provide useful design considerations for practicing engineers based on the numerical results obtained with nonlinear finite element analyses.

2 Case study and numerical modelling

This study presents the continuation of a previous work of the authors on the progressive collapse performance of a frame structure [15] by analysing different scenarios of floor column removal locations. Furthermore, the dependency of the robustness assessment on the mechanical properties of ties has been investigated by paying particular attention to the selection of the ultimate strain values.

The building consisted of a three-storey RC structure, with plan dimensions equal to 43.2 and 28.8 m along the longitudinal (x) and transversal (y) directions. The inter-storey height is 3.3 m at the first floor and 3.6 m at remaining floors, see Figure 1a-b.

Figure 1 Case study building: (a) plan with orientation of one-way floor systems; (b) elevation with different scenarios of peripheral column’s removal; (c) detailing of beam-to-column connection; (d) link relationship of beam-to-column connections (dimensions in mm).

Both moment resisting frame (MRF) and precast concrete frame (PCF) systems were analysed. Columns have 600 × 600 mm² square cross section, beams have 500 × 700 mm² at the first and second floor levels, while beams with 500 × 500 mm² square cross section are used at the 2nd floor level and along longitudinal direction. Longitudinal reinforcing bars have 22-mm diameter and 12-mm stirrups.
with 60 mm spacing are used in beams. In the precast concrete system, beams and columns are connected through a single Ø18 threaded dowel and joints are completed with in-situ fillings. Two 28-mm diameter tying rebars were provided in the PCF system to meet requirements of fib Bulletin 63 [16].

In the case of PCF system, the connections between precast beams and columns were modelled using “link” elements. Uncoupled moment-rotation and shear-displacement behaviours were modelled using isotropic hardening quadrilinear asymmetric relationship, as shown in Figure 1c-d. To model the continuous tying reinforcement, a truss element is placed between column and beam nodes. Dynamic analyses were carried out according to the following steps: (i) gravity loads were applied to the structure through distributed masses on beams, (ii) the column was removed at 0.01 s after the structural static equilibrium was achieved, and the total analysis time was fixed to 3 s. Gravity loads were derived from the Unified Facilities Criteria [17].

Nonlinear time history analyses (NLTHA) were carried out using SeismoStruct FE code [18]. The force-based (FB) fibre modelling approach [19], with distributed plasticity formulation, was used to model material nonlinearity of beam and column members. Geometrical nonlinearities were also included through a total corotational formulation. Bilinear stress-strain curve for steel and Mander model [20] for concrete were used. Compressive cylinder strength of concrete $f_{c}$ was assumed equal to 50 MPa with elastic modulus $E_c = 32.5$ GPa. Both longitudinal, transverse reinforcement and threaded dowels had yield strength $f_y = 500$ MPa and elastic modulus $E_s = 200$ GPa. In a previous study by the authors [15], the hardening ratio $k$ and steel ultimate strain $\varepsilon_{su}$ were set equal to 1% and 20%, respectively, according to other studies in the literature [21], [22]. Diaphragm contribution was neglected, and only its self-weight was considered in the simulation. Moreover, the diaphragm was removed at the column-loss level, while an in-plane rigid behaviour was used at other floors not directly affected by column removal. For further details about FE modelling and connection can be found in [15].

<table>
<thead>
<tr>
<th>Column removal</th>
<th>Load $Q_b$</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ground and 1st Floor</td>
<td>47.50</td>
<td>[kN/m]</td>
</tr>
<tr>
<td>2nd floor</td>
<td>19.00</td>
<td>[kN/m]</td>
</tr>
</tbody>
</table>

In this study, steel properties are varied in terms of strain hardening ratio $k$ and ultimate strain $\varepsilon_{su}$, to investigate their influence on progressive collapse response of both MRF and PC frames, see Table 2. Corresponding ultimate to yield strength ratios $f_u/f_y$ are also reported. Three different steel properties (denoted with label SP) were investigated. Column removals at different floors, one at time, are carried out to identify the most dangerous scenario and related failure modes, see Figure 1b.

<table>
<thead>
<tr>
<th>Steel properties</th>
<th>SP1</th>
<th>SP2</th>
<th>SP3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hardening ratio $k$ (%)</td>
<td>0.51</td>
<td>0.37</td>
<td>0.25</td>
</tr>
<tr>
<td>Ultimate strain $\varepsilon_{su}$ (%)</td>
<td>10.00</td>
<td>17.00</td>
<td>25.00</td>
</tr>
<tr>
<td>Ratio $f_u/f_y$</td>
<td>1.20</td>
<td>1.25</td>
<td>1.25</td>
</tr>
</tbody>
</table>

### 3 Results and discussion

Hereafter, the main results obtained analysing the mentioned scenarios and cases are discussed, focusing on both MRF and PCF systems:

- For MRF system, the main indicators of progressive collapse are the attainment of either a deflection limit equal to the inter-storey height or the rebar fracture in beams, whatever is achieved first.
- For PCF system, the main indicator is the loss of the corbel support, which could be a consequence of dowel and tying rebar fractures. In analogy to the previous study [15], the lateral displacement at which the support is supposed lost is taken as 200 mm. The deflection limit equal to the storey height remains.
Ductile and brittle failure modes are checked for columns in the vicinity of each column loss scenario.

To identify the actual load level at which the structural system is at incipient collapse and at collapse compared with the reference load reported in Table 1, multiple analyses by varying the load multiplier ($\alpha$) are carried out for all floor levels and steel properties reported in Table 2 using the above failure criteria. Results are reported in Table 3. The vertical drift is calculated as the ratio between the vertical displacement at the removed column location and the span length (equal to 10.8 m).

<table>
<thead>
<tr>
<th>Column loss location</th>
<th>Case</th>
<th>System</th>
<th>$\alpha$</th>
<th>Drift [%]</th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ground</td>
<td>SP1</td>
<td>(k=0.51%, $\varepsilon_{su}$=10%)</td>
<td>MRF</td>
<td>SW</td>
<td>9.0</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.4</td>
<td>17.5</td>
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<td></td>
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<td></td>
<td></td>
<td>0.6</td>
<td>15.5</td>
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<td></td>
<td>0.7</td>
<td>16.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.5</td>
<td>18.0</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.6</td>
<td>20.0</td>
</tr>
<tr>
<td></td>
<td>SP2</td>
<td>(k=0.37%, $\varepsilon_{su}$=17%)</td>
<td>MRF</td>
<td>1.1</td>
<td>22.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1.2</td>
<td>23.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.7</td>
<td>20.5</td>
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<td></td>
<td></td>
<td>0.8</td>
<td>21.0</td>
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<td></td>
<td>SP3</td>
<td>(k=0.25%, $\varepsilon_{su}$=25%)</td>
<td>MRF</td>
<td>1.1</td>
<td>22.5</td>
</tr>
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<td>0.8</td>
<td>21.0</td>
</tr>
<tr>
<td>1st Floor</td>
<td>SP1</td>
<td>(k=0.51%, $\varepsilon_{su}$=10%)</td>
<td>MRF</td>
<td>0.5</td>
<td>18.0</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.6</td>
<td>20.0</td>
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<td></td>
<td>0.7</td>
<td>18.0</td>
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<td>0.8</td>
<td>19.0</td>
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<td>0.7</td>
<td>19.0</td>
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<td></td>
<td>0.8</td>
<td>21.0</td>
</tr>
<tr>
<td></td>
<td>SP2</td>
<td>(k=0.37%, $\varepsilon_{su}$=17%)</td>
<td>MRF</td>
<td>1.1</td>
<td>23.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
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<td>24.0</td>
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<td>21.0</td>
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<tr>
<td></td>
<td>SP3</td>
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<td>MRF</td>
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<td>23.5</td>
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<tr>
<td></td>
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<tr>
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<td>20.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.8</td>
<td>21.0</td>
</tr>
<tr>
<td>2nd floor</td>
<td>SP1</td>
<td>(k=0.51%, $\varepsilon_{su}$=10%)</td>
<td>MRF</td>
<td>0.8</td>
<td>19.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.9</td>
<td>15.5</td>
</tr>
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<td></td>
<td>0.8</td>
<td>19.5</td>
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<td></td>
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<td></td>
<td>0.9</td>
<td>20.5</td>
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<td></td>
<td>0.8</td>
<td>19.5</td>
</tr>
<tr>
<td></td>
<td>SP2</td>
<td>(k=0.37%, $\varepsilon_{su}$=17%)</td>
<td>MRF</td>
<td>1.1</td>
<td>17.5</td>
</tr>
<tr>
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<td></td>
<td>0.9</td>
<td>20.5</td>
</tr>
<tr>
<td></td>
<td>SP3</td>
<td>(k=0.25%, $\varepsilon_{su}$=25%)</td>
<td>MRF</td>
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</table>

3.1 Dependency on ultimate steel strain at different floor levels

Referring to Table 3, vertical drifts and load multipliers are referred to both incipient collapse ($\alpha$) and at collapse ($\alpha_u$) using the mentioned criteria. Important observations are:

a. For both MRF and PCF systems, higher ultimate steel strain $\varepsilon_{su}$ increase the actual load which can be sustained by the structure after the loss of a vertical bearing element. For example, at the ground-floor level, load multipliers at collapse varied between SW (corresponding to self-weight value) to 1.2 times the load reported in Table 1 for MRF system and from SW to 0.8 for PCF system, respectively, see Figure 2a.

b. Due to lower load multipliers, the ground level column removal seems to be the most demanding scenario, according to other studies [21]. At the other floors, the difference in the
actual loads sustainable by both systems was lower, see Figure 2b. This was attributed to the ability of tying reinforcement in PCF system to accommodate the deformation demand and allow redistribution of applied loads. The case of 2nd floor column removal scenario will be explained in the next paragraph.

c. In general, a direct comparison between MRF and PCF is not suitable due to different tying reinforcement and dowel contribution activated during progressive collapse phenomenon.

Moment–rotation relations for MRF system are shown in Figure 3a for ground element removal scenario for two cases: \( \alpha = \text{SW} \) with \( \varepsilon_{su} = 10\% \), and \( \alpha = 0.8 \) with \( \varepsilon_{su} = 25\% \). It is evident that for the first case, bars fractured in tension at left and right plastic hinge locations under only self-weight, while for the second case bars did not fracture and the structure was able to sustain further load. This indicates a considerable reduction of system ductility and load redistribution capacity due to lower ultimate steel strain. In addition, for the same cases, it is possible to compare axial load vs elongation at left-span beam for MRF system and axial tie load vs elongation for PCF system at left-span, see Figure 3b. It can be observed that for the first case, due to rebar fractures, MRF system was not able to develop further axial load, while PCF tying system is still able to redistribute the applied loads. For the second case, PCF tying bars fractured and support was lost, while MRF system was able to sustain further loads due to large yielding of tensile bars without any fracture.

![Figure 2](image.png)  
Figure 2 (a) Vertical drift at collapse vs ultimate steel strain \( \varepsilon_{su} \) at ground floor removal; (b) load multiplier at collapse vs ultimate steel strain \( \varepsilon_{su} \) for different floor removals.

![Figure 3](image.png)  
Figure 3 (a) MRF moment vs rotation of beams at ground-floor removal; (b) axial load vs elongation for MRF left-span beam and left-span tying bar for PCF system at ground-floor removal.
3.2 Failure modes at different floor levels

Figure 4 shows the vertical drift time history of the MRF system subject to the 1st floor column removal in the SP2 case. The ultimate load multiplier $\alpha_u$ is 0.8 which was associated with the tensile rebar fracture at end-joints, while tensile rebars at mid-joints were at incipient fracture. Such failure mode is in accordance with experimental and numerical tests for members subjected to distributed loads [23]. The load multiplier $\alpha = 0.7$ did not trigger fracture of rebars, so the structure was deemed able to sustain and redistribute loads. As stated previously, although beam members crossing the removed column experience an increase in bending moment and shear demand, flexural and brittle failure modes must be checked also in nearby members. In detail, columns are subjected to considerable lateral loads and deformations due to the onset of catenary action in beam members. This condition was evident at the 2nd floor level for both MRF and PCF systems, where the load multiplier is limited by the steel rupture detected at the right adjacent column, see Figure 1b, while no shear failures were detected.

Figure 4 MRF system – SP2 case, loss at 1st floor: (a) vertical drift vs time; (b) rebar fracture locations.

Figure 5a shows the vertical drift time history of the PCF system subjected to the 1st floor column loss in the SP2 case. Under a load multiplier $\alpha = 0.8$, the structure experienced support loss at mid-joint location, because of dowel and tie fractures, as reported in Figure 5b. In Figure 5c and d, the column loss at 2nd floor level was analysed for SP2 case. In this case, it is interesting to observe that the load multiplier was related to the achievement of column failure. Indeed, the column failure governed the whole structural behaviour. Although it is not reported, no beams’ failures and support losses were observed, since the tying reinforcement provided continuity between precast beam and column members and the structure was able to carry the applied loads and accommodate the deformed shape. The column failed due to rebar fracture – as shown in the moment-rotation plot of Figure 5c, indicating that this member was crucial for load redistribution consequently to column loss. Thus, the column must be designed as a “key element”, according to UFC guidelines [17], requiring a careful check of the hierarchy of resistances between RC members [9]. Indeed, checks and verifications aimed to the designated element should be used for planning of eventual flexural/shear strengthening interventions:

- To improve flexural resistance, change in cross section and/or higher reinforcement ratio could be used, eventually in conjunction with improved steel properties (including for example high ductility bars).
- To improve shear resistance, change in cross section and/or higher reinforcement ratio could be used, eventually with lower stirrups spacing (useful to avoid any eventual buckling of longitudinal bars) and the use of high ductility bars.

