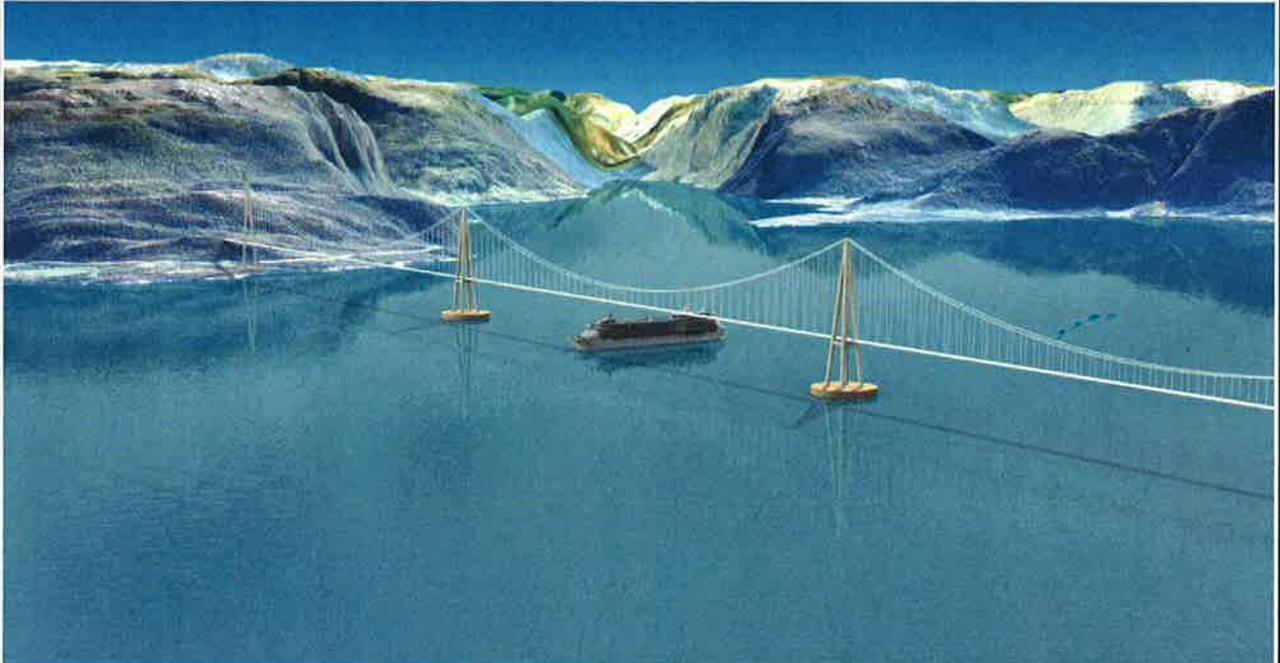




Statens vegvesen
Region vest

Sognefjorden Feasibility Study of Floating Bridge



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bygg- og anleggsteknikk

COWI



SKANSKA

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SUMMARY

A design group consisting of the companies Aas-Jakobsen, Johs.Holt, Cowi, NGI and Skanska have been awarded by Norwegian Public Roads Administration Western Region to do a feasibility study of how to cross the Sognefjord at the site Lavik-Oppedal by means of a floating bridge. At the site, the fjord is 3700m deep and 1250m deep, making the crossing extremely challenging. The concept that is investigated is a floating suspension bridge with three spans and onshore viaducts. The span widths are 1234m and the total length of the bridge is 4400m. The offshore towers are supported on concrete pontoons, while the towers itself are made of steel. The two towers at shoreline are made of concrete. The bridge deck is an orthotropic steel box girder. The pontoons are anchored to the sea bottom and to the shore through suction anchors and a combination of subsea steel stays and offshore chains. The concept is developed by the design group through initial studies involving concept development and a dialogue phase with the Client.

The feasibility study consider technical, constructional, environmental and risk evaluations. Construction costs have not been a part of the scope.

The response of the structure has been calculated and evaluated for every relevant loading, and all structural parts are designed to find the adequate dimensions and solutions. The structure is also checked for relevant accidental loading, for instance ship collision.

All design checks have been found satisfactory; implying that the design group has concluded with that the concept is feasible for crossing the Sognefjord.

The design group has utilized a number of commercial design and analysis programs in order to investigate the behaviour of the structure, such as RM Bridge (global analysis, dynamic wind analysis, ship collision), USFOS (ship collision), MIMOSA (anchor- and movement analysis), WAMIT (wave analysis), Focus (pontoon design), FEM-design (pontoon design) and Novaframe (initial global analysis). In addition, a lot of spread sheets and longhand calculations are developed.

Below is given an overview of vital data for the structure:

Total length suspension bridge part	3 700m
Total length viaduct	ca. 700m
Tower height above sea level	211m
Pontoon dimensions	ca. 75 x 180m (diameter x height)
Concrete volume pontoons	105 000m ³ per pontoon
Ballast volume pontoons	155 000m ³ per pontoon (olivine)
Steel weight towers	15 350t

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Steel weight bridge deck incl. viaduct	26 270t
Steel weight main cable	Ca. 12 000t (for 2 cables)
Total length anchor cables	61 630 m (58 810 m stays and 2 820 m chain)
Dimensions suction anchors	ca. 6 x 19m (diameter x height)
Number of suction anchors	32

For upcoming phases of the project, the following items should be looked further into:

- Further evaluation of anchor configuration
- Further evaluation of boundary conditions for the bridge deck
- Further evaluation of anchor stay materials
- Further evaluation of eccentric and directional ship collisions
- Further evaluation of damping effects for structures in water due to wind excitation

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1 INTRODUCTION

1.1 General

The feasibility study for a floating bridge crossing the Sognefjord in highway E39 is performed as one activity in the project «Ferryfree E39» hosted by Norwegian Public Road Administration. This site is chosen for a feasibility study since the fjord at the actual place is app. 3700m wide and 1250m deep. The idea has been that if a feasible solution is found here, also crossing the other fjords on the same project will be feasible, making the whole project feasible.

A design group consisting of the Aas-Jakobsen, Johs.Holt, Cowi, NGI and Skanska was summer 2012 awarded the contract to perform such a feasibility study for a floating bridge concept. This was done after an initial concept phase and a dialogue phase with the client. The five companies involved are joint responsible for the results and is to be looked upon as equal parties. In addition the design group has hired specialists in several important areas. The participants in the design team are shown in the table below.

AAS-JAKOBSEN	JOHS HOLT	COWI	NGI	SKANSKA	ANDRE
Svein Erik Jakobsen	Per Norum Larsen	Erik Sundet	Kjell Karlsrud	Lars Bjerkeli	Øyvind Nedrebø (eget)
Rolf Magne Larssen	Gunnar Egset	Bernt Jakobsen	Per Sparevik	Olai Meland	Øyvind Johnsen (MTC)
Ketil Aas-Jakobsen	Carl Hansvold	Johan M. Kjelling	Roger Olsson	Torbjørn Kjoberg	Odd Faltinsen (NTNU)
Liv Eltvik	Ame Cristensen	Jianhu Ma	Thomas Langford		Ame Bruer (TDA)
Komelius D. Hole		Inger Birgitte Kroon			Poul O. Jensen (D&W)
Knut Aas-Jakobsen		Mads Jorgensen			
Per Meaas					
Anders Fosnes					

Table 1.1: Project team

The client has been «Statens vegvesen Region vest».

Project leader has been Lidvard Skorpa.

Contact person for the client has been Johannes Veie.

1.2 Definitions

Nomenclature used in the report:

Floating bridge:

- Floating structure or the part of the structure that is supported by pontoons or similar.

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Anchor system:

- Arrangement of cables or other structural elements to keep the floating bridge in position. The system is divided into the following elements: Bottom anchor, anchor lines and top fastening

Pontoons:

- Buoyancy members attached to the superstructure and possibly supported by anchor systems. Can in general be fully submerged or partly submerged.

Shore fastening:

- Supportive structures to the floating bridge on shore. The shore fastening may be a part of the anchor system.

Splash zone:

- Level from 3m below MSLV to 3m above MSHV, where
MSLV is mean extreme low tide level, and
MSHV is mean extreme high tide level.

Life cycle period:

- The required life cycle in years from opening.

1.3 Project specifications

The design basis is outlined in a separate report, report no. 11258-01.

Vital to the content has been the SINTEF report: «Sognefjorden Opedal-Lavik. Estimert på bølger og strøm, 2009-12-03», ref. /3/. In case of lack of information, relevant assumptions have been made.

As a part of the feasibility study, risk analyses of ship collisions has been performed by Rambøll, see ref. /12/. Current revisions of their reports have always been used as a basis for the work.

Vital parts of the structure have been designed by means of self-imposed requirements for deflections and rotations.

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These are:

Load combination	Response	Allowable limit
«SLS characteristic» with wind only as variable load	Rotation about longitudinal axis of the bridge	0,50°
«SLS characteristic» with traffic only as variable load	Vertical deflection of bridge deck	Between L/200 and L/150, where L=span width
«SLS often occurring» with load factor 0.7 for traffic	Rotational angle at support	1,91°

Table 1.2: Response requirements

Throughout the feasibility study, various sea bottom depth maps have been available. In the study the basis has been available information at the start of the project. An idealized topography has been made based on this, see figure below. At a later stage of the project, the client came up with a more detailed topography. The idealized topography was however kept through the design since the more detailed one to a large extent confirmed the assumptions. The drawings do however reflect the detailed information. As can be seen on the drawings, placing of the anchors are optimized accordingly.

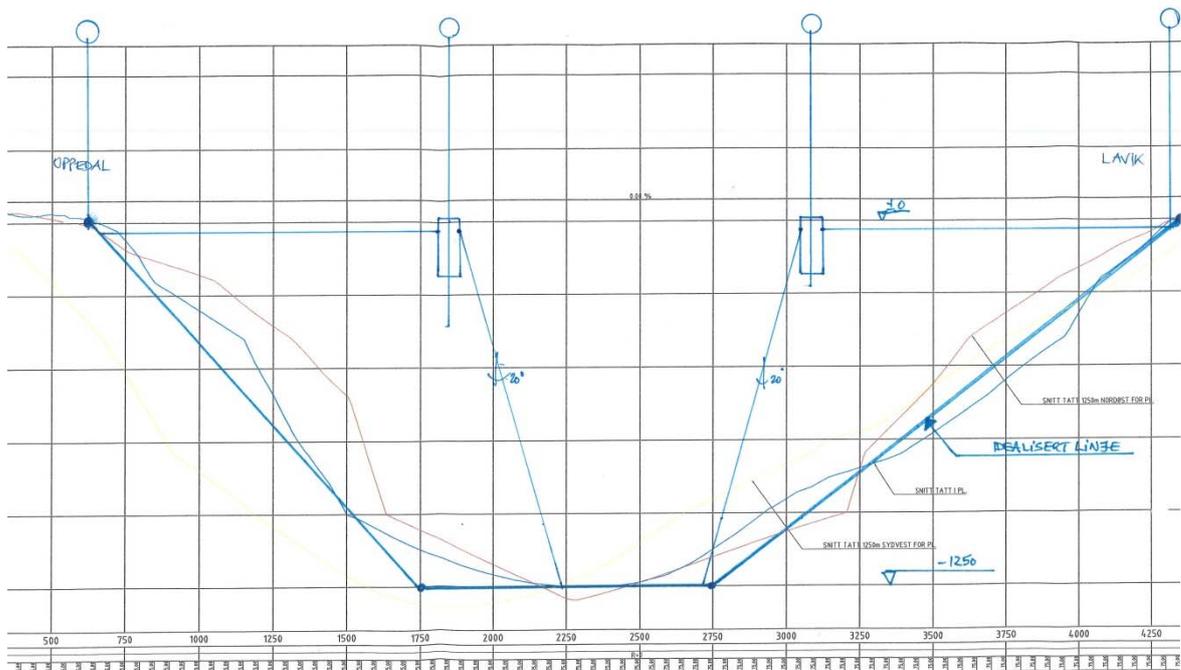


Figure 1.1: Idealized topography

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1.4 Structural design

The structure is shown on the following drawings, see also separate chapter:

11258-K001	Oversikt
11258-K011	Byggefaser, pontonger og tårn I
11258-K012	Byggefaser, pontonger og tårn II
11258-K013	Byggefaser, pontonger og tårn III
11258-K014	Byggefaser, pontonger og tårn IV
11258-K015	Byggefaser, pontonger og tårn , dokk/kai
11258-K016	Byggefaser, hovedkabel og brubane
11258-K021	Tårn akse 2 og 5
11258-K022	Tårn akse 3 og 4
11258-K023	Landkar/Horisontalfjær
11258-K031	Forankringskamre
11258-K041	Avstivingsbærer
11258-K101	Pontong. Oppriss og vertikalsnitt
11258-K102	Pontong. Plansnitt kt -130 til kt -175
11258-K103	Pontong. Plansnitt kt -25.5m
11258-K104	Pontong. Plansnitt skipsstøtsone
11258-K301	Forankringssystem

Table 1.3: List of drawings

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A brief explanation of the chosen solution for each structural part is given as follows:

Structural part	Solution
Bottom anchors	Suction anchors with diameter 6m and depth 19m
Anchor lines	Steel stays with diameter app. 150mm
Top fastening	«Fairleads» fastened to the pontoon wall and supported and anchored at top slab. Jacking device at top slab.
Pontoons	Nine cell concrete floater
Towers upon pontoons	Steel tower with four legs
Towers ashore	Concrete tower with two legs founded on rock
Columns ashore	Concrete columns
Bridge deck	Orthotropic steel box
Viaduct	Orthotropic steel box
Abutment	Concrete abutment founded on rock
Tower saddles	Welded steel
Spreading saddles	Welded steel
Spreading chamber	Concrete structure founded on rock
Anchor chamber	Concrete structure in rock tunnel

3D-plots of the structure are shown in the following.

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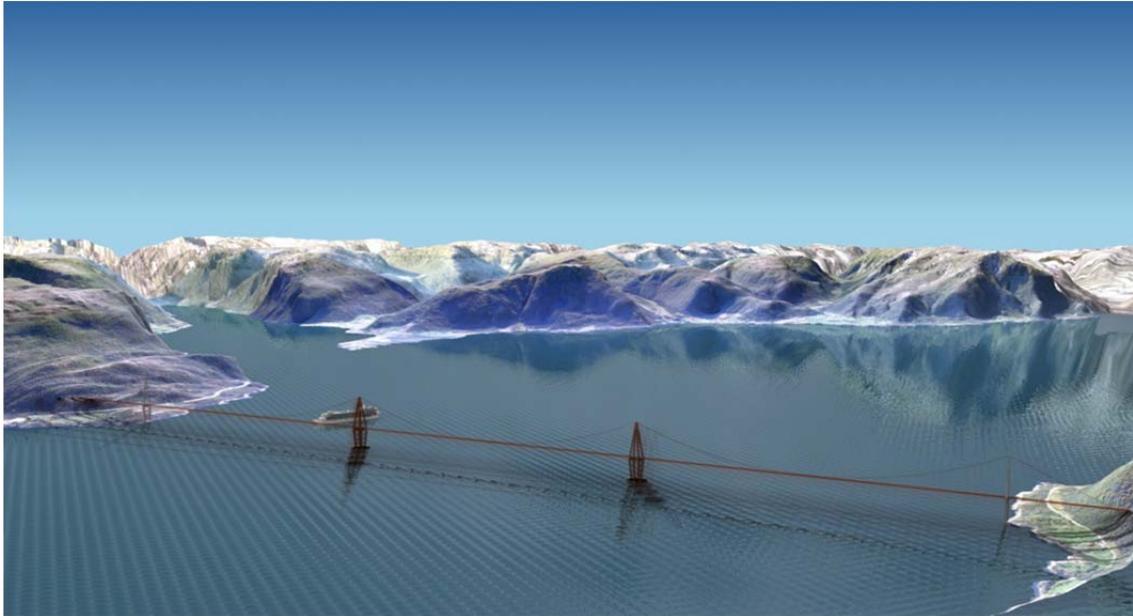


Figure 1.2: Overview I

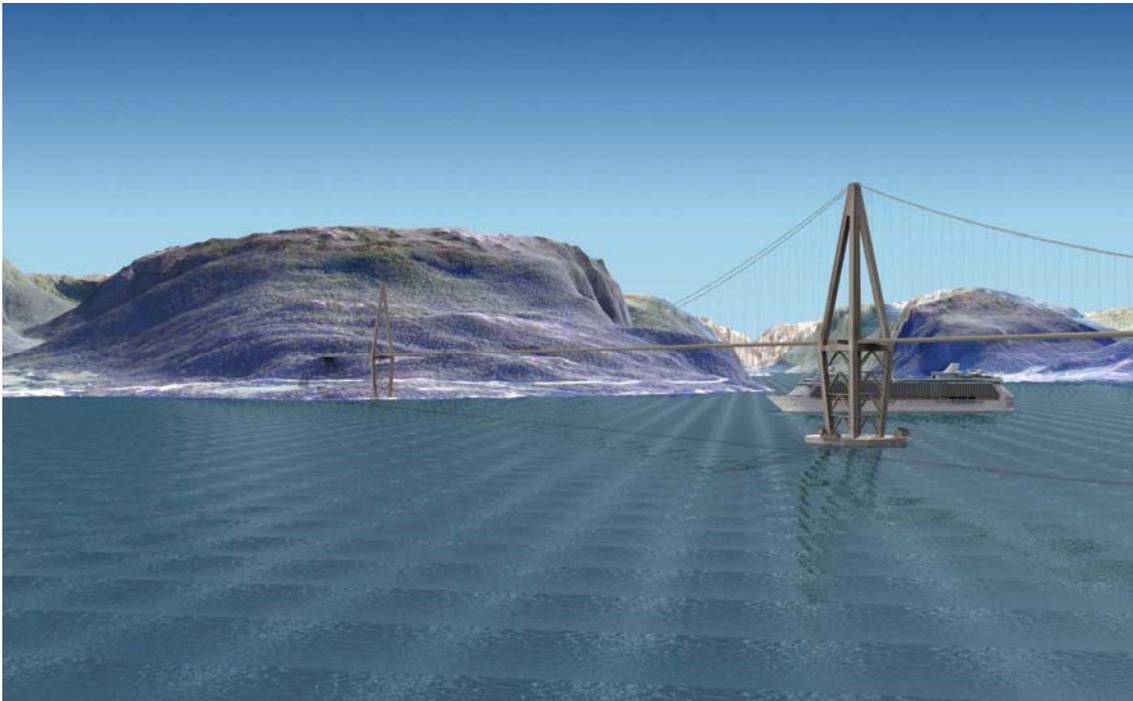


Figure 1.3: Overview II

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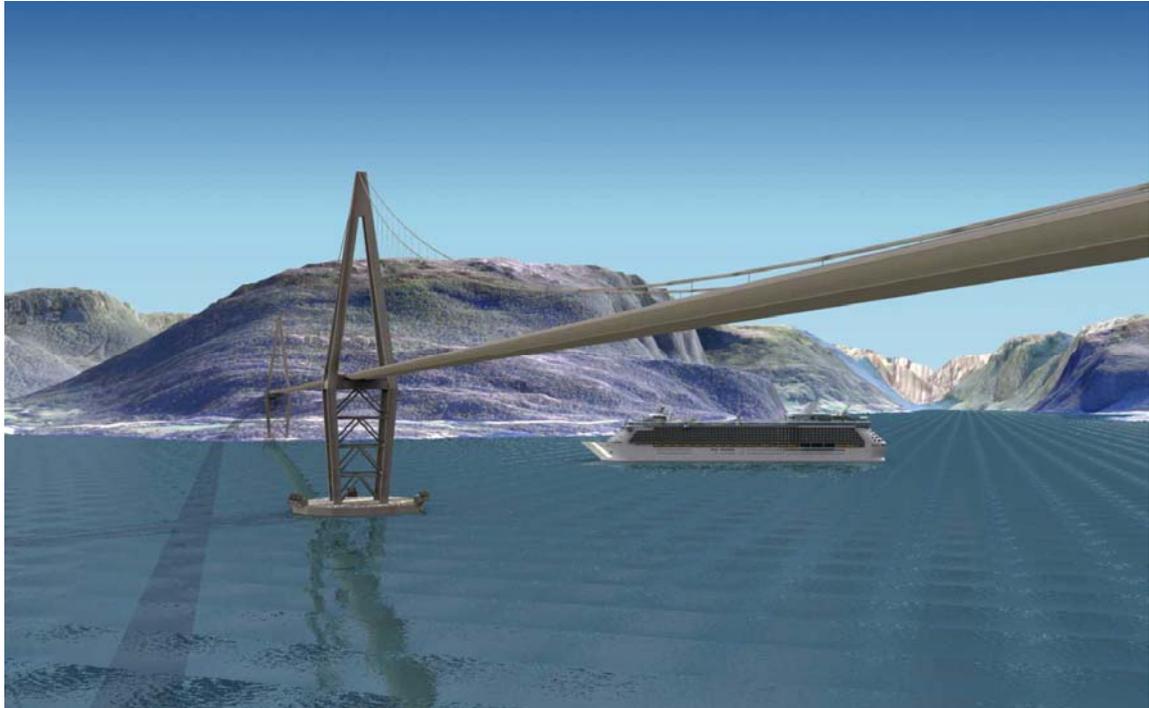


Figure 1.4: Overview III

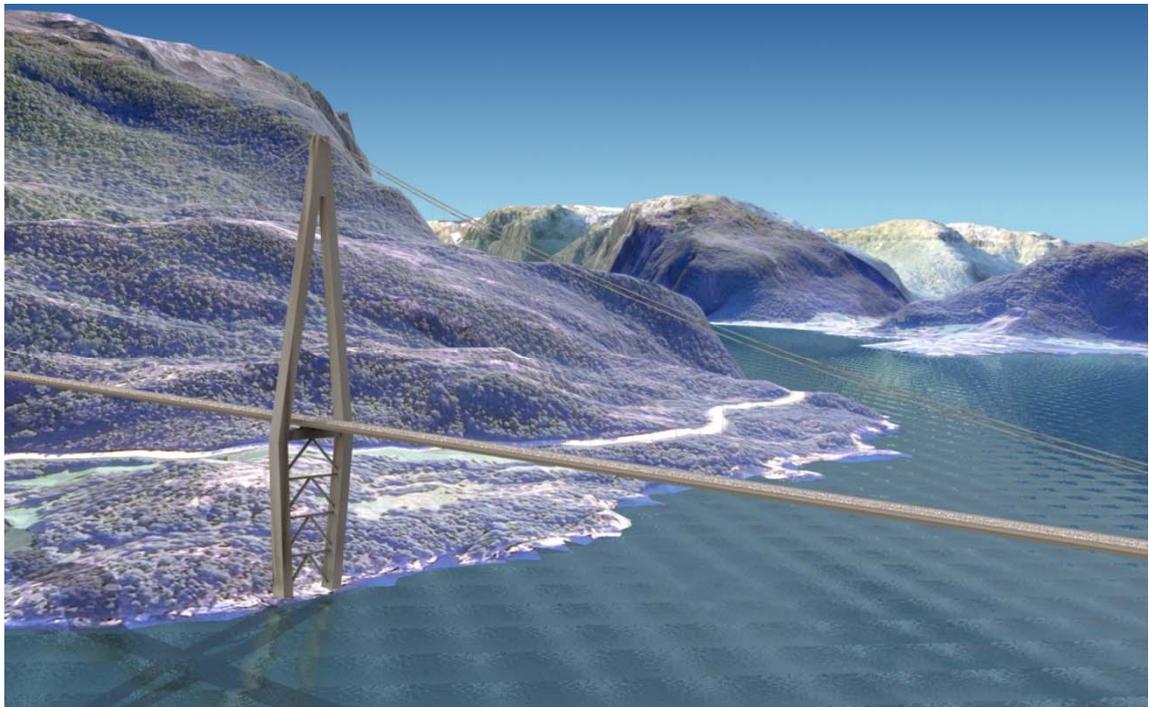


Figure 1.5: Overview IV

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Figure 1.6: Under water view (anchor lines not shown)

1.5 Design procedures

Overview of used analysis programs:

Program	Used for
RM	Global analysis, dynamic wind analysis, ship collision
NOVAFRAME	Initial global analysis
USFOS	Ship impact
WAMIT	Wave analysis
MIMOSA	Anchor analysis, wave response analysis
FOCUS konstruksjon	Pontoon analysis
Fem-design 3D-structure	Pontoon analysis

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1.6 Construction methodology

1.6.1 Suction anchors

The construction and erection methodology will depend on the types chosen.

Two types are described in chapter 8 and in Appendix F. The shallow type is typically constructed in dry dock and floated out before installation. It has buoyancy tanks that keep it floating temporarily. The deep type is typically transported on barges before installation.

Both types will be installed using heavy offshore cranes controlling the anchor during installation. The anchors will for the assumed soil conditions penetrate almost to final position just by its own weight. Additional suction will easily be managed using pumping devices available in the market.

1.6.2 Anchor lines

The anchor system is shown on dwg. 11258-K351. The suction anchors (see description above) are installed with bottom anchor chain attached to the anchor. In the top of the chain, there is a joint device to be attached to the anchor cable. This operation is done with remote driven equipment. The top part of the anchor is attached to floating devices and will be linked to the pontoon upon arrival. The jacking device at top slab will tighten the anchor lines to their final force and thereby finally position the pontoon. It will also be possible to install the whole anchor system at one go prior to pontoon fastening.

1.6.3 pontoons and towers

The construction of the large concrete pontoons and the steel towers can mainly be based on well-known construction technology from marine structures for the oil and gas industry in the North Sea.

The main principles for this work is shown in chapter 11, see also details in Appendix I.

1.6.4 Main anchor chambers

The anchor chambers and spreading chambers are constructed in rock tunnels or inside blasted away rock, similar to e.g. Hardanger Bridge.

1.6.5 Main cables and hangers

The main cable is constructed through aerial spinning. At first, a "catwalk" is erected over the whole cable length. The catwalk may be secured through a storm system to the shore and to the pontoons. Each tread is than spinned over the bridge length. The treads are collected to bundles, assumed to be 19 in each main cable. The first bundle is placed thoroughly in good weather without wind, waves and sun. Additional cables to pull back towers or ballasting of pontoon cells is not considered at this stage of the project, but will be possible to optimize the geometry.

Aerial spinning will be a timeconsuming erection method for this bridge. Therefore, one should also evaluate the use of prefabricated parallell bundles with anchors in each end. The

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bundles is collected to a circular shape as for aerial spinning. A challenge for this method will be the large weight of each bundle, calculated to be 300 tons for 19 bundles.

The completion works of the main cable as well as erection of the hangers are done in a traditional way.

1.6.6 Bridge deck

The bridge deck is produced in the traditional sectional way. It is assumed that the bridge deck is transported and lifted from three different ships/barges. The sections are lifted from cranes mounted on the main cable, symmetric from the centre of the span but simultaneously on each span to avoid eccentric loading of the dead weight. The sections in between the tower legs must be erected before the adjacent sections.

After erection and temporary fastening, the sections are welded together, before final placing of equipment and asphalt.

1.6.7 Viaduct

The viaduct is designed as steel boxes. These can be erected from sea by lifting in between the columns or by incremental launching from the abutments.

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1.7 Discussion of results

Bridge deck

The design is based upon a continuous beam from axis 1 to 6 with expansion joints at each end. The beam is kept in place by external post tensioning cables, 500m long. Since the viaducts are 300m, end anchorage of these tendons will be on the sea side of axis 2 and 5. This implies that to move the expansion joints to axis 2 and 5 should be investigated further.

The chosen concept has viaducts supported by columns with sliding bearings on top. These bearings have very large movements that may cause some maintenance challenges. Further investigations should therefore be made to avoid these movements, either by moving the expansion joints as described above (recommended) or by utilizing the main cable as vertical support to the beam.

Anchor system

In the feasibility study, steel anchor lines are used. In the work, also composite material was evaluated. These are much lighter than steel and will have less sag. This is beneficial for the characteristics of the cable system. Even if composite material is widely used in the oil and gas industry, the design group chose not to adapt this solution, mainly due to lack of long-term experience in the market and limited time in the study.

In the feasibility study, the possibility to avoid bottom anchorages in favour of shore anchorages only, has not been investigated. This is due to the fact that the distance to shore in south east direction is large. Given another and more suitable site, this should however be looked at.

It is the design group's opinion that a combination of vertical tension legs and skew anchorages as chosen, should be looked into for the next phase of the project. This will even be more beneficial for smaller water depths.

Future traffic and number of lanes

The concept is designed for two lane traffic and a pedestrian lane. Increasing the number of lanes will increase the challenges with limiting the deflection in the bridge deck and controlling the friction between the main cables and tower saddles.

Skip impact

The pontoons are designed to resist the given ship collision. The superstructure is also designed for this, resulting in minor local strengthening. This is based on the assumed 2% damping in the system. We find this value to be realistic conservative. It should however be verified through theory and models tests in later stages of the project.

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A fender system should be introduced in order to protect the pontoons for minor damage and to protect the ship and its crew and passengers in an appropriate manner: For the given ship collision, the major impact will be on the ship and not the bridge. Also the effect of eccentric impact should be evaluated further.

Wind response

The wind response analyses are performed for a lower limit of damping involving only structural damping (0.5% of critical) and aerodynamic damping. The damping from movement in water is not taken into account. Since this is the governing loading, a more detailed analysis where all damping effects are accounted for, could give a substantial reduction of the utilization of the structure.

To account for this effect, calculation tools must be developed further to ensure consistency and correct damping should be verified through further investigations in theory and model tests.

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2 RISK EVALUATION

2.1 General

The risk evaluation within the project is performed as a continuous activity, emphasizing the risk items identified in the initial development of the concept. In this work, structural safety and integrity has been the main issue through developing solutions that withstand all adequate loading conditions in the construction and operation phase as well as accidental scenarios.

The most adequate accidental scenario is ship collision. This has been a vital issue through the whole study. The basis for the discussions has been the risk analysis performed by Rambøll, ref. /12/. This analysis has also been developed further through the project.

In addition the design group has performed a special risk evaluation of the robustness of the structure facing identified risks moments and happenings. The outcome of this risk analysis is documented in Appendix J.

In addition to a site specific risk evaluation, the concept has been undergoing a special evaluation with respect to use in other site locations.

2.2 Risks related to the operational phase

The pontoon and the superstructure are checked for ship collision. The bridge deck will require some local strengthening to avoid utilization to be exceeded if taking the ship collision scenario with return period of 10^4 years into account. The damping factors, which are crucial to the response, are conservatively chosen. If later investigations show that increased damping is needed, one should evaluate steps to increase the damping, for instance through the fendering system or by introduction of vertical ribs at the outside lower parts of the pontoon.

Collisions from submarines are not considered likely at this stage. It is also assumed that the pontoons will be equipped with hydroacoustic transmitters to alert submarines in the neighbourhood. If later assessments shows that further steps should be taken, this could imply structural strengthening of the pontoons.

