

Double-layer Curved Steel-structure with Bent Glazing; New Departure Station Erasmusline, The Hague (NL)

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ZJA Zwarts & Jansma Architects have designed a new departure light rail station in The Hague, Netherlands. The spatial roof structure of the station is made of rolled rectangular steel hollow sections, arranged in two independent layers rigidly connected to each other. A glass envelope covering the roof structure is matching the contours of the steel exactly. Since the diamond shaped glass panels can only be attached to the outer layer of the steel grid, the glass with edge lengths of approximately 1,30m is two-side supported. When optimizing the overall geometry the double-curved area at the nose of the roof structure became a special focus. Knippers Helbig Advanced Engineering has managed to minimize the deviation of each single glass panel from the single curved geometry to a maximum out-of-plane deformation of only 3mm. Thus, stresses due to warping effects caused by imposed loads including deformation of the primary structure are within the allowable range for heat strengthened (HS) as well as for float glass (AN). The project is a great example of how geometry development can influence glass design, enabling new approaches.

Keywords: bent glazing, geometry optimization, curved steel-structure, SGP, doubly-curved, double curvature

1. Urban context and Architecture of the New Departure Station Erasmusline (HSE)

The Erasmusline is part of RandstadRail, a network of light rail systems connecting the cities of The Hague, Zoetermeer, Rotterdam and the region in between. At this moment, platforms 11 and 12 of the existing train station ‘Den Haag Centraal’ are in use by Randstadrail but in the future will be needed for further expansion of the heavy rail tracks. After several studies, the new tracks were positioned: one at a height of ca. 15 meters above grade over an existing tunnel, one at grade in the Anna van Buerenstraat and one at the level of the bus platform (cp. Figure 1).

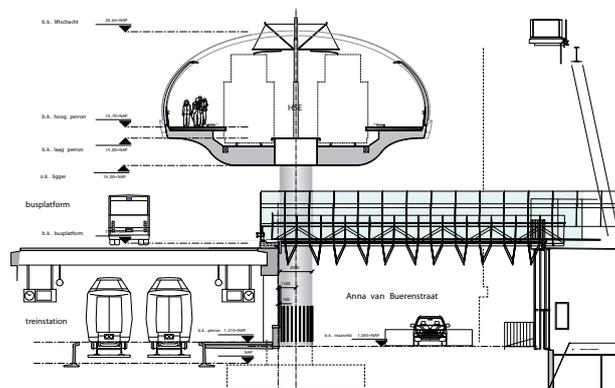


Fig. 1 Cross section – positioning of the piers.

A tunnel was too expensive and at grade there was not enough room in the Anna Van Buerenstraat; moreover, this option turned out to be conflicting with the entrance of an underground parking facility already in use. The option to land the track at the level of the bus station failed because of traffic issues at the crossing with the ‘Prins Bernhard viaduct’ (cp. Figure 2).

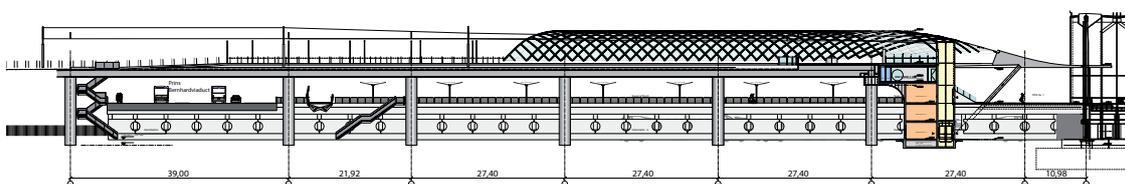


Fig. 2 Longitudinal section.

At first, the track and the station were positioned right above the bus platform, but due to the phasing issues and structural complications with the existing bus platform, the decision was taken to position it in the Anna van Buerenstraat as close as possible to the structures of the combined train station and bus platform.

Locating the track and the station at this level made it highly visible, and thus, placed emphasis on the overall appearance of the combined viaduct and station. The client (in first case the municipality of The Hague) asked ZJA to design a station with its own architectural identity towards 'Den Haag Centraal Station' and the recently new established 'Openbaar Vervoer Terminal' (OVT) (Public Transport Terminal) and related in idiom to the Light Rail Station at the Beatrixlaan (designed and elaborated by ZJA in the period 1998-2008). At the same time, the design needed to seamlessly blend with an extremely complex urban context and become programmatically a part of the 'transition machine' called the OVT. Programmatically, the client asked for two light rail platforms and an arrival platform for the bus station.

The overall architectural shape represents the smooth transition from the 'flat' viaduct, to the spatial station and to the canopy that intersects the façade of the OVT. The form therefore expresses the station's main function - to provide passengers with a clear, protective and comfortable route towards the OVT and the bus platform, while also providing clear sightlines and visibility from the OVT towards the HSE. Passenger flow and transportation flow are uniquely captured by a single sculptural gesture (cp. Figure 3).

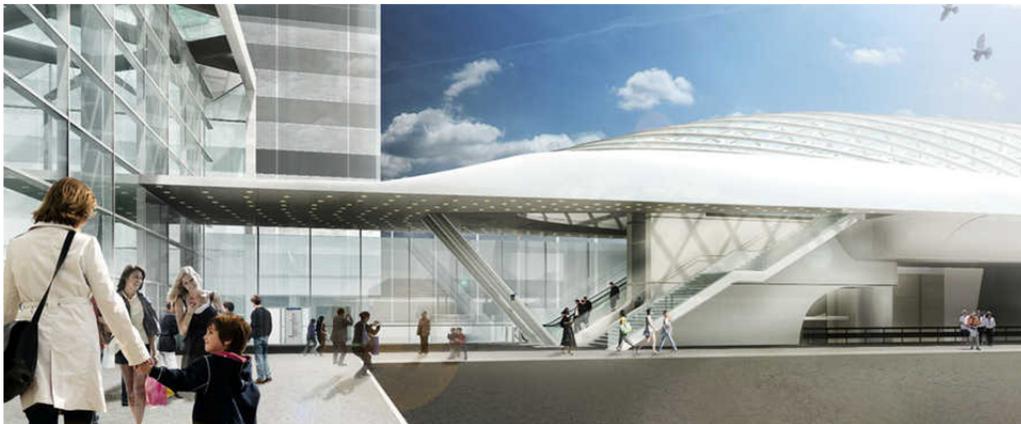


Fig. 3 View from bus platform - rendering by ZJA Zwarts & Jansma Architects.

The actual roof has a total length of approximately 90m and a maximum width (in plan) of 17m; 22m of the roof length is the canopy while 68m is placed on the bridge deck. The maximum height between roof support and roof center is approximately 6m. Roof supports are set 14.7m above ground, on top of the bridge deck and canopy. The structural members and the glazing create a quad-pattern. The steel members, arranged in two separate layers, are curved and twisted following the overall shape. Curved glass stripes follow the outer steel grid (cp. Figure 4).

