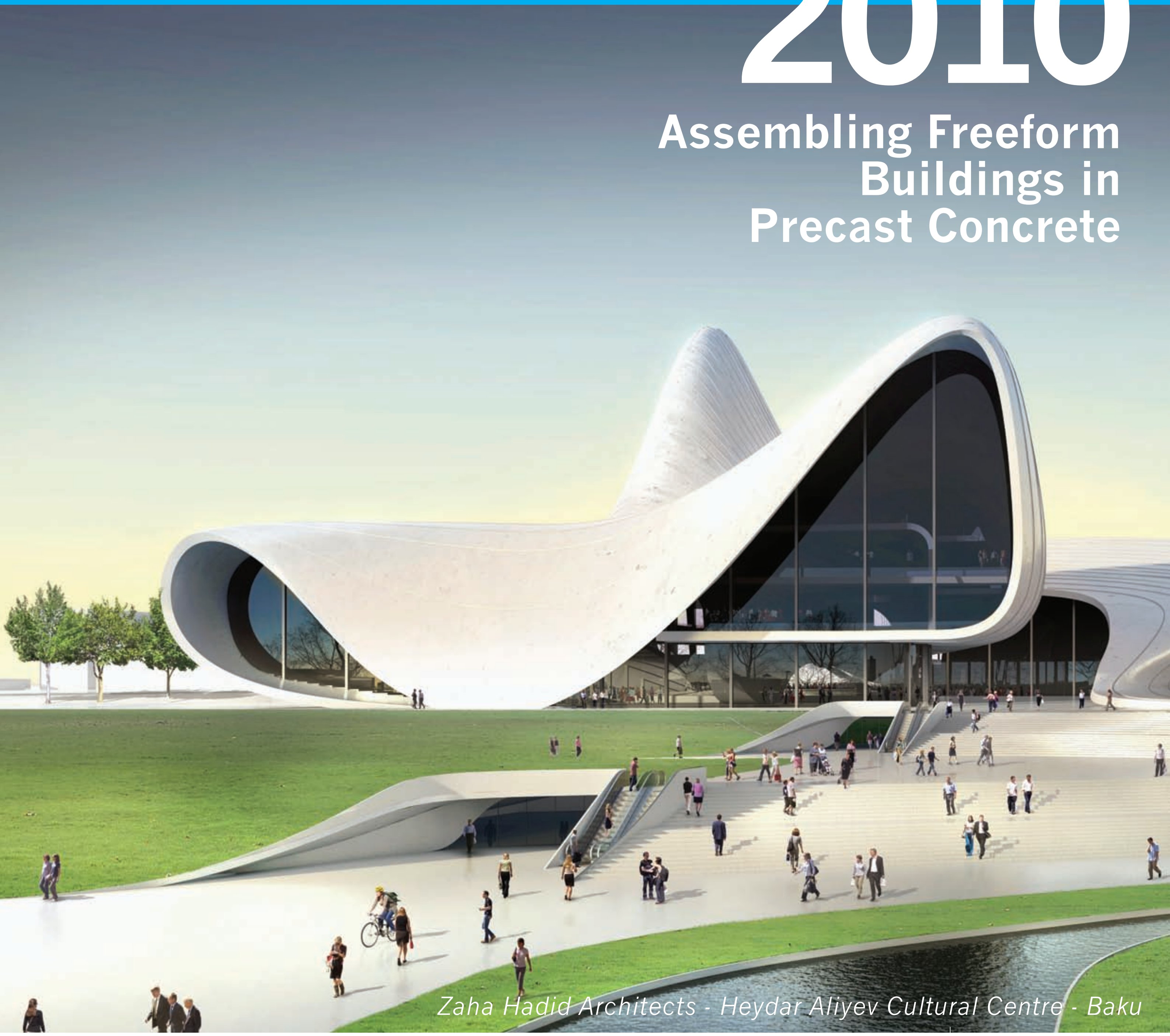
PRECAST 2010

Assembling Freeform Buildings in Precast Concrete



Zaha Hadid Architects - Heydar Aliyev Cultural Centre - Baku

Reader Symposium
TU Delft
15 juni 2010

TU Delft
Delft University of Technology

AB-FAB
ABSOLUTELY PREFABULOUS

This publication was made at the occasion of Precast2010, a Symposium held at June 15, 2010 in Delft. This symposium was organized by the Faculty of Civil Engineering and Geosciences, Delft University of Technology (Chair of Building Engineering) in cooperation with AB-FAB (Associatie van Beton-Fabrikanten van constructieve elementen).

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Editors
prof. dipl-ing. Jan Vamberský j.n.j.a.vambersky@tudelft.nl
Roel Schipper, MSc h.r.schipper@tudelft.nl

Layout cover page
Robert Schipper

Building on cover page
Heydar Aliyev Cultural Centre, Baku, Azerbaijan (under construction 2010)
Zaha Hadid Architects (London)

PRECAST**2010**

Het Nieuwe Bouwen in Prefab Beton
Assembling Freeform Buildings in Precast Concrete

Architecture – Highrise Buildings - Industrialisation

edited by
Jan Vamberský and Roel Schipper

AB-FAB Associatie van Beton-Fabrikanten van constructieve elementen. Het doel van AB-FAB is om marktpartijen te helpen bij het succesvol toepassen van prefab betonnen bouwelementen. Om optimaal te kunnen profiteren van de voordelen die deze bouwoplossingen bieden is van belang dat ze op een juiste wijze en in goede onderlinge samenhang worden gebruikt
Dat vereist kennis. Dit symposium draagt hieraan bij.

PROGRAMMA, 15 JUNI 2010

09:00-09:30 ontvangst

09:30-09:45 opening door
ing. Lambert Teunissen (voorzitter AB-FAB) en
prof. Jan Vambersky (gastheer namens TU)

09:45-10:30 ***keynote speach 1****:*
Freeform architecture assembled on site
Mr. Saffet Bekiroglu, Zaha Hadid architects

10:30-11:15 ***keynote speach 2****:*
Toekomstvisie op prefab beton door twee recent
bij prof. Jan Vamberky afgestudeerde ingenieurs
ir. Diederik Veenendaal (Witteveen+Bos/ETH Zurich
ir. Koos Tolsma (Ingenieursstudio DCK)

11:45-12:30 Precast in Ultra High Performance Concrete
prof. Joost Walraven, TU Delft

14:00-16:45 ***Parallelsessie 1 - architectuur***
- Jacco van Dijk (Hurks Beton)
- prof. Rudy Uytenhaak (Uytenhaak Architectenbureau)
- Roel Schipper (TU Delft)

14:00-16:45 ***Parallelsessie 2 - hoogbouw***
- prof. Björn Engström (TU Göteborg, Zweden)
- Rob Huijben (Hurks Delphi Engineering)
- Dick van Keulen (Ingenieursstudio DCK / TU Delft)

14:00-16:45 ***Parallelsessie 3 - industrialisatie***
- Fred Reurings (Bouwcombinatie Erasmus MC)
- prof. Dick Hordijk (Adviesbureau Hageman / TU/e)
- prof. Hennes de Ridder (TU Delft)

16:45-17:00 plenaire afsluiting

Voorwoord
Ing. L.H.W. Teunissen

LEAN BOUWEN, DENK PREFAB

De huidige financiële crisis dwingt bedrijven in de bouw ertoe om anders naar hun processen te kijken. Dit omdat veel bedrijven in financieringsproblemen terecht zijn gekomen. Onderzoek van ING heeft uitgewezen dat in de afgelopen decennia de bouw er niet in is geslaagd de productiviteitsverbetering te realiseren die industrieel Europa wel heeft gerealiseerd.

Ook met de veiligheid binnen de bouw is het droevig gesteld. De bouwplaats behoort tot de meest risicovolle arbeidsplekken met slechte werkomstandigheden. Dus ook op dit vlak hebben we de voortgang van andere industrieën niet bij kunnen houden. Dit resulteert er dan in dat jongeren Europa-wide de bouw gaan mijden met als gevolg een zeer lage instroom. Door de hoge gemiddelde leeftijd van werknemers zal de komende 5-10 jaar een geweldige uitstroom plaatsvinden. Vakmensen gaan weg en kennis van het bouwen gaat verloren, met als logisch gevolg een nog lagere productiviteit.

Indien wordt gekeken naar doorlooptijden (waarde van alle goederen in de bouwfase gedeeld door de omzet maal het aantal werkdagen per jaar) dan is er geen enkele industrie waar zo veel geld ‘nutteloos vastzit in de modder’ en dus niet rendeert. Toyota heeft zich met zijn Lean-aanpak gefocust op deze doorlooptijd en heeft deze inmiddels gehalveerd. Gevolg was dat er een geweldige hoeveelheid werkkapitaal vrijkwam die werd aangewend voor groei. Waar Lean niet voor was bedoeld, maar juist ook in resulteerde, was de toename van de efficiency. Men ontdekte dat slechts 20% van de arbeid zit in het echte voortbrengingsproces. De geheime

Preface
Ing. L.H.W. Teunissen

LEAN BUILDING, THINK PRECAST

The current financial crisis is forcing companies in the building industry to have a different at their processes. This is because many companies have ended in financing problems. ING Research has shown that in recent decades the building indsutry has not been able to achieve the same productivity gains that have been realized in industrial Europe.

Furthermore, the safety and reliability level of the building industry shows a sad picture. The building site is among the most dangerous work places with poor working conditions. In this area we definitely did not keep up with the progress in other fields of industry. The result is that young people Europe-wide avoid the construction industry, resulting in very low inflows. The high average age of employees over the next 5-10 years will result in a tremendous outflow. Professionals will retire and knowledge of the building is lost, with as the corollary a lower productivity.

When looking at throughput (value of all goods in the construction phase divided by sales times the number of working days per year) there is no industry where so much money is “useless and stuck in the mud" and therefore is not profitable. Toyota, with its Lean approach, has focused on this throughput, resulting in double speed. Result was that a great amount of working capital was released and could be used for growth. Although not intended with the Lean approach, this also resulted in increased efficiency. It was found that only 20% of work is in the real production process. The secret key to the success of Toyota is in the fact that together with their partner-suppliers they work on continuous improvement. If the sleutel van het succes van Toyota zit in het gegeven dat men samen met hun partner-toeleveranciers aan de verbeteringen werkt. Indien de bouw eenzelfde focus zou aanbrengen, zou dit voldoende kapitaal genereren om alle financiering af te kunnen lossen. De grote bouwbedrijven maken zich gelukkig inmiddels op voor het implementeren van deze benadering in hun organisatie.

Indien u de indruk mocht krijgen dat ik hel en verdoemenis predik voor de bouw, dan is niets minder waar. Want…. de oplossing voor de problemen is er: PREFABRICAGE.

Aan de steeds hogere eisen van comfort, duurzaamheid en esthetica kan juist met prefabricage worden voldaan. Ontwerpen in prefab is een absolute vereiste om juist de voordelen van prefabricage ten volle uit te nutten (Design to Costs).

Kortom, de bouwsnelheid zal enorm toenemen, inefficiency en faalkosten zullen sterk afnemen en leiden tot een gezonde bedrijfstak, waar het goed werken is.

Dus denk erom:
LEAN BOUWEN? DENK PREFAB!

construction industry would have the same focus, that would generate sufficient capital to finance all the repaying of debths. Fortunately, the big construction companies now are implementing this approach in their organization.

If you would get the impression that I might preach hell and damnation for the construction industry, then nothing is further from the truth. Because .... the solution to the problem is: prefabrication.

The increasing demands of comfort, durability and aesthetics can be satisfied with prefabrication. Design in precast concrete is a prerequisite to fully benefit of the principles described above (design to cost).

In short, the construction speed will increase dramatically, inefficiency and failure will decrease significantly and lead to a healthy industry, where working becomes a pleasure again.

So remember:
BUILDING LEAN? THINK PRECAST!

Lambert Teunissen
Voorzitter AB-FAB
Algemeen Directeur VBI / Spanbeton

Voorwoord
Prof. dipl-ing. J.N.J.A. Vamberský

Prefabricage is een uitstekende ontwikkeling die een briljante toekomst tegemoet gaat! Dit geldt voor alle vormen van vooraf vervaardigen van bouwproducten die daardoor op de bouwplaats alleen nog geassembleerd hoeven te worden. Dit in tegenstelling tot bouwproducten die in zijn geheel op de bouwplaats worden gemaakt. Het laatste, het in zijn geheel op de bouwplaats vervaardigen - is de traditionele wijze van bouwen - die wij nog steeds op de bouwplaats tegenkomen, maar die naar mijn stelligste overtuiging steeds verder naar de achtergrond zal worden teruggedrongen. Deze ontwikkeling is al enige decennia aan de gang. Er worden diverse producten uit diverse materialen en met diverse technologieën vooraf vervaardigd, om vervolgens op de bouwplaats te worden gemonteerd. Bouwelementen uit staal, hout, beton, kunststoffen en combinaties daarvan, voor hoofddraagconstructies, funderingen, gevels, afbouwconstructies, dakconstructies, maar ook bouwelementen van sanitair, elektra, liften, telecommunicatieonderdelen, hulpconstructies ten behoeve van de uitvoering. Allemaal worden zij vandaag de dag in toenemende mate vooraf vervaardigd, en zoals gezegd: dit zal in de toekomst alleen maar meer worden.

Er zijn diverse redenen aan te wijzen waarom deze ontwikkeling een dergelijke vlucht neemt en in de toekomst alleen nog verder zal nemen. Al in de eerste symposia van het "*Delft Precast Concrete Institute*" - instituut dat eind jaren tachtig door collega Prof. Dr. Ir. Walraven en de ondergetekende is opgericht - werd op meerdere van deze redenen duidelijk gewezen, andere kunnen pas vandaag worden benoemd Deze redenen zijn onder andere als volgt:

Preface
Prof. dipl-Ing. J.N.J.A. Vamberský

Prefabrication is an excellent development that has a brilliant future! This applies to all forms of prior manufacturing of products that on the site only need to be assembled. This in contrast to products that are made entirely on site. The latter, on site production, is the traditional way of building which we still often encounter, but which I firmly believe will be pushed more and more into the background. This development has been underway for some decades. There are already various products in different materials and technologies that are pre-made and then installed on site. Building products from steel, wood, concrete, plastic and combinations, main load supporting structures, foundations, façades, finishing constructions, roof structures, but also components of plumbing, electricity, elevators, telecommunications components and auxiliary structures for construction. They all are increasingly pre-manufactured nowadays, and as said this will only be more in future.

Several reasons can be pointed out why this development flies high and will continue to do so in future. Already during the first of the symposia organized by the *"Precast Concrete Institute Delft"* which in the late eighties was founded by colleague Prof. Dr. Ir. Walraven and the undersigned - several of these reasons were clearly pointed out, other reasons may only be appointed today.

These reasons include the following:

- *De toenemende welvaart* met als resultaat stijgende eisen aan de prestaties en uitrusting van gebouwen (ICT-voorzieningen, communicatievoorzieningen, nieuwe materialen en technologieën, …). De voor deze uitrusting en prestaties vereiste kwaliteit kan niet meer op de bouwplaats op traditionele wijze gerealiseerd worden. Vaak is dit alleen nog in de beschermde en schone omgeving van een werkplaats mogelijk.

- *De stijgende eisen ten aanzien van de kwaliteit en de uitstraling van het eindproduct* leiden er toe, dat diverse gebouwonderdelen die in het zicht blijven, dan wel de vormgeving van het gebouw bepalen, de vereiste kwaliteit alleen kunnen behalen wanneer zij in de beschermde omgeving van de werkplaats worden gemaakt.

- *Het 3D Syndroom van de bouw*. 3D staat hier voor "*Dirty, Dangerous* en *Difficult*". Er zijn steeds minder mensen die vanwege dit syndroom op de bouwplaats willen werken – en er worden terecht hogere eisen aan de arbeidsomstandigheden gesteld. Het resultaat is opnieuw verplaatsing van de arbeid van de bouwplaats naar de meer beschermde omgeving van een werkplaats. Er wordt steeds meer buiten de bouwplaats geprefabriceerd en op de bouwplaats slechts geassembleerd in plaats van op de bouwplaats gemaakt.

- *Increasing prosperity*, resulting in increasing demands on the performance of buildings and equipment (ICT facilities, communications equipment, new materials and technologies, ...). For such equipment and performance the required quality can no longer be realized traditionally on site. Often this is only possible in the protected and clean environment of a workplace.

- *Increasing requirements for quality and appearance of the finished result* result in the fact that for various building components that remain in sight, or that determine the design of the building the required quality can only be achieved if they are made in the protected area of a workshop.

- *3D Building Syndrome*. 3D stands for "Dirty, Dangerous and Difficult". Because of this syndrome there are fewer and fewer people that like to work on the site - and higher demands are set on working conditions. The result is again movement of labor from the construction site to the more protected environment of a workshop. There is a growing trend to prefabricate off site and assemble on site.

- *De stijgende kosten van arbeid*. Met de toenemende welvaart stijgen ook de kosten van arbeid. Het zijn niet meer de kosten van materialen, maar de kosten van arbeid die winst en risico van de aannemer bepalen. Bij niet goed aan elkaar aansluitende stromen van activiteiten van bijvoorbeeld een ploeg metselaars of betonwerkers, zal de aannemer deze vaklieden– als zij op zijn loonlijst staan – toch moeten doorbetalen en zullen, bij hoge lonen, ook zijn verliezen hoog zijn. De huidige trend bij de aannemers is om deze primaire werkzaamheden uit te besteden in plaats van zelf te doen. Veel van deze uitbestede werkzaamheden vinden dan ook plaats in de fabrieken en werkplaatsen om het proces van vervaardigen en de kosten en kwaliteit hiervan beter te kunnen beheersen.

- *De blijvende ambities van de ontwerpers om de natuurwetten te ontkennen en de zwaartekracht te negeren.* Deze ambities zijn essentieel voor nieuwe ontwikkelingen, vooruitgang en voor de uitstraling van de gerealiseerde bouwwerken, maar moeten dan ook liefst eenvoudig maakbaar zijn. Vooraf fabriceren en op de bouwplaats - zonder kostbare tijdelijke ondersteuningen - in elkaar zetten is meestal de oplossing.

Afnemende belangstelling van jonge mensen om de traditionele bouwvakken te leren leidt er toe, dat men meer prefabriceert. In de werkplaats / productiefabriek kan men immers met relatief kleiner aantal geschoolde werknemers, meer en met beter kwaliteit produceren, dan op de bouwplaats in weer en wind.

- *The rising cost of labor*. With increasing prosperity, the costs of labor rise. It's not the cost of materials, but the cost of labor that determine the risk for and profit of the contractor. It is difficult to interconnect streams of activities on site such as teams of masons, concrete workers, etc. The contractor, if these professionals are on his payroll – still must continue to pay and will, with high wages, also have high losses if they cannot be set in efficiently. The current trend for contractors is to outsource these primary activities. Much of the outsourced work is than done in factories and workshops, in order to be able to better control the process of manufacturing, the cost and quality.

- *The lasting ambitions of the designers to the laws of nature to deny and ignore gravity.* These ambitions are essential for new developments, progress and the appearance of the completed buildings, but must also preferably be easily feasible. Manufacturing in advance and assembling on site - without expensive temporary supports - is the solution.

- *Decreasing interest of young people to learn the traditional building trade* leads to more pre-manufacturing. In the workshop facility it is possible, although with relatively fewer skilled workers, to produce more and with better quality than on site.

Prefabricage van beton, het maken van betonproducten onder de geconditioneerde omstandigheden in de prefab betonfabriek, zorgt voor een onlosmakelijk deel van deze ontwikkeling. De antwoorden die deze technologie biedt voor de nieuwe eisen van hedendaagse ontwerpers en bouwers, is het hoofdonderwerp van deze dag.

Prefabrication of concrete, making concrete under controlled conditions in the precast concrete factory is an inseparable part of this development. The response of concrete technology to the new demands of today's designers and builders, is the main topic of the day.

Jan Vamberský
emeritus-hoogleraar
Gebouwen TU Delft

Delft University of Technology

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Assembling Freeform Buildings in Precast Concrete

Heydar Aliyev Cultural Center

by Zaha Hadid Architects

Saffet Kaya Bekiroglu, Project Architect
Zaha Hadid Architects, 10 Bowling Green Lane, London EC1R 0BQ
saffet.bekiroglu@zaha-hadid.com

Figure 1. Exterior View of Main Entrance

I. Introduction

As part of the disbanded Soviet Union, the urbanism and architecture of Baku, the capital of Azerbaijan, on the western coast of Caspian Sea, has a strong Soviet influence. Since declaring independence from Soviet Union in 1991, Azerbaijan has invested heavily in modernizing and developing Baku's infrastructure and architecture. Zaha Hadid Architects were appointed as design architects of the Heydar Aliyev Cultural Centre following a competition entry in 2007. The cultural centre, designed to be the primary focal building for the nation's cultural programmes, breaks from the existing rigid soviet monumentalist architecture that is so prevalent in Baku, and reflects the sensual nature of the Azeri culture whilst at the same time expressing a strength and optimism that looks into the future. In a project of such magnitude, the approach to the building becomes an important factor in the design process. The

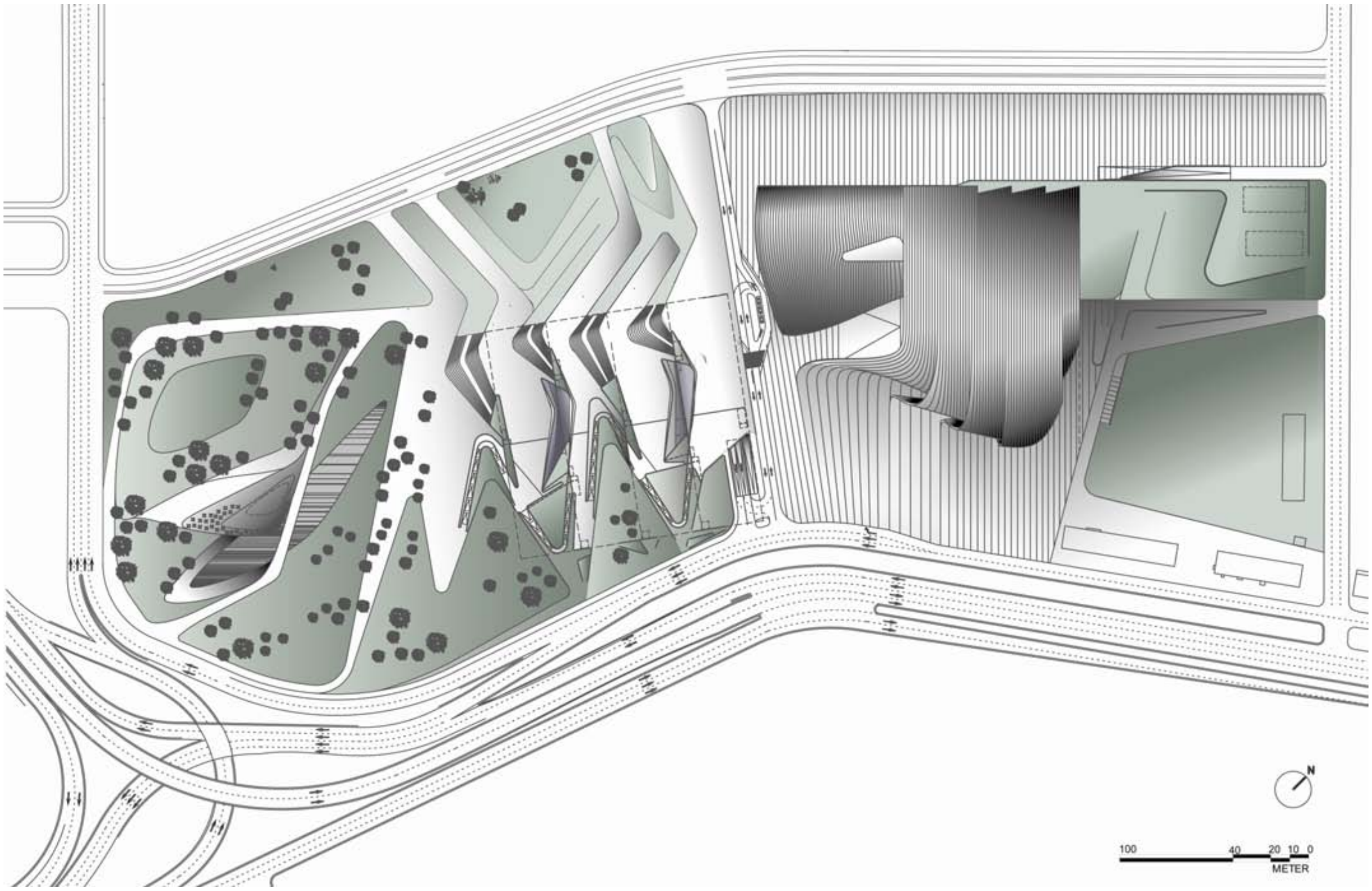

Figure 3. Site Plan

project is located on the main boulevard from the old city centre to the international airport, which is one of Baku's primary urban thoroughfares; and not being surrounded with immediate historical context, allowed greater degree of freedom for its architecture to be expressive.