The pontoons have large safety against loss of buoyancy in the upper part above elevation - 25 m. Water filling below this point is however disastrous. The concept is designed in such a way that the probability of failure below this point is negligible. This will however imply strict control and monitoring through the construction and operation phase as well as an extensive pumping system.

The concept will have large robustness against increase of the ship traffic in the fjord, due to the large spans and water depths.

The concept has also large robustness against variation of underwater topology, see Appendix J, section A.2.

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In general, the implication on the environment in operational phase is considered low.

The risk for injuries to personnel is considered for inspection and maintenance workers, for users and for 3rd party personnel. The work on the structure imply both diving and climbing, but is not much different from other major bridges, taking into account that deep sea inspection must be carried out with ROVs and that special access devices will be installed in all other parts of the structure. The users of the bridge will not experience this one as different from other major bridges. For 3rd party personnel, passengers on passing ships will be a vital group. In this respect, investigations must be done on the behaviour of the ships during collision with the pontoons. It is however assumed that this risk is less than other risks on the same boat journey. The risk can also be reduced through signs and marking of the ship channel, see also Appendix J, section D.3.

2.3 Risks related to the construction phase

The risk for personnel in the construction phase is considerable due to the large heights and to extensive marine operations. This will be similar to construction and installation of marine structures in the oil and gas industry. One must therefore adapt the same safety regulations that are provided in this sector.

The structure is quite flexible compared to a regular fixed bridge. This must be taken into account in the time schedule planning and in the restrictions for work, and may result in a longer construction period.

2.4 Using the concept in other bridge sites

The concept may also be used for areas with larger traffic volume, implying more lanes. This will increase the construction costs since it will have an impact on towers, main cable and deflections.

The following aspects need to be considered for other sites:

- To use the concept in open sea, the effect of larger wave heights and longer wave periods needs to be considered. This may imply other types of pontoons and anchor systems.
- If the site has low terrain on none or both sides of the crossing, the length of the viaducts has to be increased a lot. Ship channel clearance will then be vital.
- Sites with different soil conditions at the bottom of the sea may require different anchor solutions.
- Sites with more shallow water may require using tension leg platform technology for vertical and horizontal support of the towers, possibly in a combination with stays.

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2.5 Conclusion

It is concluded that the proposed concept is feasible with respect to risk assessments, both in operation and construction phase. Later risk analyses will reveal the necessity to adjust the concept, but it is not believed to be likely that further investigation will alter the conclusion above.

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3 GLOBAL STATIC ANALYSIS

3.1 General

The global analysis is performed utilizing the program system RM-bridge. The analysis take into account large deflections, which is vital to give a correct layout and design of a multispan suspension bridge. In Appendix A «Globalanalyse» geometry, materials, boundary conditions and loads are described.

The considered loads include dead weight, superimposed load, buoyancy, temperature, traffic loading and wind loading. Dead weight and superimposed load correspond to a uniform loading of app. 120kN/m. For temperature loading only uniform increase and decrease of the material temperature is considered. Traffic loading includes 2 loaded lanes and pedestrian loading. This corresponds to a uniform loading of app. 20kN/m in addition to concentrated loading in accordance to HB185 issue 2009.

Environmental loading from waves and current are described in chapter 5. The investigations have revealed only minor response and thereby minor sectional forces on the structure from 1st order waves and current. These loading are therefore omitted in the global analysis. The investigations of the slow drift 2nd order wave forces shows a response of app. 2m transverse on the pontoons, compared to static and dynamic wind response and app.10m. This imply that in a ULS situation with load factor 1.6 on wind and 1.12 on waves, the wave effect will only contribute 12% to the total loading and very little to stresses in the bridge deck. It is therefore omitted.

For pontoons anchored in three directions horizontally, there will be a longitudinal component to transverse deflection. This will cause imposed loading to the tower and bridge deck. Estimated loading is approximately 8.5MN in ULS. This load is included in the global analysis.

In ULS, the governing loads are primarily dead weight, traffic loads and wind loads. The focus has therefore been on load combinations where one of these is dominant. For dominant wind, no traffic is assumed. For dominant traffic, both static and dynamic wind loading with a load factor of 1.12 is assumed. This is meant to be conservative.

Results for applied loading are shown in Appendix A. For the chosen concept, maximum deflection from traffic is calculated to be app. 7.5m in mid span and 5.5m in side span. Side span is defined as spans between axis 2 and 3 or between 4 and 5. For mid span the deflection correspond to a ratio of L/165 for traffic with load factor 1.0. This is within the working criteria chosen to be between L/200 and L/150. The largest twist of the bridge deck due to eccentric traffic is 2.9 degrees. The largest rotation of pontoons (height 175m) from traffic is 0.5 degrees.

Design and evaluation of stresses in ULS in towers, bridge deck and main cables, for the applied loadings are treated in Chapter 10 and Appendix H. Ship collision loading is handled in Chapter 7 and Appendix E.

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3.2 Parameter study

In Appendix A chapter 6, a parametric study is performed in order to evaluate different boundary conditions and loading. The following items is looked into:

- Rotational stiffness of the pontoons
- Axial spring stiffness of the end of the bridge deck
- Spring stiffness of anchor system in longitudinal direction
- Spring stiffness of anchor system in transverse direction (only for wind loading as reported in Appendix B)
- Varying traffic loads
- Varying bridge deck stiffness at supports

Another effect investigated, but not reported in detail, is locking the main cables to the bridge deck in the centre of midspan.

Rotational stiffness of the pontoons

The rotational stiffness of the pontoon is investigated through varying the size of the pontoon. Dimensions $\varnothing 75 \times 150$, $\varnothing 75 \times 175$, $\varnothing 75 \times 225$ and $\varnothing 100 \times 175$ are tried out. Sensitivity with respect to deflection of bridge deck and load transfer at the top of the towers for traffic loading are investigated. By increasing the rotational stiffness by a factor of 1.68 corresponding to increasing the dimensions from $\varnothing 75 \times 175$ to $\varnothing 75 \times 225$, deflection due to traffic is reduced by app. 15%. A reduction to $\varnothing 75 \times 150$ increases the deflection by 6% to $L/155$. Corresponding friction loads at top of piers increases by 17% and reduces by 13%. Corresponding rotation of the pontoon is 0.5degrees, 0.33degrees and 0.66degrees.

It is therefore concluded that a $\varnothing 75 \times 150$ is feasible with respect to traffic loading requirements.

Axial spring stiffness of the end of the bridge deck

At the bridge deck border, external post tensioned cables are used to create a spring that keeps the bridge in position. Different spring stiffnesses of 1MN/m, 5MN/m, 15MN/m, 30MN/m and 50MN/m are investigated.

For traffic loading in centre span, the effect of spring stiffness is minor due to symmetric loading, while for traffic loading in side span deflection will increase rapidly if the spring is not stiff enough. At zero stiffness this deflection is app. 8m. Increased stiffness reduces the deflection, however when stiffness increase above 15MN/m only minor reduction in deflection occur. This level is therefore chosen as a basis for the design.

For temperature loading of +36 degrees, longitudinal deflection at abutment is 0.9m for zero stiffness, 0.73m for stiffness 15MN/m and 0.62m for stiffness 30MN/m. Corresponding force in the spring is =0MN, 11MN and 18MN. Chosen stiffness of 15MN/m gives satisfactory results with respect to deformations and stresses due to temperature.

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Spring stiffness of anchor system in longitudinal direction

Stiffness in longitudinal direction is varied between 0MN/m, 0.5MN/m, 1MN/m and 2MN/m.

Traffic loading and traffic do not affect the response within these values. The reason is that the anchor stiffness is much less than corresponding stiffness from tower and bridge deck.

Spring stiffness of anchor system in transverse direction

This variation is only affected by wind loads and is examined in Appendix B.

Varying traffic loading

Testing of increased traffic loading is performed. The vertical deflections are almost proportional to the increase. This is the case up to 4 times the traffic loading. Looking at the rotational stiffness of the pontoon it is found that an increase from $\varnothing 75 \times 175$ to $\varnothing 75 \times 225$ only decrease the deflection with 15% (From L/83 to L/98 for double traffic loading). Since the pontoon is the most expensive structural part, other measures like increasing deck stiffness and main cable area should also be investigated in case of increased traffic load requirements.

Varying bridge deck stiffness at supports

The effect of increasing the bridge deck stiffness close to the supports at axis 3 and 4 is investigated through an increase of the moment of inertia about horizontal axis by 100% over a 57m length from the axis. The effect of this stiffness change is an increase in the bending moment from uniformly distributed vertical loading by 66%. Similarly, an increase of the moment of inertia about vertical axis give an increase of the effect of horizontal distributed loading of 15%. This implies that such strengthening may be beneficial, especially for wind loading, see also Appendix H.

Locking of main cables to bridge deck in mid span

The introduction of such locking of the cables reduces deflection by 20%. The solutions will however need to transfer large forces, and should be looked at in upcoming phases of the project.

3.3 Simplifications in the analysis

The following simplifications have been done in the analysis:

- Cable mass is underestimated. Used dead weight is 14.1kN/m which correspond to 1.44 tons/m. For mass $m = (0.17\text{m}^2 \times 78.5 \text{ kN/m}^3) = 13.345 \text{ kN/m}$ is used,

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corresponding to 1.36 ton/m. The discrepancy has little effect on the analysis results.

- Anchor lines are stipulated using a horizontal spring (element 4920 and 5920) at elevation -25 at centre point of the pontoon. Since the anchor lines have a vertical component and are attached to the outer walls of the pontoon, rotational stiffness will be underestimated. This stiffness is however only app. 1% of the total value, and is thereby negligible.
- Hangers are modelled as vertical elements 15m apart. The chosen concept has however skew cable planes and is 15m apart only close to the deck, while are only 3m apart at the tower tops. This simplification has little effect on the design of main cables, hangers and bridge deck, but may have larger influence on torsional stiffness of the tower top and thereby friction force between cable and tower saddle for twisting of the tower. This effect should be looked into in the next phase of the project.
- The towers in axis 2 and 5 are modelled as the towers axis 3 and 4, with four legs. In the chosen concept, the towers ashore do however have two legs only. This discrepancy implies a stiffer tower that will attract more forces from traffic. With a less stiff tower, this force (total app. 5MN) will be added to the back cable force. This corresponds to 5% of the total force.
- Steel plate thickness of upper face of the bridge deck is 12mm in the analysis, while the requirement in EU codes is 14mm. The consequence of this discrepancy is believed to be negligible.
- The bearings for horizontal load transfer between bridge deck and tower is modelled to be in the cog. of the bridge deck, giving no eccentricity in the load transfer. In the drawings, these are however placed underneath the bridge deck. The additional stresses this causes will however be small for the chosen steel dimensions.
- The dead weight of the viaducts is not taken into account, nor is traffic on the viaducts. This reason is that these structural parts are not believed to be crucial to feasibility.
- There is some discrepancy between chosen span widths on the viaducts in the model compared to the drawings.

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4 WIND RESPONSE ANALYSIS

4.1 General

A dynamic response analysis for wind loading is performed for the complete bridge system developed in this Sognefjorden Feasibility Study of Floating Bridge. A dynamic structural model is developed based on the static model used for the Global Analysis described in Chapter 3. The analysis is performed in the program RM Bridge V8i 08.10.07.01 and is based on large deformation theory which is mandatory in order to give correct deformations and response in a suspension bridge. Detailed description of the set of analyses is given in Appendix B. This chapter summarize the basis for the analyses and present the major findings.

A set of analyses have been performed. A basic configuration of the bridge system is analysed and results are given in detail. Further a parameter study is performed in order to determine firstly the main load carrying elements for wind loading in this rather new bridge system and secondly how changes in dimensions of these elements influence the response of the system.

A main topic in defining the basis for the analyses has been the determination of the wind environment to be used. It is obvious that the response of this bridge, in particular in the cross-wind direction, will be quite different from other suspension bridges. The extreme long natural periods for response in the cross direction, first natural period is 169.5 seconds, will influence the response and thus must be included in the evaluation of the design wind environment. Deep water floating offshore production platforms are examples of structures having similar natural periods and similar wind generated response. A review of the wind environment used for this kind of structures was performed in the early phase of this work. This review revealed that the differences in surrounding topography must be more essential for the definition of wind loading, than the similarities in response periods. The bridge here is situated in a rather closed fjord landscape with steep hills surrounding the location and thus giving a source for turbulence that dissolve the large eddies that will be present in the large open areas in open sea. The conclusion was thus that until more detailed measurements of the coherence of the wind in this fjord landscape are performed, the up to now used description of the wind environment for design of vibration sensitive bridges in Norway should be used.

As a result the wind environment used is taken from the appropriate standards for bridge design without any major changes. The only major change introduced is to use 1 hour mean for the static mean wind instead of 10 min mean which is the common procedure. Both energy spectra and coherence spectra are taken as specified in Norwegian standards and the parameterisation is based on the design specification for the Hardanger suspension bridge.

Shape factors for the cross section of the bridge beam are taken from other relevant projects, from available wind tunnel investigations and from appropriate standards. Frequency dependant factors are not included. Quasi-static aerodynamic derivatives have been used.

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4.2 Calculations

The calculation of response is as common for these analyses, separated into a mean part and a variable part, i.e. a static part and a dynamic part.

The mean part of the response is in principle the result of a static analysis where the mean wind generates a distributed load on the beam elements of the structural model. This distributed load is a function of the wind velocity and the shape factor for the cross-section and is thus independent of the structural response.

The variable part of the response is calculated based on a buffeting wind response analysis. This is a stochastic modal analysis and the response is thus dependent upon the structural response and the natural frequencies of the structure. The dynamic analysis is performed as a mode by mode analysis, where the standard deviation of total response is found by SRSS summation of the contribution from each mode. The extreme value of the response is then found from extreme value statistics for this response. This procedure has been the common design procedure for suspension bridges of similar span width.

NO	OMEGA	HERTZ	T
1	0.037	0.0059	169.5
2	0.039	0.0062	161.3
3	0.068	0.0109	91.7
4	0.081	0.013	76.9
5	0.209	0.0332	30.1
6	0.215	0.0342	29.2
7	0.226	0.0359	27.9
8	0.239	0.038	26.3
9	0.24	0.0382	26.2
10	0.246	0.0392	25.5

Table 4.1: Natural frequencies and periods

First mode contribute by 87% of the cross response from the wind and have an eigenperiod of 169.5 seconds. As can be seen from Table 4.1, the natural frequencies are rather close for period 1 and 2 and for periods 5 to 10. A further development of the used procedure would have been to perform a multi-mode calculation in order to include the effect of cross-mode response in these areas. The calculation tool, RM Bridge, have not yet implemented this procedure. Such an analysis would have given valuable additional information but it is not believed that the results would change by any significance.

A further modification of the performed procedure would have been to include additional damping for the first eigenmodes. Some of these modes are based on large pontoon movements giving source of fluid damping that are not included in the present analyses.

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The analysis model used in these reported analyses, has not implemented the needed reinforced cross-section of the bridge beam towards the towers. Utilization of the bridge beam towards towers is thus rather high for these results. This is evaluated in the design part of our work where the wind analysis is rerun. For more detailed information, see Appendix H.

4.3 Results

Detailed results are given in Appendix B. Main results are as follows (static plus dynamic response for wind):

Max bridge beam deformation in cross direction	13.55 m
Max static deformation in cross direction	8.50 m
Max cable force	7 996 kN
Max tower deformation in cross direction	11.46 m
Max tower-top tilt	0.96 degree
Max tower-top tilt from static wind	0.36 degree
Max pontoon deformation in cross direction	10.19 m
Max anchor stay force - in cross direction	20 379 kN
Max anchor stay force – in-line direction	526 kN
Max static anchor stay force - in cross direction	9 360 kN

Analysis is performed based on structural and aerodynamic damping alone. Structural damping is set to 0.5 % of critical damping and aerodynamic damping is calculated directly based on the movements of the structure in the wind field.

Accordingly, damping generated by the structural movements of the pontoons in the sea is not included. Such damping may be considerable if the movements are large and have a sufficient velocity. The movement may in any case be used to achieve the damping needed by adding vertical ribs on the lower outside of the pontoon. Guidance note in DnV standard DNV-S-E301 propose 5 to 10% of critical as relevant damping levels for sway movements of moored floating production structures. As this increased damping only is relevant for the first few modes, a general increase is not feasible. Increased damping from 0.5 to 5 % for mode 1 alone, reduces the cross direction dynamic wind generated movement from 5.1 m to 1.5 m. Bending moment about the vertical axis reduces from 353 MNm to 21 MNm. This exercise indicates the sensitivity of the response for damping. As a conclusion one may say that the calculated response related to cross direction movements is far on the conservative side considering both the effect of damping and the effect of the wind environment.

Aerodynamic stability

Aerodynamic stability should not be any major challenge for this concept as the span width for the bridge is quite similar to bridges already built. The major challenges for this concept are the effect of multi span and the floating foundations for the towers. Accordingly the vertical and torsional modes for this concept should be quite similar to suspension bridges already

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build or under construction in Norway today and conditions for aerodynamic stability for flutter should be quite similar.

Documentation of aerodynamic stability is according to the regulations a rather extensive work that must be based on wind tunnels investigations. Such investigations are not possible within this feasibility study due to limitations both in time and in resources. Documentation within the feasibility study is therefore based on simplified procedures originating in the formulae's developed by Prof. Selberg.

The detailed calculations are reported in Appendix B, here only a summary of the results are given. Critical flutter velocity calculated according to Selberg is 64.3 m/s. Velocity at the bridge beam level is 36.9 m/s for a return period of 50 years and 41.3 for a return period of 500 years. Based on ref. /1/, the 2011 edition of the Hb 185, this give a limiting velocity of 66.1 m/s and thus above the calculated critical velocity based on the simplified formula acc. to Prof. Selberg.

This limiting velocity is far more conservative than what has been used earlier, e.g. the 2006 edition of the Hb 185, and far more conservative than used for ongoing building projects like the Hardanger Bridge. Based on these regulations the limiting velocity would have been 55.3 m/s.

Further is the experience from earlier suspension bridge projects that a calculation of the critical flutter velocity based on wind tunnel investigations and measured aerodynamic derivatives, give a more correct and higher value for the critical flutter velocity. At this stage of the project the needed data for such calculations are not available.

Based on our long experience with suspension bridges we feel that this problem do not influence the feasibility of the concept. This subject was as stated in the start of this section, not believed to be any particular problem for this particular bridge and this we feel is still the case. A more detailed study, including some of the measures mentioned above, will in our opinion show the needed safety for aerodynamic stability of the concept.

4.4 Parameter study

In Appendix B (Chapter 8) a parameter study of the dynamic behaviour is documented. The study is focused on:

1. to determine the main load carrying elements for wind loading
2. to determine how changes in dimensions of these elements influence the response of the system

The study documents effects in wind response of changes in anchor stiffness for pontoons, depth of pontoons, axial posttensioning of bridge beam and continuity/discontinuity of bridge beam.

A large number of different setups of the structural model are investigated. A more detailed description is given in Appendix B; see in particular the list in Section 8.2 of appendix.

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Main load carrying element for cross wind is the anchor stays. Continuity or discontinuity towards viaduct have little influence on the deformation of the bridge and force in main suspension cable but clearly influence the stresses of the bridge beam.

Static wind is source of between 35% and 65% of translation Vz and rotation Rx of pontoon and top of tower. It is mode 1 to 10 that gives the main contribution to pontoon translations and to bending moment in the bridge beam and mode 1 alone contributes with 87 % of the cross wind translation of the pontoon. Forces of suspension cable and the torsional moment in the bridge beam are governed by higher modes.

The increased depth of the pontoon reduces the cross wind translation of the tower top and stabilizes the tower for tilt, but have no particular influence on the translation of the pontoon at sea level. In these calculations only the mass effect of the surrounding fluid is included, but this is the major effect of an increased depth of the pontoon.

Some recommendations for the concept regarding dynamic behaviour may be given based on the performed investigations. The recommendations are given regarding the dynamic behaviour and thus the final conclusions may be different. Based on this the following items may be noted:

- Continuity towards viaduct may be omitted
- The deep pontoon is preferred in order to limit the tilt behaviour

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5 WAVE AND CURRENT RESPONSE ANALYSIS

5.1 General

In this chapter the effect of wave and current loading on the proposed concept for the floating bridge is evaluated. This evaluation is performed separately and only included into the global analysis as equivalent static loading if appropriate.

In reference /3/ an evaluation of wave and current for the bridge location is given as input to the feasibility study from the client. This report do not define the complete loading environment for wave and current, so the data given in the report is supplemented with information taken from the Høgsfjord project and experience within the design group from other typical Norwegian fjords.

Wave loading may be generated from local wind generated waves within the fjord basin, from degenerated remains of ocean waves entering the basin through the mouth of the fjord or from landslide generated waves from a possible landslide occurring at Stampa, nearby Flåm, in the inner part of the fjord system. Effects of both first-order waves and second-order drift waves are estimated for the concept. Further is the effect of finite water depth estimated for the long periodic landslide generated waves by including calculations with 700 m water depth in addition to infinite water depth.

Wave phenomena as internal waves are not included in the study, as the conclusions from the Høgsfjord project was that such phenomena not will define a critical combination. The reason for this is that internal waves only will have conditions for occurring during the summer season and at this part of the year the other environmental loadings to be combined with is much smaller than during the winter season. Further are waves from passing ships also excluded from the study, as these not will be able to excite the large pontoons.

Current will generate both direct loading and possible vortex excitations. Both effects are evaluated.

Detailed information of the calculations performed may be found in Appendix C. In this appendix one also may find the basis for the performed hydrodynamic evaluation as this was developed during discussions with Professor O.M. Faltinsen, both in meetings and on email.

5.2 Wave and current loading

Current data taken from ref. /3/ is given in Table 5.1. Only values to 75 m depth are given in ref. /3/. Figures for larger depths are extrapolated from the given data.

In Table 5.2 and 5.3 the wave parameters used in the study are given. These are based on the values given in report 11258-01, Project Basis, which is accepted by the client.

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Depth (m)	Velocity, u (m/s)
0-10 m	1,25
30 m	0,70
75 m	0,45
100 m and below	0,40

Table 5.1 – Current data having a 50 year return period

Wind generated waves in basin			
Return period (year)	H _{mo} (m)	Tp (s)	γ
1	1,1	3,0-4,5	2-4,5
10	1,6	3,2-4,7	2-4,5
50	2,1	3,6-5,1	2-4,5
Swell from ocean waves			
50	0,1	10-15	7
Landslide generated waves – from Stampa, Flåm			
10000	0,3	80-90	7

Table 5.2 – Wave data from ref. /3/

Return period (year)	H _{mo} (m)	Tp (s)	γ
50	2,1	5,1	3,3
50	0,1	14	7
10000	0,3	85	7

Table 5.3 – Wave parameters used in study

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5.3 Mass- and stiffness of the pontoons

In Table 5.4 mass- and stiffness values are shown for different depths of the pontoon. A depth of 175 m is chosen for further studies.

Pontongdyppgang (m)		150	175	225	250
Translasjons- masser	M _P	0,65	0,74	0,95	1,05
	AM _S	0,60	0,71	0,91	1,05
	AM _H	0,12	0,12	0,12	0,12
(x10 ⁶ t)	∑M _S	1,25	1,45	1,86	2,10
	∑M _H	0,77	0,86	1,07	1,17
Tyngde- punkt (m)	y _P	-101,7	-121,0	-157,0	-175,9
	y _{AM}	-68,5	-78,8	-103,5	-118,0
	∑y	-85,7	-100,3	-130,8	-147,0
Rotasjons- treghet om hor. akse (x10 ⁹ tm ²)	I _P	1,6	2,18	3,96	5,5
	I _{AM}	1,2	1,1	2,7	7,5
	∑I	2,55	3,9	8,0	11,1
Rotasjons- treghet om sylinderaksen (x10 ⁹ tm ²)	I _p	0,46	0,52	0,67	0,74
Rotasjons- stivhet (x10 ⁶ kNm/rad)	k _φ	184	259	436	553
Stivhet i heave (x 10 ⁴ kN/m)	k _h	5,00	5,00	5,00	5,00

Symboler:

Indeks P: Pontong
AM: "Added mass"
Indeks H: "Heave"
Indeks S: "Surge"
∑: Pontong og "added mass"
Rotasjonstreggheter er oppgitt om enkeltbidragenes tyngdepunkt, og totaltregheten er oppgitt om felles tyngdepunkt.
Stivhet i "heave" er regnet ut fra en diameter i vannlinjen på 80 m.

Table 5.4: Mass and stiffness data for different pontoon depths

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5.4 Response from wave loading

Wave generated responses are calculated using the wave loading program Wamit and the load response program Mimosa based on the wave parameters defined in Table 5.3.

A more detailed description of the calculations is given in Appendix C and an overview of the response results is given in Table 5.5. Here horizontal movement in the cross direction is given as surge, rotation about the bridge axis is given as pitch and vertical movement as heave. The values given are extreme values based on a 3 hours storm condition.

Small movements are found in general. Response from wind generated waves and swell are both negligible. It is only the dynamic response from second-order drift waves that give any significant response.

Generated by:	H _{mo} (m)	Tp (s)	1st order wave response			Wave drift response		
			3 hours extreme			3 hours extreme		
			Surge (m)	Heave (m)	Pitch (deg)	Surge (m)	Heave (m)	Pitch (deg)
Wind	2,1	5,1	0.04	0.00	0.01	2.13	0.01	0.30
Swell	0,1	14	0.09	0.00	0.04	0.01	0.00	0.00
Landslide	0,3	85	0.35	0.36	0.11	0.00	0.00	0.00

¹ 1ste order heave = 0.36m

Table 5.5 – Extreme values of response for surge, pitch and heave– 1st order waves and 2nd order drift waves

	Surge	Surge	Heave	Heave	Pitch	Pitch
	RAO	Increase	RAO	Increase	RAO	Increase
	(m/m)	(%)	(m/m)	(%)	(rad/m)	(%)
Infinite WD						
40 sec	0.37	-	1.20	-	0.0045	-
85 sec	1.07		1.01		0.0015	
200 sec	3.40		1.00		0.0025	
700 m WD						
40 sec	0.40	1.06	1.23	1.02	0.0047	1.05
85 sec	1.83	1.72	1.02	1.01	0.0025	1.71
200 sec	12.98	3.82	1.00	1.00	0.0094	3.82

(RAO: Response amplitude operator)

Table 5.6 – Comparison of response – finite and infinite water depths

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Table 5.6 indicate the effect of performing the calculations with a finite water depth of 700 m. This is only relevant for the landslide generated wave as this is the only wave that will reach to this depth. Even with the rather conservative assumption of 700 m depth (maximum depth at the location is 1250 m) the pontoon will move only 0.6 m. As this is an exceptional event, this will cause no problems.

In addition to the dynamic drift response given in Table 5.5, there will be a mean force from the wave drift. This force is calculated to 210 kN based on a formula given by Prof. Faltinsen in Appendix C.

5.5 Response from current

Based on the current profile given in Table 5.1 and a drag coefficient of 0.76 (calculated as given by Prof. Faltinsen in Appendix C) a current force with a return period of 50 years is:

$$F_s = 2160 \text{ kN}$$

Current will also give cause of vortex shedding that may generate both in line and cross flow vibrations. Cross flow vibrations may reach amplitudes of 1.3 times the diameter of the structure while in line vibrations are limited to 0.15 times the diameter. A vital parameter is thus the reduced velocity:

$$v_{red} = v_s / (f \times D)$$

where v_s is the current velocity, f is the natural frequency to the structure and D is the diameter. Cross-flow vibrations are likely to occur if v_{red} is larger than 3 and in-line vibrations may occur for v_{red} in the area of 1-4.5, ref DNV Guidelines No. 14.

For this particular structure the diameter is 79 m. Based on the current velocities given in Table 5.1, the natural period must be above $T = 190$ s in order to have large cross-flow vibrations. Natural periods for our structure for such cross-flow movements are:

$$T = 87 \text{ s and } T = 72 \text{ s}$$

and thus no danger for such vibrations.

To have large in-line vibrations the natural periods should be between $T = 63$ s and $T = 889$ s. The pontoon move in surge with a natural period of $T = 167$ s and in pitch with a natural period of $T = 28$ s. Thus the large current velocity in the upper part of the depth will have conditions where such vibration may occur. But the current velocity over the larger part of the pontoon will be far from having such conditions and cancelation effects are likely to stop such vibrations.

As a conclusion, vortex shedding phenomena are not likely to occur for the pontoon. A potential danger has been identified for small in-line vibrations of the upper part of the pontoon that must be studied at a later stage. If it is found that current may give possible cause of vibrations, measures to rectify will be introduction of spoilers.

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6 DESIGN OF THE ANCHOR SYSTEM

6.1 General

The anchor system for the pontoons consists of stay cables anchored to the seabed and to the pontoons. The anchor system is used for achieving lateral stiffness only and thus limits the movements for environmental loading. The rotational stiffness required is achieved by the floating stability of the pontoon. The bottom end of the anchor line is fixed to the seabed by suction anchors (or suction caissons), see Chapter 8. This type of foundation which is developed for the oil industry, have been installed in many of the deep water oil producing areas around the world.

In this section and in Appendix D, the system is described and a summary of performed design calculations are given. Further is the flexibility of the system to absorb variations in the bottom topography described.

6.2 Anchor system

Anchor cables used for permanent deep water oil producing installations usually consist of different material segments:

- Bottom foundation, usually a suction caisson or anchor or similar
- Bottom chain, from the bottom anchor up to a certain level above sea bottom
- Anchor line, usually locked coil steel spiral strands for subsea use, but composite material could also be used for this part
- Top chain, in the upper area close towards the pontoon
- «Fairlead», system for guiding the top chain to the top of the pontoon and giving the possibility to tensioning the system

A more detailed description is given in Appendix D.

6.3 Design

In Appendix D two principle different systems are designed in order to determine the possible dimensions of such a system for the bridge. As the basis for this investigation is to determine the possibilities to cross the Sognefjord prior to any detailed investigations of the topography of the sea bottom, different solutions have been investigated in order to have a robust solution that may absorb variations in the bottom topography.

The basis for the selected system is that all foundation points are located in the flat part of the fjord at approximately 1300 m depth. Based on the assumed simplified topography, ref. Section 1.3, the system will be as given in Figure 6.1.

