



Statens vegvesen

Ferry free E39 –Fjord crossings Bjørnafjorden

304624

Rev.	Publish date	Description	Made by	Checked by	Project appro.	Client appro.
0	15.08.2019	Issued for use	HeSky	KGS	KH	
Client						
 Statens vegvesen						
Contractor		Contract no.:				
Norconsult  		18/91094				

Document name:

Design of cable stayed bridge

Document no.:

SBJ-33-C5-OON-22-RE-019

Rev.:

0

Pages:

46



CONCEPT DEVELOPMENT FLOATING BRIDGE E39 BJØRNAFJORDEN

DESIGN OF CABLE STAYED BRIDGE

Norconsult 

 DR. TECHN.
OLAV OLSEN

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REPORT

Project name:

CONCEPT DEVELOPMENT FLOATING BRIDGE E39
BJØRNAFJORDEN

Document name:

DESIGN OF CABLE STAYED BRIDGE

Project number: 5187772/12777
Document number: SBJ-33-C5-OON-22-RE-019

Date: 15.08.2019
Revision: 0
Number of pages: 46

Prepared by: Henrik Skyvulstad, Simon Bjonness, Knut Gjerding-Smith

Controlled by: Knut Gjerding-Smith

Approved by: Kolbjørn Høyland

Summary

The cable-stayed part of the Bjørnafjorden bridge is well within span-lengths of conventional cable-stayed bridges. Untraditional features as in-length inclination of the tower, and grouping of cables makes the design more challenging, but has advantages in a smoother stiffness transition to the floating bridge and esthetics.

This report aims to reduce the technological and economical risk regarding the stay-cable bridge part, and the conclusion is that the solution is robust and flexible for further changes in design. Detailed design of well-known components is kept to the minimum without introducing higher risk to the project.

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1 INTRODUCTION

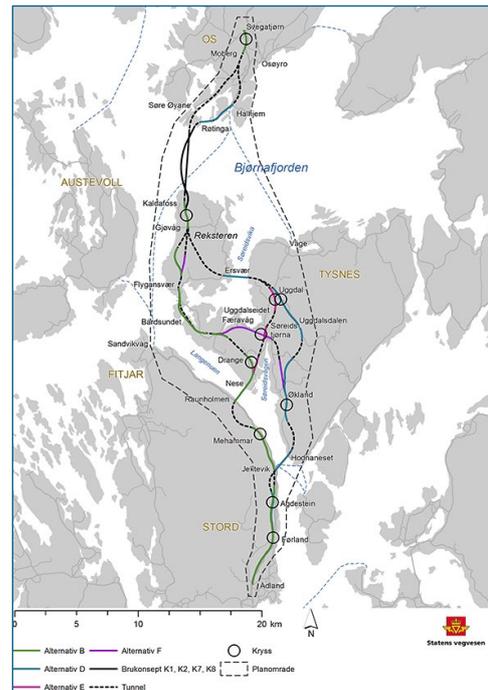
1.1 Current report

This report describes the preliminary analysis of the stay-cable part of the bridge.

1.2 Project context

Statens vegvesen (SVV) has been commissioned by the Norwegian Ministry of Transport and Communications to develop plans for a ferry free coastal highway E39 between Kristiansand and Trondheim. The 1100 km long coastal corridor comprise today 8 ferry connections, most of them wide and deep fjord crossings that will require massive investments and longer spanning structures than previously installed in Norway. Based on the choice of concept evaluation (KVU) E39 Akسدal Bergen, the Ministry of Transport and Communications has decided that E39 shall cross Bjørnafjorden between Reksteren and Os.

SVV is finalizing the work on a governmental regional plan with consequence assessment for E39 Stord-Os. This plan recommends a route from Stord to Os, including crossing solution for Bjørnafjorden, and shall be approved by the ministry of Local Government and Modernisation. In this fifth phase of the concept development, only floating bridge alternatives remain under consideration.



1.3 Project team

Norconsult AS and Dr.techn.Olav Olsen AS have a joint work collaboration for execution of this project. Norconsult is the largest multidiscipline consultant in Norway, and is a leading player within engineering for transportation and communication. Dr.techn.Olav Olsen is an independent structural engineering and marine technology consultant firm, who has a specialty in design of large floating structures. The team has been strengthened with selected subcontractors who are all highly qualified within their respective areas of expertise:

- Prodtex AS is a consultancy company specializing in the development of modern production and design processes. Prodtex sits on a highly qualified staff who have experience from design and operation of automated factories, where robots are used to handle materials and to carry out welding processes.
- Pure Logic AS is a consultancy firm specializing in cost- and uncertainty analyses for prediction of design effects to optimize large-scale constructs, ensuring optimal feedback for a multidisciplinary project team.
- Institute for Energy Technology (IFE) is an independent nonprofit foundation with 600 employees dedicated to research on energy technologies. IFE has been working on high-performance computing software based on the Finite-Element-Method for the industry, wind, wind loads and aero-elasticity for more than 40 years.
- Buksér og Berging AS (BB) provides turn-key solutions, quality vessels and maritime personnel for the marine operations market. BB is currently operating 30 vessels for harbour assistance, project work and offshore support from headquarter at Lysaker, Norway.

- Miko Marine AS is a Norwegian registered company, established in 1996. The company specializes in products and services for oil pollution prevention and in-water repair of ship and floating rigs, and is further offering marine operation services for transport, handling and installation of heavy construction elements in the marine environment.
- Heyerdahl Arkitekter AS has in the last 20 years been providing architect services to major national infrastructural projects, both for roads and rails. The company shares has been sold to Norconsult, and the companies will be merged by 2020.
- Haug og Blom-Bakke AS is a structural engineering consultancy firm, who has extensive experience in bridge design.
- FORCE Technology AS is engineering company supplying assistance within many fields, and has in this project phase provided services within corrosion protection by use of coating technology and inspection/maintenance/monitoring.
- Swerim is a newly founded Metals and Mining research institute. It originates from Swerea-KIMAB and Swerea-MEFOS and the metals research institute IM founded in 1921. Core competences are within Manufacturing of and with metals, including application technologies for infrastructure, vehicles / transport, and the manufacturing industry.

In order to strengthen our expertise further on risk and uncertainties management in execution of large construction projects Kåre Dybwad has been seconded to the team as a consultant.

1.4 Project scope

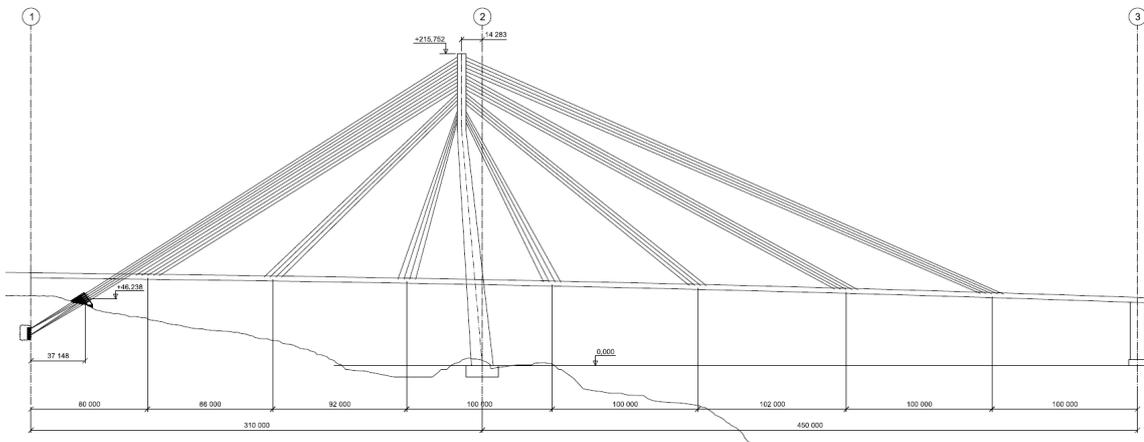
The objective of the current project phase is to develop 4 nominated floating bridge concepts, document all 4 concepts sufficiently for ranking, and recommend the best suited alternative. The characteristics of the 4 concepts are as follows:

- K11: End-anchored floating bridge. In previous phase named K7.
- K12: End-anchored floating bridge with mooring system for increase robustness and redundancy.
- K13: Straight side-anchored bridge with expansion joint. In previous phase named K8.
- K14: Side-anchored bridge without expansion joint.

In order to make the correct recommendation all available documentation from previous phases have been thoroughly examined. Design and construction premises as well as selection criteria have been carefully considered and discussed with the Client. This form basis for the documentation of work performed and the conclusions presented. Key tasks are:

- Global analyses including sensitivity studies and validation of results
- Prediction of aerodynamic loads
- Prediction of hydrodynamic loads
- Ship impact analyses, investigation of local and global effects
- Fatigue analyses
- Design of structural elements
- Marine geotechnical evaluations
- Steel fabrication
- Bridge assembly and installation
- Architectural design
- Risk assessment

2 OVERVIEW OF THE DESIGN

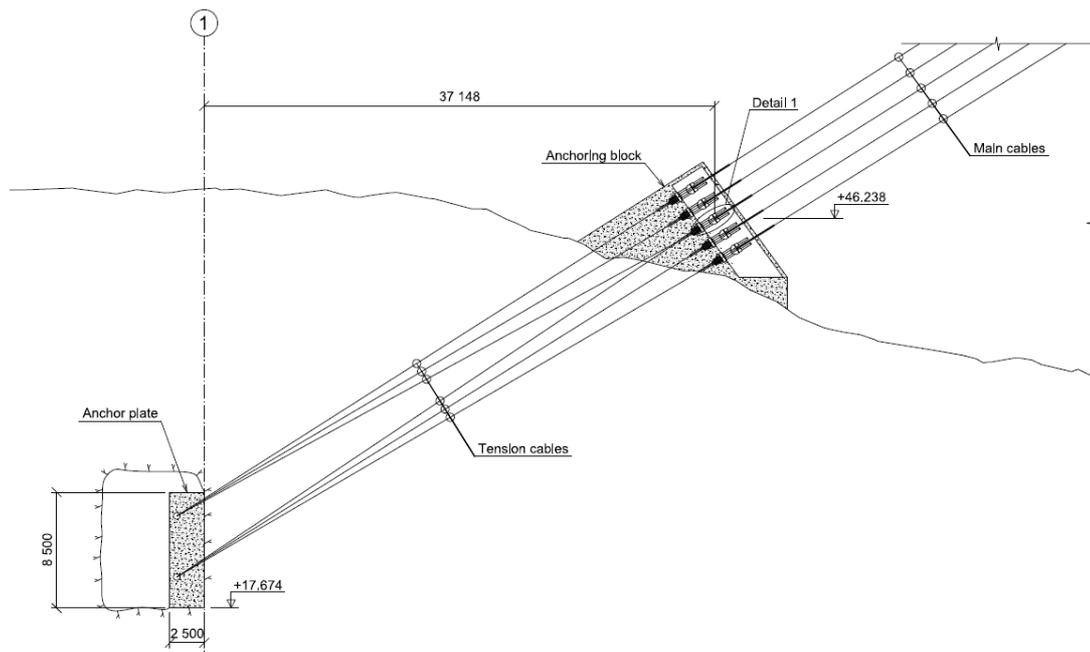


> Figure 1 Stay cable bridge - Elevation

2.1 Main components

The cable stayed bridge part of the Bjørnafjorden bridge is a 450m span one tower cable stayed bridge. Figure 1 show an overview of the bridge, for more in detail drawing see drawings -151 to -164. The bridge has the following parts:

Rock anchoring – standard rock anchoring with post tensioned cables from cable sockets to anchoring chamber at the other end. This is a traditional way of anchoring cables for both suspension and cable stayed bridges.



> Figure 2 Rock anchoring

Abutment – The abutment is a massive prestressed concrete construction. The abutment is not a part of this report, see report [1].

Cable system – The cable system with sockets and connection to tower and girders can be seen on drawing -153 to -158. The cable system with locked coil cables, dampers, sockets and connections are well known and within reasonable dimensions that have been built before. The new thing about the cable system is the distance between the cable groups. This is done to utilize the weak axis bending moment capacity of the stay-cable bridge.