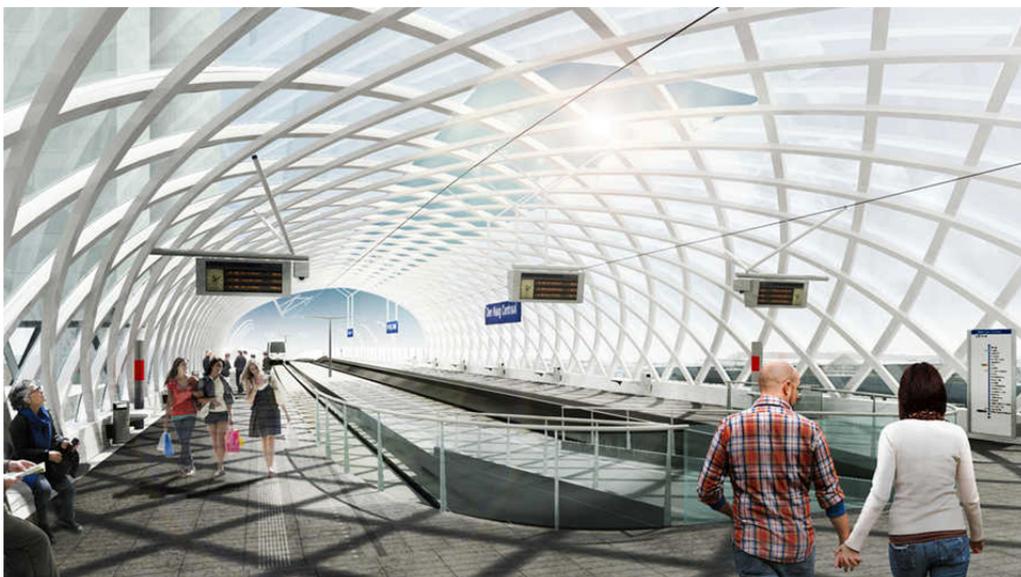


Fig. 4 Interior View – rendering by Zwarts & Jansma Architects.

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ZJA's earlier design for the Station at the Beatrixlaan was based on a spatial truss with a section that transformed from a circular one at the viaduct to an oval section at the platforms and back again. The triangular grid was covered with planar glazed panels where needed at the station sections. For the HSE, there was the ambition to bring the design to the next level and to avoid both triangular glass panels (resulting in substantial material waste) and complex welded nodes (expensive, esthetically unfavorable). Therefore during the HSE assignment, an experiment began in creating a quad-grid geometry combined with rolled steel members (strips) stacked in two layers. This allowed the nodes to appear very simple and - in combination with cold bent glass - resulted in a single design direction.

The first roof and overall shape was much larger than the pending one; it provided shelter for a large middle platform with tracks positioned to the side. Due to the curved contours in plan view, the whole quad-grid was irregular. This brings the advantage that it can structurally work as a grid shell and that it resists deformation in opposition to a regular grid.

Finally, the largest part of the roof became 'regular' within section three radii. Here, where the generic section of the roof transforms to meet the canopy, it is still an irregular grid. Unfortunately, this irregular part does not provide enough stiffness to allow the nodes of the stacked members to act as hinges and therefore must be executed using rigid connections.

2. Geometry

2.1. General

The selected design is the outcome of an intense study in an early project stage where various options were investigated including different grid sizes resulting into different tonnages, number of nodes, and different glass built-ups. Different node types were also studied, having various standing / lying profiles; in one plane, in two separate layers, with hinged nodes / bending stiff nodes.

The geometry of the roof structure was developed under consideration of architectural requirements, global structural design, glass analysis and fabrication constraints.

The roof geometry consists of a regular / generic part with a constant cross section and an irregular doubly-curved geometry towards the central station (cp. Figure 5).

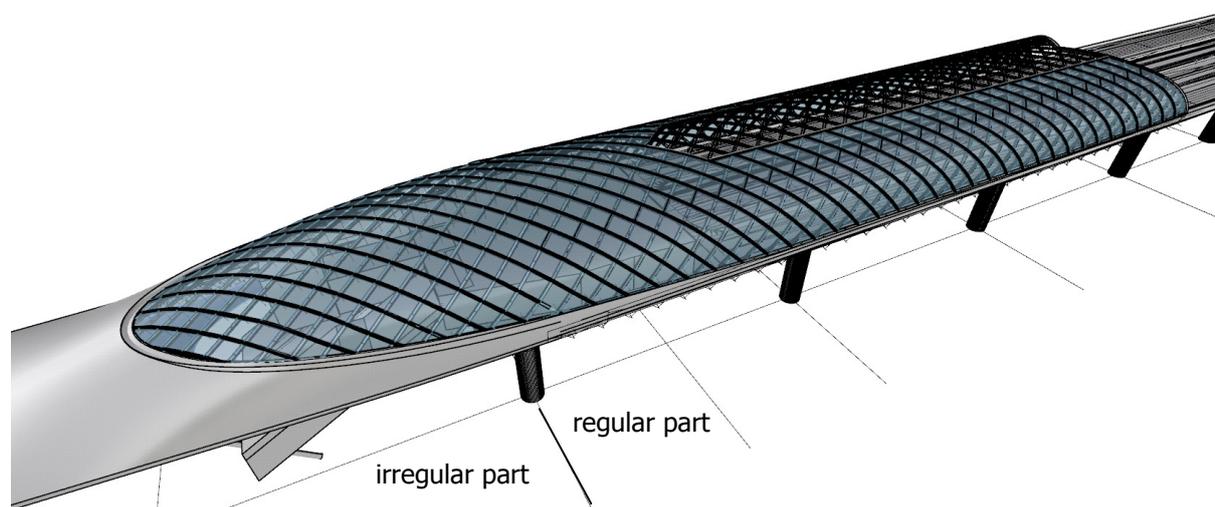


Fig. 5 Roof geometriy consisting of irregular (doubly-curved) and regular (generic) part.

2.2. Regular part

2.2.1. Form finding

The shape of the regular part is the result of various geometrical constraints and parameters. From an architectural standpoint, the clearances for trains, pedestrians, etc. had to be kept free from structural elements. At the same time multiple constraints resulting from fabrication techniques had to be considered. Notably, state-of-the-art steel bending machines as well as glass bending machines can only produce single-curved elements in an economic and automated process.

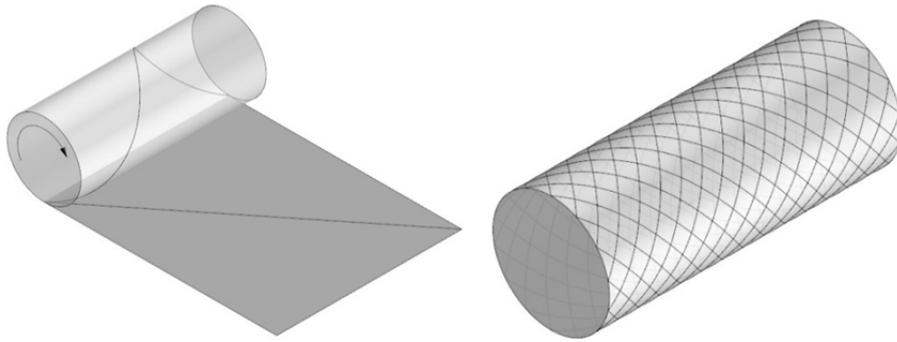


Fig. 6: General principle behind glass and steel geometry.