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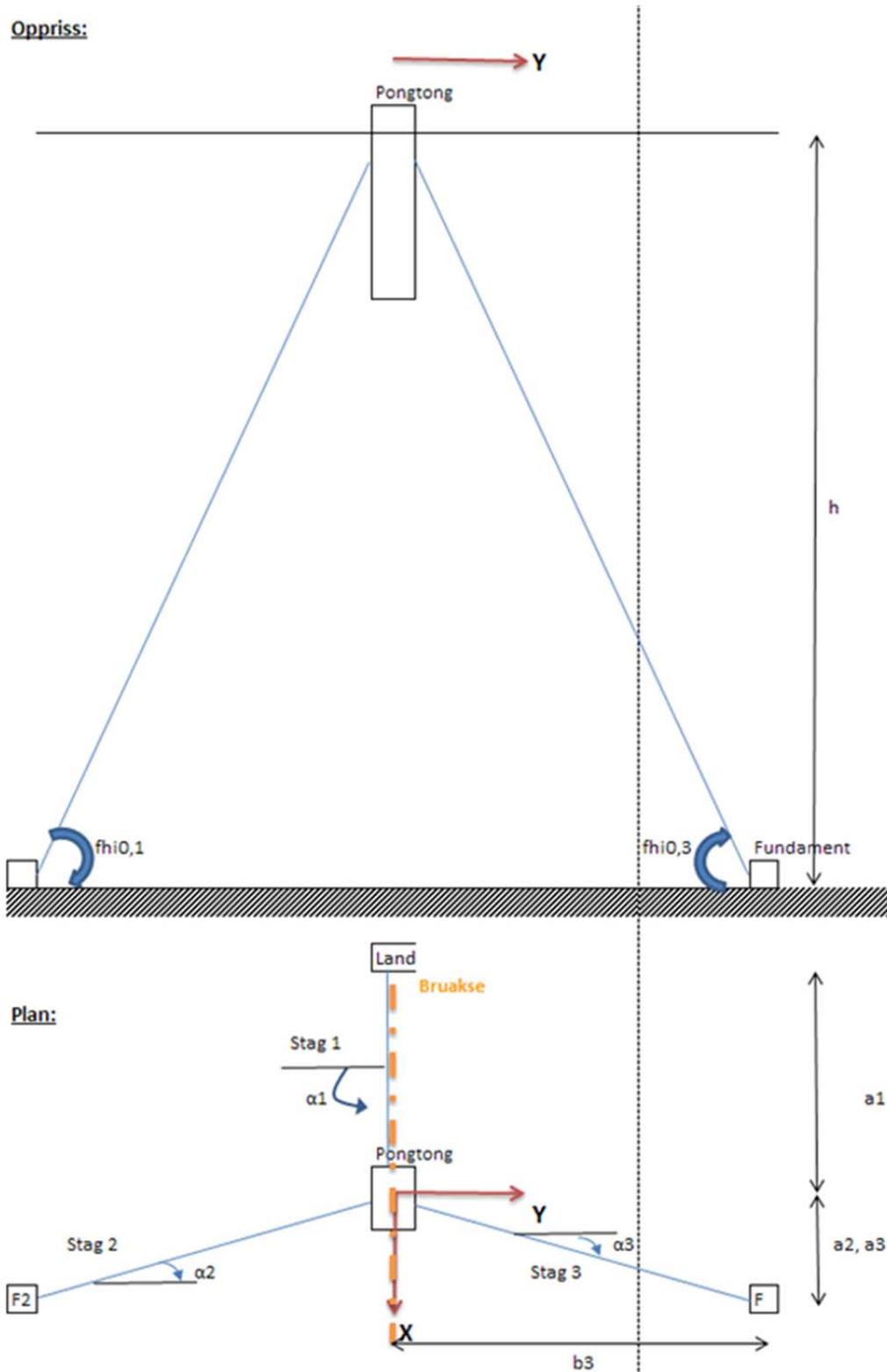


Figure 6.1: Layout of anchor system

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This is an asymmetric system having one horizontal "stay" towards shore and two inclined "stays" towards the fjord bottom. The notion "stay" is here used for a group of anchor lines having approximately the same direction. The lines in one "stay" must likely have a minimum spacing in order to avoid vortex shedding problems.

The selected system is described in detail in Appendix D. Each "stay" to the bottom consists of 8 lines (giving a total of 16 lines from each pontoon to the fjord bottom) and the horizontal "stay" towards shore contains 2 lines.

Each of these anchor lines has the following properties:

Cross-section:	$A_s = 13\,846\text{ mm}^2$
Weight:	$g = 100\text{ kg/m}$ submerged, i.e. 1 kN/m
E-modulus for line:	$E_0 = 1.5 \cdot 10^5\text{ MPa}$
Maximum breaking load	$MBL = 22.07\text{ MN}$ (Bridon Spiral Strands)

In a vertical section through the pontoon and the bottom foundation, the line is supposed to have an angle of 45° degree towards the sea bottom and this section form a horizontal angle of 70° degree towards the bridge (i.e. angle α in Figure 6.1 is 20°). Each line does then have a utilization of 46 % of MBL.

Positioned into the available digital model of the fjord, one of the anchor positions do not end up in the flat part of the fjord bottom as requested. This may easily be solved by changing the horizontal angle in one or both of the systems so that all four anchor points end up in the flat part of the fjord. In Appendix D the effect of this change is investigated and the increase in utilization of the anchor lines is marginal, from 46% to 49.6 %.

6.4 Results

Chosen system:

- Bottom fixation by suction anchors, see Chapter 8
- Bottom connection by chain, from suction anchor to some distance above sea floor, i.e. Vicinay Studless chain R4S diameter 152 mm and MBL 22 363 kN
- Anchor lines, i.e. Bridon Spiral Strands with $A_s = 13\,846\text{ mm}^2$ and MBL 22.070 kN
- Top connection by chain, as bottom connection but with corrosion allowance, i.e. Vicinay Studless chain R4S diameter 175 mm (20 mm corrosion allowance)
- «Fairleads», designed to the selected chain dimensions

From each pontoon one then have 16 anchor lines to the fjord bottom and 2 lines towards shore, i.e. 32 lines to fjord bottom and 4 lines to shore.

This give:

Anchor line:	58 810 m
Chain 152 mm	1 200 m
Chain 175 mm	<u>1 620 m</u>
Total	61 630 m

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7 DESIGN FOR SHIP COLLISION

7.1 General

At the bridge location there is a large amount of ships passing the proposed bridge. The type of ships includes large cruise ships, tankers, cargo ships and high speed passenger crafts. A total of 3300 large vessels pass the actual area yearly. These passing ships represent a potential risk for impact loading on the pontoons.

A separate risk assessment study of the vessel traffic in Sognefjorden has been performed, see ref. /12/. This study should deliver a decision basis for Statens Vegvesens' evaluation of marine safety and navigational arrangements with regard to the bridge design of a possible new bridge crossing Sognefjorden. As such, this study should then define the complete basis for our feasibility study regarding design for ship collision.

Based on the available information, three events regarding ship collision have been identified and investigated as a part of this feasibility study. These are as described in report 11258-01, Project Basis, which has been accepted by the client.

The ship collision events are analysed by the program USFOS which is a program capable of handling in-elastic impacts in a proper manner. In the simplified USFOS-model the structure is represented by its relevant inertia and stiffness properties. The bridge superstructure is only represented by its mass contribution. In the USFOS analysis, the impact force and deflection/velocity/acceleration time histories are determined. These are in the next step put as loading on the global RM-Bridge model (as described in Chapter 3 and 4) in order to perform a dynamic time step analysis to determine the effects of the impact on the superstructure.

7.2 Ship collision events for the bridge

The bridge structure must be checked for the loading caused by a ship collision.

Based on the risk assessment study three events regarding ship collision have been defined and are investigated:

1. A bow collision centrally on a pontoon where the ship is sailing orthogonal to the bridge axis
2. A bow collision centrally on a pontoon where the ship is sailing with an angle towards the bridge axis equal to the main ship route (i.e. 78° to the bridge axis)
3. The bow impact of item 2 hits eccentric on the pontoon

These events should reflect all ship collision events having a higher probability than 10⁻⁴.

Impact from drifting ships is disregarded in this feasibility study as the impact energy of these events is far below the investigated events.

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The following impact event is defined:

Ship data:

Length: 208 m
Width: 29 m
Depth: 8,8 m
Displacement: 31456 t

Speed at impact: 17,7 knop (9,1 m/s)

An "added mass"-coefficient of 0,2 for bow impact is given in ref/12/.

This gives a kinetic energy at the impact moment of 1563 MNm.

Connection area for the ship towards the pontoon is assumed as: 5x5m (not given)

The load- indentation curve given in Figure 7.1 is defined from the risk assessment study, see addendum to ref/12/. The curve for 40000 DWT-ship, marked b, is used in the calculations performed in the study.

For the eccentric bow collision, a part of the energy will be lost in sliding towards the pontoon. With an impact angel of below 28° is the transferred energy reduced to approximately 24 %, while the transferred energy is about 66 % at 50°. For a circular pontoon of 79 m diameter, this is equivalent to an eccentricity of 35 and 36 m. Only the last condition is further analysed as this is evaluated as most critical condition.

Ship impact is an accidental limit state and the pontoon must survive with two cells filled with water. Impact below -25 m is assumed as an event having a lower probability than 10^{-4} and is thus not evaluated. The structure must be equipped with hydro-acoustic equipment that warn any possible submarines sailing in this area.

- A 150,000DWT bulk carrier at an initial speed of 9,3 m/s
- A 40,000DWT container ship at an initial speed of 12,9 m/s

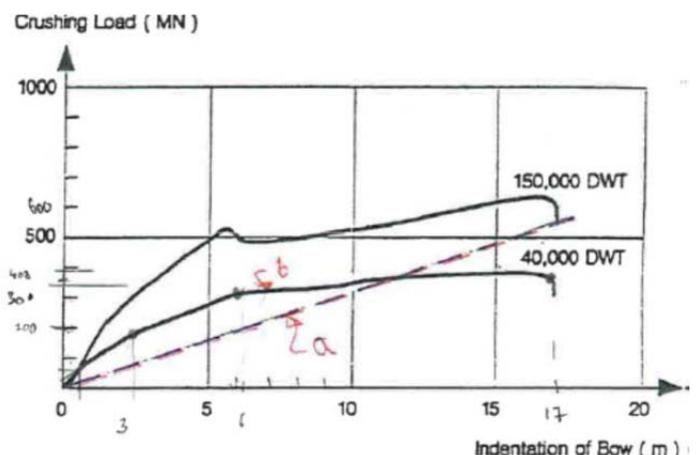


Figure 7.1: The load- indentation curve – given in addendum to/12/

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7.3 Analyses

The ship collision events are analysed by the program USFOS which is a program capable of handling in-elastic impacts in a proper manner. In the simplified USFOS-model the structure is represented by its relevant inertia and stiffness properties. The bridge superstructure is only represented by its mass contribution. In the USFOS analysis, the impact force and deflection/velocity/acceleration time histories are determined. These are in the next step put as loading on the global RM-Bridge model (as described in Chapter 3 and 4) in order to perform a dynamic time step analysis to determine the effects of the impact on the superstructure.

For the ship collision event, analyses have been performed at three levels:

1. Dynamic in-elastic impacts analyses on simplified model using USFOS
2. Dynamic global response analyses for the bridge using RM Bridge using the impact time history from USFOS analysis as loading
3. Hand calculations based on the principle of conservation of momentum for checking the USFOS analysis

The analyses are shortly described in the following, see Appendix E for further description.

1. In the USFOS-analysis the pontoon is represented as an infinite stiff cylinder with inertia represented in surge and pitch directions. The inertia includes both structural mass (for the pontoon and the superstructure) and hydrodynamic mass. The frequency variation of the hydrodynamic mass is not included as the program is not able to represent this feature. This variation is on the other hand small (only 12% in surge and 2.5 % in pitch) and the error induced is thus limited. The ship is modelled as point mass (ship mass plus 20 added hydrodynamic mass) with an incoming velocity as defined. Between cylinder and ship an inelastic, non-linear spring is introduced, the spring having the defined load- indentation curve. For the phase after the collision, the force in the spring reduces rapidly to zero.

The stiffness of the superstructure for deformations in the transverse direction is small compared to the stiffness of the anchor system. This stiffness is thus neglected. Accordingly is the stiffness in the transverse direction represented by a spring representing the anchor system (2 MN/m) at level -25 m. For deformation in the bridge direction, the stiffness of the superstructure will be noticeable (15-30 MN/m). For collision not perpendicular to the bridge axis, this stiffness is represented at the level of the bridge beam (+ 75m).

The present version of USFOS was not able to handle the effect of eccentric collision including friction, sliding and impact. This event is hand calculated, see below.

2. The global dynamic analysis model in RM Bridge (ref. Chapter 3 and 4), is used to calculate the response time history for the bridge exposed to the impact time history calculated by USFOS. The impact time history is given at the waterline of one pontoon. From this the response history of the complete bridge system is calculated in same the model as used for other global loadings.

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This split of the analysis was needed as the present version of RM Bridge was not able of calculating the in-elastic impact. The split do not represent any reduced accuracy of the results as the effect of the superstructure is negligible on the impact development during the 1.5 s the impact lasts. In addition, the mass of the superstructure is small compared to the mass of the pontoon and would as such not contribute to the interaction between ship and pontoon at any noticeable degree even for a longer lasting impact.

3. Hand calculations are based on the principle of conservation of momentum. This principle states that if you have two objects that are moving toward each other, or one object explodes into two different objects, at all-time the sum of the linear momentum of the objects will give a constant number. Hence the principle of conservation of linear momentum states that in a close system the sum of linear momentum is constant. Then in any collision the sum of linear momentum of the objects before the collision would be equal to the sum of linear momentum after the collision i.e.:

$$m_1 \times v_1 + m_2 \times v_2 = m_1 \times v_1' + m_2 \times v_2'$$

where the left side represents the linear momentum before the collision and the right side after the collision.

In our case the system may be regarded as a closed system based on that the duration of the collision is short and that the movements of the pontoon and thus the outer force from the anchor system is negligible during the collision itself. Further one may assume a perfectly inelastic collision (i.e. both bodies have the same motion afterwards) based on that the mass of the ship is small compared to the mass of the pontoon. This give:

$$v_2' = m_1 / (m_1 + m_2) \times v_1$$

where m_1 is ship mass and m_2 is pontoon mass, velocity of pontoon before collision, v_2 is assumed to be zero.

The energy lost in the collision ΔE (mainly due to plastic deformation of the ship) is equal to the difference between the kinetic energies before and after the collision:

$$\begin{aligned} \Delta E &= \frac{1}{2} \times m_1 \times v_1^2 - \frac{1}{2} (m_1 + m_2) \times v_2'^2 \\ &= \frac{1}{2} m_1 / (1 + m_1/m_2) \times v_1^2 \end{aligned}$$

This show that the relation between the masses m_1 and m_2 govern the energy lost in the collision.

The hand calculations may then directly be used to determine the velocity of the system at the end of collision. Further this give the kinetic energy of the vibrating system after the impact and the energy lost in the impact. Based on a defined load- indentation curve, the impact force and the indentation of the pontoon into the ship may be calculated and maximum acceleration/retardation may be determined. Deformations may be estimated based on the start velocities or alternatively estimated based on the assumption that all kinetic energy will be stored as elastic energy in the anchor system.

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7.4 Results

1. The USFOS- analyses give results for an assumed infinite stiff cylinder as:

- Impact force and its time history
- Displacement, velocity and accelerations on chosen location, i.e. of the pontoon, at anchor connection (level -25m), and at superstructure (level +75m)

Detailed results and time histories are given in Appendix E. Here only a summary of results are given. The maximum value of the impact force is about 350 MN and the impact last about 1.5 s. Maximum translation of the pontoon is about 6 m and maximum rotation about 2.0°. This give a maximum displacement of 8 m at anchor level, 9 m at waterline and 11.5 at bridge beam level. Velocity and accelerations at bridge beam level have a maximum value of 1.8 m/s and 0.4 m/s².

Maximum retardation of ship is 9 m/s², i.e. 0.9 g.

2. The RM Bridge analysis do show considerable vibration of the superstructure. In Appendix E time histories for selected response values are given. Both perpendicular collision and angular collision, ship following the main route direction at 78°, are investigated. The effect on utilization of the superstructure is shown in Appendix H.

For this type of response, the actual damping of the system is of major importance. For large volume structures and this type of movements the hydrodynamic damping may be rather low. In order to increase this damping, adding of vertical ribs at the outside lower parts of the pontoon may give the needed damping from the modification of the flow around the cylinder. Utilizing the lower 30-50 m of the pontoon may give the needed additional damping without increasing other loading significantly.

Analyses are performed with 1%, 2% and 5% relative damping in the frequency area for the pitch movement. These show a considerable effect of the damping size. More investigations must be performed to quantify achievable damping including both analytical investigations and model tests. A relative damping of 2 % seems achievable now.

3. The Hand calculations show quite similar results as the USFOS analysis as one may see in Appendix F for the centric impact.

For the eccentric impact the same calculations may be performed using the conservation of momentum for translation, rotation about horizontal axis and rotation about vertical axis. The given input from the risk assessment study, ref. /12/, is rather insufficient for the eccentric collision. A load- indentation curve for such a condition is not given. This curve should be quite different from the curve reproduced in Figure 7.1. The only input given is a suggestion on how much of the kinetic energy of the ship that is assumed to be transferred to the pontoon. An eccentric collision will involve changes in the ships translational and rotational energy which both depends on the mass of the ship, the shape of the ship, the shape of the pontoon,

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and the additional hydrodynamic mass. To get a reasonable correct representation of these items, a rather comprehensive amount of calculations should have been performed as a part of the risk assessment study. As such information is not available, the calculations of the eccentric collision must be based on simplifications and an extrapolation of the procedure used for the centric collision.

The eccentric collision is based on the following assumptions:

- The post impact velocity of the ship is determined by the difference in kinetic energy prior and posterior to the impact based on the figures given by the risk assessment study
- The rotation of the ship is neglected
- The ship will have the same eccentricity posterior to the impact

For the figures in our case the ship will thus have a posterior velocity of 5.3 m/s (based on an incoming velocity of 9.1 m/s) and the maximum rotation of the pontoon is:

$$\alpha = \underline{1,6}^{\circ}$$

not including any dynamic effects from the superstructure.

The calculations show that 95 % of the energy that supposedly will be transferred to the pontoon after the collision, according to ref. /12/, will be taken as plastic energy in the ship and only 5% will be transferred as kinetic energy to the construction. The energy giving tilt is then now only 20 % of the same energy from the centric collision. This was the part giving the large dynamic effects for the centric collision. One do therefore assume that the vibrations of the super-structure is reduced to 20 % of the centric collision. These are the load effects for the superstructure that should be combined with the effect from the pontoon tilt.

These results are rather uncertain as they are based on limited information on the behaviour of an eccentric collision and the calculations are rather crude as appropriate calculation tools and information is lacking.

An alternative more conservative calculation is based on that the ship is having a velocity which equals that kinetic energy that is supposed to be transferred to the pontoon and that the ship stops completely at the collision. This gives the following rotation:

$$\alpha = \underline{3,0}^{\circ}$$

7.5 Possible measures to reduce risk

If ship collision at a later phase should give rise of substantial problems the following measures should be considered:

Increase of damping:

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This could be very effective, since the calculations show that the most severe load effects for the superstructure appear quite some time after the impact due to build-up of vibrations effects.

The hydrodynamic damping could be increased by adding vertical fins at the outside lower parts of the pontoon. Size and shape should be investigated by theoretical investigations and models tests.

The anchor system will also contribute to the damping of the movements of a ship collision. This effect should also be investigated at a later stage.

Artificial dampers could also be installed e.g. in the central part of the pontoon. Mechanical devices like pendulum dampers have been used in high rise buildings and towers and could be used here as well.

Fender:

As an alternative to the proposed strengthening of the upper part of the pontoon, a fender system could be introduced outside the pontoon in order to protect the pontoons for minor damage and to protect the ship and its crew and passengers in an appropriate manner. This fender system could be fixed to the pontoon or floating outside the pontoon. It should be designed in order to spread the localised impact from the ship before transforming the force to the pontoon. For the given ship collision, the major impact will be on the ship (0.9 g) and not on the bridge. A fender system that could reduce the heavily 0.9 g retardation would perhaps be preferred in order to protect the ship and its crew and passengers.

Speed reduction:

It should be evaluated to introduce speed reduction for the ships upon passage of the bridge site in order to reduce the impact from any passing ship and by that reduce the need for costly strengthening of the bridge.

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8 DESIGN OF SUCTION CAISSONS

8.1 General

A suction caisson or suction anchor can simply be described as an upside down bucket embedded in the marine sediment. For Sognefjord crossing the suction anchors are located in the middle of the fjord, where the soil surface is relatively even. Clay sediments are assumed to be found in this location. The dimensions of the anchor are calculated to be approximately 18m into the soil and with an outer diameter of 6m. The design of the suction anchor is based on the permanent loading being carried by submerged weight of the anchor, while the fluctuating load is carried by skirt friction and suction.

Here capacity calculations are performed for the suction anchors needed to anchor the pontoons of proposed Sognefjorden Floating Bridge in the deep part of the fjord, at approximately 1300 m depth.

A suction caisson or suction anchor can simply be described as an upside down bucket embedded in the marine sediment. Use of such anchors may be described as well tested and well developed technology which has been used for anchoring of different offshore structures in the last 25 years. This technology for anchors has been used for water depths up to 1500 m. The calculations performed herein are equal to what has been used for several other offshore suction anchors designed and documented by NGI.

Appendix F documents the assumptions the design of this particular anchor is based on and the calculations performed in order to determine the actual dimensions this anchor must have in order to withstand the actual loading. The design is based on the assumption that the vertical component of the permanent part of the loading is taken by dead the weight of the anchor while the variable part is taken by a mobilization of the soil strength including mobilization of suction at the bottom of the anchor.

It is further assumed that each of the 8 anchor lines in on stay is supported by one anchor. In order that the capacity of each caisson should not be influenced by a nearby caisson, the distance between each caisson must be equal to the lager of the caissons height or width (typical 20 m for the caissons documented here)

8.2 Possible solutions for actual loadings

Design checks of two different proposals are performed:

- 1) **One wide and shallow anchor** with width approx. 15x15 m and depth 8 m below sea bottom. Such a structure will normally be constructed in steel and consist of 4 connected cylinders and having a common top structure. Figure 8.1 show a principle sketch of such an anchor. This anchor will have connection of the anchor line to the top structure. This means that anchor line may be changed and that the life time of the anchor may be the needed 100 years by designing the support structure for the line in a sufficient robust manner.

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The construction of such an anchor may be performed in a dry-dock and the anchor may be transported to the bridge location in a floating condition. It is designed by internal buoyancy tanks that keep the structure floating until installation. Installation is performed by deballasting the buoyancy tanks and assisted by a crane vessel lowering it down to the sea bottom. Based on the assumed soil conditions, the anchor will penetrate down to quite close to wanted penetration depth and the need for suction in addition to the hydrostatic pressure will be limited.

- 2) **One narrow and deep anchor** with diameter 6 m penetrating down to 19 m below sea bottom. The design of this anchor is based on connection of the anchor line at 11 m depth which is the optimal for this type of anchor. This connection will be by chain up to some level above sea bottom. Using an appropriate design for the connection and the chain, this connection may also achieve the needed service lifetime.

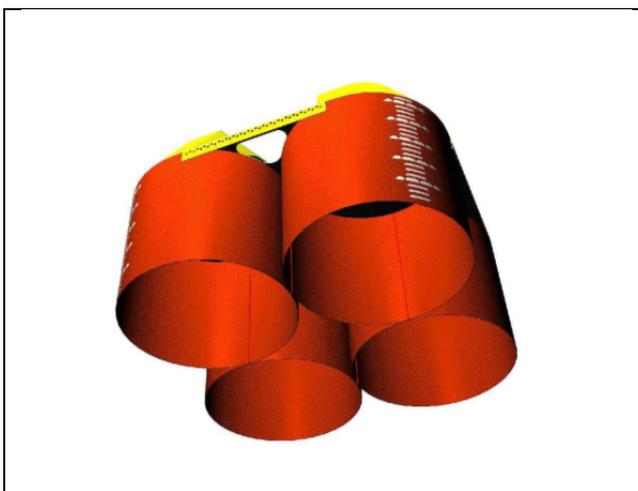
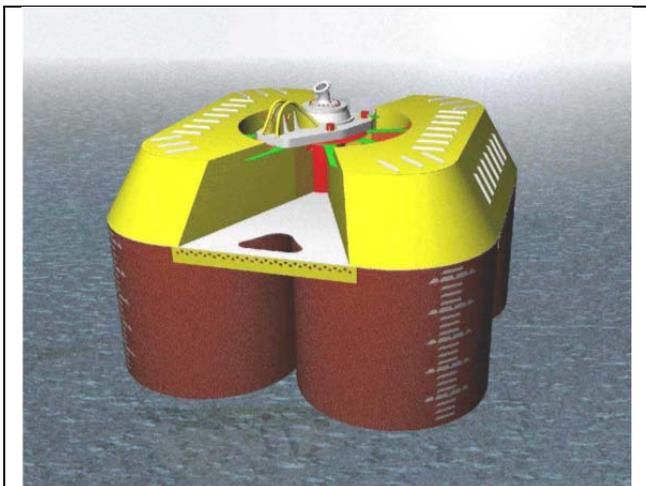


Figure 8.1 Principle drawings of a shallow anchor often used by National Oilwell Varco APL Norway AS

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An exchange of anchor line for such a connection will probably be more challenging than for the shallow anchor type.

Such a deep anchor will normally be transported on a barge to the construction site and installed by a heavy lifting vessel in the same manner as for the shallow anchor. The need for suction in addition to the hydrostatic pressure will also for this case be limited and well within the present capabilities of existing equipment.

8.3 Results

Capacity calculations for two types of suction anchors for the floating bridge are shown in Appendix F. Both concepts represents well tested and developed technologies and have been used for anchoring of different offshore structures. Both concepts have sufficient capacities to support one anchor line within already tested and built dimensions. Thus supporting an anchor line at 1300 m depth represents no new step of technology.

In the further illustrations of the bridge concept, the deep and narrow concept is used. An anchor with diameter 6 m and penetrating down to 18 m below sea bottom is shown. The calculation in Appendix F is based on a depth of 19 m.



Figure 8.2 Illustration of deep narrow anchor showing anchors installed for the Diana platform in the Gulf of Mexico on 1500 m water depth

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9 DESIGN OF PONTOONS

9.1 General

The two towers in the fjord are supported by floating concrete pontoons anchored to the bottom and to shore. The anchor system limit the horizontal movement of the pontoons, while the pontoon itself must have enough buoyancy to carry its own weight as well as vertical forces from the superstructure and the anchor system. In addition it must have sufficient rotational stiffness to resist eccentric loading from traffic, wind, waves and current.

Action loads on the pontoon itself is in addition for instance water pressure, current, waves and ship impact

9.2 Layout

The pontoons have a draft of 175 m and a nominal freeboard of 7 m. Due to variations in water weight of $\pm 1\%$ the freeboard will vary between 5.2 m and 8.8 m.

The governing loading for the pontoon will be different in the ship impact zone compared to the rest of the pontoon. This zone is defined down to elevation -20m. Below this level, outer water pressure will be governing.

Slabs are introduced in top and bottom as well as between the zones described above. The latter is placed at elevation -25 to be beneath the ship impact zone.

Lower part of the pontoon consist of eight outer cylindrical cells with overall radius of 10.85 m and one inner cylinder with overall radius of 17.5 m. These radiuses are constant with depth. The outer surfaces which are exposed to water pressure are circular shaped, while all inner parts consist of straight walls, see Appendix G. The wall thicknesses are the same up to 45m above the bottom of the pontoon. From this level they have a linear variation up to the middle slab.

The bottom slab thickness is 2.7m except in centre cell where it is increased to 7.0 m. This kind of thickness requires a good logistic plan for the casting process. Alternatively, the slabs may be replaced with domes. It is however beneficial with heavy weight in the bottom of the floater, and the slabs also have a general cost benefit to domes.

The chosen wall configuration in bottom part is not very suitable for local impact from ships. Therefore, it is proposed another configuration in the top part, see Appendix G. A uniform 1m thick circular outer wall is introduced. This coincides with outer face of the cell walls below.

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This wall is supported horizontally by means of a system of cell walls running through the whole pontoon. All walls are 0.6m thick except for diagonals in outer parts. These have wall thicknesses up to 0.75m. Inner ring is post tensioned with horizontal hoop cables 6-19 c/c 0.5m.

The loading from ship impact is calculated to be app. 350 MN. We have assumed a bulb impact area of 25m², corresponding to a bulb diameter of 5.6m. This assumption is not critical to the solution, the important thing is to recognize that the area is so small that a direct hit can take place in between the walls. Therefore the outer space between the walls are filled with LWA concrete with unit weight 11 kN/m³ and concrete quality 12 MPa. This imply that the fill in concrete do not crush during the impact. Another advantage is that this solution prevents unintentional water filling. Anyway is the adjacent cell walls design for outer water pressure in case unintentional water filling still occurs.

In later phases of the project, one should look for a fender system outside the pontoon to prevent local damage and reduce retardation of the ship.

The tricells beneath the tower legs are filled with concrete to anchor the towers in an area of 7x7m.

The fastening of the anchor system to the pontoon is done by means of a “fairlead” system including an anchor winch for each anchor line. The fairlead is placed beneath the ship impact zone while the anchor winch is in top of the top slab. The chains that are a part of the anchor line’s upper part will be protected by concrete ribs. The fastening of the fairleads is through prestressed bolts into the mid slab.

De lowers app. 45m of the pontoon is filled with water filled olivine rock with a unit weight of 28kN/m³for ballast.

It is assumed that the cells below mid slab cannot be water filled from mechanical damage. This at least implies that hydro acoustic transmitters should be installed to alert possible submarines in the area. Anyway, it is recommended that every cell is equipped with permanent pumping systems with sensors for water leakage so that the cells can be emptied.

The tricells between the circular cells may also be filled with foam or similar to avoid pumps here.

The material quantities in the pontoons are:

Concrete B45:	105000 m ³
LWA-concrete, B12:	38500 m ³
Reinforcement:	16000 tons
Post tensioning:	48500 MNm
Olivine ballast:	155000 m ³

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9.3 Design

The design of lower part of the pontoons are based on hand calculations verified with two analyses, one 3D global analysis of the pontoon to determine implosion/buckling from outer water pressure and one plane sectional model to calculate sectional forces in the members.

The bottom slab is calculated by hand using tabulated response for circular plates.

Buckling shapes and factors are given in Appendix G. The corresponding design is conservatively done based on supplementary hand calculations.

Sectional forces from outer water pressure are determined in the sectional model, see Appendix G. These results are also conservative since it does not take into account vertical force transfer.

Local bending is based on 1. order bending moments from outer water pressure added an imperfection of maximum 50mm, shaped as lowest buckling mode. In addition to these two effects, a 2. order scaling factor of $1 / (1 - N/N_{cr})$ is taken into account. N is then actual axial force and N_{cr} is $3/4$ x buckling load on actual spot. The reduction factor of $3/4$ takes into account that water pressure is a non-conservative loading that give a lower buckling load than regular axial loading. The latter remain in the same direction during buckling action.

The top slab and middle slab is design by means of hand calculations.

The upper part walls are also design by means of hand calculations. They are checked for crushing just inside the outer walls, and also that they have the capacity to transfer ship impact loads to adjacent walls and that these can transfer the forces to the slabs.

9.4 Hydrostatic stability and rotational stiffness

The hydrostatic stability is expressed by means of metacentric height GM, containing difference between centre of buoyancy and centre of gravity as well as water plane stiffness, see also Appendix G.