Bridge girder – The bridge girder is an orthotropic sharp edge box girder section. Capacity of the girder is calculated in report [2], but the forces are presented here with a simplified stress calculation. The girder is traditional, the distance between the stay-cables is not. The girder can be seen on drawing -142.

Bridge tower – The bridge tower is made of concrete with two legs in a Y-shape. On top of the inclination in the transversal direction of the bridge, the tower has an inclination in the length direction of the bridge. This inclination is structurally unnecessary and is there from an aesthetic point of view. This increases the cost of the tower but compared to the total bridge, the investment in aesthetic is concluded to be worth it. The inclination of the tower is counter measured structurally with post-tensioning of the tower legs. This gives good control of the stiffness and displacement of the tower. The tower can be seen on drawing -152.

Temporary connection between tower and girder – The temporary connection between tower and girder is established to stiffen the girder horizontally in the construction phase. The connection is made from steel beams with a bearing. The girder is only fixed in its transversal direction at the tower. The temporary connection can be found on drawing -162.

Connection to the floating bridge – The connection to the floating bridge is a complex structure with cables, guiding and sockets. The connection is designed in report [3]. Calculation of the ballasting to get the adjustments of the stay-cable bridge and the floating bridge together is done in this report. The ballasting is done through tanks, asphalt and a length adjustment of one of the cable groups. The temporary connection can be found on drawing -159 and the ballasting procedure can be found on drawing -164.

Note that most of the construction details and elements are within traditional bridge design. In this phase of the project, the goal is to choose between 4 different concepts and show that the concepts are feasible. Detailed design of well-known components is kept to the minimum without introducing higher risk to the project.

2.2 Installation procedure

The whole installation procedure can be found in report [3], but the main operations is stated here to show and identify the critical design stages of the stay-cable bridge part. Analysis of construction from the first tower foundation to the connection with the full bridge is calculated here. Construction phases for the stay-cable bridge can be found in drawings -022 and -161.

Stage 1: Free standing Stay-cable tower. This stage is shown on drawing -161. Since the bridge tower is post-tensioned, more construction stages are necessary.

- 1a. Foundation of the tower with rock anchoring is casted.
- 1b. Tower leg 1 and 2 is casted up to about 55m, where the first temporary crossbeam is connected. In this phase the first post-tensioning of cables take place.
- 1.c The tower is casted to about 110m, where the secondary crossbeam is connected. Post-tensioning of cables.
- 1.d The tower is casted about 8m above the connection of the tower legs. The last stretch of the cables is post-tensioned, and the temporary beams are removed.
- 1.e The top of the tower with the internal steel frame for stay-cable sockets are casted.

Stage 1a-d is called free tower in construction phase.

Stage 1e is called freestanding tower.

Length direction of the tower is referred to as in length of the girder. Transverse is referred to transverse direction of the girder. North side is the main span side, south side is the side span side.

Stage 2: Girder and stay cable. This stage is shown on drawing -164. The distance between the cable groups gives an untraditional solution compared with regular stay-cable bridges, but this is well within other types of bridge constructions.

- 2.a Establish temporary scaffolding in the side span
- 2.b Installing and welding of the bridge girder in side-span
- 2.c. Successive installing of cable groups and bridge elements in main span using temporary lifting structures, cranes and barges
- 2.d Installing the temporary connection to the tower before the cantilever reaches 200 m.
- 2.e Complete the installation of girder and cable elements.

Stage 3: Floating bridge and stay cable connection.

- 3.a Ballasting of floating and cable-stayed bridge with asphalt and water tanks
- 3.b Mounting of technical gear for the connection
- 3.c Course adjustments of the stay-cable and floating bridge, some through pullies on the cable stayed girder
- 3.d Connection of the two parts with a connection that can withstand 10-year summer storm
- 3.e removing the temporary connection to the tower.
- 3.f Welding of the girder

Stage 2 to stage 3c is referred to as the free-standing cable stayed bridge.

2.2.1 Critical construction phases for design

The following stages of the installation stages is calculated:

- 1.d – Freestanding tower in construction phase before the post-tensioning of cables, in 10-year wind, in longitudinal and transversal direction
- 1.e – Freestanding tower, in wind with 10-year return period
- 2.e – Freestanding cable-stayed bridge, in wind with 50-year return period
- 3.c – Freestanding cable-stayed bridge with ballasting and maximum forces from course adjustment of the elements. No wind here due to the short installation period with good weather. Full ballasting with 10-year summer storm also calculated.

In later stages of the design, a more complete calculation of the different stages is deemed necessary, but these stages should be less critical for the bridge than these stages. Stage 3.d and onwards behaves the same as the complete bridge and is outside the scope of the report.

3 STRUCTURAL ANALYSIS

Short overview of the program used:

3DFloat – Time-domain analysis of environmental condition, used in ULS/SLS

Sofistik – Static loads and load-combination in ULS/SLS

PatranPre – Modelling of tower geometry and loading

TenLoad – Application of tendon loads on FEM model

ShellDesign – Local analysis and code design check of tower. Used in all analysis of the tower

Abaqus – Used in ALS ship collision, together with DynNO for fatigue design, and construction phase analysis

DynNo – Fatigue analysis and dynamic analysis of freestanding tower and freestanding cable stayed bridge

3.1 ULS/SLS

The operational phase ULS/SLS is analyzed with 3Dfloat for the dynamic loads and Sofistik for the static loads. Load-combination is done in Sofistik. Full description of these models can be found in rapport [4]. The model is used for design-force extraction of cables, rock anchoring, sockets and girder.

The tower model is a volume-element model from Patran with post-tensioning forces from TenLoad. The design calculations are done in ShellDesign and accounts for the nonlinear behavior of reinforced concrete due to cracking etc. when establishing the response of the cross-section. ShellDesign has also the capability to include the non-linear material behavior of reinforced concrete into the structural FE analysis (nonlinear FE-analysis). The method is based on an iterative analysis/design process. The linear elastic analysis with nonlinear design calculation is used here, and nonlinear analysis is only partly used to show robustness. Load from self-weight, post-tensioning and wind on the tower are combined with the cable-loads from Sofistik and 3D-float in SLS/ULS.

To be able to attain the correct operational phase geometry of the bridge a form finding routine is applied on the bridge giving the correct initial lengths of the cables. The form finding is explained in [4].

A selection of forces from ULS/SLS can be found in appendix D. Rest of the forces can be found in [4] and [5].

The Result from the ShellDesign analysis with utilization levels in the tower for concrete and reinforcement from ULS/SLS in temporary and operational phase can be found in appendix F.

3.2 ALS

3.2.1 Ship impact

The ALS-design is conducted in Abaqus. The full ALS model is reported here [5]. Note that the ship collision only hits the girder and the pontoons, not the tower. The tower distance from the shore is so large that the ship will stop on the rocks before hitting the tower.

Ship impact has been combined with $\psi_2=0,5$ for traffic load according to Design Basis table 9. The ALS forces for the cables is shown in appendix D. These forces including load factors are considerably smaller than the forces from ULS design.

A special check from ALS-design is that the girder should not get major dents from hitting the bridge tower. The critical distance between the top point of the girder and the tower leg is 3.8 m, and the maximum horizontal deflection is 3.85m in collision 711_16. Se report [6]. This is deemed sufficient.

3.2.2 Sudden loss of stay cable

Sudden loss of stay cables is not calculated in this phase of the project. In traditional cable-bridges (due to a high safety factor) this analysis is not design driver for the cables but could be design driver for girder/tower attachments, but usually not more than maximum 5-10% of added force. These connections are designed partly by practical steel size and is not utilized 100%. This means that a sudden loss of cables is probably not a design driver, and at least not adding any cost risk to the project.

The girder will have minimal additional load from the sudden loss of a cable. The grouping of the cables will make the force only redistribute to the other cables that are nearby and the girder will be almost unaffected. The girder will usually not get critical even for suspension bridges with 20m center distance between hangers.

3.3 FLS

The fatigue analysis is done with the same procedure as in the report [7], with stochastic dynamic loads from DynNo and traffic-loads from Sofistik. Cholesky-decomposition of the frequency domain data combined with traffic loads gives time-series that are rainflow-counted to get the correct stress-cycles.

The fatigue analysis can be found in appendix E. Summary of findings:

Minimum fatigue life of cable is approximately 1000 years.

Minimum fatigue life of cable connection to tower and girder is approximately 800years.

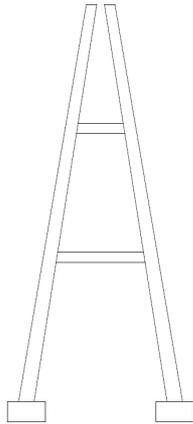
Note that the fatigue analysis of the cables is according to Eurocode, and the rest according to DNV-GL. Furthermore, it is assumed that one stay cable can fail without the collapse of the bridge. This is a design case in ALS.

3.4 Construction phase

The following design-situation is checked for the stay-cable bridge:

1. Freestanding tower in construction

Wind loading on in transverse and longitudinal direction, with added wind area for formwork, temporary beam between the tower legs, before the post-tensioning of stage 3 is finished. See figure below:



> *Figure 3 Tower in critical construction phase*

2. Freestanding tower

Wind loading of the full free-standing tower in longitudinal and transversal direction. Added wind area from formwork.

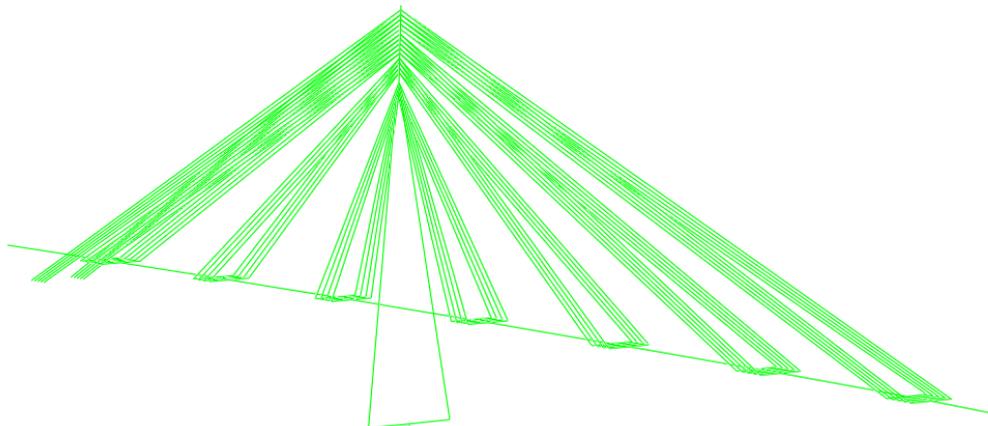
3. Freestanding cable-stayed bridge

Wind loading in transverse direction, without asphalt weight. Including formwork area at the end of the cantilever.

4. Freestanding cable-stayed bridge with ballasting

Maximum forces from course adjustment of the elements. Separate model with ballast and 10-year summer storm.

3.4.1 Modell description



> *Figure 4 Abaqus model of construction phases*

The full construction phase model is a part of the operational phase Abaqus model. This model is modelled with output from the GreenBox system and will in general look the same as the Abaqus model used as a basis for DynNo analysis in Abaqus. Some adjustments have been made, that mostly concerning the number of elements in the tower. For the full description of the operational phase model see rapport [4].

General description of the model:

- The full model is produced with Greenbox input in Abaqus
- Iteration procedure on the full model to tune cable lengths and buoyancy
- The bridge is split into two independent parts
- The weight of asphalt is removed from the cable-stayed part of the model, both in respect of vertical load and mass moment of inertia.