The previous observations highlight the importance of the structural design stage as an integral part of overall design process. The identification of the most dangerous removal scenarios and critical members is crucial for planning structural upgrading interventions and strategies for mitigation of progressive collapse risk.
4 Conclusions

In this paper, a reinforced concrete frame building was analysed under different column loss scenarios related to unspecified events. MR and PC frame systems were investigated. The influence of steel mechanical properties, tying reinforcement and column removals at different floors were critically analysed. The study aims to highlight the importance of structural design stage inserted in an overall design process, by focusing on the progressive collapse phenomenon of precast concrete structures. The following observations are related to structural design process:

- Load which can be sustained by both MRF and PCF systems increases with ultimate steel strain, allowing more efficient load redistribution among adjacent members. The MRF system generally performs better than the PCF system at the ground floor level, while the difference between such structural typologies is less evident at upper floors. The ground-floor level seems to be the most demanding scenario.
- Beam members at different floor levels must be checked for ductile/brittle failure modes, due to the increase of bending moment and shear forces associated with column loss. Structural failures were related to established collapse criteria. Such event may lead to structural collapse, and the structure is considered safe if gravity loads can be efficiently redistributed through adjacent members. In particular, the column removal at 2nd floor level have shown the vulnerability of adjacent columns.
- Current design requirements and experimental studies still lack in the current literature, especially aimed to tying systems in PC structures. For these reasons, further experimental and numerical investigations could help on this front.

Figure 5  PCF system – SP2 case: (a) drift vs time and support loss location after column removal at the 1st floor; (b) support relative displacement vs time after column removal at the 1st floor; (c) moment-rotation response of adjacent column after column removal at the 2nd floor and (d) location of column failure after column removal at the 2nd floor.
References


Conceptual Design of Fiber Reinforced Concrete Elements

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Link to the video: https://youtu.be/Vwc3koJX1tc
R-funicularity for shells’ shape optimization

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Abstract
The request for sustainable structures has encouraged the development of structural optimization methods aimed to design efficient structures and to minimize the use of construction materials. Funicular structures are an example of optimized structural systems, being able to bear the design load without exploiting bending moments. Here, we have used the eccentricity measure to study and quantify the funicular behaviour of 1D and 2D structures. Then, we have formulated an optimization process where the structures’ shape is updated in order to make them as much as possible funicular in a relaxed form. We have carried out numerical experiments to test the algorithm, the early results shown an improvement of the structural efficiency.

1 Introduction
The minimization of construction material is a relevant theme nowadays, in fact the construction industry largely contributes to depletion of natural resources and CO₂ emissions [1]. Structural designers could reduce this negative impact by targeting efficient structural behaviour and by minimizing the use of construction materials. Among efficient and optimized structural systems, funicular structures adopt the “right” shape in accordance with the applied load. As a result they are ideally able to bear that particular load without introducing bending, pursuing a full exploitation of the resistant cross section and a minimum waste of material.

1.1 About funicularity
Before the analytical formalization of the idea of funicularity, occurred between 15th and 17th century, many funicular structures had already been constructed thanks to experience and static considerations of the designers and constructors [2]. The earliest written essays concerning this topic date back to the 13th century. The medieval architect Villard de Honnecourt in his manuscript “Livre de portraiture”, describes how to construct a cross vault optimizing the entire process with the application of the rule of the three arches [3]. At a later date, it has been confirmed that cross vaults constructed respecting this rule have a better structural behaviour because the bending stresses are reduced [4], demonstrating an intuition of the relationship between shape and performance of the structure. Starting from the 15th century, the first studies on arches and cables appear. Leon Battista Alberti (1404–1472), Andrea Palladio (1508–1580), Leonardo da Vinci (1452–1519) and Simon Stevin (1548–1620) are some of the most celebrated scientists to give a fundamental contribution to the formulation of the behaviour of curved structure and of the arch equilibrium [2]. Galileo Galilei (1564–1642) was the first one who attempted to give a mathematical description of a cable; in his writing “Dialogues Concerning Two New Sciences” (1638), mistakenly using an erroneous analogy with the parabolic motion of projectiles, he confused catenary and parabola [5]. Joachim Jungius (1587–1657) rejected Galilei’s statement, demonstrating the difference between the two. Jungius’s writing “Geometria Empyrica” was published in 1669 after his death [2]. The correct equation of a cable’s geometry was written in 1691 by Gottfried Wilhelm von Leibniz (1646–1716), Christiaan Huygens (1629–1695) and Johann Bernoulli (1667–1748) [6]. Huygens was the first one to use the term “catenaria” in one of his missive to Leibniz. English engineer and scientist Robert Hooke (1635–1703) gave an additional fundamental contribution in 1676 publishing a Latin anagram whose solution was published by the secretary of the Royal Society, Richard Waller in 1705 and read “Ut pendet continuum flexile, sic stabit contiguum rigidum inversum”,
translation is “As hangs the flexible line, so but inverted will stand the rigid arch” [7]. The concept is simple: in order to obtain an arch that acts in pure compression, the shape of the equivalent hanging chain needs to be inverted. During the same years, David Gregory (1659–1708) stated that an arch is stable if the thrust line, that is the line representing the path of the resultants of forces acting in a structure, lies within its thickness [8]. This is the basic concept behind the structural assessment of masonry structures. Some years later, following Gregory’s studies, Claude-Louis Navier (1785–1836) and E. Méry (1840) supposed that in order to have an arch fully compressed, the thrust line would have to lie within the middle third of his section [9]. The thrust line becomes an indicator of the stability of arches: the more this line lies away from the axis of the arch, the more its thickness needs to be increased. Hence the “right” shape for an arch is the one corresponding to the funicular of the loads applied.

1.2 Funicularity evaluation

When a 1D structure is considered to be inextensible and locally statically underdetermined the pure equilibrium problem can’t be solved. It is then possible to introduce the shape as unknown in order to find the equilibrium solution. This allows to determine a proper geometry for the 1D structure to be in static equilibrium with the considered load case in a pure axial regime, i.e. a funicular structure. As shown in [10] the elastic problem of a 2D structure without bending stiffness, i.e. a membrane, is statically determinate. In fact, in theory, as confirmed by different authors, a shell properly supported can carry any load by membrane action only. Belluzzi [11] declares that “the behaviour of a membrane differs from that of a cable. […] The membrane is always in equilibrium for every external force and irrespective of its initial shape, and this equilibrium is satisfied solely by means of the internal membrane forces”. In Pizzetti et al. [12], it is stated that “from a theoretical point of view, one could expect to oppose any curved thin surface to any load, confident that this surface will organize to perform statically at its best, that is a bending-free behaviour.” However bending-free behaviour is only valid under two assumptions: the boundary conditions are congruent with the shape considered and the load applied, and the ratio between membrane stiffness and flexural stiffness tends to infinity; therefore a pure membrane model is applicable. In this circumstance the structure is locally statically determinate and the equilibrium equations can be solved directly. Calladine [13] confirms that for a membrane “the nature of the solution may depend on the shape of the shell surface and the nature of the boundary conditions.”. Summarizing for a given shape and load, a funicular behaviour can be found assigning the right boundary conditions to the analysed structure; otherwise, for a given load and boundary conditions, the right shape needs to be found in order to obtain a no-bending behaviour. Hence defining the “right” shape becomes crucial and different form finding techniques can be used for this purpose. Furthermore, when designing an actual surface, an elastic shell problem has to be solved: the flexural stiffness starts playing a role and the problem becomes statically indeterminate. Bending moments, even if minimal, will arise. This also happens for form found shell surfaces when the thickness increases and consequently the ratio between membrane stiffness and flexural stiffness decreases.

The mutual dependency between shape and applied load leads to the common criticism made on the effective application of form finding methodologies; they can be suited to preliminary design, but no-bending behaviour cannot be guaranteed for multiple relevant load cases [14]. When this is the case and bending moments cannot be avoided, can the shell be still considered funicular? An answer to this question has been given in [15] introducing a relaxed definition of the term funicularity and proposing an instrument able to quantify the funicularity. An interesting example of redefinition of funicular arches, whose initial shape is not funicular under given applied loads and boundary conditions, can be found in [16], where the funicularity is recovered by an optimized system of post-tensioning cables. A useful parameter to define the condition of funicularity in arch structures is the eccentricity associated to the normal force and related to the bending moment acting on a section. This parameter has been used also for shell structures by defining a proper generalization of it. To the authors’ knowledge, the first ones to use eccentricity as a parameter to quantify the funicularity of optimized shells are Marino et al. [17]. Earlier Lucchesi et al. [18],[19] proposed a numerical method allowing for analyzing masonry vaults, where the calculation of the maximum modulus eccentricities surface (MMES) has been proposed in the context of no-tension materials, as a tool for limit analysis.

In the next section, the measure of eccentricity is used to study the funicular behaviour of a parabolic arch. In section 3, a shape optimization process for 1D structures, based on the minimization of eccentricity, is introduced; the optimized shape and the behaviour of the arch are discussed. The shell’s
shape optimization process, devoted to finding a shape for the shell to be funicular in a relaxed form, is briefly presented in section 4. Some conclusions and future perspectives close the paper.

2 Funicularity investigation of a two hinged parabolic arch

Let’s consider a two hinged parabolic arch subject to a vertical and uniform load \( q \), as shown in Fig. 1.

![Fig. 1 Two hinged parabolic arch subject to the vertical and uniform load \( q \). Cross section of the arch.](image)

Its shape is described by the equation (1),

\[
y(x) = \frac{x^2}{2a} + C_1 x + C_2
\]

where \( a = \frac{H}{q} \), being \( H \) the horizontal thrust, \( C_1 \) and \( C_2 \) have been determined by imposing the conditions (2).

\[
\begin{align*}
y(0) & = 0 \\
y(L/2) & = f \\
y(L) & = 0
\end{align*}
\]

where \( L \) is the arch span and \( f \) the arch rise.

With the aim of investigating the mechanical behaviour of the arch an algorithm linking a MATLAB script with the Finite Element (FE) software SAP2000 has been developed. This allows to evaluate the shape coordinates, to build and analyse a FE model (see Fig. 2) of the structure. For all the analyses presented in this work the spacing of the \( x \) coordinates is \( 0.2 \text{ m} \), the arch span \( L = 6 \text{ m} \) and the uniform vertical load \( q = 10 \text{ kN/m} \). A rectangular concrete cross section with an elastic modulus \( E = 32000 \text{ MPa} \) has been considered.

![Fig. 2 Finite Element Model of a two hinged parabolic arch with \( f/L = 0.5 \). Beam elements of different lengths have been used to model the structure.](image)

Then, we have carried out two parametric analyses in order to study the funicular behaviour of the structure. The first one is characterized by fixed cross section dimensions and decreasing values of the \( \frac{f}{L} \) ratio. Conversely, in the second analysis, the ratio between axial and bending stiffness, that is the cross section height, has been varied whereas the ratio \( \frac{f}{L} \) has been fixed. The bending moments \( M \) and the axial force \( P \) trend has been observed in both the analyses (see Fig. 5). In addition, the mean values of \( M \) and \( P \) measured in the 15th element have been linked through the nondimensional eccentricity \( e = \frac{M}{h*P} \) (see Fig. 3).
behind the curtain

Fig. 3  Solid blue line: $e$ versus $\frac{f}{L}$ decreasing values. Dotted black line: eccentricity threshold for the arch to be compressed, according to the middle third criterion: $e_{\text{lim}} = \frac{1}{6}$. Left: $b \times h = 0.3 \times 0.15 \text{ m}$. Right: $b \times h = 0.3 \times 1 \text{ m}$.

It can be noticed that when the bending stiffness $EJ$ is low compared to the axial one $EA$, $e < e_{\text{lim}}$ for a wide range of $\frac{f}{L}$, exceeding the limit for $\frac{f}{L} < 0.024$ (see Fig. 3, left). A different behaviour is shown for a high ratio $EJ/EA$: $> e_{\text{lim}}$ when $\frac{f}{L} < 0.16$.

Fig. 4  Nondimensional eccentricity along the arch for different values of cross section height. Left: $\frac{f}{L} = 0.5$. Centre: $\frac{f}{L} = 0.15$. Right: $\frac{f}{L} = 0.05$.

In Fig. 4 the trend of $e$ along the arch is shown for different values of cross section height. It emerges that for high values of $\frac{f}{L}$ the structure is funicular in a relaxed form, meaning that $e$ belongs to the middle third of the considered section, in mathematical terms: $-e_{\text{lim}} < e < e_{\text{lim}}$. As $\frac{f}{L}$ grows a non funicular behaviour emerges as $h$ increases. It is worth to specify that for each frame element of the structure the mean value of $M$ and $P$ has been used in order to evaluate $e$ and plotted in the centre of its length projected along the $x$ direction. Because the sections close to the arch keystone are representative of its behaviour, we have summarized in Fig. 5 the analyses results expressed in terms of mean of $M$, $P$ and $e$ for the 15th element.

