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DR. TECHN.
OLAV OLSEN

REPORT

LANGENUEN SUSPENSION BRIDGE
ALUMINIUM BRIDGE GIRDER ALTERNATIVE

REPORT

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DR. TECHN. OLAV OLSEN
Statens vegvesen
Hydro
LEIRVIK
NTNU

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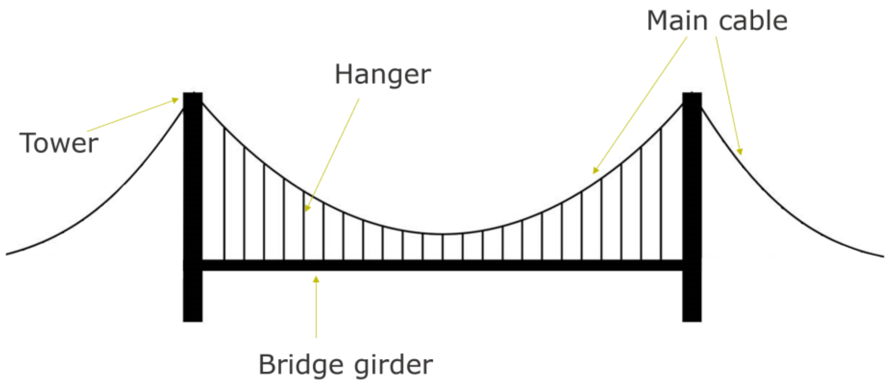
1 SUMMARY AND CONCLUSION

This study has been initiated by NPRA to investigate the feasibility and cost of substitute the use of conventional steel with aluminium as construction material for long suspension bridge girders. The project is a joint effort between Hydro, Leirvik AS, NPRA, NTNU and Dr.techn. Olav Olsen.

The Langenuen fjord crossing prospect on the west coast of Norway has been selected as an appropriate case study. This bridge is planned as a suspension bridge with a span of 1235 meters and the total length of the bridge is 1775 meters. Using aluminium girder for such a bridge span would be a ground-breaking step from current designs.

It is shown that it is feasible to meet all the relevant design criteria for the bridge, including global stability, local stability, fatigue, ultimate global stress levels, and serviceability deflections.

The Capex cost figures for the suspension bridge is similar to the steel alternative studied before. It can be seen that the aluminium bridge girder itself is slightly more costly. However, the increased cost of the aluminium bridge girder is compensated by reduced costs for the other main elements: tower, hangers and main cables.



> *Figure 1-1 Suspension bridge elements included in the cost figures*

Two different alternatives were focused in previous revision of this document: one using common principles of stiffened welded plates as for steel bridge girders, denoted "Plate Concept", and one based on friction stir welded extruded panels, denoted "Panel Concept".

In this last revision of this document, a third concept has also been evaluated, where more robust details in general has been chosen with respect to fatigue performance. This concept has been denoted "transverse panel concept" and is in its entirety described in Appendix I. Comments from 3rd party verification have been taken into account when developing this alternative. It is believed that this concept possesses a considerable potential for optimisation at a later stage. Note that for simplicity, Appendices B through H has not been modified after transverse panel concept was introduced, either because the discussions are considered general and valid also for the last concept (Appendix B,C,D,E,G,H) or they are considered irrelevant (Appendix F).

Table 1-1 below, a summary of estimated cost is given for the girder, towers, hangers and cable cost.

> *Table 1-1 Overall Cost Figures (+-25%)*

Concept	Cost Bridge Girder [MNOK]	Cost of Towers, Hangers and Cables [MNOK]	Total Cost [MNOK]
Panel Concept	816	1531	2347
Plate Concept	878	1421	2299
Transverse Plate Concept	973	1421	2394
Steel Benchmark	786	1620	2406

Any cost savings from lower maintenance costs and higher recycling potential of using aluminium alloys have not been included.

The realization of an aluminium bridge girder will be a ground-breaking step from current design practice. The need for a technology qualification program will add cost and time compared with steel alternatives and this also increases the uncertainty with respect to the aluminium bridge girder cost relative to a steel bridge girder. This uncertainty is not included in the cost table above.

It is anticipated that fatigue lifetime in excess of the required 100 years is achievable for all concepts.

Critical wind velocity is above 76 m/s for the concepts investigated. Latest results from wind tunnel tests conducted in Trondheim on the applied sections this spring, indicates that the critical velocity is so high that there is a good potential for later optimisation.

A special emphasis has been put on fatigue calculations, as the properties of welded aluminium structures are in general more challenging than for steel. Critical details have been classified and reviewed, and response from both wind and traffic actions has been assessed. The calculations show that both actions contribute to the cumulated fatigue damage.

Environmental impact seems to be equal or lower than for the steel alternative. But the outcome of such evaluations varies highly based on where materials are sourced and assembled, as is the case for steel bridges as well.

2 INTRODUCTION

2.1 Background and main objective of the study

In the project Stord-Os, there has been identified a potential for further cost reductions. For the Langenuen bridge crossing, the investment cost estimate is around 5 billion NOK.

As of today, no suspension bridges with aluminium bridge girder has been built. Tailor-made solutions and technology are not yet available in the market. Realisation of this application therefore requires development of new products, and first and foremost new application of known technology and knowledge. This study is part of an R&D effort to meet this challenge.

The project hypothesis is that the aluminium bridge girder will be lighter, but maybe more costly than a conventional steel bridge girder.

Furthermore, a reduction in cost of other elements such as towers, cables and hangers may give a total benefit in the end.

Other benefits of using aluminium as listed below are discussed in this report, even though they are not included in the direct cost comparison:

- No maintenance, humidity control or corrosion/surface protection needed for the aluminium
- If end of life recycling is included, aluminium has a lower CO_2-footprint than steel, depending on sourcing location.
- Aluminium is faster to fabricate than steel

2.2 Project activities

The project includes the following main activities:

1) Review of literature and investigate previous experience with aluminium bridge girder. Identify opportunities and limitations with respect to use of aluminium
2) Document global girder stability
3) Documentation of fatigue life, subject to traffic loads.
4) Report of potential for suspension bridge in aluminium generally and Langenuen specifically. Quantification of differences in loads, volumes and costs.

2.3 Position and main dimensions

Langenuen is located about 45km south of Bergen. Several bridge locations have been investigated in the preliminary design, which is shown in Figure 2-1. Alternative B, shown with a red circle, will serve as the base case for the feasibility study. The southern crossing has later been chosen as the main crossing, but for the purpose of this study it makes little difference.

The bridge is planned as a suspension bridge with a span of 1235 meters and the total length of the bridge is 1775 meters.

> *Figure 2-1 Location of the proposed bridge crossings over Langenuen*

The bridge design shall be in accordance with relevant design rules in Eurocodes (EC) and NPRA (Norwegian Public Road Administration) publication N400 [1] and other rules and regulations by the NPRA. See Appendix H – Design Brief for further details.

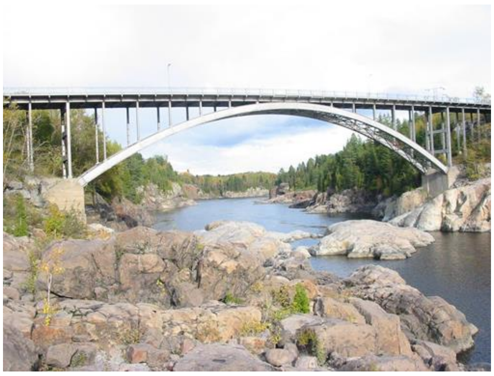
3 ALUMINIUM

3.1 Large aluminium structures

Aluminium has been used in bridge structures for decades, starting with the Pittsburgh's Smithfield Street Bridge in 1933 [2]. In order to increase the load-carrying capacity, an existing steel and wood deck was replaced by a riveted aluminium construction.

In 1950, the all-aluminium bridge crossing the Saguenay River in Arvida shown in Figure 3-1 was completed [3]. The decision to build the bridge in aluminium was based on the bridge being located in the centre of the Canadian aluminium industry and was supported by Alcan. Although the cost of the aluminium alternative was higher than for a steel bridge, it was expected that this would be offset over time through significantly lower maintenance cost. The bridge is still in operation, and recent reports have verified that only minor maintenance work has been required [3] [4].

The first all-aluminium bridge in Norway, the Forsmo bridge shown in Figure 3-2, was built by Leirvik AS and completed in 1996 and was developed in cooperation with the R&D program Expomat M11 Brokonstruksjoner [5].



> *Figure 3-1 Arvida Bridge*

> *Figure 3-2 Forsmo bridge (NTNU)*

Although some of the early bridges were built only to promote aluminium as a structural material, properties such as density and corrosion resistance makes aluminium a durable material well suited for bridge structures [6] [7] [8]. In rehabilitation of existing bridges, lightweight aluminium deck structures are used to increase the load-carrying capacity of the bridge without replacing fundaments nor main structure. Moreover, reduced weight allows for installation of larger sections, and thereby reduced installation time and cost.

Due to its excellent corrosion resistance, surface treatment is not required for aluminium bridge structures [4]. As demonstrated by the Arvida Bridge, this represents a significant reduction in operational cost compared to steel structures that required periodical replacement of surface coatings.

The favourable properties of aluminium are also beneficial in other large structures. Especially within marine and offshore applications, combination of low weight and excellent corrosion resistance offer competitive solutions. Recent applications are illustrated in Figure 3-3 and Figure 3-4.

> *Figure 3-3 Aluminium living quarter structure from Leirvik AS, size 92 m x 28 m x 30 m (total of 1500 tons aluminium material)*

> *Figure 3-4 MS Ampere hull at Fjellstrand AS*

3.2 Aluminium alloys

Pure aluminium is relatively soft. Its strength can be increased by alloying. Most of the commercially available aluminium is therefore alloyed with one or more other elements. The resulting alloy has different properties depending on which alloying elements are added. This makes the choice of alloy an important matter.

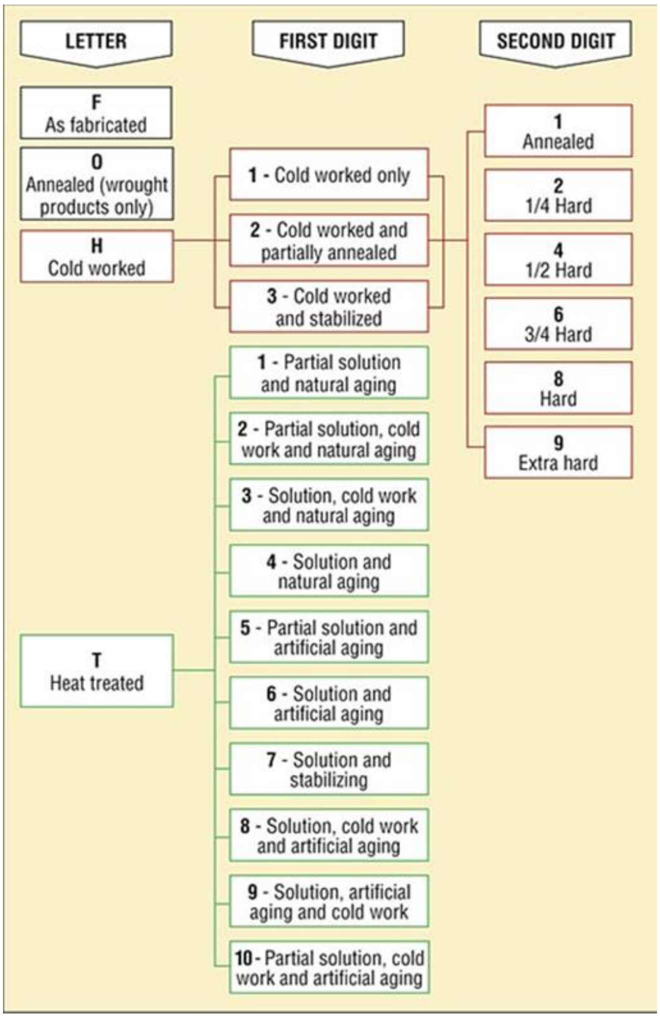
In the long-established international system for identifying aluminium alloys, the first digit in a four-digit alloy code identifies the major alloying element. The European standard uses the same codes. The general system is outlined in Table 3-1.

> *Table 3-1 Aluminium alloying series*

Alloying element	Alloy code	Alloy type
Pure aluminium	1000 series	Non-hardenable
Copper	2000 series	Heat-treatable
Manganese	3000 series	Hardenable through deformation hardening
Silicon	4000 series	Hardenable through deformation hardening
Magnesium	5000 series	Hardenable through deformation hardening
Magnesium + Silicon	6000 series	Heat-treatable
Zinc	7000 series	Heat-treatable
Other alloying elements	8000-series	

The non-heat-treatable alloys are mostly used for rolling, since the only way to increase their strength is by cold working. In extrusion, on the other hand, hardenable alloys are the most commonly used. Hardenable alloys achieve their final strength as a result of solution heat treatment and ageing (precipitation hardening). Solution heat treatment is carried out by heating to high temperature. This can be done as a separate process but is normally done during extrusion. Ageing (heat-treatment) is then carried out in special furnaces over a period of a few hours.

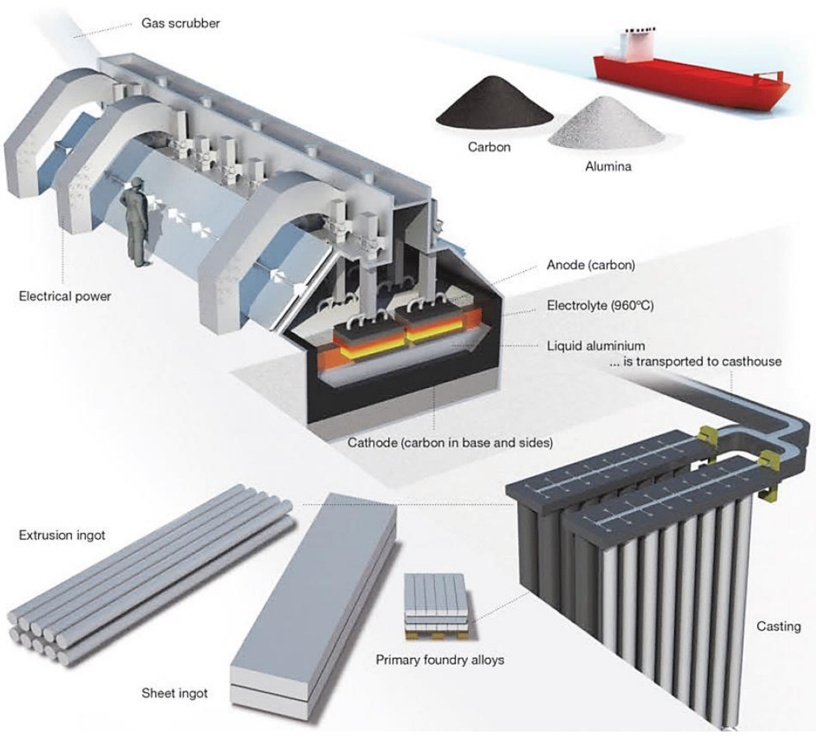
A description of the different temper designations for aluminium alloys is shown in Figure 3-5.



> *Figure 3-5 Aluminium temper designation*

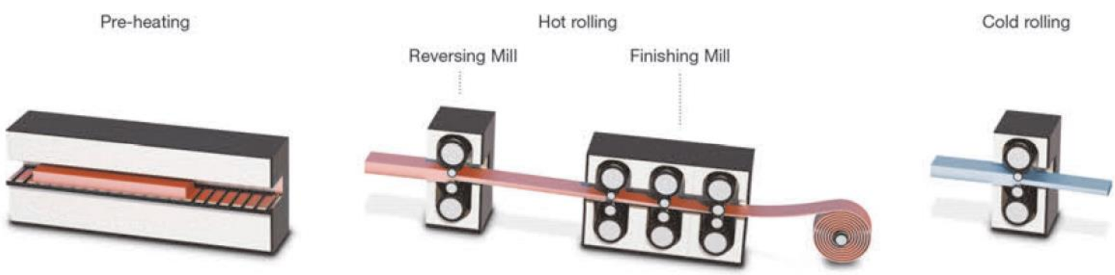
3.3 Aluminium production

Primary aluminium is extracted from alumina raw material through the smelting process illustrated in Figure 3-6. Depending on the further processing chain, the material is cast into extrusion ingots, sheet ingots and foundry alloys.



> *Figure 3-6 Aluminium smelting process (Hydro)*

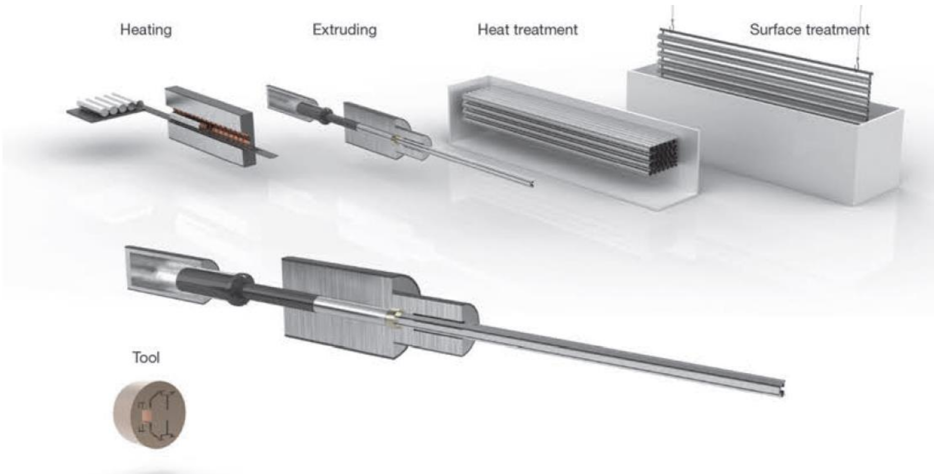
Large aluminium structures like a suspension bridge girder or a ship hull, are mainly based on flat-rolled plates and extruded profiles. Figure 3-7 shows the typical process route for rolled products. The ingot is prepared for rolling by removing ends as well as top- and bottom surfaces, and then preheated. The material is then rolled to the correct thickness in a set of rolling steps. While some customers receive coiled material directly from the rolling mill, some material is further processed to provide flat sheets in requested dimensions.



> *Figure 3-7 Aluminium rolling process (Hydro)*

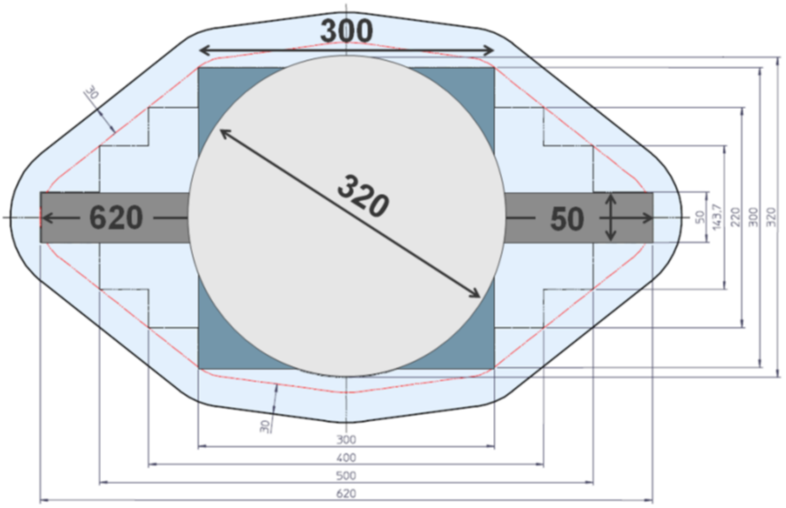
Profile extrusion starts with ingots of aluminium alloy. These are cut into billets, which are then heated in an induction furnace to the right extrusion temperature of 450-500°C. The heated billet is then forced through a die under great pressure, and the finished profile is squeezed out of the die rather like toothpaste from a tube. The profile emerges at a speed of 5-50 meters per minute and is normally between 25 and 45 meters long. The profile is cooled using air or water as it leaves the die. After cooling, the profile is stretched to relieve any stress and to give it the desired straightness. The quality of the surface and any dimensions that are important to the function of the profile are checked at the same time. The profile is then cut to a suitable length or to the length requested by the customer. The final strength of the material is controlled by natural or artificial ageing.

The aluminium extrusion process may also include surface treatment such as anodizing. As aluminium bridge girders do not require any surface treatment however, this step is not included. Bridge parts like handrails, stairs etc. will however benefit from the smooth and aesthetical surface given by analyzation.



> *Figure 3-8 Aluminium extrusion process (Hydro)*

The maximum outer dimensions of the extruded profile are limited by the extrusion press mouth size. A typical limitation chart is shown in Figure 3-9. In addition to limitations on outer dimensions, limitations also apply to maximum cross-sectional area. For larger cross-sectional area, the possible extrusion length may be limited by the maximum billet length the press can handle. All these limitations need to be communicated with the extrusion supplier during the design phase.



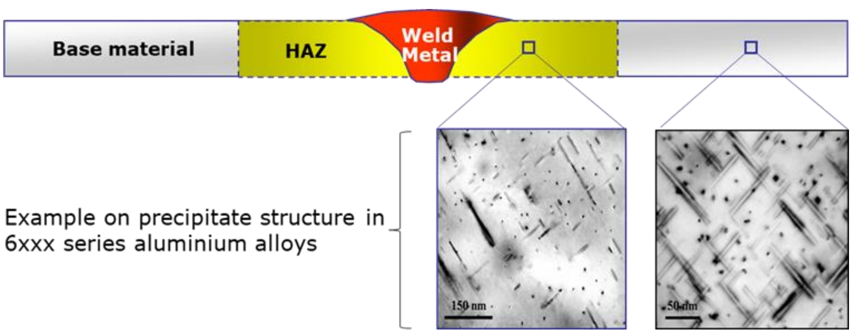
> *Figure 3-9 Example of allowable profile dimensions at one extrusion plant (Hydro)*

3.4 Fusion welding

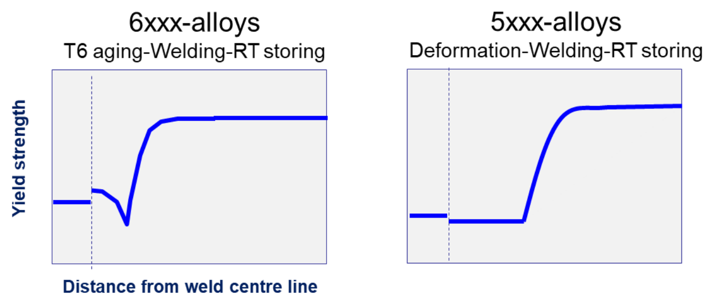
Aluminium is very suitable for welding. However, the strong oxide layer, high heat capacity and excellent thermal conductivity of aluminium mean that welding procedures differ from those of other metals. When welding aluminium, precautions must be made for the oxide layer that is present at the aluminium surface due to the metal's reaction with oxygen and the resulting oxide that is quickly formed. The oxide is strong, has a high melting point (approx. 2 050°C) and can easily cause welding defects. It is therefore important to either remove oxides from the joint surfaces before welding, or to keep the thickness of the oxide layer thin and consistent by proper storing of the aluminium parts before welding.

A common challenge in aluminium welding is reduced mechanical properties of the welded joint compared to the base material. As illustrated in Figure 3-10, the welded cross-section consists of the weld metal which is a mixture of the base material and the filler material, and the heat-affected zone (HAZ). While the mechanical properties of the weld metal can be altered by selection of filler metal alloy, the HAZ properties deviates from the base material due to heat input from the welding process.

For heat-treatable alloys, the main effect is precipitation reactions at elevated temperatures, where a low number density of coarse precipitates leads to reduced strength. For non-heat treatable alloys, the main effects are recovery from the deformation hardening process and recrystallization. Figure 3-11 shows typical yield strength profiles for welded joints in 6000- and 5000 series alloys.



> *Figure 3-10 Cross-section of welded joint*

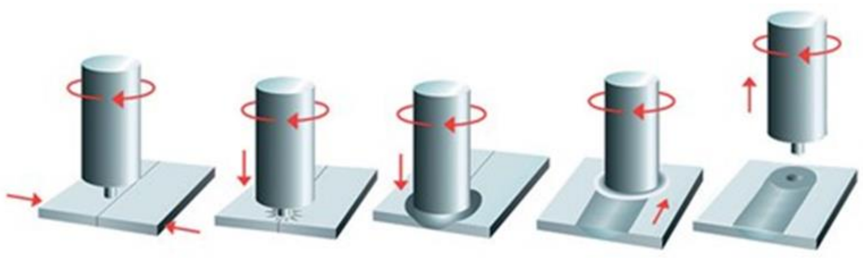


> *Figure 3-11 Yield strength profile in welded joint (Hydro)*

The reduced properties in the heat-affected zones are included in allowable stress levels defined by standard such as Eurocode 9. In general, the mechanical properties assumed for calculation of a welded structure is similar to the O-temper for non-hardenable alloys and T4-properties for hardenable alloys. Although the reduced properties are limited to an area close to the fusion line, calculations are commonly performed assuming HAZ properties in the entire structure. The negative effect of HAZ can for longitudinal welds of extruded profiles be compensated by increasing material thickness locally on both sides of the weld.

3.5 Friction stir welding

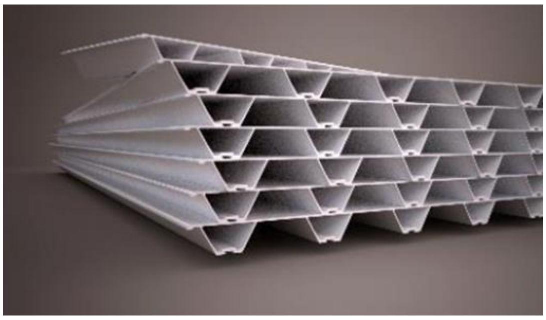
The friction stir welding process was developed by The Welding Institute and is based on a rotating tool being pressed into the metal and traversed along the joint. No filler metal or shielding gas is required. The FSW process takes place at a temperature below the melting point of the metal. The result is very little thermal deformation - and flat surfaces.



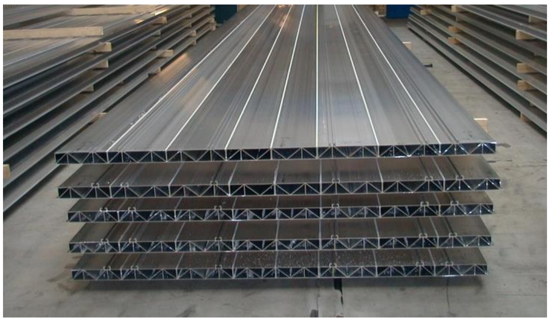
> *Figure 3-12 Friction stir welding process (Hydro)*

For large aluminium structures, friction stir welded panels are often used for reduced amount of fusion welding and improved efficiency of the assembly process. The panels consist of several extruded profiles that are joined longitudinally by FSW. By utilizing the flexibility on cross-section geometry provided by the extrusion process, the panels can be optimized for each specific application. In addition to weight optimization, this allows for integration of various functions such as backing for fusion welding in the assembly process, or mounting features for attachment of other elements to the structure.