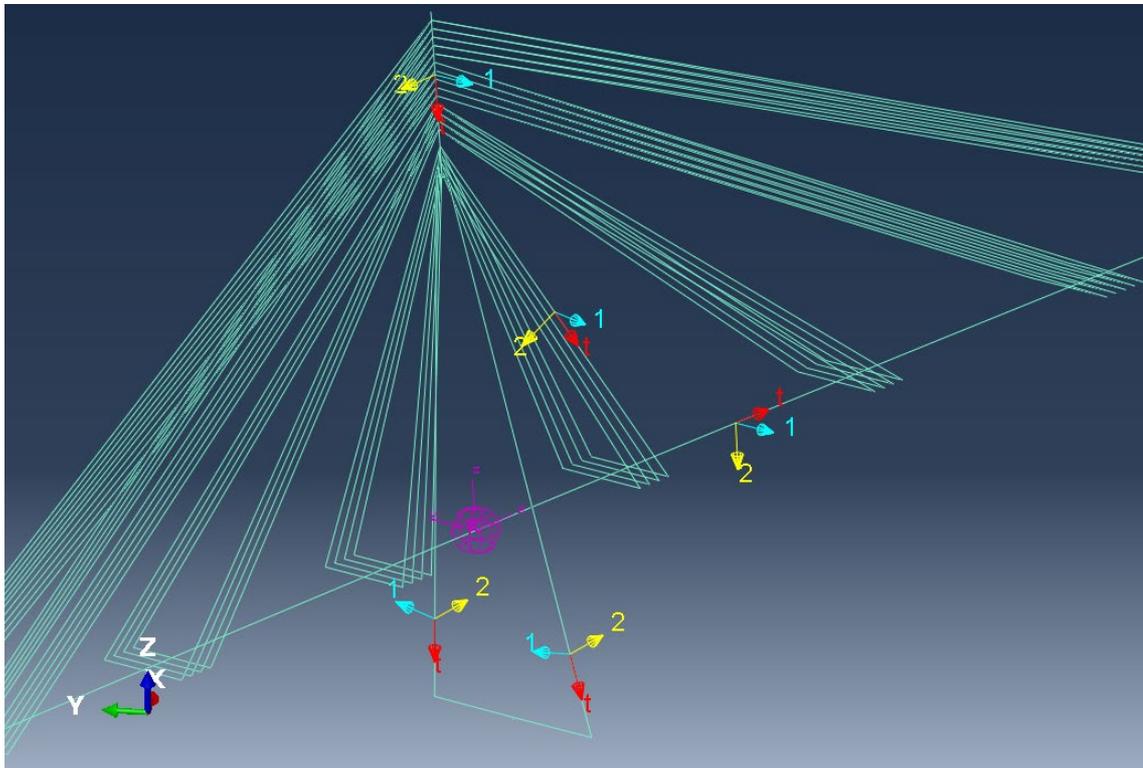
After this model is made, all of the remaining construction phase models can be made:

1. Free Standing cable-stayed bridge; is modelled with a stiff spring at the girder node exactly where the tower is, in transversal direction
2. Free Standing tower; the cables and girder are removed from the model
3. Free Standing tower in construction phase; removed the tower top of the tower and reduces the stiffness of the upper parts of the model. Adds transversal temporary beams.

Sign convention

Beam force convention.

SM1	Moment about axis 1
SM2	Moment about axis 2
SM3	Moment about longitudinal axis(torsion)
SF1	Axial force
SF2	Shear force in direction 2
SF3	Shear force in direction 1



> Figure 5 Local beam coordinates for beam elements for the stay-cable bridge

Elements

Cables: Element types B31 that are 2 nodes linear elements. One element over the length of the cables. The axial stiffness of the cables has been adjusted according to NS-EN1993-1-11 5.4.2. The mass of the elements is lumped at the nodes.

Girder: Element type B31, 2 nodes linear elements. 6 elements between cable groups, and one element between every cable pair. The vertical mass and rotational moment of inertia is modelled with 2 lumped masses over and under the element with the correct center distance to create the correct mass and mass moment of inertia.

Girder to cable connection: Element type B31, massless with high stiffness.

Tower elements: Element type B31. Elements between all cable connection for the top. Rigid elements to the tower legs. Two bottom tower legs are connected to rigid link and boundary condition. For analysis of the stiffness of the tower, see dedicated section 3.6 in this report. 12 elements over the leg height.

Temporary beam element: B33 element with equivalent concrete cross-section of 2x1m.

Element properties

The element properties for most of the elements is given below:

	Density [kg/m ³]	Area, A[m ²]	Moment of inertia, I ₁₁ [m ⁴]	Moment of inertia, I ₂₂ [m ⁴]	Torsional constant, J[m ³]	Youngs- modulus, E[GPa]	Torsional shear modulus, G[GPa]	Coefficient of thermal expansion
Top tower	2650	21.6	92.5	88.8	181.3	36	15	0.000012
Top tower leg	2650	17.7	94	37	146	36	15	0.000012
Bottom tower leg	2650	43.8	1220	202	823	36	15	0.000012
Rigid elements	0	11.6	25.5	25.5	51	64200	24700	0
Girder element [section BCS1]	N/A	1.47	2.714	114.928	6.553	210	80.7	0.000012
Cable elements	VAR	VAR	0.00001	0.00001	0.000001	VAR	0.001	0.000012

The mass moment of inertia of the girder section is modelled with 2 masses with a distance from the girder, this is to get the correct vertical, horizontal and rotational moment of inertia. Se report [4] for information.

	Mass 1			Mass 2		
	Mass per unit length	Local 1-coordinate of center of mass	Local 2-coordinate of center of mass	Mass per unit length	Local 1-coordinate of center of mass	Local 2-coordinate of center of mass
Girder element, without asphalt	3745	0	15.23	11386	0	5
Girder element, without asphalt	4366	0	15.44	13486	0	5

There is a lot of cables with different cross-section area and stiffness. It is chosen to give the information of the maximum and the minimum of the cables:

	Area, A[m ²]	Youngs-modulus, E [GPa]
Min of cable	0.003815	147
Max of cable	0.0133811	158

Note: The area of the cables is iterated to give stress of 520MPa from permanent loads. This limit is a good starting point for most cable-stayed bridges. This is a bit too high stress limit for most of the cables. The actual size has been increased without a new analysis. Different iterating goal is going to be set in the next phase. The errors introduced are minimal for the design.

Boundary conditions

There are two boundary conditions, tower bottom and end of girder. Both are set to fully fixed in all directions.

Types of analysis

All the analysis performed in Abaqus is geometrical nonlinear to get the updated geometrical stiffness of the model. The eigenvalue extraction for stochastic analysis in frequency domain are linearized steps around mean wind position, and in general mean position of permanent- and static wind load.

Form finding procedure

The geometry of the analysis model should be according to the design with self-weight applied to the model. To find this form a form finding routine is applied. The routine runs small increments of the self-weight countering the displacement with temperature at the elements. Tower, cable and girder elements are all iterated on. The form finding procedure works iterates the geometry within 2mm error for 30 iteration steps. For more explanation on the form finding procedure see rapport [4].

3.4.2 Wind loading in temporary phases

Wind loads are calculated from this table:

	Direction	Return period [y]	Seasonal factor	Terrain category	Directional factor	Basic wind velocity, 1h, 10m [m/s]	Turbulence intensity 10m	Turbulence intensity curves
Free standing tower, construction	North	10	1	1	0.7	18.06	14.5 %	EC
Free standing tower, construction	South	10	1	2	0.85	18.74	30 %	DB
Free standing tower, construction	East/West	10	1	1	1	25.8	14.50 %	EC
Free standing tower	North	10	1	1	0.7	18.06	14.5 %	EC
Free standing tower	South	10	1	2	0.85	18.74	30 %	DB
Free standing tower	East/West	10	1	1	1	25.8	14.50 %	EC
Free standing cable-stayed bridge, wind	East/West	50	1	1	1	28.5	14.50 %	EC
Free standing cable-stayed bridge, ballast	East/West	10	0.8	1	1	20.64	14.50 %	EC

Return period of wind loading

According to NS-EN1991-1-6 and N400 section 5.4.1 structures with less than 1-year construction period can use 10-year return period on environmental loads. This is used for all the tower-phases. For the free-standing cable stayed bridge it is chosen to design for a longer period. The time schedule for the installation procedure [3] dictates that this situation will be less than 5 months, but some robustness for later changes in the project is applied. The free-standing cantilever with ballasting is designed for a summer period so 10-year summer storm is applied.

Cables

Because of element type of the cables, the wind load on the cables is lumped at the tower and the girder. Static coefficient for cables is calculated according to EC1991-1-4 section 7.9.2 to be around $C_f=1,2$ and this value is used for all cables independent on windspeed.

Girder

The static coefficients on the girder is set to the chosen values from the Aerodynamics report [8]. To account for the added area for equipment in the building phase the static coefficients for cross-section with traffic box is used for the outmost 50m of the bridge.

Tower

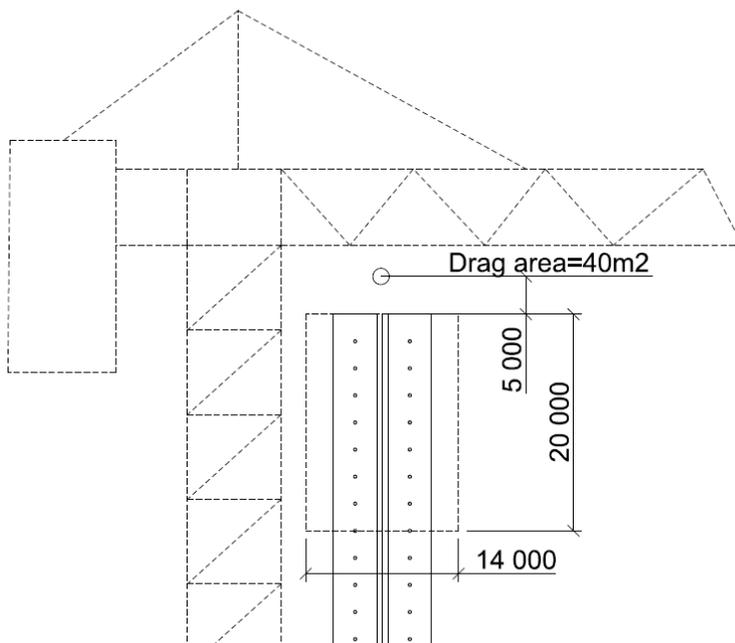
The static coefficients of the tower will in general vary along the height of the tower. Wind in transversal direction will also give a complex static coefficient value due to shielding effects of the legs. To simplify the approach static coefficients from EC1991-1-4 section 7.6 is used. The rounded edges of the tower decrease the drag according to figure 7.24(EC1-1-4). With all of this, the static coefficient ends up being close to the value of 2.0.

The additional area for formwork:

Crane = 40m^2 , with 6m eccentricity above top of tower

Formwork area= 14m wide, 20m high,

Assumed static coefficient=2.0.



3.4.3 Self-weight in temporary phases

The self-weight in the Abaqus model is calculated by the program. Note that the load on the girder is reduced by removing the asphalt. Mass properties of the girder without asphalt is as follows:

	Mass/m girder, m, [kg/m]	Mass moment of inertia, I_M , [kgm ² /m]
Girder without asphalt	15132	1152035

3.4.4 Ballast loads

The ballast loads are according to appendix A and report [3]. The ballasting is composing of:

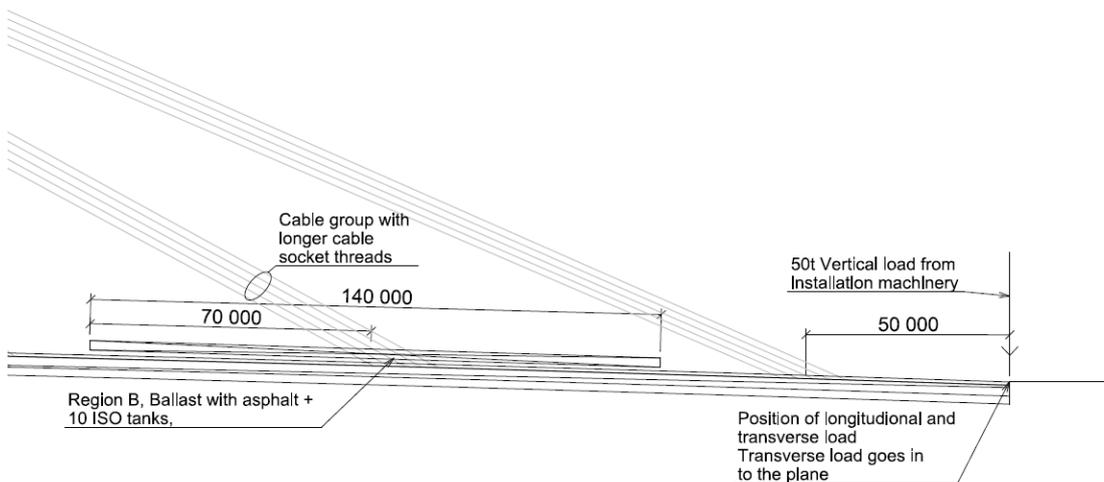
- 35cm longer cables in the second longest group (modelled with temperature)
- 10 000kN of weight evenly distributed centered around the second longest group over 70 meters
- 50t point load at the end of the free cantilever from installation machinery

3.4.5 Forces from course adjustments of floating bridge installation

The forces from the installation of the floating bridge is set to:

$F_{\text{Transverse}}=150\text{t}$, load in transversal direction applied at the end of the cantilever.