Fig. 5  $M$, $P$ and $e$ evaluated for the 15th element of the arch for increasing values of the cross section height.
The performed parametric analyses highlighted that, for a given cross section, the funicular behaviour of the studied parabolic arch is influenced by the ratio $f/L$. For a given arch shape, instead, the $h/L$ ratio represents a crucial parameter for funicularity. In the light of this, we have developed an optimization process where the shape of an arch with given cross section is modified in order to obtain an as much as possible funicular structure. The arch’s shape optimization process, discussed in the section below, constitutes the basis for the shell’s shape optimization process.

3 Arch’s shape optimization based on R-Funicularity

According to the definition of Relaxed Funicularity (R-Funicularity) presented in [15], a structure can be considered R-Funicular if the eccentricity belongs to an admissibility range that, applying the middle-third criterion, can be defined as $\left[-\frac{h}{6}, \frac{h}{6}\right]$. In the light of this we can assert that a 1D structure can be considered funicular in a relaxed way when:

$$e(s) = \frac{M(s)}{hP(s)} \in \left[-\frac{1}{6}, \frac{1}{6}\right]$$

(3)

The parametric analyses performed and presented in the previous section have highlighted that the behaviour of a statically indeterminate arch, in general, is not funicular, that is in fact, the eccentricity is not null all over the structure. It emerged, instead, that for a wide range of $f/L$ ratios and $E/E_A$ values the two hinged arch behaves as an R-Funicular structure.

3.1 Shape optimization process for 1D structure

Here, we have developed an optimization process aimed at modifying the shape of an arch in order to make it R-Funicular in each section. The iterative optimization procedure is summarized in the following diagram:

For optimization purposes the arch’s shape has been represented by a 4th order polynomial equation:

$$y(x) = c_4 x^4 + c_3 x^3 + c_2 x^2 + c_1 x + c_0$$

(4)
By applying the two boundary conditions and fixing the arch’s rise (see Eq. (2)) the unknown coefficients $c_0, c_1, c_2$ have been written in function of the arch’s geometric parameters and of $c_3$ and $c_4$, as follows:

\[
\begin{align*}
c_0 &= f \\
c_1 &= -\frac{c_3 L^2}{4} \\
c_2 &= -\frac{(16f + c_4 L^4)}{4L^2}
\end{align*}
\]  

(5)

The starting shape is a parabola obtained as the reversed funicular shape of a cable subject to a uniform vertical load. That parabola can be obtained from equation (4) by imposing $c_3 = c_4 = 0$.

### Table 1  Shape optimization settings

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value/Description</th>
</tr>
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<tbody>
<tr>
<td>$f/L$</td>
<td>0.15</td>
</tr>
<tr>
<td>$L$</td>
<td>6 m</td>
</tr>
<tr>
<td>$b \times h$</td>
<td>0.3 $\times$ 1 m</td>
</tr>
<tr>
<td>Boundary conditions</td>
<td>Pinned Boundary</td>
</tr>
<tr>
<td>Objective Function (based on the R-Funicularity criterion)</td>
<td>$f_{err}(e) = \sqrt{\sum(e_i - \frac{1}{6})^2} + \sum e_i^2$</td>
</tr>
<tr>
<td>Software</td>
<td>MATLAB (scripts for evaluation of eccentricities, optimization, shape update)</td>
</tr>
<tr>
<td></td>
<td>SAP2000 (FE solver)</td>
</tr>
</tbody>
</table>

The independent coefficient values after the optimization are: $c_3 = 3.06 \times 10^{-9}$, $c_4 = 2.35 \times 10^{-3}$ and the error function value decreases slightly, from 0.790 to 0.768. The resulting shape and the eccentricity distribution is shown in the Fig. 6.

The slight decrease of error function corresponds to a more uniform eccentricity distribution. The eccentricity worsens next to the arch’s extremities, but still remains within the boundaries. The elements closer to the arch keystone show the eccentricity that decreases, resulting the entire structure to be R-Funicular.

![Fig. 6](attachment:image.png)
4 Shells’ shape optimization based on R-Funcularity

The concept of R-Funicularity for bidimensional curved structures, i.e. shell structures, presented in [15], is based on the generalized eccentricity \( e(\theta) \) that allows to quantify the eccentricity in each point \( P \) of the surface and in each direction \( \theta \) (see Fig. 7).

The generalized eccentricity is defined as the ratio between the generalized bending and membrane internal forces:

\[
e(\theta) = \frac{M(\theta)}{N(\theta)}
\]

being

\[
N(\theta) = u^T Nu \quad M(\theta) = u^T Mu
\]

where \( u, N \) and \( M \) are, respectively, a generic direction from \( P \), the membrane forces and the bending moments tensors:

\[
N = \begin{pmatrix} N_{11} & N_{12} \\ N_{12} & N_{22} \end{pmatrix} \quad M = \begin{pmatrix} M_{11} & M_{12} \\ M_{12} & M_{22} \end{pmatrix}
\]

(8)

From Eq. 6 and Eq. 7 it is possible to express \( e \) as the Rayleigh quotient

\[
e(\theta) = \frac{u^T Nu}{u^T M u}
\]

whose associated eigenvalue problem is:

\[
(M - eN)u = 0
\]

(9)

The eigenvalues of problem (9) represent the maximum and minimum eccentricities \( e_{\text{max}}, e_{\text{min}} \).

Here, we just want to show that the same optimization process described for the 1D curved structure in the previous sections, can be reformulated and applied to shells. In this case, the goal is to minimize the generalized eccentricity all over the surface; a proper objective function depending on the eccentricity extrema is defined on purpose. A more detailed discussion can be found in [20]

As an example, in Fig. 8, the result of the optimization of a concrete parabolic velaroidal shell subject to self-weight load is depicted. The structure is clamped in the boundary vertices and along the edges every four meters, the rest of the boundary is pinned. The shell’s shape is obtained by a linear combination of three shape functions, the first of which is a parabolic velaroid. The coefficients of the linear combination are updated in the optimization process. Besides the slight shape change, a decrease of the eccentricity can be noticed, together with a more uniform eccentricity distribution in the optimized shell also towards the restrained boundaries, where bending moments are expected to be larger.

Fig. 8 Eccentricity distribution of a parabolic velaroid shell, with thickness \( t = 0.10 \) m before (Left) and after (Right) the optimization process. The colorbar shows \( \max(\{|e_{\text{min}}|, |e_{\text{max}}|\}) \) where \( e_{\text{min}} \) and \( e_{\text{max}} \) are, respectively, the minimum and the maximum dimensionless eccentricity as defined in [15]. The dimensionless eccentricity grows going from blue to red.
5 Conclusions

The results of the parametric analyses carried out have shown that the funicularity of a statically indetermined arch is not always achieved. On the other hand, the arch’s behaviour appeared to be R-Funicular for most of the $f_L$ ratios considered in this work. The optimization procedure has proved to be a useful tool to improve the mechanical response of a double hinged arch. In fact, its non funicular behaviour in the keystone area turned into an R-Funicular behaviour after the optimization. The same optimization problem, adapted to 2D structures, has been able to improve the eccentricity distribution of a parabolic velaroid shell.

Further analyses are required to test the performance of the optimization procedure and will comprise the use of different boundary conditions and applied loads.

We believe that this approach could become a valid tool for optimal shells’ shape definition from the preliminary design phase, a tool for further verification of shapes changes and for analysis of already designed shell structures.

References

A prototype pavilion in textile reinforced concrete: a tool for research and pedagogy

Patricia Guaita, Raffael Baur, Miguel Fernández Ruiz, David Fernández-Ordóñez

Abstract

This paper presents a work performed in the last years on a Textile Reinforced Concrete (TRC) Prototype Pavilion, raising questions about research on architecture and engineering within a pedagogical context. The construction of the pavilion explores how a hands-on approach builds up multi-layered knowledge and constitutes a common ground of communication on which architects and engineers meet. Through the act of construction, architects and engineers work together, generating new knowledge and experiencing how tacit knowledge is built-up and transmitted. It also allows for the processes of conception and manufacture to feed and to enrich each other. This action of making knowledge (τέχνη, techne) constructs a thought or a concept as a tangible physical entity, acting as a communication interface between the work and the mind, spanning over different disciplines.

The paper highlights the fact that the act of building is not only an intellectual and technical task. Within an education context, students experience the complete process of observation, analysis, conception, execution and testing. The direct investigation of materiality is thus essentially an invention and innovation process: an iterative cycle building up knowledge through observation on the making. Through their corporal experience, students identify and engage with the research work, leading to collective action and to individual responsibility. Such engagement opens up perspectives on architectural, engineering as well as social, economic and environmental questions for the 21st century: sustainable and resilient construction, economy of means, adequacy of expression.

1 A Pavilion as an instrument to develop and to exchange knowledge

This contribution is based on a long experience of pedagogical research in developing transdisciplinary strategies for designing and building projects in full scale and real-world conditions. The format of building a prototype construction by students of architecture and engineering offers the opportunity to work at the intersection between research and practice, of teaching and learning, of engineering and architecture. Our professions become more and more specialized and technical and thus we have less and less contexts to have an exchange of knowledge and to advance together in the necessity and evolution of developing the built environment.
Within this context, this paper presents the process of conception and construction of a Textile Reinforced Concrete (TRC) Prototype Pavilion designed and built as part of a research collaboration between a team of architects and engineers at Ecole Polytechnique Fédérale de Lausanne (EPFL, Switzerland) [1]. The project was originated as an investigation on the potential of TRC to update lightweight building techniques with concrete (as argamassa armada [2]), and has evolved into an exploration in the pedagogy of making through prefabrication [3] and tectonic innovation.

The TRC Prototype Pavilion (see Fig. 1) was constructed during two summers at EPFL Fribourg by a group of students and researchers both from architecture and civil engineering. However, it is the result of a series of prior explorations during several academic years. Through observation, drawing and testing, different elements were developed. It is thanks to this iterative process that the necessary knowledge and innovation on the process of construction (formwork, reinforcement, concrete casting, curing and erection) is generated and allows defining useful and efficient building elements to constitute a prototype pavilion.

The TRC prototype pavilion is not thought as a final product, with a plan to be designed, built and exploited. On the contrary, it remains open to transformation. The interest of its construction is learning in the process, improving and understanding the potential of TRC as a sustainable alternative to ordinary concrete construction. The pavilion is thus intended to be a first step towards an adaptable tectonic system, a construction that will be further developed the next years and that leads to a new lightweight building paradigm, forecasting a new and different conception and image of concrete construction.

Fig. 1 TRC Prototype Pavilion: assembly and exposition of elements, EPFL Fribourg 2019 (photo Raffael Baur), EPFL Fribourg 2020 (photo Ana Carvalho).

2 Materiality: lightness, structural performance and architectural language

A major breakthrough in current construction approaches using cementitious-based materials (mostly concrete) can be identified in the use of non-corrosive reinforcement as for instance fabrics in carbon fibres, (see Fig 2). Such an approach removes the need to protect reinforcement from corrosion. As a consequence, reinforcement covers can be significantly reduced with respect to current values. Required centimetres for the covers in ordinary concrete construction become millimetres in TRC [1].

This leads to thinner and lighter structures, where the concrete matrix is used for static needs only and not for protection of reinforcement. In addition, the amounts of clinker required for the cementitious matrix can be dramatically reduced (no need for rebar passivation) reducing the total CO₂ footprint of the construction. Other than sustainability aspects related to the environment, the possibility of building light-weight elements opens the door to self-construction by local communities. This allows for a socially-sustainable approach, allowing to build in high- and low-tech environments with a particular opening to the informal city.
Despite the new potential offered by this material, a clear vision on how to implement it in construction is yet unclear. Probably, inspirations shall not come as an evolution from ordinary (massive) concrete construction, but from thin and lightweight elements, such as shells, Nervi’s work in ferroment and Filgueiras Lima (Lélê)’s work in argamassa armada [2]. Interestingly, the possibilities offered by a light material with a high potential to be prefabricated are also connected to steel construction and its approach to assemble and to connect pieces.

The construction of thin members with TRC allows for a rational manner to exploit the sustainable and durable qualities of TRC while affording spatial quality and allowing inhabitants of the informal city to participate in the production of their own shelter and habitat. The use of TRC allows to combine industrial processes (factory production) with local craftsmanship and labour. Such an approach allows for an efficient transfer of knowledge and technology to multiple environments, allowing also for its adaptation to local contexts. Another important aspect is the potential of TRC for recycling industrial waste products (as fly ash) and its potential capacity to be recycled in the future as for ordinary concrete construction. The use of precast elements in TRC, with convenient connections allows to create flexible and reusable structures in agreement with the concepts of circular economy.

Fig. 2  TRC elements: shell 9mm thick, EPFL Fribourg 2019 (photo Graeg Eaves). Foundation beams, column, slab, EPFL Fribourg 2020 (photo Ana Carvalho).

3  Atelier: tools and actions

One of the hypothesis of the TRC research project is that craftsmanship constitutes a common ground on which architects and engineers meet. The practical, artisanal construction of a structural element and formwork system (i.e. a concrete reality which embodies a unity of architecture and engineering, of research and practical experience), does not demand for a division of analysis (calculation) and design but fosters a constructive understanding including the above mentioned environmental and social considerations.

Working in an atelier (studio) format allows the interaction of our hands and minds, which is an efficient manner to build up and exchange not only technical knowledge but also savoir faire and tacit knowledge between architects and engineers, teachers and students as well as between cultures of different continents.