FSW panels are either produced as single welded panels as shown in Figure 3-13 or double welded from hollow section as shown in Figure 3-14. The double welded panels can be designed to provide high bending stiffness in both directions and are often used in heavily loaded deck structures.



> *Figure 3-13 Single welded FSW panel*



> *Figure 3-14 Double welded FSW panels from hollow profiles*

3.6 Production capacities

The global primary aluminium production for 2017 was 63,4 million tons, whereof the European production was 7,8 million tons and the Norwegian production was 1.2 million tons [9].

The value chain for production of semi-finished aluminium products also includes large amounts of recycled material, whereas the total volume exceeds the primary material production. The global production of aluminium semis for 2017 was 77,8 million tons, within the product types and areas as shown in Table 3-2.

> *Table 3-2 Production of aluminium semis in million tons, 2017 [10]*

Area	Extrusion	Flat rolled products	Castings	Total
Global	29.7	26.3	21.8	77.8
EU	2.8	4.5	3.5	10.8

3.7 Design with aluminium

Proper material selection is crucial, but the determining factor is often the affordability of the product, i.e. its cost must be acceptable to the customer. Generally, aluminium is attractive in many applications, because of a favourable life-cycle cost, which is given by the sum of the initial cost of the finished product, the cost of operating or maintaining the product over its life and the cost of disposing of or recycling it after its useful life.

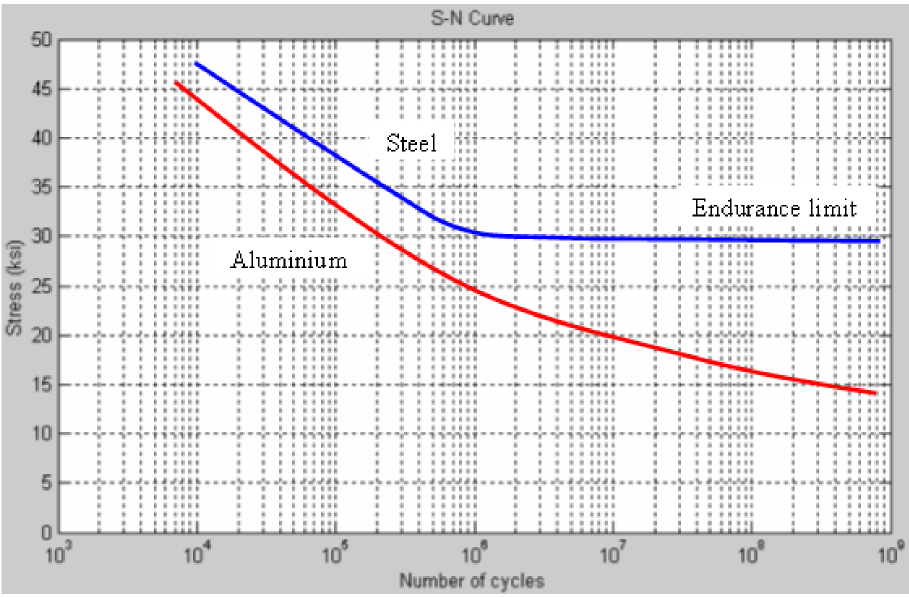
A comprehensive comparison with steel, not only in terms of cost, is important to identify the conditions and fields of application where aluminium alloys can be competitive. The main prerequisites of aluminium are:

- High strength to weight ratio: the density is 2700 kg/m3, approximately one-third that of steel;
- Corrosion resistance: the exposed surface of aluminium reacts with oxygen to form a thin, inert aluminium oxide film which blocks further oxidation; steel must always be corrosion protected in any corrosive environment.

An important parameter for comparing structural materials is the ratio between strength and density. For a given strength the increasing value of this ratio corresponds to a reduction in weight and, therefore, represents a good index of the material structural efficiency, which is favourable to aluminium alloys.

However, it is not always possible to take complete advantage of this structural benefit offered by aluminium alloys. Because of the smaller value of Young's modulus, instability phenomena are more likely to occur than in steel structures. Moreover, full utilization of weight reduction potential will require increase section height for structures where elastic deflection is the limiting factor.

Compared to steel structures, aluminium structures are more sensitive to fatigue failure when exposed to cyclic loading. As shown in Figure 3-15, the aluminium SN-curve does not show as well-defined fatigue limit as for steel. This effect is included in the fatigue limits defined by design standards such as Eurocode 9. However, aluminium structures should always be designed to avoid stress sharp transitions and stress concentration.

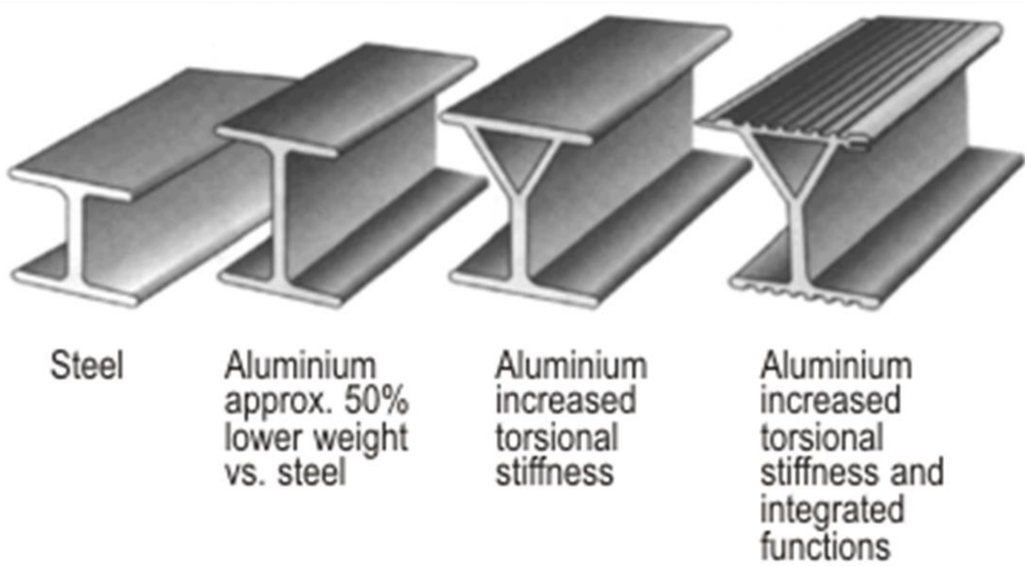


> *Figure 3-15 Representative SN-curves for streel and aluminium*

In addition to the issues described above, the following should be noted when designing aluminium structures:

- Structures made of aluminium alloys are more sensitive to thermal variations, because the coefficients of thermal expansion of this metal is twice the one of steel.
- By contrast, residual stresses produced by constraining thermal deformations are about 30% lower those in steel structures

- The extrusion process is of particular interest as it allows fabrication of profiles of any shape, contrary to steel, the shapes of which are standardized as I, H, C or L sections, being limited by the hot-rolling process. Special shapes can be built-up in steel only by welding, whereas they are easily obtained by extruding aluminium alloys. Of course, in the case of cold-formed sections, steel and aluminium possess the same advantages. An illustration of possibilities provided by aluminium extrusions is shown in Figure 3-16



> *Figure 3-16 Design of aluminium extrusion (AZOM)*

3.8 Rules and regulations

Norway is a member of CEN (European Committee for Standardization) and all CEN standards (EN) are Norwegian Standards (NS-EN). For the design of aluminium structures, the following design and fabrication standards are valid:

Design standards:

- NS-EN 1999-1-1:2007+A2:2013+NA:2009: Eurocode 9: Design of aluminium structures – Part 1-1: General structural rules
- NS-EN 1999-1-2:2007+NA:2010: Eurocode 9: Design of aluminium structures – Part 1-2: Structural fire design
- NS-EN 1999-1-3:2007+A1:2011+NA:2010: Eurocode 9: Design of aluminium structures – Part 1-3: Structures susceptible to fatigue
- NS-EN 1999-1-4:2007+NA:2010: Eurocode 9: Design of aluminium structures – Part 1-4: Cold-formed structural sheeting
- NS-EN 1999-1-5:2007+NA:2010: Eurocode 9: Design of aluminium structures – Part 1-5: Shell structures

Fabrication standards:

- NS-EN 1090-1:2009+A1:2011: Execution of steel structures and aluminium structures – Part 1: Requirements for conformity assessment of structural components
- NS-EN 1090-3:2019: Execution of steel structures and aluminium structures – Part 3: Technical requirements for aluminium structures
- NS-EN 1090-5:2017: Execution of steel structures and aluminium structures – Part 5: Technical requirements for cold-formed structural aluminium elements and cold-formed structures for roof, ceiling, floor and wall applications
- NS-EN-ISO10042:2018: Welding. Arc-welded joints in aluminium and its alloys. Quality level for imperfections

EN 1999-1-1 is currently being revised, with updates that are relevant for design of an aluminium bridge girder. Many formulas are updated, and new rules are added. Some of the main changes are listed below:

- The additional strength reduction in heat-affected zone by welding for thicknesses above 15 mm is changed to thicknesses above 30 mm.
- More exact calculation of buckling, buckling class C is introduced.
- Revised formula for buckling for welded cross-sections, in the existing standard, the reduction factor of longitudinal welding is included twice.
- Resistance of unstiffened and stiffened plates with transverse load is included.
- Interaction formulas for stiffened plates included.
- Explicit on use partial penetration butt welds in load bearing structures, and rules for resistance included.
- Rules for design of FSW included, also in Part 1-3.
- Special aluminium connections included.

The revised EN 1090-3 is now valid. The standard is generally updated to correct editorial misprints and some technical failures. The main changes from the previous standard are:

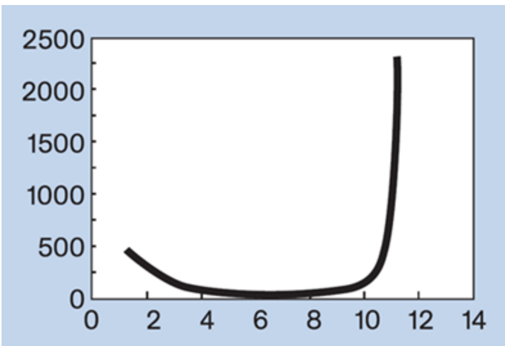
- Reference standard updated.
- Rules for single sided welds both with and without backing (partial penetration weld) included.
- Rules for FSW included.
- Acceptance criteria for UT included.
- Procedure for determination of slip factor updated.

3.9 Corrosion and maintenance

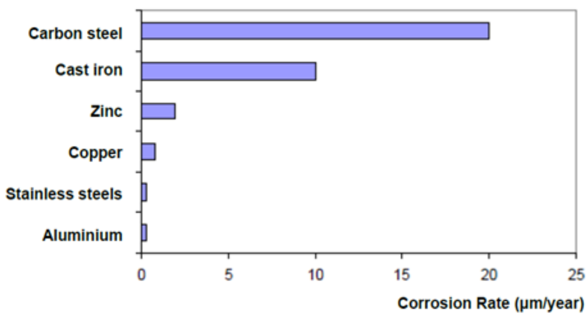
Aluminium is naturally protected – its oxide layer forms instantaneously and very effectively prevents the metal from corroding. The layer is stable in the general pH range 4-9. In strongly acid or alkaline environments, aluminium normally corrodes relatively rapidly, as illustrated in Figure 3-17.

Unlike steel, aluminium never corrodes uniformly in natural environments (Figure 3-18). Even in aggressive offshore conditions, only small localized pits occur. Consequently, uncoated aluminium sheets and profiles have been successfully used in marine environments for many decades. In seawater splash zones, corrosion pits typically do not grow any deeper than 500 µm. If some basic rules of corrosion design are followed, aluminium will resist all conceivable marine environments:

- Avoid direct contact with more noble metals
  Especially carbon steel and copper can cause severe galvanic corrosion; make sure they get separated by non-conducting materials. Stainless or galvanized steel screws cause only slight galvanic corrosion and have proven viable in many cases.
- Avoid crevices and wet spots
  Permanent contact with seawater, especially in poorly aerated crevices, can accelerate and intensify corrosion.
- Allow for natural cleaning
  The removal of salt from the surface by rain is beneficial and significantly reduces the depth of pitting.
- Add galvanic anodes
  If aluminium is permanently immersed in seawater, it must be protected by sacrificial zinc anodes.
- Clean thoroughly before painting
  Coatings are generally not required for corrosion protection but may be applied for decorative reasons. In this case, the aluminium surface should be prepared by blasting, sweeping or chemical etching before painting.



> *Figure 3-17 Corrosion rate vs. pH*



> *Figure 3-18 Uniform corrosion in marine atmosphere*

EN 1090-3, 10.1 states: “Structures made of aluminium alloys listed in EN 1999-1-1 do not need protective treatment during service under normal atmospheric conditions. Nevertheless, appropriate measures shall be taken that no corrosion or contamination occurs during execution.”

Some aluminium structures, such as bridge structures, marine applications, and power pylons, have experienced some corrosion issues. In most of the known cases, the problems have been related to unfavourable alloy selection, or galvanic corrosion as a result of incorrect connection to other metals. It is important to learn from these cases in order to avoid similar problems in future solutions.

However, experience from marine and offshore industry shows also that unprotected aluminium of alloy inn 5000 and 6000 series located above the splash zone and designed correctly have long durability and very limited corrosion. If evaporated salt from sea spray accumulates, the best way of protection is cleaning with fresh water. For an aluminium bridge 70m above sea level, it is assumed that natural rainwater only will prevent salt from accumulating on the material.

Inspection of aluminium bridge structure, both in Norway and abroad, verifies no or limited maintenance after several decades of use, ref [4], [6], [7].

4 CONCEPT DEVELOPMENT PROCESS

4.1 General

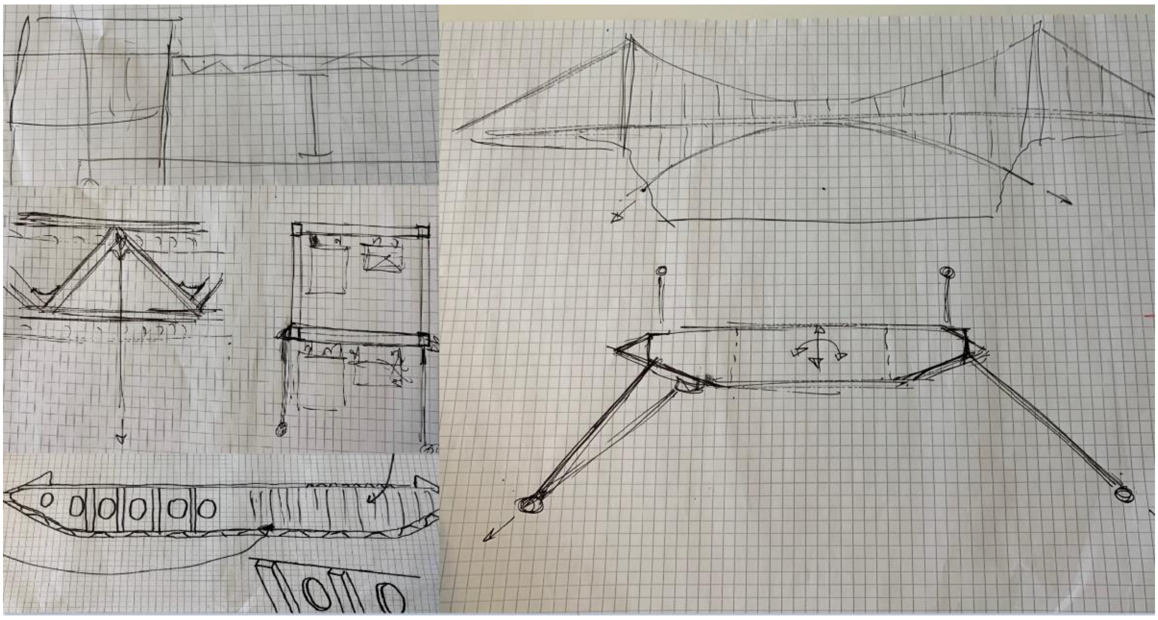
The process of developing and reporting the concepts in this study was carried out as follows:

- **Define a common ground of functional criteria, regulatory requirements, and knowledge of material and applications of aluminium**
- **Run a brainstorming process to define a handful of potential concept options**
- **Narrow down to 2-3 selected concepts to be pursued further**
- **Calculate performance, report and select as applicable**

The following sections describe details of some of the steps above.

4.2 Brain storming sessions and main concept chosen

A workshop has been held, in which different workgroups proposed concepts to be pursued. See figure for illustrations of some of the drafts.



> *Figure 4-1 Brainstorming sketches*

After maturing the proposals, and weighing against possible stability shortcomings, it was concluded that a conventional bridge girder shape was preferred to verify aero-dynamic derivatives. The non-dimensional parameters here are in the same range as in the model tests from which the derivatives are extracted. Therefore, the following concepts were proposed for further perusal:

Plate concept based on plates with stiffeners welded on. This is in line with the base case steel concept, ref [11], and enables many similar parameters to be transferred directly from that concept.

Panel concept, based on extruded double skin sandwich profile (denoted MIR 257), joined with FSW welding, as applied for several marine applications.

Inverted cable concept. The purpose behind the use of an inverted cable was to increase the global stability with respect to critical wind speed.

4.3 Concept component evaluation

A number of configurations of the different components of the bridge have been included in concept evaluations. In Table 4-1 below, those are given, with some general comment of applicability.

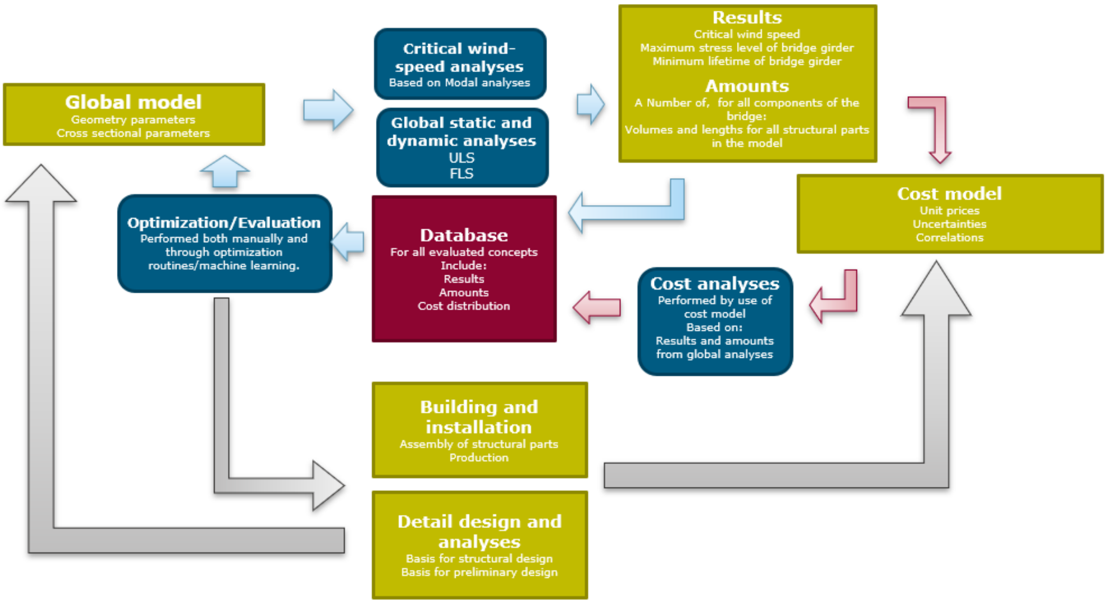
> *Table 4-1 Main concept components evaluated*

Component	**Comment**
Transverse bulkhead spacing, bridge girder	Spacing from 4 to 12 m has been included as a concept variable
Transverse stiffening	Truss structure, stiffened plate, stress-skin plate has been evaluated
Longitudinal vs transverse girder beams	To obtain sufficient stiffness, both longitudinal and transverse girders have been evaluated
Bridge girder skin layout	Both stiffened welded plates, extruded double skin, framed stress-skin and combined solutions have been evaluated
Hanger spacing	Hanger spacing of 12 and 24 m has been evaluated
Tower height and cables layout	Height of tower and cable geometry and number has been varied freely in the iterations
Assembly Section Dimension	Typical bridge girder section size similar to steel has been used (~100 l length)

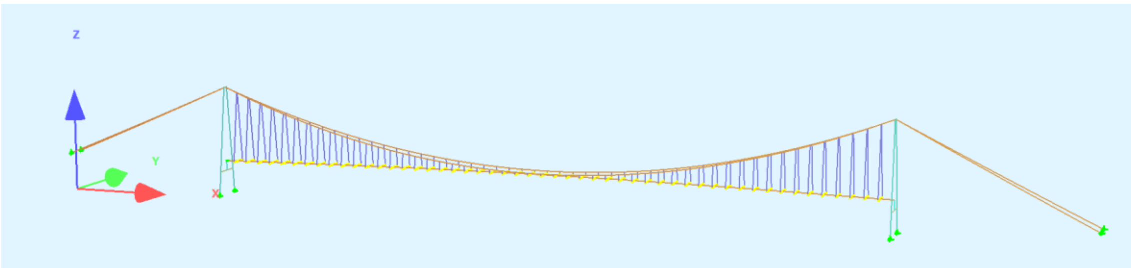
4.4 Concept performance calculation process

A schematic representation of the concept development process is given below. Where possible, the iteration processes have been automated to decrease turnaround time for every iteration and enable screening of a large number of variations. Based on a high number of coupled analyses and design checks executed, a neural network has been educated, to increase the efficiency of iterations even further.

The cost evaluations have been done both on a deterministic and statistical basis to increase understanding of governing parameters.



4.5 Parametrized global model



> *Figure 4-2 Example of global model*

The global analysis model seen in Figure 4-2 is based on parametric input values. The most relevant parametric inputs to the model are:

1. **Bridge girder cross-section (all cross-section parameters)**
2. **Prescribed tension in hangers in self-weight condition**
3. **Prescribed tension in main cables in self-weight condition**

4. **Prescribed pressure in tower top (below saddle point) in self-weight condition**
5. **Prescribed pressure in tower bottom (above foundation) in self-weight condition**
6. **Saddle point height**
7. **Minimum distance between main cables and bridge girder in the centre of the bridge.**
8. **Distance between hangers in length direction of the bridge**
9. **Spans**

4.5.1 Simplified design of hangers and concrete towers

The tower cross-sections are assumed quadratic at all elevations. The areas in the top of the tower are based on the vertical force from the main cables and the prescribed pressure, while the cross-sections areas at the bottom are based on the vertical force from the main cables as well as the self-weight of the tower and the prescribed pressure. The prescribed pressure in both tower top and bottom in self-weight condition is set to *4MPa*.

The hangers carry local weight of the bridge girder. The prescribed tension in the hangers in the self-weight condition is set to *490MPa*. The diameter of the hangers is updated based on the prescribed tension in the cables and the weight of the bridge girder.

In all cost estimations in this project, the hanger amounts are based on the assumptions given above. That is why the hanger cost vary between the concepts.

The construction phase has not been evaluated in the design of the towers. This will often be the dimensioning case for the towers. The relative cost difference of the towers is therefore not considered in this project.

4.6 Global stability evaluation

Aerodynamic stability is in general one of the critical topics and limiting factors in long-span bridge design. This has also been one of the major concerns for the Langenuen aluminium concept, since reduced mass often leads to reduced stability limit.

To determine the critical wind velocity, i.e. the wind speed at which the bridge becomes unstable, we have incorporated a comprehensive, frequency-domain procedure based on a multimode approach and unsteady forces. The framework captures coupling and interaction between modes, as well as the classical "1DOF" (1 Degree Of Freedom) flutter and static divergence and covers all the instability phenomena required in [1].

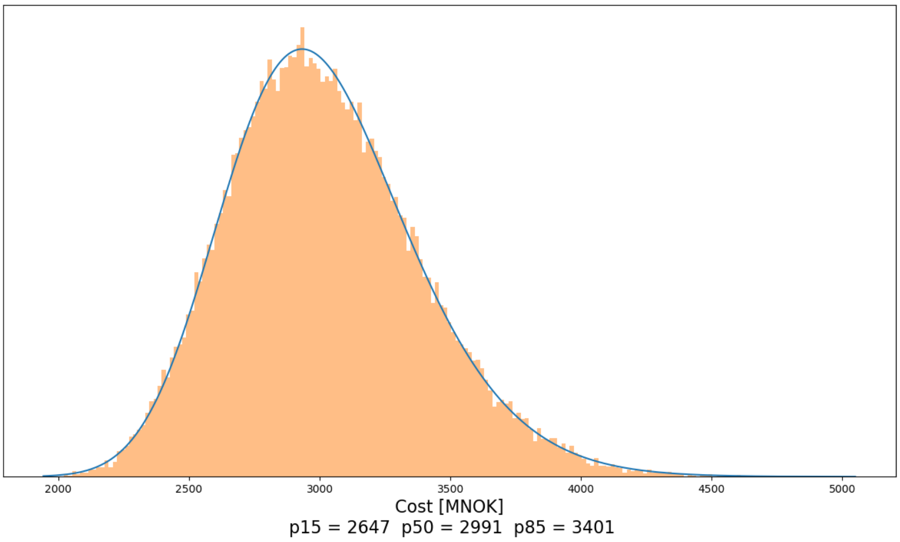
A detailed description of the theoretical principles and the practical implementation of the framework is given in Appendix G [12].

4.7 Cost estimation methodology

The cost estimation methodology is based on three main input matrices;

1) **Cost matrix:**
   - Consist of unit costs and amounts
2) **Global variables:**
   - Outer environmental uncertainties
3) **Correlation matrix:**
   - The link between the global variables and the cost matrix

Through these input matrices, cost analyses have been performed by use of Monte Carlo simulations and extract information such as the cost density seen in Figure 4-3.



> *Figure 4-3 Cost density distribution*

Other relevant information, such as correlation between uncertainties and total cost, has been extracted to pinpoint what is most important when it comes to further design.

Details of cost estimation are given in Appendix C – Cost Analysis

4.8 ULS stress and local stability estimation

Details of ULS stress and local stability calculation methodology are given in Appendix D - Global analysis and Appendix E - Local calculations, respectively.

4.9 Fatigue evaluation methodology

The fatigue evaluation performed in this project is based on the approach recommended by DNV GL for the Bjørnafjorden crossing. Due to missing information, such as relevant mean wind distributions, assumptions and simplification have been performed when found necessary.