$F_{\text{Longitudinal}}=1500\text{t}$, load in the longitudinal direction, applied at the end of the cantilever.



3.4.6 Load combination

Load combination	Load factor set 1	Load factor set 2
Permanent loads	1.35/1.0	1.2/1.0
Wind load	1.12	1.6
Technical equipment	1.05	1.2
Semi-permanent ballast	1.05	1.2
Installation force	1.05	1.5

According to N400 section 13.2.1 the load-factor for cable-bridges over 500m for ULS 6.10a EQU for self-weight may be assumed to have the total value of: $1,15 \cdot 1,05 = 1,2075$. This is because the weight in these bridges is calculated in detail, and the self-weight is often the dominant force. Load factor set 1 will not be used for the stay cable bridge.

3.4.7 Dynamic calculations

Dynamic wind calculation is found using DynNO. It is a program using eigenfrequencies and modes from Abaqus, and other inputs of the model, making use of the frequency-domain to calculate the stochastic dynamic wind-response. The wind load model can both be linear-quasi steady theory and aerodynamic derivatives. For more information about the program see report [4].

In this section all phases have been calculated in DynNO with the mean displaced position as linearization point.

3.4.8 Response

Full set of forces and eigenfrequencies for the different models can be found in appendix D.

3.5 Reduced stiffness due to cable sag

According to N400 section 13.2.2 can 1. order models be used for stay-cable bridges if the effects of reduced cable stiffness due to sag is accounted for. According to NS-EN1993-1-11 section 5.4.2 this can be considered accordingly:

$$E_t = \frac{E}{1 + \frac{w^2 l^2 E}{12 \sigma^3}}$$

E : *E-modulus of cable*

w : *Cable weight*

l : *Horizontal span*

σ : *Cable stress*

This is used for all cables with the cable stress calculated from permanent loads.

3.6 Tower stiffness

The actual stiffness of the tower has contributions from different sources:

1. Modulus of elasticity of concrete
2. Reinforcement amount
3. Degree of cracking of the section
4. Creep and shrinkage
5. Geometric stiffness

The inclination of the tower in longitudinal direction of the bridge makes the entire situation complicated. This inclination is not normal for stay cable bridges of this size. The transverse inclination of the tower legs is normal.

To have more control over the stiffness and displacement of the tower in all phases the towers are post-tensioned. This increases the effective stiffness of the tower forcing it to stay inside stadium I for an increased loading period. On top of that the displacement from the post-tensioning reduces the displacement from self-weight. Giving smaller 2nd order effects.

On the other hand, the calculation procedure, with construction phases with and without post-tensioning, long term effects on both concrete and the reduction on post-tensioning gets more complicated.

The effects of the tower stiffness on the structure is different in the different phases of the bridge. A stiff tower in construction phase gives higher eigenfrequencies giving lower dynamic wind load, smaller displacements, and less 2nd order effects from geometric stiffness.

A stiffer tower in operational phase gives higher eigenfrequencies, but the dynamics of the full bridge is less sensitive for the tower stiffness. Displacement from the deformed freestanding tower needs to be straightened with the stay cables to get final geometry, giving different forces in the cables. The effects of creep in the full bridge is mostly countered with changes in cable forces. In general, the conservative for the bridge is to use the highest reduced stiffness plausible.

The concrete in the tower is B70. The Modulus of elasticity of the concrete from EC2 is 41GPa. In traditional Norwegian bridge design, the stiffness for B45 is reduced from 36GPa to 32GPa due to the softness of the concrete aggregate. This is conservatively done here also from 41 to 36GPa.

3.6.1 Free standing tower stiffness

It has been done a study to see the effects of the geometrical stiffness of the free-standing tower in the longitudinal direction. Geometrical stiffness effects have been checked in Abaqus beam model, and the cracked stiffness of the concrete with increased stiffness of the post-tensioning and reinforcement is done in volume elements in ShellDesign.

Cracked stiffness analysis:

Load case,	Linear analysis, top tower displacement[m]	NL, fct=1,5MPa, top tower displacement[m]	NL, fct=0,5MPa, top tower displacement[m]	NL, fct=0,1MPa, top tower displacement[m]
Self-Weight	-0,403	-0,36	-0,483	-0,643
Post-tensioning	0,406	0,359	0,437	0,625
Diff	-0,07	-0,06	-0,06	-0,018

Note that the B70 has $f_{ctk0.05}=3,2\text{MPa}$.

The effects of geometrical stiffness increase with the mean displacement from the vertical position.

Load case	Without geometrical stiffness top tower displacement[m]	With geometrical stiffness, top tower displacement[m]
Self-Weight	0,3841	0,3942
Self-Weight + Mean wind load	0,4676	0,4797
Mean wind load displacement	0,0835	0,0855
	Diff	97,6%

Note that this change will be less for the construction almost without initial displacement due to post-tensioning.

Conclusion: The effects from geometrical stiffness, cracking of the cross-section, modulus of the concrete, reinforcement amount, are all included in an elastic modulus of elasticity of 36GPa.

3.6.2 Stiffness of the tower in full phase

The stiffness of the tower when the cables are built has less influence on the structural response. No further analysis is necessary and a modulus of elasticity of 36GPa is used. Note that Abaqus, and 3D-float analysis includes the geometrical nonlinearity in the analysis.

3.6.3 Effects of long-term effects on the full bridge

The long-term effects on the post-tensioned tower will decrease the stiffness of the tower. This will in turn deflect the tower. The tower is restricted to move by the cables. The increase of cable force from the long-term effects is estimated to be about $\pm 0,5\%$. This effect is neglected in this phase but should be included in the later phase.

3.6.4 Eccentricity due to building errors

Unwanted eccentricity due to building errors should be considered for vertical towers. The eccentricity used for the Hardanger bridge tower, with comparable height, is 0,09m. This eccentricity compared with the offset of tower top of 14,25m is neglectable, but should be included in the next phase.

3.7 Wind stability

The flutter wind speed of the stay-cable bridge is calculated in report [9]. The flutter wind speed is approximately 112m/s and is well within the criteria. Other instability phenomena are included in the same report. The wind stability is sufficient. But the ViV wind speed of the girder in construction phase estimated are in the lower bounds of what is acceptable. This should be investigated further in the next phase.