With regard to these major fabrication constraints, the cross section of the concourse had to be created out of circular segments. The construction lines are defined by geodesics on a cylinder consisting of helices (cp. Figure 6). Several options with three to five circle segments were investigated; finally an option consisting of three circular arc segments - two identical ones located on the outside with a larger tangential one in the middle - was chosen. The tangent vector of the lower circle segments defines the radius of the larger central one. The steel members of each arc segment could be fabricated as a single piece – each steel member had to be joined with the adjacent one on site with hidden head plates and bolts. In order to simplify these bolted field joints and the typical node detail, the changeover between circle segments (“blend-line”) has been located at the center between two structural nodes (cp. Figure 7).

2.2.2. Pattern

The rationalization correction of the surface also followed various constraints. For economic reasons it was decided to generate the grid with only 90° corners in an unrolled condition (cp. Figure 7). Further, the cross section was only filled with identical glass panes, thus, avoiding any odd or irregular pieces; this resulted in a specific number of glass panes, radii and distances between grid lines.

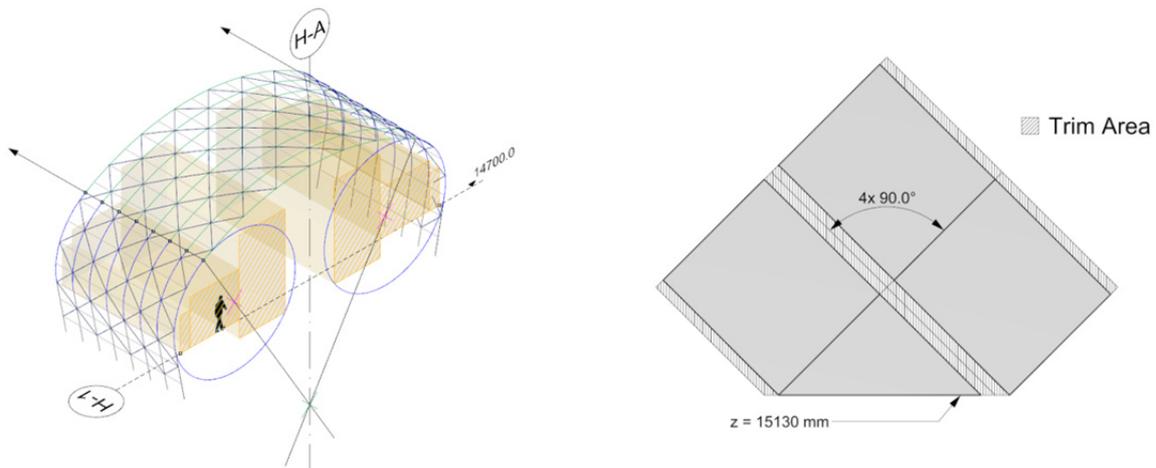


Fig. 7 Generic cross section with diamond pattern (left) / two-side supported glass panes with 90° corners in an unrolled condition (right).

2.3. Irregular part

2.3.1. Form finding

Similar to the regular part, the geometry of the irregular / doubly-curved part is the result of a form finding strategy based on a series of technical constraints and multiple geometrical issues.

To achieve an economically reasonable solution, it was necessary to find a doubly-curved geometry that could be described by single curved glass panels (cylindrical glass). The best results were achieved by a combination of a stretched translational surface and a grid relaxation, which applied a shear force to every group of panel-vertexes constraining them to a corresponding normal plane. This made it possible to generate cylindrical glass panels on top of the resulting mesh.

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Further on, the optimization process aimed to align all lateral structural members and equalize all angles between them, to help generate the steel geometry (circular arcs segments) below the glass layer (cp. Figure 8).

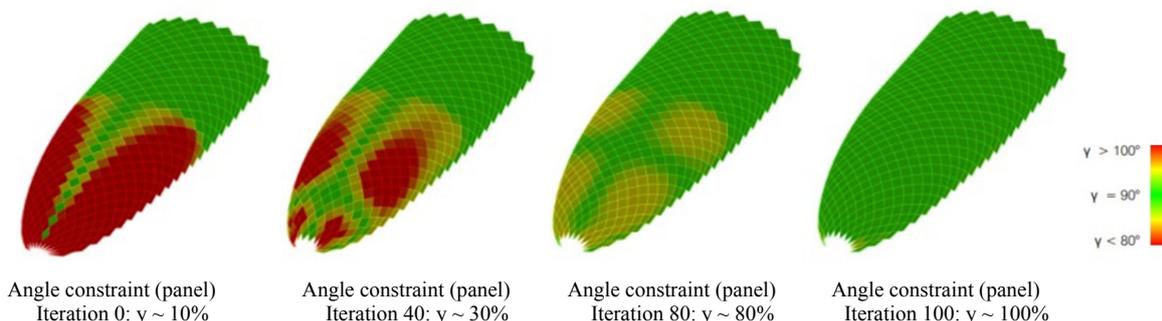


Fig. 8 Optimization of grid in the doubly-curved part.

2.3.2. Steel Segmentation

After generating a “perfect” geometry for the glass pattern; the steel grid still had to be generated following the glass joints. This meant generating a series of arc segments with constant curvature, constant twist and tangential changeover between arc segments, minimizing the deviation to the glass grid.

In a last step, the steel and glass geometries were compared using a custom grasshopper tool with the result of max. 3mm deviation (cp. Figure 9). This value had to be considered besides other effects in the glass analysis (cp. chapter 5).

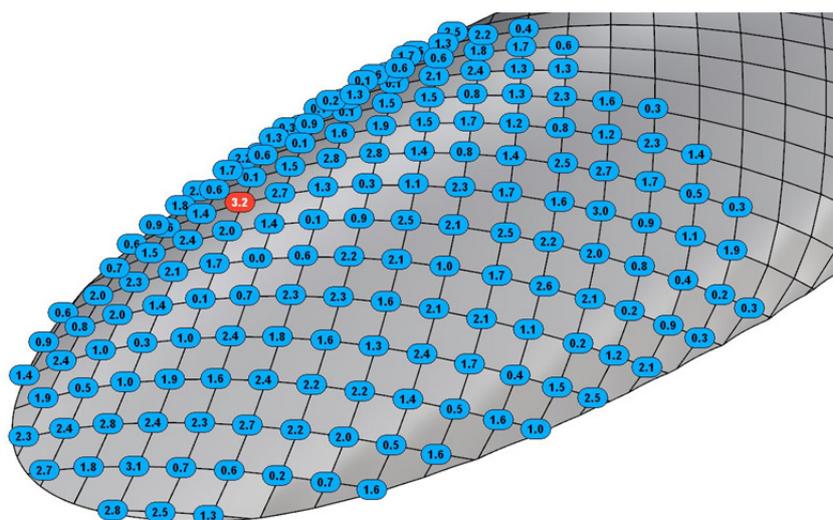


Fig. 9 Distance between glass and steel geometry (mm).