II. Concept

The design of the Heydar Aliyev Cultural Centre establishes a continuous fluid relationship between the external plaza and the centre's interior, where the public are drawn into the building in a single, seamless gesture: blurring the differentiation between architecture and urban landscape, figure and ground, interior and exterior, private and public. A series of undulations, bifurcations, folds and inflections modify the artificial landscape of the plaza to create a surface that performs a multitude of functions; welcoming, embracing and directing all visitors throughout the different levels of the interior (Figure 3).

The primary access to the building is located on the main boulevard. This entrance road bifurcates, with one road leading to the main entrance, and the other to the underground parking. The existing site initially had a 20 meter topographical shear drop that split the site into two. The main building is situated to the northern, higher end of the site, whilst the parking, landscape, pond and exterior café are to the south. Connecting these two levels (platforms) to read and perform as a continuous fluid surface was one of the design's biggest challenges (Figure 4 on the next page).

By introducing a terraced hard landscape that is connected with alternative routes such as ramps, stairs and escalators, ZHA created both pedestrian and wheelchair access offering the smoothest transition between the two separate levels. The volume below this transitional zone is used for underground parking for 1500 vehicles, as security requirements prevented the car park from being located underneath such a high profile building. Consequently, additional excavation and landfill to level the site was avoided, resulting in increased cost efficiencies and converting an initial disadvantage of the site into an opportunity. An underground link tunnel connects the parking structure to the cultural complex, where escalators and elevators take visitors up to the ground level main entrance.

III. Program

While the Heydar Aliyev Centre's continuous architectural landscape merges various architectural components i.e. stairs, slab, wall, roof and bridge, it contains 3 major programmes. Convention Centre: 1200 seat auditorium with hydraulic orchestra pit will be used for both conventions and musical performances. The auditorium has

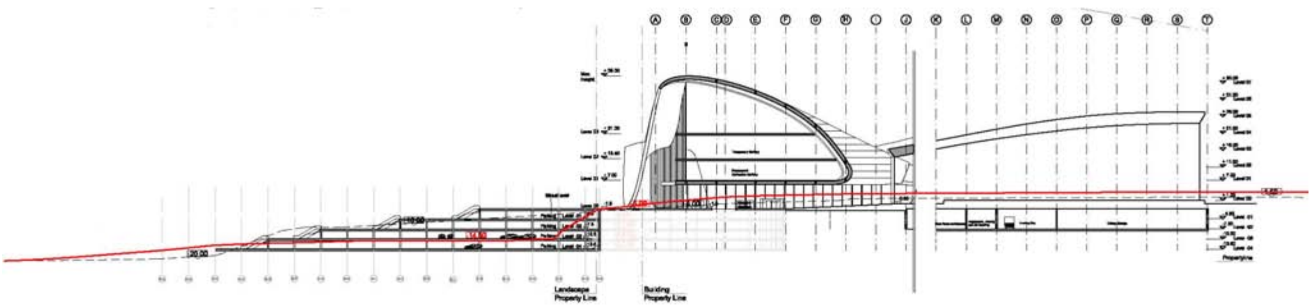

Figure 4. Longitudinal Section through Landscape, Car park and Museum

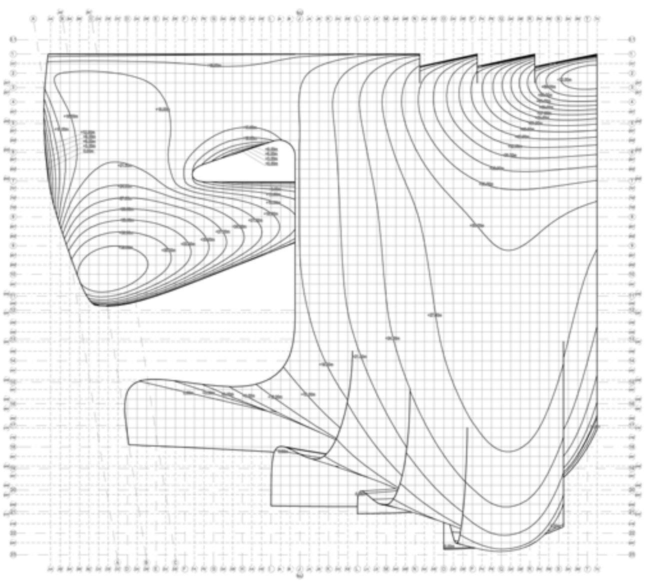

Figure 5. Topographical Analysis of Building Envelope

been designed for naked sound acoustic values, but with transformable coupling slots it can adjust to meeting purposes. Adjacent to the auditorium, the Multi-purpose hall is used as a banquet, event space or projection room to show films. This hall also expands into the garden to the north and subdivides into three smaller halls – each with separate direct public and service access. Upon arrival, the Museum greets visitors with a double-height space lobby that houses a grand staircase which seamlessly peels off from the interior skin of the building. This staircase, which leads to the higher museum levels where the heritage, permanent and temporary exhibitions will be showcased, is also used to display sculptures. Capping the museum volume, the presidential level looks out over the old city centre of Baku. The 8 storey Library is situated to face north, to take advantage of indirect diffused northern light. When viewed from the exterior, the volumetric massing and surfaces of the centre ensure that three distinct singularities are easily identifiable. The design layout allows each of these three entities to operate independently with their own entry and security areas. Internally, these three different zones function separately from each other but share some service areas. These communal zones contain cafeterias, meeting rooms, bars, restaurants and other supporting services (Figure 5).

IV. Structure

Baku, which in old Farsi means ‘where wind beats’, is subject to high wind loads throughout the year, and as the city lies within a seismic zone, the project’s structural engineers faced a multitude of challenges. The freeform structure of the project derives from the architectural design concept of modifying a single surface to adopt different functional requirements. The aim was to create a large column-free space giving visitors the opportunity of experiencing the fluidity of the interior. To achieve this, vertical elements are absorbed by the envelope and curtain wall system. The Heydar Aliyev Centre consists of 2 structural systems: Concrete and Space Frame with a single movement joint (Figure 6 and 7 on the following page).

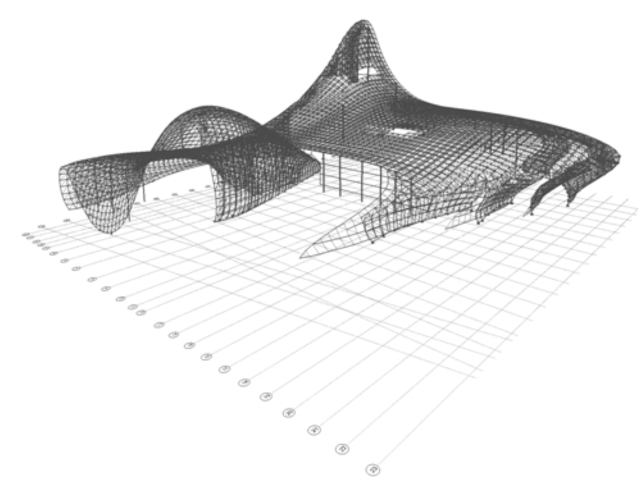

Figure 6. Structural System - Space Frame

The space frame enables the construction of this free form structure while offering significant savings in time throughout the construction process. The surface geometry driven by the architecture, dictates the need to pursue unconventional structural solutions; the introduction of curved 'boot columns' to achieve the inverse peel of the surface from the ground at the west, and the cantilever beams 'dovetails' tapering towards the free end, supporting the building envelope at the east. The substructure enables the incorporation of a flexible relationship between the rigid structural grid of the space frame and the free-formed exterior cladding seams which derive from complex geometry rationalization, architectural aesthetics and usage (Figure 6 on the previous page).

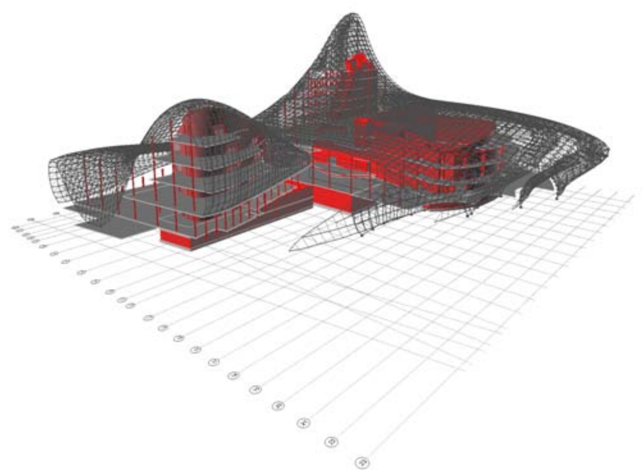

Figure 7. Structural System - Overall View

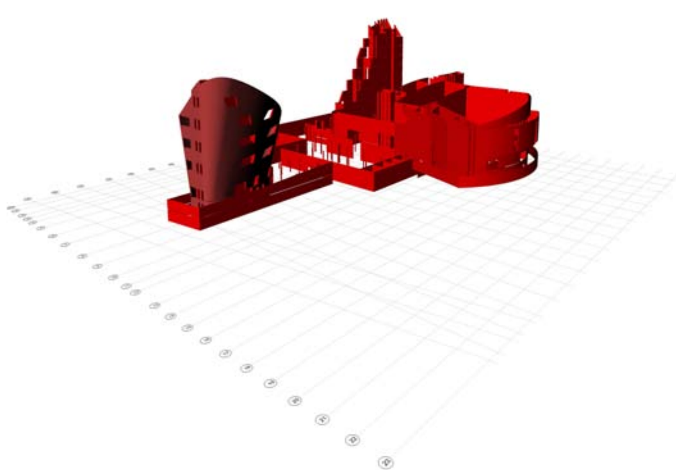

Figure 8. Structural System - Concrete Cores

V. Geometry and Materiality

A primary element that differentiates a rectilinear-surfaced volume from a volume with a fluid geometry is the way it reflects light. Each side of a cube or a box will reflect only one tone of light; however volumes with fluid geometries will reflect varying shades that continually

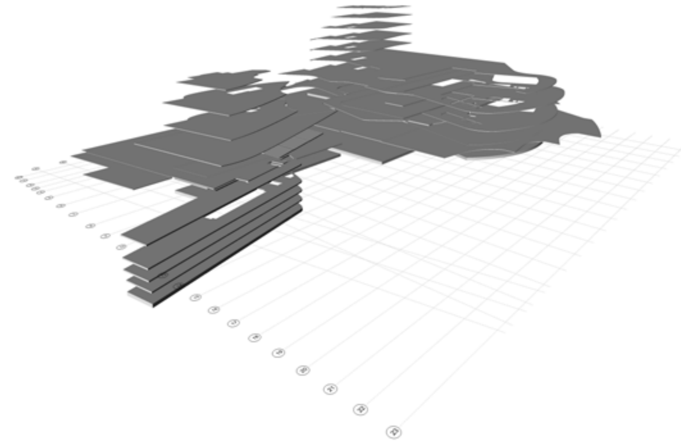

Figure 9. Structural System - Concrete Slabs

transform and flow into each other, creating much richer surface composition. The Heydar Aliyev Cultural Centre design achieves the ideas of a continuous architectural landscape by using two primary elements that are profoundly interlinked: complex geometry and materiality. The choice of the building's materiality enabled further development of the project's form. One of the most challenging and critical elements of the project was its external skin. The aim was to use a unifying material for both the plaza flooring and also the envelope cladding. This material needed to adapt to the plasticity of the geometry, whilst at the same time, offer the required colour, sheen, texture and technical specifications of UV protection, graffiti-proofing and slip resistance. Glass Fibre Reinforced Concrete, GFRC, is the ideal material, allowing the creation of the unique free form building design. A special extrusion process incorporates layers of glass-fiber into a concrete matrix. In the top and bottom layers the fibers are undirected and scattered; in the middle layer, they are set in fiber bundles that take the form of the roof. The omission of steel reinforcement allows the construction

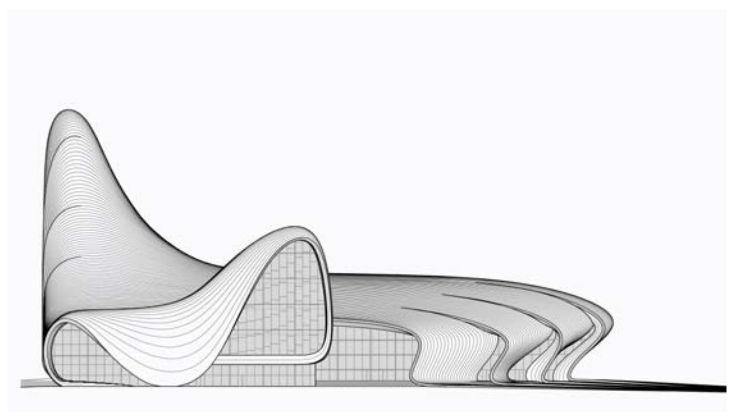

Figure 10. South Elevation

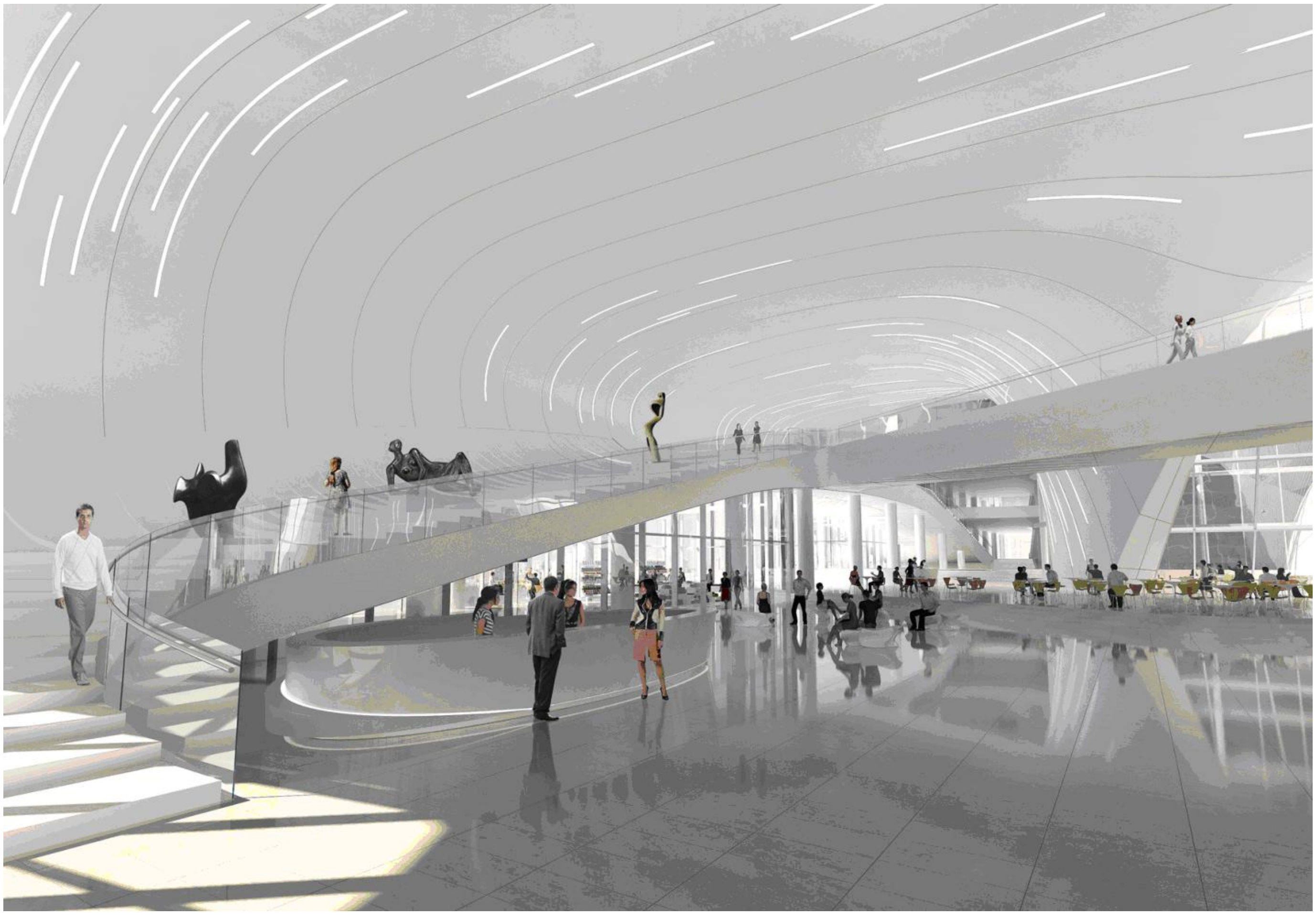

Figure 11. Interior View from Museum Entrance

of slim concrete elements which can accommodate high stress loads despite having a very thin section. The result is an extremely thin slab of 8-13 mm thickness that is very light-weight yet has a high flexural strength. Besides extrusion methods, using moulds allows GRFC panels to be created with complex geometries. In addition to fabrication restrictions, the panel dimensions must take transportation, installation, handling and assemblage into account. Geometrically the panels are divided into three categories: flat-planar, single curvature and double curvature. Flat panels are fabricated through an extrusion bed while single and double curvature panels are moulded. For obvious reasons, the panels fabricated from moulds are more expensive. The panels are also classified as per their location and consequently usage: plaza, transitional and envelope panels. Flat plaza panels and panels used in the transition zone must be treated differently due to their exposure to human reach, heavy foot load, anti-slip requirements, graffiti-proofing and scratch protection. To prevent dirt gathering in-between the panels and create a smooth surface to the plaza that enables pedestrians to cross with ease, metal and rubber gaskets are located between the panels. As the surface undulates upwards, these gaskets peal inwards implying a restricted pedestrian access through the transition zones (Figure 10 on the preceding page).

In addition, the support structure varies at each of the above mentioned locations. While the plaza panels rest on gravel and concrete, the panels at the transitional zone are supported on a wedge-shaped concrete footing. The envelope panels working as rain screen cladding system are connected to the space frame nodes via pedestals and the sub-structure, enabling maximum flexibility between the structure and cladding. High wind loads, maintenance and lightning protection were among many other issues that posed challenges during the development of the envelope panels. The surface's homogenous appearance actually has underlying heterogeneous properties which respond to various functional necessities. Unlike the exterior skin's unifying material used for cladding and flooring, the interior surface consist of two different materials: The areas where the envelope folds inwards, creating slabs and stairs, required a different treatment (Museum's first floor exhibition area, Library's ground level lobby and the Grand Stair which performs as a bridge displaying sculptures as it connects the Museum to Library on the first floor). Bolidt, synthetic resin flooring enables a smooth transition of the geometry from floor to wall, while providing the required

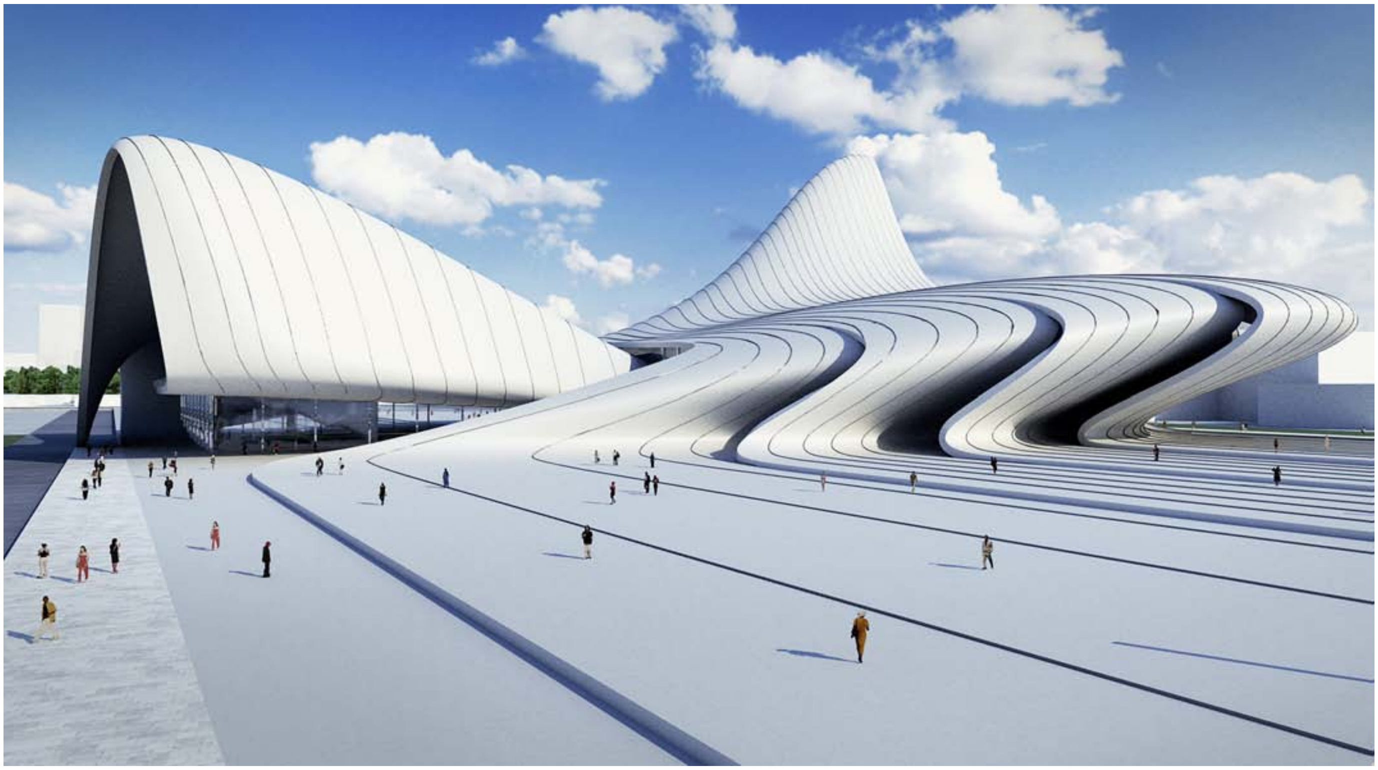

Figure 12. Exterior View from East Plaza

acoustical and technical values, as well as colour, geometry adaptability and aesthetic requirements.

The interior surface (Figure 11 on the previous page) of the walls and ceiling will be constructed of composite gypsum boards with white matt paint finish, responding to acoustical and lighting requirements, as well as parameters such as load (resulting from hanging art work at the museum) and location (required strength at areas within human reach).

VI. Seams

In this architectural composition, if the surface is the music, then the seams are the rhythm. Numerous studies were carried out on the surface geometry of the Heydar Aliyev Centre to rationalise and panelise it while maintaining the continuity of the entity and landscape. Through various design stages our goal was to achieve a seamless transition between disparate elements. However, on a large architectural scale the introduction of seams is necessary as manufacturing, handling, transportation and assembly become important parameters determining the panel size limitations. The expansion joints are also incorporated within the seams allowing movement due to deflection, external loads, temperature change, seismic activity and wind loads. The seams also give a better understanding of the project's scale and emphasize the continual transformation and implied motion of the project's fluid geometry; offering a pragmatic solution to practical construction issues. The seams are an essential element of the design, generating an elegant relationship between repetition and continuous variation of the Centre's surfaces (Figure 12).

VII. Conclusion

Good communication and coordination amongst architects, engineers, fabricators and contractors are key factors in realizing an architectural vision of such complexity; enabling a better understanding of materiality and assembly systems, whilst also contributing to the further evolution of the architecture and construction industries as a whole. The Heydar Aliyev Cultural Centre has used this advanced concrete technology to marry the sensual nature of Azerbaijan's culture with the ambition, optimism and boundless energy of the Azeri people. As with all our work, we investigate and research the landscape, topography and circulation of the site to inform our design; ensuring the building becomes "embedded" into its surroundings – giving the cultural centre the strongest relationship with its unique context within Baku. The GRFC paneling used throughout the centre plays a critical role in its design – allowing the building to sit perfectly within its environment and embrace the future possibilities of the nation.

Fabric Formwork: The State-of-the-Art and Future Endeavors

Diederik Veenendaal

Department of Architecture, ETH Zurich
Wolfgang-Pauli-Strasse 15, CH-8093 Zurich, Switzerland
Department of Buildings, Witteveen+Bos
Leeuwenbrug 37, P.O. box 233, 7400 AE Deventer, Netherlands
veenendaal@arch.ethz.ch

***Abstract*— This document gives a brief overview of what fabric formwork technology entails, as well as an overview of current applications and research efforts. Furthermore, it discusses research that has been carried out at the Delft University of Technology as part of the author's Master's thesis. The topic was evolutionary optimisation of fabric formed structural elements. Further proposed research based on the results is discussed.**

I. INTRODUCTION

Traditionally concrete structures are thought to be generally rectangular in appearance, and perceived as crude in nature. This public image of concrete sharply contrasts the fact that it is a cast material with all the geometric freedom that implies. On a large scale this property is more often fully utilized, illustrated by seminal work of shell builders such as Heinz Isler, or by more contemporary free form architecture by the likes of Santiago Calatrava and Zaha Hadid. However, free form architecture is often capital and labor intensive, and only comes to fruition under specific socio-economic circumstances.