The expression is as follows:

$$GM = VCB - VCG + I \rho g / \Delta$$

where VCB and VCG are the two centre points and $I \rho g / \Delta$ is an expression of the water plane area stiffness divided by the displacement of the pontoon.

In our case this lead to

$$\begin{aligned} GM &= -86,0 - (-117,5) + 189,373/75,546 \\ &= 31,5 + 2,5 = \underline{34,0 \text{ m}} \end{aligned}$$

Rotational stiffness is correspondingly:

$$K\phi = GM \times \Delta = 34,0 \times 75,546 \times 10^5 = \underline{2569 \times 10^5 \text{ kNm/rad}}$$

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10 DESIGN OF BRIDGE BEAM AND TOWERS

10.1 General

The superstructure and the towers are designed using sectional forces from the global analysis as described in chapter 3. The design checks show acceptable utilization ratios and stresses for the chosen dimensions. The towers and bridge deck are in addition checked for accidental ship collision. There are a few critical points showing too low capacity, but this is easily solved through local increase of steel quality or plate thickness.

10.2 Bridge beam

The bridge deck is an orthotropic steel box girder with longitudinal trough stiffeners and with diaphragms every 4m.

The bridge deck is continuous from axis 1 to axis 6 and with expansion joints at each end. The longitudinal fixation at each end is achieved through external post tensioned cables. The viaduct part at each end is supported with columns every 80m. These have sliding bearings with transverse load transfer. In axis 2 and 5, the bridge deck is supported in transverse direction and is sliding in longitudinal direction. In axis 3 and 4 the bridge deck is not supported vertically other than through the hangers. Thereby neither torsion nor vertical shear in the bridge deck are transferred to the tower. The bridge deck is however fixed to the deck for longitudinal deformation, transverse deformation and rotational deformation about vertical axis.

The bridge deck is checked against allowable stresses in ULS. The design checks show that the bridge deck needs to be strengthened adjacent to towers axis 3 and 4. This is mainly due to support moments from wind loading. The strengthening is done partly by increasing steel plate thicknesses and partly by increased steel quality from S355 steel to S420 steel. The strengthened area is chosen to be ± 150 m from axis 3 and 4. Further strengthening for future increased ship collision loads can be done by increasing the steel quality to S460 or by increased plate thicknesses.

The bridge deck has in general steel plate thickness 8mm in bottom face and 12 mm in top face. The drawings are adjusted to 14mm in top face due to EU regulations. In the strengthened area corresponding plate thicknesses are 14mm and 20mm. This gives a steel area increase of 50% while moments of inertia increase by 54% about horizontal axis and 62% about vertical axis.

To simplify the design evaluation, base case section have been run through both with base case area through the whole bridge deck and with strengthened area where chosen.

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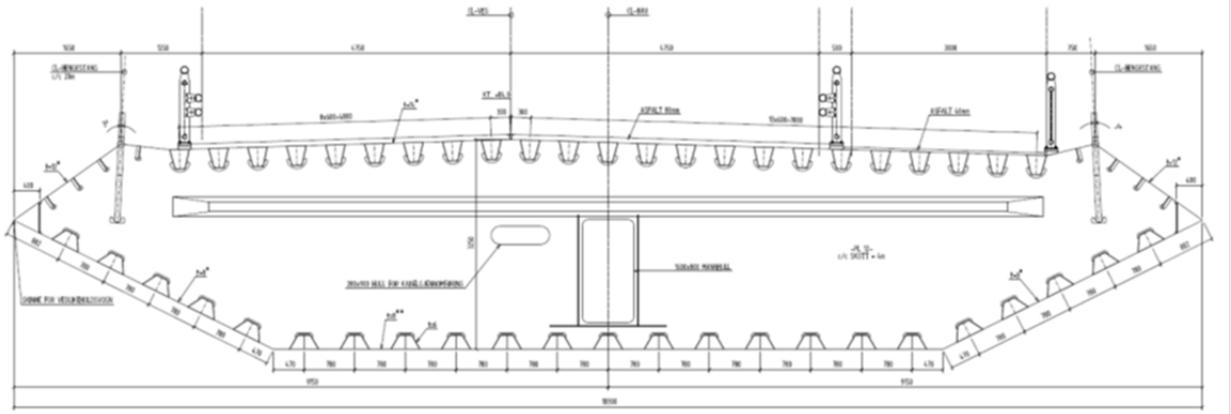


Figure 10.1: Cross section bridge deck

10.2.1 Steel box, ULS

ULS combinations are made out of permanent loads, traffic, temperature, wind, post tensioning and imposed forces from pontoon drift forces. Dominant wind, permanent loads and traffic are considered. Governing load combination is mainly a dominant wind with load factor 1.6. The sketch in Figure 10.2 shows the defined stress points. Maximum stresses occur in point SP-R adjacent to axis 3 and 4. After strengthening, stresses in this point reduce from 535MPa to 365MPa. This corresponds to a reduction of 32% for 50% increase of the area. Therefore, increased strength S420 is chosen adjacent to axis 3 and 4. In the rest of the bridge deck, S355 is used. Note that the analysis also is strengthened in an area $\pm 100\text{m}$ adjacent to axis 2 and 5, even if this is not required.

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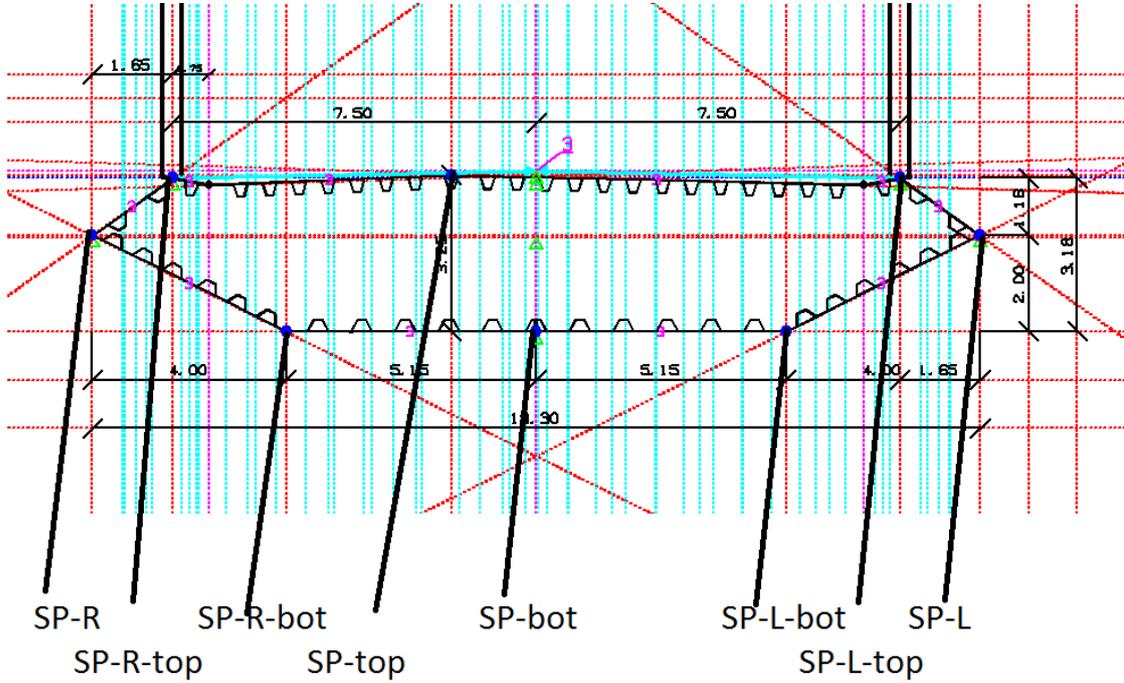


Figure 10.2: Explanation of where stresses are calculated in the global analysis. Increasing profile number towards the reader.

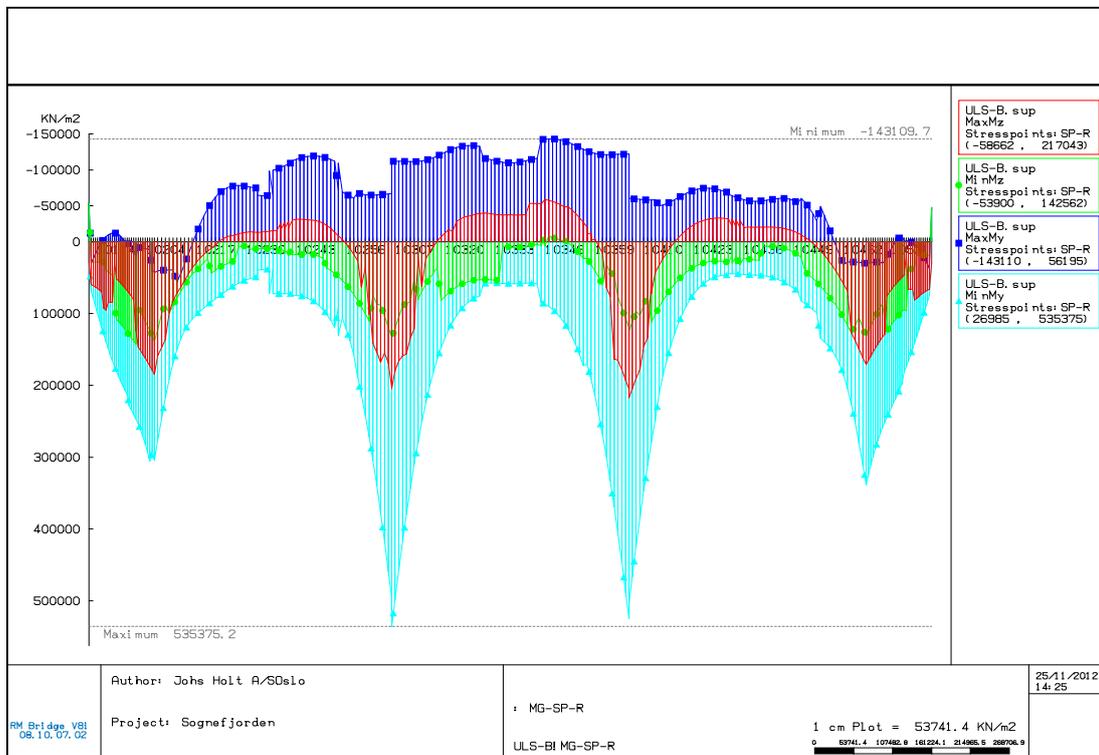


Figure 10.3: Stresses in node SP-R in the general bridge deck

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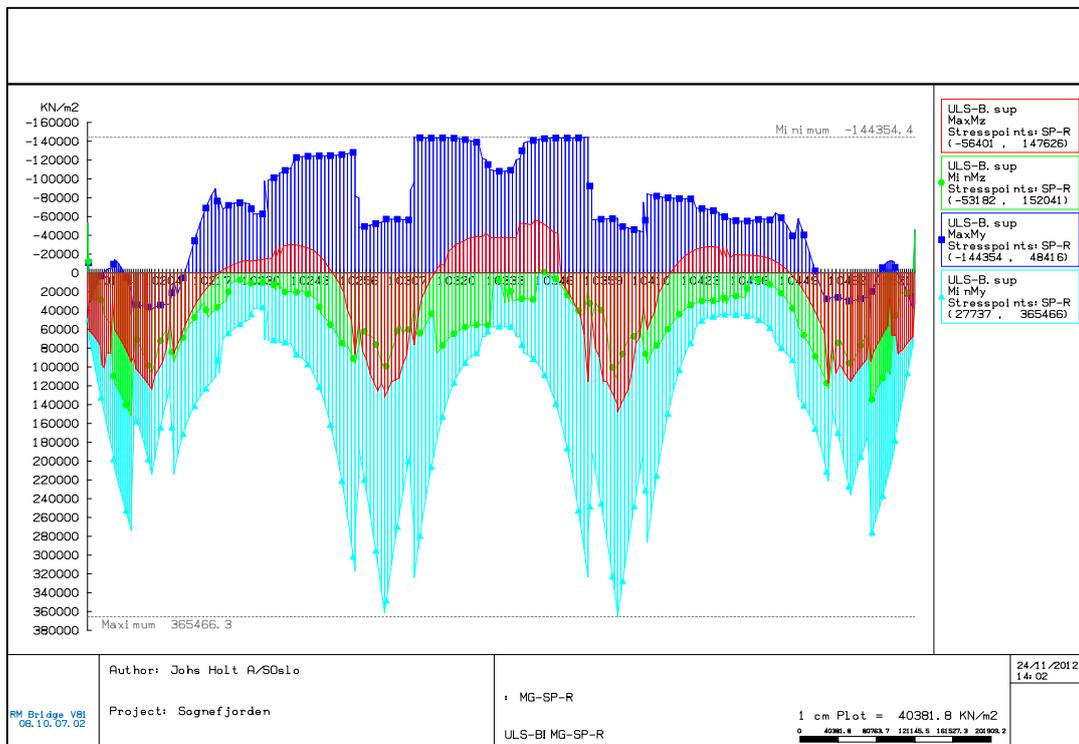


Figure 10.4: Stresses in node SP-R in the strengthened bridge deck.

10.2.2 Bridge deck. Ship collision

The bridge deck is checked for loading from ship collision onto the pontoons. In the global analysis a time series of the impact is applied. The time series is determined in a local USFOS analysis. Two scenarios are looked at, one with a direct hit transverse to the bridge (90 degrees to the bridge axis), and one with an eccentric hit that will cause a rotation of the pontoon. The check is done for a damping 1%, 2% and 5%. In this report results only for 2% damping is given. The ship collision analysis is described in Appendix E.

The stresses in the bridge deck are shown below. This shows a max. stress of 728MPa, but scaling to the strengthened cross section as described for the ULS check, max. stress reduce to just below 500MPa. For the eccentric collision, max. stress is 327 MPa for 3.1° rotation of the pontoon. These are stresses that can be handled through local strengthening. The concept therefore seems feasible for direct ship impact on the pontoons.

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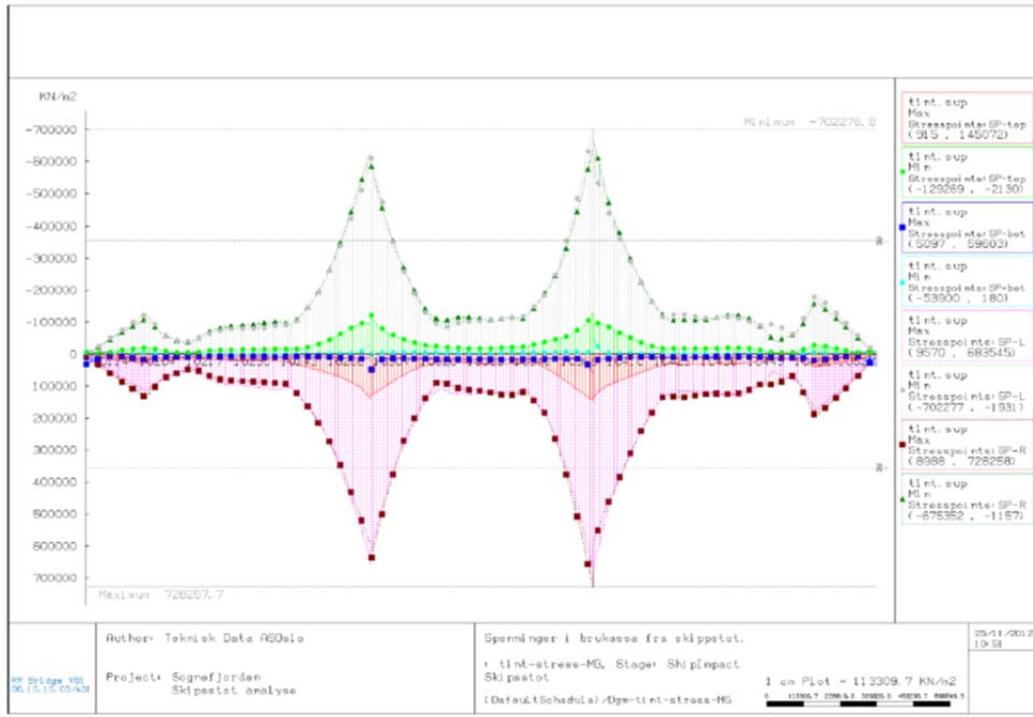


Figure 10.5: Ship impact 90°

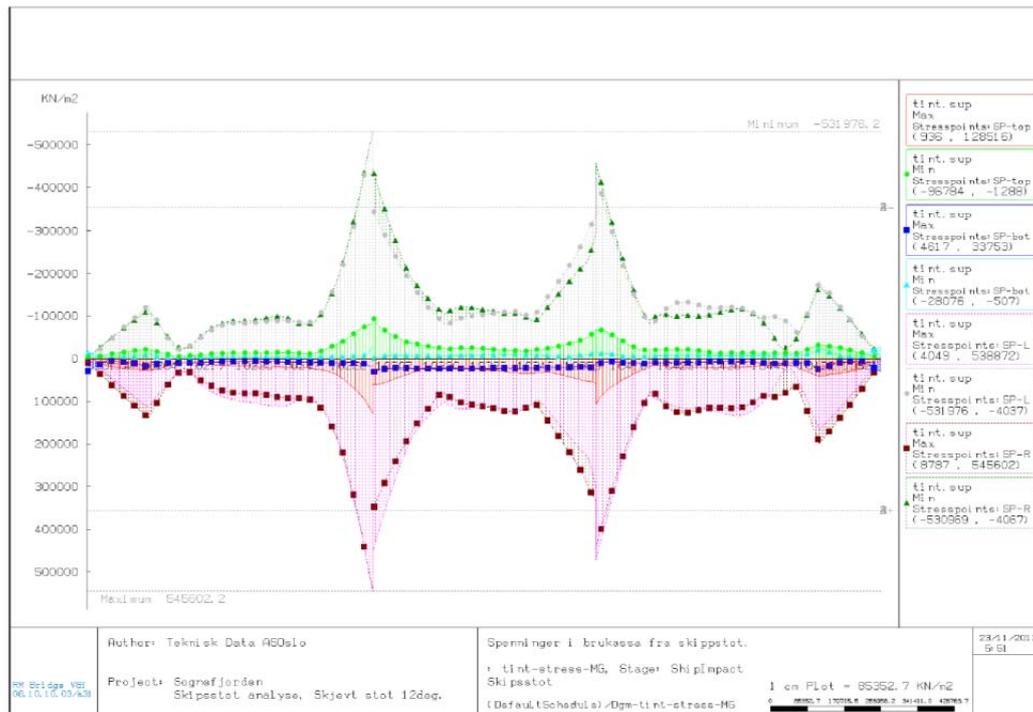


Figure 10.6: Ship impact 78°

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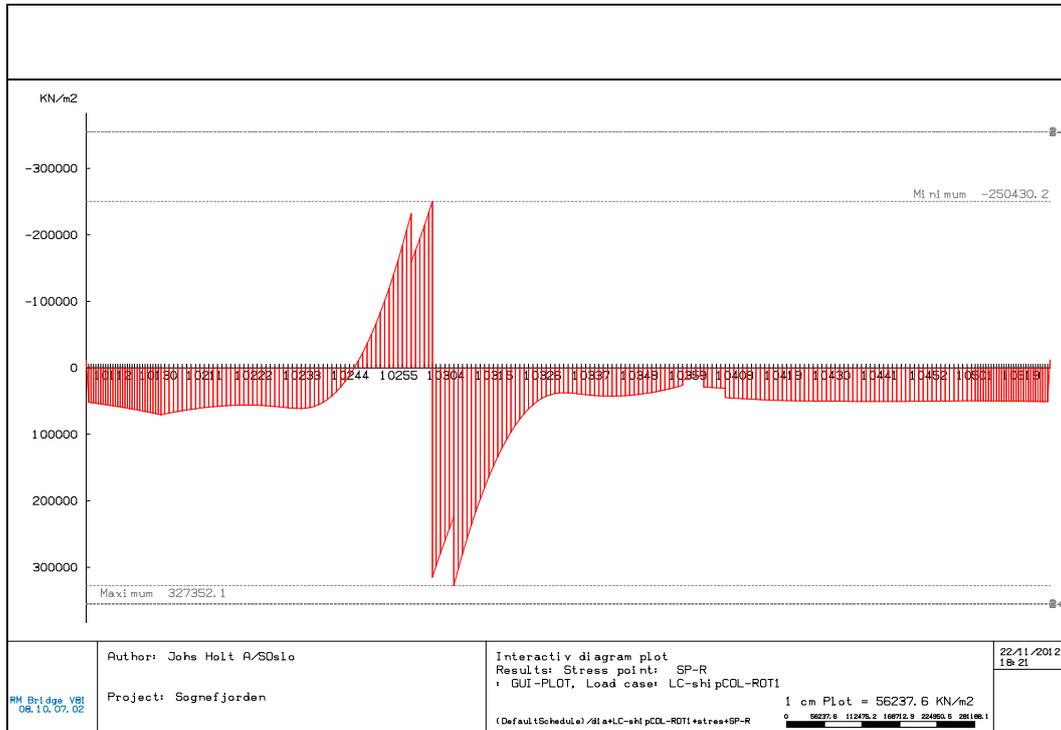


Figure 10.7: Ship impact, 3.1° rotation of the pontoon

10.2.3 Horizontal spring at abutments

To keep the bridge in place and avoid that wind and drift forces in longitudinal direction will deflect the whole structure towards the opposite shore, a horizontal spring is introduced at each abutment. This spring is made of four 6-61 external post tensioned tendons with a length of 500m each, giving a string stiffness of 15 MN/m. The tendons need to be able to deform according to $\pm 36^{\circ}\text{C}$ temperature movement and adequate traffic loading, corresponding to 720mm and 630mm, in total $\pm 1168\text{mm}$. In order to do this, the tendons must be post tensioned to 4‰. This strain correspond to a force of 28,5 MN. Max. force in the tendons are $6.6 \cdot 195000 \cdot 36600 / 1\text{E}6 = 47.1\text{ MN}$.

Tensioning should be done simultaneously on each end to avoid longitudinal deflection. Alternatively, app. 90 % of the tensioning can be applied on one side with the deck fixed to the abutment, and afterwards the tensioning is done on the other side to the point where the fixation is lost. Thereafter the last 10% are used to get equal joint opening at each side.

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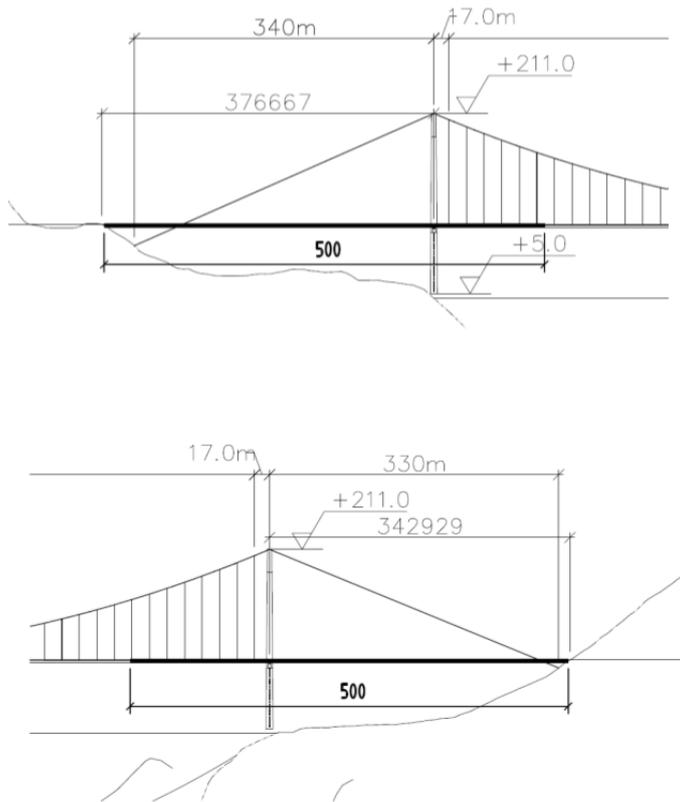


Figure 10.8: Horizontal spring in axis 1 and 6

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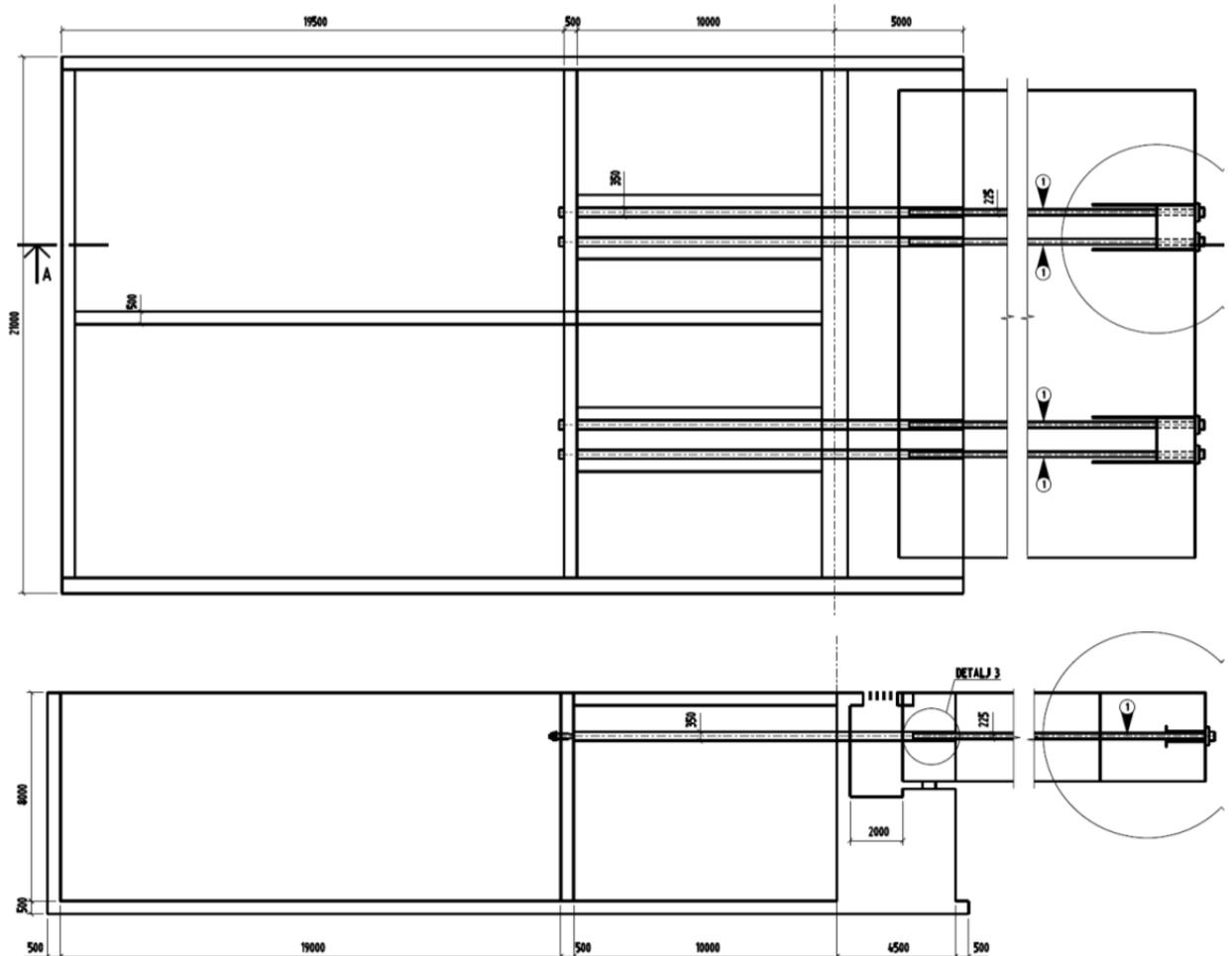


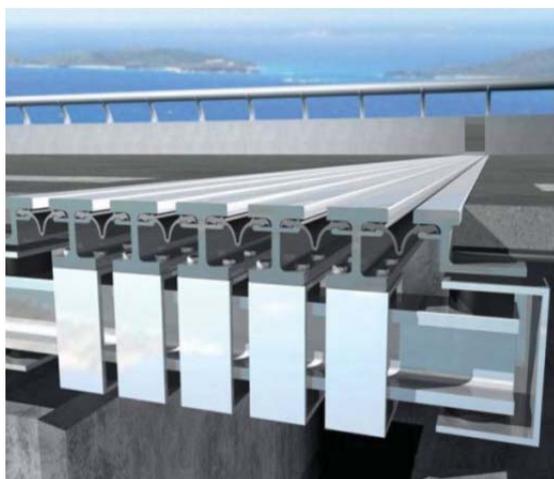
Figure 10.9: Horizontal spring in abutment in axis 1 and 6

10.2.4 Expansion joint and bearings

The expansion joints and bearings are designed for temperature loads corresponding to $\pm 36^{\circ}\text{C} + \pm 10^{\circ}\text{C} = \pm 46^{\circ}\text{C}$ in accordance to deformation from traffic loading. In total this implies a movement of $\pm 1144\text{mm}$. This corresponds to an expansion type LR30 that has a total movement capacity of 2400mm .

The bearings in axis 1 and 6 must have the same capacities, implying upper sliding plates with capacity $\pm 1200\text{mm}$.

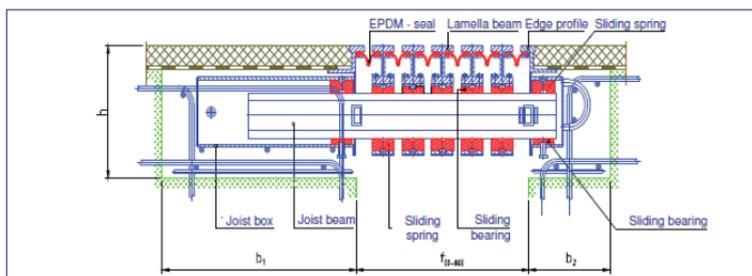
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mageba

Type designation 3

Type designation, movements and dimensions



Type & Size	Max. range of movement mm	Recess space			Joint width				Weight kg/m
		b ₁ mm	b ₂ mm	h mm	f ₀ mm	f _s mm	f _{ss} mm	f _{so} mm	
LR 2	160	400	300	400	140	150	270	300	150
LR 3	240	480	300	400	220	235	415	460	240
LR 4	320	560	300	400	300	320	560	620	330
LR 5	400	640	300	420	380	405	705	780	420
LR 6	480	720	300	420	460	490	850	940	510
LR 7	560	800	300	420	540	575	995	1'100	600
LR 8	640	880	300	440	620	660	1'140	1'260	690
LR 9	720	960	300	460	700	745	1'285	1'420	790
LR 10	800	1'040	300	480	780	830	1'430	1'580	900
LR 11	880	1'120	300	510	860	915	1'575	1'740	1'020
LR 12	960	1'200	300	510	940	1'000	1'720	1'900	1'140
LR 15	1'200	1'490	350	590	1'320	1'395	2'295	2'520	1'400
LR 20	1'600	1'860	350	655	1'770	1'870	3'070	3'370	2'050
LR 25	2'000	2'260	350	675	2'220	2'345	3'845	4'220	2'750
LR 30	2'400	2'660	350	735	2'670	2'820	4'620	5'070	3'500

Figure 10.10: Data Mageba LR joints

In the viaduct area between axis 1 and 2 it is planned sliding bearings in one direction. These will have much sliding that may affect the friction close to the support. This can be avoided by moving the expansion joints to axis 2 and 5.