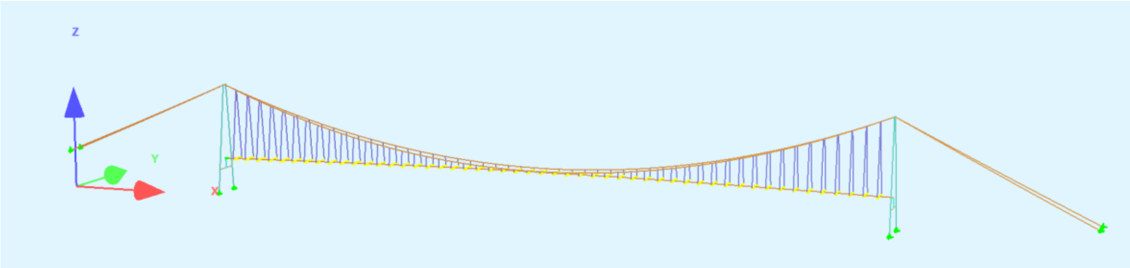
To summarize the process;

1) **Critical sections of the structure have been located and relevant fatigue curves are extracted.**
2) **The design premises with regards to the two most relevant loads (traffic and wind dynamics) are set.**
3) **Stress ranges are calculated for traffic and wind dynamic.**
4) **Design life calculations have been performed for both traffic and wind dynamics as well as for the combination of them.**

Details of fatigue evaluation methodology are given in Appendix A – Fatigue.

4.10 Sensitivity studies

A total of 1300 analyses have been performed to map the behaviour of the suspension bridge with respect to ultimate stresses, critical wind speed and cost. Each analysis is based on a set of 6 input variables that have been applied in a parametric global model seen in Figure 4-4. Each of the variables has been picked randomly from reasonable pre-set intervals.

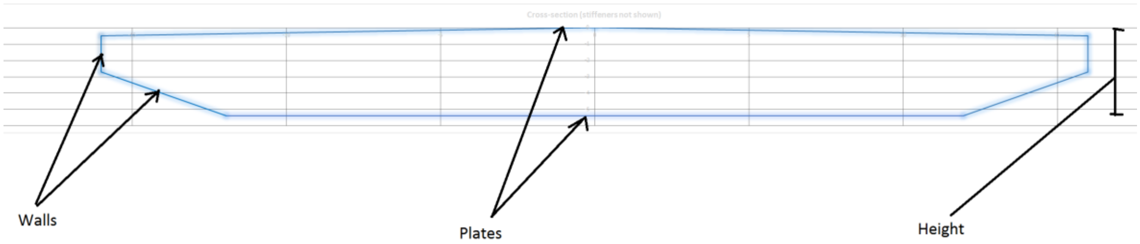


> *Figure 4-4 Parametric global model*

The six variables and their intervals are presented in Table 4-2. A description of the cross-section parameters is seen in Figure 4-5.

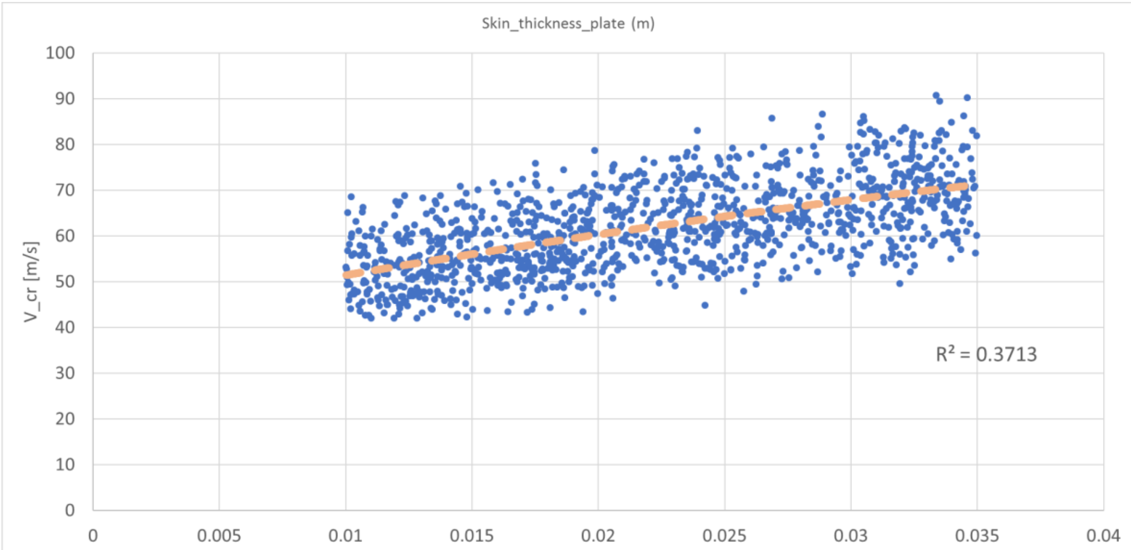
> *Table 4-2 Input to sensitivity study*

Properties	Low	High	Unit
Thickness of bridge girder cross-section walls	10	35	mm
Thickness of bridge girder cross-section plates	10	35	mm
Height of bridge girder	3	6	m
Prescribed tension in main cables	300	500	MPa
Saddle elevation	150	235	m
Minimum distance between main cables and bridge girder	4	20	m



> *Figure 4-5 Parametric cross-section model*

The main finding in this study was that increasing the plate and wall thickness, as well as the girder height, was the most efficient way to increase the critical wind speed. See Figure 4-6. Even though it increases the cost of the main girder significantly they proved to be the most efficient cost measures, considering all costs (tower, main cable). It also has a positive effect on the stress level and fatigue life.



> *Figure 4-6 Critical wind speed with respect to the skin thickness of the top and bottom plate*

More details of the sensitivity study are given in Appendix B – Optimization study.

4.11 Aspects not included in evaluation

For simplicity and to gain maximum benefit for the effort, several aspects considered similar to steel or not governing has been omitted from further evaluation in this study.
A list of those is given in Table 4-3.

> *Table 4-3 Aspects not included in evaluation*

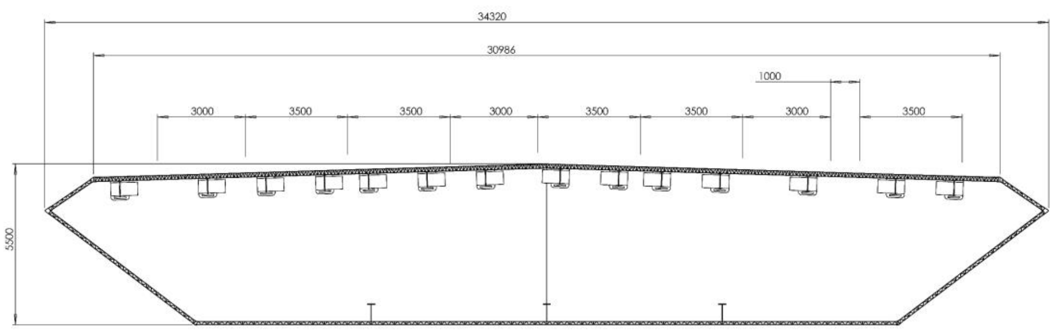
Aspect	**Reasons for exclusion**
Geotechnics and foundation	Similar to steel alternative
Wave, current	Not governing for design
Architecture	Similar to steel alternative
Viaduct, road entrance	Similar to steel, not assessed further here
Outfitting	Discussed on cost section only
Spoilers/Strikes	Omitted for later optimisation
PLS and SLS limit states	Other limit states governing for bridge girder, tower, stays and cables
Temporary phases	

5 MAIN CONCEPT DESCRIPTION

5.1 Panel concept

5.1.1 Geometry

The basic strategy behind this concept is to use the possibilities of extruded aluminium profiles in combination with Friction Stir Welding (FSW).
Using extruded profiles makes it possible to optimize material thickness and geometry, thereby reducing weight and stress levels in the design.
By using panels made from hollow profiles, a very stiff structure with little need for supporting structure is created, meaning reduced weight and assembly time. The thermal distortion is also reduced or eliminated by using the stiff panels.
This type of design concept (aluminium sandwich panels) is widely used for other applications with high demands for structural strength (train car bodies, ship decks, bulkheads, offshore living quarters etc.).

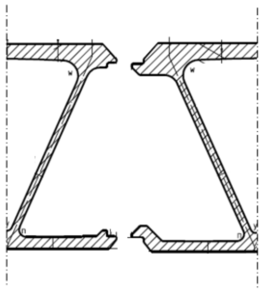


> *Figure 5-1 Panel concept, cross-section*

The overall dimensions of the bridge girder cross-section are 34 m x 5,5 m.
The bridge girder is built up from 12m long sections with a bulkhead every 12 m where the cables are connected via hangers. All profiles and panels are extruded in 12 m length, which minimizes the welding and number of joints. The sections are assembled together to a 120 m long module, which is to be transported to the bridge site and connected to the rest of the bridge structure. See also Chapter 8.

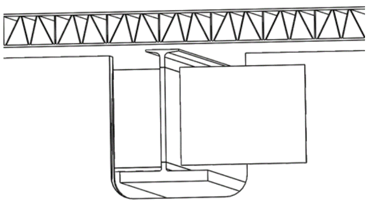
To compensate for the lower Young's modulus compared with steel, the height of the bridge girder has been increased from 4 to 5,5m.

The panels on top (deck panels) are built up from 150 mm high extrusions that are FSW together to panels 12 m x 3,09 m. When MIG welding these together, weld preparation details (root support etc.) are integrated in the sides of the panels to simplify the joining process.



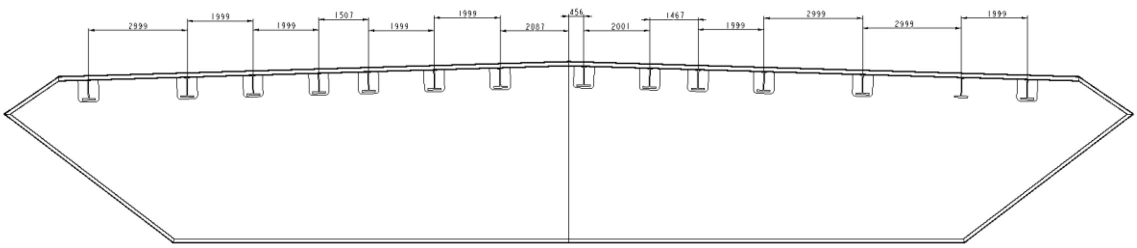
> *Figure 5-2 Example of weld preparations that are integrated in the profiles*

The deck panels are resting on longitudinal I-beams that are attached to the underside of the deck panels. These I-beams are located to match the position of the wheels of the heavy vehicles (trucks) to reduce the stresses in the deck panel joints.
The load from the deck panel goes via the I-beams and is transferred to the Bulkhead panel via shear plates or gusset connections. Those details have not been studied in detail in this phase.



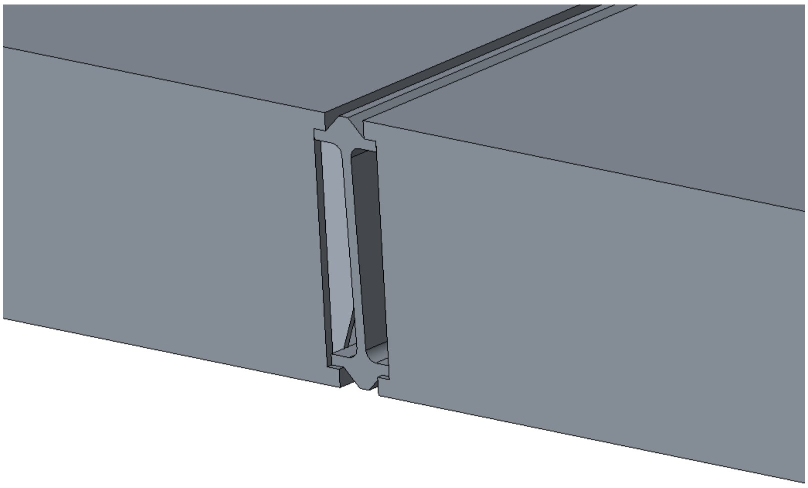
> *Figure 5-3 Shear plates to connect the I-beams to the bulkhead panels*

Outside the heavily loaded areas the I-beams have bigger lateral spacing, see Figure 5-4.



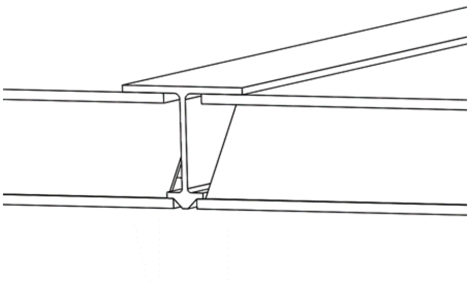
> *Figure 5-4 Distribution of lateral spacing between longitudinal beams*

The deck panel transversal joints have been located 1 m away from the bulkhead, thereby minimizing the bending stresses in the joint. The transversal joint welding is simplified by using a weld support profile that also provides integrated root support (see Figure 5-5). This profile provides a backing for the weld that gives it a better weld detail with respect to fatigue (welded one side only, full penetration with backing, see Appendix A) than without a backing. Additionally, the joiner profile provides better tolerances and assembly conditions.



> *Figure 5-5 Weld support profile for transversal weld of deck panels*

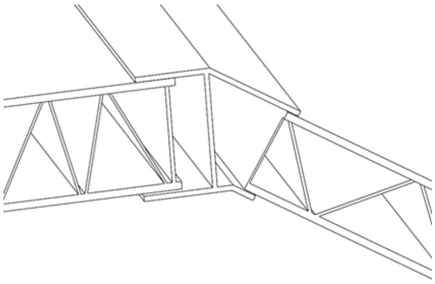
The weld support profile in Figure 5-6 is suggested for the side and bottom panels. This joint type has less fatigue strength (fillet welds on one side). However, these locations are not exposed to local stresses from traffic and are therefore expected to be less critical with respect to fatigue. The detail in Figure 5-6 is better for assembly, as the joiner profile will rest on the flange.



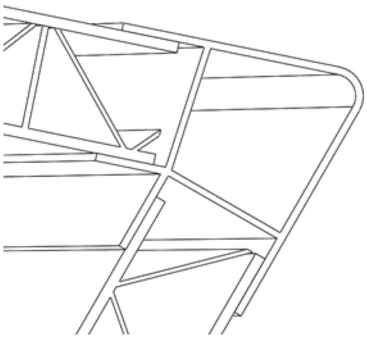
> *Figure 5-6 Weld support profile or transversal weld of side/bottom panels*

Side panels and bottom panels are built up similarly as the top panels, but with 115 mm thick FSW panels, 12 m x 2,4 m.
The bottom panels have three T beams to give sufficient buckling resistance in the lower part of the bridge girder (marked blue in the picture below).

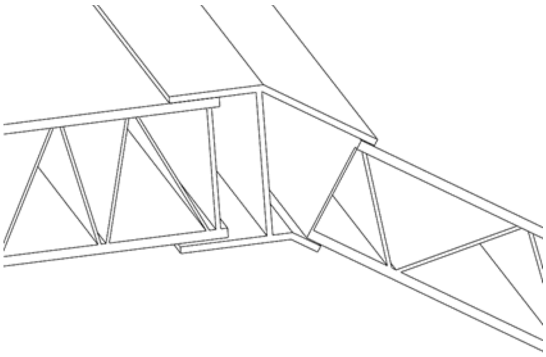
In the corners of the bridge girder, specially designed joiner profiles are used to join the different panels (marked yellow in the picture below). The purpose of these is to simplify the corner welding, while giving possibility to compensate for tolerances of the individual panels and the assembly process (see pictures below). An alternative to these joiner profiles could also be bent plates that are places on in- and outside of the girder at the corners.



> *Figure 5-7 Joining deck panel to upper side panel using joiner profile*



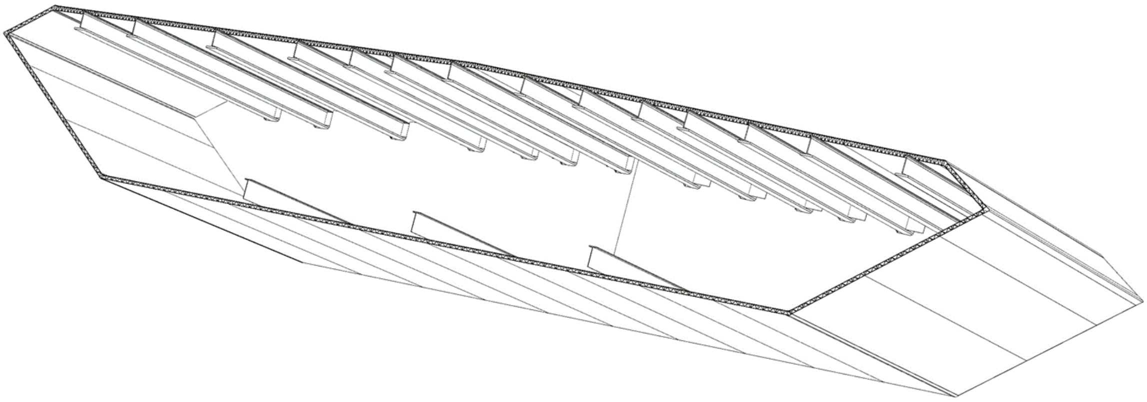
> *Figure 5-8 Joining upper side panel to lower side panel using joiner profile*



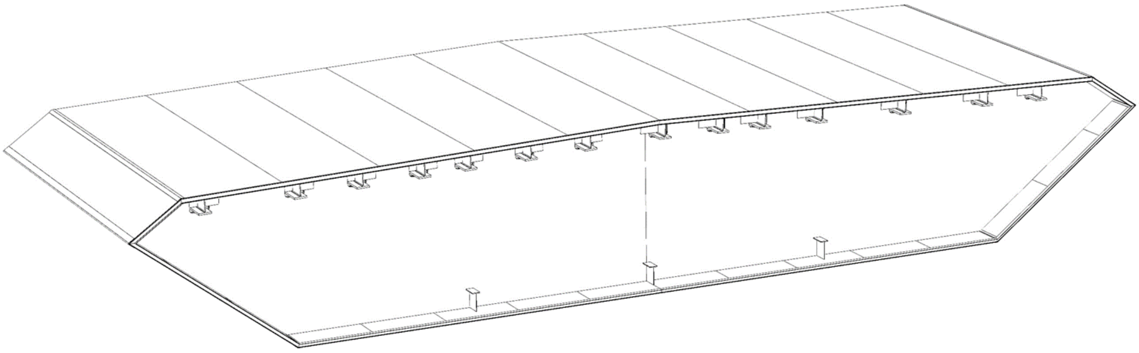
> *Figure 5-9 Joining lower side panel to bottom panel using joiner profile*

The transverse bulkhead is built up from the same panels as is being used in the sides and bottom. The reason for this instead of a traditional truss design is to simplify assembly (reducing MIG welding).

The bulkhead panel is welded directly to the inner-skin of the adjacent sandwich-panels on all sides.



> *Figure 5-10 Main view of a girder section*



> *Figure 5-11 Main view of a girder section with bulkhead*

See Appendix M - Panel concept drawings for more drawings of the panel concept.

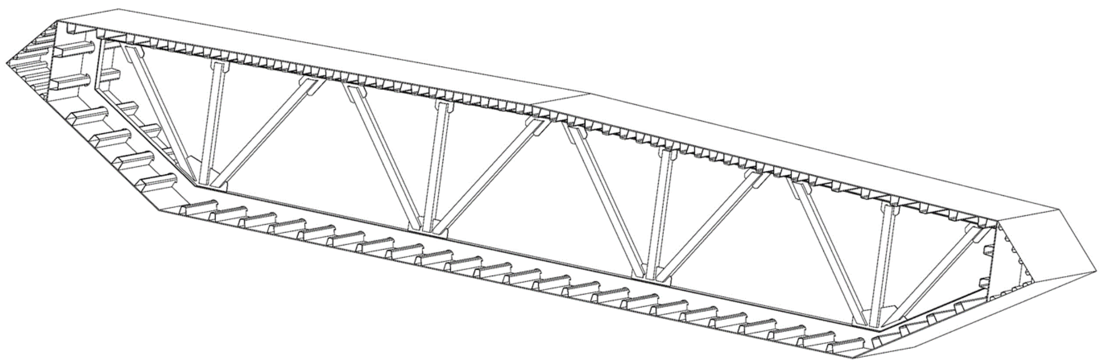
5.1.2 Material

The alloy that is intended to be used for all profiles is EN AW6005A-T6. A possible alternative could be EN AW6082-T6, but as the structural strength in the HAZ of the welds is quite similar for these two alloys it is not worth the extra cost for the 6082 alloy. Gussets etc. can be made from 5083 alloy or similar that is very weldable to the 6000-alloys. This is not detailed further here.

5.2 Plate concept

5.2.1 Geometry

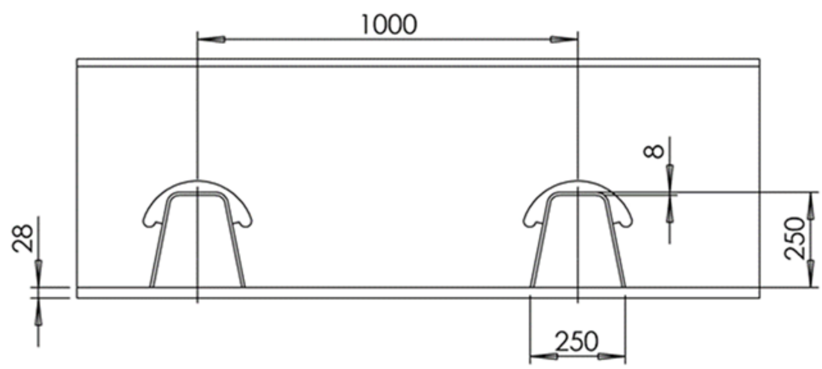
The plate concept is based on a more standardized bridge girder cross-section design approach as seen in Figure 5-12.



> *Figure 5-12 Plate concept cross-section (4m section)*

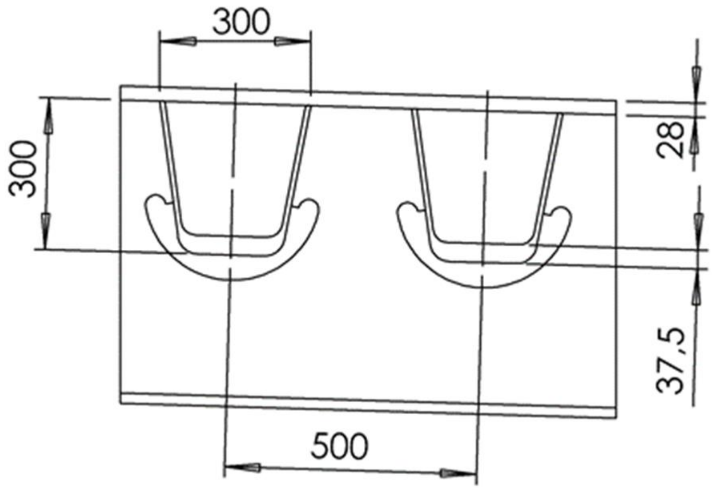
The cross-section skin consists of stiffened plates with a thickness of 28 mm. The skin thickness is based on a sensitivity study (ref Chapter 4.10) which showed that increasing the skin is the most economic measure in order to ensure a critical wind speed above the project criterion of 76 m/s. Also, the skin thickness increased the horizontal stiffness of the cross-section which reduces the global stresses from wind loads.

In general, the stiffeners have a maximum centre distance of 1m to avoid local buckling issues with respect to the plates as shown in Figure 5-13.



> *Figure 5-13 Detail bottom plate skin*

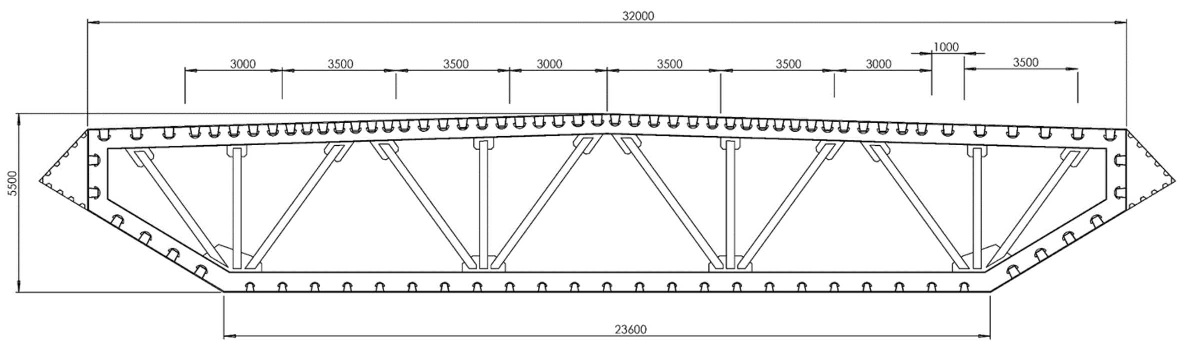
Below the road deck, the stiffeners are placed closer together and with increased flanges, so that the deck can withstand the largest traffic loads in the ultimate limit state, and to avoid large stress cycles from lorries in the fatigue limit state. See Figure 5-14. The stiffeners go through cut-outs in the transverse t-beam, and the t-beam is then later on welded on the stiffeners. This is to avoid transferring longitudinal forces through the welds. The cut-outs are shaped as seen in the figure to avoid stress concentration that could be problematic in a fatigue perspective.



> *Figure 5-14 Detail road deck*

The bridge deck is supported every 4 m with bulkheads. The bulkheads transfer the traffic loads to the outer edge of the cross-section where the bridge girder is supported by hangers every 12m.

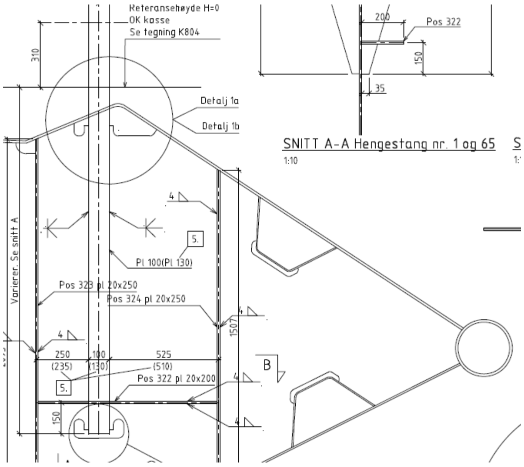
The truss-work bulkhead, as seen in Figure 5-15, consists of a T-beam going around the entire cross-section, and four sets of trusses to transfer local forces to the edges. The truss-work provides enough stiffness so that the global figures are maintained, and local buckling lengths are restrained. The T-beams have a web height of 600mm and a flange width of 250mm, both 20mm thick. The trusses are rectangular hollow sections that are 250mm wide and 12mm thick.



> *Figure 5-15 Cross-section 2D*

The spoilers/wings are as seen in Figure 5-12, not part of the structural cross-section to avoid narrow corners with respect to welding. They must be connected later.
See Appendix L – Plate concept drawings for more drawings of the plate concept.

The hanger joint has not been evaluated in detail in this project. It is a very important detail of the bridge girder, but for now, we assume that similar joints to those created for Hardangerbrua and Askøybrua (steel boxed bridge girders) can be applied for this concept as well. A 120mm thick plate is included in the material quantity for the cost evaluation in both concepts of the bridge girder, see Chapter 7.1.1 and 7.1.2 (OO: add decent cross-reference).