3. Steel-structure

3.1. Concept

The spatial roof structure is made of lying rolled rectangular steel hollow sections, arranged in two independent layers rigidly connected to each other. This diamond-shaped steel grid is supported by a continuous edge beam, which creates a series of stiff triangles along the base.

The cross-section of the roof is a compromise between the minimum clearance necessary for trains and platforms and the structural elements required to provide arch-like behaviour. The central opening – located at a position where all additional loads are most unfavourable to a flat arch – helps to reduce the high bending and allows for a flatter arch shape. In order to achieve the required amount of lateral stiffness, a stronger edge beam is used along the front edge of the roof (“edge beam at entrance”). Furthermore, while the diagonal orientation of the stripes / beams is unfavourable from a structural point of view, this is somewhat compensated given that the structural depth is increased by the stacking of the two layers.

At the closed end of the roof (oriented towards central station) the shape provides 2-dimensional curvature, which helps to create a shell effect. Due to this effect the higher load coming from the fully closed glazing can be transferred to the ground.

The standard grid beams are consisting of 180×100×6.3mm rectangular rolled hollow section. In order to achieve the required structural performance, some elements require a stronger wall thickness of 8mm or 10mm. All profiles are steel grade S355.

3.2. Bearing conditions

One of the most important aspects is the fact that the roof is supported by three different main structures. Figure 10 broadly outlines the relation between bridge deck, bridge-end-column and canopy:

- bridge deck supported by a series of cantilevering columns (grey),
- bridge-end-column consisting of a very high transversal stiffness (red),
- canopy made of steel framework providing access to the central train station with a relatively low stiffness (light blue)

During design process these three structural parts were analysed by three different engineering companies. However, due to the close cooperation between involved parties (as coordinated by the architects), this interface was handled with special care. Stiffness and support reaction forces were exchanged back and forth and discussed regularly.

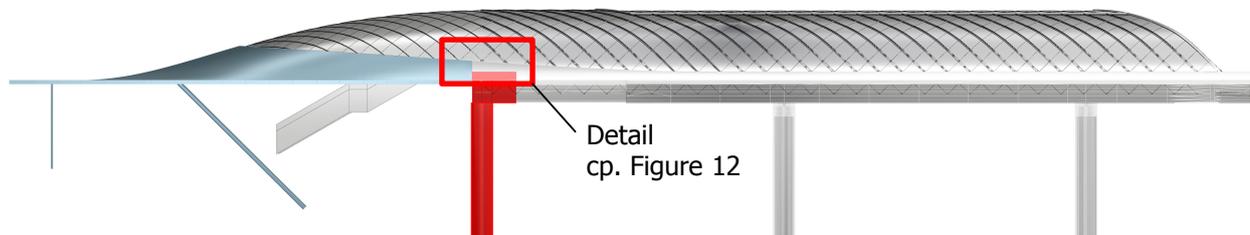


Fig. 10: Roof supported by three different main structures.

Differential settlement and stiffness of roof supports were identified as the cause of internal forces and moments in the structural members of the roof. Very local differential deformation had to be considered during the design, especially near the bridge-end-column, while forces such as wind were leading to horizontal deformation of the main structures. Since bridge and canopy are not connected rigidly, a horizontal kink occurs at the bridge-end-column when both structures tend to deform horizontally. Due to its large weight and the reaction forces transferred to the bridge-end-column by elastomeric support plates, the bridge deck also shows differential vertical deformation at the bridge-end-column (cp. Figure 11). Stiffness and performance behavior of the different parts of the sub-structure have been considered within the structural model by means of springs with certain stiffnesses.

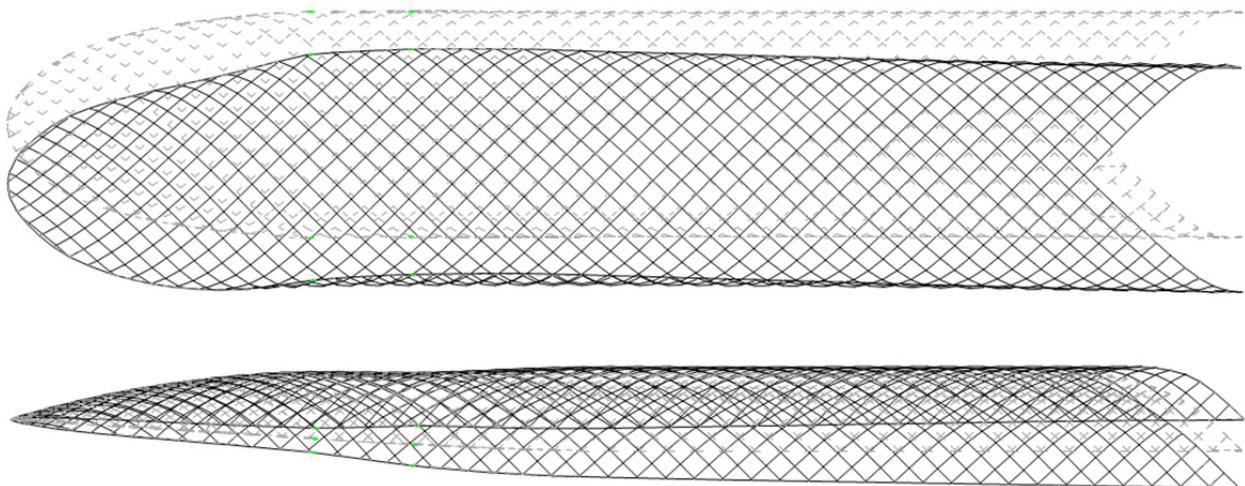


Fig. 11 Horizontal (top) and vertical (bottom) deformation of supporting sub-structure under wind-load conditions.

- A) Welded saw-tooth plates - one welded on the steel profile from inside; another one acting as a washer below the bolt head. Loads can be transferred between the “rips” of the checker plate.
- B) Hilti-Hit injection - the cavity of the hole in the steel plate (which is bigger than the bolt diameter) is filled with Hilti hit through small openings.
- C) Shear Pins - drilled through the washer into the steel profiles.
- D) ‘Araldite’ - the cavity of the hole in the steel plate (which is bigger than the bolt diameter) is filled with liquid material called ‘Araldite’.