4 BRIDGE TOWER

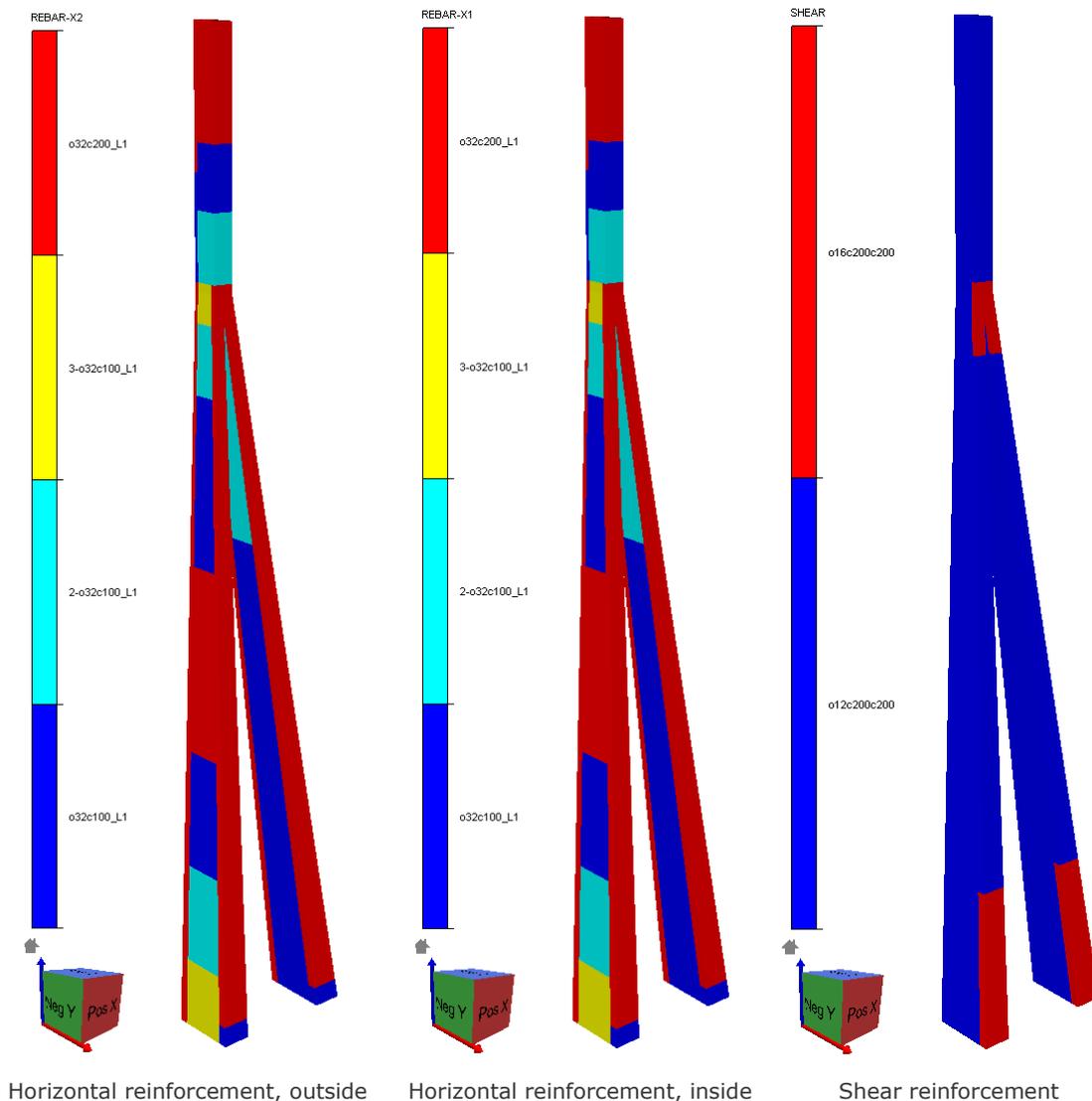
4.1 Tower design

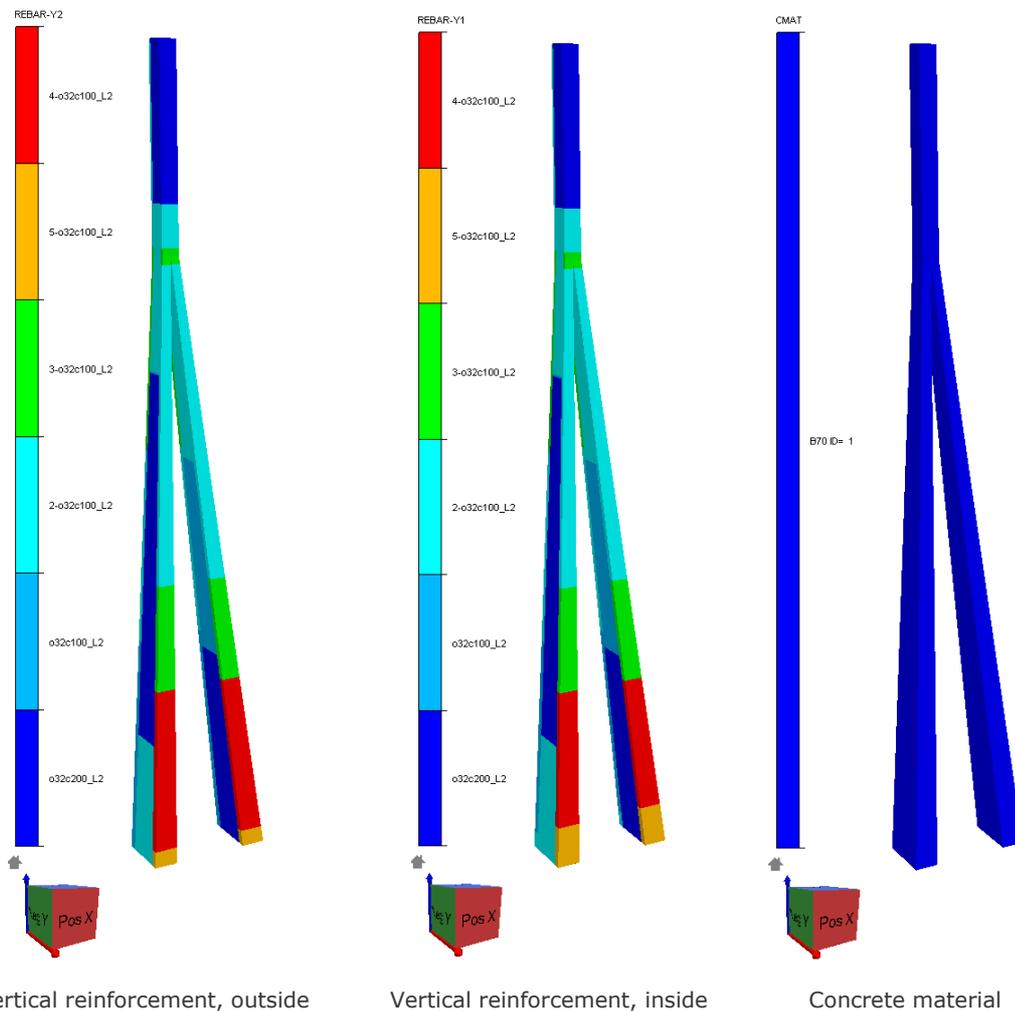
4.1.1 Minimum reinforcement

Minimum reinforcement in according to NS-EN 1992-1 is $1900\text{mm}^2/\text{m}$ ($\phi 25\text{c}250$) for walls with thickness 900mm and $4530\text{mm}^2/\text{m}$ ($\phi 32\text{c}175$) for walls with thickness 2m . The north and south walls are 2m thick at the bottom and 0.9m where the legs meet. All other wall sections are 0.9m thick.

4.1.2 Actual reinforcement

The actual reinforcement is shown below and are well beyond the minimum required. In a later phase it could be investigated whether some of the reinforcement should be replaced by more post-tensioning and concrete to reduce the highest intensities. It is assumed that the reinforcement can be grouped together and put in different layers.





4.1.3 Material quality and cover

The material quality of B70 is used. On the outside of the towers cross section $C_{min}=60+10=70\text{mm}$ because of spray from seawater and the use of slip forming. On the inside $C_{min}=35+10=45\text{mm}$. $\Delta C_{dev}= \pm 15\text{mm}$.

Surface	Cover
Outside of tower	85+/-15mm
Inside of tower	60+/-15mm

4.1.4 SLS-phase

The maximum crack width in temporary phases is 0.6mm in accordance with [11]. For the operational phase the limit is set to 0.2 for both the infrequent and quasi permanent loads situation. The crack width is not dimensioning for the tower and the utilization is low for the entire tower both in temporary and operational phases. In the quasi permanent situation there should not be tension where there is post-tensioning cables present.

4.2 Analytical model

The analytical model for the bridge tower is made in PatranPre with volumetric 20-noded elements. The bottom of each tower leg is fixed in all directions. Wind is applied as element pressure varying with height and direction according to [11]. Tendon forces is added to the model with TenLoad a program that evaluates the FEM-model geometry and adds nodal forces to elements that the tendons pass through from anchoring forces, losses and curvature.

Each height elevation in the GreenBox model is matched in the volume model. The cable forces are distributed across the entire cross section at that elevation and the direction of each cable and axial force is fetched from the GreenBox model and combined with self-weight of the tower, post-tensioning forces and wind on tower. The forces from GreenBox (cable forces) are combined with wind on tower from all 4 directions.

This model is then used as input to ShellDesign where code checking is done, and utilization level is reported for concrete and each group of reinforcement. Please refer to appendix F for further information and results.

4.3 Operation phase

4.3.1 Wind loads on ULS-design

The tower with cables attached is so stiff that wind directly on the tower gives very low dynamic amplification factor. The peak velocity pressure according to EC1991-1-4 equation 4.8 is 7 times the turbulence intensity. This is a single occurring event and combining peak velocity on the tower at the same time as the maximum load from the dynamic is overconservative. A peak scaling factor of 3,5 is used on wind on the tower in the Shell Design model.

4.4 Static equivalent wind loads

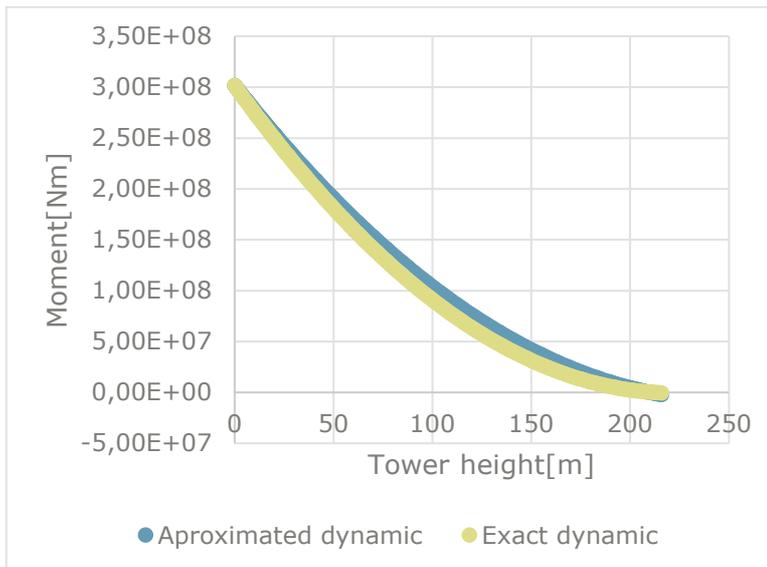
The ShellDesign model of the bridge tower is a static model. The workaround is to make static equivalent loads from Abaqus and DynNo. The procedure is as follows:

1. NL deflection from permanent loading and mean wind in Abaqus
2. Extraction of linearized modes around the displaced equilibrium
3. Stochastic frequency domain calculation in DynNO
4. Static equivalent forces from scaling mean wind loadings

Modal shapes, eigenfrequencies and forces can be found in appendix D.

The goal is to have the exact same force distribution along the tower, but this is not feasible. Since the main design driver for the tower is the in-wind moment, it is this section force that is matched by the static equivalent load.

Typical plots of Scaled static vs. dynamic moments along the tower looks can look like this:



The form finding procedure forces the moment at ground level to be the same for both dynamic and static winds. The error introduced is within acceptable bounds.

The table below shows the scaling of the mean wind load to get the total load from both stochastic and dynamic

	Direction	Approximated moment of tower leg [MNm]	Scaling factor mean wind
Free standing tower, construction	North	N/A	3.6*
Free standing tower, construction	South	N/A	2.5*
Free standing tower, construction	East/West	115	2.25
Free standing tower	North	550	2.23
Free standing tower	South	662	3.6
Free standing tower	East/West	225	2.5

*) the freestanding tower in construction has higher eigenfrequencies e.i less dynamic loading, and less area for stochastic loading in north/south direction. The longitudinal wind in the construction phase is not design driver.

4.5 Construction phase

4.5.1 Free-standing tower

This phase is dimensioning for the vertical reinforcement in the south wall. All utilizations are well within limits and can be seen in appendix F.

4.5.2 Free-standing stay-cable bridge

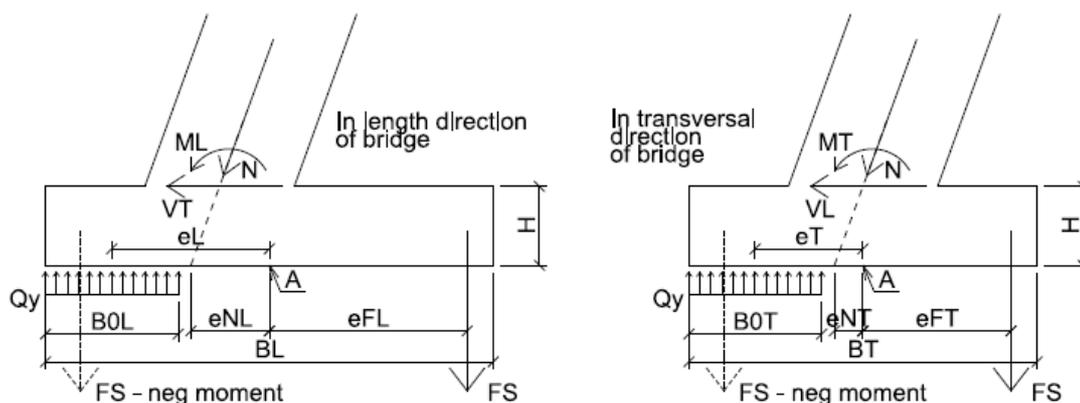
All utilizations are well within limits for this phase and can be seen in appendix F. Only the resulting beam forces from the dynamic analysis is matched in the bottom part of each tower leg. For this reason, only the lower part of the model displays the correct results. The utilization levels are low and for this reason the rest of the sections are not controlled in this phase.

4.6 Local analysis of connection to tower

The temporary bracing between the girder and the tower is resulting in a force of about 16MN on the tower. In this elevation local strengthening is necessary. The added strength could be in the form of a concrete slab or a steel bracing inside the tower. The concrete slab will distribute the force to the entire cross section and could be used as a natural platform for access to the bridge girder.

4.7 Tower foundation

The following figure is used to calculate the bridge foundations:



Explanation	Naming convention	Value	Unit
Width foundation, length direction	BL	22	m
Width foundation, transversal direction	BT	15	m
Height of foundation	H	8	m
Eccentricity of normal force	eNL	0.70	m
Eccentricity of normal force	eNT	1.28	m

Density of concrete	GammaBet	25	kN/m ³
Number of rock anchors	n	6	
Eccentricity of rock anchor, in length	eFL	9.5	m
Eccentricity of rock anchor, in transversal	eFT	6	m
Force single rock anchor	FKar	5022	kN
Reduction coefficient for anchor	GammaStag	0.65	
Reduced force from rock anchor	FDim	3264.3	kN

Forces from the free-standing tower in construction phase is used as an example:

Load-Combination	N[kN]	VL[kN]	VT[kN]	ML[kNm]	MT[kNm]	YG
Free tower, wind from north	123443	-11192	-13612	-1249961	308081	0.9

Without rock anchors

	Without anchors	
Moment around A, in lenth dir	$MLA=ML+VT*H+N*eNL$	-1272787 kNm
Moment around A, in trans dir	$MTA=MT+VL*H+N*eNT$	377115 kNm
Weight of foundation	$Nfund=BL*BT*H*yBet$	66000 kN
Normal force at A	$Na=N+Nfund*YG$	182843.2 kN
Eccentricity of the reaction force	$eL=MLA/NA$	7.0 m
Effective width, length dir	$BOL=BL-2*eL$	8.1 m
Eccentricity of the reaction force	$eT=MTA/NA$	2.1 m
Effective width, transversal dir	$BOT=BOT-2*eT$	10.9 m
Ground pressure	$qy=NA/(BOL*BOT)$	2081.4 kN/m ²

Rock anchors: Rock anchor type is 18 strands of 150mm² with f_{pk} of 1860MPa. V220 chapter 10.5.2.1 gives reduction factor of 0,65.