> *Figure 5-16 Hanger joint from Hardangerbrua*

5.2.2 Material

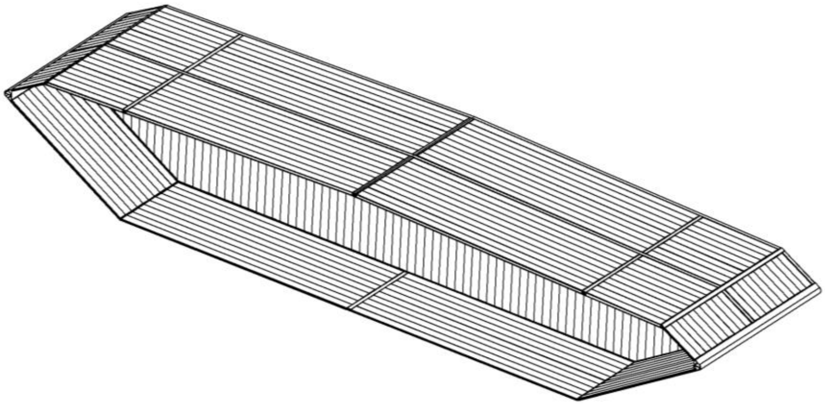
The materials applied in the plate concept is presented in Table 5-1.

> *Table 5-1 Material applied in the plate concept*

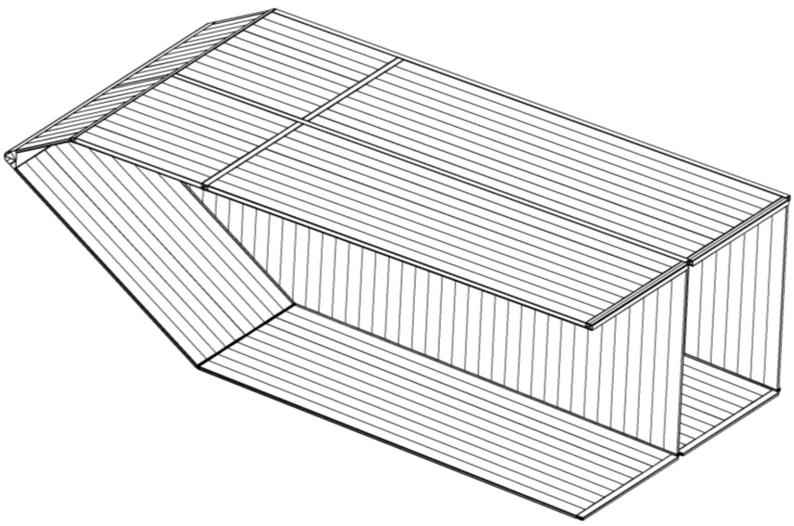
Alloy		**Thickness [mm]**	f_o **[MPa]**	f_u **[MPa]**	$\rho_{o,haz}$	$\rho_{u,haz}$	$f_{o,haz}$ **[MPa]**	$f_{u,haz}$ **[MPa]**	**Buckling class**
5083-H116	Plate	20	215	305	0.72	0.9	155	275	B
		28	215	305	0.72	0.9	155	275	B
5383-0	Plate	120	130	275	1	1	130	275	C
6005A-T6	Extruded open profile	30-37.5	215	255	0.53	0.65	115	165	A
		8-10	215	260	0.53	0.63	115	165	A
6082-T6	Extruded hollow profile	12	260	310	0.48	0.6	125	185	A

5.3 Transverse panel concept

The transverse panel bridge girder concept consists of panels in all parts of the outer skin. The panels are oriented transverse to the bridge direction. The deck is supported by transverse bulkheads every 3.9 m. Hangers are located every 11.7 m, so there are in total three bulkheads per hanger set connections. Principle sketches are shown in Figure 5-17 and Figure 5-18 below.



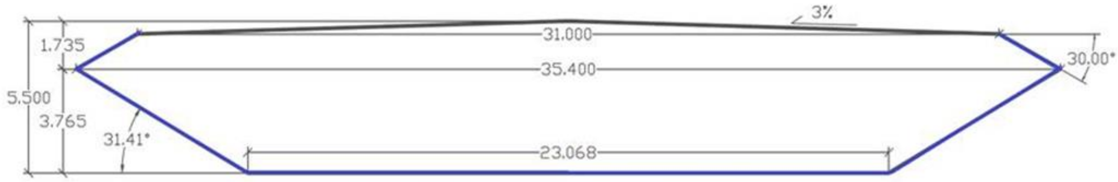
> *Figure 5-17 Principle sketch of transverse panel concept*



> *Figure 5-18 Principle sketch #2 of transverse panel concept (here, for simplicity, symmetric about CL)*

5.3.1 Outer layout

The outer layout of the bridge girder cross section is based on what is tested in a wind-tunnel by two master students led by Ole Øiseth at NTNU the spring of 2020 [13]. The thesis was issued at the very end of this project and the results, consisting of new sets of aerodynamic derivatives, have therefore not been applied in the stability calculations. However, the study indicates that the critical velocity with respect to flutter/galloping of a similar bridge girder is considerably higher than what has been calculated in this project. Thus, there is a substantial potential for further optimisation.



> *Figure 5-19 Outer layout*

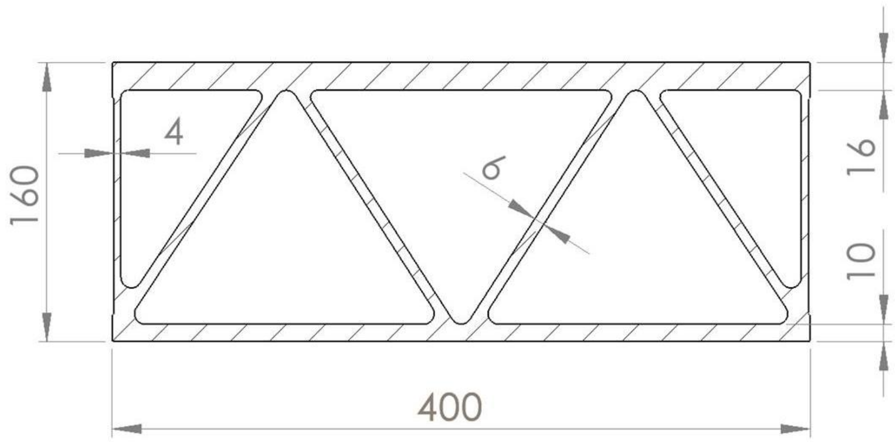
5.3.2 Orientation of panels

Due to limited access to the internal webs when you place two panels next to each other in the longitudinal direction, only the top and bottom plates are accessible to welding. The web will therefore be discontinuous in this joint. FEM-analyses showed Appendix F – SCF Analysis that this discontinuity produces stress concentrations up to a factor of 2. This issue cannot be handled by a local increase of section material due to the production method of these panels (extrusion).

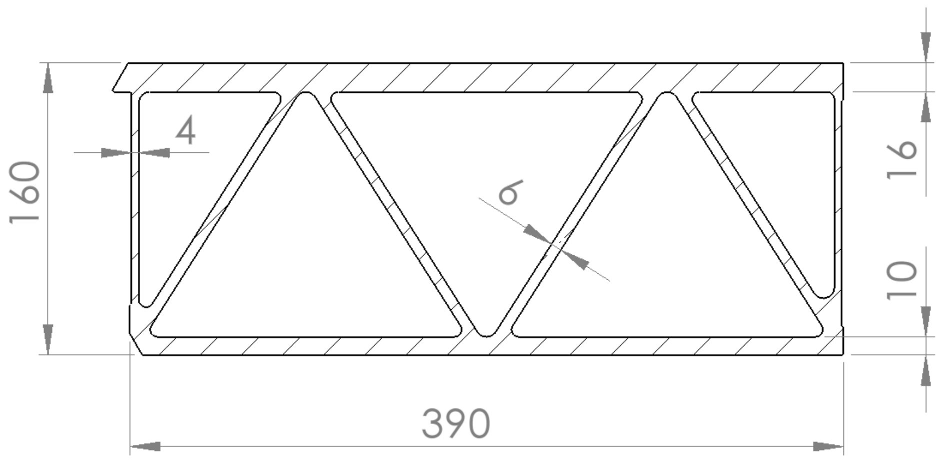
By orienting the panels in the transverse direction, this discontinuity is avoided. However, larger local bending stresses due to the wheel-pressure is introduced, which must be taken into account in the local stress evaluation. This matter is however easier to design for by decreasing the web distance.

5.3.3 Panel dimensions

5.3.3.1 Top panel



> *Figure 5-20 Top deck panel*



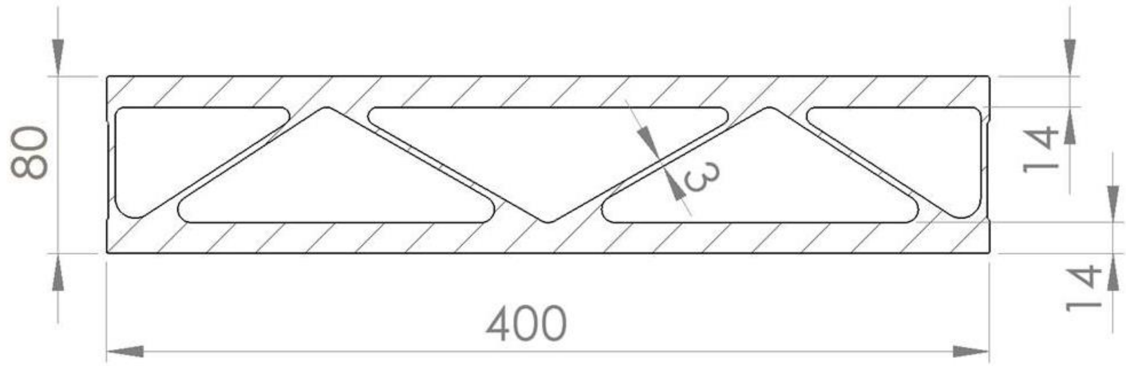
> *Figure 5-21 Top deck panel toward joiner profile in bulkhead (right side)*

The following dimensions are given for the top deck:

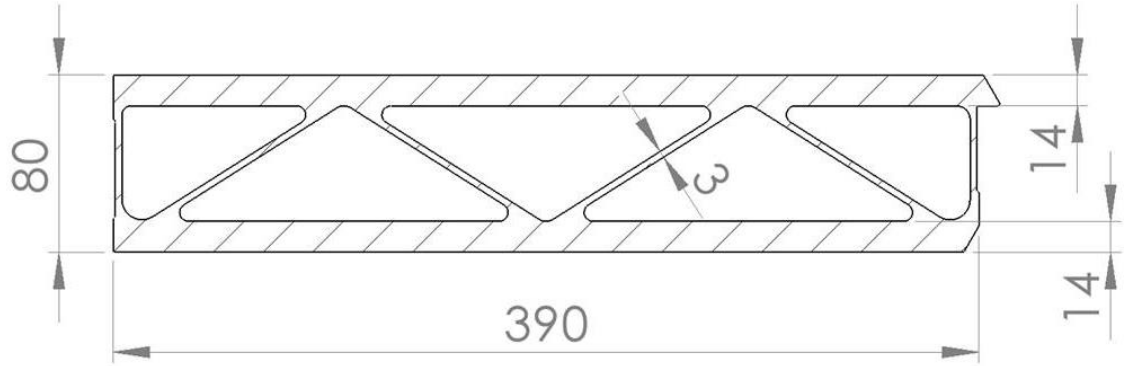
- **Top plate = 16 mm**
- **Bottom plate = 10 mm**
- **Diagonals = 6 mm**
- **Verticals = 4 mm**

The top plate has to carry local bending stresses from the wheel load and are therefore thicker than the bottom plate which is dominated by membrane stresses.

5.3.3.2 Bottom panel



> Figure 5-22 Bottom and side deck panel



> Figure 5-23 Bottom and side deck panel towards joiner profile in bulkhead (right side)

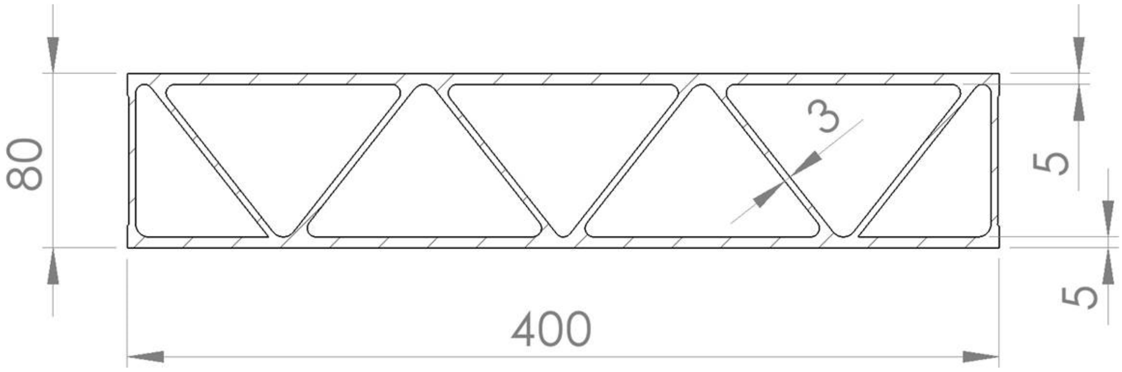
The following dimensions are given for the bottom and side deck:

- **Top plate = 14 mm**
- **Bottom plate = 14 mm**
- **Diagonals = 3 mm**
- **Verticals =3 mm**

The thicknesses of the bottom and top plate are oversized with respect to ULS and FLS requirements. However, the optimization study performed in this project has shown that increasing the torsional stiffness by increasing the thickness of the skin is the most cost-efficient measure (up to a certain level) to obtain the required critical wind speed of the concept (flutter/instability etc.)

The distance between the plates has been reduced (more efficient with respect to torsional stiffness) but is kept large enough to avoid local buckling issues.

5.3.3.3 Bulkhead panel



The following dimensions are given for the bulkhead panels and side deck:

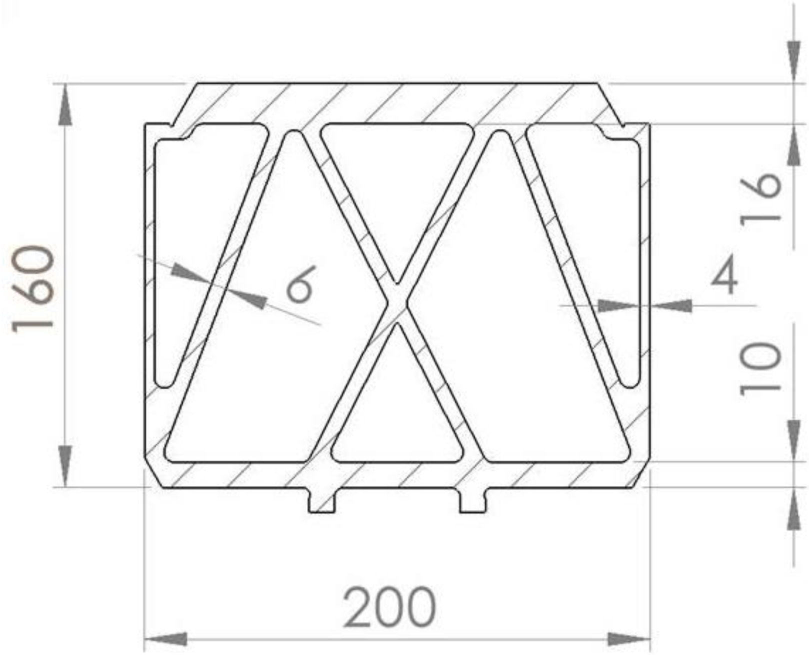
- **Plates = 5mm**
- **Diagonals = 3mm**
- **Verticals =3mm**
- **Height = 80mm**

5.3.3.4 Joiner profiles for connection of deck panels to vertical bulkhead

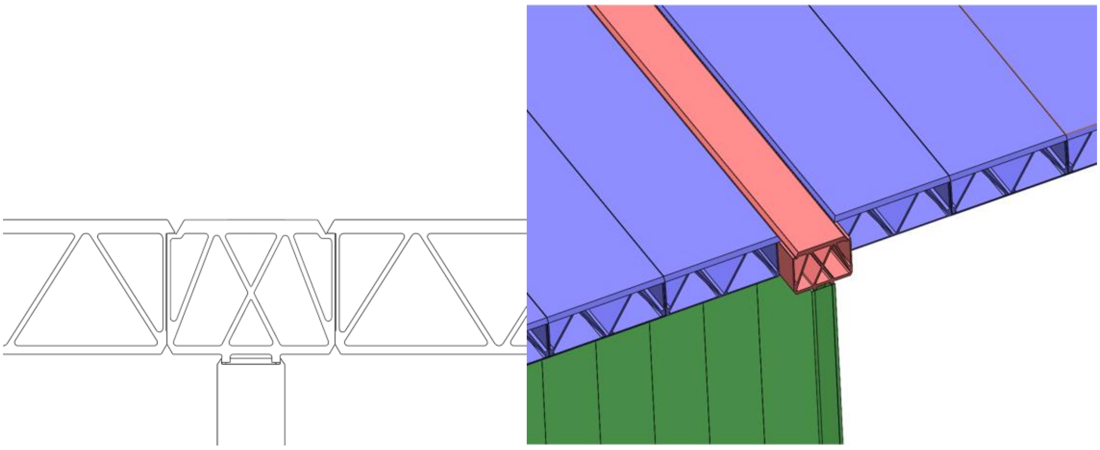
The connection of the decks to the bulkhead is handled by use of a joiner profile as seen in Figure 5-24 through **Error! Reference source not found.**. The connection shown is for the top deck. However, the same principle is applied for the bottom deck and side deck as well.

The section is designed in order to transfer forces across the bulkhead as well as into the bulkhead in a satisfactory manner to avoid high stress concentrations.

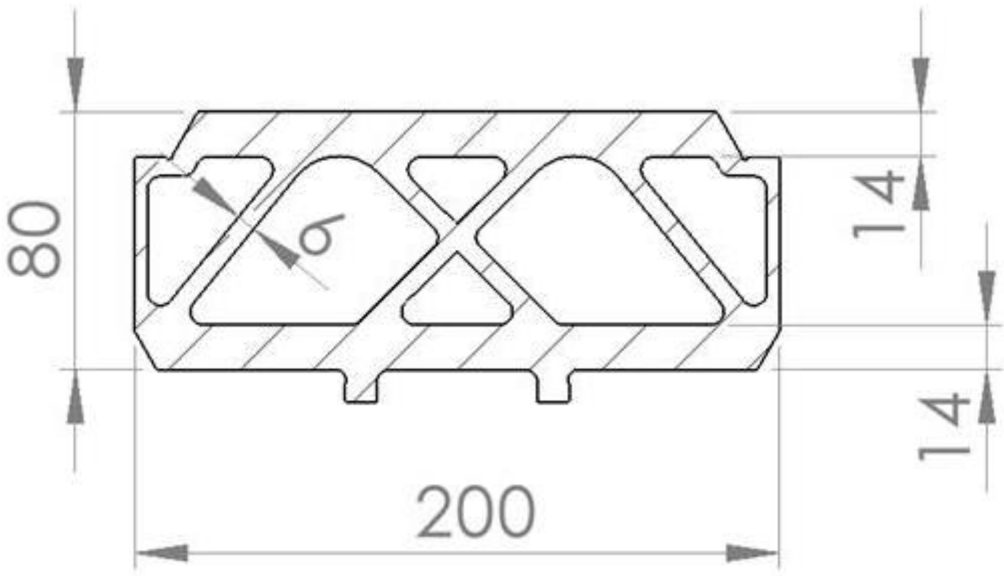
By use of the possibilities from extrusion techniques when manufacturing aluminium profiles, groove, bevel and radius details can be included in the joiner and decking profile in order to account for single sided full penetration welding with backing, which has improved fatigue life compared to fillet and partial penetration welds. Any weld root failure will then be outside the load-carrying material.



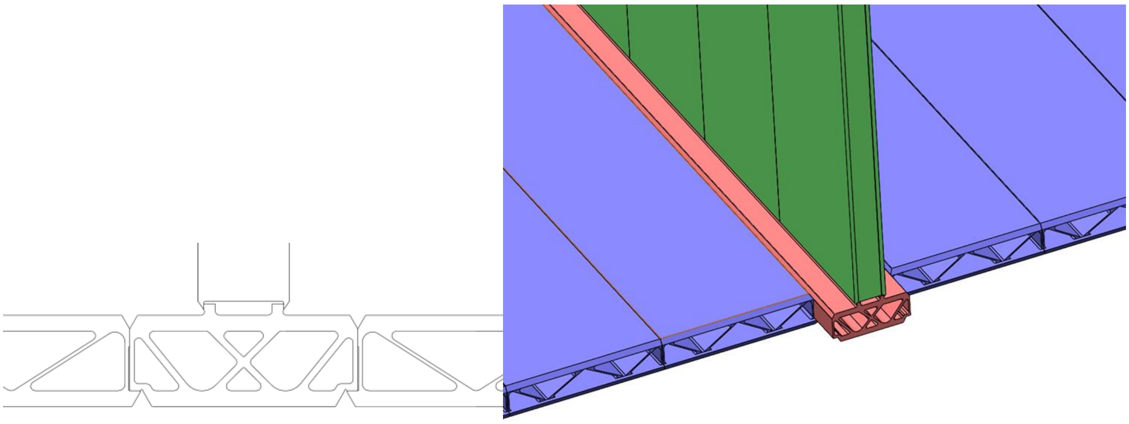
> *Figure 5-24 Joiner profile between top deck and bulkhead, dimensions*



> *Figure 5-25 Joiner profile between top deck and bulkhead*



> Figure 5-26 Joiner profile between bottom deck and transverse bulkhead, dimensions

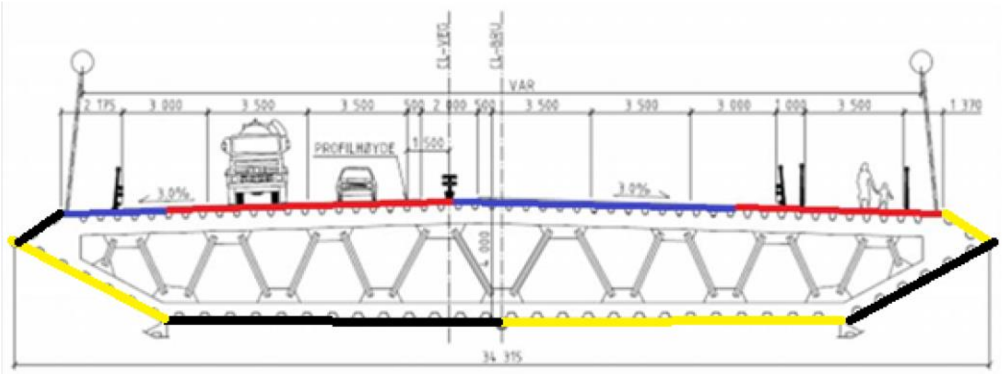


> Figure 5-27 Joiner profile between bottom deck and transverse bulkhead

5.3.3.5 Joiner profiles for transverse connection of longitudinal panels

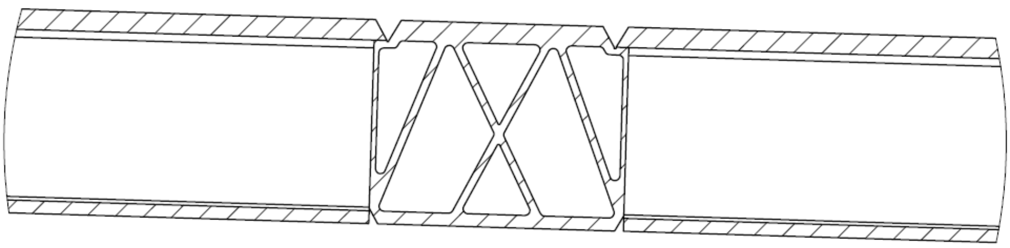
The design philosophy of these joints is to keep them as far away from the fatigue wheel loads as possible. This is to avoid stress concentrations in fatigue load exposed areas.

A possible layout is shown in the figure below. Connections or joiner profiles are placed where the colour changes in Figure 5-28. In total 10 profiles are applied in order to connect the panels in the transverse bridge direction. They are not designed further, but they are considered less challenging.

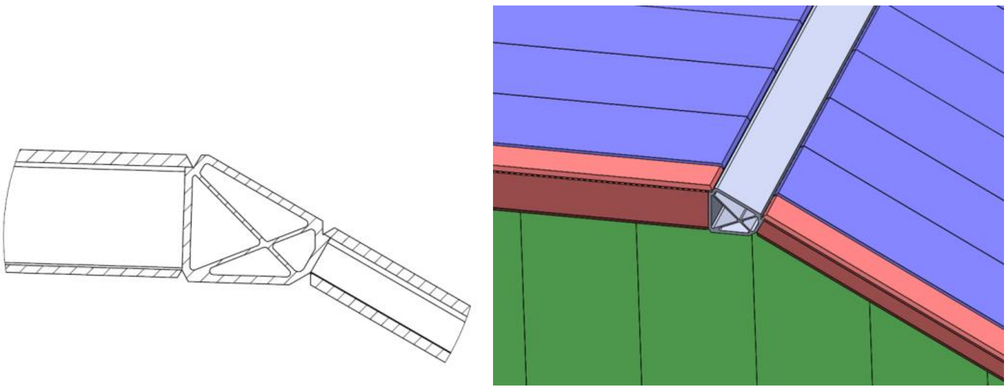


> Figure 5-28 Panel layout in the transverse bridge direction

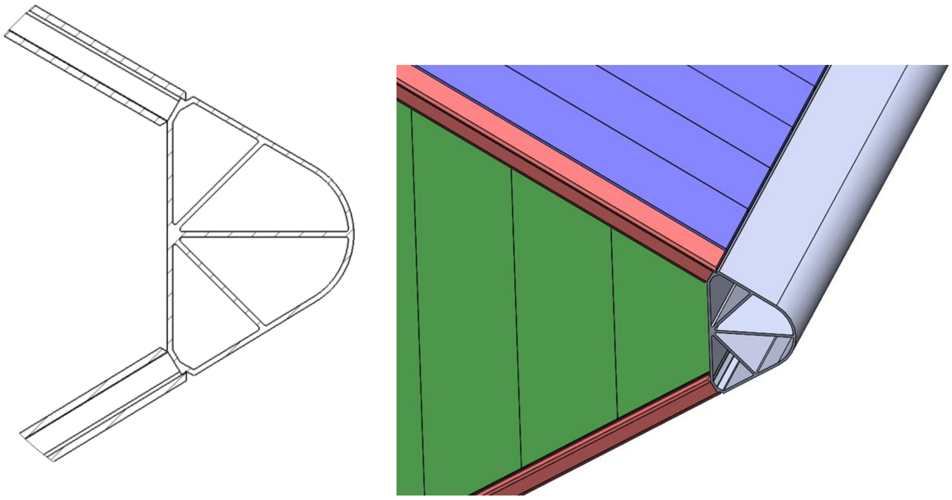
Possible joiner profiles are sketched in Figure 5-29 through Figure 5-32**Error! Reference source not found.**.