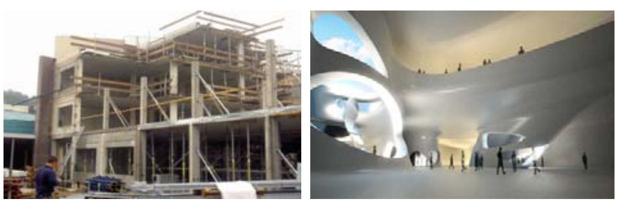

Fig. 1 Contrasting orthogonal, rectangular prefabrication with free form architecture and prevailing esthetics

On a smaller scale, on the level of structural elements, applying free form to concrete implies intricate formworks or complex computer-driven production methods. Fabric formwork technology, as it is now envisioned, addresses this apparent contradiction (Fig. 1) of concrete's inherent fluidity, yet angular application. It can offer relatively simple production for economically feasible and esthetically pleasing designs.

II. FABRIC FORMWORK TECHNOLOGY

Fabric formwork is characterized by the use of coated fabrics or geotextiles as the main material for a concrete mold. One or more layers of fabric are filled or injected with fresh concrete. The fabric can be either prestressed or slack, as the hydrostatic pressure of the fresh concrete ultimately stresses the formwork. The design considerations for these formworks is similar to those in the design and engineering of tensioned membrane structures, involving the interaction of prestress, non-linear material behavior and the support conditions. Additionally, fabric formwork has concrete pressures and fluid structure interaction as complicating factors. There are two aspects that distinguish the design of fabric formwork from that of membrane structures, caused by its short term use. Firstly, the formwork invites the designer to apply not only fixed, but also supports along which the fabric may slide during stressing and casting, normally leading to long-term wear and tear. Secondly, the stress distribution within the fabric may be highly uneven. One result of these possibilities is shown in Fig. 2.

Fig. 2 Concrete truss cast at the University of Manitoba, with the timber and fabric formwork shown below in two seperated parts

Practical applications of fabric formwork are commonly found in the construction of foundations, especially for hydraulic structures. Other examples are mostly confined to simple columns or walls, or non-structural applications.

Research into more geometrically pronounced structural elements, such as shells or non-prismatic beams [1], has yet to lead to widespread use of this technology. The lack of sufficient engineering understanding of these elements is one of the contributing causes. Computational research at the Delft University of Technology focused on this issue.

III. EVOLUTIONARY OPTIMISATION OF FABRIC FORMWORK

The design of structurally efficient non-prismatic shapes has been investigated for the last few decades with Evolutionary Structural Optimisation (ESO) as one of the most prominent methods of finding optimal forms [2]. This and similar algorithms remove inefficiently used material and produce results that are often described as organic or skeletal (Fig. 3). However, resulting shapes are difficult to manufacture economically by conventional means and also do not take constraints posed by fabric formwork into account.

Fig. 3 Simply supported beam optimized with ESO

A new computational framework was devised in which three steps necessary in fabric formwork design were integrated, the form finding of the fabric, the analysis of the resulting beam and finally, the optimisation of the beam shape. The entire framework was written in Java and interfaced with ANSYS for finite element analysis of the concrete beam. There are a few form finding algorithms available for the design and engineering tensioned membrane structures. One commonly used and well defined algorithm, dynamic relaxation [3], was chosen and adapted to use for fabric formwork (Fig. 4).

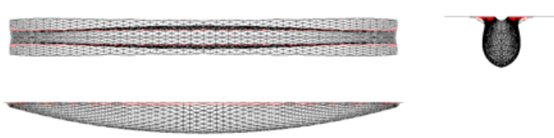

Fig. 4 Example of beam shape developed with dynamic relaxation

The fabric mesh was then translated to a three dimensional concrete mesh in ANSYS and then analyzed to determine the volume and overall stiffness in terms of strain energy. These properties were then used to evaluate the beam. Optimisation of the beam shape was performed by using a genetic algorithm, differential evolution [4]. Genetic algorithms use an analogy with biological evolution by continuously generating and evaluating a certain number of solutions, then combining their properties based on the evaluation to form a new generation of solutions. In this case a set of beams was continuously generated by form finding and subsequently analyzed and rated using the finite element analysis in ANSYS (Fig. 5). The entire process is fully automated and produced optimized, manufacturable fabric formed beams.

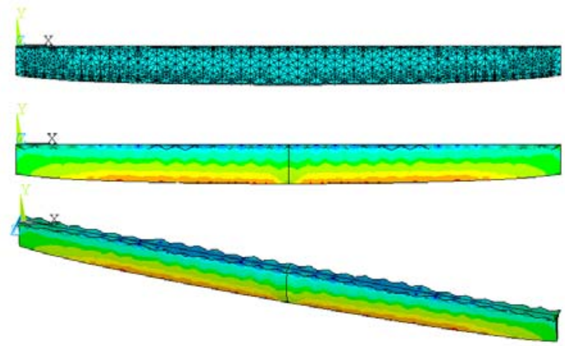

Fig. 5 Example of meshing and analysis of optimised result

It has been shown that constraints posed by fabric formwork can be integrated in a single functional design tool, thereby bridging the gap between computational optimisation and manufacturability [5]. A linear elastic comparison between the resulting beam shapes and rectangular beams shows that significant material reductions can be realised (Fig.6, Table I).

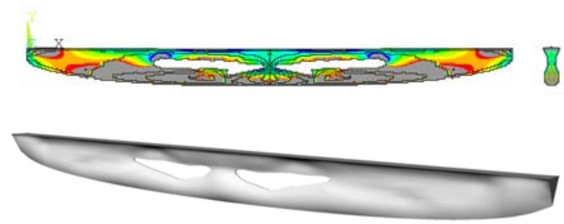

Fig. 6 One of the final results obtained from optimization, used for comparison in Table I.

TABLE I
LINEAR ELASTIC COMPARISONS OF FABRIC FORMED BEAM WITH RECTANGULAR BEAM

Beam	**volume**	**height**	**deflection**
reference beam	100%	100%	100%
fabric formed beam, equal volume, equal slenderness	100%	>100%	9%
fabric formed, equal volume equal construction height	100%	100%	22%
fabric formed, equal deflection equal construction height	42%	100%	100%

IV. Future research

Several recommendations were made that form the basis for future research at the ETH Zurich. The automated evolutionary optimisation will be (partially) abandoned due to its high computational demands in favor of more user interaction and engineering judgement. The scope of the research will also be broadened to include entire structural systems, whilst investigating reinforcement strategies, the role of fabric patterning (sewing or welding fabric together) as well as implications of the design on the supporting frame of the fabric. Parallel to this computational work, quantitative information will be collected on completed projects and prototypes to gain further insight into the true economy and value of the technique. This should offer better understanding of the full potential of fabric formwork and work towards handing both architects and engineers better tools to design, analyze and ultimately realize fabric formed structures.

Acknowledgment

The author wishes to acknowledge the supervisory committee of his Master's thesis for their help and guidance during this research carried out at the Delft University of Technology; prof. dipl.-ing. J.N.J.A. Vambersky, ir. J.L. Coenders, dr.ir. P.C.J. Hoogenboom and dr.ir. C. van der Veen. Furthermore, prof. M. West of the University of Manitoba in Winnipeg, Canada provided significant amounts of information on fabric formwork technology and enthusiastically corresponded on the topic. Finally, prof. dr. P. Block has provided a PhD position at the ETH Zürich for continuation of this research and further exploration of the topic.

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Precast Concrete Cores in High-rise Buildings

Structural Behaviour of Precast Corner Connections

Koos Tolsma

Faculty of Civil Engineering and Geosciences, Delft University of Technology
Stevinweg 1, 2628 CN Delft, Netherlands
koos@studio-dck.nl

***Abstract*— This paper describes the results of a MSc thesis on the structural behaviour of a high-rise core composed of precast elements. The structural design of a high-rise building is governed by requirements for stiffness. To determine the stiffness of a precast concrete core, a time-consuming finite element calculation is required. This paper proposes a reduction factor which can be applied on the stiffness of a monolithic core to estimate the stiffness of a precast concrete core in the design phase. To determine this reduction factor the influence of three precast corner connections on the stiffness of a core is studied.**

I. INTRODUCTION

Over the last decades one can see a clear increase in the use of precast concrete technology in high-rise buildings. Main advantages are the high speed of construction and the reduction of the amount of labour on the building site. Recent high-rise projects like Strijkijzer in The Hague and Maastoren in Rotterdam, where precast elements in the façade provide structural stability, pushed the limits in terms of height and construction speed. However, with the precast elements located at the façade a rather closed façade was obtained. To realise an architectural design with a transparent glass façade combined with a structural design entirely in precast concrete, this thesis aims at the structural design of a core composed of precast elements.

A core composed of precast elements differs from a cast in situ core in having connections between the precast elements. From preceding research [1] the stiffness reduction due to the horizontal joints and the open vertical joints can be estimated. In addition to this, the stiffness of a core depends to a large extent on the structural behaviour of the corner connections. Although various corner connections in high-rise buildings are applied frequently, little is known about their structural behaviour.

What is the best precast corner connection and what is its influence on the stiffness of a high-rise core composed of precast elements?

II. APPROACH

Three types of precast corner connections are considered (Fig. 1):

1. Staggered connection (SC)
2. Interlocking above ceiling connection (IACC)
3. Interlocking halfway connection (IHC)

A. Height of shear key

First, the connections differ with regard to height of the shear key (0.8 m for IACC, 1.7 m for IHC and 3.4 m for SC). The influence is studied with a 2D FE model. The 2D model (Fig. 3) results in a discrete connection stiffness:

$$K_{discrete} = \frac{F_r}{\delta_A - \delta_B}\ [MN/m]$$

B. Connection density

Secondly, the connections differ with regard to connection density. Because the IACC and IHC are spread over one story and the SC over two stories, the connection density of the IACC and IHC is twice as large. This influence is taken into account by dividing the discrete stiffness by the variable connection height and the constant wall depth:

$$K_{smeared} = \frac{K_{discrete}}{h \cdot d}\ [MN/m^3]$$

This smeared stiffness is subsequently imported between perpendicular core walls of the global 3D model (Fig. 6) to study the influence of the corner connections on the stiffness of the core.

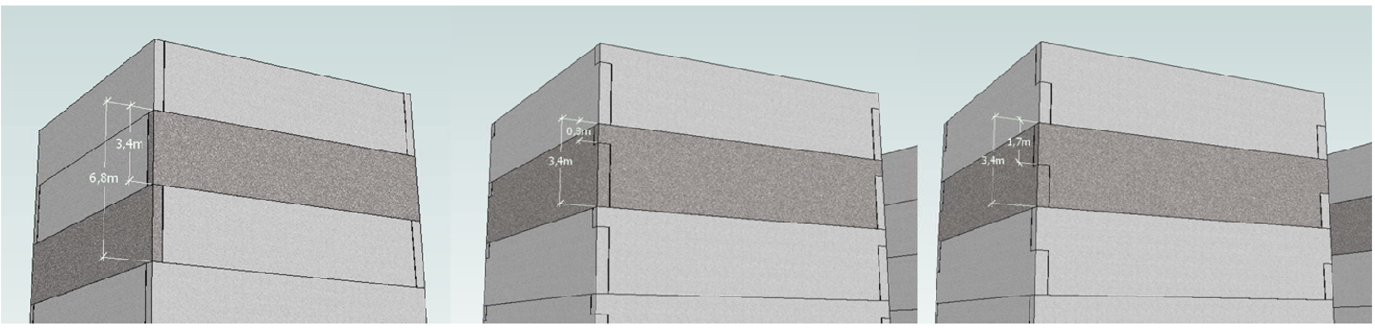

Fig. 1 Three considered corner connections, from left to right: SC, IACC and IHC

III. 2D FE MODEL OF CORNER CONNECTIONS

The 2D FE model is derived from a core composed of precast elements as depicted in Fig. 2.

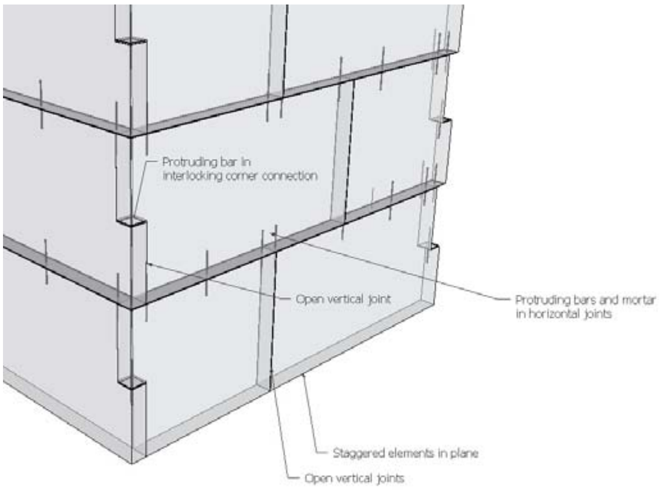

Fig. 2 Connections in core composed of precast elements

In plane the elements are placed in a staggered pattern with open vertical joints. The boundary conditions $K_{nn;hor.joint}$ and $K_{tt;vert.joint}$ are derived from the properties of horizontal joints with mortar and protruding bars [2]. The load displacement diagram is obtained by plotting the vertical load against the mutual displacement $\delta_A - \delta_B$.

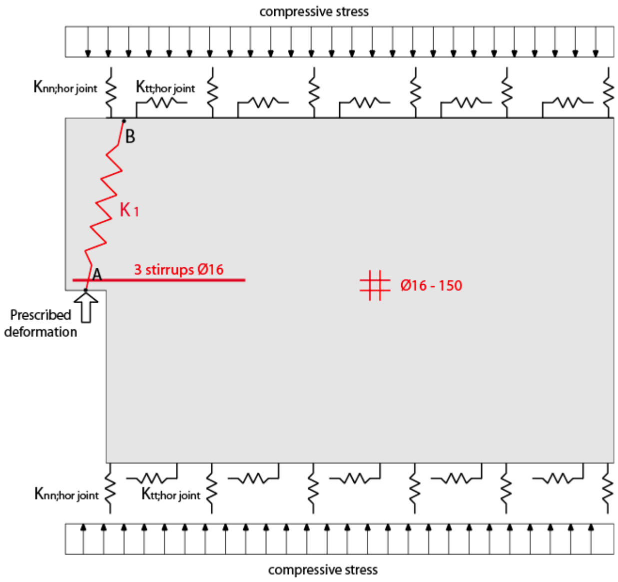

Fig. 3 2D FE model of IHC

The load displacement diagram of Fig. 4 shows for all considered connections an important difference in the behaviour before the concrete ruptures and after. Before F_r the behaviour is linear elastic and the shear key is compressed vertically. The vertical strain diagrams of the FE models show that the compressed zone is equal for all connections, explaining the comparable values of the discrete stiffness until F_r. On beforehand one might have expected that the SC would be stiffer due to its higher shear key, but the FE model shows that only a part of the concrete near the load is compressed vertically and the concrete above this zone is not compressed. In other words: the influence area is limited.

After F_r the shear key rotates and the horizontal reinforcement is activated. The amount of horizontal reinforcement determines the behaviour until failure. As Fig. 5 shows, the amount of reinforcement determines the stiffness after F_r, but the stiffness is significantly reduced after F_r.

In this research it is assumed that the vertical shear forces in the corner connections of the reference project should not exceed F_r to assure linear elastic behaviour and the high value for the discrete stiffness.

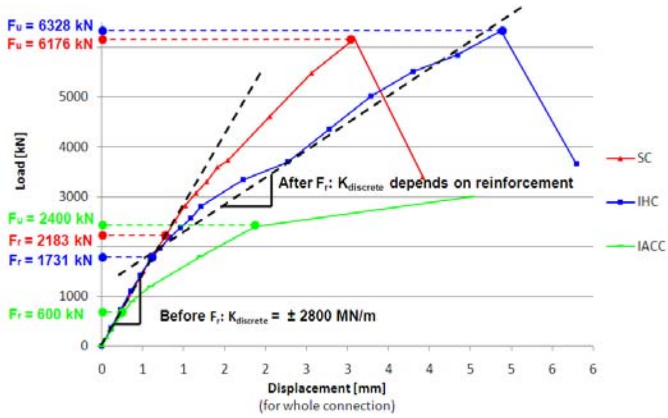

Fig. 4 Load displacement diagram of the considered corner connections

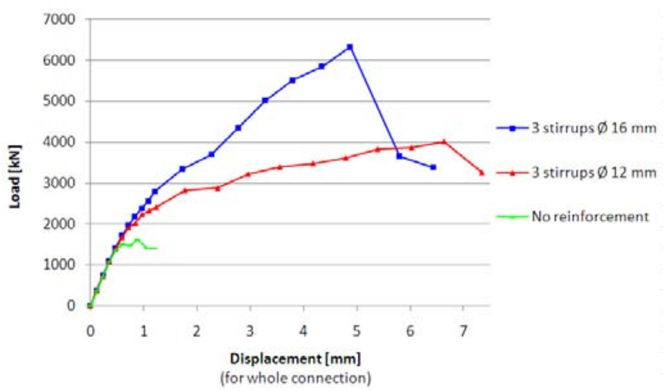

Fig. 5 Influence of reinforcement on load displacement diagram of IHC

IV. 3D FE MODEL OF CORE

To obtain realistic results the dimensions and loadings were adopted of the reference project the Rembrandt Tower in Amsterdam. The global 3D model is composed of simple core walls connected by interface elements in the corners (Fig. 6). This interface has the parameters of the smeared connection stiffness.

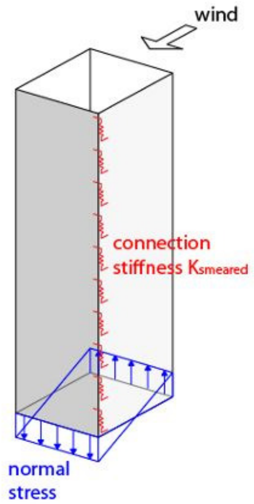

Fig. 6 3D FE model of core

Monitoring points in the 3D model showed that the maximum shear stress in the corner connection is 0.33 MPa. The strength of the connections f_v should be below the maximum stress. To take into account the influence of cyclic loading, a reduction factor of 0.6 was applied at the strength. From various tests in the research on dowels [3] was concluded that at a maximum load of 60 % of the failure load a specimen will not fail. Implementation of this factor on the strength is given in Table 1.

TABLE 1
$K_{DISCRETE}$ FROM FE MODEL OF CORNER CONNECTION, $K_{SMEARED}$ FOR INTERFACE ELEMENTS OF FE MODEL OF CORE, STRENGTH OF CONNECTIONS F_V

	$K_{discrete} = \frac{F_r}{\delta}$ [MN/m]	$K_{smeared} = \frac{K_{discrete}}{h \cdot d}$ [MN/m^3]	$f_v = 0.6 \cdot \frac{F_r}{h \cdot d}$ [MPa]
IHC	2924	1720	0.61
IACC	2689	1582	0.21
SC	2852	839	0.39

The strength f_v of the IACC is lower than the shear stresses in the corner connections of the global 3D core and is therefore not suitable to be applied in the core of the reference project.

The smeared stiffness of the IHC and SC is subsequently imported at the interfaces between the perpendicular core walls of the global 3D model, and the deflections at the top under wind loading is monitored. The influence of the corner connection stiffness is shown in Table 2.

TABLE 2
INFLUENCE OF CORNER CONNECTIONS ON LATERAL DEFLECTION CORE

Corner connection	**Deflection at top** [mm]	**Difference** [%]
Monolithic	47,7	100%
IHC	49.3	103.3%
SC	50.5	105.9%

V. CONCLUSIONS

- Of the three considered precast corner connections the IHC has the best structural behaviour since it has the highest smeared stiffness and the highest strength.
- Although the corner connections transfer shear forces, the elements are compressed locally at the shear key due to normal stress. This compressed zone is limited and comparable for all considered connections.
- Compared to a monolithic corner connection the IHC shows an increase of lateral deflections of just 3.3 %. The SC results in an increase of 5.9 %.
- Since the strength of the IACC is lower than the shear stresses in the core of the reference project, the IACC is not suitable to be applied in this high-rise core.

VI. RECOMMENDATIONS

- As stated in III the discrete stiffness is equal for all connections since only a certain part of the concrete near the load is compressed. Further research is required to determine the height of this influence zone.
- Besides the stiffness reduction due to the corner connections the stiffness is also reduced due to staggering of the elements in plane. From [1] was concluded that with staggered elements with open vertical joints the deformations increase with 5-8%. Further research is required how these reduction factors relate to each other.
- The reduction factor for cyclic loading is derived from a research on dowel action. With shear keys at the precast elements this factor could be different and should be further studied.

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High performance Concrete: a Material with a Large Potential

Joost WALRAVEN
Professor, Dept. of Civil Engineering, Delft University of Technology, The Netherlands.

ABSTRACT:
High Performance Concrete is a material that was regarded as "academic" for quite a number of years. Now, the profits of this material are becoming to be recognized. The high compressive strength is not the only advantage of this material. The fibers lead to small crack distances and give the material large ductility. The very dense material structure can as well result in high durability. This makes the material suitable for the design of lightweight slender structures with a long service life, as well as surprising architectural structures. On the other hand the material is appropriate for repair of structures, such as bridge decks. First applications show convincingly a large potential. At this moment an international committee (fib Task Group 8.6) works on producing an international recommendation.

Keywords: High performance fiber concrete, design recommendations

1. INTRODUCTION

High performance fiber concrete is a material with a rather short history. Its introduction was relatively sudden, if one regards the very gradual development from conventional strength C45-C65 to high strength C95-C115. High performance concrete was not the continuation of this development, but a major step ahead. The idea to realize a material with an optimum particle packing, to limit the maximum particle diameter to a maximum of 1-2 mm, to use water – cement ratio's which are so low that all the water is used for hydration and to add fibers for ductility meant a revolutionary step forward. All at once concrete strengths over 200 MPa were possible. Probably the first concrete technologist who produced high strength fiber concrete was Hans Hendrik Bache in Denmark. He published already in 1981 about ultra fine particle based materials [1], Fig. 1.

Fig. 1. Hans Hendrik Bache, pioneer in the development of ultra fine particle based materials.

The brittleness of high strength concrete inspired researchers to add short steel fibers to the mixtures to provide adequate ductility. Especially in France experience was gained with steel fibers as an additional component, which resulted in a material which was not only very strong but as well very ductile. It was realized that the term "High Strength Concrete" would basically non cover the real significance of this material. Very soon therefore the name was changed into "High Performance Fiber Reinforced Concrete".

In the last decade of the previous century, increased interest developed into service life design. Many experiences had learned that concrete is not the eternal material that it was thought to be for a long time. Penetration of chlorides through the concrete, as an example, turned out to be a major cause of deterioration of concrete structures. The large scale deterioration of structures was the reason that design for durability was introduced into the codes with the same significance as design for safety and serviceability. High Performance Fiber Reinforced Concrete (HPFRC) if composed and produced well, can have a very small permeability for damaging chemicals like chlorides. This is a further reason to speak about "high performance" than "high strength". In this paper some applications are shown and trends are sketched.

In order to introduce HPFRC into building practice reliable recommendations are necessary to increase the confidence of users. In this respect it should be noted that codes are available for traditional fiber concrete already. A difficulty is that it is not easy to develop a consistent code for fiber concretes of any arbitrary strength. Nevertheless it is worthwhile to put substantial effort into the development of consistent and compatible codes for fiber concrete in all its variety. This fits well into the idea of developing a new design philosophy for materials and structures, recognizing the large potential of new types of concrete to be designed for performance and not only for strength. This way of designing is denoted as “defined performance design” and the introduction of this new design philosophy is regarded as one of the most important challenges for future codes.

2. THE POTENTIAL OF UHPFRC FOR NEW TYPES OF STRUCTURES

The potential of the HPFRC has been demonstrated during the last years by quite a number of interesting applications. Fig. 2 and 3 show the potential of HPFRC for the design of structures with a high architectural quality. Fig. 2 shows an elegant spirol staircase as realized in Denmark. The stairs are made of a concrete which contains both fibers and traditional reinforcing bars. The idea to combine

Fig. 2. Staircase of UHPFRC in Denmark (Courtesy B. Aarup [2]).

fibers with traditional reinforcement is very good. Early applications in Denmark showed that it is possible to combine large volumes of fibers (2-4%) with high reinforcing ratio’s of traditional reinforcing bars (5-10%). This composite material was designated as CRC (Compact Reinforced Composite). The advantage of combining fibers with rebars is that rebars can be economically used for the main bearing function, whereas fibers allow very thin structural elements since they control splitting and spalling mechanisms and very effectively control cracking.