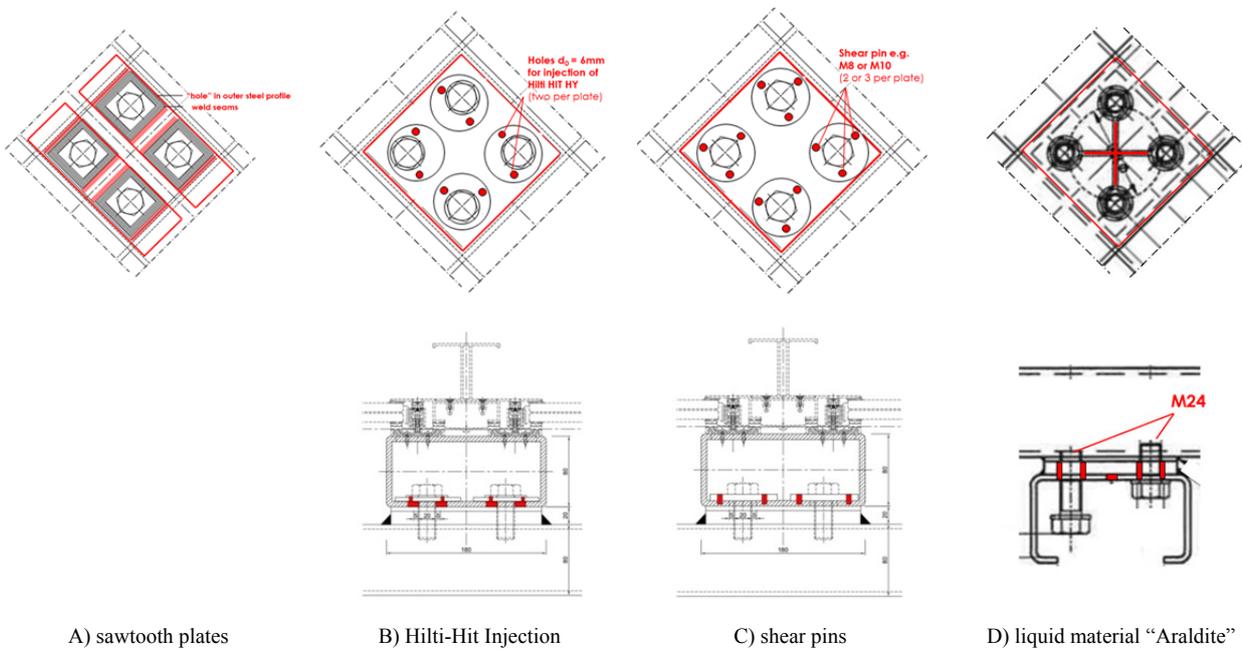


Fig. 14 Options for bolted connection.

Finally, Option D with oversized holes filled by a liquid material called “Araldite” was chosen as the design option. A mockup test was carried out in order to evaluate the range of torsional stiffness achieved with this solution (cp. Figure 15). During these tests a high plastic deformation was observed at the maximum loaded location and therefore the global structural design of the entire roof structure had to be rechecked accordingly, under consideration of reduced torsional stiffness of the grid joints.

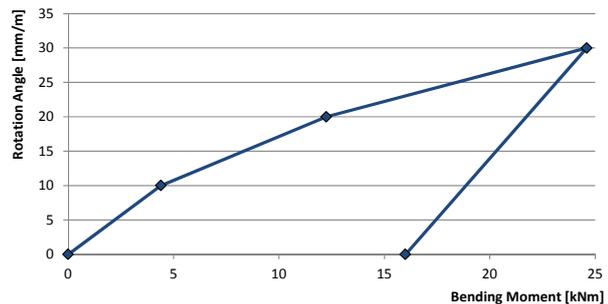


Fig. 15 Bolted connection, oversized holes filled with ‘Araldite’ (left) and load-deflection behaviour (right).

3.3.2. Edge beam

The roof structure always ends at an edge beam; the profiles are connected with a spigot and bolted connection (cp. Figure 16). The edge beam with profile dimensions 250×150mm is able to move by means of a special horizontal sliding detail. The lowest glass pane is supported by a slender horizontal profile that is connected between two outer steel profiles. First and second drainage layers are draining into a gutter; the gutter is locally supported by a special steel bracket, which also holds an outer architectural steel plate.

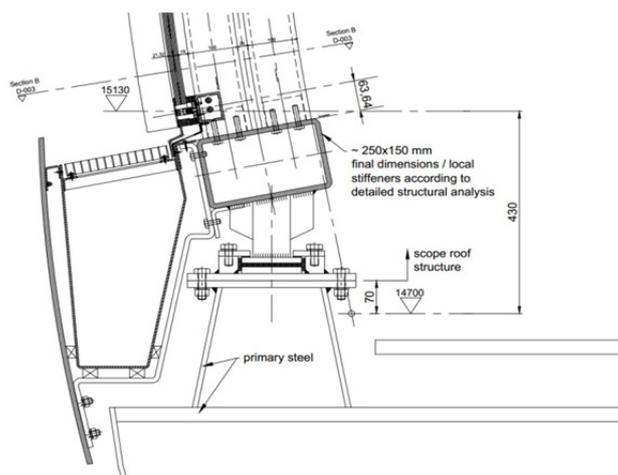


Fig. 16 Typical bearing detail (section and elevation).

4. Glass Design

4.1. Glass types

Designing with curved glass requires a series of technical considerations in order to define which bending technique is applicable for fabrication; these are predominantly shape, size, strength and coating. For example, automated glass bending processes like on-line roll bending (which use a robotic press to rapidly form and heat-treat glass along a production line), can only be used to produce single curved glass with a constant bending radius. Other geometries, like cones, spheres, paraboloids and hyperbolic paraboloids can all be formed by means of gravity bending (slumping process), but require individual custom molds to achieve different glass geometries. Heat-treatments are typically unavailable with slumped glass, since adding heat after slumping would only undo the original forming. If a design requires slumping for geometrical or optical quality reasons it can only be annealed or chemically treated glass (Timm and Chase 2014).

Since all glass panels located at the blend line are consisting of two different radii, automated glass bending processes can only be used for fabrication of glass type 1, consisting of two heat strengthened ($2 \times 10\text{mm}$) plies laminated with PVB interlayer (cp. Figure 17). All other glass panels (type 2) have to be fabricated by means of gravity bending, therefore the glass cannot be tempered and a glass build-up consisting of float glass layers ($2 \times 10\text{mm}$) laminated with sentry glass interlayer (SGP) has been chosen

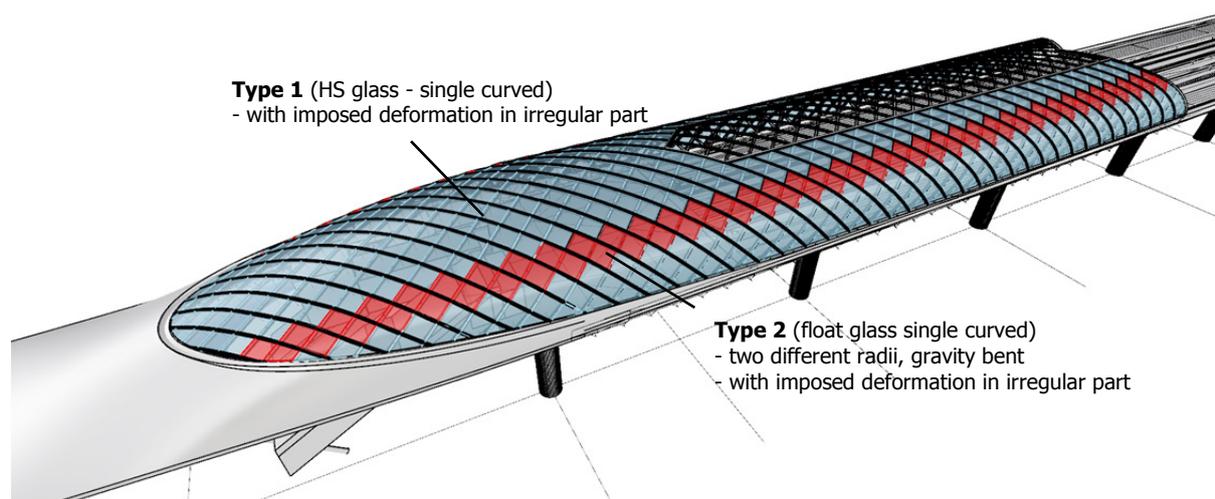


Fig. 17 Location of different glass types.