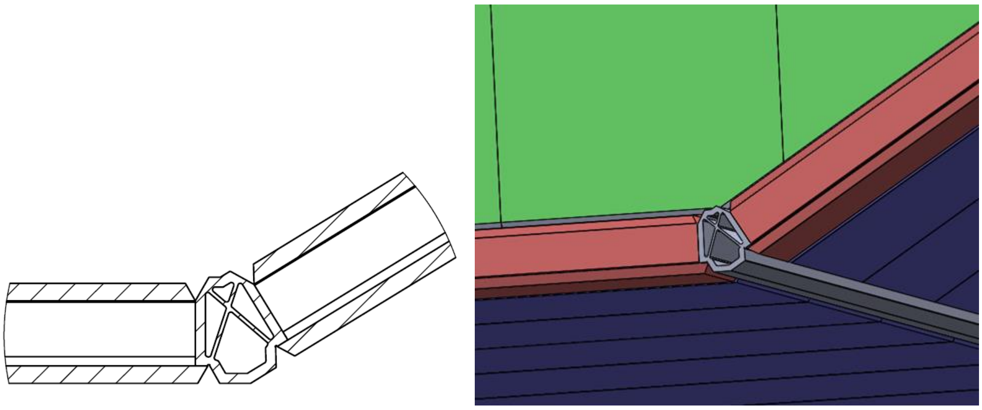
> Figure 5-29 Possible joiner profile 1



> Figure 5-30 Possible joiner profile 2



> *Figure 5-31 Possible joiner profile 3*



> *Figure 5-32 Possible joiner profile 4*

5.4 Other concepts

5.4.1 Inverted cable concept

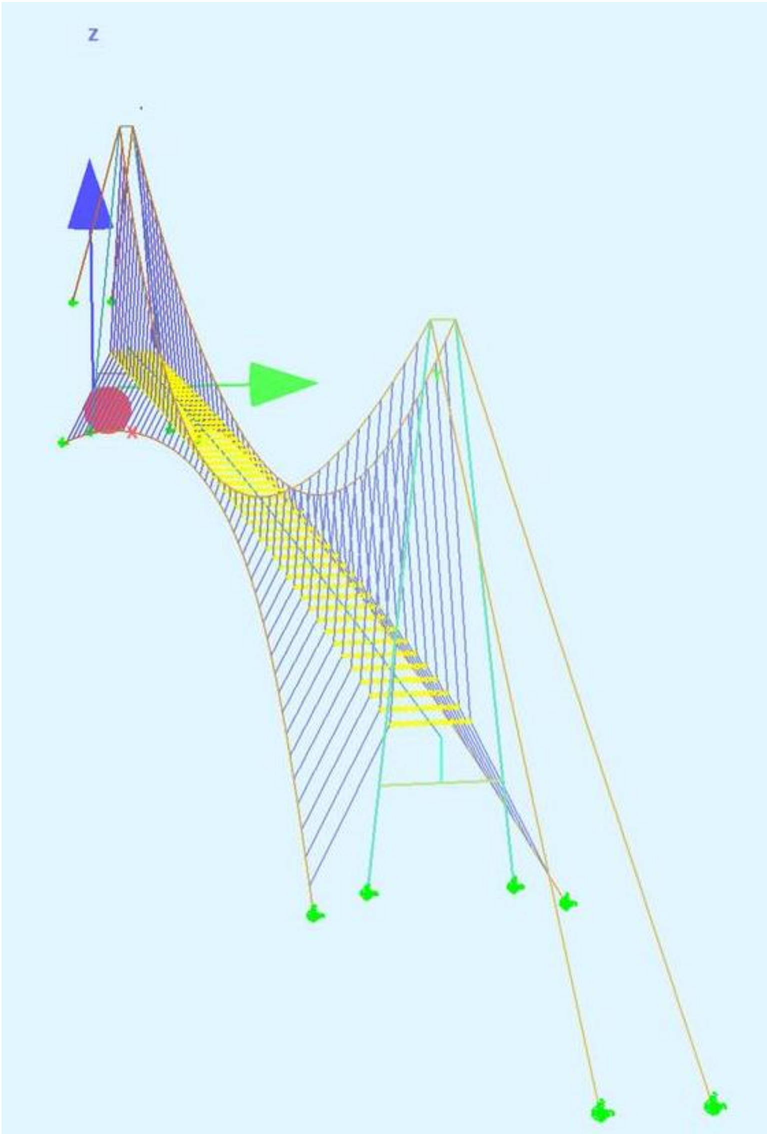
A proposal for using inverted cables below the bridge in the same fashion as the one in Figure 5-33 has been investigated.

> *Figure 5-33 Dodhara Chandani suspension bridge, Far Western Nepal*

The purpose behind the use of an inverted cable was to increase the global stability with respect to critical wind speed. The stability of the bridge is governed by the relationship between the torsional and vertical, mass and stiffness relations. In general, increasing the torsional stiffness improves the stability.

The torsional stiffness given to a suspension bridge consists of two main parts, the girder contribution and main cable contribution. Adding a vertical inverted main cable has somewhat the same effect as increasing the main cable size. However, it is more complex, less efficient and most likely more expensive than just increasing the diameter of the main cable. This will also entail a further elevated girder to give the same clearance as the solution without an inverted cable system. Based on these evaluations the idea of a straight vertical inverted cable was disregarded.

On the other side, by applying the inverted cable as a more horizontal cable instead (see Figure 5-34) have some positive repercussions. Analyses showed that the cable reduced the global stresses in the bridge girder from wind. This opened the possibility of increasing the main girder height (which has a positive effect on the torsional stiffness), and still be within the ULS requirements of global stresses. Whether it is a cost-efficient design philosophy taking everything into consideration, was not concluded upon within this project.



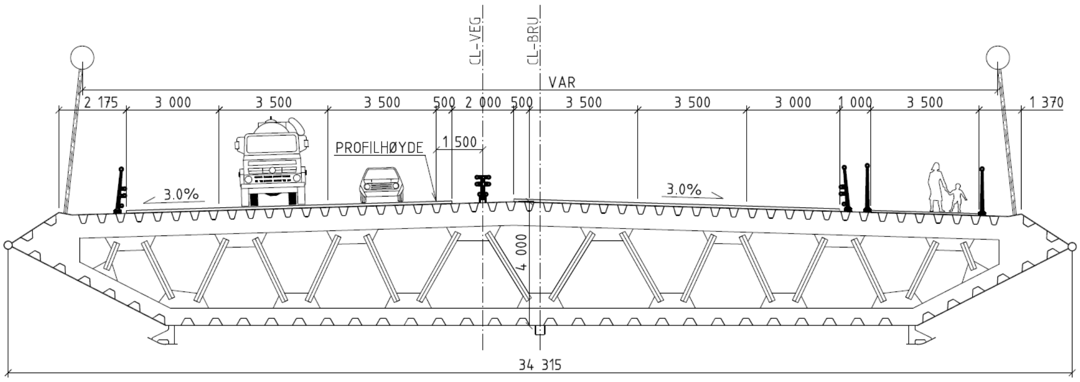
> *Figure 5-34 Example of partly horizontal, partly vertical inverted cable suspension bridge model.*

At the time further investigation of this concept was to take place, the two previous mentioned concepts showed promising results with respect to global stresses, critical wind speed and cost relative to a steel bridge. More thorough fatigue evaluations and analyses were prioritized at the expense of further investigation of the inverted cable concept which would have required a full re-design of the bridge girder. The concept should be looked further in to in the future.

The design was not developed enough to present any dimensions. They would at best be misleading. A philosophy of a pretension corresponding to from a half to two-thirds of the self-weight pretension of the main cable was analysed, but no conclusions were made.

5.4.2 Steel benchmark concept

A steel concept has in this report been used as a reference in comparison with the aluminium concept. The bridge girder applied for this concept is the one developed by Norconsult [11] in a previous project for crossing of Langenuen. The cross-section is shown in Figure 5-35.



> *Figure 5-35 Steel concept developed by Norconsult [11]*

The same design philosophy has been used for the hangers and the concrete towers as for the aluminium concepts. This is described in Chapter 4.5.1.

As seen in Chapter 6 only the results with respect to some of the performance criteria have been answered. These are global stability and cost, respectively.

5.5 Summary of global values

A summary of the global parameters for the concepts evaluated in Chapter 6 is presented in Table 5-2.

> *Table 5-2 Summary of global parameters and information*

	Panel concept	**Plate concept**	**Transverse panel concept**	**Steel benchmark concept**
Bridge girder:				
Cross-section height [m]	5.5	5.5	5.5	4.0
Cross-section area [m^2]	2.45	2.81	2.48	1.23
Torsional stiffness, J [m^4]	31.0	39.1	40.6	9.43
Second area moment about horizontal axis, Iy [m^4]	12.0	15.28	13.2	3.42
Second area moment about vertical axis, Iz [m^4]	238.3	267.1	266.0	120.7
Mass [kg/m]	7 598	9 045	8969	12 010
Total weight [ton]	9 391	11 080	10987	14 831
Concrete towers:				
Saddle elevation [m]	214 (higher to reach critical wind speed criterium)	206.1	206.1	206.1
Main cable:				
Distance between cable and bridge girder in centre of bridge [m]	3.9	3.9	3.9	3.9
Tension in self-weight condition [MPa]	400	490	487	490

Diameter [m]	0.681	0.644	0.644	0.711
Total weight [ton]	11 437	10 178	10178	12 387
Hangers:				
Centre distance [m]	12	12	12	24
Diameter [m]	0.044	0.046	0.046	0.072
Total weight [ton]	122	127	127	151

6 PERFORMANCE

In this chapter the performance of the three concepts is presented. Global and local ULS have not been checked for the last transverse panel concept. However, due to similar or reinforced global and local design, the concept is assumed to have similar or better performance than the two other concepts with respect to ULS.

6.1 Global stability/critical velocity

All concepts fulfil the requirement (76m/s) with respect to critical wind speed.

	Critical wind speed	Unit
Panel concept	76.6	m/s
Plate concept	76.5	m/s
Transverse panel concept	76.0	m/s
Steel benchmark concept	76.0	m/s

6.2 Global stress levels (ULS)

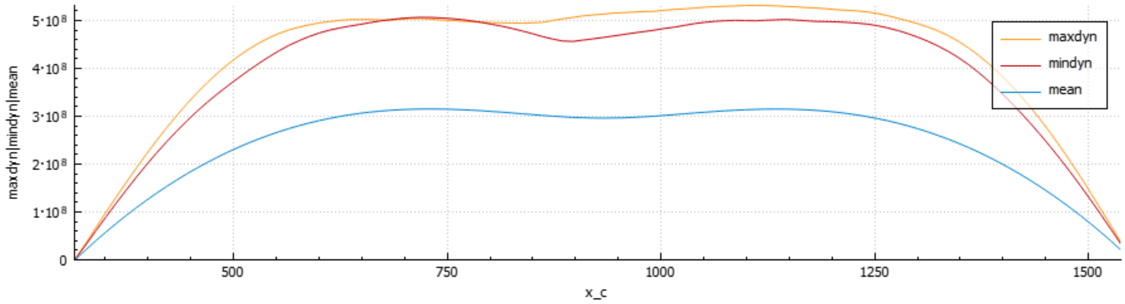
The global stress levels are below 100 MPa for both concepts, which has been set to an upper limit. Depending on the material applied, a total stress level of up to 115 MPa is acceptable – as long as local stability is also preserved.

	Maximum stress	Unit
Panel concept	93.2	MPa
Plate concept	87.5	MPa
Transverse panel concept	similar	MPa

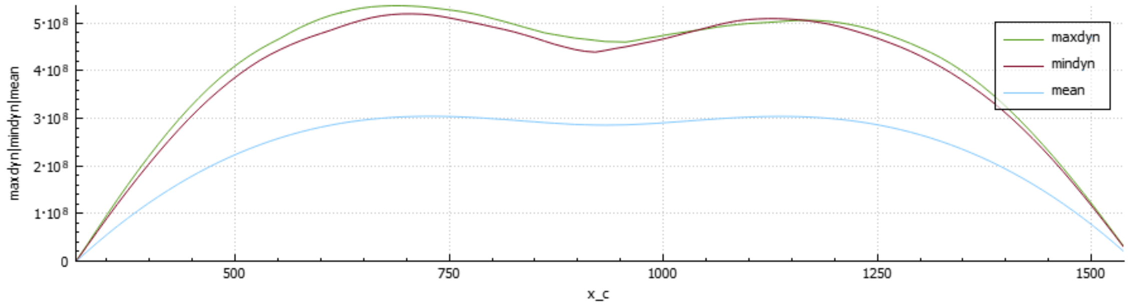
An excerpt of characteristic response and eigenmodes are presented below. More details can be found in Appendix D – Global analyses. Local stability is evaluated in Appendix E – Local calculations and found to be within acceptable limits.

6.2.1 Characteristic response

The largest global stress levels in the bridge girder are driven by wind loads of 50-year return period. Moments about vertical axis are shown here, for static (*mean*) and dynamic wind load from turbulence components in opposite directions both normal to the bridge (*maxdyn* and *mindyn*). Due to the random nature of time domain analyses, the responses from turbulence are not equal in both directions.

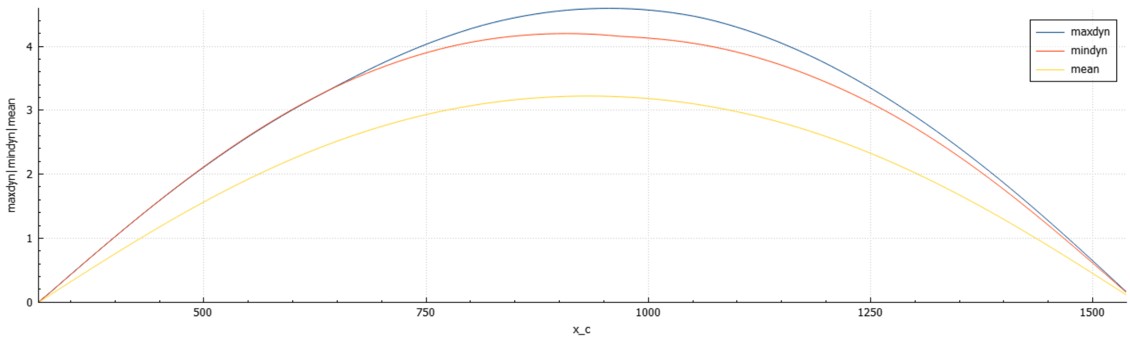


> *Figure 6-1 Panel concept: Moment [Nm] about vertical axis. Maximum and minimum dynamic contribution and mean (static) contribution.*

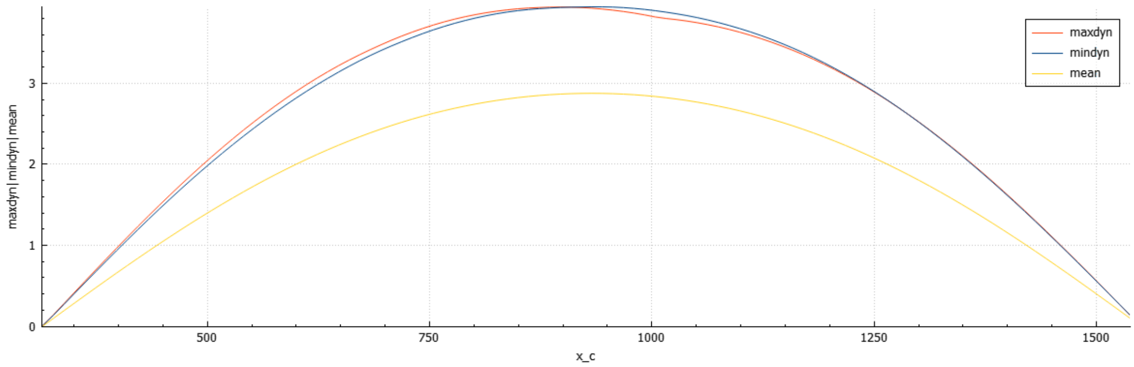


> *Figure 6-2 Plate concept: Moment [Nm] about vertical axis. Maximum and minimum dynamic contribution and mean (static) contribution.*

There are no big differences between the two concepts with respect to the dynamic contribution. The panel concept exhibits slightly larger moment from static wind. This may be because it has a higher tower and less prestress in the cable, which leads to the girder taking a larger portion of the horizontal wind load compared to the cable. Correspondingly, the panel concept also exhibits larger lateral displacements, see Figure 6-3 and Figure 6-4.



> *Figure 6-3 Panel concept: Lateral displacement [m] of the bridge girder*



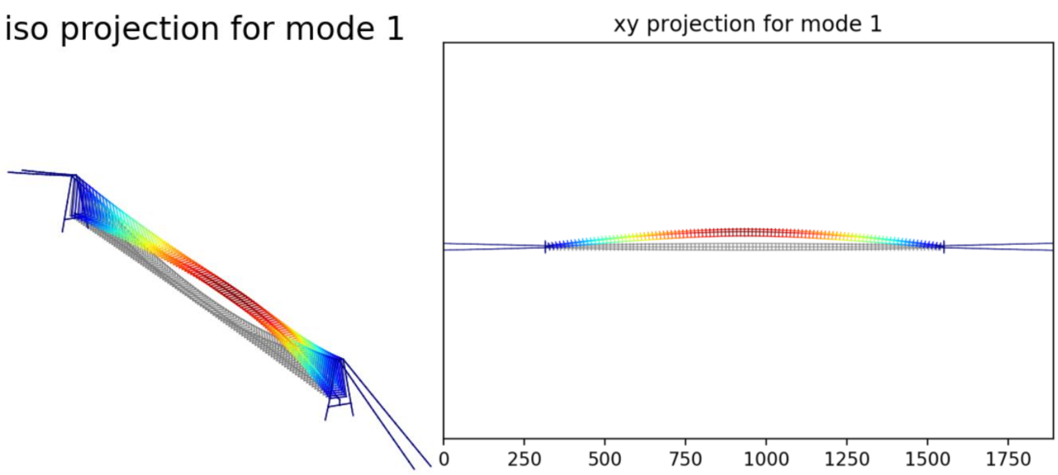
> *Figure 6-4 Plate concept: Lateral displacement [m] of the bridge girder.*

6.2.2 Eigenmodes

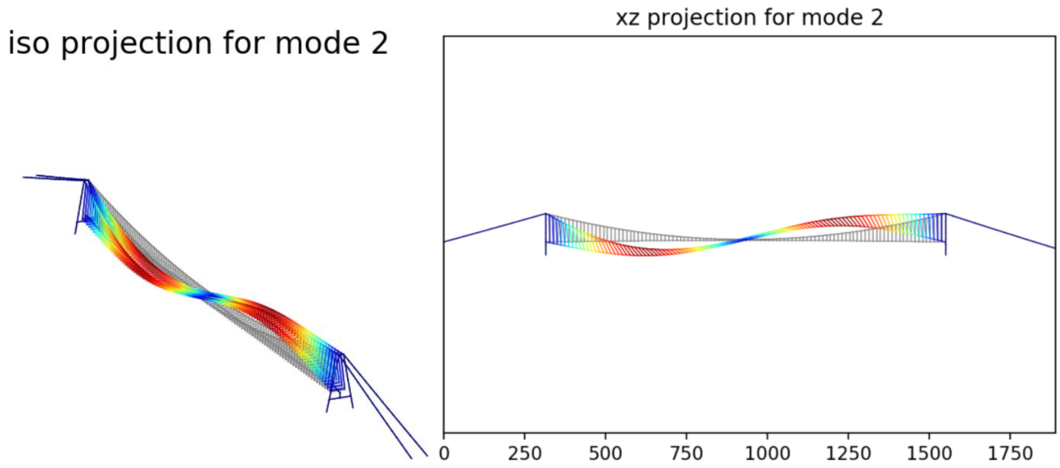
The frequency and period of the two first eigenmodes are shown in Table 6-1 for both concepts. The corresponding mode shapes are visualized in the figures below. It can be seen that the panel concept has slightly higher eigen periods than the plate concept. The panel concept has a higher tower, the girder is in general softer and the girder is heavier, all of which may lead to a less stiff construction and thereby higher eigen periods.

> *Table 6-1 Eigenfrequencies and -periods of the two concepts*

	Panel concept			Plate concept			Transverse panel concept
Mode	**Frequency [Hz]**	**T [s]**	**Shape**	**Frequency [Hz]**	**T [s]**	**Shape**	
1	0.0578	17.3	First horizontal	0.0594	16.84	First horizontal	similar
2	0.1112	8.99	First vertical	0.1146	8.73	First vertical	similar



> *Figure 6-5 The mode shape of the first eigenmode for the plate concept*



> *Figure 6-6 The mode shape of the second eigenmode for the plate concept*

6.3 Local stability (ULS)

Local stability has been evaluated for both concepts in critical parts of the structure:

- **Panel concept: Bottom panel**
- **Plate concept: U-stiffeners in top, bottom and side plate**
- **Transverse panel concept: similar to panel concept**

The capacities of the structural parts are well above the stresses of which the parts are exposed to.

6.4 Fatigue life

The fatigue calculations performed in this report are simplified, especially with respect to wind dynamics.

In general, the fatigue requirements (fatigue curves) are tougher relative to the material capacity than what is the case for steel.

Based on the assumptions presented in this report it is concluded that a sufficient lifetime is achievable for both concepts.

The lifetime is too small for the panel concept in the slow lanes. This can be compensated for by increasing the moment of resistance W, by 15% for both the deck and bottom of the longitudinal beam. This should not have very much impact on the global analyses. The impact on the cost estimates is about 5 MNOK.

> *Table 6-2 Lifetime of concepts summarized*

Fatigue loads	Plate concept	Panel concept	Transverse panel concept
Traffic (slow lane)	147 years	115 years	>100 years[1]
Wind (outer edge of cross-section)	105 years	105 years	105 years[2]
Combined (slow lane)	100 years	100 years[3]	>100 years

.

[1] Many of the details have stresses below cut-off, i.e. lifetime is ∞. A few are lower than cut-off, but those details are considered easy to improve and optimise.
[2] Not studied in detail, but similar to the other concepts.
[3] Note that in Appendix A, 49 years lifetime is calculated for combined action of traffic and wind for the most critical weld of the panel concept. It is concluded that by increasing flange of the girder below the slow-lane from 49 mm to 60 mm a lifetime beyond 100 years is achievable.

6.5 Cost of main construction parts

The cost of the different concepts more in detail is shown in Table 6-3. These are best estimate cost numbers where uncertainty is not taken into account. The uncertainty of the cost estimates is +-25%.

> *Table 6-3 Cost of main structural parts*

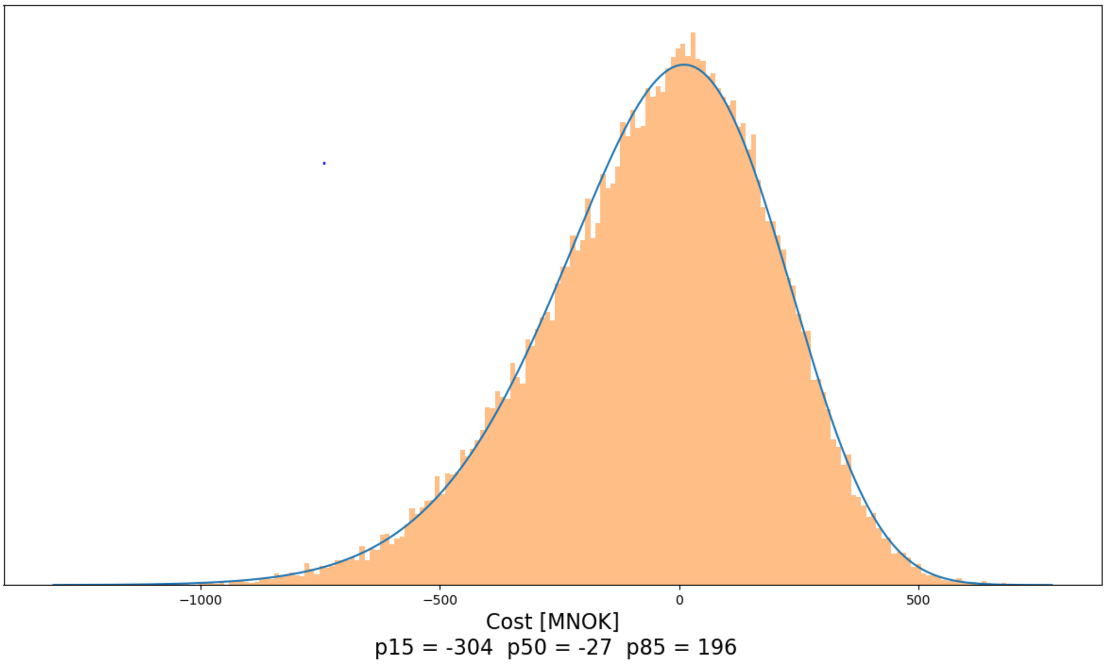
	Plate concept [MNOK]	Panel concept [MNOK]	Transverse panel concept [MNOK]	Steel benchmark concept [MNOK]
Concrete tower	500	500	500	500
Main girder	878	816	973	786
Hanger steel	25	24	25	30
Main cable steel	896	1007	896	1090
Total	2299	2347	2394	2406

The cost of the tower foundation is included in the concrete tower cost. However, the cost of the main cable anchor chambers is not included in the cost of the main cable. The ultimate limit state force in the anchors are calculated to be 252 MN and 234 MN for the plate concept and panel concept, respectively. The anchor force for the steel concept is not calculated but will most likely be higher than both aluminium concepts. Including the cost of these chambers will benefit the aluminium concepts. Those benefits have not been quantified at this stage. The construction phase has not been evaluated in the design of the towers. This is most likely the dimensioning phase for the towers. Due to this, the tower cost is set equal for all concepts in Table 6-3. The cost of the tower is estimated to be about 500 MNOK. The hanger cost is based on the total required material volume in the hangers and not the number of hangers. This should be included at a later stage. The increased thermal expansion of aluminium will increase the cost of the expansion joint at one end of the bridge. This extra cost is not included in the table above.

6.6 Relative cost estimates

The results shown in Figure 6-7 is the economic difference density distribution of the steel and the plated aluminium concept. In this study, the plated aluminium concept amounts have been subtracted from the steel concept amounts, and a cost analysis based on these relative amounts has been run; taking unit cost, market and design-related uncertainties into account.

By using relative amounts, we imply that all equal amounts in between the concepts are fully correlated, which is a fair assumption. E.g. whether a cable is used to carry a steel girder or aluminium girder does not affect any relative uncertainty in between the concepts. It is still a cable and it must be constructed independently of the chosen concept.



> *Figure 6-7 Probability density with regards to difference in cost between the steel concept and the plated aluminium concept.*

Every simulation that gives a positive value represents realities/analyses where the steel concept costs exceed the aluminium concept cost. The p15, p50 and p90 values from the distributions are shown in the figure.

In general, the cost analysis showed that there are two global effects/uncertainties that affect the results. Steel suspension bridges of this scale have been built for decades. Aluminium bridges of this size and method have never been built before. This naturally increases the uncertainty when it comes to aluminium design. However, by using aluminium instead of steel in the bridge girder, a wider range of materials is mobilized. This effect mitigates some of the risk/uncertainty with respect to market variations.