Meanwhile a number of medium span bridges have been built in HPFRC. The most famous bridge is the first one, a pedestrian bridge in Sherbrook. In France the bridge in Bourg les Valence is well known, whereas in Japan the Sakata Bridge was an interesting demonstration of the potential of HPFRC. Those bridges do not only have the aim to get design engineers acquainted with the new technology, but as well to demonstrate potential customers that practical applications with HPFRC are possible. A recent example of such a pilot project was the Gärtnerplatz bridge in Kassel [3], Germany, which was opened to the public in 2007. The bridge has a total length of 133m and consists of an upper slab of concrete with a strength of 185 N/mm2 and a thickness of 80-120 mm. The slab is supported by a three dimensional steel truss. The high performance concrete deck is connected to the truss with a glued connection. This concept is promising: high performance concrete and steel can be an excellent combination if the best properties of both materials are combined in an optimum way. Research on interface shear design concepts deserves therefore due attention. Substantial recent experience in various types of structures has been gained as well by French engineers (e.g. Behloul [4]).

Fig. 3. Gärtnerplatzbridge in Kassel, Germany (2007), consisting of a high performance deck glued to a 3-D steel truss

These new structures show that HPFRC is a material to realize light, slender and durable structures, which are appealing from an esthetic point of view and ecologically interesting by the possibility to minimize the use of materials.

Further to its strength and ductility HPFRC offers other favorable properties. Considering the wish to design structures according to a specified (long) service life, the low permeability of the material offers chances.

Important work in this respect was carried out at the University of Kassel in Germany. Schmidt [5] studied the porosity of two HPFRC mixtures (C180-200) and compared them with the porosity of conventional concrete C45/55 and high strength concrete C105. As a result of the very low W/C ratio of about 0,20 and the high packing density of the aggregate particles and the fillers, the total porosity (air pores + capillary pores + gelpores) of self compacting or nearly fully consolidated HPFRC sinks down to about 4-6 Vol.-%. Fig. 4 shows the distribution of the pore-radii, measured by mercury intrusion. It is shown that the capillary pores, responsible for the transport of O_2, H_2O, CO_2 and Cl^-, are practically absent. This is the reason that a skillfully produced HSFRC has a very high resistance against carbonation and chloride ingress and against frost-thaw salt exposure.

In additional tests on carbonation Schmidt [5] showed, that for the HSFRC mixtures investigated, after half a year a carbonation depth of only 0,3-0,5 mm was reached. After 3 years a depth of 2 mm was measured which is much smaller than found in conventional mixtures.

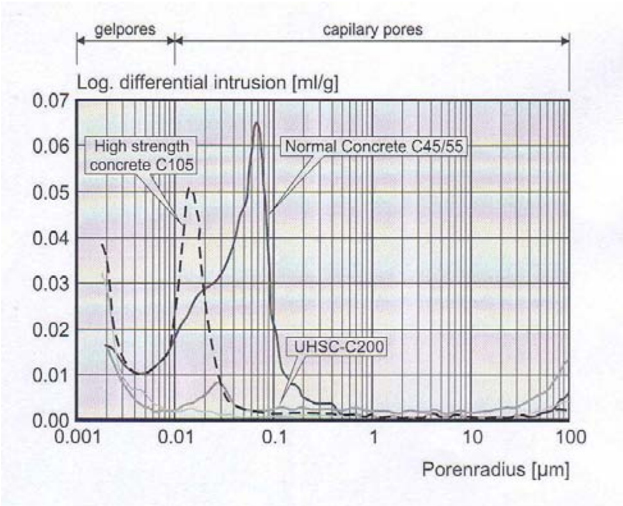

Fig. 4. Comparison of pore radii distribution for normal concrete C45/55, High Strength Concrete C105 and Ultra High Strength Concrete C200 (Schmidt [5]).

In the scope of the same program chloride penetration tests were carried out. Chloride diffusion was tested with the rapid test method developed by Tang and Nilsson [6]. According to this method a short concrete cylinder with a thickness of 35mm is placed in between two chambers. In the one chamber there is water, and in the other a 10-% chloride solution. The chloride diffusion is accelerated by applying a voltage difference of 40 V between the chambers for a period of 6 hours. In reference specimens of normal concrete a chloride penetration depth of 23 mm was measured. In the HSFRC the chloride ions only penetrated over a depth of 1mm in the concrete.

Finally Schmidt [5] carried out pilot tests on the resistance of HPFRC to frost-thaw cycli and simultaneous salt exposure. Fig. 5 shows the loss of

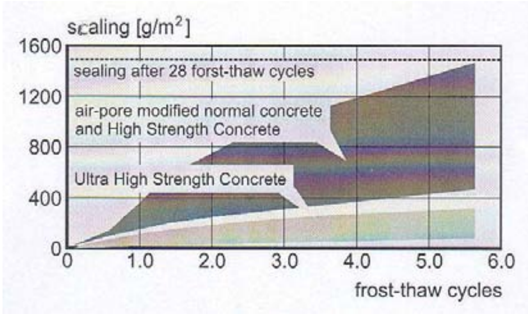

Fig. 5. Comparison of scaling due to frost-thaw cycles between UHSC, HSC and aerated normal concrete according to Schmidt [5]

material due to scaling (g/m^2) for HSFRC in comparison with conventional high strength concrete (C100) and normal concrete with air entrainment. The values measured for HPFRC appear to be very low.

An example of an application where both the strength of the material and its durability are combined is the use of HPFRC for anchor elements of prestressing tendons in a sea environment, Fig. 6.

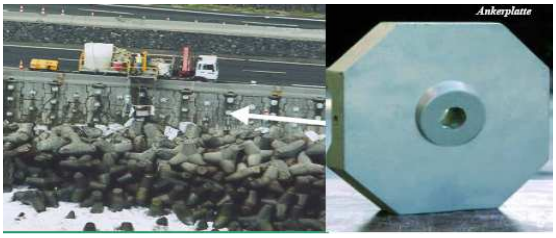

Fig. 6. Application of HPFRC for prestressing anchors in a coastal area (Ile de la Reunion, [7])

In the Netherlands an analysis was made of the suitability of UHPFRC for the gates of the Storm Surge Barrier “Eastern Scheldt” in The Netherlands, fig. 7.

Fig. 7. Storm Surge Barrier in the Eastern Scheldt, The Netherlands

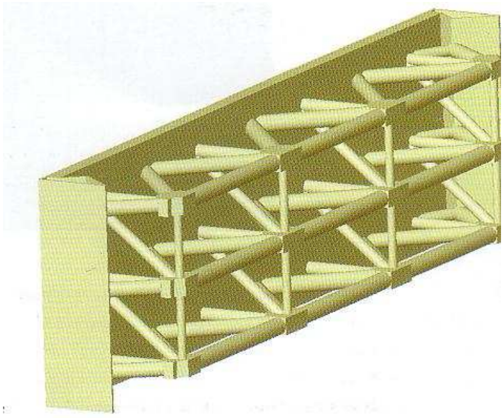

Fig. 8. Design of HPFRC door as an alternative for the existing, maintenance intensive, steel doors.

The storm surge barrier was built in 1980-1986 in order to avoid in future a flood disaster like the one that occurred in 1954. It is an open semi-barrier with 65 lifting doors made of steel, which are closed if a storm tide is announced. The doors have a width of 45 m and a height of about 15m. Because of the very aggressive marine environment the steel doors have been provided with a coating in order to protect them against corrosion. However, inspection in the early nineties showed, that nevertheless substantial corrosion developed. So, the coating had to be replaced, which increased the maintenance cost significantly. At TU Delft, in cooperation with the Dutch Ministry of Infrastructure, an investigation was carried out in order to find out whether a door made of HPFRC would be a viable alternative. From the point of view of durability this is an excellent solution. It was shown that in the case of an eventual new structure, a door in HPFRC would be the cheapest solution with regard to the integral cost, including maintenance, followed by an aluminized steel door. The investigation showed furthermore, that a door in HPFRC would have a weight of 640 tons, whereas the weight of a steel door is 450 tons. This means that even an exchange of doors could be considered, because the lifting equipment had been over-designed and nowadays low-friction materials are available which would reduce the frictional forces, exerted on the door during lifting.
A plan has been made to produce one door in HPFRC and substitute it for a steel door, as a large scale pilot project. Due to a reorganization of the Ministry the definite approval is still standing out.

Another very interesting application is the use of HPFRC for prestressed sheet piles. Prestressed sheet piles are normally made of steel, but have been produced in concrete C55/65 as well. The thickness of those piles, which were both reinforced and prestressed, was 120mm. A new self-compacting high strength fiber mixture was developed in order to reduce the slenderness of the piles. The new piles are prestressed in longitudinal direction by strands, but contain no mild reinforcement, Fig. 9. The mixture developed consisted of 913 kg/m^3 cement, 61 kg/m^3 silicafume, 207 liters/m^3 of water, 1098 kg/m^3 aggregate with a maximum diameter of 1mm, 125 kg/m^3 of straight steel fibers with a length of 13mm and a diameter of 0.16mm, and 21 l/m^3 superplasticizer. The cube compressive strength after 24 hours was already 74 MPa. Hence, the elements could be demoulded very quickly. After 28 days the cube compressive strength was 120 MPa. The centric tensile strength was 6 MPa after 1 day, 12 MPa after 1 week and 13,5 MPa after 28 days. The price of the concrete was 450 Euro, which was about 4 times the price of 1 m^3 concrete C55/65. At first sight this seems quite high, but it should be realized that the piles have a thickness of only 45mm in stead of the 120 mm for concrete C55/65. So, the necessary volume of concrete is reduced to about 1/3. Because of the smaller cross sectional area less prestressing steel is necessary than in a pile in C55/65, whereas mild steel was omitted totally. Another advantage is that the high strength concrete piles, contrary to the C55/65 piles, can be stacked on one another and can therefore very economically be stored and transported. At the site they can be handled more easily and it takes less time to bring them into the bottom. This is an important advantage, because sheet pile walls have often a length of hundreds of meters (Fig. 10), so that considerable savings in time and money can be achieved. Moreover, by virtue of the fibers hardly any damage occurs during hitting the piles into the bottom. So finally a very competitive product has been achieved. It should be noted that the durability of the concrete is crucial for its success. The new piles, with a thickness of 45mm, are prestressed with strands ½", so that the concrete cover is only about 15 mm.

Fig. 9. Formwork for a high performance fiber reinforced sheet pile, Spanbeton, The Netherlands

Fig. 10. Earth retaining wall of concrete sheet piles

4. RETROFITTING BRIDGE DECKS WITH HIGH PERFORMANCE CONCRETE

A further interesting application of high performance concretes regards the repair of bridge decks. In The Netherlands a substantial number of bridges have been built with orthotropic steel decks with an asphalt layer on the top. This type of bridges, however, often show problems, related to flexural deformations of the deck plate under traffic and their effect on the ribs or stiffeners, cross beams and girders. Due to the increase of both the traffic intensity and the axle loads, which were not foreseen in the initial design, premature fatigue cracks occurred. Repairing the cracks and applying a new asphalt layer can give a solution for a number of years, but it is not satisfactory, also because of the shutdown time for the traffic necessary for repair. Another solution that was proposed was the replacement of the asphalt layer by a reinforced high performance concrete overlay, which is bonded to the bridge deck (Boersma [8] Braam [9]). Tests have been carried out into the effect of concrete overlays, which contained one or more layers of welded mild reinforcement (bar diameter 8mm and bar spacing 50mm). The concrete contains both steel fibers and acrylic fibers. The average concrete cube strength is 120 MPa. The concrete contains about 70 kg/m^3 steel fibers (12mm x 0,4mm). The thickness of the overlay is about 50-60 mm. The connection to the steel beams is made by at first applying an epoxy-layer (2mm) at the steel surface, on which split (4-6mm) is sprayed in order to obtain an interface layer with sufficient bonding capacity. Fig. 11 shows a steel beam with an overlay, subjected to a fatigue test. The replacement of the asphalt layer by the high performance fiber concrete results in a considerable reduction of the stresses in the steel girders. Stress amplitudes of 124 MPa are reduced to 28 MPa. This means that the mass of the concrete plays an important role. Of course, the layer thickness could be smaller by increasing the strength of the concrete, but then the stress amplitudes would become proportionally larger as well. Tests have been carried out with regard to time dependent behavior of the concrete, the adhesion capacity, the frost-thaw resistance in combination with de-icing chemicals and chloride penetration. These tests confirmed the durability of the solution. Fig. 12 shows the spreading of the concrete at the site. Now, various Dutch bridges get new bridge decks in high strength fiber reinforced concrete. Another interesting application of an ultra high strength fiber reinforced concrete is the replacement of the bridge decks of the "Kaag"- Bridges in The Netherlands (Buitelaar [10],

Fig. 11. Fatigue test on a bridge deck with high performance concrete.

Fig. 12. Spreading the concrete at the site

Fig. 13 shows the assembly of the precast HPFRC bridge decks. In this case, the concrete is not only reinforced with steel fibers, but as well with mild reinforcement. As was already shown by Bache [1] in the beginning of the eighties, very dense mild reinforcement in combination with high steel fiber percentages can be a very suitable combination. The strong matrix in combination with the fibers gives an excellent composite. This effect is optimized by using small reinforcing bar diameters at small distances. This composite was called Compact Reinforced Composite (CRC). Structural members with CRC are very ductile, durable and slender. The resistance against fatigue and impact is large. The concrete mixture is a combination of CEM III 52,5, silicafume, bauxite 0-1 and 5-8mm, steel fibers 0,4x12,5 mm, superplasticizers, and an air entraining agent. The water binder ratio was 0.18. The strength class obtained with this mixture was about C180. After one day hardening the compressive strength was already 90 MPa. The bridge decks shown in Fig.

13 have a thickness of only 45 mm. They are reinforced with 3 meshes 8mm-40mm. Those three layers fit in a height of about 26mm, The concrete cover at the upper side is 9mm, at the lower side 10mm. The length of a deck element is 7,2m and the width is 3,0m. The element is supported on beams at a distance of 865mm. The specific weight of the concrete is 2850 kg/m^3. The weight of a panel per m^2 is 170 kg. Although the price of the material per m^3 is high, the solution in total is competitive because of the small amount of materials which are needed, the easy transport, the quick assembly and the high durability. The very low concrete cover is realistic because of the very large density of the material. The diffusion coefficient for chlorides is for instance a factor 50 smaller than that of a concrete C45/55 CEM III. Test on various HPFRC's, which were exposed to wet/dry cycles during a number of years at a NaCl- concentration of 3.5% have shown that the chloride penetration was restricted to the outer 1 to 2 mm, even in permanently cracked concrete.

Fig. 13. Assembly of precast bridge deck panels in reinforced ultra high strength fiber reinforced concrete (Kaag Bridges, The Netherlands, 2003)

It may be wondered whether applications of HPFRC in tunnels are possible as well. Tunnel linings are predominantly subjected to radial forces so that the concrete is generally loaded in compression. According to this argument HSFRC could be expected to be an excellent application, since the lining elements are lighter and more easily transportable and mountable, which could have a favorable effect of the speed of construction.

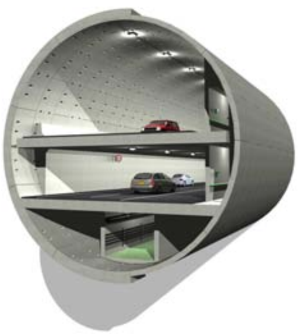

Fig. 14. Cross section of a traffic tunnel: a case for HPFRC?

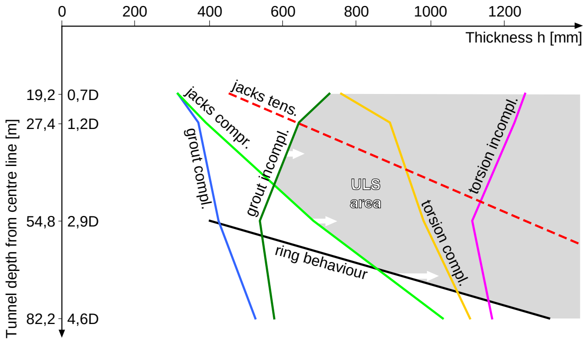

Fig. 15. Tunnel lining thickness for concrete C35/45

The minimum lining thickness, however, does not only depend on the magnitude of the radial compressive force exerted by the soil and eventual water. In general the construction stage is governing the design. Effects that play a role are splitting forces due to the jacks, compressive forces, uplift forces due to fresh grout, dimensional inaccuracies etc.

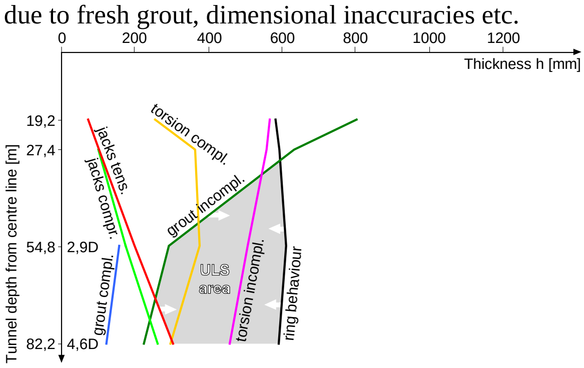

Fig. 16. Tunnel lining thickness for concrete C180/210

Taking those design criteria into account, it can be demonstrated that the use of UHPFRC can lead to much thinner linings (compare the grey areas in Fig. 15 (concrete C35/45 and Fig. 16 (concrete C180/210). In these figures the allowable lining thickness h is shown as a function of the depth of the tunnel below the surface (Groeneweg [11])

4. RECENT RESULTS OF RESEARCH AT TU DELFT

4.1 Hybrid fiber concrete

There have been fundamental discussions on the role of the fibers in FRC. In conventional reinforced fiber concrete the fibers are relatively large in comparison with the aggregate particles. The fibers are activated as soon as a major crack in the concrete occurs: by bridging the crack, fibers more or less act in the same way as reinforcing steel. In HPFRC the fibers are much finer: they are already activated when microcracks occur in the concrete. It may therefore be wondered whether those fine fibers act as reinforcement, or whether they are an integral part of the composite on a lower (micro) level. Markovic [12] combined in one mixture fine fibers with long fibers. The fine fibers react immediately on microcracking in the concrete. The further growth of those microcracks is therefore counteracted from their origin. Hence the concrete appears to stay longer in the elastic phase. When, under the influence of the increasing external load finally macrocracks occur, the long fibers are activated. Fig. 17 shows the results of a number of bending tests on short beams made of hybrid fiber concrete. Here various combinations of long (40 or 60mm) and short (13 mm) fibers have been used. Very high flexural strengths (up to 45 N/mm2) have been measured. The compressive strength of this concrete was about 120 N/mm2. It turned out that there are considerable differences between concretes with one type of fiber and concretes with combinations of fibers. Fig. 17 shows for instance that a mixture with 2 Vol.% of fibers with l =13 mm reached a flexural tensile strength of 25 N/mm2 whereas a mixture with 1 Vol.% short fibers (13mm) and 1 Vol. % long fibers (40mm) reached a flexural tensile strength of 40 N/mm^2. It can as well be seen in the diagram that 1 Vol.% of short fibers combined with 0,5 Vol.% of long fibers (40 mm) offers the same flexural tensile strength as 2 Vol.% of short fibers. This shows that by combining different types of fibers optimization of properties can be achieved.

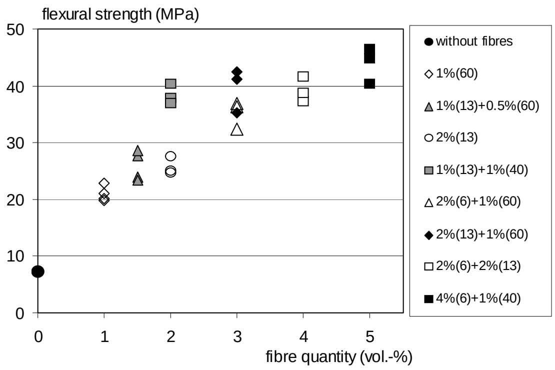

Fig. 17. Bending tensile strengths of various types of hybrid fiber concrete [12]

3.2 Fatigue

One of the advantages of HPFRC is that the material allows light structures. A consequence is, however, that fatigue – a criterion that hardly plays a significant role in massive conventional concrete structures, can now become decisive. Fatigue can occur for instance due to traffic loads (bridges) wind (off-shore wind turbines). At TU Delft a research project was carried out in which the behavior of different types of HPFRC under fatigue loading was investigated. The concrete with the highest strength was BSI/Ceracem. This concrete had a relatively large maximum coarse aggregate diameter (7mm) with 2,5 Vol.% (200 kg/m3) fibers 20/0,3 mm. The mean compressive strength was 220 MPa. Another mixture, denoted as HSFRC, was developed at TU Delft [13]. This mixture was a.o. used for the production of the prestressed sheet piles (Fig. 9). The mixture contained 1.6 Vol.% fibers with a length of 13mm and a width of 0,16 mm. The average compressive strength was 145 N/mm2. The third mixture was a hybrid mixture, with 0.5vol% short fibers (l = 13mm and d= 0,2mm) and 1 Vol.% long fibers with hooked ends (l = 60 mm, d = 0,75mm) according to Markovic [13]. The compressive strength of this concrete was about 120 N/mm2. With all mixtures beams 125 x 125 x 1000mm were made , which were subjected to four point bending. Both static and fatigue tests have been carried out. Fig. 18 shows the results of the tests under static loading, represented by the relation between calculated flexural stress at the bottom of the beam and the deflection at mid-span. The “hardening” part of the curves, which is a proof of a well-designed HPFRC is clearly visible in all curves. The results of the fatigue loading tests are represented in Fig.19. The mixture HSFRC demonstrated the best behaviour: with an upper load of 70% of the average static failure load only one of the seven beams failed within 107 cycles. For the mixtures BSI/CERACEM and the hybrid mixture an equivalent behavior was found at 60-65% of the static strength. Moreover the BSI/CERACEM, used here, showed a higher scatter. The research demonstrated, that better workability leads to smaller scatter in test results under fatigue loading.

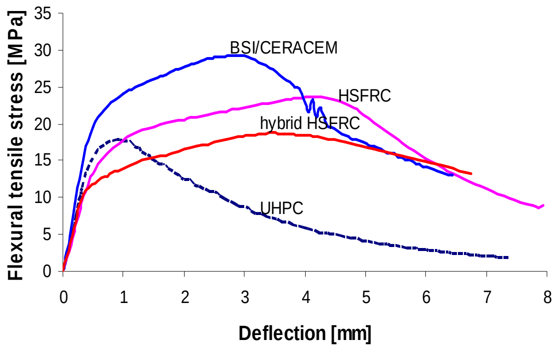

Fig. 18. Relation between bending tensile stress and deflection for three different types of HPFRC under static loading, Lappa [14].

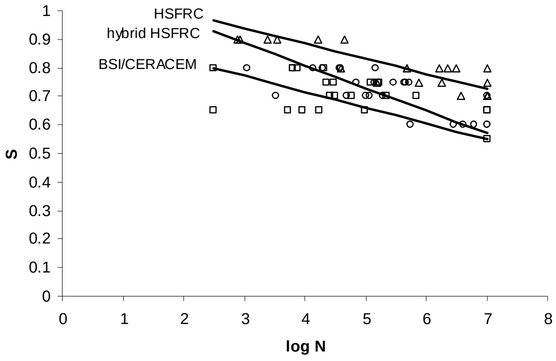

Fig. 19. Results of fatigue tests for the same three mixtures as shown in Fig. 18, Lappa 149].

3.3 Cracking behavior of HPFRC in combination with reinforcing steel.

The cost of high and ultra high strength fiber concrete is predominantly governed by the steel fibers. In this respect it is illustrating to analyze the meaning of 125 kg/m3 fibers 13/0,16 mm in an unconventional way. A simple calculation shows that 1 m^3 concrete contains 60 millions of such fibers, altogether representing a wire with a total length of 791 km. This means as well that any cm^3 contains 60 fibers. These fibers especially influence the behavior on a micro scale (counteracting the growth of microcracks). Previously, when treating hybrid fiber concrete, it was demonstrated that long fibers can be a favorable additional component, because they take over the role of the short fibers after macro cracking. Of course the task of the long fibers can be adopted by traditional reinforcing steel or prestressing steel as well. An interesting example of a combination of wire mesh and steel fibers was shown in Fig. 13, showing the placement of a deck plate made of reinforced fiber concrete. The three meshes d_s = 8mm - 40mm represent a reinforcing ratio of 8,4% which shows, in combination with the 200 kg/m^3 steel fibers 12,5/0,4 mm, a high strength, high ductility and durability. In spite of these excellent characteristics it is clear that the material can be further optimized. On the one hand this refers to the production technology, on the other hand the cracking behaviour under tension and shrinkage. Moreover the question could be raised where one finds the optimum between mixture composition and mechanical properties. In order to give a contribution to the answer on this question at TU Delft tests have been carried out on combinations of high and ultra high strength fiber concrete, provided with reinforcement in combination with various volumes and types of fibers. As a part of the research axial tension tests were conducted on reinforced prismatic bars. The concrete used had compressive strengths of about 130 and 180 N/mm^2 respectively. The volume of fibers was 0 vol. %, 0,8 Vol.% and 1,6 Vol. %, which corresponds to 0, 60 and 120 kg/m^3 steel fibers. Fig. 20 shows the crack pattern which was obtained by tensioning the prismatic bars 50x50mm, reinforced with a reinforcing bar d_s = 6mm in the centre of the cross section.