All parameters for Type 2 glazing are similar not matter whether they are located within irregular or regular part of the roof structure. The only difference is the additional imposed deformation due to the double-curved geometry within the irregular part which is based on the fact, that these glass panes must follow the geometry of the outer members at the nose of the steel structure where the global geometry is double-curved; and therefore slightly deviate from single curvature (cp. Figure 9). The imposed deformation is applied during installation by pressing one single corner of the glass panel into the position required. Imposed deformation has a long-term (permanent) load character.

The glazing system follows the outer steel beam, therefore the glass panels are two-side supported. A pressure plate consisting of an aluminum extrusion emphasizes the orientation of the steel grid architecturally while also acting as a gutter. Glass supports shall be hidden within the glass layer; the second drainage layer is continuous from the apex edge to the gutter at the base. The frame bite considers the issue that the steel profiles basically have curved edges; however the glass edges will be cut straight before bending. The unsupported edges are sealed by structural sealant silicone (SSG), dimensions 20×20mm, providing an elastic modulus in shear G of at least 0.63 Mpa. Furthermore, a design stress for dynamic shear σ_{des} of at least 0.04Mpa had to be reached in order to guarantee a proper load transfer for e.g. single point loads to the adjacent panel.

4.2. Design Concept

4.2.1. General

Design of the bent glass panels has been carried out in accordance with current European Codes and specific guidelines. Loads have been applied according to relevant parts of NEN EN 1991, whereas glass resistance has been verified by NEN 2608+C1:2012 under consideration of ‘The Guidelines for thermally-curved glass in the building industry’ (Bundesverband Flachglas 2011).

Ultimate Limit State (ULS)

$$\sigma_{res.} \leq f_{mt;u;d} \quad (1)$$

where design value of the breaking strength of annealed (equ. 2) and pre-stressed (equ. 3) glass are:

$$f_{mt;u;d} = \frac{k_e \times k_a \times k_{mod} \times k_{sp} \times f_{g;k}}{\gamma_{m;A}} \quad (2)$$

$$f_{mt;u;d} = \frac{k_e \times k_a \times k_{mod} \times k_{sp} \times f_{g;k}}{\gamma_{m;A}} + \frac{k_e \times k_z \times (f_{b;k} - k_{sp} \times f_{g;k})}{\gamma_{m;V}} \quad (3)$$

with:

- k_e edge quality factor of the glass pane
- k_a load-area-effect factor
- k_{mod} modification factor depending on the load duration and the reference period
- k_{sp} surface factor of the glass pane
- $f_{g;k}$ characteristic value of the breaking strength of glass, here $f_{g;k} = 45\text{N/mm}^2$;
- k_z zone factor of the glass pane
- $f_{b;k}$ is the characteristic breaking strength of pre-stressed glass given in N/mm^2
- $\gamma_{m;A}$ glass material factor, which can take the following values:
 - $\gamma_{m;A} = 1.8$ in cases where wind is the dominant variable load
 - $\gamma_{m;A} = 2.0$ in all other cases
- $\gamma_{m;V}$ is the material factor of the pre-stress of tempered glass, where $\gamma_{m;V} = 1.2$

The shortest load duration for each load combination is the driving factor to determine k_{mod} . Application of equation 2 and 3 respectively is resulting in design values of breaking strengths as summarized in the following table.

Table 1: Design values of breaking strengths for float (ANG) and heat-strengthened glass (HSG) according to NEN 2608 + C1:2015.

No.	Load combination (LCC) [-]	Load Duration [-]	Float Glass (ANG) $f_{mt;u;d} [\text{N/mm}^2]$		Heat Strengthened Glass (HSG) $f_{mt;u;d} [\text{N/mm}^2]$
			surface	edge	
1	1.35 SW	50 y	6.5	5.2	19.0
2	1.2 SW + 1.35 S	1 month	9.8	7.8	22.3
3	0.9 SW + 1.35 W	5 sec	25.0	20.0	37.5
4	1.2 SW + 1.35 W ⁺	5 sec	25.0	20.0	37.5
5	1.2 SW + 1.5 V ₁	2 days	11.6	9.3	24.1
6	1.2 SW + 1.5 q ₁	1 hour	15.3	12.2	27.8
7	1.2 SW + 1.5 F ₁	1 hour	16.8	13.4	29.3

Serviceability Limit State (SLS)

$$f_{res.} \leq f_{allow} \tag{3}$$

According to the NEN 2608+C1:2012, the deformations perpendicular to the glass pane have to be limited to 1% of the span, thus $f_{allow} = \text{span} / 100$.

Determination of deflections and stresses of the glass panes has been carried out using nonlinear finite element methods (SJ Mepla Version 3.5.8). The finite element calculations to verify the stresses caused by geometrical restraints have been performed using SOFISTIK (Version 13.08-27). Within each calculation the equivalent glass thicknesses corresponding to the load duration have been taken into account.

4.2.2. Loads

Besides application of external actions such as wind, snow and maintenance loads, an out-of-plane deflection of a single corner of the glass panels (caused by global deformations of the steel structure and local geometrical restraints) had to be considered within the glass design.

The roof deformation, due to wind and snow, changes the curvature of the roof section; the most important effect occurs under snow load at the sides where two different radii in section occur in a series of glass panes made of gravity-bent float glass. This leads to a conjunction between global deformation and design of glass, which was more important than in a typical case. In general, this applies to all glass types, apart from some panels in irregular zone where the out-of-plane deflection is caused by global deformation of steel structure due to self-weight, external loads and imposed deformations due to geometric deviation in the irregular doubly-curved part at the nose of the structure (cp. Figure 9).