By integrating over all positive values in the distribution we get about 45% probability that the steel concept cost exceeds the aluminium concept cost.

Details of cost estimation are given in Appendix C – Cost Analysis.

7 BREAKDOWN OF BRIDGE GIRDER FABRICATION AND ASSEMBLY COST

7.1 Key cost elements

7.1.1 General overview with cost elements for Aluminium alternative

The cost overview, for both Plate Concept and Panel Concept, is split into 4 elements:

- **Material**
- **Welding**
- **Handling**
- **Facility & installation**

Each of these elements has been estimated with Low-Medium-High rates. For welding, the High rate is cost for 100% manual welding based on today's welding technology. For the three other cost elements, Low and High rates are generally +/- 10% from the Medium rate, which shall account for opportunities (-10%) and risks (+10%).

7.1.1.1 Material cost

For material, prices are collected from multiple suppliers to obtain reasonable values. These prices are factored with 20% addition due to procurement profit and cut & waste.

7.1.1.2 Welding cost

For welding, the different rates are based on estimated improved efficiency due to automatic welding processes and high degree of repetitive work, as the 12 m sections shall be fabricated in 100 identical copies.

More detailed, the welding cost is based on a manual/automatic-ratio, welding arc rates, welding speeds, number of weld-passes to each weld category (18 different weld types defined), robot/person-ratio and cost per man-hour. When these parameters are combined, welding unit cost [NOK/m] from Low-Medium-High for each weld category is deduced. These cost rates are then applied to the weld quantum estimated for each of the concepts.

In Table 7-1, different welds for different applications are assigned with a category code, W1 to W19. For example, W2 is a 7mm fillet weld with cost (NOK/m) ranging from 136,6 to 311,7. These codes are referenced in the following calculations in Table 7-4, Table 7-7 and Table X (table I section 7.4.2).

> *Table 7-1 Girder Welding Cost Details*

Cat	Description	Type	Thickn [mm]	Manual – Manual [%] Low	Manual – Manual [%] Med	Manual – Manual [%] High	Manual – Arc rate [%]	Manual – Speed [mm/min]	Manual – Passes [ ]	Automatic – Automatic [%] Low	Automatic – Automatic [%] Med	Automatic – Automatic [%] High	Automatic – Arc rate [%] Low	Automatic – Arc rate [%] Med	Automatic – Arc rate [%] High	Automatic – Speed [mm/min]	Automatic – Passes [ ]	Automatic – Automats per Person Low	Automatic – Automats per Person Med	Automatic – Automats per Person High	Cost [NOK/hrs] Low	Cost [NOK/hrs] Med	Cost [NOK/hrs] High	Cost [NOK/m] Low	Cost [NOK/m] Med	Cost [NOK/m] High
W1	Fillet	F	5																					72,8	118,8	155,8
W2	Fillet	F	7																					136,6	225,1	311,7
W3	Fillet	F	8																					218,5	356,5	467,5
W4	Fillet	F	10																					291,3	475,4	623,3
W5	Fillet	F	12																					436,9	713,1	935,0
W6	Fillet	F	14																					582,5	950,7	1246,7
W7	Part pen + fillet	PP+F	5+5																					145,6	237,7	311,7
W8	Part pen (single side)	PP1	7																					145,6	237,7	311,7
W9	Part pen (single side)	PP1	10																					291,3	475,4	623,3
W10	Part pen (double side)	PP2	20																					582,5	950,7	1246,7
W11	Part pen (double side)	PP2	28																					873,8	1426,1	1870,0
W12	Full pen (single side w/backing)	FP1	10																					291,3	475,4	623,3
W13	Full pen (double sided)	FP2	20																					582,5	950,7	1246,7
W14	Full pen (double sided)	FP2	22																					728,2	1188,4	1558,3
W15	Full pen (single side w/backing)	FP1	28																					728,2	1188,4	1558,3
W16	Full pen (double sided)	FP2	28																					1583,8	2589,4	3428,3
W17	Full pen (single side w/backing)	FP1	35																					2038,9	3327,6	4363,3
W18	Full pen (double sided)	FP2	49																					2475,8	4040,6	5298,3
W19	Full pen (double sided)	FP2	60																					3276,8	5347,9	7012,5
W20	Full pen (single side w/backing)	FP1	5																					72,8	118,8	155,8
W21	Full pen (single side w/backing)	FP1	12																					291,3	475,4	623,3
W22	Full pen (single side w/backing)	FP1	14																					364,1	594,2	779,2
W23	Full pen (single side w/backing)	FP1	16																					509,7	831,9	1090,8

The welds are split in type and thickness. The types are:

- **F: Fillet weld**
- **PP+F: Partial penetration with fillet weld above**
- **PP1: Partial penetration, from one side only (no access from opposite side)**
- **PP2: Partial penetration, double-sided access**
- **FP1: Full penetration, single sided which requires permanent backing (no access from opposite side)**
- **FP2: Full penetration, double-sided**

7.1.1.3 Handling cost

Handling includes, amongst others, prefab work (cutting, bevels, tac welding), NDT and dimensional control, local movement of material and parts, internal transportation from prefab to assembly yard, rigging and repositioning of the work.

The plate concept uses Leirvik AS's benchmark numbers for handling in a typical Living Quarter (400 – 600 tons aluminium), since this concept has the most complex fabrication method of the two. The panel concept has a lower handling cost due to fewer parts to be pre-assembled and less anticipated dimensional deviations during fabrication.

7.1.1.4 Facility and installation cost

The Facility & installation cost element is equal to the two concepts. Here, the cost of yard rent, new yard hall and equipment, skid tracks, temporary structures for assembly of girder, cranes and trucks, transportation of 120m-sections etc. are included.

> *Table 7-2 Facility and Installation Cost Breakdown (numbers omitted for commercial property rights)*

Item	Description	Size	Unit	Unit cost	Unit	Duration	Unit	Cost	Unit	Assumptions
001	Rent of area for fabrication	45 000	m2						NOK	
002	Fabrication hall	4 050	m2						NOK	
003	Office rental	8	units						NOK	
004	Overhead crane	1	of						NOK	25 tonn SWL
005	Track system for sections	900	m						NOK	Skid tracks 2 * 3 *150m HEA700
006	Temporay building supports and frames	90 000	kg						NOK	Including fabrication
007	Boogie/trailers for transp of compl units	1	of						NOK	
008	Truck rental	1	of						NOK	
009	Mobile crane rental	1	of						NOK	
010	Other temporary facilities (gas, electricity etc.)	1	of						NOK	
011	Jack rental for installation	1	of						NOK	
012	Robot equipment	1	of						NOK	
013	Transportation of sections to site	10	of						NOK	
							Total	89 850 000	NOK	

7.2 Panel concept

7.2.1 Material cost

The panel concept utilizes to a great extent the advantages with extrusion of aluminium profiles and Friction Stir Welding (FSW) joining technology (98.7%). The content of rolled plate products is very low (1.3%).

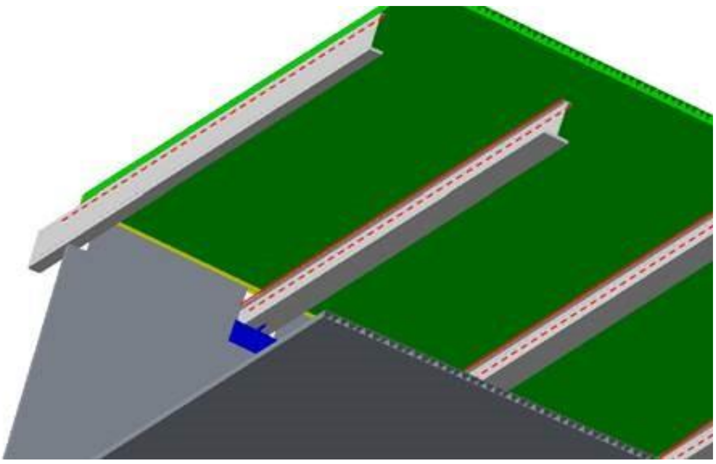
> *Table 7-3 Panel Concept Material Cost, 12m section*

Material - Panel concept						Cost [NOK/kg]			Cost [NOK]		
Description	Alloy	Quantum	Unit	Mass/unit	Mass [kg]	Low	Med	High	Low	Med	High
FSW Top deck	6005A-T6	372	m2	81,6	30355	55,84	62,04	68,24	1 694 913	1 883 237	2 071 560
FSW Transverse bulhead	6005A-T6	143	m2	56,3	8062	55,84	62,04	68,24	450 150	500 166	550 183
FSW Bottom deck	6005A-T6	289	m2	71,2	20545	55,84	62,04	68,24	1 147 137	1 274 597	1 402 057
FSW Side bulkheads	6005A-T6	194	m2	71,2	13824	55,84	62,04	68,24	771 888	857 653	943 418
Joiner profiles	6005A-T6	71	m	4,8	344	37,57	41,75	45,92	12 925	14 361	15 797
Corner profiles	6005A-T6	72	m	10,5	759	37,57	41,75	45,92	28 518	31 687	34 855
Beams - Top deck - reinf.	6005A-T6	48	m	85,9	4123	59,40	66,00	72,60	244 918	272 131	299 344
Beams - Top deck	6005A-T6	120	m	90,8	10896	59,40	66,00	72,60	647 222	719 136	791 050
Beams - Bottom deck	6005A-T6	36	m	29,2	1051	59,40	66,00	72,60	62 441	69 379	76 317
Gusset assembly - PL15	5083-H116	6	m2	40,5	241	32,40	36,00	39,60	7 821	8 690	9 559
Hangers - PL120	5383-0	3	m2	324,0	972	40,88	45,42	49,96	39 733	44 148	48 563
Total					91173				5 107 667	5 675 185	6 242 704
Mass per m					7598			Cost/kg	56,02	62,25	68,47
Transverse mass				8,8 %	8062						

7.2.2 Welding cost

The welding cost is split into three segments. First, welding cost of the parts that form each 12m-section is calculated based on calculated lengths and assumed weld category. Second, the welds required for assembling two 12m-sections are calculated. Last, the site weld for joining 120m-sections, including an efficiency scale factor of 3.0, are calculated and distributed (divided by 10) on the cost of each 12m-section.

When the welding cost is estimated in Table 7-4, reference is given to codes from Table 7-1, which provides the estimated cost per category. This is further multiplied with the welding length per 12m sections. For example, the weld between the deck beams and the top deck profiles is assumed to be 7mm fillet and category W2, see also Figure 7-1.



> *Figure 7-1 The weld joining the deck beams and top deck panel (W2 - 7mm fillet)*

> *Table 7-4 Panel Concept Girder Welding Cost*

Assembly of parts to 12m sections	Weld info				Cost [NOK/m]			Cost [NOK]		
Description	Weld [m]	Weld cat	Weld type	Weld [mm]	Low	Med	High	Low	Med	High
Top/side/bottom panels - longitudinal	528	W9	PP1	10	291,3	475,4	623,3	153 789	250 996	329 120
Bulkhead panels	284	W8	PP1	7	145,6	237,7	311,7	41 360	67 503	88 513
Top - Beams to deck	336	W2	F	7	136,6	225,1	311,7	45 886	75 631	104 720
Bottom - Beams to deck	72	W1	F	5	72,8	118,8	155,8	5 243	8 557	11 220
Joiner profiles to panels	142	W12	FP1	10	291,3	475,4	623,3	41 360	67 503	88 513
Corner profiles to panels	288	W2	F	7	136,6	225,1	311,7	39 331	64 827	89 760
Gusset assembly - PL15	70	W2	F	7	136,6	225,1	311,7	9 560	15 756	21 817
Nodes / hangers [kg]	12	W6	F	14	582,5	950,7	1246,7	6 990	11 409	14 960
Total	1732							343 519	562 181	748 623
COST/kg								3,17	5,18	6,90

Joining 12m sections	Weld info				Cost [NOK/m]			Cost [NOK]		
Description	Weld [m]	Weld cat	Weld type	Weld [mm]	Low	Med	High	Low	Med	High
Top/side/bottom panels - transverse	142	W12	FP1	10	291,3	475,4	623,3	41 360	67 503	88 513
Beams - top deck webs - reinforced	2	W13	FP2	20	582,5	950,7	1246,7	1 258	2 054	2 693
Beams - top deck flanges - reinf.	1	W19	FP2	60	3276,8	5347,9	7012,5	4 587	7 487	9 818
Beams - top deck webs	10	W14	FP2	22	728,2	1188,4	1558,3	7 065	11 530	15 119
Beams - top deck flanges	5	W18	FP2	49	2475,8	4040,6	5298,3	12 131	19 799	25 962
Beams - bottom deck webs	2	W12	FP1	10	291,3	475,4	623,3	524	856	1 122
Beams - bottom deck flanges	1	W13	FP2	20	582,5	950,7	1246,7	437	713	935
Total	163							67 363	109 941	144 161
COST/kg								0,62	1,01	1,33

Joining 120m sections - at site	Efficiency factor				Cost			Cost		
Description					Low	Med	High	Low	Med	High
Cost - Joining 12m sections	3,0							202 088	329 824	432 484
Total per 12m section								20 209	32 982	43 248
COST/kg								0,19	0,30	0,40

TOTAL COST [NOK]								431 091	705 104	936 033
TOTAL COST [NOK/kg]								3,97	6,50	8,62

7.2.3 Cost summary for Panel Concept

Then, summarized, the cost of the Panel Concept is estimated to be:

> *Table 7-5 Panel Concept, Girder Cost Summary*

PANEL CONCEPT	Cost [NOK]			Cost [NOK/kg]		
	Low	Med	High	Low	Med	High
Material	5 107 667	5 675 185	6 242 704	56,02	62,25	68,47
Weld	431 091	705 104	936 033	3,97	6,50	8,62
Handling	836 337	899 360	952 474	9,17	9,86	10,45
Facility & inst.	808 650	898 500	988 350	7,45	8,28	9,11
TOTAL 12m	7 183 745	8 178 150	9 119 561	76,62	86,89	96,65
TOTAL 1200m	718 374 450	817 814 989	911 956 103	76,62	86,89	96,65

7.3 Plate concept

7.3.1 Material cost

The material for the plate concept is divided by rolled plate products (60% - PL120/28/20) and extruded profiles (40% - T-profile, U-stiffener, RHS). Only the aerodynamic side skirts are FSW-panels (1.5%). Therefore, the material cost is relatively low.

> *Table 7-6 Plate Concept, Girder Material Cost, 12m section.*

Material - Plate concept						Cost [NOK/kg]			Cost [NOK]		
Description	Alloy	Quantum	Unit	Mass/unit	Mass [kg]	Low	Med	High	Low	Med	High
PL28 x 70.18m in skin	5083-H116	842	m2	75,6	63667	37,64	41,82	46,00	2 396 310	2 662 566	2 928 823
T600x250x20x20 - 3off	6005A-T6	211	m	44,8	9432	33,26	36,96	40,66	313 752	348 614	383 475
RHS250x12 - 52m - 3off	6082-T6	156	m	31,0	4836	33,26	36,96	40,66	160 865	178 739	196 612
PL20 Gusset - 8m2 - 3off	5083-H116	24	m2	54,0	1296	32,40	36,00	39,60	41 990	46 656	51 322
U250x250x150x8x8	6005A-T6	276	m	13,9	3836	33,26	36,96	40,66	127 614	141 793	155 973
U300x300x250x10x10	6005A-T6	312	m	22,4	6989	33,26	36,96	40,66	232 475	258 306	284 137
U300x300x250x30x10	6005A-T6	120	m	34,9	4188	33,26	36,96	40,66	139 310	154 788	170 267
U300x300x200x37.5x10	6005A-T6	312	m	37,5	11700	33,26	36,96	40,66	389 189	432 432	475 675
Hangers - PL120	5383-0	3	m2	324,0	972	40,88	45,42	49,96	39 733	44 148	48 563
Areodynamic side skirts		108	m2	15,0	1620	56,68	62,98	69,27	91 819	102 021	112 223
Total					108537				3 933 058	4 370 064	4 807 070
Mass per m					9045			Cost/kg	36,24	40,26	44,29
Transverse mass				14,3 %	15564						

7.3.2 Welding cost

Just as for the panel concept, the welding cost is split in three segments. First, welding cost of the parts that form each 12m-section is calculated based on calculated lengths and assumed weld category. Second, the welds required for assembling two 12m-sections are calculated. Last, the site weld for joining 120m-sections, including an efficiency scale factor of 3.0, is calculated and distributed (divided by 10) on the cost of each 12m-section.

As already described in 7.2.2, welding cost is estimated in Table 7-7 with reference to codes from Table 7-1 providing the estimated cost per category. This is further multiplied with the welding length per 12m sections.

> *Table 7-7 Plate Concept Girder Welding Cost*

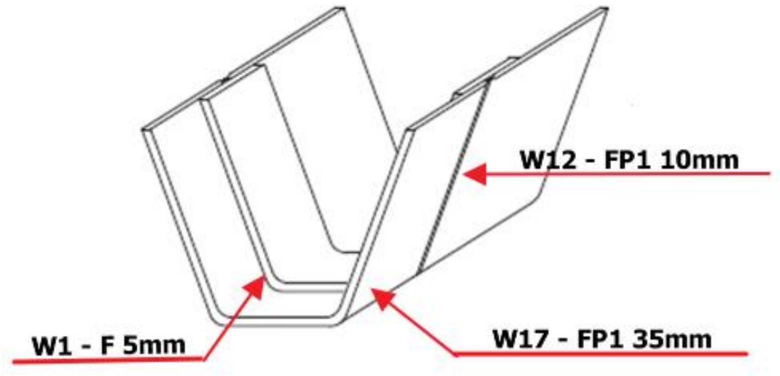
Assembly of parts to 12m sections	Weld info				Cost [NOK/m]			Cost [NOK]		
Description	Weld [m]	Weld cat	Weld type	Weld [mm]	Low	Med	High	Low	Med	High
Deck plate - longitudinal to bridge	240	W11	PP2	28	873,8	1426,1	1870,0	209 712	342 267	448 800
Stiffener to deck - 85no	2040	W7	PP+F	5+5	145,6	237,7	311,7	297 092	484 878	635 800
Transvers frame to stiffener pr 12m	273	W7	PP+F	5+5	145,6	237,7	311,7	39 758	64 888	85 085
Transvers frame to PL30 pr 12m	264	W7	PP+F	5+5	145,6	237,7	311,7	38 473	62 792	82 336
Gussets in frame pr 12m	86	W7	PP+F	5+5	145,6	237,7	311,7	12 583	20 536	26 928
Braces to gussets pr 12m	36	W7	PP+F	5+5	145,6	237,7	311,7	5 243	8 557	11 220
Nodes / hangers [kg]	12	W6	F	14	582,5	950,7	1246,7	6 990	11 409	14 960
Total	2952							609 851	995 326	1 305 129
COST/kg								5,62	9,17	12,02

Joining 12m sections	Weld info				Cost [NOK/m]			Cost [NOK]		
Description	Weld [m]	Weld cat	Weld type	Weld [mm]	Low	Med	High	Low	Med	High
Deck plate - transverse to bridge	70	W16	FP2	28	1583,8	2589,4	3428,3	111 153	181 721	240 600
Backing plates inside stiffener	164	W1	F	5	72,8	118,8	155,8	11 927	19 466	25 526
Stiffener joint with backing - webs	55	W12	FP1	10	291,3	475,4	623,3	16 092	26 264	34 439
Stiffener joint with backing - flanges	5	W17	FP1	35	2038,9	3327,6	4363,3	10 602	17 303	22 689
Stiffener joint with backing - flanges	3	W15	FP1	28	728,2	1188,4	1558,3	1 820	2 971	3 896
Stiffener joint with backing - flanges	12	W12	FP1	10	291,3	475,4	623,3	3 568	5 823	7 636
Total	309							155 164	253 549	334 786
COST/kg								1,43	2,34	3,08

Joining 120m sections - at site	Efficiency factor	Cost [NOK/m]			Cost [NOK]		
Description		Low	Med	High	Low	Med	High
Cost - Joining 12m sections	3,0				465 491	760 648	1 004 358
Total per 12m section					46 549	76 065	100 436
COST/kg					0,43	0,70	0,93

TOTAL COST [NOK]								811 564	1 324 940	1 740 351
TOTAL COST [NOK/kg]								7,48	12,21	16,03

As an example, joining the stiffeners to the deck requires weld of backing plate (W1) and single-sided full penetration welds with permanent backing with varying thickness in webs and flanges (W12, W17). See Figure 7-2 below for an illustration of these welds.



> *Figure 7-2 Joining of stiffeners*

7.3.3 Cost summary for Plate Concept

Then, summarized, the cost of the Plate Concept is estimated to be:

> *Table 7-8 Plate Concept, Girder Cost Summary*

PLATE CONCEPT	Cost [NOK]			Cost [NOK/kg]		
	Low	Med	High	Low	Med	High
Material	3 933 058	4 370 064	4 807 070	36,24	40,26	44,29
Weld	811 564	1 324 940	1 740 351	7,48	12,21	16,03
Handling	1 816 336	1 934 412	2 029 957	16,73	17,82	18,70
Facility & inst.	808 650	898 500	988 350	7,45	8,28	9,11
TOTAL 12m	7 369 607	8 527 915	9 565 728	67,90	78,57	88,13
TOTAL 1200m	736 960 727	852 791 548	956 572 786	67,90	78,57	88,13

7.4 Transverse panel concept

7.4.1 Material cost

As for the previous panel concept, this transverse panel concept utilizes to a great extent the advantages with extrusion of aluminium profiles and Friction Stir Welding (FSW) joining technology (99.3%). The content of rolled plate products is very low (0.7%).

> *Table 7-9 Transverse Panel Concept Material Cost, 3.9m section*

Material - Revised Panel concept						Cost [NOK/kg]			Cost [NOK]		
Description	Alloy	Quantum	Unit	Mass/unit	Mass [kg]	Low	Med	High	Low	Med	High
FSW Top deck	6005A-T6	115	m2	103,5	11882	55,84	62,04	68,24	663 453	737 170	810 887
FSW Transverse bulkhead	6005A-T6	153	m2	42,0	6426	55,84	62,04	68,24	358 802	398 669	438 536
FSW Bottom deck	6005A-T6	89	m2	91,7	8143	55,84	62,04	68,24	454 670	505 189	555 708
FSW Side bulkheads	6005A-T6	59	m2	91,7	5429	55,84	62,04	68,24	303 114	336 793	370 472
Joiner profiles - transv.	6005A-T6	71	m	26,2	1861	37,57	41,75	45,92	69 921	77 690	85 459
Joiner profiles - long.	6005A-T6	37	m	26,2	969	37,57	41,75	45,92	36 423	40 471	44 518
Hangers - PL120	5383-0	1	m2	324,0	249	40,88	45,42	49,96	10 188	11 320	12 452
Total					34959				1 896 572	2 107 302	2 318 032
Mass per m					8964			Cost/kg	54,25	60,28	66,31
Transverse mass				18,4 %	6426						

7.4.2 Welding cost

The welding cost is split in two segments. First, welding cost for assembling each of the 3.9m sections (individually and together). Secondly, welding cost for site assembly of 120.9m sections, including an efficiency scale factor of 3.0, distributed (divided by 31) on the cost of each of the 3.9m sections.

For this concept, almost all welding are single sided full penetration welds with backing, due to redesign of decking profiles and joiners.

> *Table 7-10 Transverse Panel Concept Girder Welding Cost*

Assembly of parts to 3.9m sections	Weld info				Cost [NOK/m]			Cost [NOK]		
Description	Weld [m]	Weld cat	Weld type	Weld [mm]	Low	Med	High	Low	Med	High
Top deck - Top deck (joiner profile)	47	W22	FP1	14	364,1	594,2	779,2	17 039	27 809	36 465
Top deck - Top deck (joiner profile)	124	W22	FP1	14	364,1	594,2	779,2	45 187	73 749	96 704
Top deck - Side deck (joiner profile)	31	W22	FP1	14	364,1	594,2	779,2	11 359	18 539	24 310
Side deck - Side deck (joiner profile)	31	W22	FP1	14	364,1	594,2	779,2	11 359	18 539	24 310
Side deck - Side deck (joiner profile)	64	W22	FP1	14	364,1	594,2	779,2	23 301	38 030	49 867
Bottom deck - Side deck (joiner profile)	31	W22	FP1	14	364,1	594,2	779,2	11 359	18 539	24 310
Bottom deck - Bottom deck (joiner profile)	16	W22	FP1	14	364,1	594,2	779,2	5 680	9 270	12 155
Bottom deck - Bottom deck (joiner profile)	96	W22	FP1	14	364,1	594,2	779,2	34 952	57 044	74 800
Transverse bulkhead - top/side/bottom	144	W20	FP1	5	72,8	118,8	155,8	10 490	17 120	22 449
Transv bulkh. - transv. bulkh.	77	W20	FP1	5	72,8	118,8	155,8	5 607	9 151	11 999
Joiner - joiner (butt connection)	7	W9	PP1	10	291,3	475,4	623,3	2 144	3 499	4 588
Joiner - joiner (T-connection)	18	W9	PP1	10	291,3	475,4	623,3	5 359	8 747	11 469
Nodes / hangers (distr. on 3 sections)	8	W6	F	14	582,5	950,7	1246,7	4 660	7 606	9 973
Total	695							188 497	307 643	403 399
COST/kg								5,39	8,80	11,54

Welds for joining of 120.9m sections	Weld info				Cost [NOK/m]			Cost [NOK]		
Description	Weld [m]	Weld cat	Weld type	Weld [mm]	Low	Med	High	Low	Med	High
Top deck - Top deck (joiner profile)	47	W22	FP1	14	364,1	594,2	779,2	17 039	27 809	36 465
Top deck - Top deck (joiner profile)	124	W22	FP1	14	364,1	594,2	779,2	45 187	73 749	96 704
Top deck - Side deck (joiner profile)	31	W22	FP1	14	364,1	594,2	779,2	11 359	18 539	24 310
Side deck - Side deck (joiner profile)	31	W22	FP1	14	364,1	594,2	779,2	11 359	18 539	24 310
Side deck - Side deck (joiner profile)	64	W22	FP1	14	364,1	594,2	779,2	23 301	38 030	49 867
Bottom deck - Side deck (joiner profile)	31	W22	FP1	14	364,1	594,2	779,2	11 359	18 539	24 310
Bottom deck - Bottom deck (joiner profile)	16	W22	FP1	14	364,1	594,2	779,2	5 680	9 270	12 155
Bottom deck - Bottom deck (joiner profile)	96	W22	FP1	14	364,1	594,2	779,2	34 952	57 044	74 800
Joiner - joiner (T-connection)	18	W9	PP1	10	291,3	475,4	623,3	5 359	8 747	11 469
Total	459							165 597	270 267	354 390
COST/kg								4,74	7,73	10,14

Joining 120.9m sections - at site	Efficiency factor				Cost			Cost		
Description					Low	Med	High	Low	Med	High
Cost - Joining 120.9m sections	3,0							496 790	810 801	1 063 170
Total per 3.9m section								16 025	26 155	34 296
COST/kg								0,46	0,75	0,98

TOTAL COST [NOK]								204 523	333 797	437 695
TOTAL COST [NOK/kg]								5,85	9,55	12,52

7.4.3 Cost summary for Transverse panel concept

Then, summarized, the cost of the transverse panel concept is estimated to be:

> *Table 7-11 Transverse Panel Concept, Girder Cost Summary*

TRANSVERSE PANEL CONCEPT	Cost [NOK]			Cost [NOK/kg]		
	Low	Med	High	Low	Med	High
Material	1 896 572	2 107 302	2 318 032	54,25	60,28	66,31
Weld	204 523	333 797	437 695	5,85	9,55	12,52
Handling	396 025	425 759	449 655	11,33	12,18	12,86
Facility & inst.	262 811	292 013	321 214	7,52	8,35	9,19
TOTAL 3.9m	2 759 931	3 158 871	3 526 596	78,95	90,36	100,88
TOTAL 1200m	850 058 830	972 932 132	1 086 191 441	78,95	90,36	100,88

7.5 Comparison between bridge girder cost for plate concept and panel concept

A summary table, comparing the cost between the two concepts, is included below. From the numbers in the table, the panel concept is slightly cheaper (4%) than the plate concept. The main driving reason for the plate concept cost to come more expensive is the welding and handling costs, which is approximately twice the plate concept. The reduced material cost for the plate concept does not completely even out the differences.