In axis 2 and 5 there is two sliding bearings beneath the bridge deck and two on the side as shown below. Both should have sliding capacity app. ±1000mm. Horizontal forces in the bridge deck will be transferred to the bearings brackets underneath the deck.

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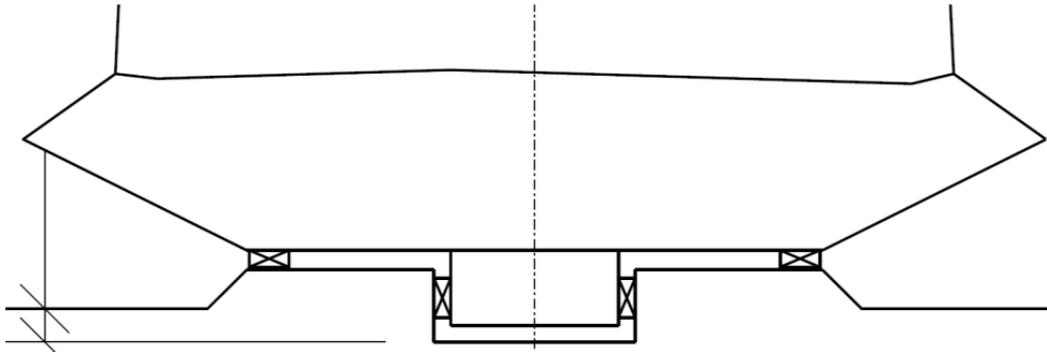


Figure 10.11 Configuration of bearings axis 2 and 5

In axis 3 and 4 force transfer is similar to axis 2 and 5. Vertical forces are transferred through the hangers. The bridge are fixed to the tower for longitudinal movement with a cross beam as shown on figure 10.13.

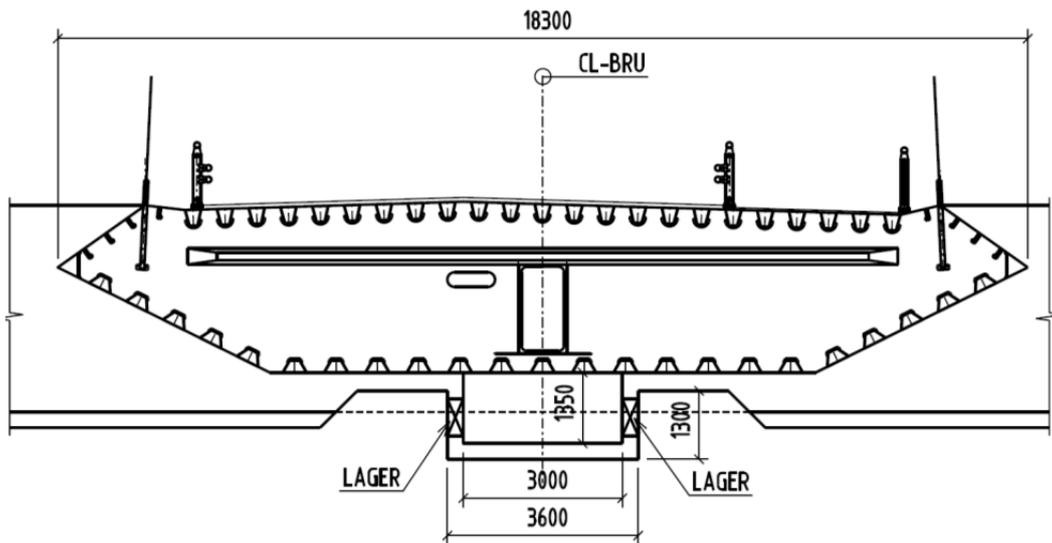


Figure 10.12: Side bearings in axis 3 and 4

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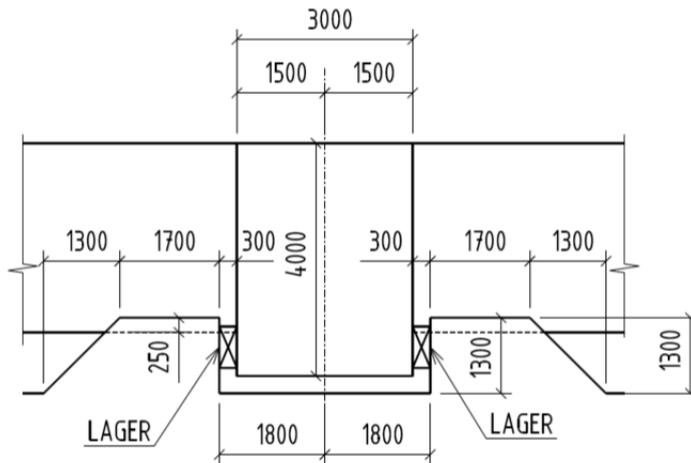


Figure 10.13: Bracket for transferring longitudinal loads in axis 3 and 4

10.3 Towers

10.3.1 General

There are two types of towers, concrete towers on shore and steel towers off shore. Steel is chosen to save weight and to ease construction at sea.

The steel towers have a diamond shape. This shape is chosen to minimize the footprint of the tower on the pontoon. In addition, it is beneficial to have a relatively wide configuration at bridge deck to reduce the deformations in top of the tower. The inclination towards the tower base also makes it possible to place bracings between the tower legs to increase the torsional and shear capacity of the tower. This is important to control the response from eccentric ship collision.

The layout of the concrete towers are chosen similar to axis 3 and 4 on side view. This is done for aesthetic reasons.

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10.3.2 Geometry

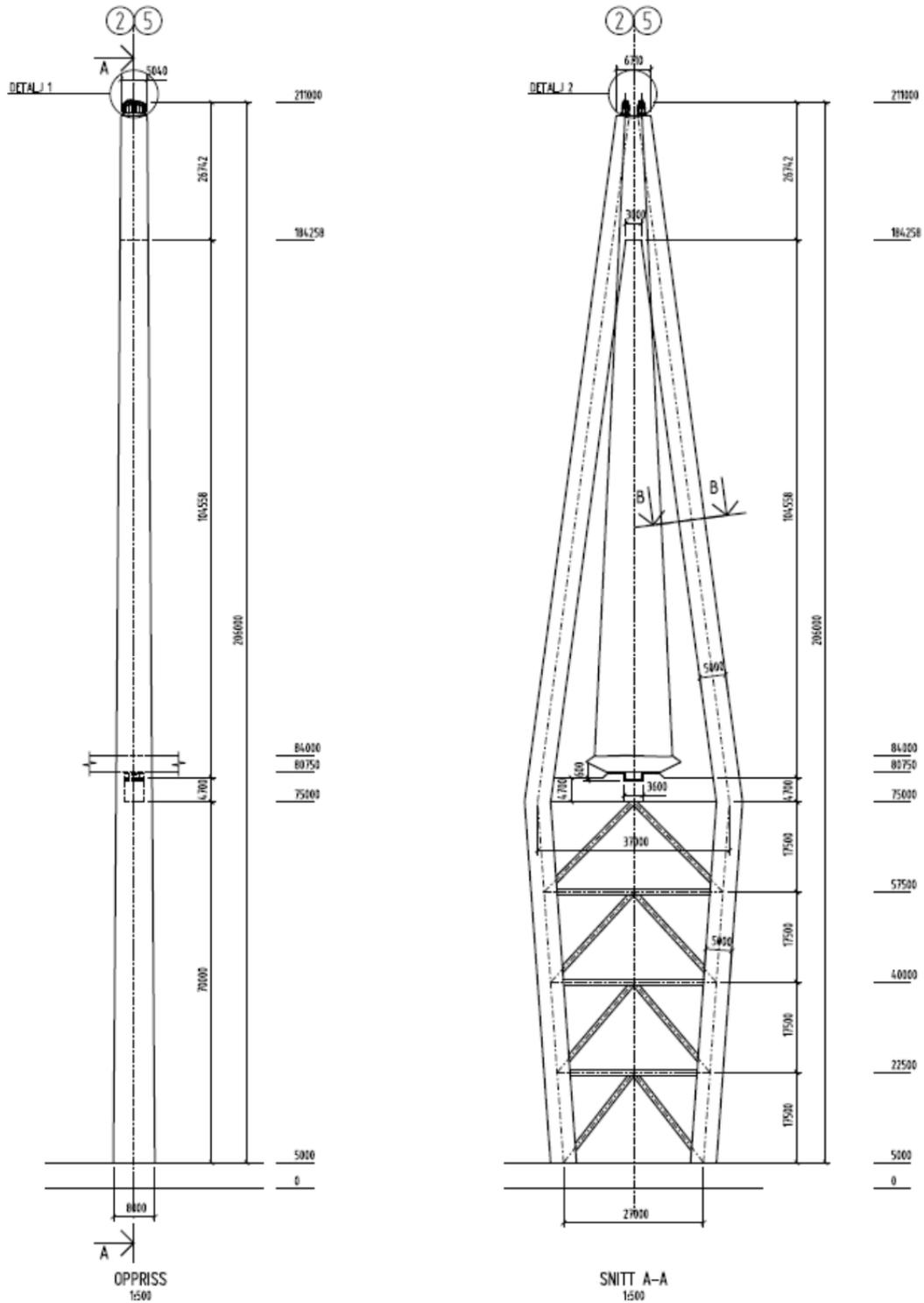


Figure 10.14: Towers axis 2 and 5

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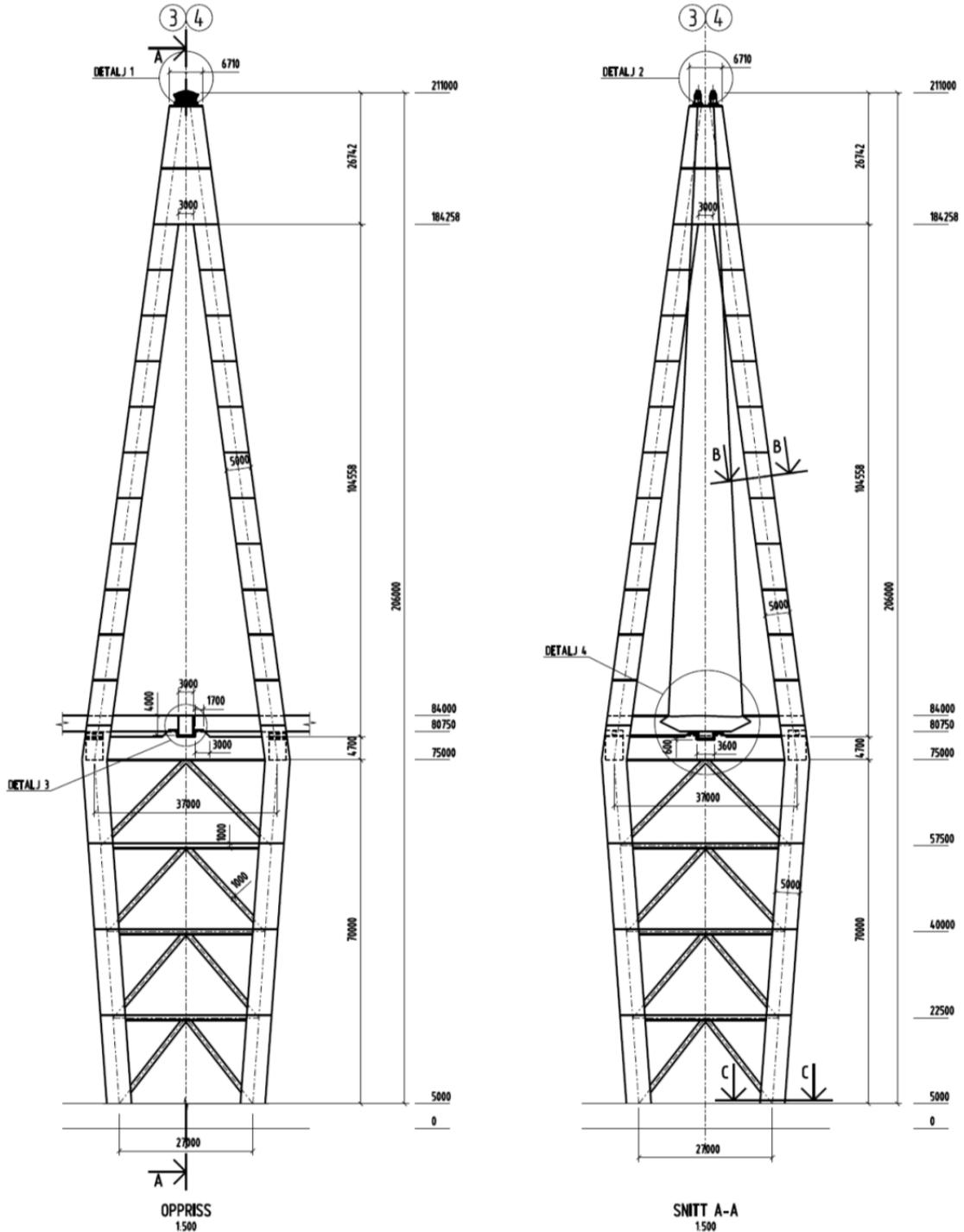
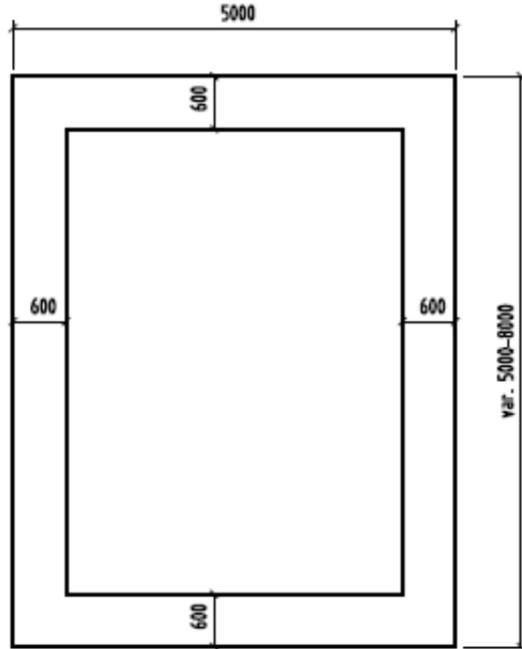


Figure 10.15: Towers axis 3 and 4

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SNITT B-B
1:50

Figure 10.16: Typical section, tower axis 2 and 3

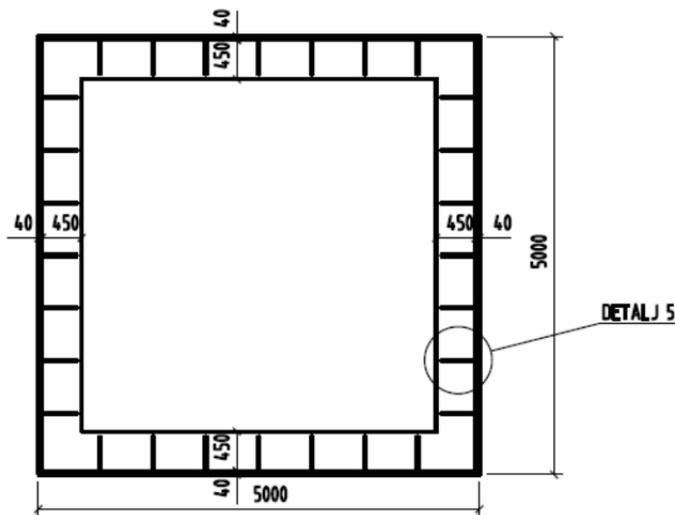


Figure 10.17: Typical section, tower axis 3 and 4

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10.3.3 Design

General

In the feasibility study the towers are designed for ULS B with combinations of permanent loads, traffic, external post tensioning and wind loading. The towers are designed for most critical load combinations and checked for local and global buckling. Using plate thicknesses 40mm, upper part of the tower get maximum utilization ratio of 100%, while lower part gets a maximum utilization ratio of 105%. This maximum value is located in the lowest design section, and can be easily coped with by increasing the plate thickness.

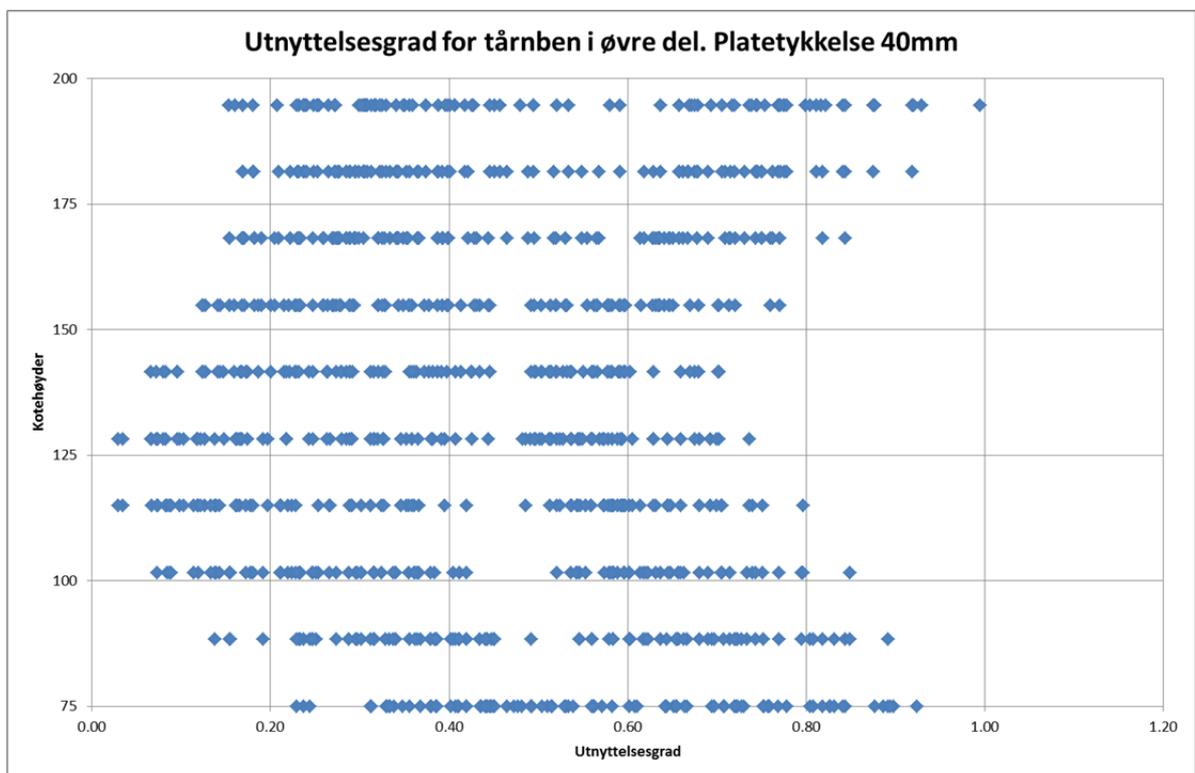


Figure 10.18: Utilization ratios in towers, upper part

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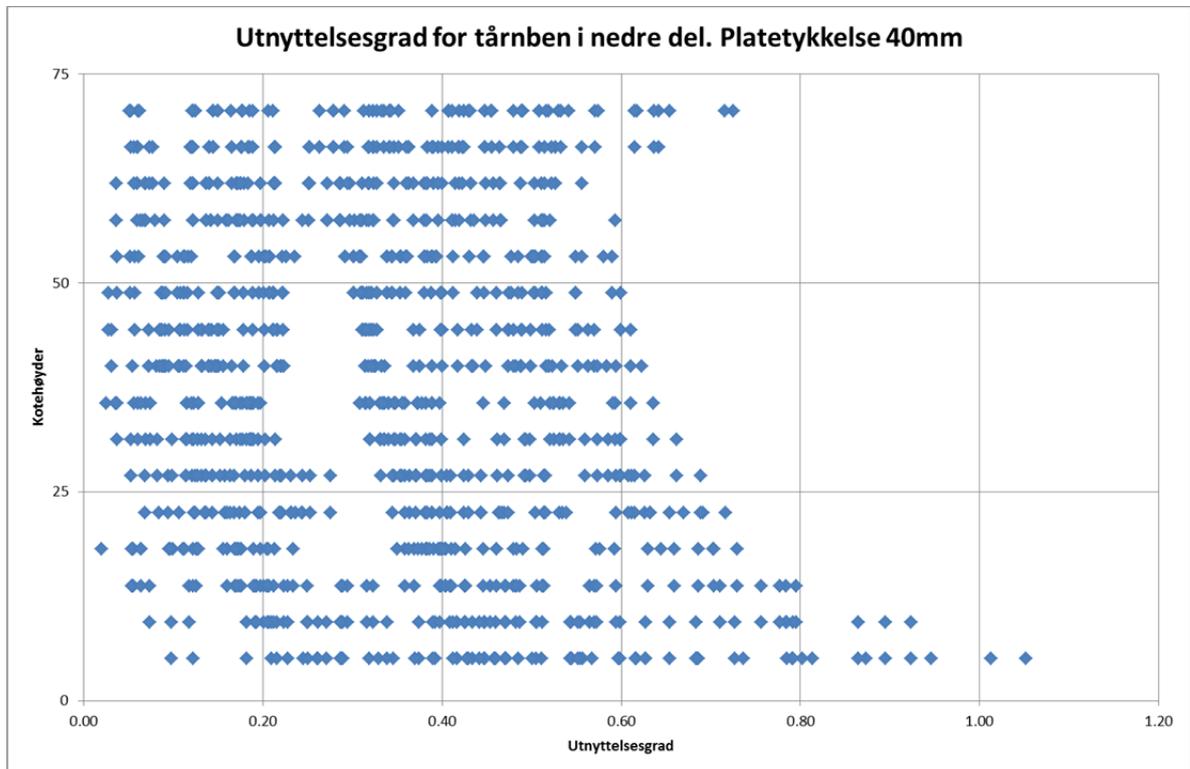


Figure 10.19: Utilization ratios in towers, lower part

Ship collision

A ship collision analysis is performed and reported in Appendix E. The superstructure and towers are checked for the occurring sectional forces. Actual impact has been 90 ° and 78° angle to the bridge axis. In addition, eccentric impact giving a rotation and the pontoon of 3.1°, is investigated.

It is the lower part of the towers that experience the largest sectional forces. Utilization ratios are shown below.

Direct hit gives a utilization ratio of maximum 93%, while the 3.1° rotation give a utilization ratio of app. 50%.

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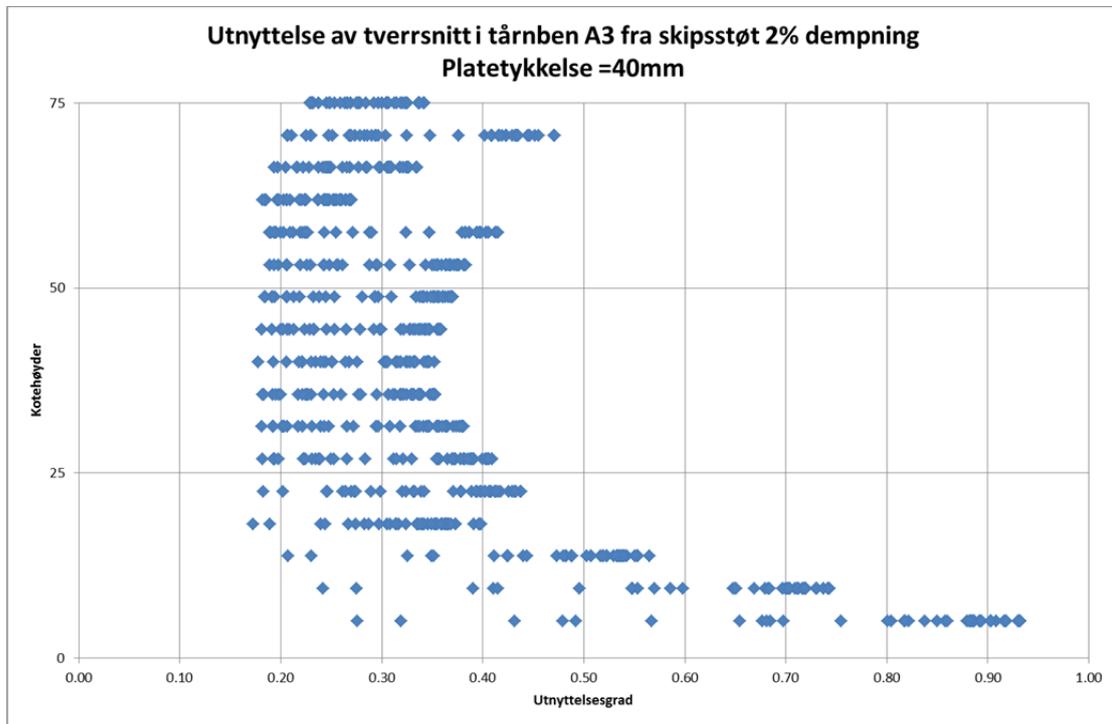


Figure 10.20: Utilization ratios in towers, 90 degree ship impact

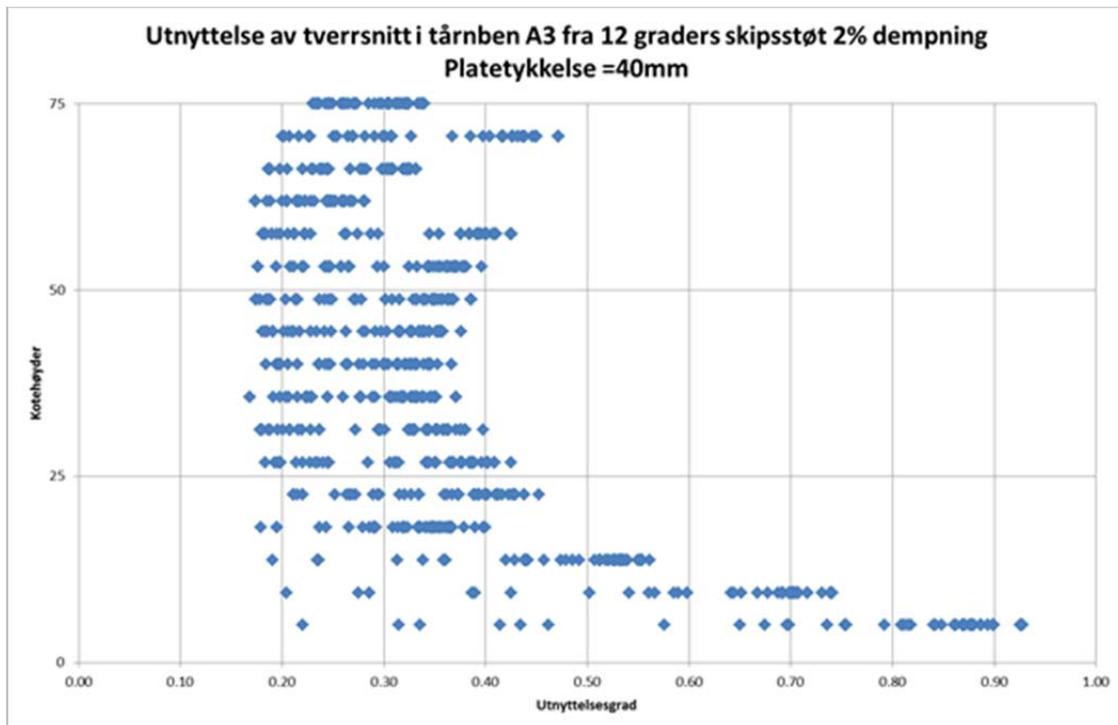


Figure 10.21: Utilization ratio towers, 78 degree ship impact

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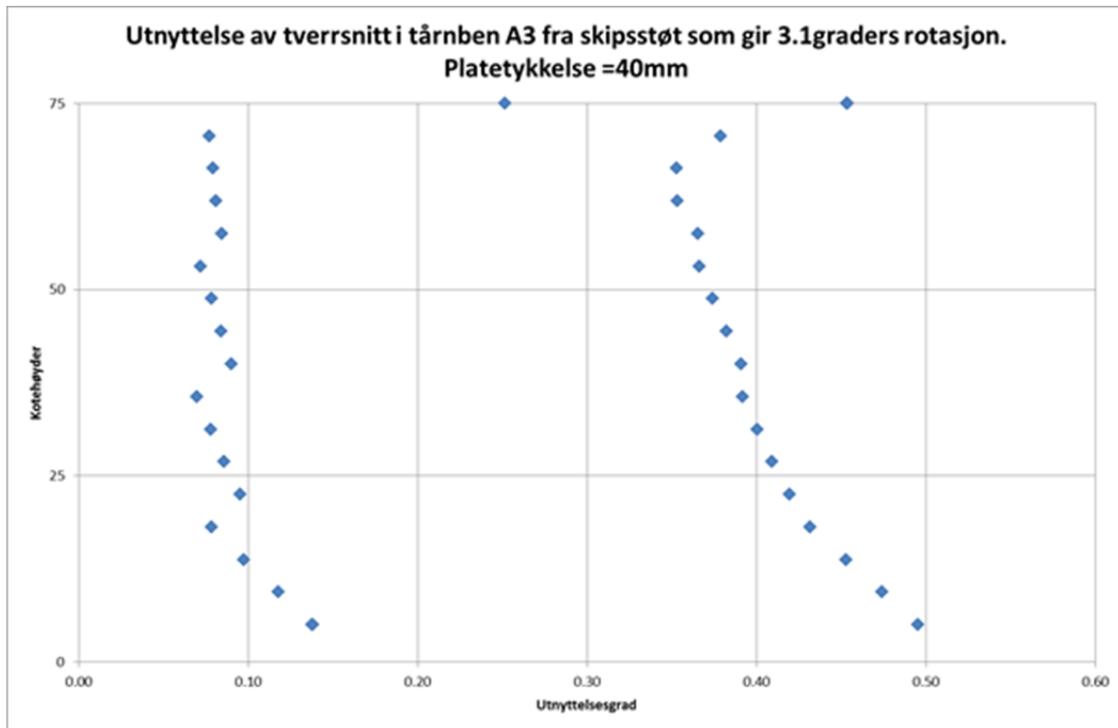


Figure 10.22: Utilization ratios towers, eccentric ship impact

10.3.4 Saddle

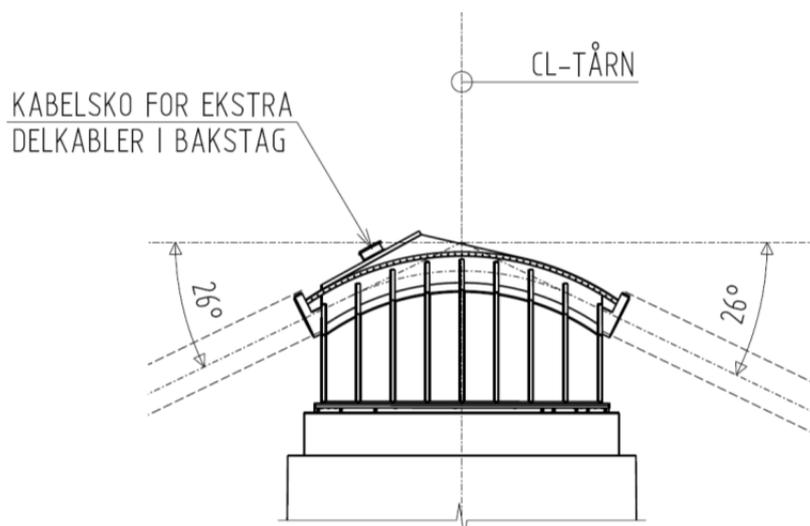


Figure 10.23: Principle sketch, tower saddle

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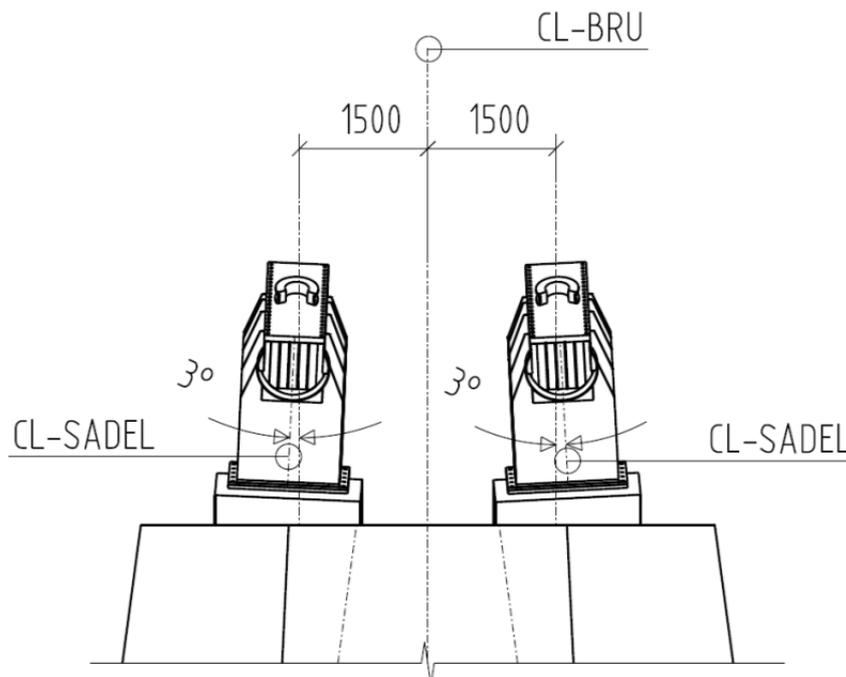


Figure 10.24: Principle sketch, tower saddles

According to design basis, nominal friction factor between tower saddle and main cable is 0.2. With a material factor of 1.65 we get a design friction factor of 0.12. This is too low compared to occurring friction in ULS, which is 0.21, see chapter 6, Appendix H.