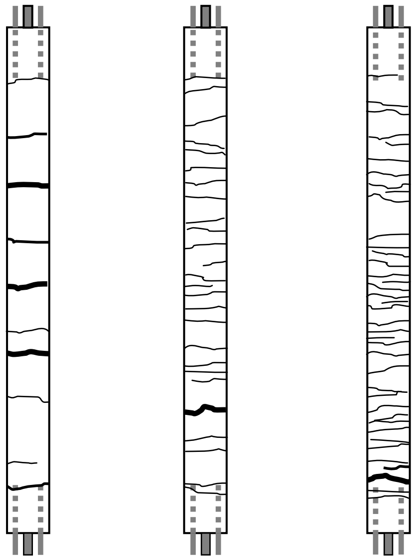

Fig. 20: Crack patterns in axially loaded reinforced concrete prisms with fiber contents (from left to right) 0, 0,8Vol.% and 1,6 Vol.%, for concrete with f’c≅130 N/mm2 (Shionaga, [15])

The results show that the number of cracks increases with increasing fiber volume. Another important conclusion is that in the bar without fibers crack localization occurs in a number of cracks, whereas in the elements with fibers localization only occurs in one crack. This is a result of the variation of the concentration and orientation of fibers in the different cross sections. Of course an important question is whether the number of cracks and their distance can be calculated with an extended version of existing code rules. To this aim the behaviour of the fiber concrete in tension was tested with centrically loaded, so called “dogbone” specimens. On the basis of the tests the response curves were simplified as shown in Fig. 21.

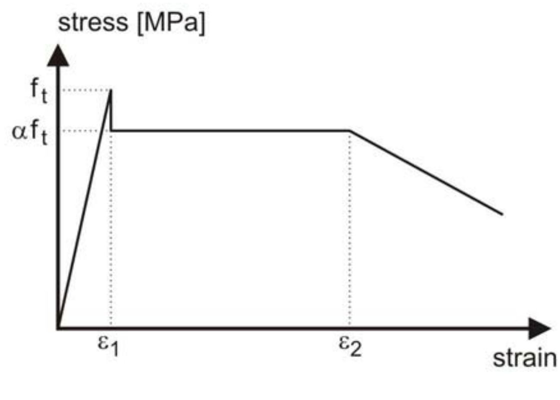

Fig. 21. Simplified stress – crack opening relation, based on tests [16].

This aimed at describing the behaviour in such a way that it represents the actual behaviour well and simultaneously offers a relation which can be used in combination with existing code rules for reinforced concrete. The relation shown in Fig. 21 consists of a linear (elastic) part until first cracking and a plastic part immediately afterwards. The plastic post-cracking tensile strength is formulated as αf_{ct}. At larger crack opening a declining branch is added, which however is often not relevant for design calculations. The simplification according to Fig. 21 turns out to be advantageous when calculating the crack width and crack distance. Because the fibers transmit stresses across a crack they reduce the length at both sides of the crack necessary to restore the undisturbed situation. This means that the mean crack distance, and as a consequence the mean crack width, will be smaller as a result of the action of the fibers. Fig. 21 gives both the tensile strength f_{ct} of the HPFRC considered, and the stress which is be transmitted across the crack αf_{ct}. With those data it is easy to derive an expression for the crack width calculation in reinforced HPFRC. Starting from the expression for reinforced plain concrete in the Model Code 90 the expression

$$w_{\max} = \frac{f_{ctm}(1-\alpha)d_s}{4\tau_{bm}\rho E_s}\left(\frac{N}{A_s} - \frac{f_{ctm}}{2\rho}(1-n\rho)(1-\alpha)\right) \quad (1)$$

for the maximum crack width is obtained, where $\alpha = \sigma_{pf}/f_{ctm}$ in which σ_{pf} = post cracking "plastic" stress by fiber action, f_{ctm} = mean tensile strength of concrete, d_s = diameter of reinforcing bar, τ_{bm} = mean bond strength, ρ = reinforcing ratio of reinforcing bars, E_s = modulus of elasticity of reinforcing steel, N = normal tensile force, A_s = area of bar cross section, $n = E_s/E_c$.
Corresponding to this, the transmission length l_t is

$$l_t = \frac{f_{ctm}(1-\alpha)\Phi}{4\tau_{bm}\omega} \quad (2)$$

The mean crack spacing s is then accordingly

$$s = 1{,}5 l_t \quad (3)$$

From centric tests on dogbone specimens for the concrete with a compressive strength of 130 N/mm^2 a centric tensile strength f_{ctm} = 5,5 N/mm^2 and a post cracking reduction factor of α = 0,72 was found. For the concrete with a compressive strength of 180 N/mm^2 a centric tensile strength f_{ctm} = 9,0 N/mm^2 and a post cracking reduction factor of α = 0,88 was found. With those values the calculated crack distances given in Table I are obtained (Yang, [16,17]). The agreement between calculated and measured values is seen to be good.

Table I: Comparison between measured and calculated crack spacings in Shionaga's[15] tests

Strength	Vf (%)	Mean crack distance measured (mm)	Mean crack distance calculated (mm)
f'c = 130 N/mm2	0	56	43
	0,8	23	21
	1,6	14	16
f'c = 180 N/mm2	0	44	58
	0,8	27	28
	1,6	26	22

3.4. Shear capacity of HPFRC

At TU Delft in 2007 a series of shear tests was varied out by Pansuk [18]. The research program contained a series of three beams according to Fig 2. The mean compressive strength of the concrete was 140 N/mm2. The concrete contained 0, 0,8 Vol.% and 1,6 Vol. % of fibers 13/0,16mm. The beams were provided with 2 bars d_s = 25mm as longitudinal reinforcement which was expected to be sufficient to avoid failure in bending. Fig. 23 shows the beams in

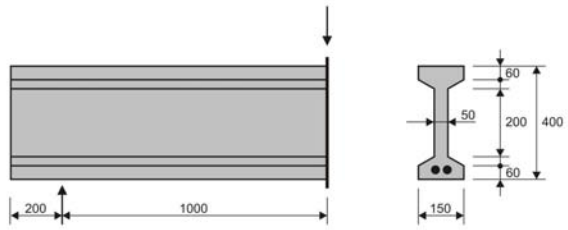

Fig. 22. Shear tests on beams made of HPFRC [18, 19]

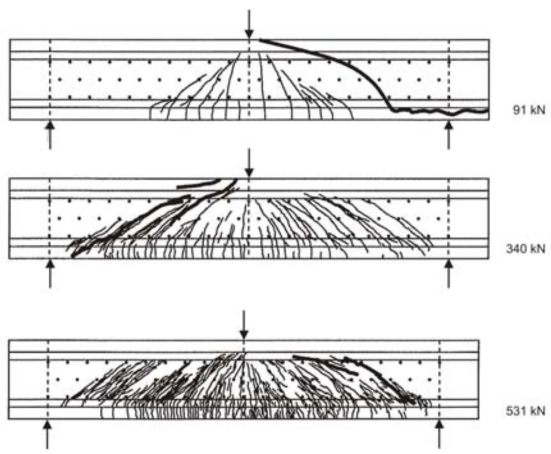

Fig. 23. Failure pattern of beams made of HPFRC with fiber volumes percentages of 0, 0.8 Vol. % and

1,6 Vol. % (from top to bottom), Pansuk [18])

the failure state. It can be seen that the fibers are adding quite substantially to the shear capacity of the beams. Also in this case existing code provisions can be used to extend their validity to HPFRC. In the Eurocode 2 the method of the variable inclination strut model to determine the shear capacity has been adopted. According to this method a strut angle can be chosen between $1 \leq \cot\theta \leq 2{,}5$ and the stirrups crossed by the inclined crack under the corresponding angle can be contribute to the shear resistance with their yielding forces . The shear capacity is then

$$V_u = b_w d \cot\vartheta \frac{A_{sw}}{t} f_y \quad (4)$$

where b_w= web width, d = effective depth of cross section, θ = strut inclination, A_{sw} = cross sectional area of stirrup, t = stirrup distance and f_y = yield strength of stirrup steel. This expression could be modified to

$$V_u = b_w h \cot\vartheta\, \sigma_{pf} \quad (5)$$

where h = full depth of section, σ_{pf} = post cracking plastic fiber strength (Fig. 21). Here the full cross sectional depth h is introduced, because the fibers contribute also below the longitudinal reinforcement level. Since the fibers are expected to add to the redistribution capacity it is assumed that the limits of strut rotation can be widened to $1 \leq \cot\theta \leq 3$. Direct tensile tests on dogbone specimens provided values of σ_{pf} = 5,6 and 9,0 N/mm^2 for fiber volumes of 0,8 and 1,6 % respectively. In combination with $\cot\theta$ = 3 the values shown in Table II are obtained.

Table II: Shear capacities from tests [18,19] in comparison with calculated values according to Eq. 4, for $\cot\theta$ = 3

	Vf	σpf (MPa)	Vu,calc (kN)	Vu,test (kN)
Mix 1	0,8%	5,6	311	340
Mix 2	1,6%	9,0	500	531

Of course these results can only be regarded as provisional and further evidence is necessary. It shows, however, that extending existing code rules to HPFRC is a promising option to be further explored.

5. ON THE WAY TO DESIGN RULES

In order to be able to quantify the mechanical properties of high performance fiber reinforced concrete (HPFRC) in order to be able to design structures, the material should be qualified by tests. In principle, the behaviour of materials subjected to tension can best be obtained by conducting an axial tensile test. However, conducting an axial tensile test is associated to a number of difficulties. First, fracture at the glued end faces of the specimen should be avoided by appropriate measures, such as tapering the specimens. Second, the result of the tests can be highly influenced by eventual load eccentricities, which are hard to avoid. The centric tensile test should therefore preferably be carried out in a highly qualified laboratory, and is not suitable for industrial use. For fiber reinforced concrete other difficulties apply. In a small cylindrical specimen there is a considerable boundary effect. The fibers tend to orient parallel to the wall of the mould and furthermore the fiber orientation is influenced by the way of casting. This latter influencing factor does not only apply to cylinders cast in a mould, but as well to cylinders sawn from a larger element like a slab.

For practical reasons a bending test on a short beam or a prism is an attractive alternative because of the relatively easy way of conducting the test. However, the scatter of such tests is large. The result is quite sensitive for the production of the concrete, the casting and the vibration. Fig 24 shows the scatter as observed in standard bending tests according to Gossla [19]. Recognizing this, RILEM defined a

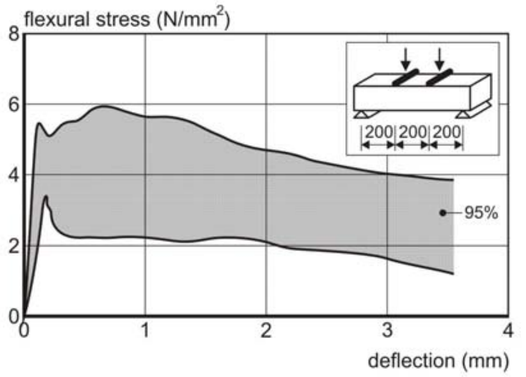

Fig. 24. Variation in load deflection relations (grey area) for 71 bending tests on FRC (Gossla [20])

standard test beam for conventional fiber reinforced concrete with a notch and prescribed exactly how to fill the mould, how to carry out mechanical compaction and how to measure the load deflection relation and load crack opening relation. Finally it is described how stress – strain relations can be derived, on the basis of reversed analysis, from the test results. Fig. 25 shows the test beam as prescribed by RILEM.

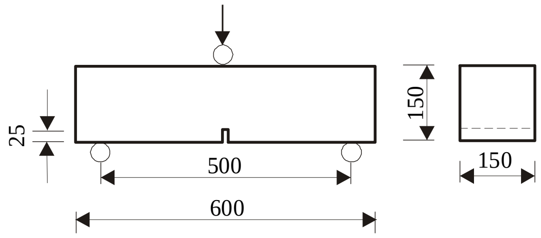

Fig. 25. RILEM standard test for conventional fiber reinforced concrete

In the French regulations for UHPFRC [21] the influence of fibre orientation in the standard test specimens and the structural elements is recognized. The solution offered is based on the introduction of two different types of tests. Here thin and thick elements are distinguished, in order to reflect as much as possible the structural behaviour of thin and thick structural members in practice:

(a) If the bending behaviour of a thin element is considered, the standard test should be carried out on a specimen where the cross-sectional height is smaller than $3l_f$, where l_f is the fiber length. By choosing a standard bending test with such a small thickness, Fig. 26, the stress – strain relation includes already somehow the effect of alignment by the boundary conditions, which is expected to occur in thin structural elements as well.

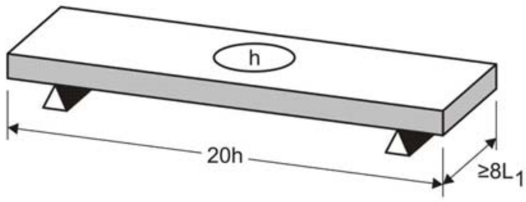

Fig.26. Standard test for thin UHPFRC members, Resplendino [20]

Anyhow this test is not considered as absolutely representative, since it is advised to apply a correction factor 1/K to the results obtained from specimens taken from the actual structural element.

(b) For the design of thicker elements ($h \geq 3l_f$) the following procedure is advised. A series of prism is cast and notched and subjected to a bending test. By inverse analysis the post-cracking stress – crack width relation (σ – w) is obtained. This relation is corrected with a factor 1/K representing the difference between the bending test on a cast prism and the actual behaviour of the structural member. To this aim prisms are sawn from a prototype of a member to be produced.

An advantage of this method is that relations are obtained which will be much alike the behaviour of the structural member. The disadvantage is that in both cases (thin and thick test elements) the standard test does not pretend to give a basic relation. The stress – crack opening curve is a general reference in relation to more detailed tests on specimens sawn from the structural element. For various types of structural elements advisory values for 1/K could be given: this needs further tests. In fact this method comes very near to "prototype testing" of complete elements.

At this moment the international fib Task Group 8.6 (UHPFRC) works on a recommendation for Ultra High Fiber Reinforced Concrete, whereas TG 8.3 works on a recommendation for conventional fiber concrete. A point to be considered is that preferably the design rules for conventional fiber reinforced concrete and ultra high performance concrete should be compatible, so that also designs with fiber concrete with intermediate strengths should be possible. The first concept of the new design code for UHPFRC is expected to appear in 2009.

6 CONCLUSIONS

(1) High performance concrete has developed in relatively short time to a material with a recognized high potential of application
(2) High performance concrete can be used with profit in combination with reinforcing steel or prestressing steel.
(3) For UHPFRC a future oriented code has to be written, which is operational at the same time. So, the code should not only regard design for ULS and SLS, but also take into account aspects like service life design and performance-based design.
(4) An important task is to get a code which is not "stand-alone", but is consistent with the codes for "classic fiber concrete". This is important since many interesting applications are in the range between conventional and ultra high performance.

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A flexible mould for double curved pre-cast concrete elements

Roel Schipper, MSc, lecturer/researcher #1 and
prof. dipl.-ing. J.N.J.A. Vambersky, chair of Building Engineering #2

Dept. of Structural and Building Engineering, Faculty of Civil Engineering and Geosciences, Delft University of Technology, P.O. Box 5048, 2600 GA, Delft, The Netherlands

[1]h.r.schipper@tudelft.nl

[2]j.n.j.a.vambersky@tudelft.nl

***Abstract*—The manufacturing of double curved precast concrete elements is still expensive, due to the high costs and limited possibilities for repetitive use of the moulds or formwork. The goal of a PhD project recently initiated at TU Delft is to develop a production method that overcomes these difficulties by enabling the mould to be used many times and by making the shape of the mould adjustable in a flexible way. This paper describes the research goal and method of the PhD project.**

I. Introduction

Freeform architecture is gradually gaining market share in everyday building practice. Until recently freeform shapes and, more in general, complex geometries were merely designed either in an academic context or in top budget buildings designed by famous architects. In Germany freeform architecture examples are for instance: Der Neue Zollhof in Düsseldorf (Frank O' Gehry Architects, 1998, see Figure 1), which is made in pre-cast concrete, the Mercedes-Benz Museum in Stuttgart (UNStudio, 2006) and BMW Welt in München (Coop Himmelblau, 2007) in cast in situ concrete. In Spain it is for instance the -in mean time famous- Guggenheim museum in Bilbao (Frank O' Gehry Architects, 1997), with the façade made in structural steel with metal sheet cladding.

Figure 1. Der Neue Zollhof, Düsseldorf: final result and the underlying pre-cast elements

In the past decade, a large number of freeform architecture projects have won design competitions worldwide [4],[5] and some of them have already been built. The possibilities of freeform shapes have apparently opened the eyes of a new generation of young architects, and a steady movement is now visible from using freeform only in high profile architecture, such as museums and corporate headquarters, towards using it also in more common buildings. Freeform is slowly becoming adopted as one of the architectural streams or shape languages, as a natural choice among other existing possibilities.

Figure 2. Heydar Aliyev Cultural Centre, Baku (Zaha Hadid Architects, under construction)

II. Pre-cast concrete and freeform architecture

Prefabricated concrete elements stand for a growing market share in the total European building construction volume, both in high and low rise buildings. Pre-cast concrete elements have the advantage of combining high quality with relatively high strength and stiffness and excellent heat accumulation properties. Usually, these pre-cast concrete elements are flat or prismatic in shape, that is, without curves, and the repetition of these elements (achieved mostly by the use of the "master mould" principle in the manufacturing) makes it possible to reuse a single mould many times during the same project. The pre-cast concrete technology is an almost perfect construction technology for freeform architecture. Pre-cast concrete offers all of the desired qualities required in our (environment conscious) human society: production by skilled workers in labor-friendly off-site environment, freedom of shape, high aesthetic value, high load bearing strength, small size tolerances and safe and fast assembly on-site.

Due to the complexity of the freeform shapes, however, it is often not possible to distinguish any repetitive elements in the building at all [3]. This is illustrated by the analysis of elements in the roof of a freeform building in 6 on the following page. Furthermore, most freeform building designs consist of many single or double curved surfaces which require complex shapes for the mould and element edges [6]. Since the costs of the moulds (formwork) for precast concrete make up a significant percentage of the final price per element or per square meter, the feasibility of freeform buildings in precast concrete is still far from optimal. The PhD research carried out at Delft University of Technology, Faculty of Civil Engineering, aims to improve the applicability of precast concrete through the development of a flexible mould.

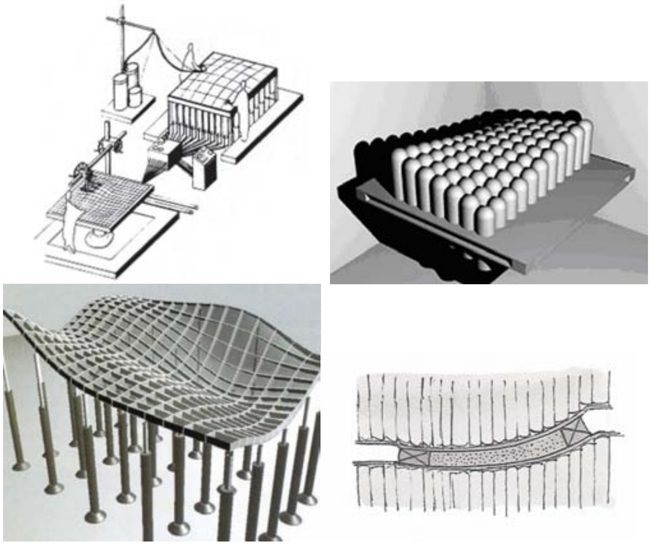

Figure 3. Several flexible mould sketches by (clockwise starting top left) Renzo Piano, Hans Jansen, Van Roosbroeck, Lars Spuybroek

III. Earlier research

Some attempts have already been made to design a flexible mould, see a.o. [7] and [1]. Mostly, those designs of a mould system were still rather conceptual (see (3)), not yet addressing all practical difficulties that occur if one tries to build the concept in reality. Dr. Vollers and ir. Rietbergen from Delft University of Technology have been working on a material-independent prototype for a flexible mould. This patented system is based on the principle of deformation of an originally flat sheet of any material into a double curved surface [8]. The application of this principle on the material concrete in 2009 by two Master students in the Stevin Lab in Delft (see Figure 4) has led to interesting observations and a better insight in the questions that have to be solved [2] . A number of experiments have been performed using a prototype for a flexible mould built earlier by Rietbergen. In these experiments the complete process starting from CAD geometry and ending with the final concrete element has been covered, revealing many difficulties and open ended research questions. The experiments helped to formulate and clarify the following research goal.

Figure 4. Experiments in the Stevin Lab with double curved precast concrete based on the patented system [8]

IV. Research goal

A production method is to be developed that overcomes the difficulties caused by the curved shapes and lack of repetition. This implies that the method should solve two obvious issues:

1) If the repetition can't be found in the building geometry, than the mould itself needs to be suitable for multiple shapes: a reusable, flexible and adjustable mould.
2) Since curvature is one of the main characteristics of freeform architecture, the mould needs to enable the manufacturing of (double) curved elements.

Based on the earlier researches and experiments the objective of the PhD project is to push the development of the flexible mould concept for the material concrete from theoretical concept towards a working industrial prototype: the necessary developments and open ended research questions have been identified. At the end of the research the design specifications of vital parts of the flexible mould should be described, and supported by theoretical and -if possible also- experimental study as a proof of concept. A number of practical and more fundamental questions, however, have to be solved first, as described in the following paragraph.

V. Practical and fundamental questions

By examining the results of the earlier experiments in the Stevin Lab, a number of important questions have been identified, some more practical, others of a more fundamental character. These questions will have to be answered first in order to determine what developments are necessary and which boundary conditions should be chosen to design a useful and working manufacturing process:

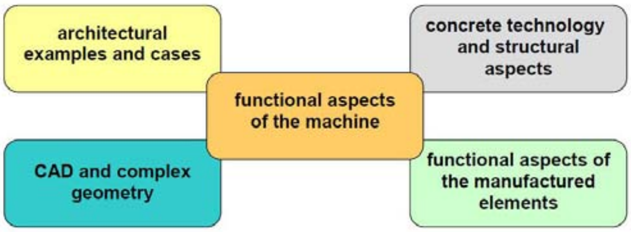

Figure 5. Simplified research map

- Which manufacturing options for making Double Curved Concrete (DCC) elements are available other than using a flexible mould? What other types of concrete panel techniques can be distinguished in current practice?
- Which obstacles are standing between the theoretical concept of a flexible mould for concrete and the practical implementation of the concept in a working prototype?
- What specific properties would make a flexible mould suitable for the industrial manufacturing of prefabricated DCC elements?
- What properties do the desired prefab DCC elements have or need in terms of size, weight, looks, application field, texture, color, curvature, structural behaviour?
- What type of concrete mixture is suitable for application in the flexible mould?
- In which applications is it necessary to reinforce elements? Which materials can be used for reinforcement of DCC elements?
- How to deal with the link between complex geometry in CAD models and an automated production process?
- How to deal with mechanical aspects and practical difficulties with the machinery that were exposed by the earlier experiments?

The research has been divided in five main aspects (see Figure 5): As can be seen, the development of a functional machine is depending on a number of aspects: in the first place a number of recent examples of freeform architecture will be analyzed in order to make sure that the developed solution will be practically applicable. Secondly, the more mathematical side of the process will be addressed by looking at data exchange between CAD and machine, and also at the way complex geometry can be broken down into elements that can be manufactured and assembled. Furthermore of course the aspect of concrete mixtures and reinforcement should be taken into consideration. And at last, also the functional aspects of the manufactured elements form a boundary condition in the development of any machine. Although a number of aspects are of a more theoretical character and need serious desk study, also the more practical approach (“just start and do it”) has proved very useful and inspiring in the earlier experiments. A number of new lab experiments is foreseen, as described in the following paragraph.