In the process of life, neurologic structures emerge simultaneously with the development of skills and knowledge. Knowledge is a result of life, of its experiences and of the consciousness of individuals [4]. Every skill, also the most abstract ones, start from physical processes [5]. Understanding of techniques is established thanks to the application of an imagination on action. This process or “material consciousness” is the extension of thinking from the mind to the nervous system [6] and from the hand
to the material world. Our hands are an interface between our brain and the real world, giving us the possibility to interact with ourselves, with materials and processes.

Thinking through action [7] allows understanding and solving situations and to develop an intuitive and tacit knowledge. In many cases, the work of the craftsman is done without a predefined theory of his own work, but by applying experience and intuition to precise contexts. Sharing experience, by dialogue and physical interaction, is thus the most efficient manner to transmit tacit knowledge [8]. Despite the fact that it is possible to distinguish between explicit and tacit knowledge, it is very hard to separate them in practice. Tacit knowledge is subjective and only by doing and through verbal exchange and physical discussion others can have access to it. Such a process generates a type of culture which can be identified as the addition of actions taken in response of specific contexts. One of the tasks of the architect and engineer is to transmit such a culture of tacit knowledge by means of different tools [9].

The distance between an architect or engineer and the process of making has become larger in the last decades as a result of intermediate actors [10]. Also, concepts and physical artefacts have become more and more separate. However, separating design and execution of a work separates the teaching and professional competences, leading to incomplete professional profiles in respect to the complexity of building.

In the atelier, learning is encouraged by the development of technical and spatial skills in reciprocal interaction, fostering a poetic understanding of construction, (Fig. 3). Supported on tools rooted in the core of the profession (drawing, models and physical prototypes), materials are transformed into spatial artefacts. Such actions promote intuition, sensitivity and knowledge transfer [11]. The work integrates all processes implied in construction: conceptual design, material innovation, fabrication and assembly. Such a physical approach is intuitive and allows students for a deeper understanding of building processes and to feel capable of modifying, transforming and developing our environment.

Fig. 3  Atelier Pop Up, EPFL Fribourg 2019, (photo Raffael Baur); prepared formwork and carbon net reinforcement of bean element, EPFL Fribourg 2019, (photo Raffael Baur).

4  Interdisciplinary
During the construction, students work directly with concrete. By designing, building and testing, they enter into a reciprocal relationship, such that construction becomes a projective activity. Everything is fabricated in the atelier, from sketch to prototype, drawing, formwork, reinforcing, casting, erecting and assembling. This way of working faces students to construction reality and its material qualities.
Every generation of students inherits elements built by previous ones, thus a constructive language evolves. Their work is thus not isolated but belongs to a context and collaborative effort, to a culture of making. The construction of the pavilion shows that design and fabrication is a collective act. The time and effort involved in building with one’s own hands reflect interactions with others.

Students need to understand and go through the complete process of analysis/observation, conception, execution and testing, (Fig. 4). This generates a deep learning impact in an interdisciplinary context. The gained experience and confidence of being actively involved in an innovation process encourages students to question and further develop any given task or reality in their coming careers. The interdisciplinary and collaborative nature of the process, which is necessary accounting for the complexity of the situation, can serve as model for any future activity.

Fig. 4  Atelier Pop Up, experimental casting of the columns and shells. EPFL Fribourg 2019 (photo Sergio Ekerman), EPFL Fribourg 2020 (photo Ana Carvalho).

5 Drawing as a cognitive tool

Drawing constitutes a very powerful and critical tool of conceptual design. By articulating thought, it acts as a communication interface between the work and the mind [12] and between different disciplines; it is the most powerful language of communication in the working together between architects and engineers. The act of drawing is physically apprehended: the care in the drawing translates into a care in the making.

During this project on TRC, it is proposed to investigate analytical drawing methods capable of exploring structural and architectural concepts and solutions. Through analytical drawing, students enter into dialogue with the construction process in a direct manner. They get a sense for the adequacy of tools and refinement of solutions. This allows investigating the relation with scale and space, with tectonic articulation in relation to structural idea. An important contribution is also devoted to structural analysis, as the tool to make transparent the parameters and dependencies of the design process and to open the work to predict the structural response.

Drawing is a cognitive process where the dynamic relation between making and thinking is essential. The construction of points and lines on a piece of paper leads students to the notions of scale, size, proportion, transparency and composition, leading to build up their own tacit knowledge. The slow and tactile nature of the drawing process encourages the development and understanding of detail as a key
moment of construction and as a mediator, relating a structure to the body and to the subject perceiving it.

Exploratory working drawings were done at different scales and incorporating calculation notes and a notion of sequence and time of the construction process, leading to generate a complete documentation of all the necessary information for the fabrication of a TRC element (meta-drawing, Fig. 5). This multi-scalar approach, opening and creating multiple relationships, overcomes typical difficulties of conventional presentations based on a linear sequence and acknowledges the potential of parallel and iterative processes in the conception and construction, allowing invention, experimentation and discoveries. These ‘tactile’ drawings allow the observing eye (mind) to survey (travel) from one detail to another while integrating all information into a mental construction.

Fig. 5  Foundation beam, connection detail ENAC UE Argamassa Armada 2019 (drawing Flavio Gorgone) / Shell, ENAC UE Argamassa Armada 2020, (drawing Gianni Verrillo).

6 Conclusion
This paper reflected on the conception and fabrication process of an experimental pavilion in TRC, where the process of conception and manufacture feed each other to develop a thought or a concept into a concrete physical entity (Fig. 6).

It is shown that interactions between architects and engineers can occur naturally with the hand as interface between the ideas and the produced artefacts. In order to make a step forward in the process of building, it is important not only to share technical aspects, but mostly tacit knowledge. The latter, compiling multisensorial experiences, can only be transferred in an efficient manner by means of the making.

Working in an atelier format, with interdisciplinary teams composed of architects and engineers with different levels of experience and backgrounds, reveals to be a very powerful tool to address the challenges of tomorrow, opening a common ground for action and experience in between the body, nature and technology.
Fig. 6 TRC Prototype Pavilion, Atelier Pop Up, Structural Elements in Textile Reinforced Concrete, 2020 (photo Ana Carvalho).

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References
Pushing concrete material usage to the limit: weight optimised, 3D printed concrete girders with external reinforcement

Nadine Stoiber, Benjamin Kromoser

Abstract
Via the consultation of topology optimisation, this study aims at finding weight optimised, externally reinforced concrete structures. The found organic geometries are realised using extrusion-based 3D concrete printing. Significant interactions between the complex girder geometry and the constraints of the manufacturing system are identified. This outlined first trail run of the entire process from preliminary design to nonlinear numerical evaluation as well as experimental testing in the course of full scale 1:1 shear force tests of optimised, externally reinforced concrete girders shows the basic feasibility of the presented process. The paper is concluded with the illustration of a future envisioned optimisation approach.

1 Introduction: Motivation and objective
Climate change represents one of the biggest global challenges of our time. According to [1], the concrete construction industry is responsible for a significant share of total CO₂ emissions worldwide. The related high environmental impact of concrete’s constituent cement with its process emissions due to its inherent chemical conversion process plays a significant role in this context [2]. Furthermore, conventionally designed concrete structures are characterised by little utilisation of their material properties. Reference [3] illustrates the latter issue consulting the example of a concrete floor element with a span of 6 m and a floor thickness of 0.28 m: In the respective case, the dead weight already accounts for approx. half of the total load. Based on this information, the necessity to increase resource-efficiency in concrete construction is demonstrated and serves as a motivator for this study. The intent is to reduce material consumption and enhance material utilisation by optimisation of the topology of concrete parts.

This conference paper outlines a first trail run of the entire process from design to testing of a topology optimised concrete girder with external reinforcement [4]. At first general strategies to weight optimise concrete structures are outlined. In the second part, the case study is presented including an explanation of the design concept, an analysis of the manufacturing restrictions as well as the experimental test series, which is subsequently evaluated by numerical simulations. In the course of the conclusion, a future vision of an optimised concrete construction strategy is depicted.

2 Theoretical background
General strategies that can be pursued to minimise material consumption in concrete structures include optimising material, manufacture, design and geometry. The material lever targets an optimisation of concrete composition or the application of high-performance materials. In regard to manufacturing, on the other hand, the possibility to reduce concrete material consumption under prefabrication requirements (compare with [5]) can be considered. Probabilistic methods contribute to the design lever, e.g. pursuing a direct determination of design values for material properties on an individual project level. The geometry lever refers to mathematical structural optimisation amongst other approaches. The listed levers are frequently reappraised and discussed within the scientific community. This study consults topology optimisation for geometry finding and 3D concrete printing as a manufacturing method. Subsequently, these two consulted methods are briefly outlined.
2.1 Topology optimisation in concrete construction

In contrast to other structural optimisation methods such as size and shape optimisation, topology optimisation enables a completely free arrangement of geometrical entities within a design domain [6]. Thus, it is commonly applied in an early stage of the design process. The topic of topology optimisation within the field of concrete construction has recently gained in popularity, especially within the last two decades [7]. Several strategies are pursued to take the nonlinear material behaviour of concrete into account during the optimisation processes, e.g. including tensile and compressive stress constraints or considering concrete damage characteristics. Exemplary, the work of [8] can be mentioned, who investigated the optimisation of plain concrete structures via the application of density-based topology optimisation under Drucker-Prager stress constraints. Reference [7] found that the number of research papers dealing with topology optimised concrete structures is numerous, but experimental investigations are clearly limited. Hence, the intention of this study is to contribute to this by presenting the entire process from the idea of a topology optimised concrete structure over manufacturing to the conduction of experiments. Practical studies comparable to the one of [9], who tested a topology optimised reinforced concrete beam as well as an optimised hybrid concrete-steel truss loaded in a four-point bending test, are recommended to be further conducted in order to enable assessment of optimised design’s load-bearing behaviour.

2.2 3D concrete printing

Besides conventional casting, extrusion-based 3D concrete printing is chosen as the manufacturing method for selected girder specimens within this study. As casting is associated with time- as well as cost-consuming construction of formwork, 3D concrete printing is seen as a potentially more efficient manufacturing method to produce organic-shaped concrete structures. An important aspect regarding the load-bearing behaviour of 3D printed concrete is the dependency of the direction of load application on the print direction. Reference [10], e.g., characterised the mechanical performance of 3D printable concrete, assigning the interfaces between the printed filaments a significant role regarding the mechanical response. A field of exhaustive investigation within 3D concrete printing includes the integration of reinforcement [11]-[12], also mentioned as one key challenge by [13] to overcome limitations within structural engineering of digitally fabricated concrete. A focus on 3D concrete printing in the context of topology optimisation within concrete construction has been recognised lately, probably enhanced by the method’s high degree of design freedom. Reference [13] identified the development of appropriate numerical modelling approaches as an opportunity to achieve further improvements on the topology optimisation capabilities of digitally fabricated concrete. To give a practical example, [14] combined the disciplines of mathematical optimisation and extrusion-based 3D concrete printing in investigating a 3D printed, post-tensioned and topology optimised concrete girder. Their findings indicate material savings of approx. 20% in terms of midspan deflection, thus demonstrating the approach’s potential regarding weight optimisation of concrete structures.

3 Case study: Externally reinforced concrete girder

A case study of an externally reinforced, optimised concrete girder is presented in this paper, where the entire design and construction process is demonstrated. The design concept is initially presented and key findings regarding print path configuration are highlighted. The numerical as well as the experimental analyses are subsequently discussed.

3.1 Design concept

Two basic types of optimised girders, as illustrated in Fig. 2, were investigated by the authors. The static system is a simple single-span girder with a rectangular cross section reinforced along the lower edge of the beam. The external reinforcement is either a carbon laminate with a sand-coated surface or a steel laminate with a welded, internal bond ridge. Via the use of epoxy resin, the carbon laminate was glued into a so-called steel implant (compare with [15]). Due to its opening, the steel implant enables easy mounting and disassembly of the girder with a bolt. For further details about the functionality of the steel implants, the reader is referred to [16]. The total girder length between the support contacts amounts to approx. 2 m. Within the 200 mm height of the concrete body the depth of the external reinforcement is not considered. A width of 60 mm is set for all girder versions. For comparison reasons, the mentioned outer dimensions were chosen according to [15]. The rounded lower beam edges close
to the support areas result from an adaptation of the geometry to principal tensile stresses in the course of a previously conducted linear elastic finite element analysis. Investigations regarding the introduction of compression forces perpendicular to the structural edges via the use of implants in thin-walled concrete elements are described in [17]. This study is embedded into an evolution of development stages of beam versions according to the idea of mass-optimised concrete beams, as illustrated and headed by [4].

Fig. 2    Side views and cross sections of an optimised girder type with carbon laminate glued into steel implants (top) as well as of an optimised girder type with a steel laminate and a welded, internal bond ridge (bottom) with point load. The support areas are shown enlarged (centre).