The cost deviation of 4% for the two concepts is, however, not significant with regards to the cost compared to the steel alternative (see chapter 6.5). As the two concepts cost more or less the same, it must be considered an advantage for the project to still have two possible design concepts for aluminium material to optimize. Given that this is an early design stage and the potential for optimization is significant, a difference of 4% makes it difficult to reach a conclusion.

A summary table of bridge girder cost for the two concepts is given below.

> *Table 7-12 Bridge Girder Cost Summary, Comparison between Concepts*

PLATE CONCEPT	Cost [NOK]			Cost [NOK/kg]		
	Low	Med	High	Low	Med	High
Material	3 933 058	4 370 064	4 807 070	36,24	40,26	44,29
Weld	811 564	1 324 940	1 740 351	7,48	12,21	16,03
Handling	1 816 336	1 934 412	2 029 957	16,73	17,82	18,70
Facility & inst.	808 650	898 500	988 350	7,45	8,28	9,11
TOTAL 12m	7 369 607	8 527 915	9 565 728	67,90	78,57	88,13
TOTAL 1200m	736 960 727	852 791 548	956 572 786	67,90	78,57	88,13

PANEL CONCEPT	Cost [NOK]			Cost [NOK/kg]		
	Low	Med	High	Low	Med	High
Material	5 107 667	5 675 185	6 242 704	56,02	62,25	68,47
Weld	431 091	705 104	936 033	3,97	6,50	8,62
Handling	836 337	899 360	952 474	9,17	9,86	10,45
Facility & inst.	808 650	898 500	988 350	7,45	8,28	9,11
TOTAL 12m	7 183 745	8 178 150	9 119 561	76,62	86,89	96,65
TOTAL 1200m	718 374 450	817 814 989	911 956 103	76,62	86,89	96,65

DIFF PLATE - PANEL	Cost [NOK]			Cost [NOK/kg]		
	Low	Med	High	Low	Med	High
Material	- 1 174 609	- 1 305 121	- 1 435 633	- 19,78	- 21,98	- 24,18
Weld	380 474	619 835	804 318	3,51	5,71	7,41
Handling	979 998	1 035 051	1 077 482	7,56	7,96	8,26
Facility & inst.	-	-	-	-	-	-
TOTAL 12m	185 863	349 766	446 167	- 8,72	- 8,31	- 8,51
TOTAL 1200m	18 586 277	34 976 559	44 616 684	- 8,72	- 8,31	- 8,51

RATIO PLATE / PANEL	Cost [NOK]		
	Low	Med	High
Material	77 %	77 %	77 %
Weld	188 %	188 %	186 %
Handling & inst.	217 %	215 %	213 %
Facility	100 %	100 %	100 %
TOTAL 12m	103 %	104 %	105 %
TOTAL 1200m	103 %	104 %	105 %

7.7 Comparison of girder cost between transverse panel concept and original panel concept

The summary table below compares the cost between the two panel concepts. As can be seen from the tables, the total cost of the bridge girder for the transverse panel concept increases with 154MNOK, or 19%, compared with the original concept. The increase is caused by additional costs for:

- **Material (52.5% of total increase)**
- **Welding (20.9% of total increase)**
- **Handling (26.6% of total increase) (see chapter 7.1.1.3 for definition)**

> *Table 7-13 Bridge Girder Cost Summary, Comparison between the two panel concepts*

TRANSVERSE PANEL CONCEPT	Cost [NOK]			Cost [NOK/kg]		
	Low	Med	High	Low	Med	High
Material	1 896 572	2 107 302	2 318 032	54,25	60,28	66,31
Weld	204 523	333 797	437 695	5,85	9,55	12,52
Handling	396 025	425 759	449 655	11,33	12,18	12,86
Facility & inst.	262 811	292 013	321 214	7,52	8,35	9,19
TOTAL 3.9m	2 759 931	3 158 871	3 526 596	78,95	90,36	100,88
TOTAL 1200m	850 058 830	972 932 132	1 086 191 441	78,95	90,36	100,88

ORIGINAL PANEL CONCEPT	Cost [NOK]			Cost [NOK/kg]		
	Low	Med	High	Low	Med	High
Material	1 659 992	1 844 435	2 028 879	56,02	62,25	68,47
Weld	140 104	229 159	304 211	4,73	7,73	10,27
Handling	271 810	292 292	309 554	9,17	9,86	10,45
Facility & inst.	262 811	292 013	321 214	8,87	9,85	10,84
TOTAL 3.9m	2 334 717	2 657 899	2 963 857	78,79	89,70	100,02
TOTAL 1200m	719 092 824	818 632 804	912 868 059	78,79	89,70	100,02

DIFF TRANSVERSE - ORIGINAL PANEL	Cost [NOK]			Cost [NOK/kg]		
	Low	Med	High	Low	Med	High
Material	236 580	262 867	289 153	- 1,77	- 1,97	- 2,16
Weld	64 418	104 639	133 484	1,12	1,81	2,25
Handling	124 216	133 467	140 101	2,16	2,31	2,42
Facility & inst.	-	-	-	- 1,35	- 1,50	- 1,65
TOTAL 3.9m	425 214	500 972	562 738	0,15	0,66	0,85
TOTAL 1200m	130 966 006	154 299 328	173 323 382	0,15	0,66	0,85

RATIO TRANSVERSE / ORIGINAL	Cost [NOK]		
	Low	Med	High
Material	114 %	114 %	114 %
Weld	146 %	146 %	144 %
Handling & inst.	146 %	146 %	145 %
Facility	100 %	100 %	100 %
TOTAL 3.9m	118 %	119 %	119 %
TOTAL 1200m	118 %	119 %	119 %

7.5 Aluminium bridge girder cost savings not quantified

For an aluminium bridge girder, some cost items that can be omitted or reduced drastically compared to a steel bridge girder are listed below. They might have both Capex, Opex and decommissioning cost/residual value implications, but those items have not been included in this study.

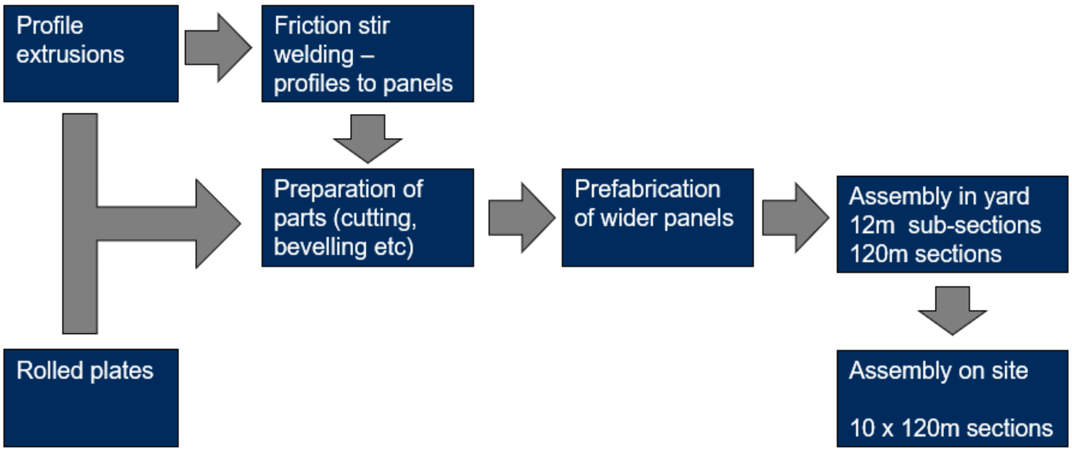
- **Maintenance / Surface protection of bridge girder structures**
- **Humidity control**
- **Higher material value when bridge girder is recycled**

8 ASSEMBLY OF 120 M SECTIONS

8.1 Fabrication/assembly of the Panel Concept

In brief, the fabrication process is based upon two phases, prefabrication of deck and bulkhead panels, and assembly of panels to form the bridge girder sections.

The prefabrication phase can be split in manufacturing of the material (extrusions, rolled products), joining profiles by Friction stir welding, preparation of parts for joining, and joining of parts to wider panels ready for the assembly phase, see figure below.



> *Figure 8-1 Assembly flowchart*

8.2 Prefabrication of wide panels

The deck and bulkhead panels are delivered from Hydro in 2.4 – 3.1m wide panels. These are welded together to larger panels, including joiner / corner profiles and deck beams in a prefabrication process and location. This work could typically be performed at Leirvik AS yard, alternatively with contribution from several other local companies which all have high expertise in aluminium welding.

8.3 Assembly of 120m bridge sections

For the assembly phase, the wider panels are joined together to form the 120m bridge girder section. A 120m section is formed by 10 12m-sections. Once the first sub-section is completely assembled, it is skidded on tracks 12m towards the end of the workshop. Then, the next sub-section is assembled and joined to the first. This process continues in 10 steps, and the bridge girder section will gradually be increased in length and "grow" out of the workshop until it is complete.

A crane can operate in the centre of the workshop, and by this serve similar welding processes in each end of the workshop. Hence, two 120m-sections can be manufactured simultaneously by use of this philosophy. For further description of possible prefabrication yards and assembly areas, see Chapter 8.7.

8.4 Assembly method for the Panel Concept

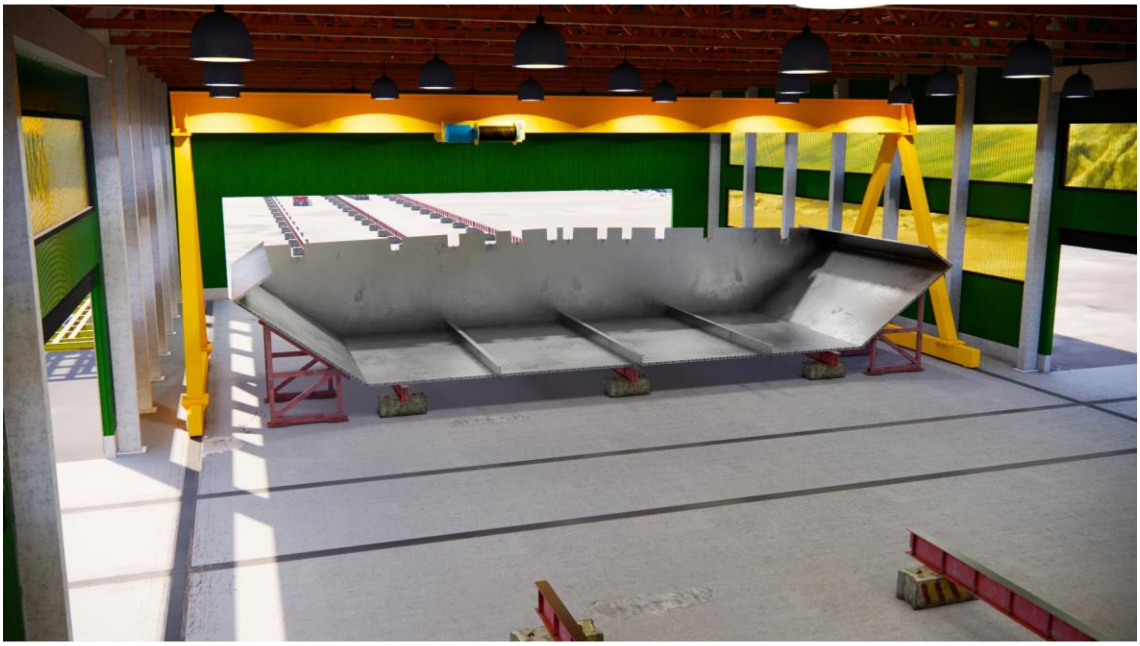
Stepwise, the assembly process is:

1. Empty workshop – with preinstalled skid tracks

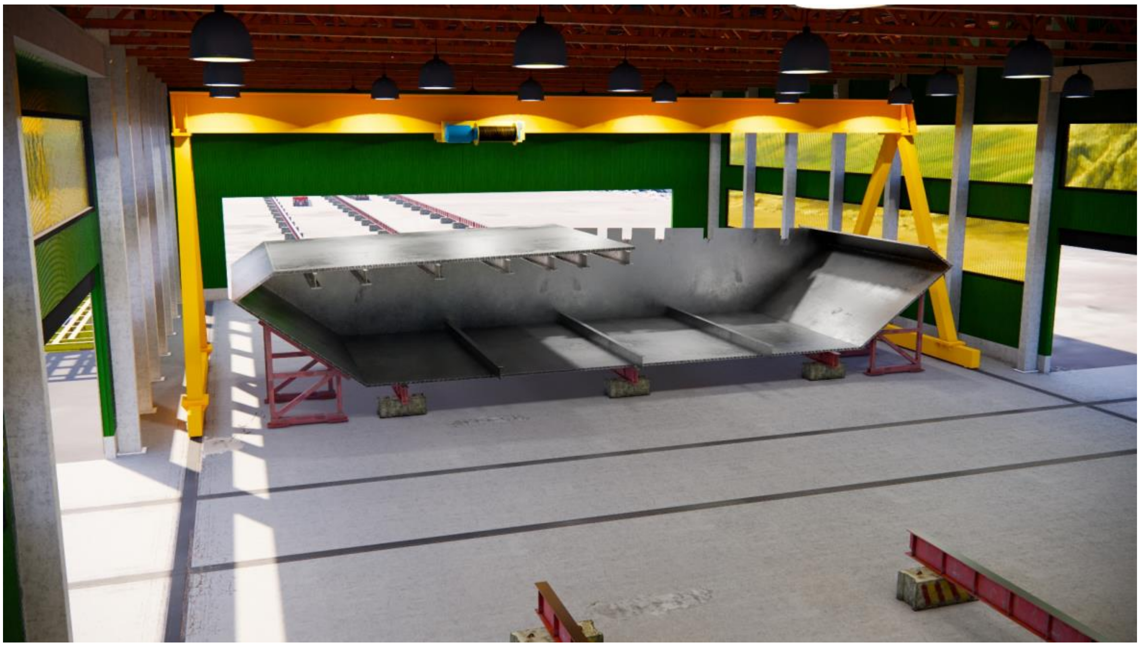
2. Start with the bottom deck panel, locate on the skid tracks

3. Install the transverse bulkheads (two pieces, temporary structures not shown)

4. Raise the side bulkheads, support them temporarily by frame structures



5. Install the top deck (two pieces, temporary structures at beam ends not shown)



6. Skid the first sub-section 12m out of the workshop to prepare for the next sub-section

7. Form the next sub-section according to the same sequence as for the first, but also join the parts together with the first sub-section.

8. Second sub-section completed, ready for skidding 12m towards workshop opening.

9. Assembly of two 120m-sections in parallel by feeding the panels through the mid-gate of the workshop, the gantry crane can be operated at both workshop ends.


LEIRVIK

8.5 Assembly method for the Plate Concept

The fabrication process for the plate concept will basically be similar to the steel concept, since its design is quite parallel. But, in brief the fabrication sequence would be:

1. **Install backing plates inside U-stiffener at end joints**
2. **Pre-assemble / weld U-stiffeners to PL28 (format 3000 x 12000)**
3. **Pre-assemble / weld T-section to PL28 & U-stiffeners (within 12m length)**
4. **Pre-assemble / weld braces to gusset plates (in groups of three)**
5. **Install bottom deck on skid tracks**
6. **Install side panels and weld to adjacent structures**
7. **Install transverse braces (4 groups of braces per 4m) and weld to adjacent structures**
8. **Install top deck and weld to adjacent structures**

Once one 12m-section is assembled, it will be skidded 12m towards the workshop end to prepare for the next sub-section, just as is planned for the panel concept.

8.6 Assembly method for the Transverse Panel Concept

In general, assembly method for the improved transverse panel concept are similar to the method for the original panel concept. As the identical sections now spans 3.9m instead of the previous 12m, the number of identical section assembly repetitions for the bridge girders increases from 100 (based on 1200m girder length) to 308 (equals 1201.2m). Further, 31off 3.9m sections would form a 120.9m long part of the girder.

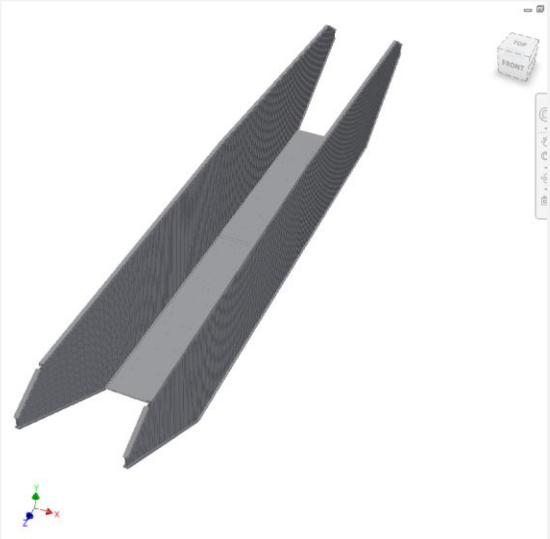
The principles from the production line set-up will be suitable for the transverse panel concept as well. The use of production line skid tracks, automated welding and handling techniques (by robots) and suitable supports and fixturing for specific operations during assembly is crucial for an effective construction flow. This set-up should be studied more thoroughly in further work with this aluminium bridge girder.

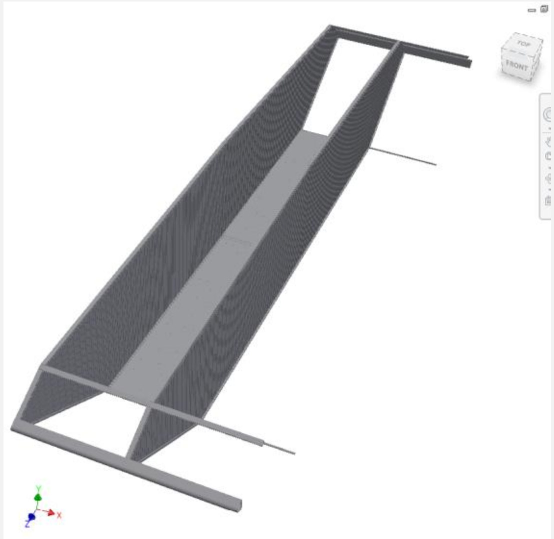
In principle, the assembly process is proposed as follows:

Assembly method for transverse panel concept

Step 1: Raise first transverse bulkhead

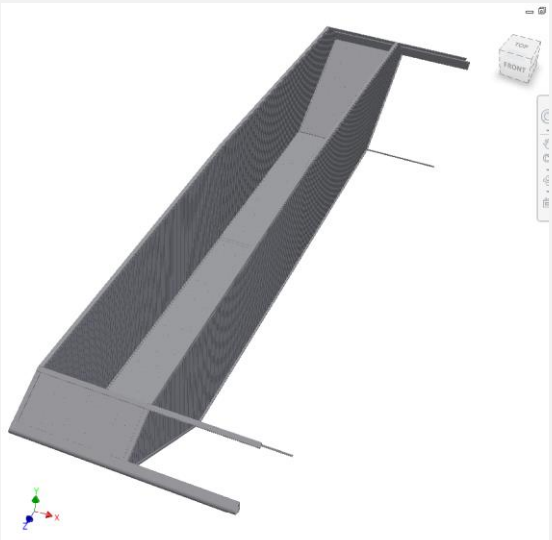
Step 2: Raise next transverse bulkhead

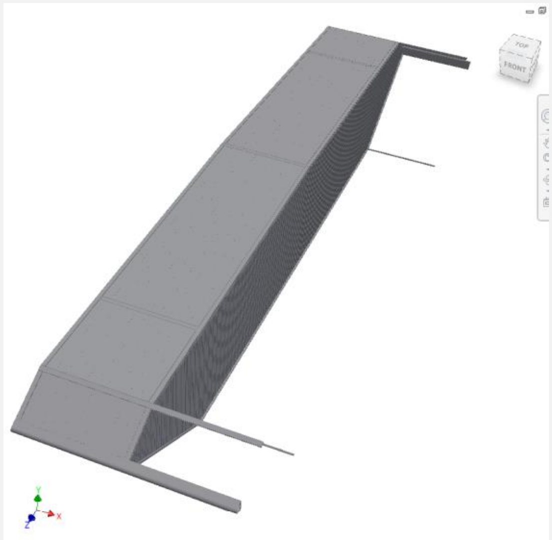




Step 3: Install bottom deck panel

Step 4: Install joiner profiles at corners

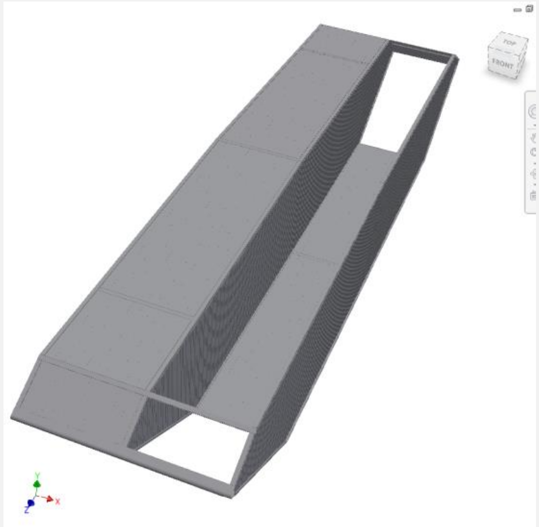




Step 5: Install lower and upper side panels

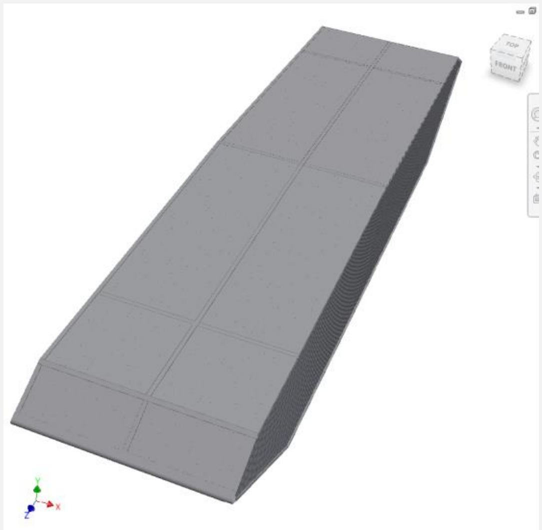
Step 6: Install top deck panel

Step 7: Raise next transverse bulkhead…



Step 8: …and continue with bottom deck…

Step 9: …and side panels…



Step 10: …and top deck.

The amount of pre-fabrication of sub-assemblies is expected to be significantly lower for this concept, as the panels and profiles could be delivered with fixed width and lengths, including any CNC-machined details which cannot be adopted by the extrusion. This is, however, not reflected in the cost estimate, and can potentially be a cost saving element. Most pre-fabrication would be related to the transverse bulkheads, where the vertical oriented panels needs be joined by MIG-welding, and the joiner profiles along the bulkhead perimeter must be attached.

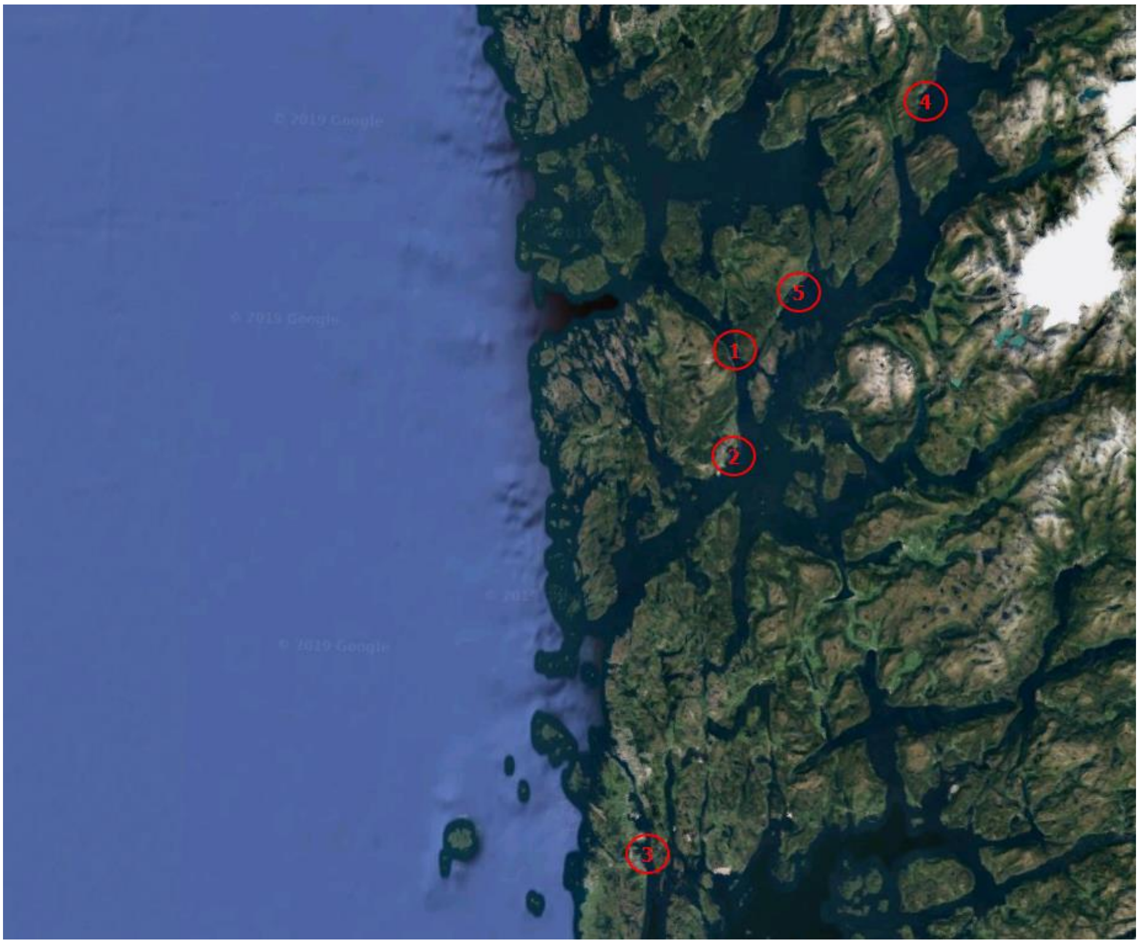
8.7 Pre-fabrication and assembly yards

Locally, nearby the Langenuen fjord crossing, several workshop companies are located, which all are world-leading aluminium providers for design and construction within the offshore, marine and infrastructure sectors. These are:

Company	Aluminium products	Location (dist. to site)
Leirvik AS	Living Quarters, Modules, Helidecks	Stord (16.5km)
Marine Aluminium	Helidecks, Telescopic bridges	Karmøy / Stord (74km / 16.5km)
Fjellstrand AS	Fast boats, ferries, elevator shafts	Omastrand (71.3km)
Oma båtbyggeri	Fast boats, ferries	Stord (16.5km)

The material quantum of this bridge girder is high, as well as the required area for both the prefabrication, assembly and storing of material and completed girder sections.