VI. Planned experiments

The experiments will be centered around the five themes shown in (5). Since the Stevin Lab offers excellent facilities for composing and casting different concrete mixtures, and the production process in mind requires a very specific constitution, this part of the research will start in the coming months. Some existing architectural CAD models have already been examined in order to, for example, determine the desired element size and curvature (See Figure 6). For now, the research

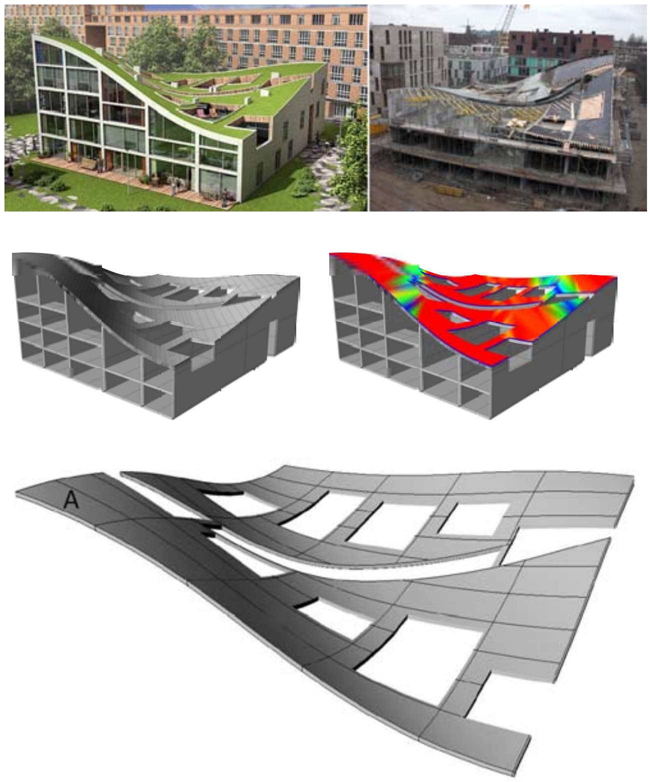

Figure 6. Analysis of a CAD model of a residential building in Het Funen in Amsterdam (NLArchitects) - the project has been realized last year using cast in site concrete

will focus on structural elements with a load bearing function, such as structural façade elements or roof elements. During the initial lab experiments the right flexibility and material for the mould will be examined using smaller test setups. For the development of a full scale prototype industrial partners will be invited to participate as soon as some tangible results follow from the theoretical study and first lab experiments. The research has to come to a final result within the coming three years.

VII. Conclusion

The PhD research described in this paper is aiming for the development of a working industrial prototype for a flexible concrete mould. This flexible mould will be able to produce double curved concrete elements that can be applied for realizing freeform architecture projects. The research is now in the stage that practical experiments will be carried out in the Stevin Lab in Delft in order to determine the right concrete mixtures and mould material. Parallel a number of more theoretical aspects will be taken into consideration. In Precast2012 we hope to report on the results.

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Structural connections in precast concrete

Björn Engström

Department of Civil and Environmental Engineering, Chalmers University of Technology
SE-412 96 Göteborg, Sweden
bjorn.engstrom@chalmers.se

***Abstract*— Proper design of structural connections for precast concrete buildings must be based on a deep understanding of the role of the connection in the structural system, the flow of forces through the connections, and basic force transfer mechanisms. Furthermore, needs with regard to functionality, simplicity, production, and easy assembly should be considered.**

I. General Considerations and Design Philosophy

The main purpose of the structural connections is to transfer forces between the prefabricated concrete elements when the structural system is loaded. By the ability to transfer forces, the connections should secure the intended structural behaviour of the superstructure and the prefabricated subsystems that are integrated in it.

To reach a proper design of the structural connections, the designer should understand how the connections are parts of the overall system and be aware of the flow of forces through the structure as well as through the connections. The structural layout, the arrangement of stabilising units, the design of the structural system and its sub-systems, and the design and detailing of the structural connections must be made consistently and with awareness of the intended structural behaviour, Fig. 1.

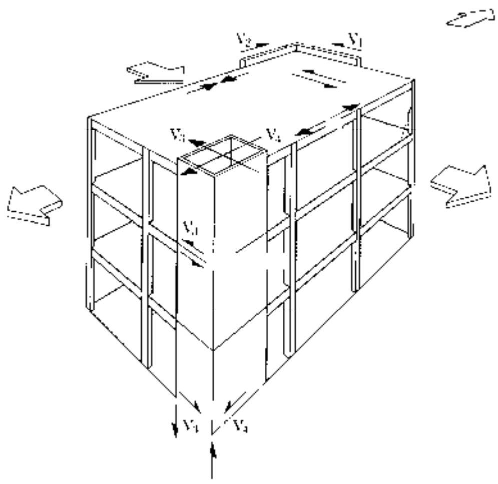

Fig. 1 The connections should be designed consistently with the intended structural behaviour

The design of the structural connections is not just a question of selecting appropriate dimensions of the connection devises, but the force path through the connection must be considered in a global view of the whole connection including the end regions of the adjacent structural members. Therefore, the connections and the elements must be designed and detailed as a unity where the flow of forces is logic and natural so that the forces that are to be resisted by the connection can be transferred into the elements and further on to the overall load-resisting system. The force transfer from tying devices, support bearings etc. into the adjacent prefabricated concrete elements must be secured by a proper design and detailing of these connection zones, Fig. 2. Hence, it may be necessary to design and reinforce the connection zones with regard to the action of concentrated forces and the corresponding risk of cracks.

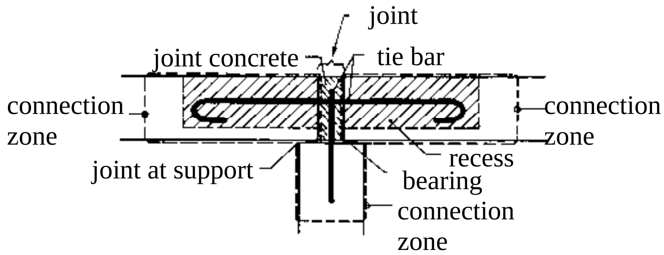

Fig. 2 A structural connection consists normally of several components. The structural behaviour and the performance of the connection depend on the interaction between these components

The various aspects that should be considered in the design and detailing of structural connections can be related to the following groups:

– the structural behaviour for ordinary and excessive loads
– the performance and appearance of the building in the service state
– structural integrity in case of fire and accidental actions
– production of the concrete elements
– handling, storage, and transportation of the concrete elements
– mounting of the prefabricated structural system

Other key aspects in the design of structural connections are simplicity, standardisation, durability, and aesthetics. Much of the advantages of precast concrete construction are due to the possibility of fast erection of the structure. To fully realise this benefit and to keep the costs within reasonable limits, the connections should be kept simple.

II. Basic Force Transfer Mechanisms and Mechanical Behaviour

When the behaviour of connections is studied more in detail, it is found that there are some rather few basic force transferring mechanisms that appear quite often in many different types of connections. A deep understanding of those basic mechanisms, of the flow of forces through the connections and of the role of the connections in the overall structural system forms the basis of proper design of structural connections.

A. Transfer of Compression

Structural connections and connection zones of precast concrete elements are often subjected to high concentrated compressive forces, Fig. 3. When these forces are transferred through the connection and further into the adjacent elements, they are spread into wider stress distributions.

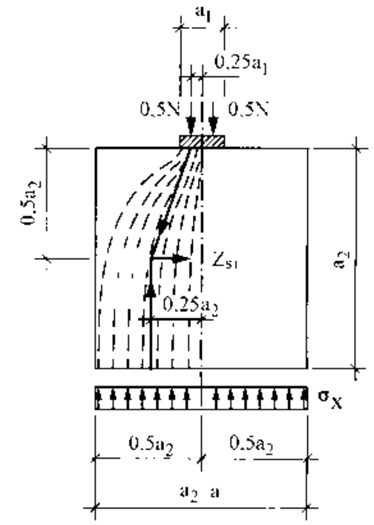

Fig. 3 Transfer of compression, local compression

The deviation of forces (i.e. change of directions) and spread of stresses might lead to high transverse stresses. If the concrete tensile strength is reached, cracks will appear in these zones. In case of improper detailing, these cracks might result in damage, which in turn might limit the capacity of the connection, for instance due to splitting failure in a support region. The 'strut and tie method' is an appropriate tool to design the connection zones and check equilibrium in the ultimate limit state. This method also reveals the flow of forces through the structural connection and, thus, helps the designer to understand the behaviour and find a proper detailing, which is consistent with the intended behaviour.

B. Transfer of Shear

Dowel action of partly embedded steel bars is a basic mechanism in the transfer of shear force. The simplest case is a bar embedded at one end and loaded by a shear force acting along the joint face or at some distance from the joint face, Fig. 4. This load case will give rise to a highly concentrated reaction in the concrete beside the dowel pin. Depending on the strengths and dimensions of the steel bar and the position of the bar relative to the element boundaries, several failure modes are possible. A weak bar in a strong concrete element might fail in shear of the bar itself. A strong steel bar in a weak element or placed with small concrete cover will more naturally result in splitting of the element itself. However, when the splitting effects are controlled by properly designed splitting reinforcement, the dowel pin will normally fail in bending by formation of a plastic hinge in the steel bar. For this bending failure, in which the steel bar yields and the concrete beside the bar deforms in a plastic mode, an eccentric application of the load reduces the shear capacity significantly and should be avoided when possible.

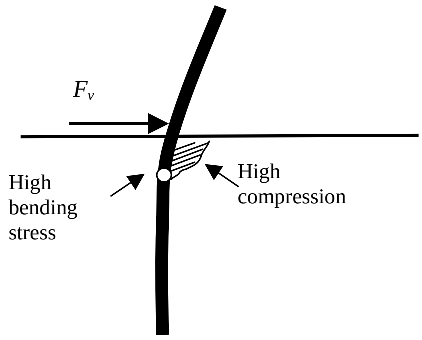

Fig. 4 Transfer of shear by dowel action

Another basic mechanism for shear transfer is frictional resistance at joint interfaces. The roughness of joint faces varies but can be controlled by treatment of the fresh concrete. Joint faces can be classified with regard to its natural roughness, roughness after special treatment or even specially formed shear keys. Under the condition that normal compressive stresses are present at the joint interface or will be generated with increasing shear slip along the joint interface, a shear resistance is possible by friction. The mechanism is principally the same as the so-called aggregate interlock.

When under shear loading slip develops along the joint interface, this will be associated with a certain joint separation, because of the irregularities of the rough joint face. If the joint interface is crossed by steel bars that are well anchored on each side of the joint, the steel bars will be tensioned due to this joint separation, Fig. 5. The tensile force in the bars must be balanced by compression at the joint interface. Thus, the wedging caused by the irregularities generates a compression force that makes shear transfer by friction possible, even in the case when there is no compression at the interface initially. The force that develops in the transverse steel bars for a certain joint separation depends on the resistance of the bar to bar pullout, Fig. 6. Here the way the bar is anchored is essential. To achieve a high self-generated compression with as little steel amount as possible, the steel should be forced to yield for a very small joint separation.

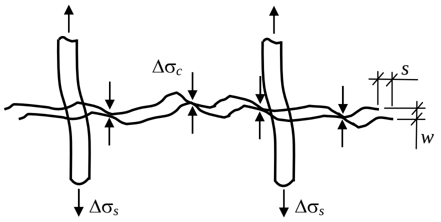

Fig. 5 Transfer of shear by friction due to the pullout resistance of reinforcement bars across rough joint interfaces

C. Transfer of Tension

For precast structures there must be a certain effort undertaken to ensure sufficient structural continuity and structural integrity. The connections act as bridging links between the precast elements. In this respect the ability to transfer tensile forces between elements is essential and tying systems should be arranged to meet these requirements. Furthermore, as mentioned previously, transverse reinforcement is also needed across shear joints to enable self-generated compression and to balance the transverse component of inclined compression forces in struts. Tie bars are also used to anchor welding plates in concrete elements.

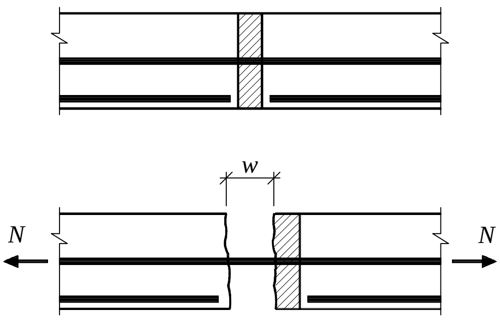

Fig. 6 Transfer of tension, pullout resistance of tie bar across a joint

In many cases connections consist of many components that act together as links of a chain. All the links contribute to the global load-displacement relationship of the composite connection. However, when a ductile behaviour is required from such a connection, it must be designed in a balanced way. Some of the components can be identified as ductile components, while the others have a more brittle behaviour. The aim of a balanced design for ductility is to secure that the full deformation capacity of the ductile links can be utilised. Brittle failures of the other components should be prevented before the full plastic deformation is obtained in the ductile ones. Hence, the other links should be designed to resist not only the yield capacity, but also the ultimate capacity of the ductile ones with a sufficient margin.

III. FIB BULLETIN ON STRUCTURAL CONNECTIONS

In order to encourage good and innovative design of structural connections in prefabricated concrete structures, the fib Commission on Prefabrication has prepared a design guide, which has been published as a fib Bulletin [1]. This guide gives basic understanding of how structural connections are parts of the overall structural system and prefabricated subsystems like floors, walls and frames. In this respect it is shown how the design of structural connections is influenced by the design philosophy of prefabricated buildings. Furthermore, the guide expresses a sound design philosophy for the connections as such and gives understanding of basic force transfer mechanisms in structural connections.

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Bijzondere gevels integraal ontwerpen als sandwich

Door ir. Rob Huijben, directeur Hurks delphi engineering bv,
constructief ontwerper

Hurks delphi engineering bv, Postbus 221, 5500 AE Veldhoven

r.huijben@hurksdelphi-engineering.nl

I. INLEIDING

Bovenstaande titel heeft in feite twee uitdagingen in zich. Het betreffen sandwich en bijzonder. Het ontwerpen met sandwich is een stuk lastiger dan het niet gecombineerd ontwerpen, omdat het buitenblad immers gelijktijdig met het constructieve binnenblad in één keer als prefabelement in de bouw geassembleerd wordt. Met integraal ontwerpen, dus ontwerpen waarbij je let op constructieve, esthetische, bouwkundige en bouwmethode aspecten, geef je hieraan invulling.

Fig. 1 Project Oosterbaken Hoogvliet Rotterdam, Architect: KuiperCompagnons – voorbeeld van een gevelblad in architectonisch beton

Bouwefficiëntie is de zakelijke reden om voor sandwich te kiezen. Maar ook vormt het voor ontwerpers de ultieme uitdaging om grenzen te verleggen en juist met sandwich, gevels te maken die anders niet haalbaar worden geacht gegeven financiële danwel technische overwegingen.

Een complete bouwstroom vervalt door deze aanpak, omdat het separaat plaatsen van kozijnen, isoleren en het aanbrengen van het buitenblad niet meer benodigd is. Een bijkomend voordeel van de bouwwijze is dat steigerloos gewerkt kan worden en dat een stuk organisatie op de bouwplaats niet meer nodig is. Dat geeft rust en overzicht voor overblijvende hoofdactiviteiten, die met de juiste aandacht zonder allerlei afleidingen kunnen worden uitgevoerd.

Logistiek gezien functioneren grote wandelementen als efficiënt transportmiddel van allerlei kleinere bouwdelen, die met zijn allen nota bene onderdeel uitmaken van een tijdens de bouwfase moeilijk bereikbare gevel. Aan- en afloop van uitvoerend personeel in grote gebouwen wordt hiermee fors teruggebracht. Het zou interessant zijn om na te gaan hoeveel kilometer aan geloop, het aantal liftbewegingen en wachtmomenten hiermee bespaard worden.

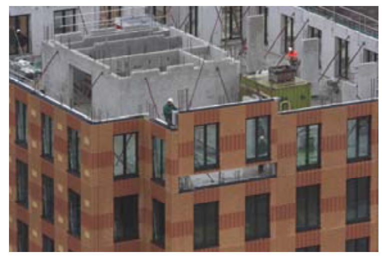


Fig. 2 Dubbeltoren Symphony Amsterdam.
Architect: de Architecten Cie
Homogene prefabmetselwerk gevels

Voordat ik verder vertel beschrijf ik eerst de sandwichelementfabricage, zoals we dat momenteel kennen. Het geheel wordt vervaardigd op een maltafel, waarbij gestart wordt met het buitenblad bestaande uit architectonisch beton, baksteen of een mix van beide. Vervolgens worden thermische isolatie en de stelkozijnen voorzien van bouwkundige slabben aangebracht en daarop wordt het betonnen binnenblad gemaakt. Verankeringen worden door het opeenvolgend storten van beton verankerd.

Als gevels van volledige prefab casco's uitgevoerd worden met sandwichelementen kan er natuurlijk vlot gebouwd worden, omdat het betreffende bouwwerk snel wind- en waterdicht te maken is mede doordat beglaasde kozijnen op het tasveld reeds kunnen worden aangebracht.

Ook speelt een volledig prefab casco goed in op een hoge maatnauwkeurigheid, die voor bouwen met sandwich benodigd is. De elementen moeten immers zodanig gesteld worden dat direct de juiste voegbreedte in het buitenblad wordt gerealiseerd.

Sandwichelementen worden ook toegepast bij bouwwerken waarvan het casco bestaat uit insitubeton en semi prefabbeton. Een voorbeeld is het Jeroen Bosch ziekenhuis in Den Bosch met een projectomvang van 140.000 m^2 vloeroppervlak, waarbij de langsgevels in sandwich zijn uitgevoerd niet zozeer vanwege bouwsnelheid, maar eerder om de omvang van de bouwactiviteit op de bouwplaats te beperken en hierdoor de bouworganisatie te ontlasten. Bij dit project zijn betonnen bekistingsplaatvloeren toegepast en betreft dus een semi prefab bouwsysteem.

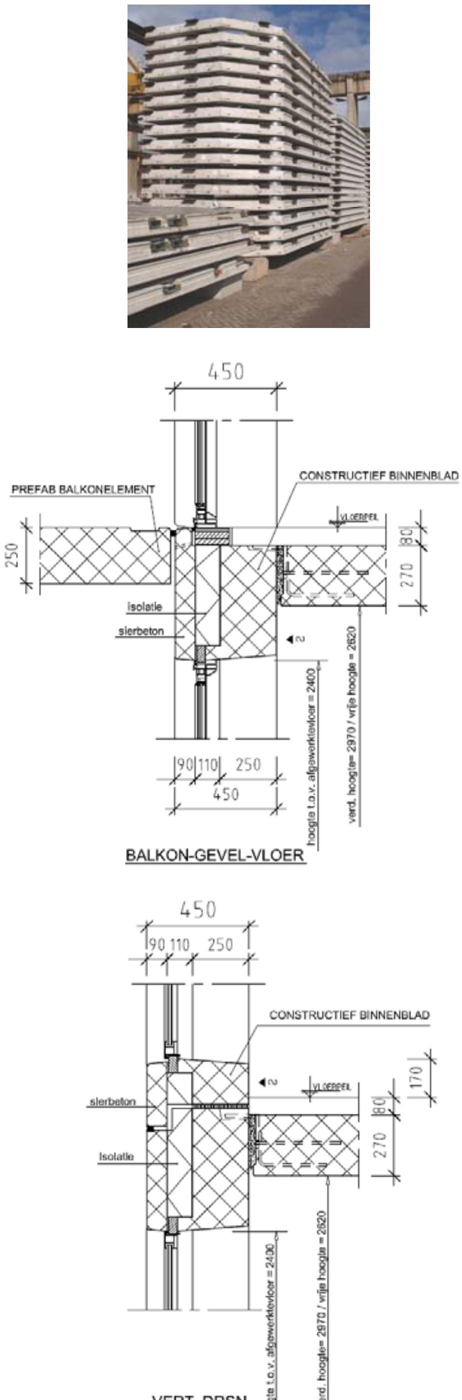


Fig. 3 Voorbeelddetails van een woongebouw en prefab volvloeren inclusief leidingen voorzien van stalen handen

In combinatie met een meer insitu betoncasco zal er meer aandacht nodig zijn voor maatnauwkeurigheid en ongewenste vervuiling van het sandwichelement ten gevolge van ruwbouw activiteiten. Een sandwichelement kan eventueel achteraf aangebracht worden, zodat ruwbouw en afbouw net zoals in de traditionele bouw weer uit elkaar getrokken worden. Dit gaat natuurlijk wel ten koste van bouwsnelheid.

II. Interactie, voegen en vormgeving

Constructie en gevelafbouw dienen gecombineerd te worden. Dit houdt onder meer in dat naden en voegen van binnen- en buitenbladen op elkaar afgestemd moeten worden. Gevelbladen van architectonisch beton worden gangbaar zonder luchtspouw uitgevoerd, zijn met betonstaal gewapend en hebben veelal een minimumdikte van 90 mm.

De gevelbladen worden bijvoorkeur maximaal h.o.h. 5,5 meter voorzien van een voeg in verband met het vermijden van scheurvorming. De voegbreedte tussen de elementen bedraagt 12 tot 24 mm afhankelijk van de werkende buitenbladbreedte en elementkleur. Voegen worden meestal in kleur afgekit en worden eventueel in combinatie met schijnvoegen geaccepteerd, maar ook gemaskeerd door elementsprongen aan te brengen. Maskeren kan ook door de voegomvang beperkt te houden en qua kleur aan te passen op de naastliggende gevelvlakken.

Sinds enige jaren worden sandwichelementen voorzien van een bakstenen gevelblad voor omvangrijke gebouwen toegepast. Een afstand tussen verticale voegen van bijvoorbeeld 7,5 meter in het bakstenen buitenblad is mogelijk op basis van de beperkte thermische uitzettingscoëfficiënt van het materiaal in vergelijking met beton, betreffende opeenvolgend 0,6- en $1{,}0 \cdot 10^{-5}$ per graad temperatuurverschil. Voegafmetingen van 12 tot 15 mm worden gehanteerd en zijn daarmee even groot als de overige voegen. Bij baksteen is het wenselijk om een spouw aan te brengen, omdat het baksteenblad inclusief voegvullingen, die uit een schuimband bestaan, onvoldoende vochtdicht zijn.

Op basis van een nauwkeurige productie en montage en diverse metselwerkpatronen blijken voegen nauwelijks te spreken en kunnen visueel grote homogene vlakken gecreëerd worden. Baksteengevels kunnen ook met baksteenstrips vervaardigd worden, die verlijmd zijn op een betonnen achterblad. Deze methode stip ik alleen maar even aan.

Vervolgens verdiepen we ons in enkele bouwkundige details. Een kozijn wordt bijvoorkeur gegeven wind- en waterdichtheid bevestigd op een door één element omkaderde opening. Bij balkonsituaties is dat niet mogelijk en zal aandacht besteed dienen te worden aan een goede onderaansluiting. Bijgevoegd zijn enkele details van een appartementengebouw die dit verduidelijken. Ook de achterliggende constructie stelt zijn eisen om zich te samen met het gevelblad als sandwich te laten vormgeven.

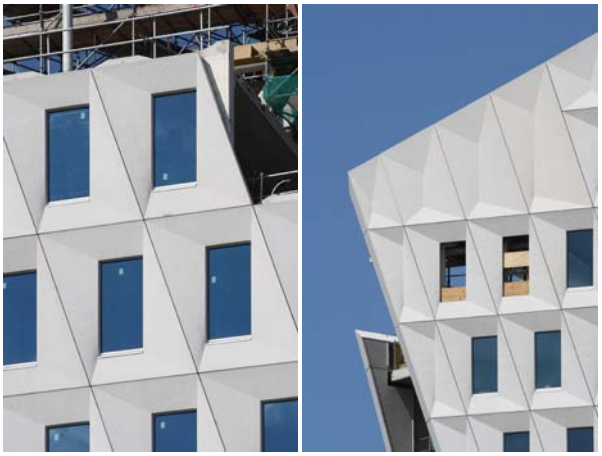


Fig. 4 Stadskantoor Leyweg Den Haag, diverse gevelfragmenten tijdens montage

III. SLANKE SANDWICHCONSTRUCTIES

Vooral bij hoge gebouwen, waarbij het binnenblad veelal een dragende en een gebouwstabiliserende functie heeft, bepaalt de vormgeving van de vloer-wandverbinding in belangrijke mate de draagkracht van de constructieve gevelwand. Voor hoge woongebouwen is de afgelopen 12 jaar deze verbinding sterk geëvolueerd Eerst was het gebruikelijk om de prefabvloerelementen middels nokken als een ligger op twee steunpunten zonder inklemming op te leggen. Dit vergde (extra) sparingen en veelal capaciteitsverlies van het wanddraagvermogen. Sparingen en nokken als constructieve verbinding bleken ook te dominant te zijn in combinatie met allerlei installatievoorzieningen.

Vervolgens zijn andere verbindingen ontwikkeld, die inmiddels grootschalig en naar tevredenheid zijn toegepast. Een prefabvloerelement wordt daarbij nog steeds gezien als een ligger op twee steunpunten en wordt vaak voorgespannen. De oplegging wordt gecreëerd door verborgen stalen handen, die plastisch kunnen vervormen en enige rotatie kunnen opnemen. De wand wordt op deze wijze niet meer deels onderbroken, zodat de hoge prefabbetonsterkteklasse volledig benut kan worden.