The specimens were chosen to be either full or optimised, with the latter case depicted in Fig. 2. The topology optimised geometry was found using the software Altair Inspire™ under the assumption of linear elastic material behaviour. A point load was introduced 584 mm away from the right support contact, compare with Fig. 5. The objective of the topology optimisation was stiffness maximisation under a user-defined volume constraint in percent of the total design space volume. Altair Inspire™ uses the algorithm Altair OptiStruct™ in the background that is based on a mathematical topology optimisation method (Solid Isotropic Material with Penalization) [6]. No manufacturing constraints of any kind were directly considered within the optimisation process. In this context it also has to be pointed out that it was not the authors’ intention to go into detail regarding the topology optimisation process at this stage, but rather to investigate the basic feasibility of the entire process from design and manufacturing to experimental and numerical evaluation in order to sharpen the future research areas in this context. Organic shapes of the girder were consciously allowed for during the design process. The manufacturing was considered within the conceptual design process in the sense that external reinforcement was chosen to facilitate manufacturing via 3D concrete printing. Furthermore, the external reinforcement also functions as part of the outer formwork. The whole test series with varying investigated girder versions is given in Table 1. When comparing the concrete material consumption of the optimised with the full structural concrete designs, concrete volume savings of 33 % were achieved. However, a consideration of load-bearing behaviour is not included herein.
Table 1 Test series overview

<table>
<thead>
<tr>
<th>Girder versions</th>
<th>External laminate reinforcement</th>
<th>Concrete geometry</th>
<th>Manufacturing method</th>
<th>Number</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ct3d</td>
<td>Carbon (sand-coated)</td>
<td>Optimised</td>
<td>3D concrete printing</td>
<td>1</td>
</tr>
<tr>
<td>Ct3dc</td>
<td>Carbon (sand-coated)</td>
<td>Optimised</td>
<td>3D concrete printing of the contours and subsequent casting (“lost formwork”)</td>
<td>failed before testing</td>
</tr>
<tr>
<td>Ctc</td>
<td>Carbon (sand-coated)</td>
<td>Optimised</td>
<td>Casting</td>
<td>1</td>
</tr>
<tr>
<td>Cf3d</td>
<td>Carbon (sand-coated)</td>
<td>Full</td>
<td>3D concrete printing</td>
<td>1</td>
</tr>
<tr>
<td>St3d</td>
<td>Steel with welded bond ridge</td>
<td>Optimised</td>
<td>3D concrete printing</td>
<td>1</td>
</tr>
<tr>
<td>Cfc</td>
<td>Carbon (sand-coated)</td>
<td>Full</td>
<td>Casting</td>
<td>1</td>
</tr>
<tr>
<td>Sfc</td>
<td>Steel with welded bond ridge</td>
<td>Full</td>
<td>Casting</td>
<td>1</td>
</tr>
</tbody>
</table>

3.2 Print path configuration

The manufacturing methods of the varying investigated girder versions are listed in Table 1. Subsequently, findings regarding manufacturing of the girder versions via extrusion-based 3D concrete printing are highlighted.

The 3D printing system BauMinator® by [18] was consulted as the manufacturing method of choice besides casting. The print paths for the presented optimised girder geometry as outlined in Fig. 2 were developed manually and empirically evaluated in an iterative manner. Due to the set print and process parameters, minimal geometry adaptions from the initial “raw” optimised layout were necessary. A print path width of 17 mm, a print path height of 7.5 mm, print filament overlapping of 0.5 mm and a printing speed of 300 mm/s were predetermined by the printing system. More than eight test layouts with varying process parameters and print layouts were produced in advance. The final print layout is illustrated in Fig. 3. In total, eight layers were printed on top of each other. The following prerequisites were derived from the findings of the test prints and subsequently preconditioned for the final print layout [19]:

- Reduction of start, deposition and stop printing sequences to result in a continuous print process. As far as necessary, they are preferably positioned in non-critical girder areas. Start sequences are characterised by unwanted local material deficit and deposition as well as stop sequences by unwanted surplus material.
- Print speed reduction to 80% of its initial value in elongated print areas to avoid cavities. As an illustrative example, the importance of print speed adaption is illustrated in Fig. 4 (left).
- Increase of overlapping in load-critical or infill beam areas close to the supports to avoid cavities and reduce the effect of vibrations of the print system resulting from frequent direction changes of the printing nozzle or sharp inversions of directions.
- Layered inversion of print layers on top of each other (z-direction) to avoid cold joints between girder webs and flanges.
- Avoidance of print discontinuities in the load introduction area and along the lower flange (load-critical areas).
- Distance minimisation of the print nozzle to the external reinforcement. Small distances show a significant influence on the resulting print and bond quality.
Fig. 3  Final print path: Side view of the top layer with contours of the carbon-reinforced beam type in black (top) and isometric view of the entire print path layout (bottom).

Fig. 4  Significance of print speed reduction in elongated girder areas (left) and fracture pattern of the lower flange of girder version Ct3dc after curing (right). (Credits: Nadine Stoiber, University of Natural Resources and Life Sciences, Vienna)

The girder version Ct3dc already failed before testing. The crack propagation, as depicted in Fig. 4 (right), indicates crack growth as a result of uneven shrinkage. The filling concrete was added directly after the lost formwork of the inner cavities was printed. Due to the addition of accelerator and a comparably higher cement share of the printed in comparison to the cast concrete, uneven shrinkage is assumed to lead to such illustrated fracture patterns. One could counter this issue in adding the filling concrete after total curing of printed concrete parts.

3.3 Experimental and numerical investigations

3.3.1 Materials

As print material the mortar product PrintCret 230 [20] (maximum grain size of 2 mm) from Baumit was used, which is exclusively developed for the BauMinator® 3D printing system. Just before the print material is extruded, an accelerator of 4.5-5.5 Vol.-% is added to the mixture. As filling material, the dry concrete product GießBeton [21] by Baumit is consulted, corresponding to a concrete quality of C25/30 according to the product data sheet. The results of performed hardened concrete tests are shown in Table 2.

Table 2  Mean values of concrete material properties: Results of hardened concrete tests on prisms (160x40x40) according to ÖNORM EN 1015-11 [22] with a concrete age of 40d

<table>
<thead>
<tr>
<th></th>
<th>Print material:</th>
<th>Casting concrete:</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flexural tensile strength [MPa]</td>
<td>7.9-11.2*</td>
<td>7.3</td>
</tr>
<tr>
<td>Compressive strength [MPa]</td>
<td>33-47*</td>
<td>37</td>
</tr>
</tbody>
</table>

*dependent on the orientation of print and loading direction, full specimens were tested as well
Regarding the external reinforcement materials, the steel laminate’s yield and tensile strength were determined in the course of tensile tests. The Young’s Moduli of both materials as given by the manufacturers were verified via the use of strain gauges. The respective material properties are listed in Table 3.

Table 3  Mean values of reinforcement material properties and respective thickness

<table>
<thead>
<tr>
<th></th>
<th>Carbon laminate: Sika® Carbodur® M 614 [23]</th>
<th>Steel laminate with welded bond ridge</th>
</tr>
</thead>
<tbody>
<tr>
<td>Young’s Modulus [MPa]</td>
<td>210,000</td>
<td>210,000</td>
</tr>
<tr>
<td>Tensile/yield strength [MPa]</td>
<td>3,500/-</td>
<td>510/355</td>
</tr>
<tr>
<td>Thickness [mm]</td>
<td>1.4</td>
<td>3</td>
</tr>
</tbody>
</table>

### 3.3.2 Experimental test setup

The bending tests were performed in a servo-hydraulic testing machine. The setup is illustrated in Fig. 5. The displacement \( w \) was measured directly at the position of load introduction by the test machine via the integrated LVDT (inductive displacement transducer) and the loads via the integrated load cell. A digital image correlation was used to track the displacements and strains in the load introduction area with an image area width of approx. 800 mm. The tests were performed in a displacement-controlled manner (speed 0.8 mm/min).

![Fig. 5  Test setup of the shear force tests as adopted from [17]. Girder version Ctc is displayed.](image)

### 3.3.3 Numerical settings

A comprehensive recalculation of the experimental findings was conducted using the nonlinear finite element software ATENA Engineering 2D, with the basic numerical settings being briefly outlined. The default material model SBETA by the software ATENA Engineering 2D with fixed crack modelling is consulted for the printed and cast concrete materials, with targeted values being adapted according to Table 2 and Table 3. Plane, bilinear quadrilateral elements are predominantly used for the concrete body to avoid effects comparable to aggregate interlock, which are accompanied by triangular-shaped elements and are commonly associated with an unrealistically stiff behaviour of the analysed structure. The finite elements have a size of 20 mm corresponding to 10 elements over the height of the girder. The solution method of choice is the Newton-Raphson method (tangent, stiffness update at each iteration) provided by the software. The iteration number limit is extended, thus being able to achieve satisfying convergence characteristics. Body force is considered and a prescribed deformation of 0.1 or 0.2 mm/analysis step for some optimised girder versions, respectively, is applied. The external reinforcement is modelled as a conventional reinforcement bar with an equivalent perimeter as well as an equivalent bar area. The reaction forces are directly measured at the point of load introduction.
3.3.4 Results

The experimental and numerical results are illustrated in Fig. 6 and successively discussed.

First, the experimental findings are outlined: A brittle failure mechanism of the carbon-reinforced, full girder versions (Fig. 6: Cfc – top right, Cf3d – centre right) can be identified, with the crack propagation during the experiments indicating compression zone failure of the concrete body close to the load introduction area. Comparing the experimental results of Cfc (Fig. 6: top right) and Cf3d (Fig. 6: centre right), a more ductile behaviour of the printed girder version is visible. The carbon-reinforced, optimised girder versions (Fig. 6: Ctc – centre left, Ct3d – bottom left) are characterised by comparably lower maximum loads and visible load drops during testing. A decisive factor regarding the failure mechanism of the carbon-reinforced girder specimens was a complete absence of shear reinforcement as well as the specific bond behaviour of the contact surface between the external carbon reinforcement and the concrete body. The experiments showed a detachment of the sand-coated laminate from the concrete body. Further information on the bond behaviour of sand-coated carbon rods are given in [24]. The recognised sudden load losses or snapbacks of the experimental curve could be explained by the rather brittle tensile failure mechanism of tension strut rupture and consequently brittle failure of the upper concrete compressive zone, respectively, which might have led to sudden movements of the displacement transducers. Shifting from the carbon to the steel-reinforced specimens, the experimental load-displacement diagram of the full, cast girder version with steel reinforcement (Fig. 6: Sfc – top left) is characterised by a significant yield plateau. A well distributed crack growth along the girder length was visible during the tests. The visible cracks that occurred just before the final load drop at the end of the yield plateau indicated a bending shear failure mechanism. The steel-reinforced optimised girder version (Fig. 6: St3d – bottom right), however, did not show yielding as the full cast pendant Sfc (Fig. 6: top left), which can be explained by the concrete structure’s optimised geometry being the limiting factor regarding load-bearing behaviour and the capacity of the reinforcement not being fully utilised.

After the conduction of the full-scale experiments, the experimental load-displacement diagrams were numerically recalculated set a basis for a numerical prediction of future optimisation results. Comprehensive parameter studies were conducted, including the variation of targeted concrete material properties, mesh size, solution methods and bond-slip models. Material properties previously determined in the course of material tests (compare with Table 2 and 3) were not altered. The results revealed that the concrete compressive strength as well as the compressive ductility $\mu$ showed a distinctive influence on the load-bearing behaviour of the varying girder versions. The latter parameter $\mu$ describes the post-peak plastic compressive displacement in the course of a uniaxial compression test, or in other words the displacement from maximum compressive strength to reaching zero stresses again. To give an example, the compressive ductility was significantly reduced for printed in comparison to cast concrete, indicating a more ductile behaviour of printed concrete. A further decisive factor within the numerical analysis was modelling of the bond behaviour, significantly challenged by the number of occurrence concrete to reinforcement surface constellations (cast concrete + steel, cast concrete + carbon, printed concrete + steel, printed concrete + carbon). Default bond-slip models provided by ATENA Engineering 2D were consulted and adopted according to the experimental load-displacement curves in an iterative manner. The bond-slip model CEP-FIP Model Code 1990 with a cold drawn wire as reinforcement type and a good bond quality, for example, served as a basis for modelling the bond behaviour of Sfc (Fig 6: top left). The bond-slip relationship between the sand-coated carbon laminate and the cast concrete body of girder specimen Ctc (Fig. 6: centre left) was directly adopted from the previously calculated girder version Cfc (Fig. 6: top right), allowing for an attribution of a certain degree of robustness to the respective bond-slip model. The less steep experimental load-displacement curve of St3d (Fig. 6: bottom right) compared to the numerical calculations can be associated with irregular and overall weaker bond characteristics due to the manufacturing method of 3D printing, which impeded a precise modelling of bond behaviour close to reality. In order to significantly improve the presented numerical results, a conduction of bond-slip experiments would be necessary. Another aspect to be mentioned in the context of the numerical analysis is the consideration of shrinkage effects via a reduction of tensile strength [25]: In order to achieve such low failure loads of the 3D printed, optimised girder versions (Fig. 6: Ct3d – bottom left, St3d – bottom right) in the numerical analysis matching those of the experiments, the tensile strength had to be reduced in comparison to the ones chosen for the analysis of the full pendant Cf3d (Fig. 6: centre right).
Comparing all investigated girder versions, following conclusions can be drawn: The topology optimised girder versions are characterised by failure at lower maximum loads than the full pendants. However, the failure mechanism, tension strut rupture, of the optimised girders was also identified in the numerical analysis. Therefore, the concordance of the numerical with the experimental results amongst all girder versions outlines the general practical feasibility of optimised concrete structures, even though a material reduction while maintaining the same load-bearing behaviour was not achieved in this specific case. The latter fact is due to the simplified topology optimisation process, which was not an intended matter of detailed investigation within this study. Due to the large number of varying parameters, a direct comparison of the girder version results is only reasonable to a limited extent.