The tower saddle has 4 internal plates that can provide side friction to the cable if they are welded to the saddle. If this is taken into account by assuming a lateral pressure of 1/6 of the vertical pressure, the total friction factor increases to 0.2 or almost the same as actual force.

One other measure to increase friction capacity is to introduce vertical compression over the saddle from anchor plates. This is done on the Taizhuo Bridge in China.

The largest transversal load on the tower saddle is app. 1900kN per saddle in ULS. Corresponding vertical load is 91000kN, which give a 1.2 degrees inclination from vertical axis. Taking into account that the main cables have an inclination of 3 degrees from permanent loads, this force should easily be absorbed.

10.3.5 Material quantities

This chapter gives tables of material quantities of concrete and steel.

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Tower axis 2 and 5

Elnr	Beskrivelse	Tårn					Vol
		L	B	T	n	Vol	
1	El nr 1	20	5	0.6	4	240.00	
2	El nr 2	20	5	0.6	4	240.00	
3	El nr 3	20	5	0.6	4	240.00	
4	El nr 4	20	5	0.6	4	240.00	
5	Riegel	33	5	0.6	2	198.00	
6	Riegel	33	3	0.6	2	118.80	
7	El nr 1 over rigel	9.3	5	0.6	4	111.60	
8	El nr 2 over rigel	9.3	5	0.6	4	111.60	
9	El nr 3 over rigel	9.3	5	0.6	4	111.60	
10	El nr 4 over rigel	9.3	5	0.6	4	111.60	
11	El nr 5 over rigel	9.3	5	0.6	4	111.60	
12	El nr 6 over rigel	9.3	5	0.6	4	111.60	
13	El nr 7 over rigel	9.3	5	0.6	4	111.60	
14	El nr 8 over rigel	9.3	5	0.6	4	111.60	
15	El nr 9 over rigel	9.3	5	0.6	4	111.60	
16	El nr 10 over rige	9.3	5	0.6	4	111.60	
17	El nr 11 over rige	9.3	5	0.6	4	111.60	
18	El nr 1 topp	13.35	5	0.6	4	160.20	
19	El nr2 topp	13.35	5	0.6	4	160.20	
					Sum	2824.8	
					Begge tårn	5,650	

Figure 10.25: Concrete quantities axis 2 and 5 (m3)

Elnr	Beskrivelse	Tårn					Stivere					Skott					Vol	densitet	Vekt (tonn)	Antall	Tot vekt (tonn)	
		L	B	T	n	Vol	L	B	T	n	Vol	L	B	T	n	Vol						
20	Diagonal 1	18	1	0.025	4	1.80	18	0.2	0.01	8	0.288	0.8	0.25	0.01	12	0.02	2.11	7.85	16.6	2	33.2	
21	Diagonal 2	18.5	1	0.025	4	1.85	18.5	0.2	0.01	8	0.296	0.8	0.25	0.01	12	0.02	2.17	7.85	17.0	2	34.1	
22	Diagonal 3	19.5	1	0.025	4	1.95	19.5	0.2	0.01	8	0.312	0.8	0.25	0.01	12	0.02	2.29	7.85	17.9	2	35.9	
23	Diagonal 4	21	1	0.025	4	2.10	21	0.2	0.01	8	0.336	0.8	0.25	0.01	12	0.02	2.46	7.85	19.3	2	38.6	
24	Horizontal 1	24	1	0.02	4	1.92	24	0.2	0.01	8	0.384	0.8	0.25	0.01	12	0.02	2.33	7.85	18.3	1	18.3	
25	Horizontal 2	27	1	0.02	4	2.16	27	0.2	0.01	8	0.432	0.8	0.25	0.01	12	0.02	2.62	7.85	20.5	1	20.5	
26	Horizontal 3	29	1	0.02	4	2.32	29	0.2	0.01	8	0.464	0.8	0.25	0.01	12	0.02	2.81	7.85	22.0	1	22.0	
																				Sum		202.6
																				Begge tårn		405

Figure 10.26: Steel quantities axis 2 and 5

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Towers axis 3 and 4

Elnr	Beskrivelse	Tårn					Stivere					Skott					Vol	densitet	Vekt (tonn)	Antall	Tot vekt (tonn)																			
		L	B	T	n	Vol	L	B	T	n	Vol	L	B	T	n	Vol																								
1	El nr 1	20		5	0.04	4	16.00	20	0.4	0.01	28	2.24	4.55	0.45	0.015	20	0.61	18.85	7.85	148.0	4	592.0																		
2	El nr 2	20		5	0.04	4	16.00	20	0.4	0.01	28	2.24	4.55	0.45	0.015	20	0.61	18.85	7.85	148.0	4	592.0																		
3	El nr 3	20		5	0.04	4	16.00	20	0.4	0.01	28	2.24	4.55	0.45	0.015	20	0.61	18.85	7.85	148.0	4	592.0																		
4	El nr 4	20		5	0.04	4	16.00	20	0.4	0.01	28	2.24	4.55	0.45	0.015	20	0.61	18.85	7.85	148.0	4	592.0																		
5	Riegel	33		5	0.03	2	9.90	33	0.4	0.01	20	2.64	4.55	0.45	0.015	10	0.31	12.85	7.85	100.8	4	403.4																		
6	Riegel	33		3	0.03	2	5.94	33	0.4	0.01	20	2.64	4.55	0.45	0.015	10	0.31	8.89	7.85	69.5	4	279.1																		
7	El nr 1 over rigel	9.3		5	0.04	4	7.44	9.3	0.4	0.01	28	1.0416	4.55	0.45	0.015	12	0.37	8.85	7.85	69.5	4	277.9																		
8	El nr 2 over rigel	9.3		5	0.04	4	7.44	9.3	0.4	0.01	28	1.0416	4.55	0.45	0.015	12	0.37	8.85	7.85	69.5	4	277.9																		
9	El nr 3 over rigel	9.3		5	0.04	4	7.44	9.3	0.4	0.01	28	1.0416	4.55	0.45	0.015	12	0.37	8.85	7.85	69.5	4	277.9																		
10	El nr 4 over rigel	9.3		5	0.04	4	7.44	9.3	0.4	0.01	28	1.0416	4.55	0.45	0.015	12	0.37	8.85	7.85	69.5	4	277.9																		
11	El nr 5 over rigel	9.3		5	0.04	4	7.44	9.3	0.4	0.01	28	1.0416	4.55	0.45	0.015	12	0.37	8.85	7.85	69.5	4	277.9																		
12	El nr 6 over rigel	9.3		5	0.04	4	7.44	9.3	0.4	0.01	28	1.0416	4.55	0.45	0.015	12	0.37	8.85	7.85	69.5	4	277.9																		
13	El nr 7 over rigel	9.3		5	0.04	4	7.44	9.3	0.4	0.01	28	1.0416	4.55	0.45	0.015	12	0.37	8.85	7.85	69.5	4	277.9																		
14	El nr 8 over rigel	9.3		5	0.04	4	7.44	9.3	0.4	0.01	28	1.0416	4.55	0.45	0.015	12	0.37	8.85	7.85	69.5	4	277.9																		
15	El nr 9 over rigel	9.3		5	0.04	4	7.44	9.3	0.4	0.01	28	1.0416	4.55	0.45	0.015	12	0.37	8.85	7.85	69.5	4	277.9																		
16	El nr 10 over rigel	9.3		5	0.04	4	7.44	9.3	0.4	0.01	28	1.0416	4.55	0.45	0.015	12	0.37	8.85	7.85	69.5	4	277.9																		
17	El nr 11 over rigel	9.3		5	0.04	4	7.44	9.3	0.4	0.01	28	1.0416	4.55	0.45	0.015	12	0.37	8.85	7.85	69.5	4	277.9																		
18	El nr 1 topp	13.35		5	0.04	4	10.68	9.3	0.4	0.01	28	1.0416	4.55	0.45	0.015	12	0.37	12.09	7.85	94.9	4	379.6																		
19	El nr 2 topp	13.35		5	0.04	4	10.68	9.3	0.4	0.01	28	1.0416	4.55	0.45	0.015	12	0.37	12.09	7.85	94.9	4	379.6																		
20	Diagonal 1	18		1	0.025	4	1.80	18	0.2	0.01	8	0.288	0.8	0.25	0.01	12	0.02	2.11	7.85	16.6	8	132.6																		
21	Diagonal 2	18.5		1	0.025	4	1.85	18.5	0.2	0.01	8	0.296	0.8	0.25	0.01	12	0.02	2.17	7.85	17.0	8	136.3																		
22	Diagonal 3	19.5		1	0.025	4	1.95	19.5	0.2	0.01	8	0.312	0.8	0.25	0.01	12	0.02	2.29	7.85	17.9	8	143.6																		
23	Diagonal 4	21		1	0.025	4	2.10	21	0.2	0.01	8	0.336	0.8	0.25	0.01	12	0.02	2.46	7.85	19.3	8	154.5																		
24	Horizontal 1	24		1	0.02	4	1.92	24	0.2	0.01	8	0.384	0.8	0.25	0.01	12	0.02	2.33	7.85	18.3	4	73.1																		
25	Horizontal 2	27		1	0.02	4	2.16	27	0.2	0.01	8	0.432	0.8	0.25	0.01	12	0.02	2.62	7.85	20.5	4	82.1																		
26	Horizontal 3	29		1	0.02	4	2.32	29	0.2	0.01	8	0.464	0.8	0.25	0.01	12	0.02	2.81	7.85	22.0	4	88.2																		
																			Sum																				7677.0	
																			Begge tårn																					15,354

Figure 10.27: Steel quantities axis 3 and 4

Superstructure

Normaltverrsnitt											
Elnr	Beskrivelse	L	B	T	n	Vol	densitet	Vekt pr m (tonn)	Antall m	Tot vekt (tonn)	
1	Topplate 1		1	5.254	0.014	1	0.07	7.85	0.6	3772	2178
2	Topplate 2		1	8.254	0.014	1	0.12	7.85	0.9	3772	3422
3	Topplate 3		1	0.754	0.014	1	0.01	7.85	0.1	3772	313
4	Topplate 4		1	0.771	0.014	1	0.01	7.85	0.1	3772	320
5	Topplate 5		1	2.029	0.014	2	0.06	7.85	0.4	3772	1682
6	Bunnplate 1		1	4.472	0.008	2	0.07	7.85	0.6	3772	2119
7	Bunnplate 2		1	10.3	0.008	1	0.08	7.85	0.6	3772	2440
8	Stiver topplate		1	0.77	0.006	23	0.11	7.85	0.8	3772	3146
9	Stiver skrå		1	0.24	0.01	8	0.02	7.85	0.2	3772	569
10	Stiver bunnplate		1	0.71	0.006	23	0.10	7.85	0.8	3772	2901
11	Skott		1	47.67	0.015	0.05	0.04	7.85	0.3	3772	1059
12	Skott		38.3	0.4	0.01	0.2	0.03	7.85	0.2	3772	907
									5.6		21,055
Forsterket tverrsnitt											
Elnr	Beskrivelse	L	B	T	n	Vol	densitet	Vekt pr m (tonn)	Antall m	Tot vekt (tonn)	
1	Topplate 1		1	5.254	0.02	1	0.11	7.85	0.8	600	495
2	Topplate 2		1	8.254	0.02	1	0.17	7.85	1.3	600	778
3	Topplate 3		1	0.754	0.02	1	0.02	7.85	0.1	600	71
4	Topplate 4		1	0.771	0.02	1	0.02	7.85	0.1	600	73
5	Topplate 5		1	2.029	0.02	2	0.08	7.85	0.6	600	382
6	Bunnplate 1		1	4.472	0.02	2	0.18	7.85	1.4	600	843
7	Bunnplate 2		1	10.3	0.014	1	0.14	7.85	1.1	600	679
8	Stiver topplate		1	0.77	0.006	23	0.11	7.85	0.8	600	500
9	Stiver skrå		1	0.24	0.01	8	0.02	7.85	0.2	600	90
10	Stiver bunnplate		1	0.71	0.006	23	0.10	7.85	0.8	600	461
11	Skott		1	47.67	0.015	0.25	0.18	7.85	1.4	600	842
12	Skott		38.3	0.4	0.01	0	0.00	7.85	0.0	600	0
									8.7		5,214
Totalt for hele bru									6.01		26,269

Figure 10.28: Material quantities superstructure

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10.4 Main cable

Maximum cable force in ULS is: $S_y = 157717$ kN

Cable capacity according to NS-EN 1993-1-11:

$$F_{Rd} = \min \{ F_{uk}/1.5*\gamma_R; F_k/\gamma_R \}; \gamma_R = 1.2 \text{ acc. NA}$$

Braking capacity is governing:

$$F_{Rd} = F_{uk}/1.5*\gamma_R \geq S_y$$

$$F_{uk} = f_{uk} * A_m$$

Using $f_{uk} = 1770$ MPa

$$\text{i.e.: } A_m \geq 160389 \text{ mm}^2$$

Assume tread diameter 5.3 mm, $A_t = 22.06 \text{ mm}^2$

Assume 19 bundles, i.e. no. of treads per bundle: $n_b \geq 382$, choose $n = 4*96 = 384$ treads

No. of treads per bundle: $n_b = 384*19 = 7315$ treads

Total cable area: $A_m = 7315 * 22.06 = 161368 \text{ mm}^2$

We have used $A_m = 170000 \text{ mm}^2$ in the analysis, i.e. we have 5% spare capacity for possible contributions not taken into account in the governing force.

Total length of main cable is app. 4530m. This give a total steel quantity of app. 12000t.

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11 CONSTRUCTION OF PONTOONS AND TOWERS

11.1 General

The construction of the large concrete pontoons and steel towers will mainly be based upon well-known construction techniques from the large marine concrete structures built for the oil and gas industry in the North Sea. In this chapter only main principles are outlined. More details are given in Appendix I.

11.2 Construction of concrete pontoons in dry dock and close to shore

The first phase of the construction of the concrete pontoons is conducted in dry dock. The only operational dock in Norway today is Hanøytangen outside Bergen. This dry dock has a draft depth of 17m inside the dock gate and an area for use of 125 x 125m. It is very suitable for the purpose.

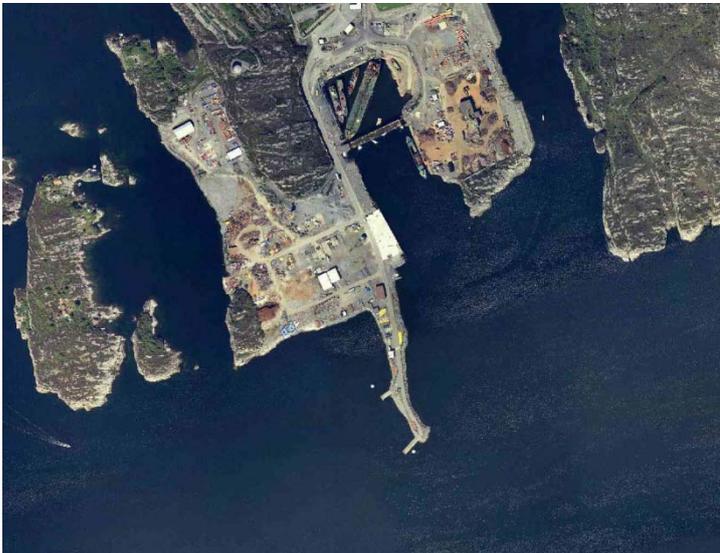


Figure 11.1: Overview Hanøytangen

The bottom slab and adequate height of cell walls (app. 21m) until the bottom part can be floated out of dock with a clearance of 1m, is constructed inside the dock. The cell walls are slip formed.

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Outside of the dry dock, a deep water construction site is established. Here the structure is constructed to a total height of 87m. The advantage with this site is that it is close to the shore. Solid ballast is placed here before tow to site.

11.3 Completion of the concrete pontoons near the bridge site

The pontoon is now ready for towing to site, where it is anchored away from the ship channel. Upper part is here completed.

11.4 Completion of the steel towers

The lower part of the steel towers is prefabricated in a steel yard and arrive Sognefjord on a barge. A division into 4 parts as shown below may be beneficial. The elements will be lifted in place by using offshore cranes, followed by fixing the joints between each part.

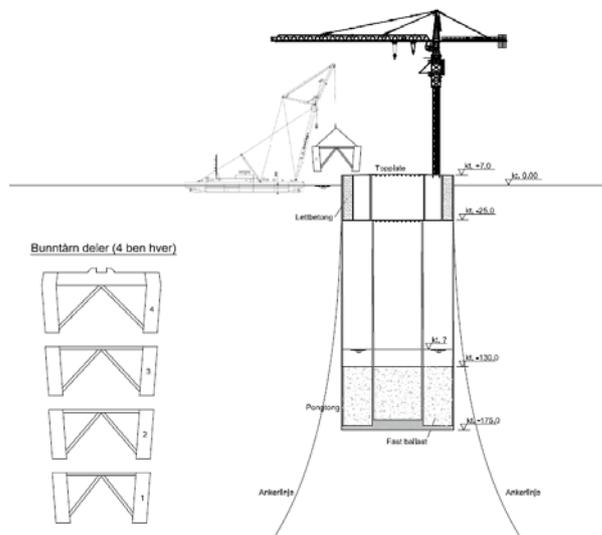


Figure 11.2: Erection of lower elements in towers

The upper parts of the tower will also be prefabricated, but lifted in place by using a large tower crane fixed to the pontoon.

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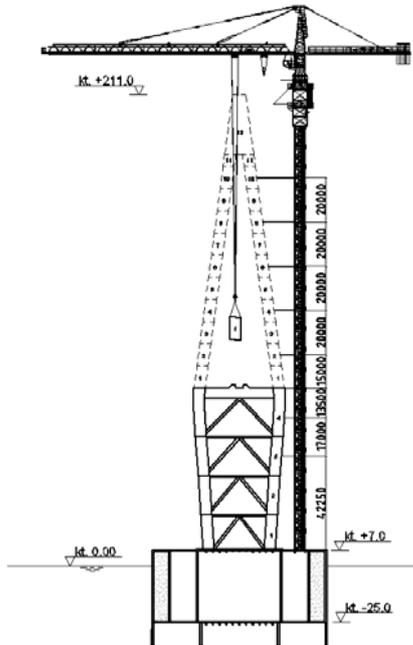


Figure 11.3: Erecting upper elements of the towers

The pontoon with steel tower are now ready for final towing in place and to be attached to the already placed permanent anchor system.

11.5 Completion of concrete towers

The concrete towers on shore are constructed in a traditional way with climbing scaffolding (as on the Hardanger Bridge) or by slip forming.

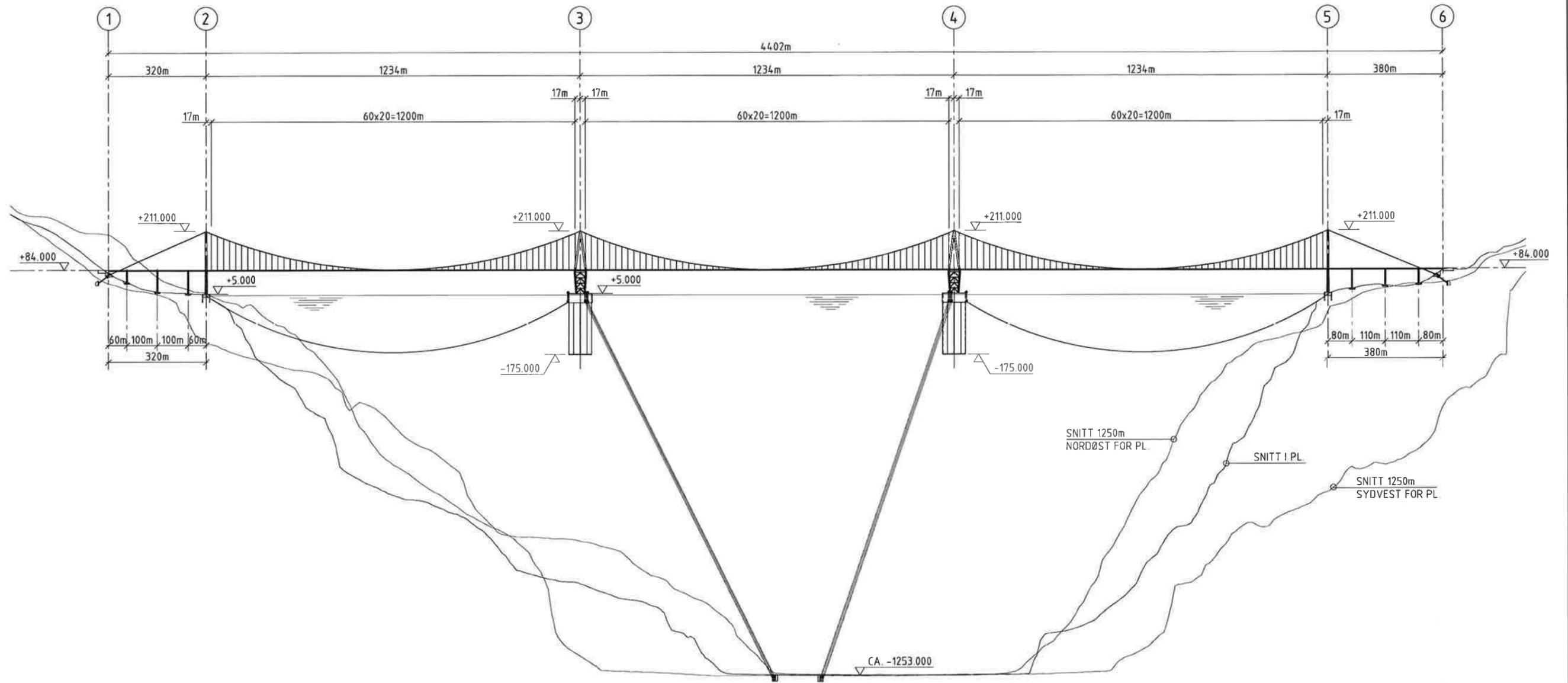
    	Pr.nr./Pr.no 11258	Dok.nr./Doc.no 11258-03	Rev. A
Prosjekt/Project Sognefjorden Feasibility Study of Floating Bridge	Dato/Date 28.11.12	Rev.dato/Date 31.05.13	
Tittel / Title Main Report	Utført/Prep.By SEJ	Rev. av/Rev by SEJ	Side/Page 71

12 REFERENCES

- A: Guidelines/handbooks from Norwegian Public Road Administration
 /1/: Statens vegvesen: Håndbok 185, Bruprosjektering (2011).
 /2/: Statens vegvesen: Håndbok 017: Veg- og gateutforming (2008).
- B: Norwegian codes:
 Referred to in the text where applicable.
- C: Reports/publications
 /3/: Rørbrukryssing Sognefjorden Opedal-Lavik. Estimert på bølger og strøm, SINTEF, 2009-12-03.
- D: Other codes/guidelines/basic documents:
 /4/: PETROLEUMSTILSYNET, Forskrift om utforming og utrustning av innretninger med mer i petroleumsvirksomheten (Innretningsforskriften), April 2010
 /5/: NORSOK, NORSOK standard N-001, Rev. 7, Juni 2010
 /6/: PETROLEUMSTILSYNET, Veiledning til Innretningsforskriften, April 2010
 /7/: DET NORSKE VERITAS, Offshore Standard DNV-OS-E301, Position Mooring, October 2010
 /8/: DET NORSKE VERITAS, Offshore Standard DNV-OS-E302, Offshore Mooring Chain, October 2010
 /9/: DET NORSKE VERITAS, Offshore Standard DNV-OS-E303, Offshore Mooring Fibre Ropes, October 2010
 /10/: DET NORSKE VERITAS, Offshore Standard DNV-OS-E304, Offshore Mooring Steel Wire Ropes, October 2010
 /11/: DET NORSKE VERITAS, Recommended Practice DNV-RP-C103, Column Stabilised Units, April 2012
 /12/: Feasibility Study – crossing of Sognefjorden, Risk assessment part I – frequency analysis – floating bridge, Rambøll

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13 DRAWINGS



A OPPRISS
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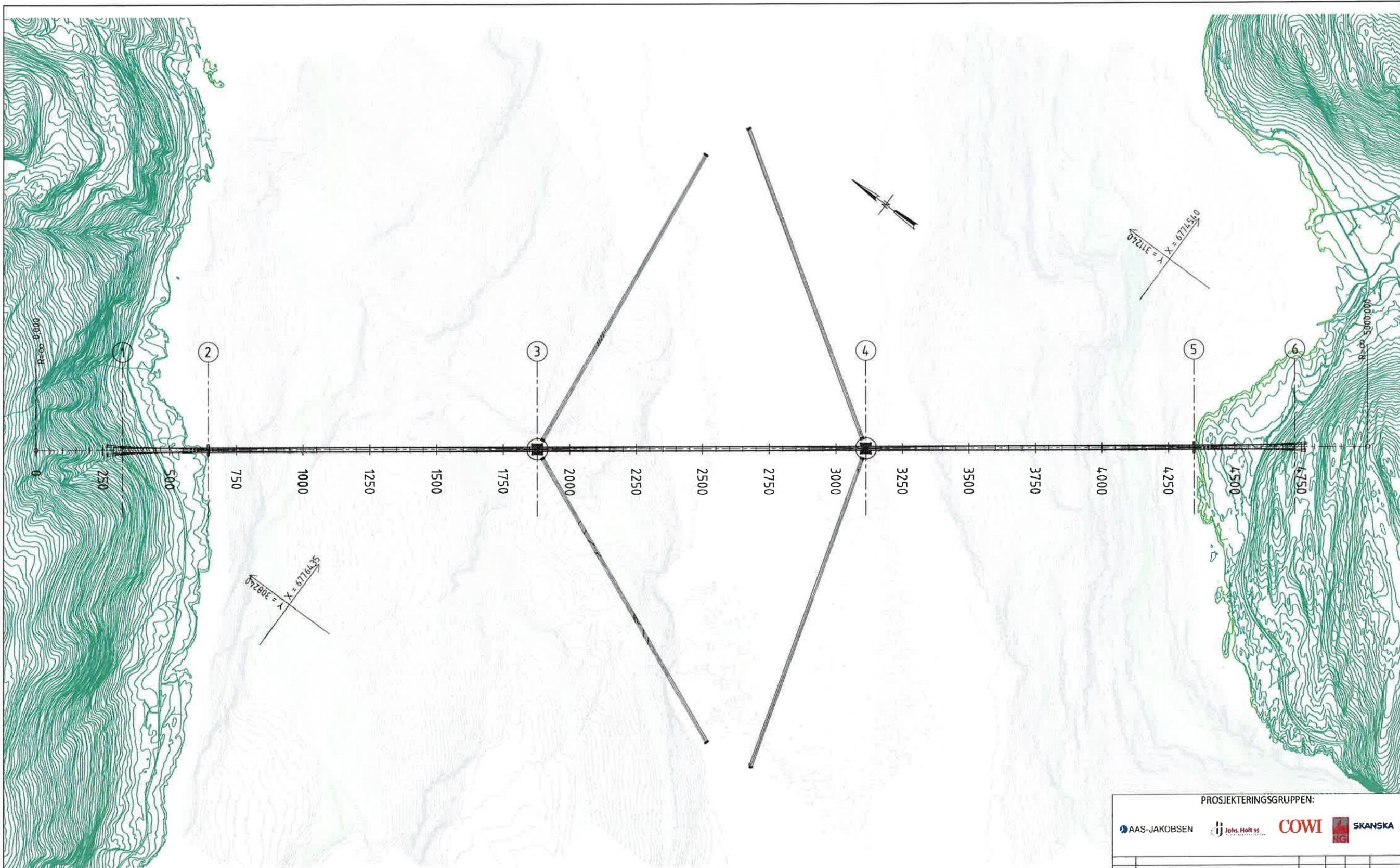
PROSJEKTERINGSGRUPPEN:

AAS-JAKOBSEN Jøhs. Holt as COWI SKANSKA

Revisjon	Revisjonen gjelder	Utdr.	Kont.	Godkjent	Rev dato
Saksnr.					
Tegningsdato		28.11.2012			
Bestiller		SVV Region vest			
Prosjekt for		SVV Region vest			
Prosjekt av		AAS-JAKOBSEN			
ISCF-nummer					
Arbeidsnummer					
Byggesaksnummer					
Helsestadi A1		SOM VST			
Utdr. av	Kontrollert av	Godkjent av	Konsulentarkiv	Tegningsnummer / revisjonsstatus	
SEJ	RHL	SEJ	11258	K001	

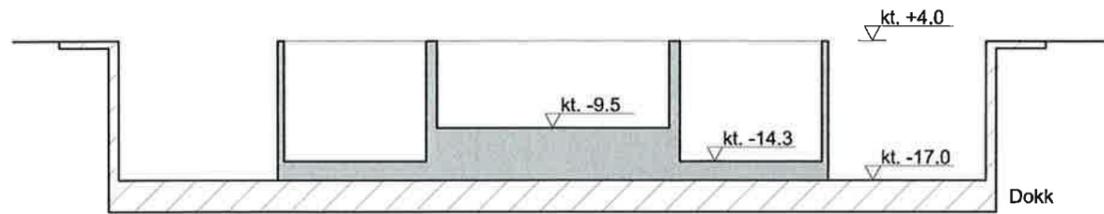
Stolens vegvesen
SOGNEFJORDEN
MULIGHETSSTUDIUM FLYTERBU

OVERSIKT, OPPRISS.