Voorts worden wandelementen ter plaatse van verticale voegen hoogwaardig schuifvast gekoppeld door ze per verdieping te vertanden of door ze te plaatsen in een verband. Bij deze laatste methode worden omlopende schuifkrachten bij de verticale voegen over een grotere maat verplaatst. Een constructeur zal de wijze van vertanden van geval tot geval dienen af te wegen.

IV. VOORBEELDPROJECTEN

Afsluitend licht ik een tweetal projecten toe, die momenteel respectievelijk in uitvoering en in productie zijn bij Hurks beton en Hurks oosthoek kemper.

Allereerst het Stadskantoor Leyweg, Den Haag. Een project dat door architect Rudy Uytenhaak is ontworpen en op deze dag ook door hem wordt toegelicht.

Het ontwerp van de dragende en naar boven toe uitwaaierende gevels is in een aanvullend ontwerptraject door ons bureau omgezet naar sandwich. Dit werd eerst technisch onhaalbaar geacht gegeven het dambordpatroon van raamsparingen. Onder elke raamsparing is namelijk sprake van een soort tweepaalspoerconstructie, om hiermee de penantkracht, die onder in het gebouw natuurlijk alleen maar groter wordt, telkens weer te verslepen.

In prefab bleek dit mogelijk te zijn door te werken met een vertande horizontale voeg. Op deze wijze worden extreem hoge schuifspanningen in mortelvoegen voorkomen en kunnen resulterende drukspanningen beheerst opgenomen worden. Het betreft hier een bijzondere sandwichgevelconstructie, die complex van vorm en constructie is. Bijgevoegde beelden zijn denk ik meer accuraat in informatieoverdracht dan veel meer woorden.

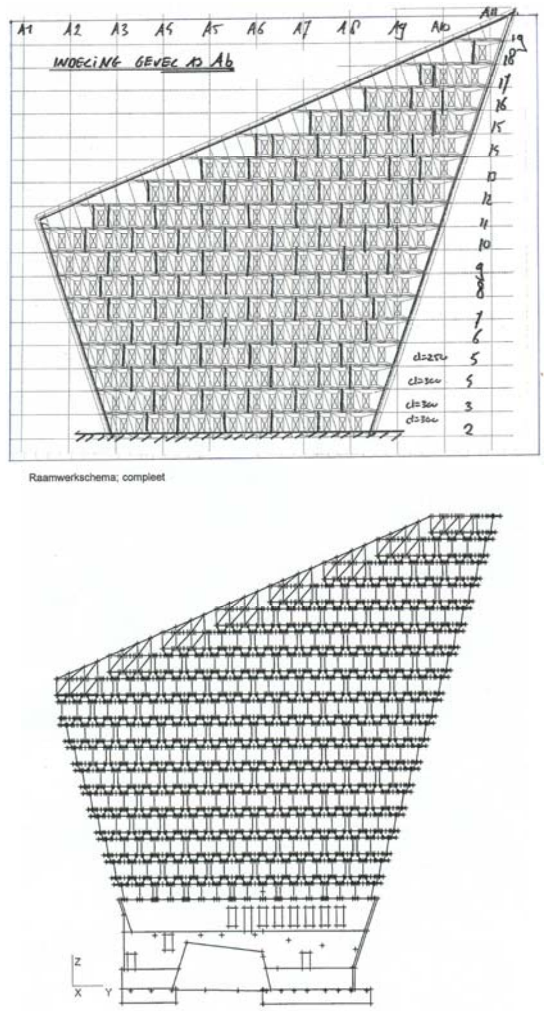


Fig. 5a Stadskantoor Leyweg Den Haag. Het betreffen een gevelaanzicht voorzien van een elementindeling en een rekenschema van de gehele dragende gevel

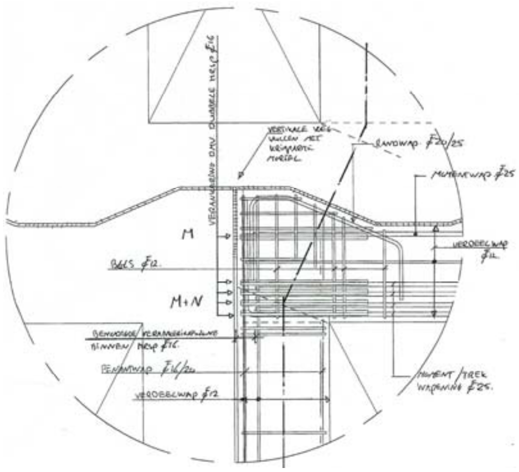


Fig. 5b Stadskantoor Leyweg Den Haag.
Knooppunt voorzien van wapening.

Het tweede project betreft Erasmus MC Rotterdam. Dit project wordt vandaag ook vanuit de bouworganisatorische kant besproken.

De hoogbouw kent een hoogte van 120 m. Het project is ontworpen door EGM architecten in samenspraak met Aronsohn Constructies raadgevende ingenieurs.

De stabiliteit van de hoogbouw en het dragen van de vloeren ter plaatse van de gevels wordt verzorgd door een gevelbuis van sandwichelementen. De constructieve binnenbladen zijn samengesteld tot een gevelbuis door ze in een "half steensverband" te plaatsen. De hoogbouw bestaat vanaf niveau 4 uit een volledig prefabbetoncasco, waarvan de engineering door ons bureau verzorgd wordt. Voor de bepaling van de wapening van de gevelbuis en uitwerking van de constructieve verbindingen worden krachtwerkingen grondig geanalyseerd. Voor dit project is ervoor gekozen om dit uit te voeren met een 3D eindige-elementen computerprogramma. Met dit rekenpakket had Aronsohn ook zijn ontwerp getoetst, zodat samenwerking optimaal gestalte kon worden gegeven.

Fig. 6 Erasmus MC Rotterdam, een element
dat gepolijst wordt en elementen op het tasveld.

V. Tot slot

Hiermee is op hoofdlijnen een overzicht gegeven van diverse ontwerpaspecten en overwegingen. Tot slot wens ik u veel goede sandwichervaringen toe.

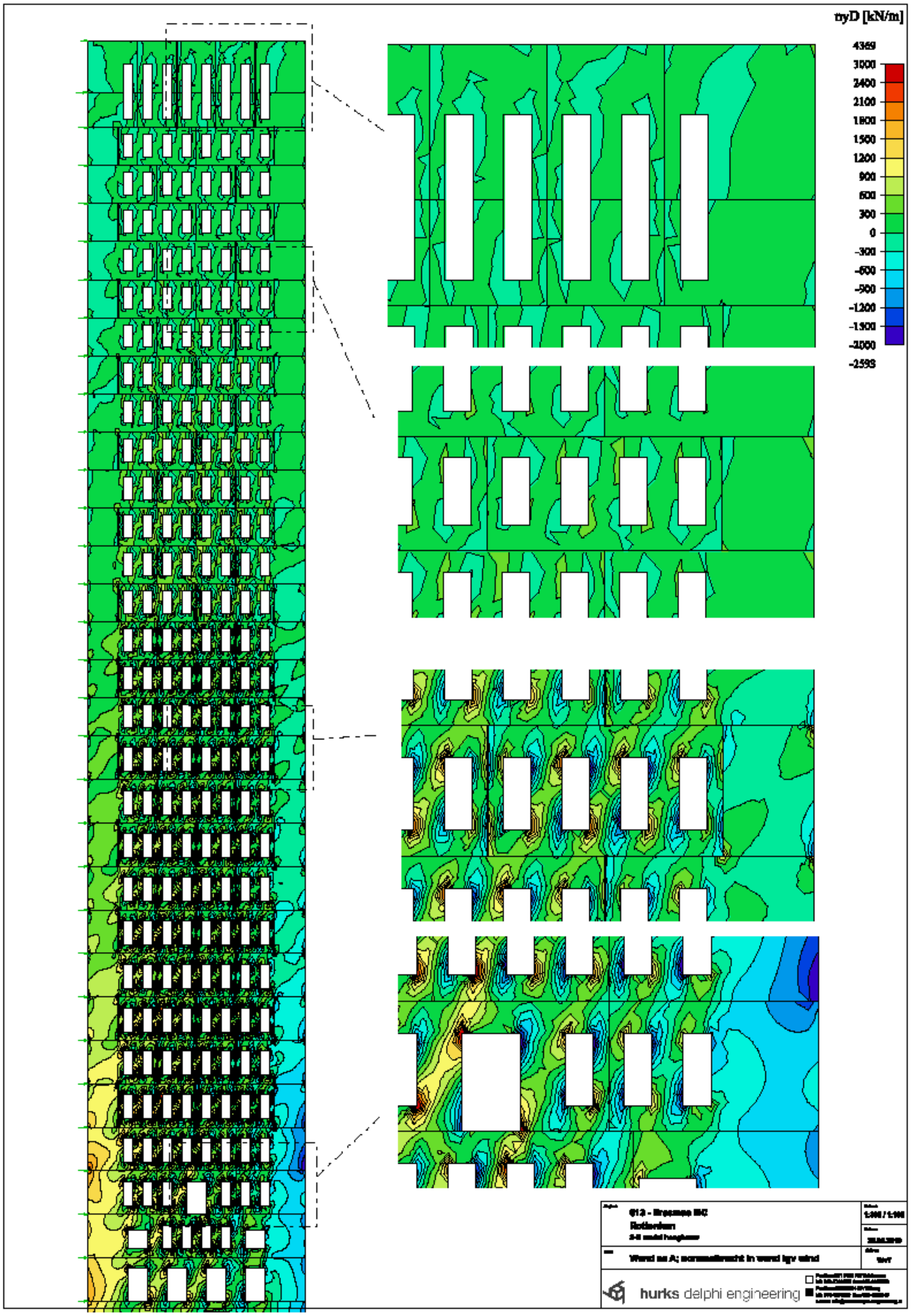


Fig. 7 Erasmus MC Rotterdam. Eerste van een tweetal visualisaties van een EEM-berekening van de 120 meter hoge kopwand van de gevelbuis. Het verloop van verticale spanningen ten gevolge van alleen windbelasting wordt op een tweetal wijzen inzichtelijk gemaakt.

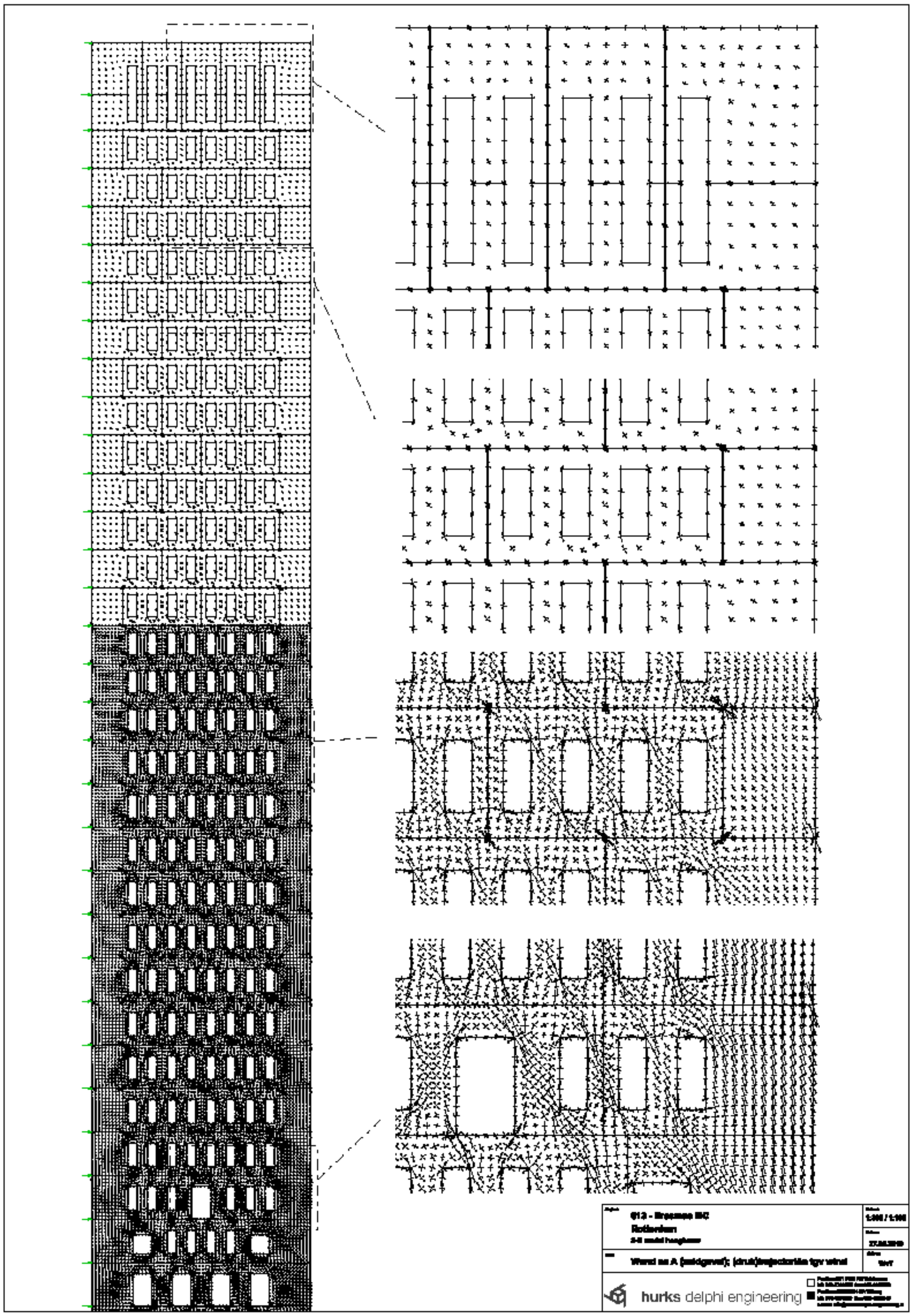


Fig. 8 Erasmus MC Rotterdam. Laatste van een tweetal visualisaties van een EEM-berekening van de 120 meter hoge kopwand van de gevelbuis. Het verloop van verticale spanningen ten gevolge van alleen windbelasting wordt op een tweetal wijzen inzichtelijk gemaakt.

Geprefabriceerde Hoogbouw

ir. D.C van Keulen[1,2]

[1] Faculteit Civiele Techniek en Geowetenschappen, Technische Universiteit Delft
[2] Ingenieursstudio DCK, Pesetastraat 48, 2991 XT Barendrecht, post@studio-DCK.nl

1. Inleiding

Het geprefabriceerde hoogbouw concept sluit aan bij de principes van het nieuwe bouwen. De trend is immers dat bouwen steeds vaker bestaat uit het monteren van vooraf vervaardigde producten. Diverse hoogbouw projecten in prefab beton zijn reeds met een goed georganiseerd bouwproces in een relatief korte bouwtijd gerealiseerd. Ook zaken als de risicospreiding voor de hoofdaannemer, de betere kwaliteit van industrieel vervaardigde producten en de gunstige bouwplaatsomstandigheden sluiten aan bij de principes van het nieuwe bouwen. Constructief ontwerpers van hoge gebouwen kiezen veelal voor monoliete betonconstructies. De reden is dat de buigstijfheid van een geprefabriceerde betonconstructie lager ligt dan die van de monoliete variant. De keuze van ontwerpers voor monoliete betonconstructies bij hoogbouw is daarmee een verdedigbare. Hoewel de trend van het nieuwe bouwen conflicteert met de keuze van constructief ontwerpers is het de moeite waard te onderzoeken hoe de prefab skeletbouwmethode voor een hoogbouwopgave presteert. In deze bijdrage komt een gerealiseerde hoogbouw aan de orde. Daarnaast wordt aandacht besteed aan het ontwerp en het gedrag van geprefabriceerde betonconstructies. Vervolgens wordt het gedrag van monoliete en geprefabriceerde wanden met open voegen berekend en onderling vergeleken.

2. Gerealiseerde geprefabriceerde hoogbouw

In 's-Gravenhage staat het hoogste gebouw – 131 meter – dat volledig is uitgevoerd met de prefab skeletbouw methode. Het is een woontoren en draagt de naam "Het Strijkijzer". Een aantal van de bij de realisatie betrokken partijen zijn architect AAArchitecten, hoofdconstructeur Corsmit Raadgevend Ingenieursbureau, prefab constructeur overall Aveco de Bondt en prefab constructeur vloeren Hurks Delphi Engineering. De hoofdaannemer was Boele & van Eesteren.

A. Realisatie

De aannemer heeft de opdrachtgever twee alternatieve bouwmethoden voorgelegd. Het zijn de in het werk gestorte methode en de prefab skeletbouwmethode. De prefab skeletbouwmethode bleek niet de meest economische, maar leverde een tijdbesparing op van 1 jaar bouwen. In dit laatste wilde de opdrachtgever investeren. Hij kon daarmee enerzijds eerder rendementen uit verkoop en verhuur genereren en anderzijds besparen op de rentekosten. De torenkraan had voldoende capaciteit voor het transporteren van prefab beton elementen. De maximale hijscapaciteit is vastgesteld op 20 ton. Deze hoge capaciteit was vooral te danken aan de korte vlucht van de prefab elementen. De weersinvloeden bij het hijsen van de betonelementen hebben niet tot grote problemen geleid. Het was slechts op één dag niet mogelijk de kraan te gebruiken. Dit kwam door het jaargetijde waarin gewerkt werd en door de beschutting van de toren zelf.

B. Constructieve analyse

De constructieve opbouw van de woontoren bestaat uit een in het werk gestort deel ter hoogte van de eerste 4 bouwlagen. Daarop zijn prefab wandelementen gestapeld. De wanden zijn 250 mm dik en hebben een betonkwaliteit C53/65. De wanden zijn met vertandingen onderling gekoppeld. De samengestelde wanden vormen een gevelbuis constructiesysteem. De vloeren zijn massief uitgevoerd met daarin opgenomen de meeste installatie voorzieningen. De vloeren liggen met een ingestort stalen "handje" op de dragende wanden. Tijdens de voorbereiding heeft men onderzocht wat het verschil in stijfheid is tussen de monoliet gestorte uitvoering en de geprefabriceerde uitvoering. Volgens [1] "bedraagt de uiteindelijke afname van de horizontale stijfheid 5 á 10%. De horizontale vervormingen liggen daarmee nog steeds binnen de daarvoor gehanteerde uitgangspunten en normen." Voor meer informatie over dit project wordt verwezen naar diverse publicaties, onder ander in het vakblad Cement [1].

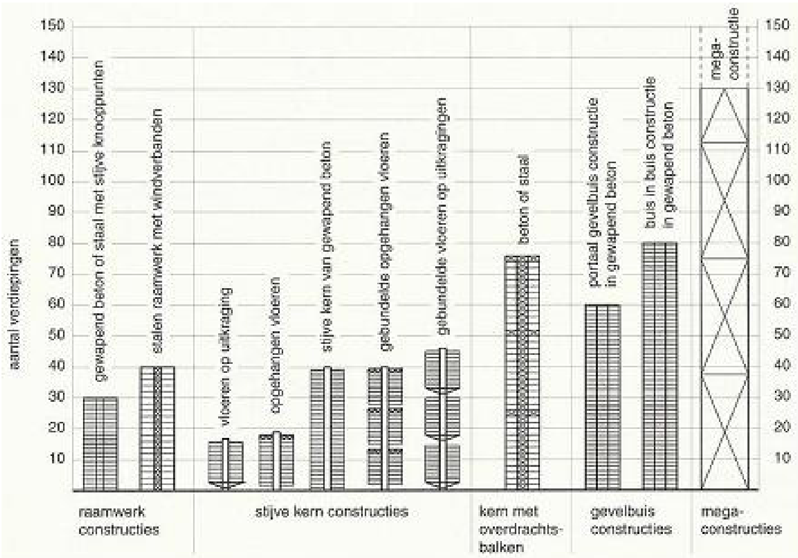
3. Ontwerpen van geprefabriceerde betonconstructies

A. Hoogbouw ontwerpen

Het ontwerpen van hoogbouw is geen reguliere opgave. Er spelen specifieke hoogbouw onderwerpen en het ontwerpproces verschilt ook enigszins van laagbouw. Een aantal van deze zaken zijn beschreven in een artikel naar aanleiding van een interview met architect Diederik Dam. [2]. Het laat zich als volgt samenvatten. Elke bouwopgave heeft een contextuele relatie, met stedenbouwkundige en architectonische aspecten. Dat geldt ook voor hoogbouw. De inpassing in de bebouwde omgeving moet met zorg plaatsvinden. Ook moeten de kenmerkende begrippen "hoog"en "verticaal" in je architectuur benadrukt worden. Hoogbouw is bij uitstek een opgave waarbij veel bij de bouw betrokken disciplines betrokken zijn. De keuze van een constructiesysteem wordt dan ook niet alleen bepaald door de constructie. Architectuur, bouwkunde, constructie, installatie, kosten deskundigheid en uitvoering spelen alle een belangrijke rol. Alleen een goede integratie van deze disciplines levert in het ontwerpproces een bevredigend resultaat op. De kosten per m^2 nemen toe naarmate het ontwerp hoger wordt. Compact ontwerpen is dan de opgave en op die compacte plattegrond gebeurt heel veel op het gebied van techniek en logistiek. “Het is dus belangrijk om geïntegreerd te ontwerpen, samen met andere adviseurs en de opdrachtgever. Als je bijvoorbeeld in een hoogbouwontwerp de liftcapaciteit wijzigt, dan wijzigt het complete ontwerp, want er is nergens een verloren hoekje waar je dat alsnog in kunt stoppen. De plattegrond bij hoogbouw is meestal alzijdig. Lift en trappen bevinden zich centraal in het gebouw. De woningen en kantoren worden daaromheen verdeeld, met een maximaal uitzicht en aanzicht. Het is de kunst om een vaste structuur van woningscheidende wanden te maken waartussen een aantrekkelijk oppervlakte ontstaat.

B. Geprefabriceerd constructief ontwerpen

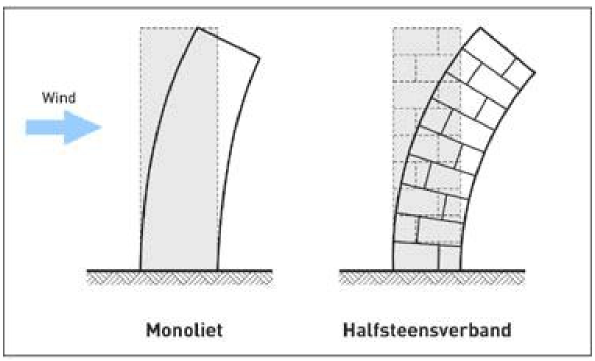
Door de constructief ontwerper wordt een constructief ontwerp voorgesteld. In het ontwerp komen verschillende constructieve onderdelen aan bod. Er moet een stabiliteitssysteem worden gekozen. Ook worden keuzes gemaakt ten aanzien van type vloerconstructies en verbindingen. Indien een geprefabriceerde betonconstructie wordt overwogen dient daarvoor een specifiek geprefabriceerd ontwerp gemaakt te worden. De gedachte dat een monoliete betonconstructie opgedeeld kan worden in prefab elementen is te eenvoudig en levert geen goede hoogbouw constructie op.



Figuur 1: Overzicht constructiesystemen. Bron: opgenomen in diverse publicaties.

C. Stabiliteitsystemen

Inmiddels zijn gebouwtypologieën voor hoogbouw ontwikkeld die worden voorzien van beproefde constructiesystemen. De constructieve maatregelen om vervormingen te beperken en de krachten op te nemen zijn zo kenmerkend voor deze constructies dat deze criteria gebruikt worden voor een indeling in constructiesystemen. Een overzicht is gegeven in figuur 1. Wanneer in dit overzicht sprake is van wanden of kernen is daar meestal ook een geprefabriceerde oplossing voor mogelijk.



Figuur 2: voorbeeld geprefabriceerde elementconfiguratie; bron: bouwwereld nr. 20

D. Elementconfiguratie

In het geprefabriceerde ontwerp moet de constructieve werking van het stabiliteitssysteem worden vastgelegd. Het opdelen van wanden en kernen gebeurt niet willekeurig. Het wordt zodanig samengesteld dat de totale constructie zo goed mogelijk presteert. In figuur 2 wordt deze gedachte verduidelijkt. In figuur 4 en 5 zijn een aantal voorbeelden van elementconfiguraties voor wanden getekend. Naast deze bestaan, afhankelijk van het ontwerp nog vele andere mogelijkheden voor wanden en kernen.