4 Conclusion

The aim of this study was to conduct the entire process from topology optimisation, layout design, manufacturing, experimental testing and numerical analysis of optimised as well as full concrete girder specimens with external carbon and steel reinforcement. The feasibility of the approach is demonstrated. One important finding of this study that came to light is the importance of multidisciplinary cooperation. The interaction between different disciplines such as mathematical optimisation, structural and mechanical engineering requires flexibility.
The goal of the authors is to elaborate a clearly defined optimisation strategy for concrete structures. In a first step, varying mathematical topology optimisation approaches are applied to a well-defined girder problem and the respective design results are evaluated in regard of manufacturability and concrete material savings while maintaining similar load-bearing behaviour. The selection of the optimisation algorithms is based on a previously conducted comprehensive systematic review on the topic of topology optimisation in concrete construction. Numerical analysis of the girder versions is intended to accompany the entire design process to allow for prediction and better assessment of the load-bearing potential of the found optimised geometries. The steel-reinforced girder versions, as investigated in the presented study, show more favourable results regarding failure announcement and maximum carrying capacity while simultaneously being easier to produce and numerically recalculate. The limited bending radius of the carbon laminate significantly impeded manufacturing. Furthermore, numerical modelling of the carbon reinforcement’s bond behaviour is not trivial [26]. The general advantage of using external reinforcement is that a relatively straightforward manufacturing of 3D printed specimens is enabled. Regarding concrete material, a possible optimisation strategy could be the pursuit of ultra-high performance concrete, as it is characterised by a significant liner-elastic behaviour under compression until failure, therefore allowing for a facilitation of the mathematical optimisation process.

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Structural design possibilities of reinforced concrete beams using eggshell

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Inspiration of interlocking wooden puzzles in precast buildings concrete construction

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Abstract

This paper presents the use of puzzle shape geometries in precast systems for reducing the probability of chain-collapse by providing desirable interlocking, eliminating the necessity of supports, and accelerating the construction process. This new approach benefits from wooden puzzle geometries and enhances the system's performance by increasing the Degree of Static Indeterminacy (DSI), which likewise improves safety in the construction process. The proposed approach is described through an example and evaluated by limited standard calculations and finite element analyses. The use of post-tensioning improves the section's performance and clamps the system better. Hence, to evaluate the possibility of utilising the post-tensioning in this geometry, the sections' capacity is entirely evaluated and explained. In the example, a well-known wooden puzzle with branched-geometry is selected to join a column to two beams. After geometrical improvement and analysis of the puzzle-connection, a two-floor building with eight connections and linear elements was modelled in Rhino. To evaluate the assembling process and manufacturing possibility, a small-scale model of the joint is experimentally manufactured. The connections performance and the applied ideas for facing this system's limitations are discussed. Despite the limitations regarding the developing manufacturing techniques (e.g. robotic CNC), the proposed interlocking approach has the potentials to be used in the building industry.

Keywords: Interlocking, dry connections, post-tensioning, precast, concrete.

1 Introduction

Coincide with the increasing world's population, recent advances in manufacturing technology offer opportunities for increased prefabrication use in more extensive and complex precast projects. Besides higher construction quality and assembling speed, the most prominent feature of prefabricated elements is their integration into modern structural approaches [1] and manufacturing technologies [2]. Other precast structures' achievements are improved productivity, economic predictability in terms of life cycle costs [3], energy performance and environmental effects [4]. Thus, the precast systems are growing much faster than on-site construction sectors and should be developed by several up to date studies [5]. The swift construction mainly needs the omitting of the scaffolding and the time for pouring and curing concrete during the assembly process in the connecting palaces.

Despite the advantages, some challenges in designing and constructing the precast systems exist, mainly related to maintaining the structures' integrity, especially during construction and under extreme loads [6]. Due to the modular and precast systems' nature, a lack of suitable integrity may lead to a chain collapse. Hereof, connections play a crucial role in maintaining integrity in the construction time, the structures' general stability and robustness [7]. The connections should ensure the structure's stability in both vertical and lateral directions [6].

The typical connections generally have three significant drawbacks: 1- Accidental separation that occurs without caution in severe loading conditions [7], 2- Stress concentration may lead to local or overall collapse [8]. 3- Lack of resistance against all degrees of freedom, e.g. the bending moment. These connections are not mainly designed for carrying bending loads at the ends of the elements and need further concrete filling in the connecting re-bar part. One of the most common connections is the Corbel connection which, despite the robust resistance against shear load, does not have any bending resistance without additional parts, causing a higher bending in the middle of the beam. This joint also
does not have any parts to stop the beam element from falling. In other words, lack of suitable interlocking and little unwanted movement may cause the beam to fall. This issue has caused several chain-collapses during construction time [9]. To increase the competitiveness of the precast systems, respecting modern potentials, such as robotic manufacturing techniques, the construction industry should focus on developing novel details and innovative systems with higher interlocking [10].

Researchers have investigated different approaches, such as innovative approaches for optimising locations of the precast elements [11] and modular integrating systems [13], to improve the integrity and self-interlocking mechanism. Additionally, efforts for developing dry structural concrete connections by inspiring from' dry-wooden connections and using high-performance concrete (UHPC) were made [15]. Based on a test of a crossing-beam-connection with a cut-out in the middle of each beam (to fit beams together), the precast beams can perform similar to two monolithic crossing beams. In this crossing joint, each beam's cut-out parts filled with the second beam and act as a tensile-compressive couple, with the tensile re-bar of the first beam section [17].

Another crucial issue that displays its effect more in the complex concrete geometries are the tolerances appearing due to several usages of the formworks, deformation caused by transportation and changes in the material (e.g. shrinkage). The tolerance problem is a significant obstacle for assembling and fitting the elements or forming gaps between the elements [7], which with new manufacturing techniques can be minimised.

To face these issues and use the precast system's mentioned advantages, appropriate interlocking systems should be proposed. In addition to being adaptable to nowadays manufacturing methods, this system by higher DSI should provide alternative load paths to stop the sudden collapse in case of local damages [18]. The system should also ensure uniform deformation and integrity in the entire system [7]. In contrast to routine analysis, integrity analysis can be problematic [19]. The integrity can be evaluated by applying lateral loads and measurement of the deformation of the building. Alternatively, supports can be gradually removed, and the structure's performance is analysed [20]. Otherwise, the degree of statistical indeterminacy as an indicator can be calculated [18].

2 Discussion:

As stated, up to now, no precast system with high geometrical interlocking was practically proposed. Nonetheless, these systems have meaningful advantages, such as 1- raising the Degree of Static Indeterminacy (DSI), 2- no need for scaffolding and 3- stopping the chain collapses. Hence, this study discusses the possibility of using a puzzle-shape dry concrete connection toward increasing the interlocking in the entire building system.

The proposed system should be developed theoretically and experimentally. The development should regard: Theoretical: developing innovative-appropriate interlocking systems (geometries) and related analysis. Experimental: facing difficulties in producing the precise (low tolerances) concrete elements and, likewise, the adversity of fitting the interlocking elements to each other. In following, through an example, the possibility of utilising the proposed interlocking system is briefly evaluated, regarding Theoretical (numerical) (2.1) and Experimental (physical) aspects (2.2).

2.1 Proposing Interlocking Joints and Theoretical Evaluation

2.1.1 Puzzle connections geometries:

The new precast structures are inspired by different wooden connections and puzzle shapes. For instance, as the most well-known puzzle, the jigsaws concept can be used for wall and floor connections. In this study, the interlocking precast frames are the subject. Hence, a few possible puzzle geometries that can form the branched shape connections for joining the crossing beams and column are discussed. Fig.1 shows that these 3D puzzles can form geometries similar to the beam-column joint, using one to four different concrete specimens. Each puzzle has a unique assembling-scenario (solution), which may change if used as a structural element. The assembling-scenarios contribute to interlocking (by increasing DSI).

In terms of the production possibilities, the grinding technology, especially utilising robots, enables the engineers to manufacture complex non-prismatic 3D geometries of concrete connections with micrometre tolerances.

In the investigation of these geometries, the following questions should be answered: I- How the cut-out parts reduce each element's structural capacity? II- How can the weaker directions of the connections regarding the load application be considered? and III- How can this puzzle-geometry can be
The mesh has to be unified in the next step, and the tiling order has to be computed as Fig. 6B-C. The first tile represents a mesh quad-face, and the triangular tiles are used to create the boundary faces. For the mesh generation, the neighbouring edges are directed to have an opposite orientation as Fig. 6B. This condition is obtained by traversing mesh faces and assessing whether edges are opposite to the neighbouring peer. Graph traversing algorithms such as Breadth-First-Search (BFS) is used to obtain the tiles' sequence. Such an algorithm contains a sequence that indicates the adjacency of the tiles. The BFS algorithm, as Fig. 6C, is used to traverse mesh faces because it explores all the neighbour nodes at the existing depth before exploring the nodes at the next depth level [3]. Then, the 2D tiles, as Fig. 6D, are assigned to the mesh faces according to the BFS sequence algorithm. This is done by checking whether it is possible to choose one of the nine tiles and the number of times a tile has to be rotated. The mapped tiles are coloured by an index (0-9) as Fig 6E. Furthermore, the associated edges are coloured to form triangles (-1: grey, 0:red, 1: pink, 2: yellow, 3: orange) as Fig. 6F. This tiling method produces the Nexorade pattern obtained by duplicating, rotating, and mapping the tile-set as Fig. 6G.

Fig. 6 Tiling method for a planar diamond mesh grid: (A) a diamond mesh, (B) a polygonal mesh with the unified edge winding, (C) Breadth-First-Search, (D) 2D tiles, (E) Tiling, (F) Tiles with coloured edges that show the edge indexing, (G) pattern generated from tiling.

Rotation of the tiles and their appropriate mapping are determined through the adjacency rule as Fig 7C. The rule has a notation of the current-edge-index and the next-edge-index. The indexing follows 0,1,2,3 for the existing tile' face edges and 3,2,1,0 for the next tile. These number pairs say that the quad must be connected, i.e., by edge 0 to the other tiles by edge 3, then 1-3, 2-1, 3-0. The edges that form the boundary are indexed as -1 and empty. The current tile and its rotation in the meshing scheme at each are shown in Fig 7. Fig. 7B illustrates a tile connected with the existing tile following the 2-1 rule (yellow-pink triangles), where index 2 belongs to the tile shown in Fig. 7A. Sequentially, the tile is placed following index pair 3-0 as Fig 7C. Furthermore, it is possible to match several edges as Fig 7D-E, while traversing through the BDS sequence order. To match these sequences, the algorithm compares strings and is determined as true for the tile string (3,2,1,0), if the current mesh faces are indexed as (0,1,2,x) or (x,x,x,3), or (x,x,3,0) or (x,x,x,x). If the tile has a match, then it is selected, and the number of rotations is revised to shift the array of tile-edge indices. The iteration stops when all mesh faces are checked or none of the tiles is chosen. The overall aim of applying such a procedure is to use this methodology for multiple patches where the target geometry is more complex than one rectangular surface.
**B. Beam:** The original B. beam's section of the puzzle has a relatively low width (b) to height (h) ratio. Fig. 2. Likewise, the moment of inertia (I=b.h^3/12) is lower, and the cut-out reduces the capacity of this section. Hence, for parts of the beam that is not in the joint zone, the rectangular-sections shape was changed to a T-shape. Besides, similar to Corbel connections, some edges added to the columns. Fig. 2 (C.column) for supporting this beam, as cantilever supports. It compensates for the lack of full beam-section shear capacity, while the shear can be transmitted to the column through these robust supports, where the section is yet full. The section also can be scaled up, like a typical section.

**C. Column:** Fig. 2 displays the column in the joint zone, which loses half of the section area. This cut-out reduces the area of the interaction diagrams (force-bending) and also can cause force eccentricity. The A. beam fills the cut-out part of the column. In the case of no-tolerances and appropriate load directions, it can act as the compressive part to semi-compensate the cut-out and face the eccentricity.

After selecting the final connection shape and the relation between the sectional dimensions, in the next step, the standard calculations of the reinforced concrete are applied, and the possibility of using the post-tensioning technique is controlled.

### 2.1.3 Application of post-tensioning technique:

A wide range of calculations and parameter analyses should be conducted to use the post-tensioning method's advantages and evaluate the reinforced elements' capacity. Nonetheless, since the target is just proposing an interlocking concept, by respecting the mentioned dimensions (Fig. 2) and ACI-318 requirements, a brief calculation's results are stated below:

To check the possibility of using the post-tension technique, the sections and all post-tensioning standard criteria were regarded, and all details calculated. In these calculations, shear and bending loads, as well as deformation, were evaluated. For instance, post-tensioning controls in the T-shape section (fc: 450 kg.cm2 and φ12@150mm stirrups), with 20 strands (Area: 51.61mm2) and Grade 270, while the pump power and slip are 125ton and 2mm etc., entirely displays acceptable performance in the B. beam for a 10m span under self-weight. Hence, this technique can be used in the proposed connections to use the following advantages.