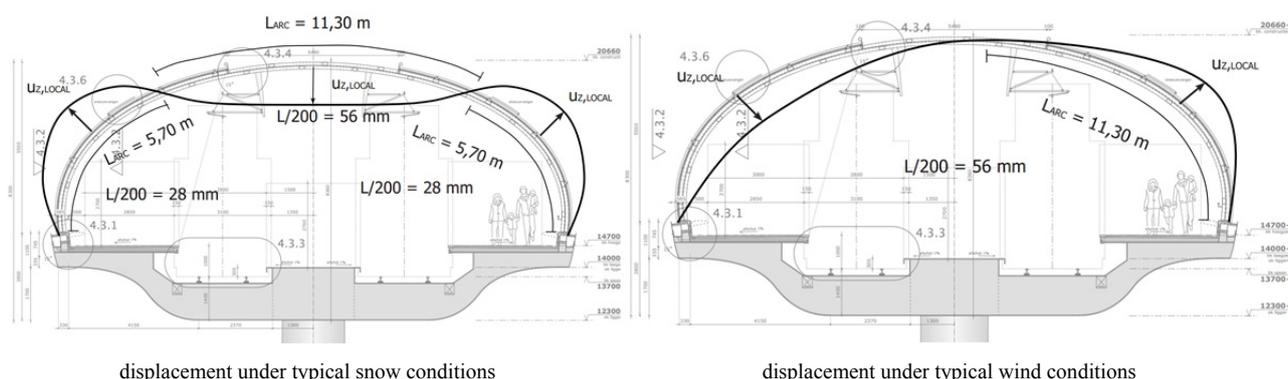


Fig. 18 Typical displacement of roof structure under snow and wind conditions respectively.

Stresses due to imposed deformation do not need to be superimposed with the stresses due to other external loads acting perpendicular to the pane, since the main tension stresses are not occurring at the same location or position (face) in the surface (cp. Figure 19).

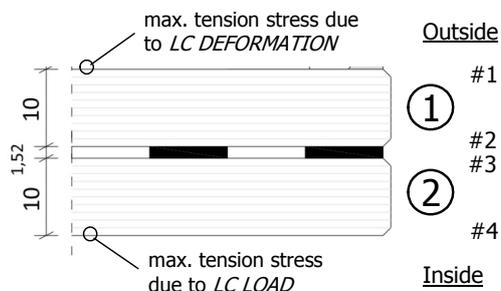


Fig. 19 Panel and position (face) numbers in glass panes.

External loads (LC LOAD) like self-weight, snow load and wind pressure are causing tension stresses at positions #4 and #2, whereas imposed deformation (LC DEFORMATION), which is always governing design, is resulting in tension stresses at #1 and #3.

The finite element model with the applied deformation is shown in Figure 20, corresponding in blue color for the non-deformed pane and in red for the deformed pane. It is obvious that the applied deformation causes stresses at #1.

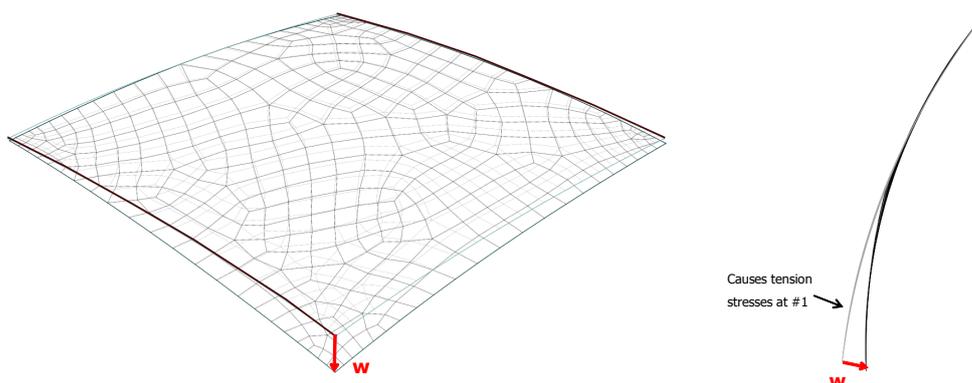


Fig. 20 Deformation due to imposed loads as applied within finite element calculation.

Load cases considered for the imposed deformation are:

- LCD 1 Maximum imposed deformation corresponding to a maximum tension stress at the edge of the glass panels of 5.2 N/mm^2 (max. allowable edge stress for bent float glass under permanent load)
- LCD 2 Maximum imposed deformation corresponding to a maximum tension stress at the edge of the glass panels of 2.6 N/mm^2 (max. allowable edge stress for bent float glass under snow load under consideration that 5.2 N/mm^2 are caused by permanent action such as self-weight and imposed deformation, cp. LCD 1)
- LCD 3 Maximum tension stress due to global deformation of the steel structure under wind load

4.2.3. Concept

Accordingly, the design concept of the stresses causes by imposed deformation is as follows:

- Determine the imposed permanent deformation that can be applied to reach a tension stress of 5.2 N/mm^2 similar to the allowable stress for permanent load (cp. LCD 1)
- Determine the imposed temporary deformation that can be applied to reach a tension stress of 2.6 N/mm^2 similar to the allowable stress for snow load left when 5.2 N/mm^2 is caused by permanent deformation (cp. LCD 2)
- Determine stresses within glass panes caused by global deformations of the steel structure due to wind loads (cp. LCD 3)
- Design of the wind loads by comparing the stresses with the corresponding allowable stresses minus allowable stresses under permanent and snow load (differentiation on the safe side)
- Determine the required modification of bending geometry of glass panes (smaller bending radius) where either permanent and / or deformations due to snow loads are exceeding the allowable limits

4.3. Design Verification

4.3.1. Glass Type 1

Single point load (maintenance load) on unsupported edge of glass panel is governing design for glass type 1, results for max. stresses and deflections are shown with the next figure. All values are within the allowable range for the chosen glass build-up, being $2 \times 10 \text{ mm}$ heat-strengthened glass with PVB interlayer.

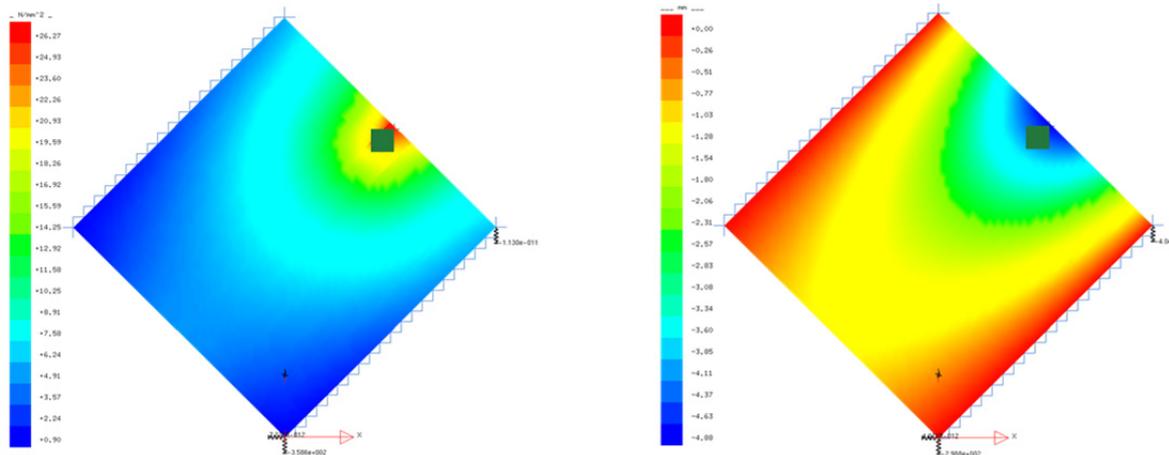


Fig. 21 Max. stresses (left) and deformations (right) for glass type 1 under governing load conditions.