With rock anchors

	With anchors	
Force from rock anchors	$FS=n*FenkDim$	19585.8 kN
Normal force at A	$NA=NA+FS$	202429.0 kN
Moment around A, in lenth dir	$MLA=MLA-FS*eFL$	1086721.7 kNm
Moment around A, in trans dir	$MTA=MTA-FS*eFT$	259600.4 kNm
Eccentricity of the reaction force	$eL=MLA/NA$	5.4 m
Effective width, length dir	$BOL=BL-2*eL$	11.3 m
Eccentricity of the reaction force	$eT=MTA/NA$	1.3 m
Effective width, transversal dir	$BOT=BOT-2*eT$	12.4 m
Ground pressure	$qy=NA/(BOL*BOT)$	1445.3 kN/m ²

The goal of the analysis is to keep the Ground pressure(qy) below 10 000kN/m², this is according to the geology report.

The rest of the analysis can be found in appendix G.

The maximum ground pressure is found to be 3600kN/m². The foundation size could be reduced in the detailed design phase, when exact allowed ground pressure is clarified.

5 BRIDGE GIRDER

This chapter includes the girder design.

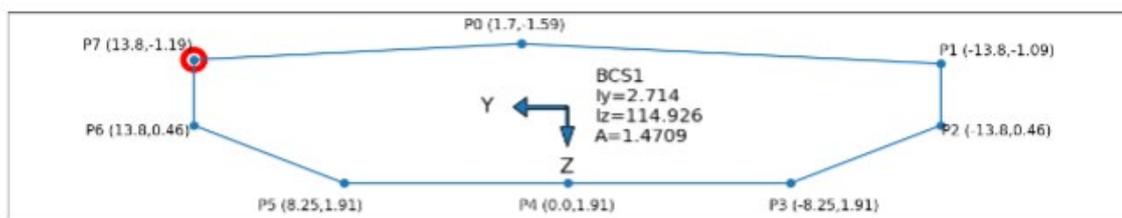
5.1 Design

The general design of the bridge girder is reported in [2]. Here, a simplified design-check of the girder in free-standing cable-stayed phase is conducted. The simplified approach consists of getting an allowable axial stress limit from the girder design report, and checking the critical point for this limit. Critical limit for stress-calculation for the girder is:

$$\sigma_{\text{crit,compression}} = 342\text{Mpa}$$

$$\sigma_{\text{crit,tension}} = 380\text{Mpa}$$

The critical stress-point for the girder when strong axis bending is dominant is stress-point 7. Se figure below:



> Figure 6 Section properties

To get the stress - only axial force, strong and weak axis bending moment is used, according to report [2].

Forces in free-cantilever beam is calculated statically by Abaqus and dynamically by DynNo. The critical design-phase is free standing stay-cable bridge with wind normal to the bridge axis. The wind is set to 50-year return period. A bridge with less building time than 1 year, could be calculated with 10-year return period according to NS-EN 1990-1-6. From the installation procedure report [3] we find that the free-standing cantilever stage will only stand there for around 4 months, this implies that we have some additional safety when using 50-year return period.

Stress-calculation is done elastically with simply:

$$\sigma = \frac{SM1 * y}{I_1} + \frac{SM2 * z}{I_2} + \frac{SF1}{A}$$

Where,

SM1 is bending moment about axis 1(strong axis)

SM2 is bending moment about axis 2(weak axis)

SF1 is axial force

A is the area of the cross-section

I1 is moment of inertia about axis 1

I2 is moment of inertia about axis 2

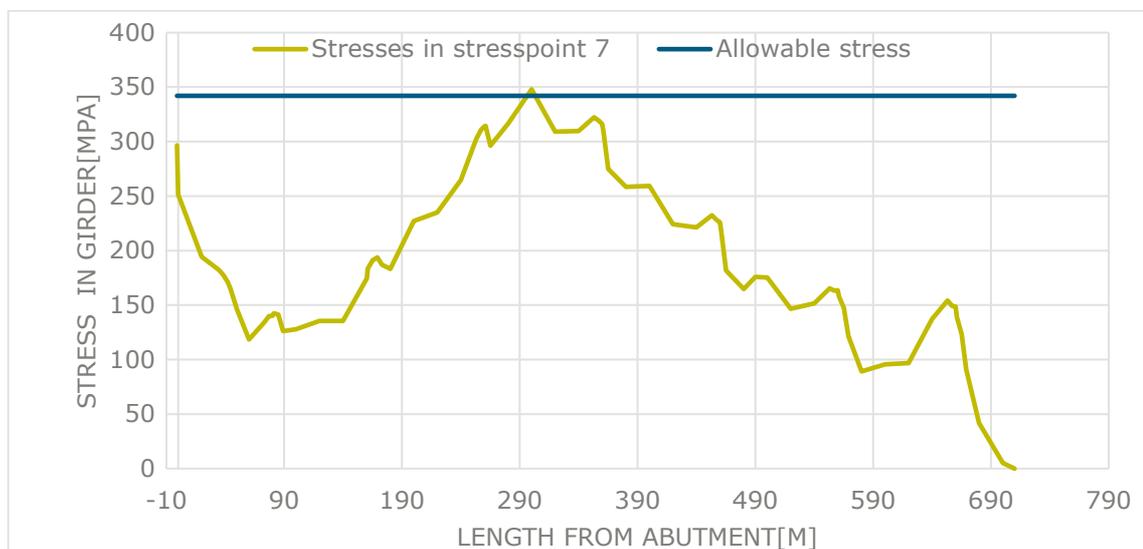
y is distance from neutral axis to stress-point along axis 1

z is distance from neutral axis to stress-point along axis 2

Since DynNo is calculating forces in frequency domain, you get maximum forces from one mode separately. The maximum forces are calculated through extreme-value statistics and this extreme will only happen once every period. It is highly unlikely that the extreme value for both SM1 and SM2 (main contribution from independent mode) happens at the same time. The dynamic stresses are therefore calculated with the maximum extremes with root sum of square method:

$$\sigma_{dynamic} = \sqrt{\sigma_{SM1}^2 + \sigma_{SM2}^2}$$

The figure below shows the stress at stress point 7 along the bridge axis for the maximum stress on the girder in the construction phases. The stresses are calculated with the section properties as given in Figure 6.



From the figure you can see that at the point where the girder is fixed to the tower, the stresses become greater than the allowable stress. This is the most critical point due to highest strong axis bending moment from wind, highest axial forces from the cables, highest weak axis bending moment due to the point being in the middle between cable pairs.

The maximum stress at the critical point is 347MPa. The following can be done with this:

1. Stresses for this exact load situation can be calculated more precise
2. Local strengthening of the cross-section of the 15m where the stresses is to large
3. Design with 10-year return period for the wind

The conclusion is that this small stress exceeds is easily prevented, and that the stress in the girder from construction phases is within the limits.

5.1.1 Shear lag

The grouping of cables increases the effects of shear lag on the girder. This effect is not included in this phase of the project and should be included in the later stages of design.

The effects of shear lag on the bridge girder in the free cable stayed bridge will have almost no or small effects due to the fact that it is the strong axis bending moment that are driving the stresses, and that the weak axis bending moment from wind also has a large distance between inflection points on the moment. The girder design report [2] concludes that the shear lag effect increases maximum the stress to about 352MPa.

6 BRIDGE CABLES

6.1 Cable design

There are several different cable types available which can be used for cable stayed bridges.

1. Locked coil cables. This is prefabricated spiral strands, with round wires in the core and normally Z-shaped wires in the 2 or 3 outer layers to give a smooth and almost watertight surface. This is the most common cable type in Norwegian suspension and cable stayed bridges. Each cable is supplied with a steel socket in each end. The cable-end is spread out like a brush and fixed inside the socket in a conus casted with zinc.
2. Parallel wire cable. The cable is built up with several round wires laid up in hexagonal form using very long helix length and put into a close-fitting polyethylene tube which is filled with grease. Same type of sockets as for locked coil. Sometimes the cable end is fitted to the socket with an epoxy-compound instead of zinc which improve the fatigue properties (Hi-Am-socket). This is the most common cable type for cable stayed bridges abroad. Former, steel tubes injected with mortar were used instead of polyethylene. Consequently, these cables could not be pre-fabricated because the injection as well the steel tube erection had to be done after installing the cable.
3. Parallel strand cable. Like the parallel wire cable, but instead of single wires, the cable is built up with single strands. Normally the strands are locked to the socket with wedges like common post-tensioned reinforcement.

The different types have different properties. The parallel wire/strand cable has a higher E-modulus than the locked coil (ca. 200 GPa compered to ca. 160 GPa). The spinning of the locked coil reduces the stiffness compered to pure steel. These cables are normally cheaper than the locked coil.

Experience shows that the locked coil has better vibration characteristics due to wind than the other.

The parallel strand is easier to tension because you can tension every single strand separately with a small jack instead of jacking the socket. On the other hand, it is very complicated to slacken the cable if that is needed during the construction phase. You must inject the cable tube with grease after the cable is installed.

For the Bjørnafjorden Bridge, we have chosen locked coil cables for this phase. This is of course not an irreversible choice for the project, but it is sensible to reduce the variables as much as possible. Regardless, the impact on the analyses from the cable type is almost insignificant.

Because every cable pair in a cable stayed bridge has different angels, the ULS-tension will be different. Consequently, an optimal design gives different cross section of all cable pairs which will be iterated in our analysis. However, in the detailing design phase, from economic reasons, one would prefer to reduce the number of different cables and therefor divide the cable dimensions into 4 or 5 groups. Hence, some cables will be oversized.

In the analysis it is important to input a correct stiffness of the cables. Because of the cable sag, the stiffness of a cable is lower than EA, (E-modulus multiplied with cross sectional area). EN 1993-1-11 has the following formula in paragraph 5.4.2 to take this effect into account:

$$E_t = \frac{E}{1 + \frac{w^2 l^2 E}{12 \sigma^3}}$$

E : *E-modulus of cable*
 w : *Cable weight*
 l : *Horizontal span*
 σ : *Cable stress*

This reduced stiffness is calculated in our analysis for all cables, using the cable stress from eigen weight. That is normal procedure.

For detailed dimensioning, see Appendix B.

6.2 Sockets

There are several available socket-types for cables. However, for heavy bridge-cables 4 types are relevant, see figures below:

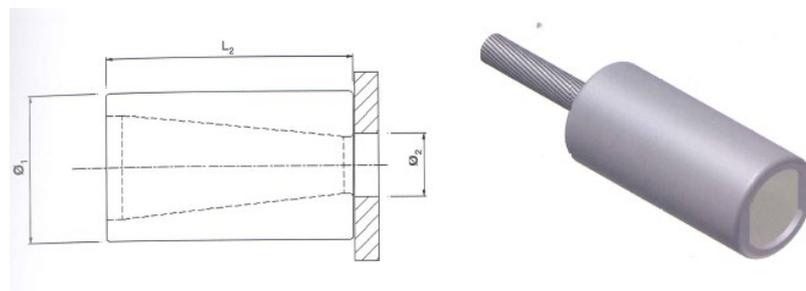


Figure 7.2-1 Plain cylindrical socket – alt. 1

The plain cylindrical socket, alt. 1, is the most common socket used for cable stayed bridges. It is cheap and simple, but the only way to regulate the cable length during installation is using shim plates between socket and supporting plate. It is most relevant for the passive end of the cable.

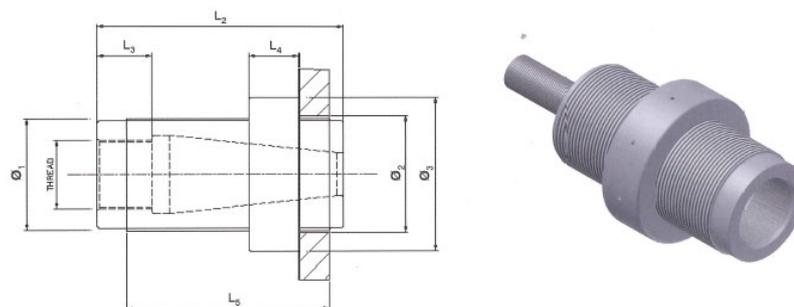


Figure 7.2-2 External thread cylindrical socket – alt. 2

The cylindrical socket with external thread, alt. 2, is a more sophisticated solution which is easy to adjust, but also more expensive. It is very suitable for the active end of the cable.