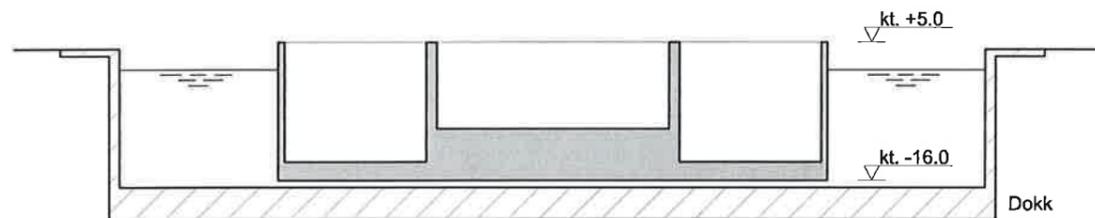
B PLAN
1:7500

PROSJEKTERINGSGRUPPEN:				
Rev. nr	Revisjonen gjelder	Utdr.	Kontroll	Godkjent
				Rev. dato
		Saknr:		
		Tegningsdato:		28.11.2012
Bestiller:		Prosjekt for:		
SOGNEFJORDEN MULIGHETSSTUDIUM FLYTEBRU		BVV Region vest		
Utarbeidet av:		Prosjektert av:		
SEJ		AAS-JAKOBSEN		
Kontrollert av:		PROF-nummer:		
RML		Adressenr:		
Godkjent av:		Byggesaksnummer:		
SEJ		Helsekontroll:		
Konsultant nr:		SOM VISIT:		
11258		Tegningsnummer /		
revisjonsnotat:		K002		



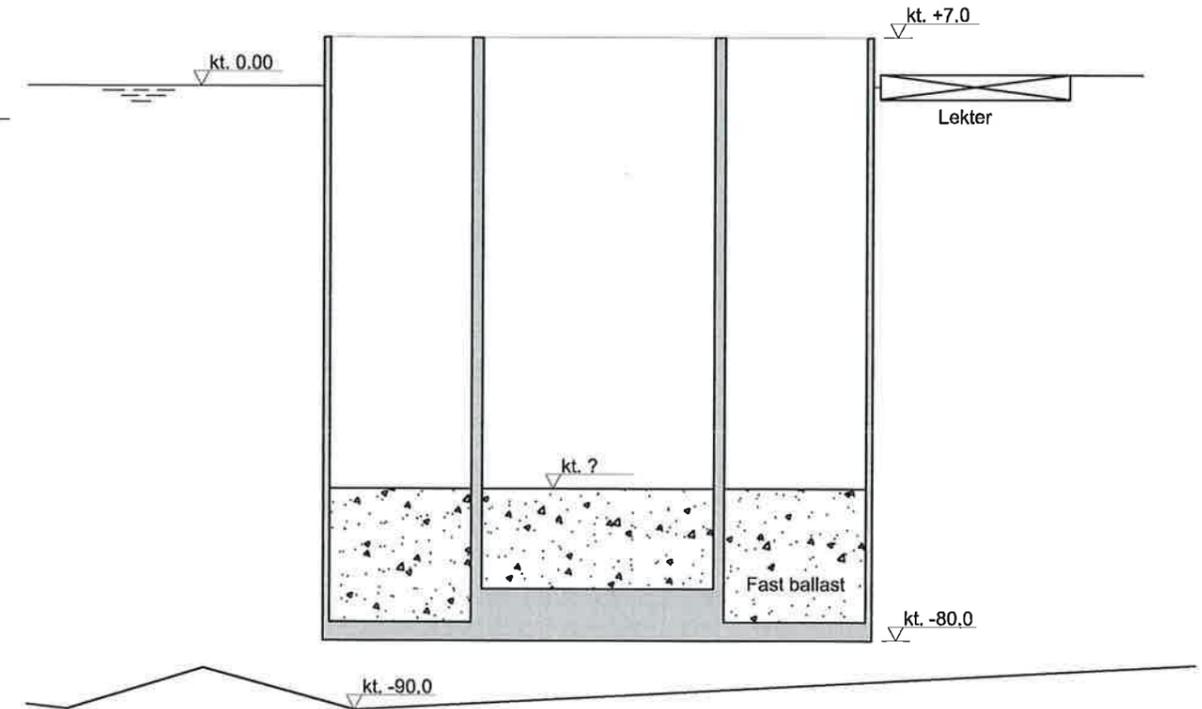
Fase 1 - Tørdokk Hanøytangen

De første 21m av pontongen støpes i dokk med full bunnplattetykkelse



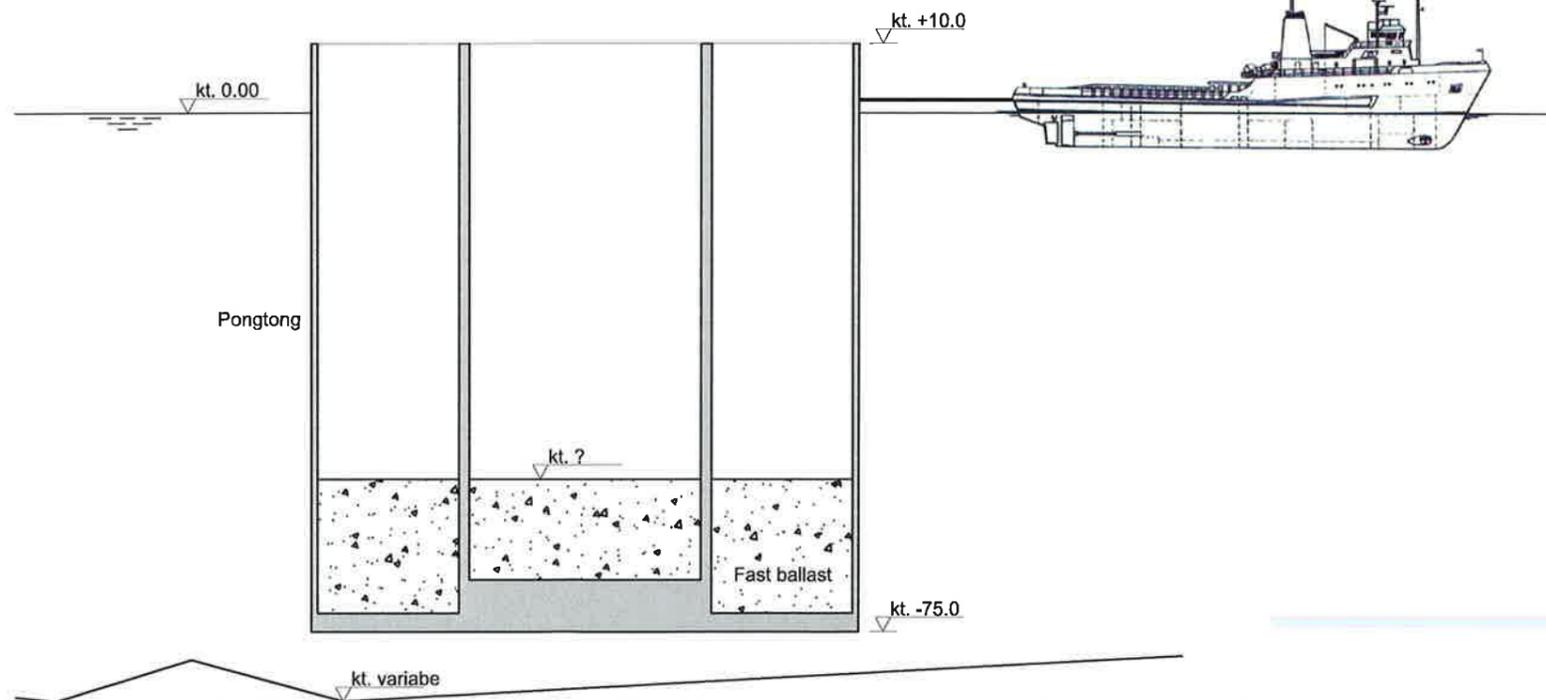
Fase 2 - Ut av dokk

Dokken fylles med vann og pontongen flyter med bunnklaringen på 1m. Fribord er ca. 4m.



Fase 3 - Flytefase ved Hanøytangen

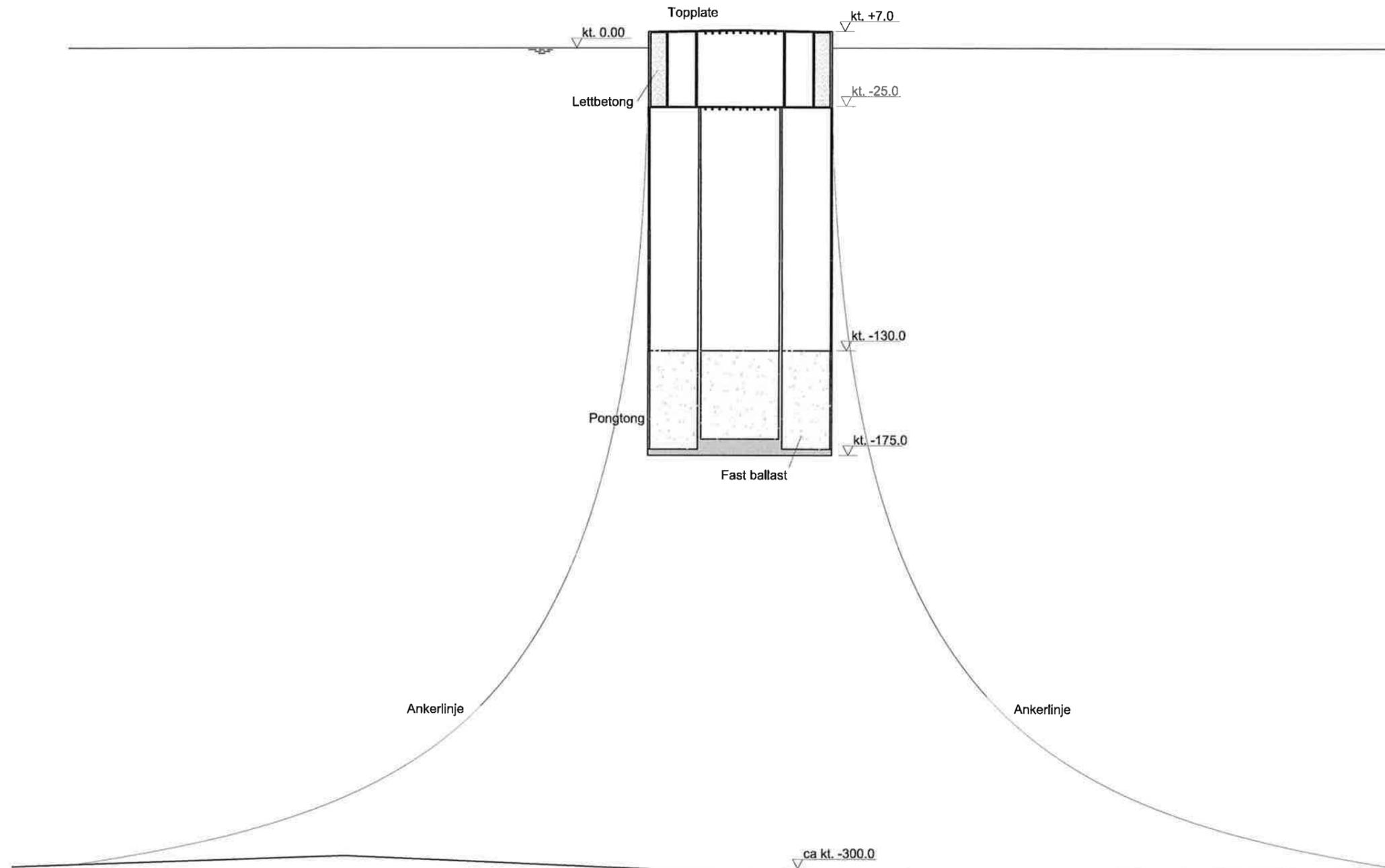
21m høy pontongen taues ut av dokken og til flytekaai med kaidybde ca 85-90m. Pga stabilitetshensyn og ønske om konstant fribord vil pontongen vannballasteres mens man støper. Fast ballast vil også legges inn i denne fasen. Her støpes ytterligere 64m. Nå er total høyde 85m, dypgangen 80m med 5m fribord.



Fase 4 - Tauing til Sognefjorden

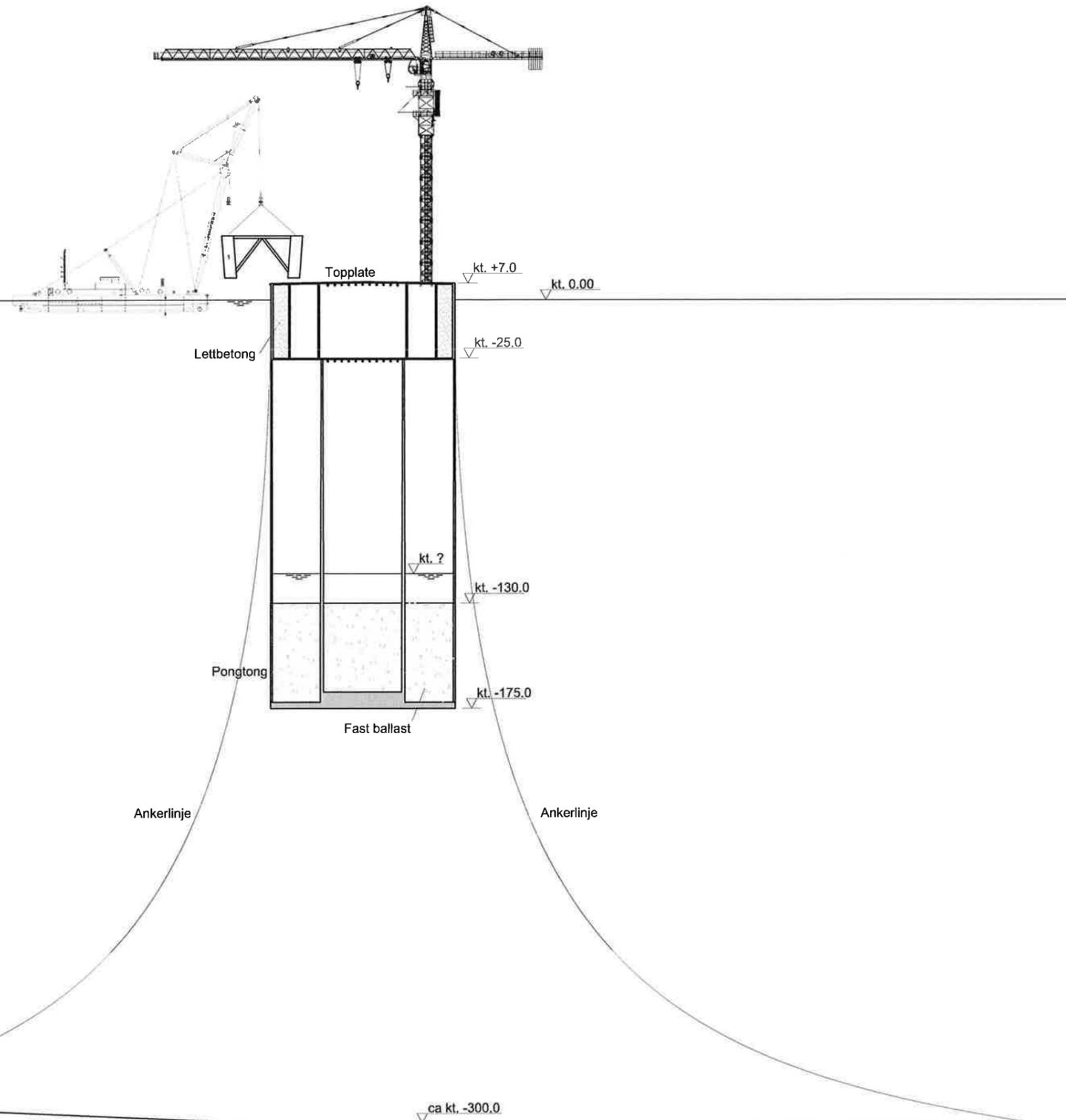
I denne fasen øker fribord til 10m. 2-4 slepebåter tauer pontongen til et eget konstruksjonssted i Sognefjorden med tilstrekkelig dybde til å fullføre pontongen (eksempelvis Brekke).

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01/12.11.2012	For interne kommentarer	SBø	LBj		
TEGNINGSSTATUS	Foreløpig tegning	Arbeidstegning			
	Anbudstegning	Som bygget			
SOGNEFJORDEN MULIGHETSSTUDIE FLYTEBRU					
Byggfaser pontonger og tårn I					
Skanska Norge AS Konstruksjonsavdelingen		Tegning nr.	K011	Rev.	02
12.11.2012					

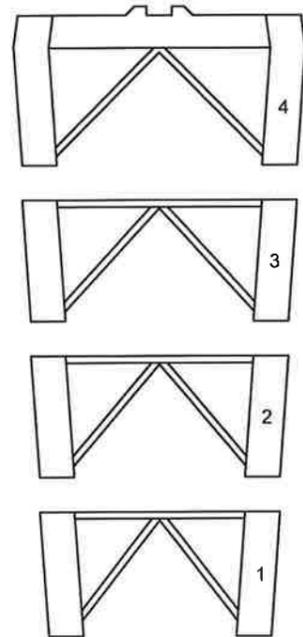


Fase 5 - Fullføre betongsarbeidene i Sognefjorden
 Pontongen forankres i en på forhånd utlagt mooring i Sognefjorden nær fjordskrysningsted og som har vanddybde minst 220m. Cellegliden fullføres. Lokket støpes ved bruk av "tapt forskaling".

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SOGNEFJORDEN MULIGHETSSTUDIE FLYTEBRU					
Byggfaser pongtonger og tårn II					
Skanska Norge AS		Konstruksjonsavdelingen		Tegning nr.	02
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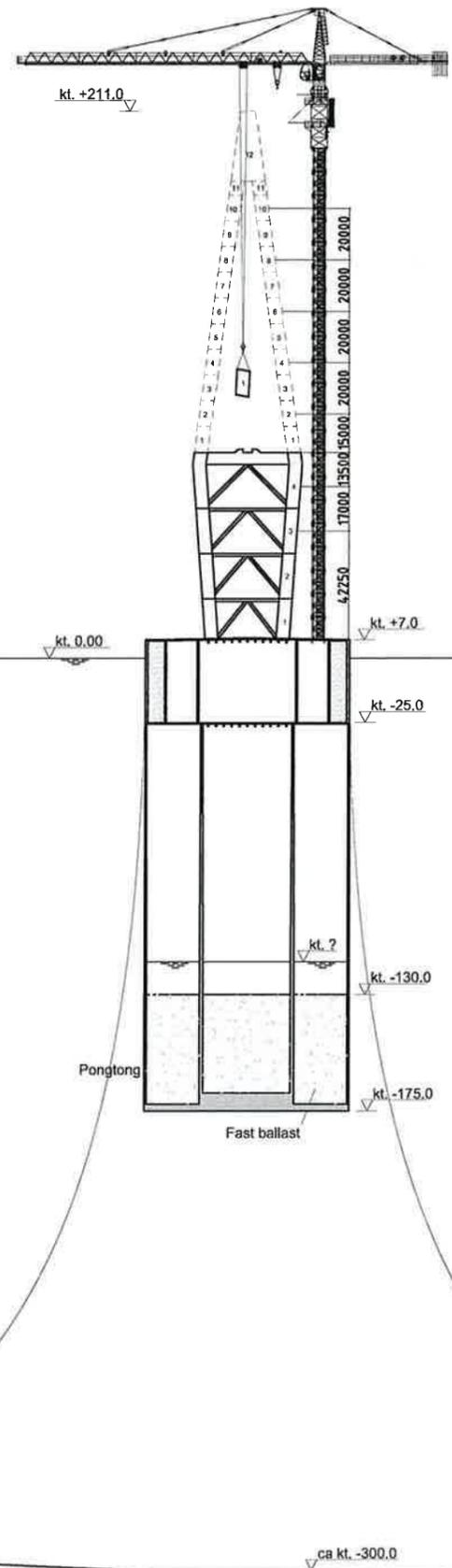
Bunntårn deler (4 ben hver)



Fase 6 - Nedre del av ståltårn monteres med flytekran

Ved bruk av flytekranen (for eks Taklift 4 eller tilsvarende) monteres nedre del av tårnet. Den nedre delen kan deles opp i f.eks. 4 deler, begrenset av kranes løftekapasitet, høyde på kranbom og rekkevidde. Hver del består av: 4 ben, 8 diagonaler, 4 horisontale elementer. Siste del, nr. 4, er den tyngste på grunn av topp box profilen. På grunn av stabilitetsforhold må man vannballastere etter at hver del er montert. Forutsetter at det skal være min. 7m fribord i alle faser. Tårnkran benyttes til mindre løft. Tårnedelene prefabrikeres på et verft og transporteres til byggesstedet på lekter.

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01	12.11.2012	For Interne kommentarer	SBa	LBj	
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Byggefaser pongtonger og tårn III					Skisse
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Dato: 12.11.2012			Dokument nr.: 41602004		

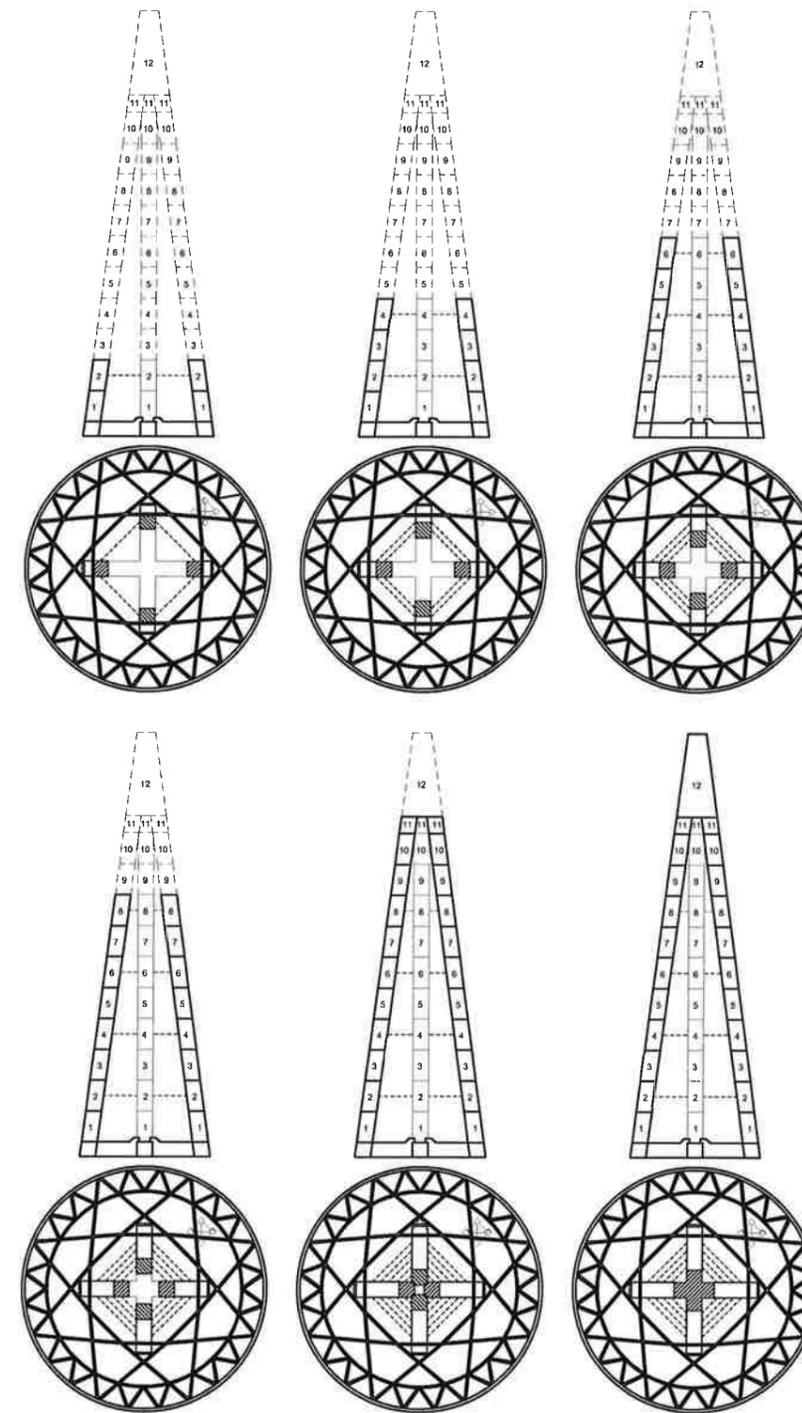


Kapasitet Tårnkran

Heavy-Load 3150 HC 60

Technical data	Lifting capacity	Max. hook height
	Driving units	
Working radius	Lifting capacity	
39.2 m	60.000 kg	
50.7 m	60.000 kg	
62.5 m	50.000 kg	
74.2 m	38.000 kg	
80.0 m	32.000 kg	

Fullføring av ståltårnet: sekvensielle faser

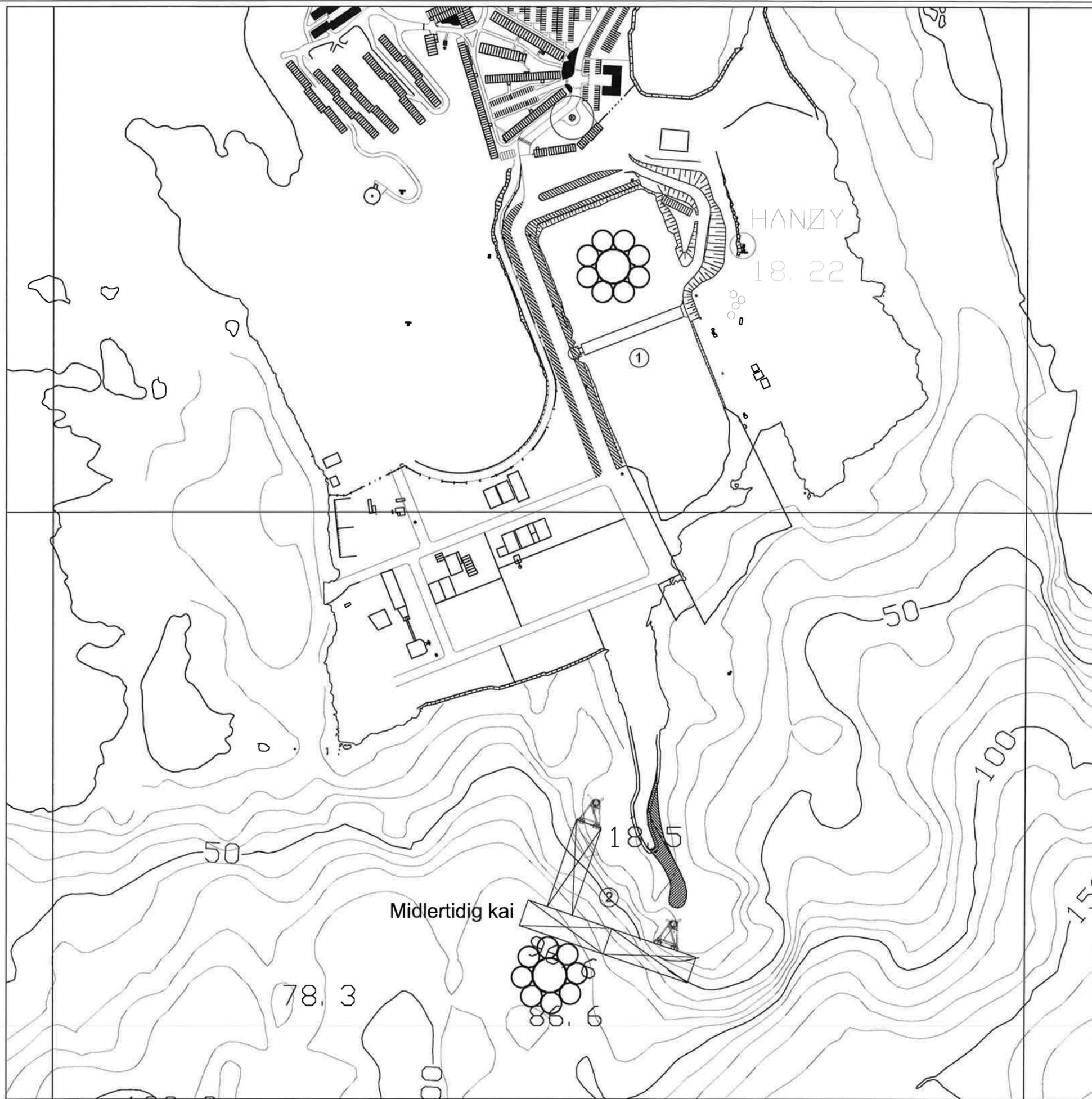


----- Midlertidige avstivninger
 Tårnkran

Fase 7 - Øvre del av ståltårn

I den siste fasen fullføres monteringen av ståltårnet. Tårnkranen (for eks. Liebherr 3150 HC 60 eller tilsvarende) skjøtes opp suksessivt og stives av med ståltårnet. Første innfestningspunkt skal plasseres ca 50m over OK betonglokk. De andre kan plasseres med ca 20m mellomrom. På høyre siden vises sekvensielle byggefaser. Del nr.12 må deles i flere deler for å kunne løftes opp. Midlertidig avstivning i ulike høyder mellom tårnkassene vil være nødvendig. Det vannballasteres i alle faser for å kontrollere fribord og eventuelt skjevhet av tårnet.