4.3.2. Glass Type 2

The following table summarizes the results of the finite elements calculation for 2×10mm float glass with SGP interlayer. All stresses and deflections were obtained applying the design concept as described above.

Table 2: Results for float glass with SGP interlayer.

Load case Deformation	Tension Stress [N/mm ²]	Deflection w [mm]	equivalent glass thickness [mm]
LCD 1 (permanent load)	5.2	8	18.2
LCD 2 (snow load)	2.6	4	18.8
LCD 3 (wind load)	4.8	7	20.0

The values highlighted in blue are considered as fixed values in the calculations. Δw_w is deformation taken from global structural design of the roof.

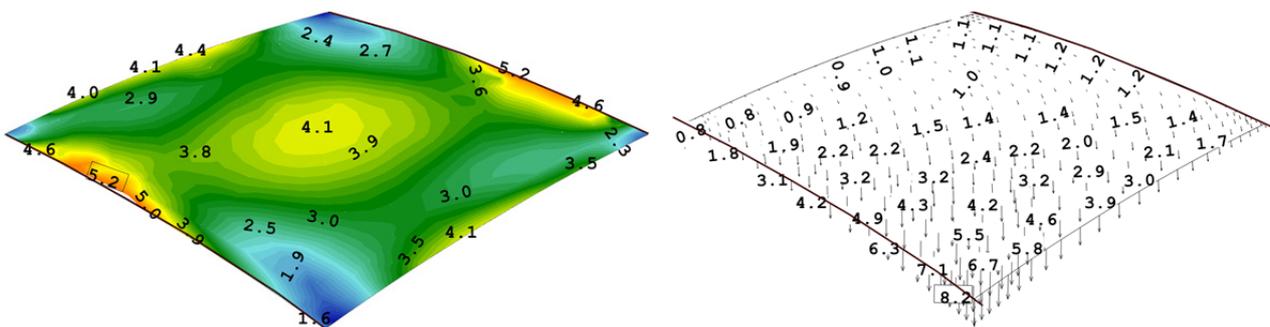


Fig. 22 Maximum tension stress [N/mm²] (left) and deformation [mm] (right) – long-term / permanent load character (LCD 1).

The following deformations (cp. Table 3) extracted from the global structural calculation of the roof need to be considered within glass design as deformations w of the corner of one single glass pane.

Table 3: Out-of-plane deformations as obtained within global structural design to be considered within glass design.

Load case	Load duration	Regular roof part [mm]	Irregular roof part [mm]
LC 0 (geometry)	Permanent	-	3
LC 1 (self-weight)	Permanent	14	13
LC 2 (snow load)	1 month	7	7
LC 3 (wind load)	5 seconds	7	1

The required modification of the bending radii in fabrication is developed off the permanent deflection and deflections due to snow load. All panels of glass type 2 where the allowable limits of 8mm (permanent action) and

4mm (snow) respectively are exceeded are therefore subject of a geometry modification. Similar to pre-cambering of steel-beams the idea behind this modification is to minimize stresses caused by constraint forces below the allowable maximum.

These modifications are highlighted within the next figure. Dimensions (bending radii) of the glass panels can be further optimized and grouped within the next step in order to reduce the number of geometry types and to minimize costs, given that the number of different modules required during fabrication has been subsequently reduced.

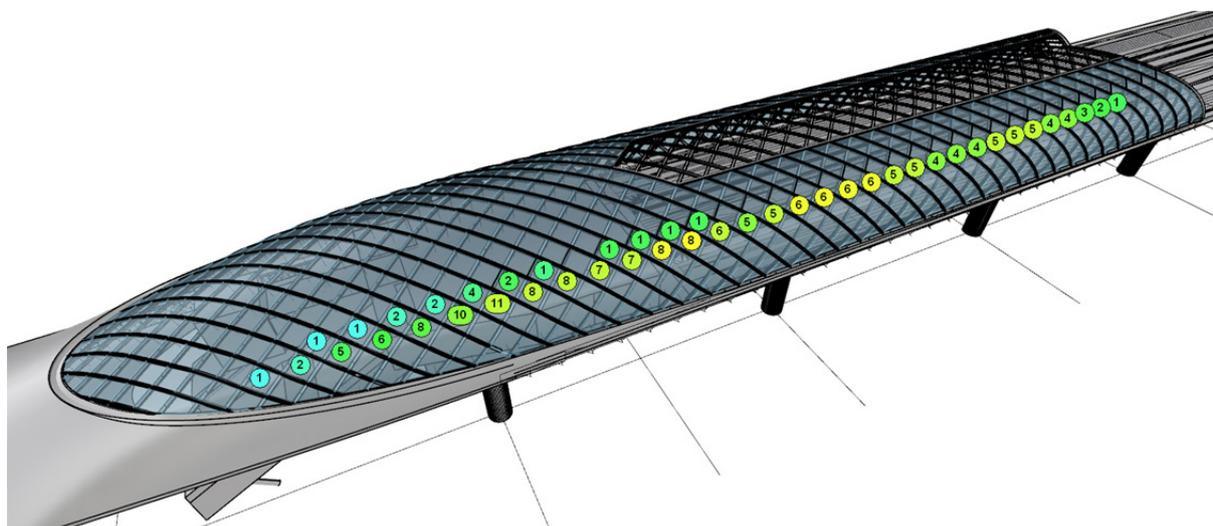


Fig. 23 Required modification of bending radii [mm].

5. Credits

- Principal: Municipality of The Hague (Netherlands)
- Client during design stages: Prorail
- Client during execution stages (Engineering & Construct): BAM Infraconsult BV
- Architect: ZJA Zwarts & Jansma Architects, Amsterdam (The Netherlands)
- Design Development and Geometry Optimization of the roof structure: Knippers Helbig GmbH, Stuttgart (Germany), ZJA Zwarts & Jansma Architects, Amsterdam (The Netherlands)
- Structural Design of the roof structure and Glass Design: Knippers Helbig GmbH, Stuttgart (Germany)
- Main Contractor: BAM Infra Nederland (The Netherlands) (part of the Royal BAM Group)
- Design, Fabrication and Erection of roof structure and glazing: Jos van den Bersselaar Constructie B.V., Brakel Atmos (The Netherlands)
- Glass Supplier: IFS-SGT (The Netherlands)
- Completion: 06.2016

References

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 NEN 2608+C1: 2012; Glass in Building – Requirements and determination method (2012)
 Timm C., Chase J.; Thermally curved glass for the building envelope; Challenging Glass 4 & COST Action TU0905 Final Conference, Louter, Bos, Belis & Lebet (Eds), Taylor & Francis Group, London, 2014, ISBN 978-1-138-00164-0, p. 141 - 149