E. Verbindingen

De betonelementen worden aan elkaar gekoppeld zodat uiteindelijk één samenhangend geheel ontstaat. Voor de hoofddraagconstructie zijn de verbindingen op te delen in drie categorieën. Het zijn de *wand-op-wand* verbindingen, de *vloer-aan-wand* verbindingen en de *wand-aan-wand* verbindingen. Wand-op-wand verbindingen worden doorgaans uitgevoerd met stekken in gains in combinatie met voegen van krimparme cementgebonden mortels. Deze verbinding heeft diverse functies. “Een voegmortel bezit hoogwaardige eigenschappen om tezamen met hoogwaardige aansluitmaterialen grote druk- en afschuifkrachten over te brengen en is tevens in staat maatafwijkingen te compenseren" [3]. Daarnaast is stekwapening in staat eventuele trekkrachten op te nemen indien in de verbinding trekspanningen ontstaan. Vanwege de samenwerking tussen verschillende wandelenten zijn *wand-aan-wand* verbindingen nodig. Er zijn verschillende soorten verbindingen voor deze toepassing in gebruik. Bekende verbindingen zijn open voegen, lasplaat verbindingen, lusverbindingen in natte knopen en vertandingen. Van lasplaten is bekend dat het gedrag slapper is dan bijvoorbeeld van natte knopen of vertandingen. Bij lasplaten is de brandwerendheid ook een belangrijk nadeel. Vanwege het sterkte en vervormingsgedrag zijn vertandingen in combinatie met in verband geplaatste wanden met verticale open voegen het meest effectief. Natte verbindingen presteren ook zeer goed maar zijn relatief bewerkelijk en dus kostbaar.

4. Gedrag van geprefabriceerde betonconstructies

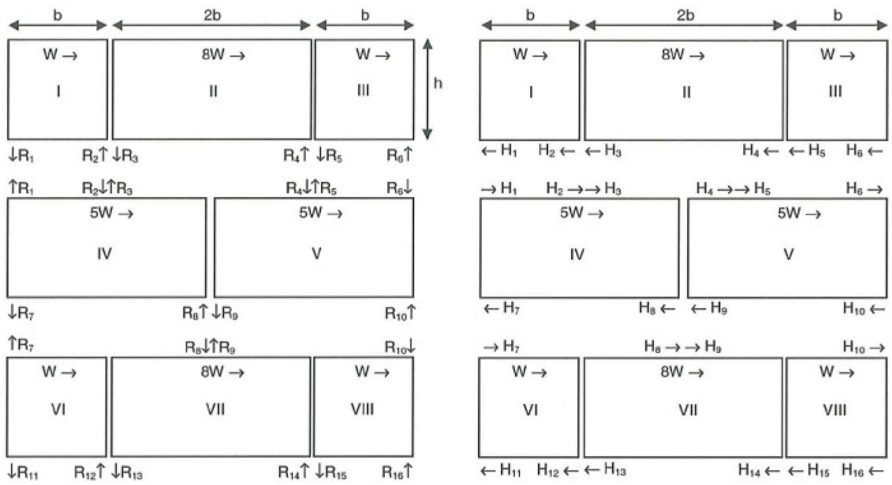
Een geprefabriceerde betonconstructie gedraagt zich anders dan de monoliete variant. Bij de monoliete betonconstructie werkt het stabiliteitssysteem als één geheel. De geprefabriceerde uitvoering is veel meer te beschouwen als een systeem van in verband geplaatste elementen die samengesteld het stabiliteitssysteem vormen. De krachtsverdeling wordt bepaald door de wijze waarop elementen elkaar onderling belasten. De totale vervorming bestaat uit een sommatie van de vervormingen van de individuele elementen.

A. Berekening krachtsverdeling

Verschillende onderzoekers hebben zich gebogen over de vraag hoe de krachtsverdeling van een geprefabriceerde constructie eenvoudig bepaald zou kunnen worden. Voor een aantal geselecteerde constructies heeft dit eenvoudige rekenmethoden opgeleverd. Voor de anderen ontbreken deze. Over de exacte bepaling van de krachtswerking is men het eens. Alleen met een EEM berekening kan het eenduidig worden vastgesteld. Het nadeel is wel dat het uitvoeren van deze berekeningen arbeidsintensief en dus kostbaar is.

Van Dorst [4] heeft een onderzoek uitgevoerd naar de schijfwerking van geprefabriceerde dragende gevelelementen waarbij de verticale voegen verspringend worden aangebracht. De onderzoeker heeft gezocht naar eenvoudige rekenmethoden voor het bepalen van de krachtswerking. In dit onderzoek komen een aantal methoden ter sprake:

1. Blokkenmethode; dit is een handberekeningsmethode waarbij de schijven in de gevel als niet vervormbare blokken worden beschouwd. De blokken zijn alleen met stekken in de horizontale voegen met elkaar verbonden. De krachten tussen de elementen worden bepaald uit de drie beschikbare evenwichtsvergelijkingen die per blok worden opgesteld. In het onderzoek van Snelders [5] komen gelijksoortige modellen aan de orde. Een daarvan is opgenomen in figuur 3.



Figuur 3: blokken model volgens Snelders [5]

2. Raamwerkanalogie; een computerberekening waarbij het element met sparingen wordt opgevat als raamwerk. De stijfheden van de kolommen (penanten) en liggers (regels) verschillen en worden afzonderlijk opgegeven in een raamwerkprogramma.
3. Schijfwerking met een gedetailleerd computermodel; het element wordt beschouwd als schijf met daarin een aantal openingen. Middels een EEM berekening wordt de krachtswerking in het gevelelement bepaald.
4. Schijfwerking met een equivalent computermodel; deze methode is afgeleid van de bovenstaande methode. Het is bedoeld als sterk vereenvoudigde methode van een uitgebreide EEM berekening. De vervormingseigenschappen worden zo goed mogelijk benaderd in één element. Na de berekening van de krachtswerking in en tussen de equivalente schijven, wordt met interpolatie de krachtsverdeling in het oorspronkelijke element berekend.

Onderzoeker Falger [6] heeft een literatuuronderzoek uitgevoerd. In deze studie zijn de bovenstaande modellen geanalyseerd. De literatuurstudie resulteert voor de krachtswerking in een aantal conclusies:

1. De krachtswerking tussen de elementen in een constructie met open verticale voegen kan relatief eenvoudig worden bepaald met de blokkenmethode.
2. De schematisering van de gevel met de raamwerkmethode geeft goede resultaten voor het bepalen van krachten en momenten in kolommen en regels, in ieder geval voor de standaard constructies. De methode is voor de ingenieurspraktijk omslachtig.
3. Berekeningen met een EEM-programma geven goede resultaten ten aanzien van de krachts-verdeling, ook voor constructies met open voegen. De invoer blijft tijdrovend en weinig inzichtelijk. De schematisering van de voegen vraagt de nodige aandacht en heeft grote invloed op de resultaten.
4. De aangepaste schematisering met equivalente schijven blijkt ook niet nauwkeurig. Een belangrijk probleem is dat het bijna onmogelijk is om vanuit de resultaten voor de equivalente schijf krachten en momenten in afzonderlijke kolommen en regels te bepalen.

B. Berekening vervormingsgedrag

Naast het bepalen van de krachtsverdeling moeten ook vervormingen worden bepaald. De literatuurstudie van Falger [6] komt ook met conclusies die betrekking hebben op het vervormingsgedrag van stabiliteitssystemen:

1. Een eenvoudige manier voor het bepalen van de vervormingen bestaat (nog) niet.
2. Het blokkenmodel geeft op een eenvoudige manier inzicht in de krachtswerking maar geeft niet aan hoe de vervormingen kunnen worden bepaald.
3. Het schematiseren van de gevelelementen tot raamwerken is een goede maar zeer bewerkelijke manier om de vervormingen van de gevel te bepalen.
4. Een EEM berekening is een vrij omslachtige berekeningsmethode. De modellering van de voegen verdient speciale aandacht. Een EEM berekening kan de vervormingen echter wel nauwkeurig voorspellen.

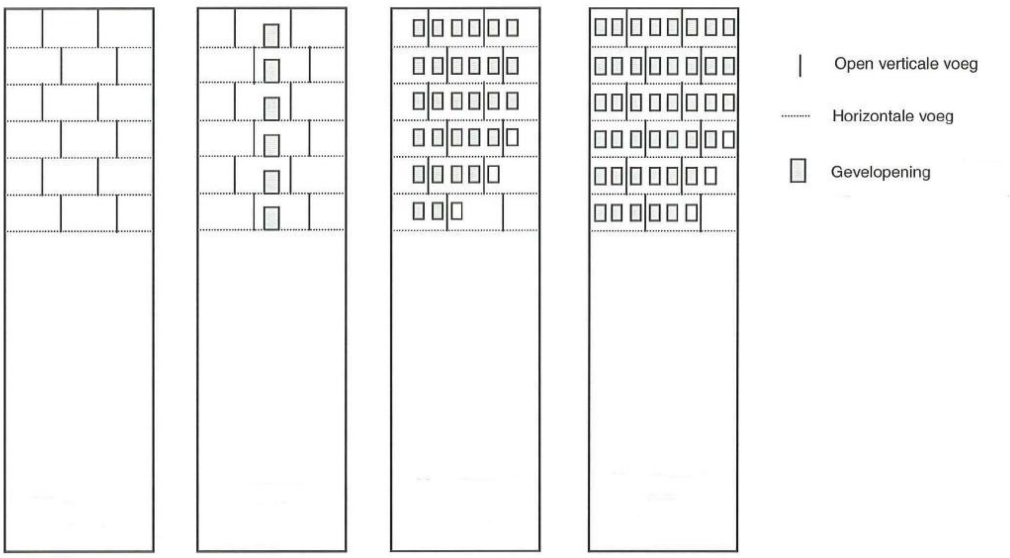
5. Geprefabriceerd en monoliet vergeleken (1)

A. Vergelijkend onderzoek

Om inzicht te krijgen in het gedrag van geprefabriceerde betonconstructies ten opzichte van de monoliete variant hebben verschillende onderzoekers vergelijkende berekeningen gemaakt. In het navolgende worden de vervormingen van een wand bepaald en vergeleken. De reden dat vervormingen worden vergeleken is dat bij hoogbouw de stijfheid en dus de vervormingen bepalend zijn.

B. Onderzoeksmodel van Falger

In het onderzoek van Falger [7] zijn voornamelijk verticale voegen onderzocht. Bekeken is hoe verschillende verbindingen zich gedragen in een samengestelde wandconstructie. De wandelementen werden in een metselwerkverband gestapeld zoals getekend in figuur 4. Voor een viertal constructietypen is onderzocht welke invloed open verticale voegen hebben op de respons van de geprefabriceerde betonnen stabiliteitsconstructies. De beschouwde constructietypen variëren van een volledig gesloten stabiliteitswand tot een dragende gevel met gevelopeningen.



Figuur 4: Constructietype A t/m D van Falger [7]

De wandconstructie is door Falger overgenomen uit het referentieproject “De Prinsenhof” in Den Haag. De wanden zijn 14,4x86,4 meter (bxh) en hebben een slankheid van 6:1. Ook de gebruikte belastingen zijn gerelateerd aan dit gebouw. Voor de stijfheden van de voegen zijn aannames gedaan op basis van realistische verbindingen. Voor de betonkwaliteit heeft men C53/65 aangehouden met een elasticiteitsmodulus van E’b = 38.500 N/mm^2. De wandconstructies zijn berekend met het EEM-programma ATENA. De in dit onderzoek gehanteerde rekenmethode is een Geometrisch (1^e orde) en Fysisch Lineaire berekening.

C. Onderzoeksresultaten

Uit dit onderzoek blijkt dat bij geprefabriceerde constructies met open voegen de momenten, de dwarskrachten en de spanningen in de prefab elementen lokaal tot 45% groter kunnen zijn ten opzicht van een monoliete constructie. Dat de krachtsverdeling lokaal in de betonconstructie tot grotere waarden leidt is normaal gesproken oplosbaar. Soms kan eenvoudig in een bepaald element de betonkwaliteit worden verhoogd. Ook kan waar nodig lokaal extra wapening worden toegepast.

Vervorming Constructie type	Monoliet [mm]	Open voegen [mm]	Open voegen [%]
A	51,5	54,2	105,2
B	63,4	68,5	108,0
C	63,7	67,6	106,1
D	84,4	89,5	106,0

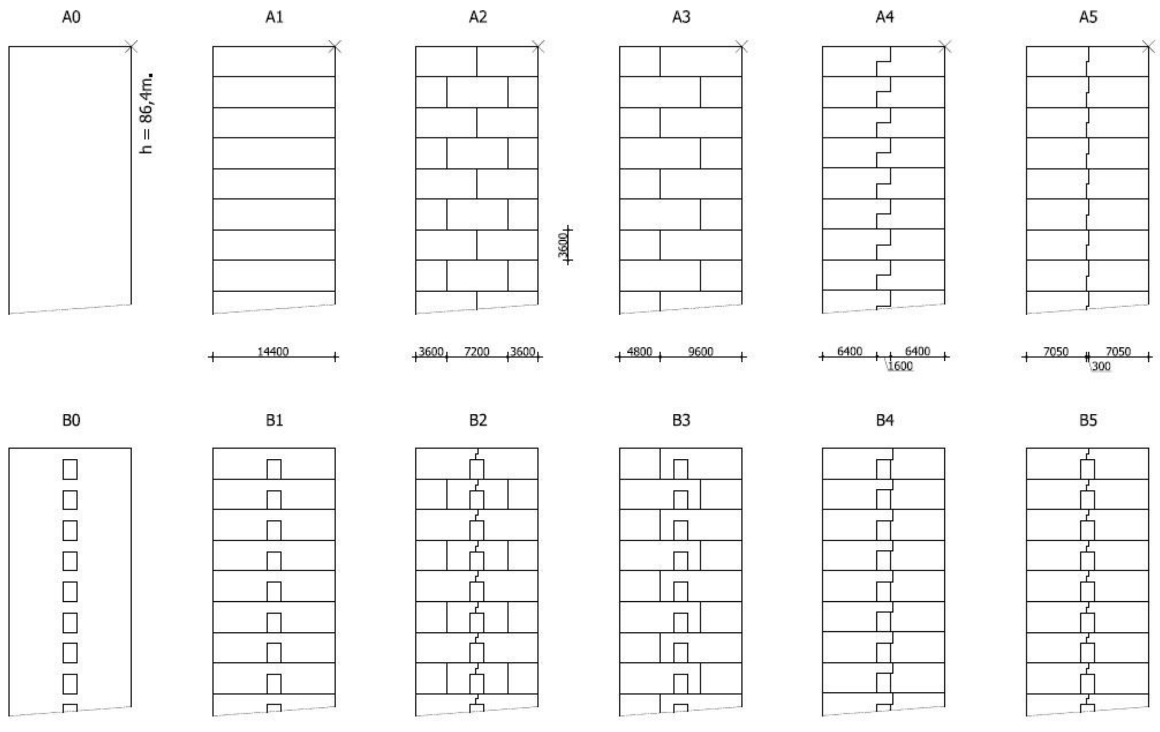
Tabel 1 Vervormingen [7]

In stabiliteitsconstructies voor hoogbouw zijn de vervormingen aan de top van een gebouw bepalend. In het onderzoek is onderzocht hoe de vervormingen ($U_{x;top}$) van de geprefabriceerde wandconstructies zijn in vergelijking met dezelfde monoliete betonconstructie. De waarden hiervan zijn opgenomen in tabel 1. De volgende hoofdconclusies werden door de onderzoeker getrokken. Een stabiliteitswand die met open voegen in metselwerkverband opgebouwd is, zal altijd een lagere overall wandstijfheid hebben. De stijfheidsafname blijkt, onafhankelijk van het toegepaste constructietype, maximaal 8% te zijn. Verder is gebleken dat lage afschuifstijfheden van voegen een groot effect op de toename van de vervormingen hebben. Is de afschuifstijfheid groter, dan blijkt de vervormingstoename minder invloed te hebben. Het onderzoek geeft aan dat een hoogbouwproject met prefab elementen in metselwerkverband het gedrag van een monoliet gestorte constructie binnen een aanvaardbare bandbreedte benadert.

6. Geprefabriceerd en monoliet vergeleken (2)

A. Open verticale voegen en monoliet vergeleken

In figuur 5 zijn wanden met open verticale voegen verder gevarieerd door de auteur van dit artikel. Door de stijfheid van de horizontale voegen monoliet te modelleren is het stijfheidsverlies ten gevolge van alleen de open verticale voegen onderzocht. Daarnaast is onderzocht hoe groot de invloed van het 2^e orde effect is op de vervormingen van de wandconstructies. Voor dit onderzoek zijn de gegevens van het referentieproject van Falger [7] overgenomen.



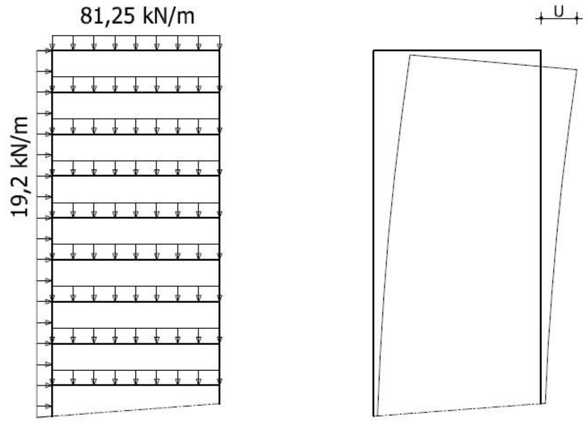
Figuur 5 Elementconfiguraties met open voegen

B. Onderzoeksmodel

De berekeningen zijn uitgevoerd met het EEM programma AxisVM. Voor zowel de horizontale als de verticale voegen zijn interface elementen gebruikt. De interface elementen van de verticale voegen hebben een stijfheid 0, terwijl voor de horizontale voegen een gekozen stijfheid is toegepast. In de EEM berekening is voor sterkteklasse C53/65 met een E-modulus van 38.000 N/mm^2 gekozen. Bij de keuze van de belasting is ervan uitgegaan dat in de wanden geen trekspanningen optreden. Dit uitgangspunt wordt ook in de praktijk veel toegepast. De belastingen zijn per verdieping aangebracht zoals in figuur 6 aangegeven.

C. Variaties met open verticale voegen

Wand A0 in figuur 5 is een monoliete constructie. Wand A1 heeft een horizontale voeg met een equivalente stijfheid van de monoliete variant A0. De vervormingen van wanden A0 en A1 zijn om die reden gelijk. Voor wanden A2 t/m A5 hebben de horizontale voegen dezelfde stijfheid. De verticale voegen zijn open. Op deze wijze is alleen de invloed van de verticale open voeg gevarieerd. Dezelfde principes zijn ook toegepast voor de wanden van de B-serie. Van deze wandconstructies zijn zowel 1e orde (GL) als 2e orde (GNL) berekeningen gemaakt. In de berekeningen gedraagt het materiaal zich lineair elastisch. De resultaten zijn opgenomen in tabel 2/3.



Figuur 6 Belastingen en vervormingen

Vervorming Wandtype	1e orde (GL) [mm]	1e orde (GL) [%]	2e orde (GNL) [mm]	2e orde (GNL) [%]	n/n-1*
A0	50,93	100,0	51,46	100,0	1,010
A1	50,94	100,0	51,47	100,0	1,010
A2	51,28	100,7	51,82	100,7	1,011
A3	51,59	101,3	52,13	101,3	1,010
A4	51,44	101,0	53,02	103,0	1,031
A5	52,86	103,8	53,43	103,8	1,011
* vergrotingsfactor zonder vergroting t.g.v. het stijfheidsverlies horizontale voegen					

Tabel 2 Vervormingen, t.g.v. open verticale voegen, A-serie

Vervorming Wandtype	1e orde (GL) [mm]	1e orde (GL) [%]	2e orde (GNL) [mm]	2e orde (GNL) [%]	n/n-1*
B0	52,31	100,0	52,87	100,0	1,011
B1	52,32	100,0	52,88	100,0	1,011
B2	54,48	104,1	54,62	103,3	1,003
B3	53,22	101,7	53,80	101,8	1,011
B4	53,02	101,4	53,59	101,4	1,011
B5	53,53	102,3	54,11	102,3	1,011
* vergrotingsfactor zonder vergroting t.g.v. het stijfheidsverlies horizontale voegen					

Tabel 3 Vervormingen, t.g.v. open verticale voegen, B-serie

Uit de berekeningen blijkt voor de A-serie dat de wandconstructies met open verticale voegen maximaal 3,8% meer vervormen dan de monoliete betonconstructie. Voor de wandtype van de B-serie is de maximale toename in vervorming berekend op 4,1%. Dit betekent dat enerzijds de geprefabriceerde oplossing minder stijfheid heeft maar dat anderzijds de vervormingstoename gering is. De vergrotingsfactor voor 2e orde effecten heeft voor beide series een grootte van 1,011. De conclusie die daaruit getrokken kan worden is dat stijfheidsverlies door open voegen in deze gevallen niet direct tot substantieel grotere 2e orde effecten leiden.

D. Variaties met open verticale voegen en horizontale mortelvoegen

In de vorige paragraaf is onderzocht hoe groot de stijfheidsverliezen zijn ten gevolge van de open verticale voegen. In deze paragraaf worden gelijksoortige berekeningen gemaakt. Nu worden de horizontale voegen gemodelleerd als horizontale mortelvoegen met verticale stekwapening. De verticale voegen blijven open voegen. De horizontale voegstijfheid is ondermeer afhankelijk van de stekwapening en bovenbelasting. Omdat de bovenbelasting per element, afhankelijk van de plaats in de wand varieert is de wand in 2 secties verdeeld. Voor de voegstijfheid is een gemiddelde waarde gebruikt. De horizontale voegen van de 11 verdiepingen bovenin hebben een afschuifstijfheid van $3{,}0 \times 10^5$ kN/m/m. De afschuifstijfheid van de 12 verdiepingen onderin de wand bedraagt $9{,}3 \times 10^5$ kN/m/m. Deze waarden representeren realistische voegen. De berekende vervormingen komen overeen met de waarden die in onderzoek [7] zijn gevonden.

Vervorming Wandtype	1e orde (GL) [mm]	1e orde (GL) [%]	2e orde (GNL) [mm]	2e orde (GNL) [%]	n/n-1
A0	50,93	100,0	51,46	100,0	1,010
A1	54,46	106,9	55,02	106,9	1,010
A2	54,54	107,1	55,12	107,1	1,012
A3	54,10	106,2	54,68	106,3	1,012
A4	53,77	105,6	54,34	105,6	1,012
A5	55,14	108,3	55,74	108,3	1,012

Tabel 4 Vervormingen, t.g.v. open verticale voegen en horizontale mortelvoegen, A-serie

Vervorming Wandtype	1e orde (GL) [mm]	1e orde (GL) [%]	2e orde (GNL) [mm]	2e orde (GNL) [%]	n/n-1
B0	52,31	100,0	52,87	100,0	1,011
B1	55,30	105,7	55,90	105,7	1,011
B2	57,63	110,2	57,78	109,3	1,003
B3	56,39	107,8	57,01	107,8	1,011
B4	54,81	104,8	55,41	104,8	1,011
B5	56,55	108,1	57,17	108,1	1,011

Tabel 5 Vervormingen, t.g.v. open verticale voegen en horizontale mortelvoegen, B-serie

In tabel 4 en 5 is gerapporteerd hoe groot de 1e orde en 2e orde vervormingen zijn. Uit de berekeningen blijkt voor de A-serie dat de wandconstructies maximaal 8,3% meer vervormen dan de monoliete betonconstructie. Voor de wandtype van de B-serie is de maximale toename in vervorming berekend op 10,2%. Bij deze waarden moet de kanttekening worden geplaatst dat de afschuifstijfheid van de horizontale voegen nauwkeuriger berekend zou kunnen worden. Er is hier namelijk gerekend met een conservatieve inschatting op basis van eerdere berekeningen. Ook hier kan de conclusie worden getrokken dat enerzijds de geprefabriceerde oplossing minder stijfheid heeft maar dat anderzijds de vervormingstoename gering is. De vergrotingsfactor voor 2e orde effecten heeft ook hier een maximale waarde van 1,012. De conclusie die daaruit getrokken kan worden is dat stijfheidsverlies door open voegen in deze gevallen niet direct tot grotere 2e orde effecten leiden.

Wand	1e orde t.g.v. verticale voegen [%]	1e orde t.g.v. horizontale voegen [%]	1e orde toename totaal [%]
A1	0,0	6,9	106,9
A2	0,7	6,4	107,1
A3	1,3	4,9	106,2
A4	1,0	4,6	105,6
A5	3,8	4,5	108,3
B1	0,0	5,7	105,7
B2	4,1	6,1	110,2
B3	1,7	6,1	107,8
B4	1,4	3,4	104,8
B5	2,3	5,8	108,1

Tabel 6 Vervormingen, opsplitsing aandeel verticale open voegen en horizontale mortelvoegen

E. Bijdrage van verschillende voegen

Zowel de verticale als de horizontale voegen leveren beide een bijdrage aan het stijfheidsverlies ten opzichte van de monoliete betonconstructie. In tabel 6 zijn deze waarden opgesplitst. Hieruit is af te lezen wat het aandeel stijfheidsverlies door de verticale open voegen en de horizontale mortelvoegen is. Uit deze opsomming kan de conclusie worden getrokken dat de toename van de vervormingen voor het grootste deel komt door de horizontale voegen. Het aandeel in het stijfheidsverlies door de open voegen is in deze wandconstructies minder.

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