In the interlocking connection, post-tensioning has multiple advantages: 1- improves the capacity of the elements, 2- applies desirable permanent loads in the parts, such as adjustable axial loads for improving the bending-performance of the columns. 3- minimises the probable gaps by pushing the elements into each other and causing better fitting. 4- applies desirable eccentricity to the system and elements, and 5- maintains the connection. Post-tensioning loads and strands, which pass from the joint's centre, bring more integrity to the entire frame system and increase the system's interlocking degree. Based on the controlling calculations, post-tensioning is applicable and can have considerable benefits. Post-tensioning instruments (e.g. strands) can be applied step-wise during the assembling process.

### 2.1.4 Evaluation of reinforced sections:

To evaluate the joint performance, three different sections of each beam and column were selected. Based on standard reinforced concrete calculations, the elements were analysed and compared.

**A. Beam:** This beam in the centre of the joint-zone loses half of its section (Fig.2), but the re-bars can be bent to continuously pass in the remained half of the section. In the full section, the bending capacity is 165.9 kN.m (Fig. 2 (A.1)), and in the reduced section, the capacity is 65.3 kN.m (Fig. 2 (A.2)), which is less than half. Likewise, the shear force was also reduced from 149 Kn to 72Kn. Nonetheless, in a perfect fitting, the B. beam acts as the compressive part of the A. beam. Hence A. beam performs as a beam without compressive re-bar while the height is similar to the full-section. This section's bending capacity is 157.2 kN.m, which is 0.05% less than the full beam. Its shear capacity is, yet, half of the original beam (74kN). That is why the Corbel supports were added to the beams to transfer the shear force to the column, where it has the full-section area.

**B. Beam:** The second beam was also evaluated in the three steps. In the full section (Fig. 2 (B.1)), the bending moment and shear capacities are 116.2 kN.m and 103.1kN (Fig. 2, B.2). While in the section with a rectangular middle cut, the bending moment and shear capacities reduced to 98.9 kN.m and 73kN. The cut-out part reduces the beam's bending and shear capacity by around 17.5% and 30%, respectively. This section was continued by the T-shape section, which has a higher bending capacity and applies the bending by pressing the column's sides. It also transfers the shear force on the Corbel...
supports. The cut-out part is also filled with a part of the column and partially acts as the beam's compressive part, to likewise improve the section's capacity.

**C. column:** Fig. 2, illustrates the interaction diagrams (Axial-Bending) of the full column (Fig. 2 (C.1)) and column with half of the section (Fig. 2 (C.3)). Theoretically, the column is the most challenging elements of this connection due to considerable reduction of the section's capacity and asymmetric performance even in one direction. To compare the full section with the cut-section, the models were analysed in SAP2000 (Section Designer). The interaction diagrams compare the bending capacity in the section's weaker direction with cut-out (Fig. 2 (C.1)) to the full section in the more robust direction (Fig. 2 (C.1)). It shows that the cut-section acts as 30% of the full-section in this column (before final assembling). Nonetheless, suppose the column section's correct direction in the design process is considered (Fig. 2 (C.2)) and filled with the beam. This section, under bending, acts as 71% of the full-section (after final assembling). All sections under pure axial load perform almost similar. The 30% reduction in the column can, by adding the post-tensioning method, be semi compensated. To improve the performance, the proposed geometry also can be scaled up freely from each dimension in a non-prismatic form, which is another advantage of these joints; for instance, the column can be enlarged (e.g. 20×40 to 30×40cm).

![I. II. III.](image)

**Fig. 3** Von Mises Stress in the connection (I), merged connection (II) and a typical joint

### 2.1.5 Evaluation of the model by numerical simulations

Up to this section, the joint performance, by standard calculation and evaluation of each element, was theoretically discussed. Nonetheless, even if the elements can be produced and assembled precisely, different items should also be evaluated; for example, the tangential behaviour between the fitted part may cause stress concentration, especially in the edges. The numerical simulation of these models enables a brief comparison. The results of numerical simulations were displayed in Fig. 3 by the Von Mises stresses, in the I) proposed geometry connections, II) in merged geometry of the proposed geometry (monolithic) and compared with a III) typical connections with the puzzle shape.

The deformation of the connections under the equal loads compared as the stiffness indicator. Results showed that the proposed connection (I) generally has around 65% of the typical joint's stiffness (III). This high reduction ratio (35%) comes from a lack of post-tensioning load and generally low friction (cohesiveness) between the touching surfaces (Fig. 3). The stiffness ratio is calculated under the low forces and in a linear range. However, after a downward movement and micro-cracks, the rebar gets activated, and precast elements touch each other entirely. Then stiffness in them increases to 80%. The merged connection (Fig. 3(II)) displays a 33% higher stiffness than the typical geometry (III), while its maximum stress is almost 60% less than in the monolithic elements (III). If instead of a connection, the stiffness in the individually filled-A. beam and filled-B. beam are compared to similar monolithic beams, the A. beam has 90%, and the B. beam (T-shape) has 85% of the stiffness of their similar monolithic beams.
Fig. 4  Assembling process for solving the puzzle and assembling the two-floor building

A. Studies of small produced joint and assembling of a two-floor 3D model

2.2.1 Construction and fitting small scale model

Calculations show the appropriate performance of the interlocking joints in the small buildings with low numbers of floors. The previous discussion was developed while the element's geometry was assumed entirely perfect, with an easy assembling process. Now the possibility of production and fitting should be discussed.

The connection as a puzzle needs to be assembled and fitted step-wise, as shown in Fig. 4. First, the column should be located, and then the opening part of B.beam should be fitted from the top of the column and move down vertically (Fig. 4 (b)). Then A.beam should be fitted from the end of B.beam and be move horizontally to the joint zone and finally centralising all parts. For post-tensioning purpose, the strands can easily pass through the predicted holes to be pulled by jacks.

To evaluate the possibility of production and fitting the connection, A prototype of the connection constructed on a small scale with regular concrete. This model was not manufactured for the loading-tests and, only a few longitudinal re-bar (φ 4mm) were used due to limited space (Fig .5). For the production of the specimens, six plastic formworks were printed. Each formwork has two pieces that connect to each other after lubrication and filled with concrete. After concrete curing and opening the formworks, the specimens have sufficient surface quality and did not need any post-processing (e.g. sanding), due to pretty accurate geometry. As shown in Fig. 5, these models were assembled similar to the graphical Rhino models (Fig. 4). The B.beam, without difficulties, almost under its weight, was fitted. Nonetheless, the fitting of A.beam needed more push, which in the construction size would be an issue. Fitting time took all in all 20S, and no big gap between the elements was visible. For further studies, ultra-high performance concrete regarding higher properties and the ability to form more precise shapes is preferred.

Fig. 5  Experimental small-scale model of the connection made by printed formwork in three parts

2.2.2 Assembling process of a small two-floor building

The assembling scenario of the full-frame is similar to the joint, just the number of beams are doubled. After locating the four columns, two A.beams should be fitted to B.beam on the ground, and then the whole four beams should be fitted from the top of the columns and move vertically down to reach the floor connection. Finally, B.beams should be centralised (Fig. 6). Besides, additive beams, in case of necessity, can be easily added to the system [17], in the case of two-direction load distribution and a higher interlocking (DSI). The Corbel supports should be eliminated on the top floors to let the B.beams of lower floors move down. It makes the connection weaker. This issue can be solved by adding more interlocking connections and segmenting the column. Fig .6 illustrates how these two-floor building can be assembled entirely. It can be predicted that due to the high friction and tolerances, the movement of the beams to their final location is the most challenging task. Also, the elements' deformation may bring unexpected difficulties to this assembling process due to shrinkage and transportation.

Alternatively, to face the issues, two leading solutions can be selected. 1- Utilising specific steel elements in the connecting surfaces with a slight slope and 2- Applying the grinding techniques (CNC machines) to produce precise connections, after a specific time, when effects of deformations caused by the shrinkages and creeps are ignorable. The new technology (e.g. robotic CNC and digital scanner) can practically manufacture these complexes geometries with minimum efforts and tolerances.
2.2.3 Compare interlocking degree to the typical precast system

Increasing the Degree of Static Indeterminacy (DSI) was one of the primary reasons for this study. Hence, the proposed assembled frame was compared to a selected typical precast system. The selected system, using the Corbel cantilever supports, provides robust vertical support without rotational stiffness. Figure 7 shows these two systems next to each other.

In the connection of the selected geometry, the joint resists two degrees of freedom (Fig. 7 (right)), while the proposed interlocking joint connection has resistance against 12 degrees of freedom. It means Static indeterminacy, in this interlocked small building, is 80 degrees higher. The mentioned DSI is not the exact calculation and just indicates the differences during the construction process. The differences decrease after adding the floor elements or probable connecting details. Nonetheless, it should be regarded that DSI in the interlocking method is considerably higher in all steps. Most chain collapses were experienced during the construction process, proving the importance of developing interlocking systems. A collapse in this precast-prestressed system against the typical precast systems needs a progressive failure in different elements and rupture in strands. Unlike the other systems, it does not collapse due to a sudden movement (e.g., chain collapse [9]) and performs more similarly to monolithic buildings. In addition to remaining detachable, the proposed system has more flexibility in plan than other systems, e.g., constructing a cantilever beam that needs a fixed connection.

In the end, it can be mentioned that the non-technical issues are also obstacles for developing and using such a technique. The issue can be highlighted as the construction industry's unwillingness to use innovative approaches. Clearly, every new idea demands a wide range of studies and a new school of thought for development and finding its position in industrial usage.

Fig. 7 Comparison of the proposed interlocking system and a typical precast method

3 Concluding remarks

Utilising more interlocking geometries in the precast building systems can bring a wide range of advantages to this industry, including a higher degree of static indeterminacy, uniform deformation, and less probability of chain collapses. The current study tries to evaluate the possibility of using this
innovative approach. Both advantages and disadvantages of the interlocking systems were discussed and proven an example based on one puzzle shaped connection that was structurally retrofitted and utilised as well as related to a simple building. Based on the results, it can be concluded that despite the drawbacks, this system has the potentials to be practically used in a limited range of buildings. Some results can be highlighted as:

- The cut-out parts in the joint can reduce the section's capacity, e.g., 40% of monolithic elements' capacity.
- The capacity reduction in the columns can be more effective.
- The direction of the forces and asymmetric performances of the interlocking element can considerably reduce this system's drawbacks.
- The utilisation of the post-tensioning technique makes this system's performance more similar to monolithic elements.
- The geometry can be locally scaled up (e.g. non-prismatic sections) to compensate the drawbacks, such as the section's dimension.
- The interlocking elements' geometries can be manufactured with high accuracy by robotic post-processing tools or innovative frameworks (e.g., wax or plastic printed formworks).
- The assembly process demands a unique step-wise fitting scenario.

References

Robotically-fabricated nexorades from whole timber

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Link to the video: https://youtu.be/DF23wTLPRh0

Abstract
This paper introduces the conceptual design of sustainable timber structures using raw wood. It presents a workflow for systematically generating Nexorades using a tiling method for multi-valence quad subdivided surfaces. The system is designed based on an integrated framework combining resources from local forestry, knowledge from geometry processing, joinery for raw wood, laser scanning and digital fabrication, and structural engineering. Particular focus is given to Cross joint geometry and tool-path generation. Also, the global structural performance of the system is assessed. The results indicate that the proposed design methodology can offer an efficient and sustainable construction technique. Furthermore, the methodology is reflected in a robotically fabricated prototype, demonstrating that the design framework can seamlessly integrate digital tools and construction.

1 Introduction and Prior Study of Nexorades for Raw Wood
Nexorades belongs to a class of self-supporting structures where elements rest on top of each other as Fig. 2, or are interconnected side-to-end, as Fig. 3. One of the key reasons for building these structures is a two-element connection instead of a multi-valence node, as Fig. 1. The Nexorade connection varies from external fasteners to wood-wood connections with additional fabrication effort to adapt to irregular raw woods. The past research-tested panel braced raw wood as Fig. 4. This study focuses on a two-layer system and open-sourcing, an in-depth study of conceptual Nexorades form-shaping methods [5]. In a larger research scope, the raw wood is used due to: a) overstocking of small-radius trees in Swiss forests (Rossiniere), b) added value for decarbonization, c) small radius trees are not considered as a construction material by local saw-mills, d) potential for local-circular economies using local timber for non-standard production, and e) increasing business interest due to low-cost robotic automation. Every raw beam contains certain surface irregularities and natural characteristics, such as knots and non-circular sections. The natural characteristics require subtractive machining operations to unify raw timbers at a connection zone using rectilinear geometries observed in Cross joints. The proposed method follows a rule of not exceeding two-thirds of a beam section [1] to have a sufficient contact zone. Lastly, structural performance is evaluated using a reliable force-flow mechanism. The methodology has four main parts: a) tiling geometry generation, b) local joinery generation, c) structural analysis, d) laser-scanning and robotic fabrication. Accordingly, the system is designed based on an integrated framework combining resources from local forestry, Computer-Aided Design (CAD) for geometry manipulation, joinery of round woods, scanning, digital fabrication, and Computer-Aided Engineering (CAE) and numerical simulations.
Fig. 2 Mathematical models for simple, platonic objects can be used to define edge rotation angles.

Fig. 3 Translation methods A-C is extended over the past work to an even number of polygons D.

Fig. 4 Fabrication of the panel braced Nexorades is tested using raw wood and translation method.