Therefore, in addition to the above companies which could focus on the prefabrication process of wider deck and bulkhead panels, an area located on Onarheim, Tysnes Island, is considered suitable for location of a new and tailor-made assembly yard. This area is located only 35km from the Langenuen crossing site.



> *Figure 8-2 Workshop companies nearby Langenuen fjord-crossing*

With respect to Figure 8-2, the locations of the following workshops are presented by corresponding numbers in the figure:
1 – Langenuen crossing site
2 – Leirvik AS / Oma Baatbyggeri / Marine Aluminium (Stord)
3 – Marine Aluminium (Karmøy)
4 – Fjellstrand AS (Omastrand)
5 – Tronds Marine (Onarheim, Tysnes) (possible assembly site)

> *Figure 8-3 Possible assembly site: Tronds Marine, Onarheim, Tysnes*

> *Figure 8-4 Possible assembly site, closeup*

> *Figure 8-5 Possible assembly site, available area*

> *Figure 8-6 Possible assembly site*

> *Figure 8-7 Possible assembly site, close-up*

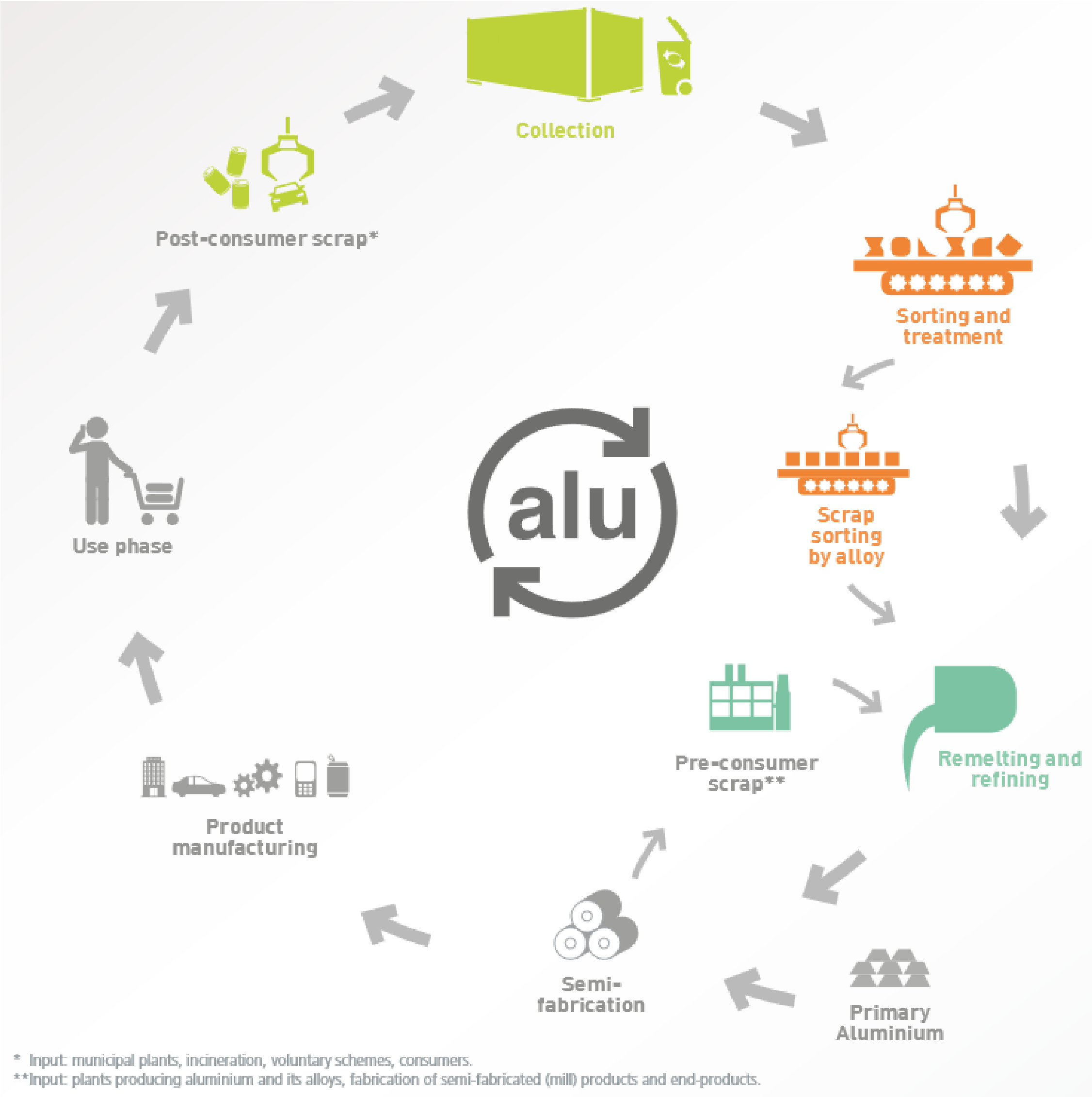
> *Figure 8-8 Possible assembly site, assembly hall*

This suggested area for an assembly yard, owned by Tronds Marine, is currently available and has a 140m quay plus a slipway. The overall dimensions of the area are 300m x 150m, hence is should fit the purpose of fabricating two 120m-sections out of each workshop end.

Other potential areas should also be investigated. A couple of alternatives may be Eldøyane (Stord), Hanøytangen (Askøy) and Kværner (Verdal), but due to the dimensions, the availability is limited. Reducing the bridge section length from 120m to e.g. 72m could increase the availability of yards for the assembly phase.

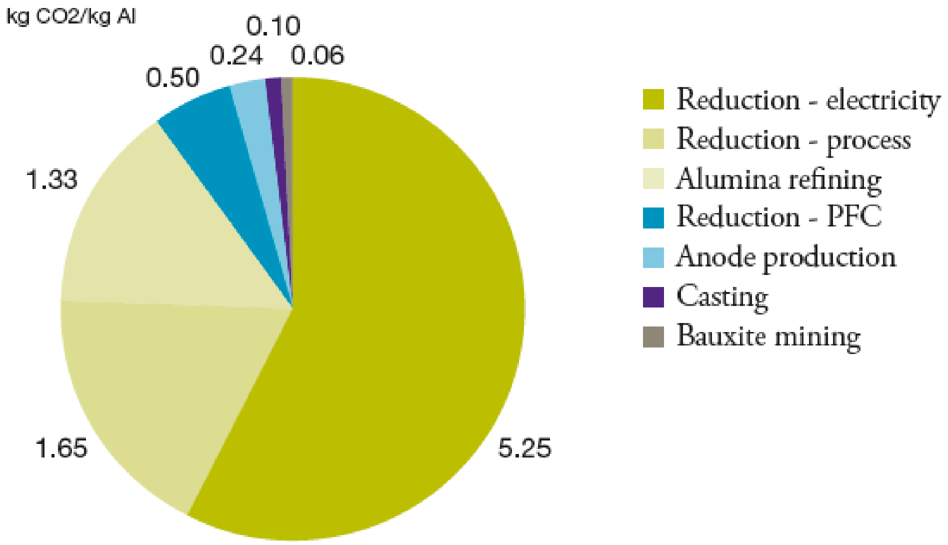
9 ENVIRONMENTAL IMPACT

9.1 Introduction

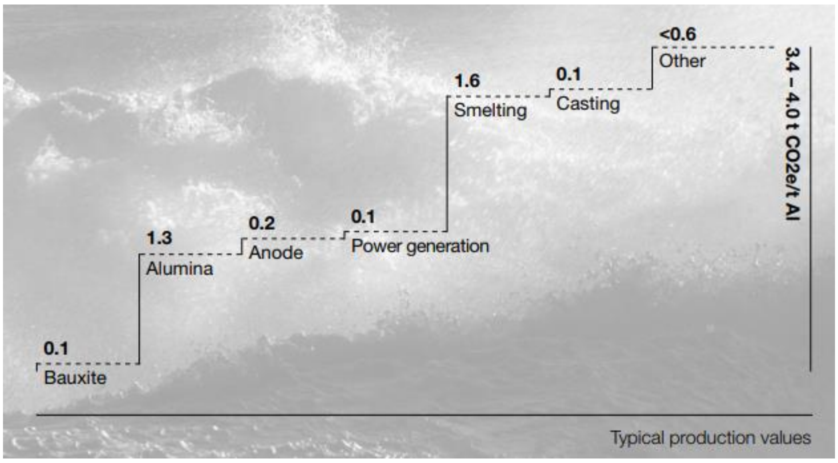


> *Figure 9-1 Aluminium value chain*

Primary aluminium production is the first step in the aluminium value chain shown in Figure 9-1. The largest contributor to CO_2 emissions in this mine to ingot value chain is the electricity to the smelter (Figure 9-2). Consequently, the power production process has a major impact on the total CO_2 emissions. As Norwegian aluminium production is based on renewable hydropower, the material has a low carbon footprint. In combination with advanced technology for low energy consumption, low direct emissions and post-consumer scrap recycling, this is utilized in the Hydro 4.0 material series, with a certified 4.0 kg CO2 per kg maximum carbon footprint [14](Figure 9-3).



> *Figure 9-2 Contributions to CO_2 emissions from the aluminium mine to ingot value chain*



> *Figure 9-3 Hydro 4.0 carbon footprint*

While primary aluminium production is energy-intensive, recycling of aluminium is extremely energy efficient. Only 5% of the energy required to produce primary aluminium is needed to re-melt aluminium. [15] In addition to the energy savings, emissions of greenhouse gases and pollutants are reduced, and encroachments in the landscape related to bauxite mining and refining are avoided.

The world's increasing stock of aluminium in use acts as a resource bank, over time delivering more and more practical use and value from the energy embodied in the metal. Around 75 percent of the almost one billion metric tons of aluminium ever produced is still in productive use, some of it has been through countless recycle loops through its lifecycle. Most of the aluminium being produced today enters long-life products like vehicles and building products. With average lifetimes of about 15 to 20 years for vehicles and 40 to 50 years for buildings, this means that most of the aluminium will not be available for recycling for many years. As a result, access to aluminium scrap is limited. Globally, less than 25 percent of the aluminium being produced came from post-consumer scrap sources in 2010. With an expected continued growth in aluminium demand, this share is not expected to increase significantly in the future.

In order to increase the use of post-consumer scrap in extrusions and flat-rolled products, new technology for collection and sorting of aluminium scrap is being developed. By utilizing this technology Hydro has established the Hydro 75R material series, that has a guaranteed content of minimum 75% post-consumer scrap. [16] This allows for a significant reduction in environmental impact, while the material quality is maintained.

For some applications such as within transportation, weight reduction realized through the use of aluminium allows reduced emissions in the use phase. For stationary structures such as suspension bridges, this is not relevant. However, reduced weight from an aluminium bridge girder, could allow reduced material consumption and thereby reduced emissions from other parts of the bridge structure such as concrete structures and steel cables. Moreover, elimination of surface treatment will have positive effect on the environmental impact.

9.2 Carbon footprint evaluation

In order to provide an initial evaluation of the environmental impact of an aluminium bridge girder for the Langenuen suspension bridge, calculation of CO_2 emissions will be used. The evaluation will be based on a relative comparison between the aluminium concepts and the steel reference concept, focusing on the main bridge elements tower, main cable, hangers and bridge girder (Table 9-1). For this initial comparison, the carbon footprint of the consumed materials only has been included. This is assumed to provide a correct comparison, as CO_2 emissions from the final fabrication and construction are small compared to the emissions from material production. Moreover, these emissions are expected to have minor impact on the differentiation between the concepts.

Emissions from transport of materials and completed structures depend of course on distance and shipping weight. Compared to steel, aluminium has an advantage related to reduced weight. Moreover, domestic production of aluminium material, as well as fabrication of the complete structure in Norway, has the potential to allow short shipping distance. The main limiting factor is extrusion and friction stir welding, as the Norwegian capacity for larger sections and double-sided panels is limited.

The paint consumption has been estimated from the covered surface area, assuming an average thickness of 0,4mm. As seen in the table, painting has been assumed for the steel girder only.

> *Table 9-1 Main volumes*

Bridge element	Steel - basis	Plate concept	Panel concept	Unit
Bridge girder	14 831	11 080	9 391	ton
Main cable (steel)	12 387	10 178	11 437	ton
Hangers (steel)	151	127	122	ton
Paint	35			ton

For the CO2 calculations, assumed emissions, as shown in Table 9-2, have been used. These numbers have been based on available Environmental Product Declarations from The Norwegian EPD foundation, and values used in an earlier study performed for the Norwegian Public Roads Administration. [16] [14] [17] [18] [19]

For the aluminium material, three different alternatives have been included:

- Extrusion ingot Europe: the average of Hydro's European ingot production
- Hydro 4.0: ingot produced according to the certified 4.0 route
- Hydro 75R: ingot produced according to the 75R certification

The EPDs used for aluminium cover production of extrusion billets. As the production of sheet ingots used for rolling of flat-rolled products are quite similar, the same EPDs have been used. Moreover, the CO_2 emissions from the rolling and extrusion process have been added. Data from European Aluminium shows 0,7 kg CO_2/kg for extrusion, and 0,4 kg CO_2/kg for sheet production. [20] However, since the detailed split between extrusions and flat-rolled plates has not been defined, an average of 0,55 kg CO_2/kg has been used for all aluminium material.

> *Table 9-2 Assumed CO_2 emissions*

Material	CO2	Unit	Source
Aluminium plates/profiles	2.88/4.55/6.25	ton/ton	EPD + European Aluminium [16] [14] [17] [20]
Steel plates	2.47	ton/ton	EPD [18]
Steel cable/hangers	2.68	ton/ton	NPRA report [19]
Paint	3.76	ton/ton	NPRA report [19]

The resulting CO_2 calculations for the bridge concepts are shown in Figure 9-4, Figure 9-5 and Figure 9-6. The aluminium concepts show reduced CO_2 emissions compared to the steel basis concept for the Hydro 75R material, while the other alternatives show increased CO_2 emissions. The correlation between CO_2 emissions from the aluminium material production and the total emission is shown in Figure 9-7. Based on this curve, it is found that Hydro 4.0 with 25% 75R will provide 40% environmental impact for the plate concept compared to steel, while only 20% is required for the panel concept.

It should be noted that this comparison is performed with no evaluation of the emissions from the steel plate production. If steel material with less environmental impact is used, this will of course change the results.

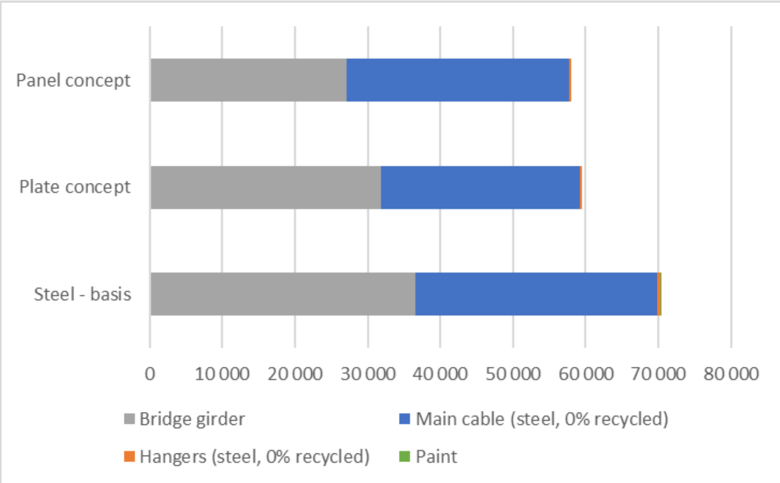


Figure 9-4 CO_2 footprint based on Hydro 75R [ton CO_2]

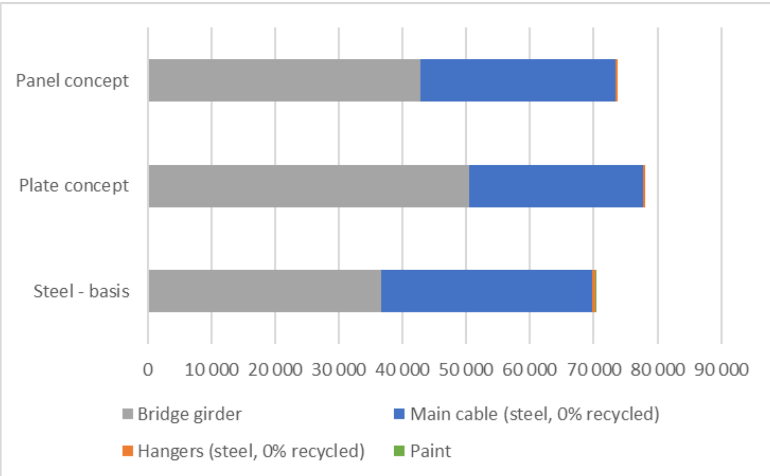


Figure 9-5 CO_2 footprint based on Hydro 4.0 material [ton CO_2]

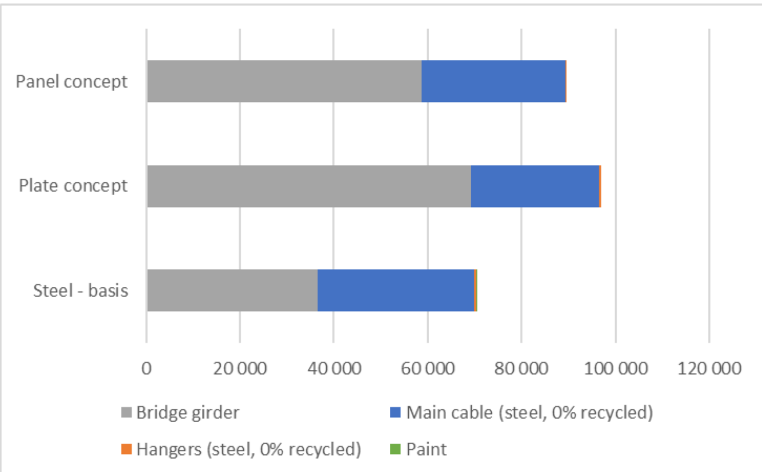
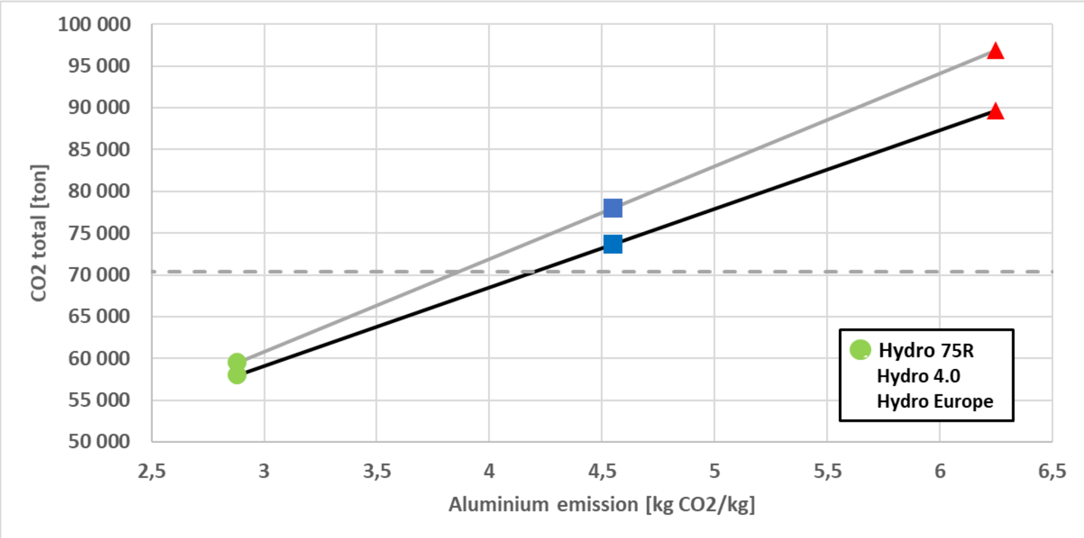


Figure 9-6 CO_2 footprint based on Hydro European average material [ton CO_2]



> *Figure 9-7 Sensitivity on aluminium CO2 emissions*

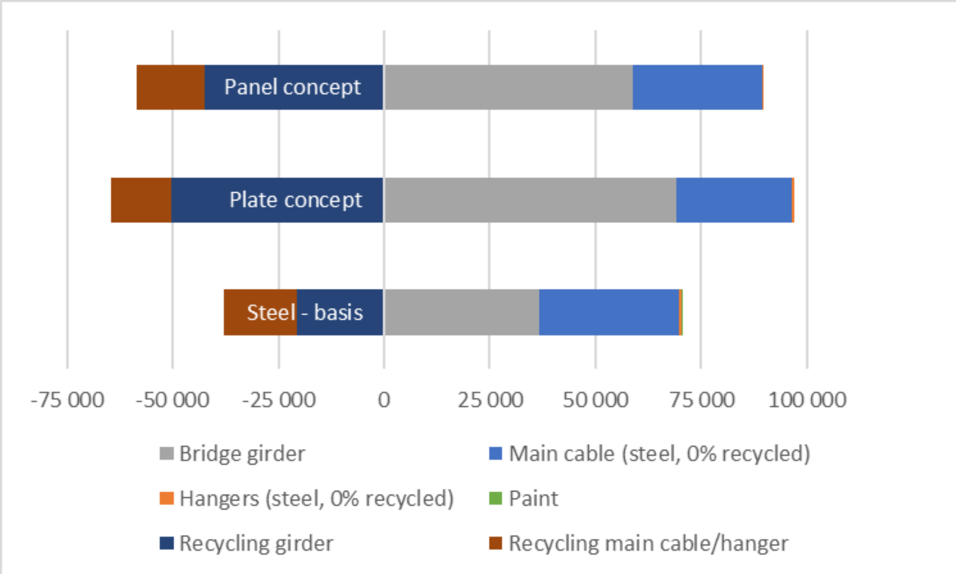
9.3 End of life evaluation

As described above, recycling of aluminium is extremely energy efficient, as only 5% of the energy required to produce primary aluminium is needed to re-melt aluminium. Moreover, aluminium can be recycled without degradation of quality. If this end of life recycling is included in the CO2 calculations, a picture as shown in Figure 9-8 and Figure 9-9 is found. This is based on Hydro European average aluminium material and recycling potential as shown in Table 9-3. For hangers and cables, values from the steel plate EPD have been assumed.

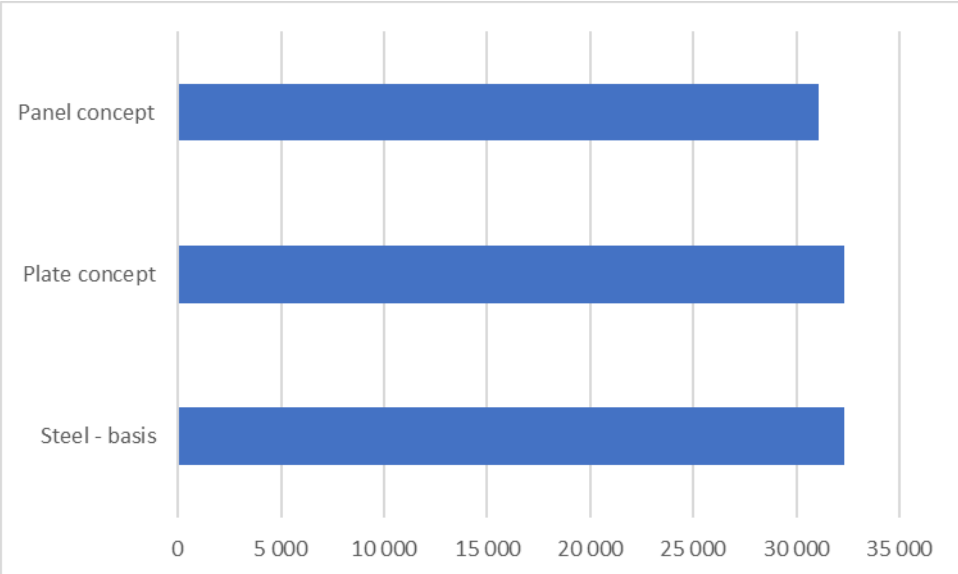
As can be seen from the results, the net CO2 emissions are lower for the aluminium panel concept compared to the steel basis design, while the plate concept is at the same level.

> *Table 9-3 Avoided burden assumptions*

	Steel	Aluminium	Unit
Recycling potential	-1.39 [18]	-4.53 [17]	kg CO_2/kg



> *Figure 9-8 CO_2 footprint including avoided burden [ton CO_2]*



> *Figure 9-9 Net CO_2 footprint including avoided burden [ton CO_2]*

10 RECOMMENDATIONS FOR FURTHER WORK

In general, all aspects of the concepts may be revisited and optimised in later phases. Below, a list of proposed actions for further investigation in later phases is populated, items rated most important in bold.

- **Geometry/Strength**
  - **Investigate fatigue properties of typical welds and details by tests and numerical methods.**
  - **Study further the connection between bulkheads and hangers, local hanger design**
  - **Optimise detailing further with respect to aluminium fatigue properties and profile design**
  - **Study effect of thermal variations**
  - **Include construction phase in the design of towers**
  - Verify torsional stiffness of complex geometry
  - Include detailing of walkway/manholes in bulkhead
  - Reduced section at 120 m splice, plate concept
  - Weighing thickness of base material plates and weld size/details to ensure, failure in base material
- **Loads**
  - **Including Proper wind data /directional mean wind distribution to calculate fatigue life more correctly**
  - **Refine fatigue calculations from traffic**
  - **Verify aerodynamics of proposed bridge girder (CFD/ Wind tunnel test)**
  - Study effect of strakes
  - Take new Metocean spec dated 02.12.2019 into account
- **Constructability and Market**
  - **Further development of production line set-up and cost model**
  - **Optimisation of assembly, handling and installation of 120m sections**
  - **Study prediction and consequences of construction/dimensional tolerances**
  - Construction area requirements, how to store sections
  - Develop details of maintenance cost savings
  - In-depth study of global availability of materials, fabricators and contractors

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