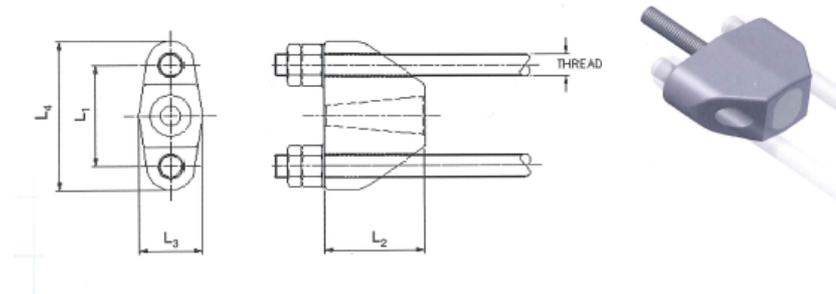


Figure 7.2-3 Block socket – alt. 3

The block socket, alt. 3, is the most common alternative for Norwegian suspension bridges, but have not been used for cable stayed bridges. It may be suitable for the rock-anchoring end, but needs a special arrangement for jacking.

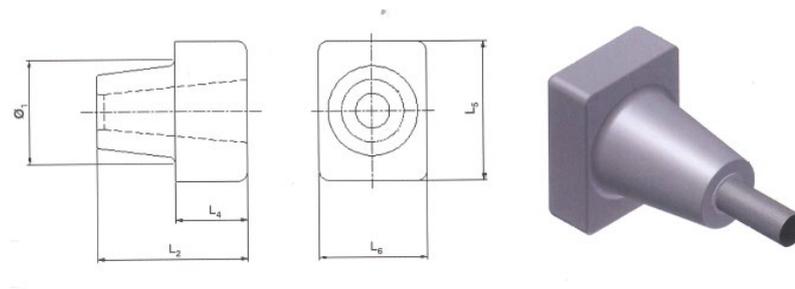


Figure 7.2-4 Hammerhead socket – alt. 4

The hammerhead socket, alt. 4, is a quite new solution. Like alt. 1, it is most suitable for the passive end of the cable. This alternative gives the possibility to inspect the area where the cable enters the socket after installation which can be difficult for alt. 1. This alternative requires a more complicated construction for support and is more expensive than alt. 1. This type was used inside the pylon for the Farris Bridge.

For the Bjørnafjorden Bridge, we have chosen alt. 1 for the passive end in the pylon and alt. 2 for the active end in the bridge deck.

6.3 Attachment of cable to bridge deck girder

We have considered two different ways of attachment between the cable and the bridge deck:

1. The cable is attached to a console outside the deck.
The console is welded to the external vertical steel plate located on each side of the steel-box and just outside an internal bulkhead which must be locally strengthened. In addition, the vertical steel plate must be strengthened with horizontal stiffeners. A tube is welded to the console with a neoprene damper and a watertight sealing in the upper end to minimize undesirable cable vibrations and penetration of water into the socket. See figure below.

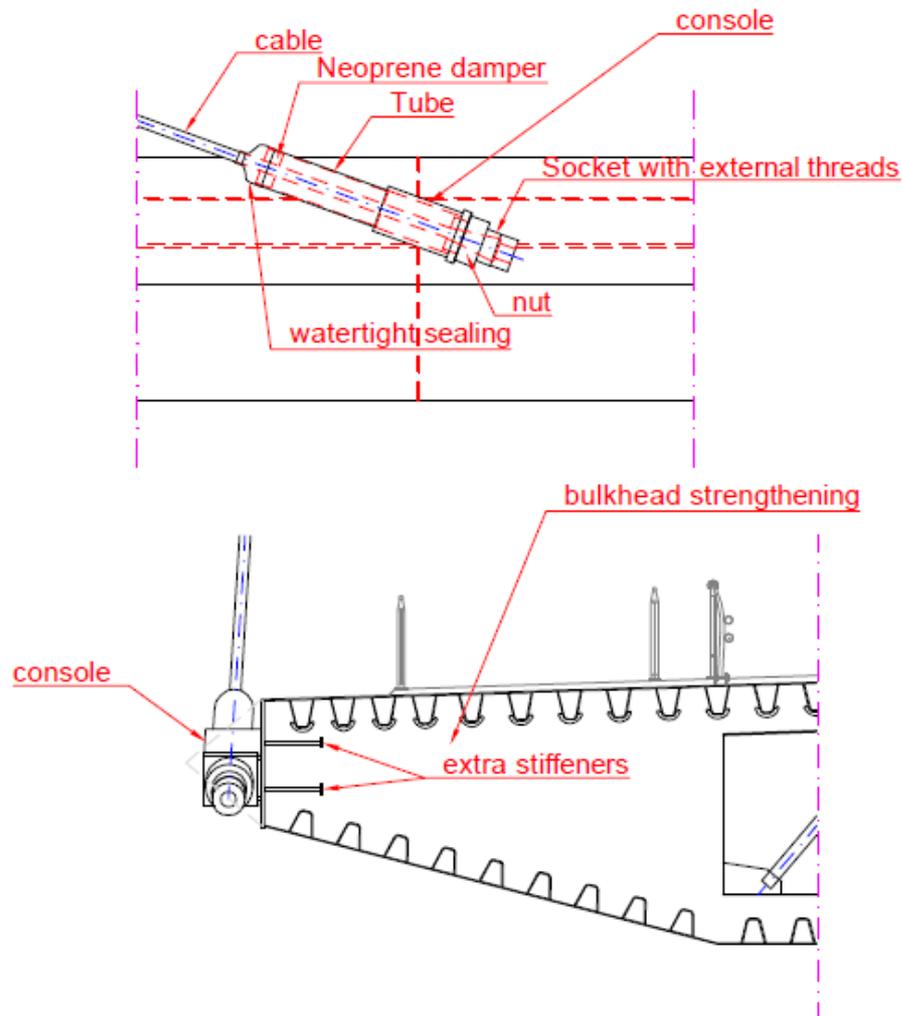


Figure 7.3-1 Cable attachment to bridge deck – alt. 1

2. The cable is attached partly inside the bridge deck. The box girder has an extra plate parallel to the external vertical plate and with two load-bearing plates in between to support the socket. The socket will be located outside, under the box girder. The tube with neoprene damper and sealing will be located on the upper side of the bridge deck. The bulkhead close to the cable attachment must be locally strengthened. With this alternative, the position of the cable attachment has not to correspond exactly to the position of the bulkhead and gives more freedom.

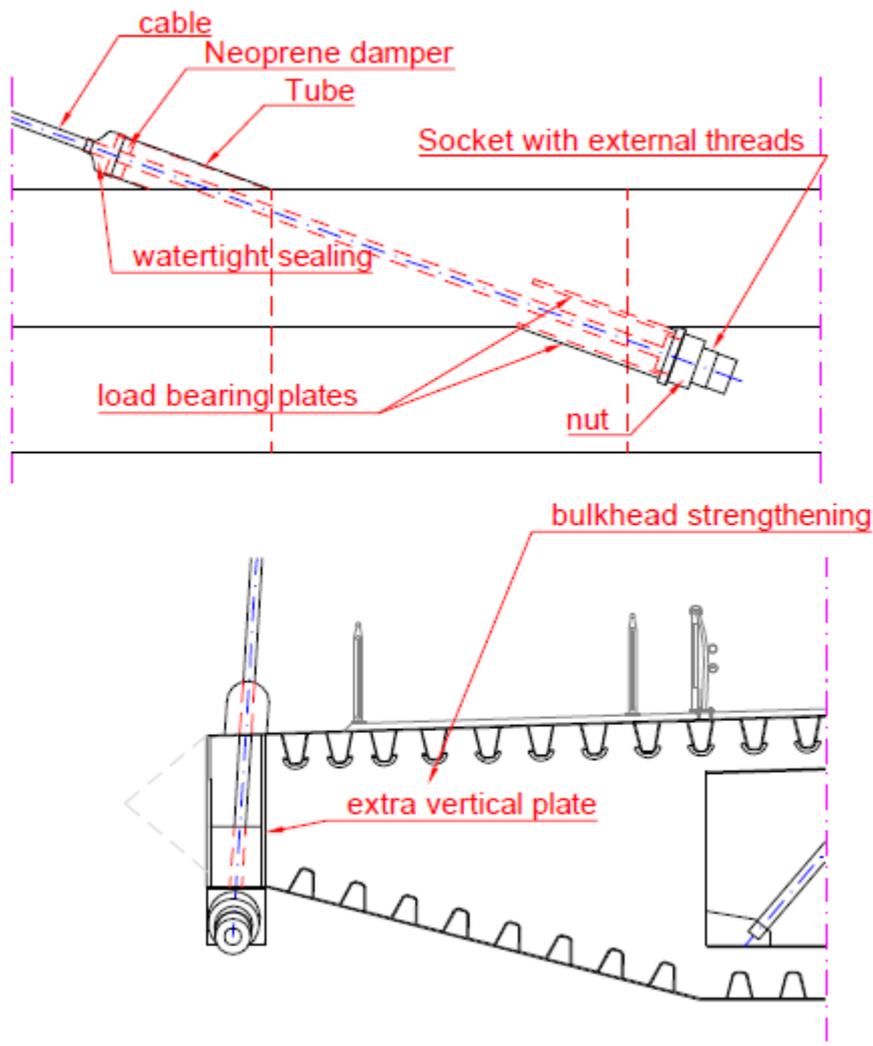


Figure 7.3-2 Cable attachment to bridge deck – alt. 2

For the Bjørnafjorden Bridge we have chosen the latter alternative. To give the bridge girder an aerodynamic shape, “noses” will be installed on both side of the girder. Alternative 2 will not affect these “noses”. Besides, the location of the attachments must not correspond to the bulkheads in this alternative.

For detailed design, see Appendix B.

6.4 Attachment of cable to pylon

Traditionally two different ways of attaching cable and a concrete pylon have been used, see figure below:

1. The cable socket is supported by a steel plate embedded in the concrete wall. The vertical component of the cable tension is transferred directly to the concrete. To transfer the horizontal component from one side of the pylon to the other, loops of post tensioned tendons are used.

2. The cable is attached to a steel-box which also serve as inner formwork for the pylon and connected to the concrete by headed stud connectors. The horizontal force component is taken care of by the side walls of the box, and the vertical component is transferred to the concrete by studs.

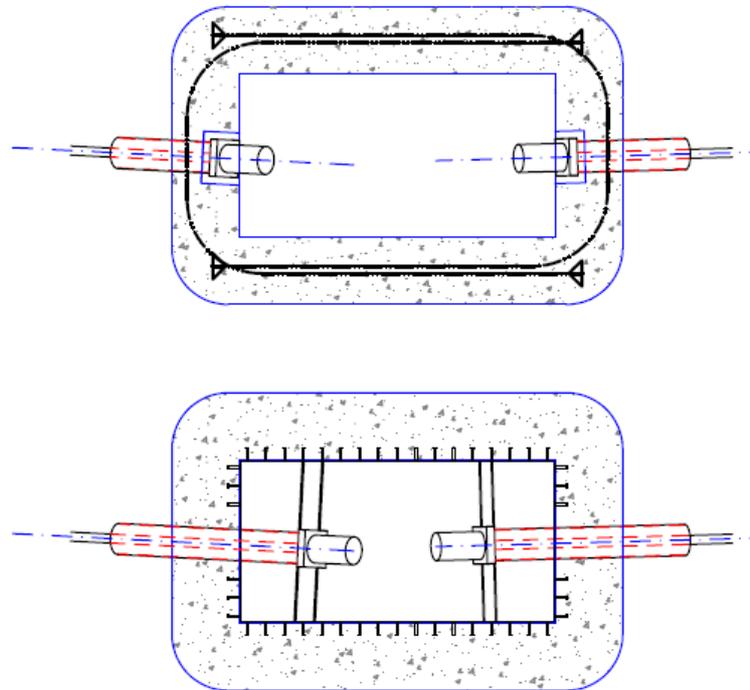


Figure 7.4-1 Cable attachment to pylon – alt. 1 and 2

If the forces are moderate and the number of cables limited, the first alternative is simple and economic. However, with a large number of heavy cables, the split-forces will be challenging, and the number and size of the tendons will complicate the construction process of the pylon.

For the Bjørnafjorden Bridge we recommend the latter alternative except for the lowermost cables which are very steep with low tension and small corresponding split-forces.