2 Linear Pattern Generation on a Quad-Mesh

The proposed tiling method aims to obtain a principle pattern representing the rotational Nexorade system using a finite set of tiles [5] as Fig. 5 F. The tiles’ set consists of nine tiles drawn as a polygon unit measuring 1x1x1 containing line segments. These line segments represent a part of the Nexorade beam where each side of the polygon is numbered. Colours are used to visualize the indexing to track adjacency between the edges as Fig. 5 A. The sides of the polygon are indexed to identify the rule-set as Fig. 5 B-C. This process improves the visual reflection of the tiles' edges, where each triangle colour represents an index. The rule-set as Fig. 5 C (i.e. [0,1,2,3] – [3,0,1,2]) is then used to match neighbouring tiles. The structure of the tiling is computed using the mesh face-edge graph when matching tile edges. The matching is detailed in the following paragraph. Finally, the tiling workflow is applied to a particular quad-dominant mesh topology as Fig. 5 E composed of mesh patches as Fig. 5 D.

Fig. 5 Tiling notation. A) sides of a Tile are indexed, B) sides are coloured for visualization purpose, C) each side has an associated bar element, D) Mesh topology composed of multiple patches, E) Barycentric mapping, F) linear pattern of the mapped tiles.
The mesh has to be unified in the next step, and the tiling order has to be computed as Fig. 6B-C. The first tile represents a mesh quad-face, and the triangular tiles are used to create the boundary faces. For the mesh generation, the neighbouring edges are directed to have an opposite orientation as Fig. 6B. This condition is obtained by traversing mesh faces and assessing whether edges are opposite to the neighbouring peer. Graph traversing algorithms such as Breadth-First-Search (BFS) is used to obtain the tiles' sequence. Such an algorithm contain a sequence that indicates the adjacency of the tiles. The BFS algorithm, as Fig. 6C, is used to traverse mesh faces because it explores all the neighbour nodes at the existing depth before exploring the nodes at the next dept level [3]. Then, the 2D tiles, as Fig. 6D, are assigned to the mesh faces according to the BFS sequence algorithm. This is done by checking whether it is possible to choose one of the nine tiles and the number of times a tile has to be rotated. The mapped tiles are coloured by an index (0-9) as Fig 6E. Furthermore, the associated edges are coloured to form triangles (-1:grey, 0:red, 1:pink, 2:yellow, 3:orange) as Fig. 6F. This tiling method produces the Nexorade pattern obtained by duplicating, rotating, and mapping the tile-set as Fig. 6G.

![Fig. 6 Tiling method for a planar diamond mesh grid: (A) a diamond mesh, (B) a polygonal mesh with the unified edge winding, (C) Breadth-First-Search, (D) 2D tiles, (E) Tiling, (F) Tiles with coloured edges that show the edge indexing, (G) pattern generated from tiling.](image)

Rotation of the tiles and their appropriate mapping are determined through the adjacency rule as Fig 7C. The rule has a notation of the current-edge-index and the next-edge-index. The indexing follows 0,1,2,3 for the existing tile’ face edges and 3,2,1,0 for the next tile. These number pairs say that the quad must be connected, i.e., by edge 0 to the other tiles by edge 3, then 1-3, 2-1, 3-0. The edges that form the boundary are indexed as -1 and empty. The current tile and its rotation in the meshing scheme at each are shown in Fig 7. Fig. 7B illustrates a tile connected with the existing tile following the 2-1 rule (yellow-pink triangles), where index 2 belongs to the tile shown in Fig 7A. Sequentially, the tile is placed following index pair 3-0 as Fig 7C. Furthermore, it is possible to match several edges as Fig 7D-E, while traversing through the BDS sequence order. To match these sequences, the algorithm compares strings and is determined as true for the tile string (3,2,1,0), if the current mesh faces are indexed as (0,1,2,x) or (x,x,x,3), or (x,x,3,0) or (x,x,x,x). If the tile has a match, then it is selected, and the number of rotations is revised to shift the array of tile-edge indices. The iteration stops when all mesh faces are checked or none of the tiles is chosen. The overall aim of applying such a procedure is to use this methodology for multiple patches where the target geometry is more complex than one rectangular surface.
Fig. 7 Tiling based on the Breadth-First-Search: A) first tile, B) the second tile and a common edge, C) third tile and a common edge, D) sixth tile, E-F) boundary tiles.

So far, the tiles are disconnected from each other. However, they contain adjacency information to reconstruct the beam's geometry. Consequently, the tiles are connected, forming a Beam data structure containing ordered line segments, connectivity information, and axes' planes. Tiles are transformed into individual beams by converting the tile's sides into an undirected graph. Subsequently, the connected component method is applied to identify the interconnecting beams. Thus, the connected components are simplified as lines, where the corresponding orientation depends on the sum of the mesh face's normal vectors of the tiles. Next, the axis perpendicular to the plane is determined for each beam. Finally, the line axis is trimmed between the ends of two neighbouring planes. The tiling sequence and the resultant volumetric geometry are tested to correspond within a set of meshes within multiple singularities, including open and closed geometries, as Fig 8.

Fig. 8 A1) Visualized Breadth-First-Search and A2) geometry output. Three selected geometries are chosen for further structural analysis: B) planar surface, C) cylindrical shape, and D) a three-valence reciprocal.

3 Dynamic relaxation

Connection zones between beams in Nexorades depends on a mesh curvature, the thickness of elements, rotation of a tile, and the Eccentricity between beams. If the distance between a pair of elements is too large, then the area to connect the two is not sufficient enough. Consequently, either the mesh topology has to be changed, or the distance between bar elements must be reduced. The eccentricities' minimization goal is used, employing a constraint-based solver [4]. In the current study, the dynamic relaxation reduces the maximum deviation between beams, as Fig 9. Beam ends are moved towards each other within the closest vector between the two beam axes. Beams can slide within the neighbour line segments while altering the pair-wise rotation. Thus, the input geometry has to be relatively close to a relaxed one. Tessellated geometries with a changing curvature mostly have different eccentricities unless the input surface has a uniform curvature, for example, a sphere or a plane shape. Therefore, dynamic relaxation aims to reduce or equalize the interval of eccentricities to a given limit, i.e., less than
one-third of a given raw-wood diameter. Consequently, the joinery generation adapts to each joint scale depending on a mesh curvature, as Fig. 9 bottom.

![Image](image1.png)

Fig. 9 Top – dynamic-relaxation of central axes to minimize and equalize eccentricities. Bottom – joinery generation at the lowest eccentricities and the connections cut-outs from closed triangle mesh using CGAL mesh boolean difference method.

4 Cross-lap Joint Robotic Fabrication

Two main algorithms can be used in the conceptual tiling method to obtain the Nexorades pattern: a) cross-lap cuts when joints intersect within less than two-thirds of the timber section and b) side-end connection. The current research explores the first option of a cross-lap joint as Fig 12. The digital joinery generation follows timber joinery existing in regular rectangular structures as Fig 11, and it applies this methodology to round sections as Fig. 10A1-A2.

![Image](image2.png)

Fig. 10 Cross-lap joint geometry generation: A1-A2) differences between rectangular and circular section, B) changes by angle, C) fasteners, D) conical cuts and its side extensions (D1-D2).
Raw timbers are not generally straight or regular in shape and section, but they stay relatively constant within the connection zone. Also, the definition of joints relies on fabrication tools and their movement relative to a timber piece. The milling tool is limited to its flat cylindrical movement, whereas the saw blade is constrained within the flat surface area resulting in rectilinear cuts. Due to timber irregularities and requirements to have as much contact area as possible, a conical timber joint is proposed as Fig. 10A2 (right). It requires nine cuts comparing to its predecessor that only employs one cut as Fig. 10A2 (left). It is also possible to decrease the number of cuts to six using an extended conical cut, as Fig. 10A2 (middle). The joint is generated using a Joinery Solver that can adapt to various design geometry angles as Fig. 10B and add necessary fasteners to interlock the cross-lap joint as Fig. 10C. The joint can be modified further by controlling side cuts parameters to control from the least contact area as Fig. 10D to the largest possible as Fig. 10 D1-D2. Various wood-wood connections are cut to understand the tool-path generation process concerning limited robotic arm movement, as Fig 13. The cutting process requires scanning because timber shape and location vary, and at least a few centimetres of misalignment can cause issues during the assembly. Consequently, the fabrication workflows require the following steps: a) scanning and point-cloud processing to obtain timber central axis as Fig. 13A b) tool-path that is generated together with the joint geometry as Fig. 13B, c) automatic tool-changer to switch between sawblade and milling as Fig. 13C and d) continuous scanning and fabrication integration after timber is mounted on the rig as Fig. 13D.

Fig. 11 A) single layer Nexorade, B) Seiwa Bunraku Theater replica in raw-wood, C) two-layer Nexorade.

Fig. 12 Raw-wood cross-lap joints vary from a single dowel, surface flattening to conical cuts.
Given the complex geometry of the Nexorades and variations in the mechanical characteristics of round woods, numerical approximations are preferred to closed-form solutions. Using the Finite Element Method (FEM), the structural design and analysis of the spatial Nexorades structures are performed. The OpenSees library [8] is employed as the backend solver for structural analysis and design feedback loop. The geometric information of the structure shown in Fig. 11A is transformed from the Rhinoceros 3D CAD environment. Each round wood element is simulated using 1D linear elastic beam elements with orthotropic wood material properties. A uniaxial elastic material associated with C24 timber grade (Table 1) is assigned to the elements. For each joint type, a connector element is defined. The connectors are modelled using a two-node link element. The element consists of six strings, which represent the translational and rotational behaviour of the joint area. The numerical models are analyzed by being subjected to the loads specified by the Eurocode standard. A combination of dead, live, wind, and snow loads is considered, leading to a uniform distributed load of 2814 N/m². Once the analysis is complete, the results are post-processed by COMPAS and stored back into the original 3D CAD object.

Table 1. Material properties of C24 timber used for round woods.

<table>
<thead>
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<th>Description</th>
<th>Unit</th>
<th>Value</th>
<th>Comment</th>
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<tr>
<td>Bending strength</td>
<td>MPa</td>
<td>24.0</td>
<td>$f_{m,k}$</td>
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<tr>
<td>Tension strength</td>
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<td>$f_{t,0,k}$</td>
</tr>
<tr>
<td>Compression strength</td>
<td>MPa</td>
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<td>$f_{c,0,k}$</td>
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<td>MPa</td>
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<td>Modulus of elasticity</td>
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<td>$E_{m,0,mean}$</td>
</tr>
<tr>
<td>Shear modulus</td>
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<tr>
<td>Density</td>
<td>Kg/m³</td>
<td>350.0</td>
<td>$\rho_k$</td>
</tr>
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</table>

The model is analyzed in 1.26 sec, and the output data is extracted from the backend in 0.119 sec. Furthermore, the data post-processing time is less than 0.005 sec which shows the computational framework's efficiency. The results indicate that the joints' rotational stiffness at the Ultimate Limit State (ULS) has a minimal contribution to the overall load-carrying system. The vertical displacement and Mises stress of the structural elements at the ULS are shown in Fig. 14a-b, respectively. The structure undergoes the ULS deformation of 234 mm at its mid-span, and the maximum Mises stress produced in the elements is 0.32 MPa. This stress state indicates that the Nexorades system with either quad or diamond grid pattern demonstrates a reliable force-flow mechanism. This is mainly related to the multiple numbers of joints within such systems and their ability to distribute the structure's forces. Moreover, given that the system under consideration has low curvature, the stress concentration in particular regions is not considerably high. Furthermore, the deformed shape obtained from the numerical FE models is 37 mm, satisfying the Serviceability Limit State (SLS) since the deflection-based criteria (the span length/300) are approximately 46 mm.
6 Conclusions

A new methodology to design bio-based spatial timber structures from raw timber is proposed. A series of geometrical steps are employed to obtain a Nexorades pattern, including tiling, dynamic-relaxation, joinery solver, structural calculation, and digital fabrication. The framework enables discrete tile mappings in non-manifold meshes to obtain a Nexorades pattern while reconstructing the beam-like geometry following the graph methods. The beams' volumetric representation included changing radii along the beam axis, and taper assignment based on curvature and elevation is also considered. Meshes with multiple singularities and variation of curvature with optimized Eccentricity are also validated. The geometry generation is open-sourced using NGon tool-set [5]. Experiments have already been conducted using laser-scanner and point-cloud processing to create a raw-wood library and alignment of beams in the robotic workspace. The geometry is also processed in a FE package, COMPAS_FEA. Furthermore, the contact zones of each high-resolution joint have to be studied in further detail. It is recommended that multiple layers of tree trunks or additional fasteners have to be used at the connection points to improve such systems' structural performance due to the non-linear rotational assembly sequence.

References

The ice shell project

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Link to the video: https://youtu.be/i-tktQe6GLs
The conceptual design of structures is at the heart of the design process and when the most fundamental and influential decisions are taken for a project. It merges experience, intuition, tradition, site constraints, technical solutions and, above all, the genius and sensitivity of the designers.

The International *fib* Symposium on the Conceptual Design of Structures 2021 generates a fruitful exchange event for academics and practitioners from engineering, architecture and other disciplines on the topic of the conceptual design of structures. The focus is placed on experiences made particularly during the design process. The discussions reflect how a project emerges, how design decisions are taken, how responsibilities are distributed, how obstacles and constraints are handled, how fundamental design principles are applied and the way the individual members of the design team collaborate.

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