The general arrangement is shown in figure 7.4-2 and 7.4-3, and the he details are shown in figure 7.4-4.

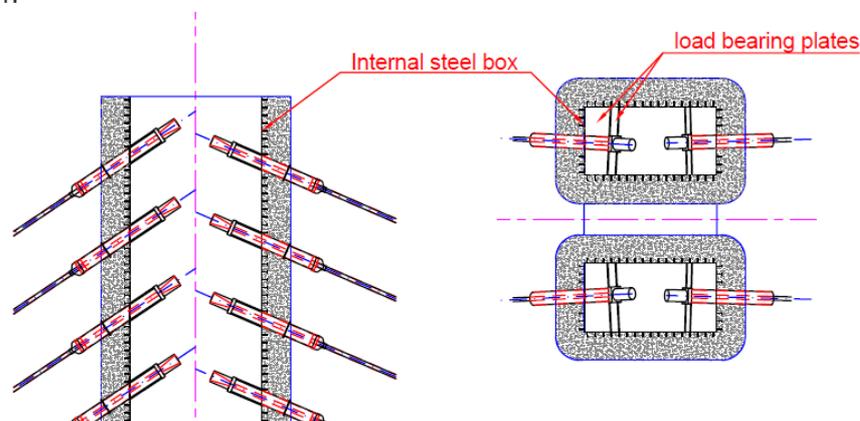


Figure 7.4-2 Cable attachment to pylon with inner steel box

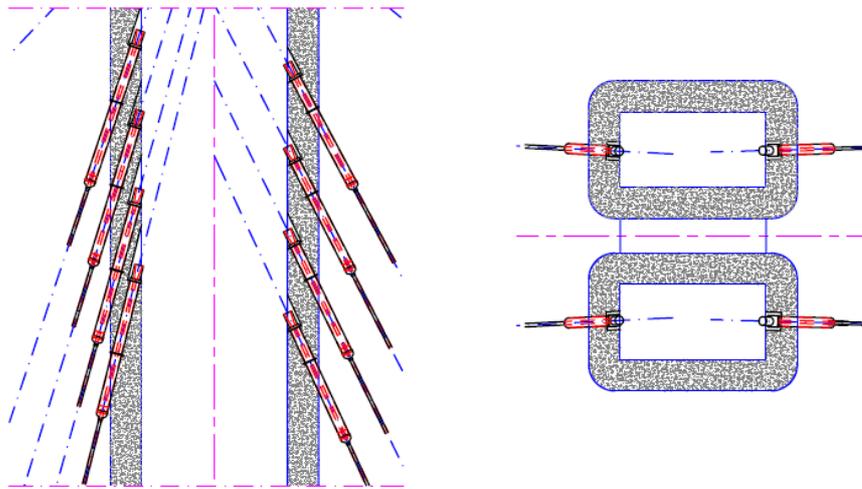


Figure 7.4-3 Cable attachment to pylon the lowermost cables

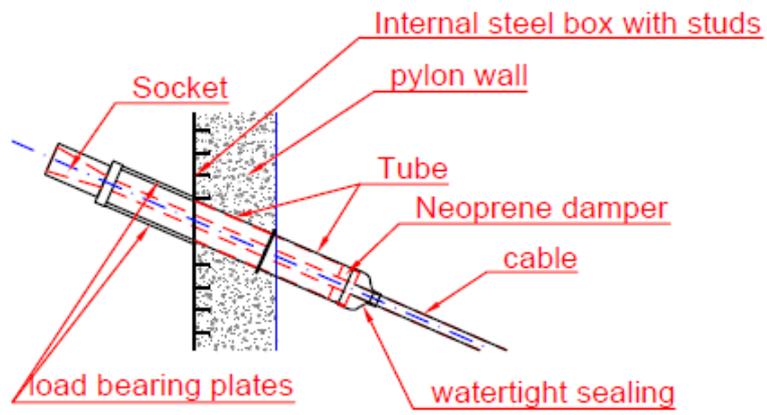


Figure 7.4-4 Cable attachment to pylon – details

For detailed design, see Appendix B.

7 ROCK ANCHORING

7.1 Design

Rock anchoring of bridge cables is used for several Norwegian suspension bridges and for one or two cable stayed bridges as well.

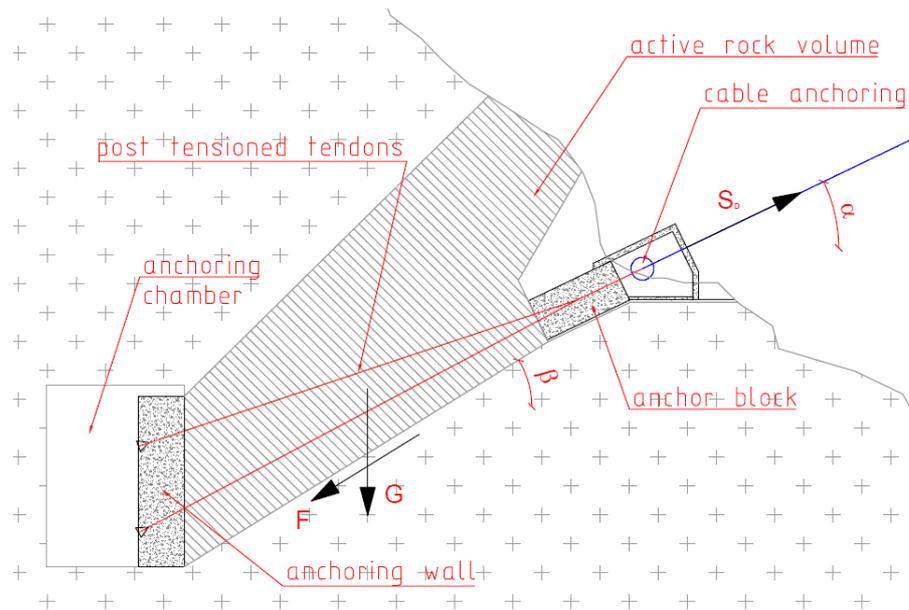


Figure 8.1-1 Rock anchoring - principle

The typical anchoring system includes a rock-chamber with an anchoring wall and an anchor block outside where the cables are attached. The anchor block and the anchoring wall are connected by post tensioned tendons through the rock, protected by polyurethane tubes, see fig. 8.1-1.

To calculate an exact capacity of a rock anchor like this is difficult and probably not possible. Therefore, simplified and conservative methods have been used. The system shown in the figure defines a rock volume which gives the capacity of the anchoring by its weight G and the friction force F only. Friction coefficient is set to 1,0. The material properties of the rock (shear and tensile strength) are not considered. In addition, the rock density is reduced because of the buoyancy from a presumed level of the groundwater.

7.2 Local cable-attachment

The easiest way to fasten the cable to the anchor block is using a block socket connected to two stays which are attached to the post tensioned tendons by a steel plate, see fig. 8.2-1. This is the traditional way which is used on a number of suspension bridges in Norway.

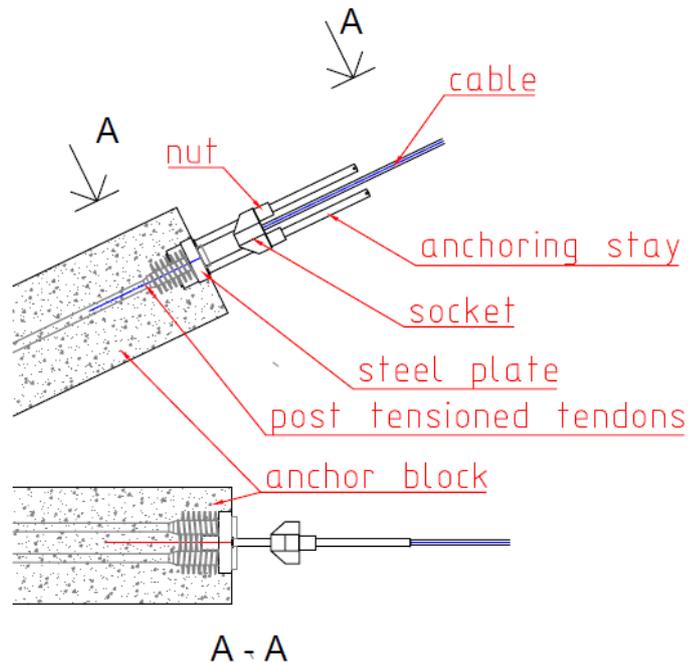


Figure 8.2-1 Attachment cable/anchor block

However, with a cable stayed bridge there is a need for jacking during the erection process which is unnecessary for a suspension bridge. A standard block socket is not suitable for jacking. Therefore, the sockets must be designed so that jacking is possible. Sockets for cables of this size will always be tailor-made, so a special solution will hardly give rise to the costs. A possible solution is showed in fig. 8.2-2.

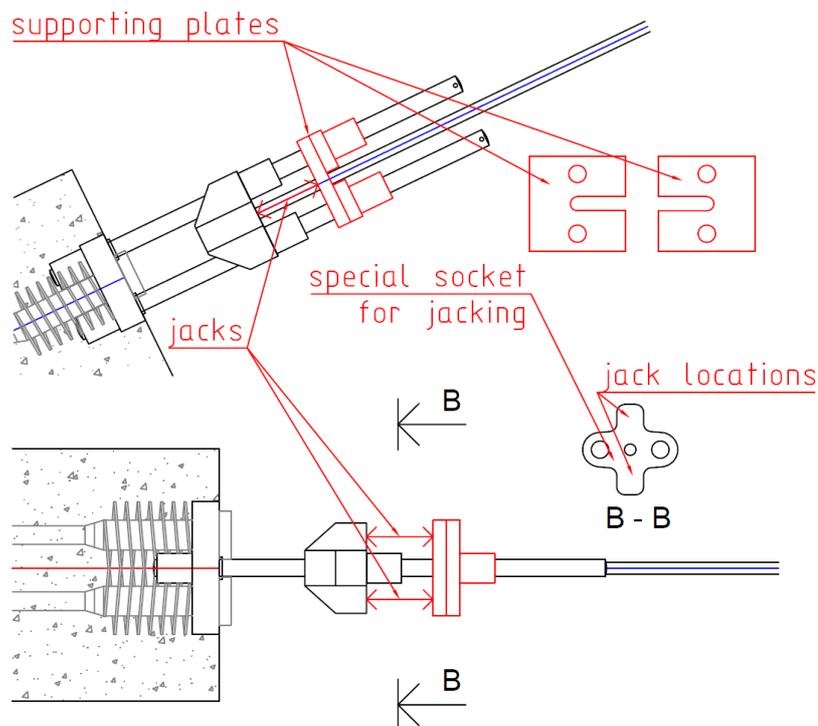


Figure 8.2-2 Possible solution for jacking

8 EQUIPMENT

8.1 Temporary bearing against tower

The temporary horizontal bearing between the girder and the tower, and the local strengthening of the box girder is calculated in appendix C

8.2 Damping on cables

Necessary dampers on stay-cables is a complicated topic. In the validation report [9] necessary added damping for galloping is found to be 0.9%, and 0.7% for parametric excitation. It is important to note that damping from parametric excitation should be mechanical dampers and not aerodynamical measures.

Additional dampers come in many shapes and forms and are used on most conventional long span cable stayed bridges. In the cable chapter, a neoprene damper is suggested, but also frictional and viscous dampers are regularly used.

The final damper properties are up to the manufacturer of the cable-system to decide, but we need to know that it is possible to produce the damper that we specify. The possible amount of damping is dependent on the placement of the damper from the support. [10] gives a good overview of some types of dampers and their properties. If the detailing phase suggest that the final additional damping amount is 0.9%, and we use a friction type damper from VLS international and place the damper 8,5m from the cable ends, we could get a damping amount of about 0.6% from a single damper. Each cable is usually fitted with dampers at both ends giving a damping of 1,2%. This shows that we can easily get dampers that satisfy our demand.



> Figure 7 Example of damper, VSL - friction damper

9 FURTHER WORK

Further work on the stay-cable part (not considering in depth design of the components) should include an updated abutment position. The size of the southern abutment and the aesthetic impact of the abutment should be investigated. It is probably a better solution moving the abutment further away from the main span giving a longer side span of the stay cable bridge.

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