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02 26.11.2012	Div korreksjoner	CL	LBj
01 12.11.2012	For Interne kommentarer	SBa	LBj
Rev	Dato	Forfatter	Godkj. av
TEGNINGSSTATUS		Foreløpig tegning	Arbeidstegning
		Anbudstegning	Som bygget
SOGNEFJORDEN MULIGHETSSTUDIE FLYTEBRU			
Byggefaser pongtonger og tårn IV			
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		K014	02
		Dato	12.11.2012
		Skanska pro. nr.	874822008



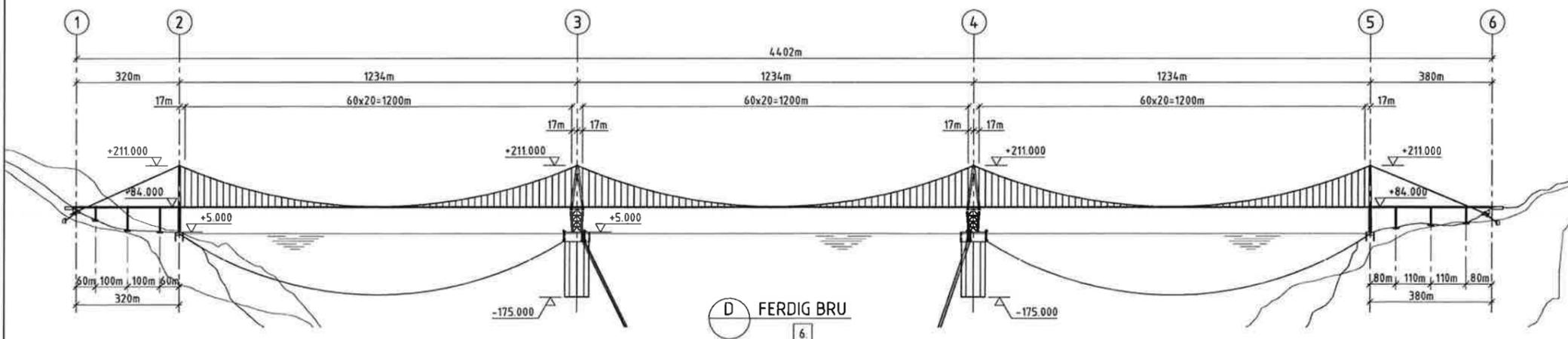
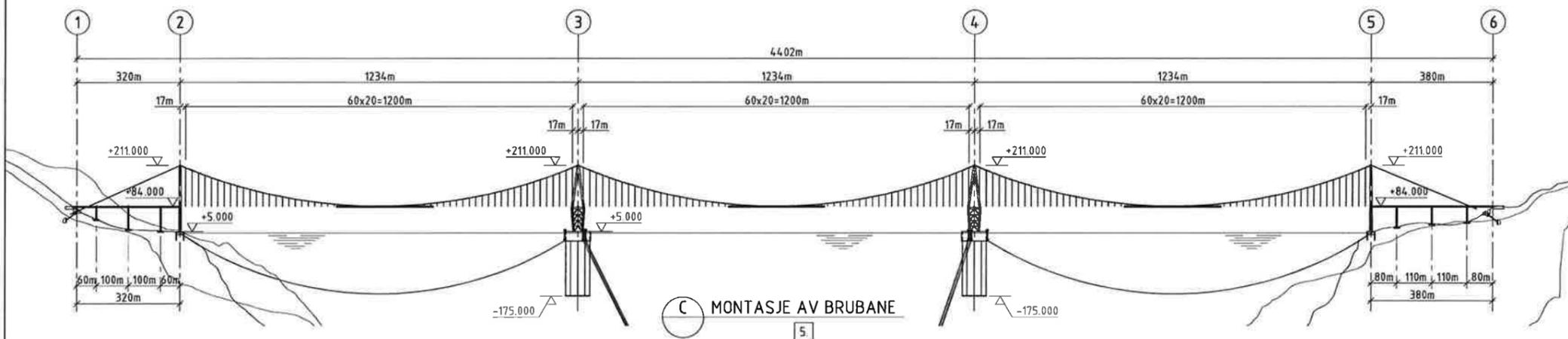
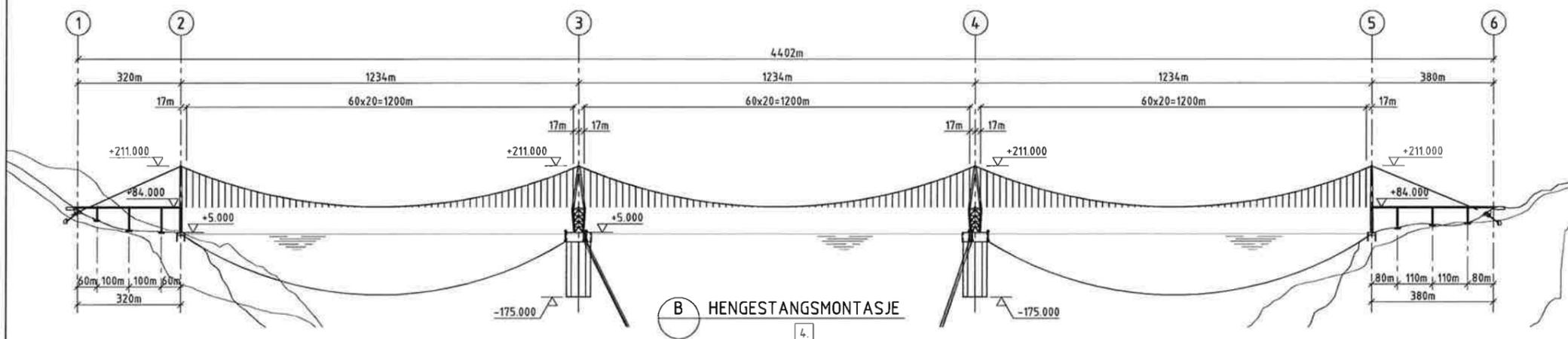
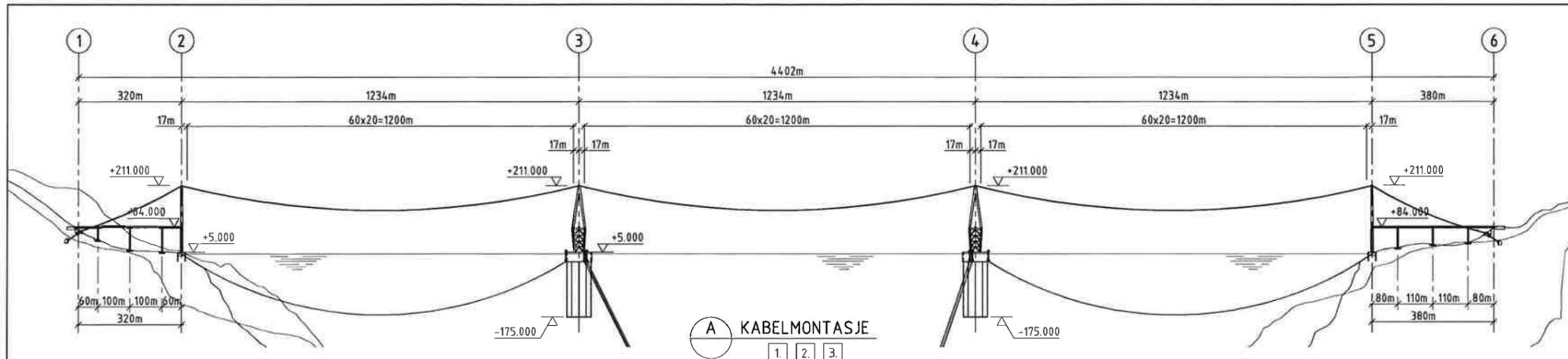
Bemerkninger

Byggingen av pontongene starter i Hanøytangen dokk. Denne dokka har vanndybde -17m og et areal på 125mx125m innenfor dokkporten (Pos 1). Referer til Fase 1 og 2.

I pos 2 etableres igjen en midlertidig kai som ble brukt på 1990 tallet for bygging av Troll olje plattformen. Referer til Fase 3.

Posisjon 2 har vanndybde ca 80-100 meter.

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02 26.11.2012	Dlv korreksjoner	CL	LBj
01 12.11.2012	For interne kommentarer	SBa	LRj
Rev	Dato	Rev	Dato
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SOGNEFJORDEN MULIGHETSSTUDIE FLYTEBRU			
Byggefaser pontonger og tårn Dokk/kai			
Skanska Norge AS Konstruksjonsavdelingen		Tegning nr. K015	Rev. 02
SKANSKA		Dato 12.11.2012	Revisjon nr. 416255104



KABELMONTASJE

1. TÅRN I AKSE 2 OG 5 TILBAKETREKES. TÅRN I AKSE 3 OG 4 FORHÅNDSINNSTILLES VED BALLASTERING AV PONTONGER.
2. SIDESPENN MONTERES FRA LAND ELLER FRA SJØEN.
3. KABEL SPINNES/MONTERES

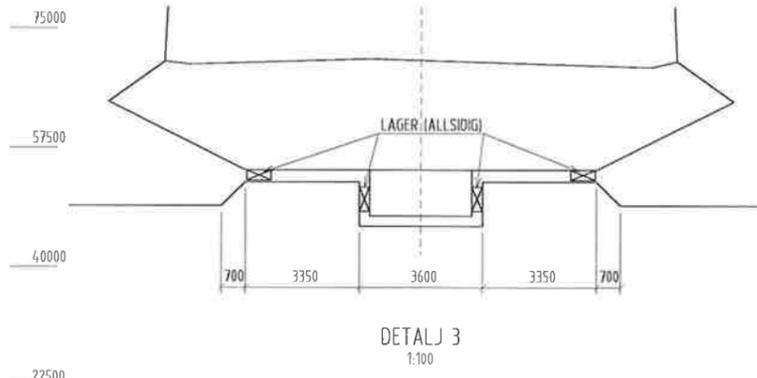
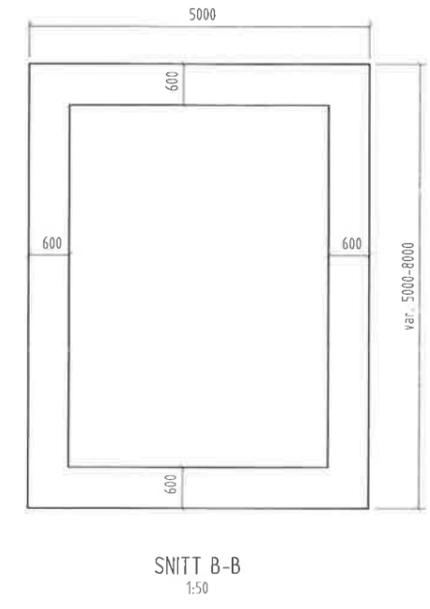
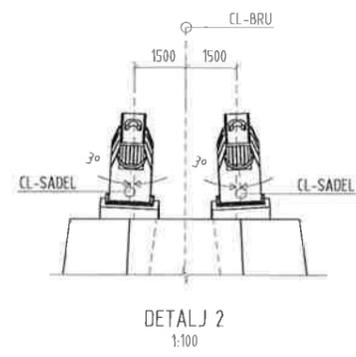
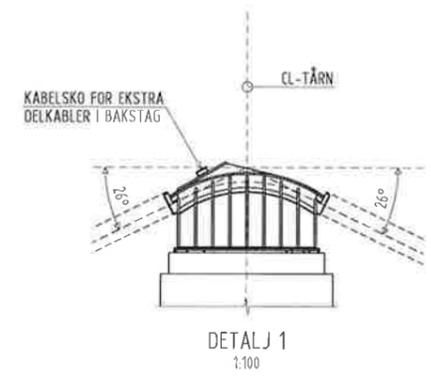
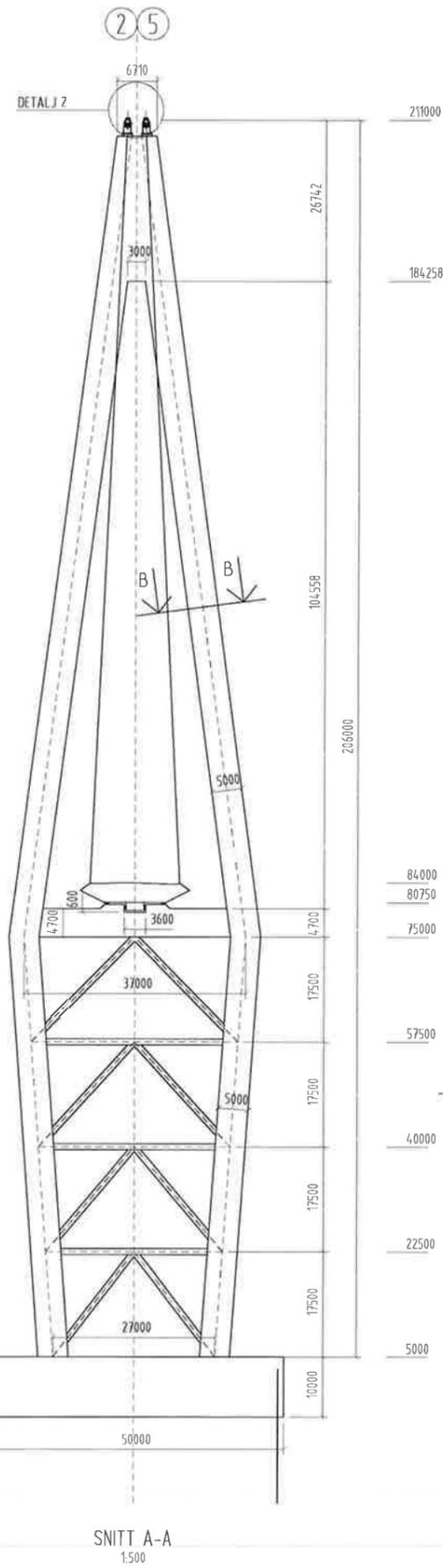
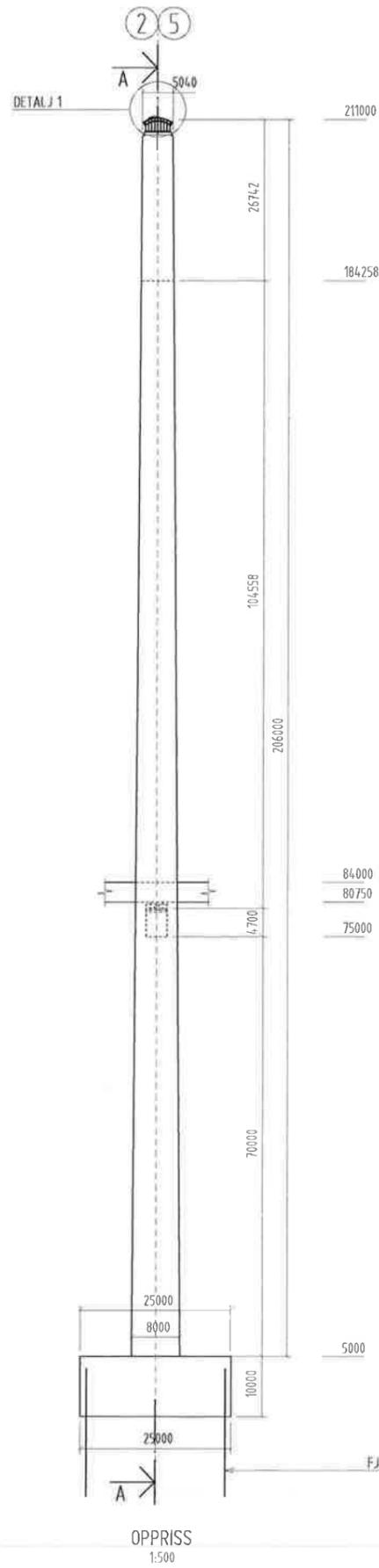
HENGESTANGMONTASJE

4. HENGESTANGSFESTER OG HENGESTENGER MONTERES

MONTASJE AV BRUBANE

5. AVSTIVNINGSBÆRER LØFTES I SEKSJONER OPP FRA LEKTER/BÅT. MONTASJEN UTFØRES SAMTIDIG I ALLE 3 SPENN, STARTER FRA SENTER SPENN. TIL SLUTT SVEISES SEKSJONENE SAMMEN. UTSTYR MONTERES OG SLITELAG UTFØRES
6. FERDIG BRU.

PROSJEKTERINGSGRUPPEN:					
AAS-JAKOBSEN		Johs. Holt as		COWI SKANSKA	
Revisjon	Revisjonen gjelder	Utdato	Kontrollert	Godkjent	Rev. dato
		Saksnr			
Statens vegvesen		Tegningsdato	28.11.2012		
SOGNEFJORDEN		Prosjekt	BYV Region vest		
MULIGHETSSTUDIUM FLYTEBRU		Prosjekt nr	AAS-JAKOBSEN		
BYGGEFASER		HØI-nummer			
HOVEDKABEL OG BRUBANE.		Arkivnummer			
		Byggesaksnummer			
		Håndtekket av	SDM VST		
Utarbeidet av	Kontrollert av	Godkjent av	Konsulentnr	Tegningsnummer /	Revisjonsstatus
LRE	SEJ	SEJ	1758		K016



Tegn. nr.	11258-K021
Fig.	-

MERKNADER

- Belong Fasthetsklasse B45 SV-40
- Armering B500NC, NS3576-3
- Bestandighetsklasse MF40
- Lufthold 5,0+-1,5%
- Tilslagsstørrelse D100=22mm
- Kontrollklasse Ufvidet kontroll, NS3420
- FORSKALING
- Synlige flater Bordforskaling
- Avfasing 20mm trekantlist på utstående hjørner

BENEVNELSER

Alle mål gis i mm. Alle koter gis i m.
 Utførelse skal være i samsvar med Statens vegvesens prosesskode 2 - hovedprosess 8
 Toleranser: nøyaktighetsklasse B, prosesskode -2

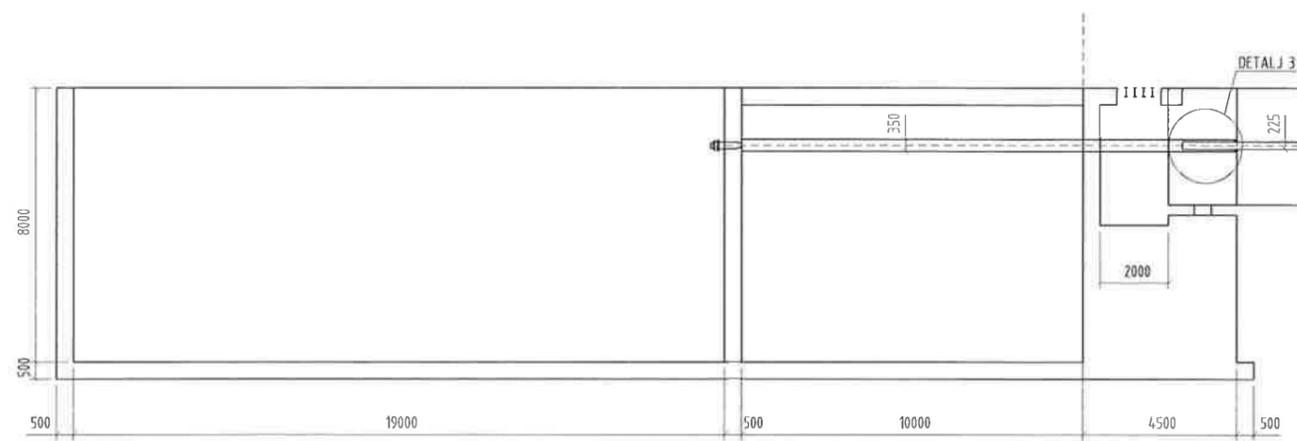
HENVISNINGER

Tegn.nr. 11258-K041 Avslivningsbærer

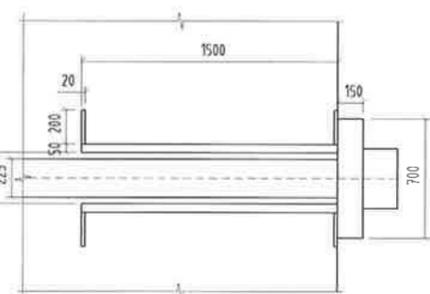
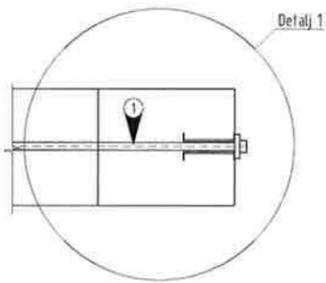
PROSJEKTERINGSGRUPPEN:			
AAS-JAKOBSEN	Johs. Holt as	COWI	SKANSKA
Revisjon	Revisjonen gjelder	Utørt	Kontr. Godkjent Rev. dato
-	-	-	-
Sinnor			
Tegningsdato		28.11.2012	
Bestiller		XX	
Statens vegvesen		Produkt for	
Mulighetsstudie for kryssing av Sognefjorden		Produkt av	
Parsell Sognefjorden		Johs. Holt AS	
Sognefjorden Flytebru		PROJ-nummer	
Tårn akse 2 og 5		Arkivnummer	
Form		Byggeskisse	
Utørbedt av		Målestokk A1	
Kontrollert av		Tegningsnummer/	
CLH		revisjonsbærer	
Godkjent av		11258-K021	
Korutentarkov		1154/3117011	

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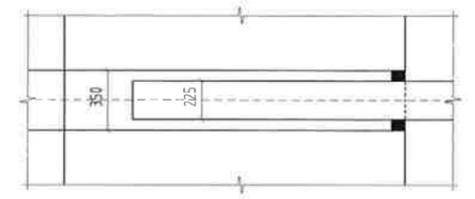




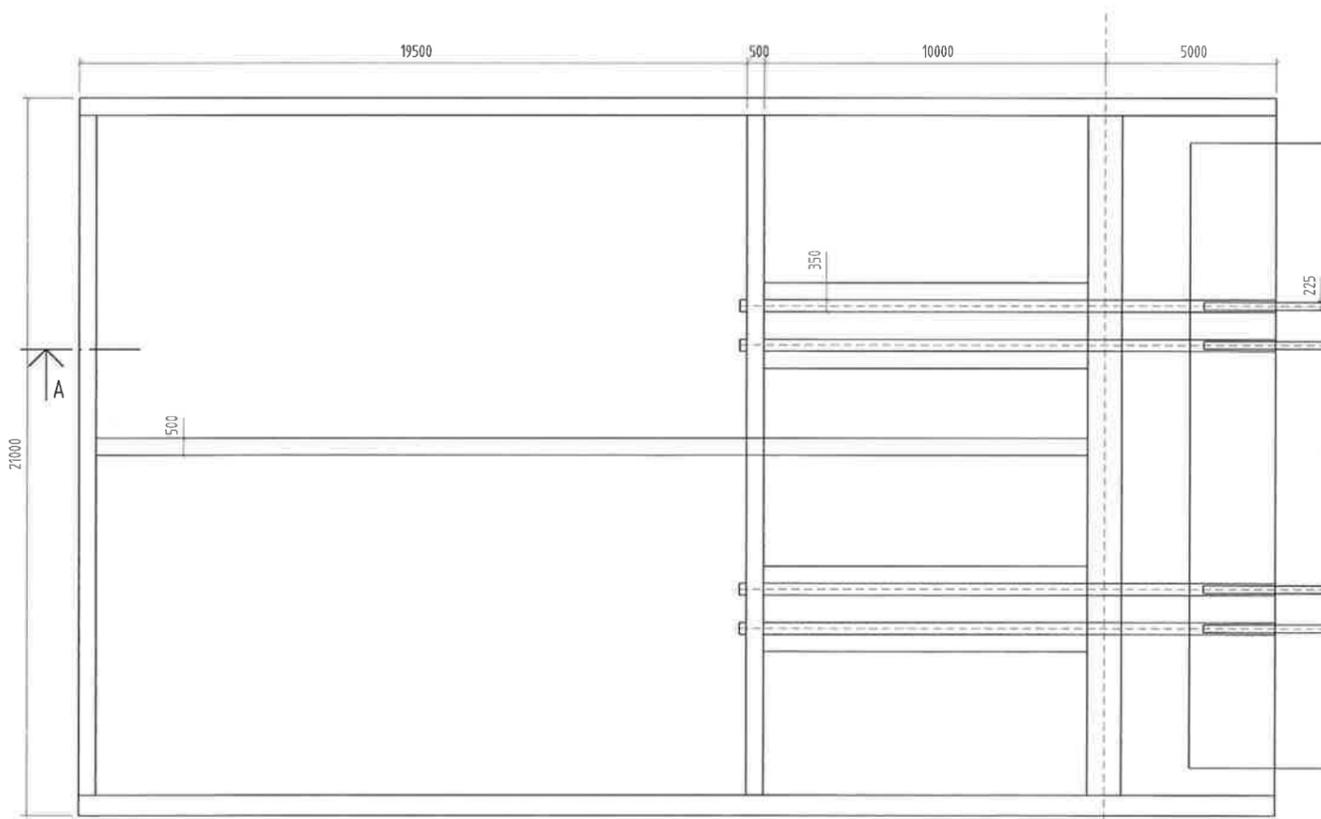
SNITT A-A
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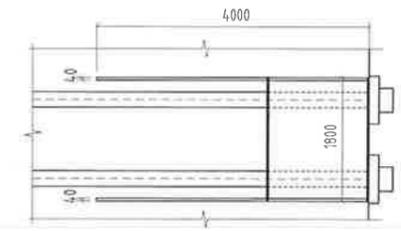
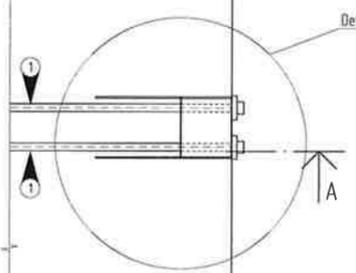
DETALJ 1
1:20



DETALJ 3
1:20



PLAN
1:100



DETALJ 2
1:50

FORKLARINGER

- ① Horisontalfjær 4 stk. 6-61 spennkabel 1860 MPa. Oppspenningskraft 7,1 MN pr. kabel.

11258-K023

MERKNADER

- Betong Fasthetsklasse B45 SV-40
- Armering B500NC, NS3576-3
- Bestandighetsklasse MF40
- Lufthold 5,0 ± 1,5%
- Tilslagsstørrelse Ø100-22mm
- Kontrollklasse Utvidet kontroll, NS3420
- FORSKALING
- Synlige flater Bordforskaling
- Avfasing 20mm trekantlist på utstående hjørner

BENEVELSER

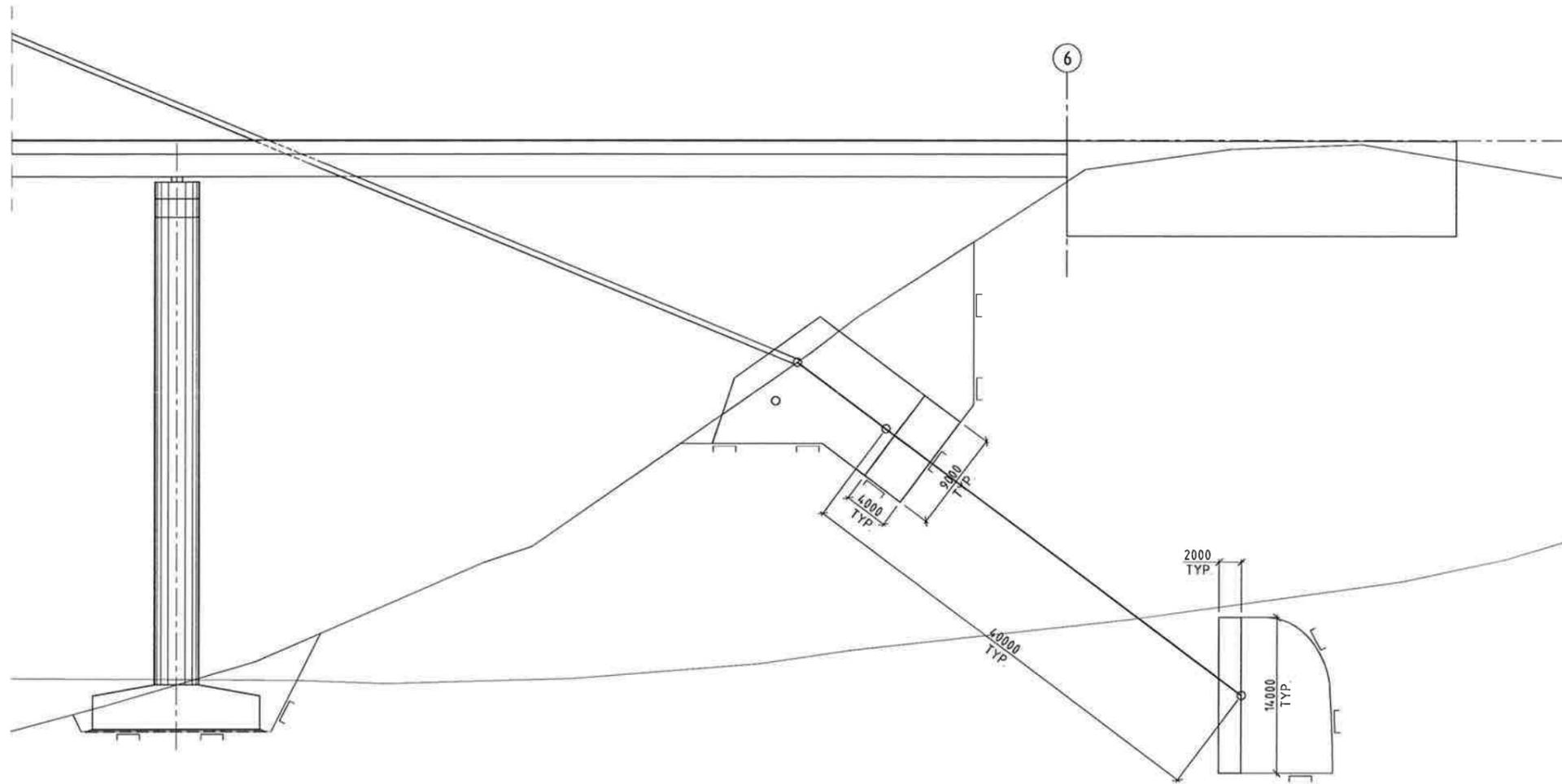
- Alle mål gis i mm. Alle koter gis i m.
- Utførelse skal være i samsvar med Statens vegvesens prosesskode 2 - hovedprosess 8
- Toleranser: nøyaktighetsklasse B, prosesskode -2

HENVISNINGER

PROSJEKTERINGSGRUPPEN:				
AAS-JAKOBSEN	Johs. Holt AS	COWI	SKANSKA	
Revisjon	Revisjonen gjelder	Utarb	Korrig	Godkjent
-	-	-	-	-
Statens vegvesen		Saksnr		
Mulighetsstudie for kryssing av Sognefjorden		Tegningsdato 28.11.2012		
Parsell Sognefjorden		Revisjon XX		
Sognefjorden Flytebru		Prosjekt nr. 11258-K023		
Landkar/ Horisontalfjær		Produkt av Johs. Holt AS		
Form		PRJF-nummer XXX		
		Arkivnummer		
		Byggesaksnummer xxx		
		Målestokk A1		
		Som vist		
Utarbeidet av	Kontrollert av	Godkjent av	Konsulentarkiv	Tegningsnummer/
CLH	GJE	PM	1154/30120011	revisjonsstatus
				11258-K023

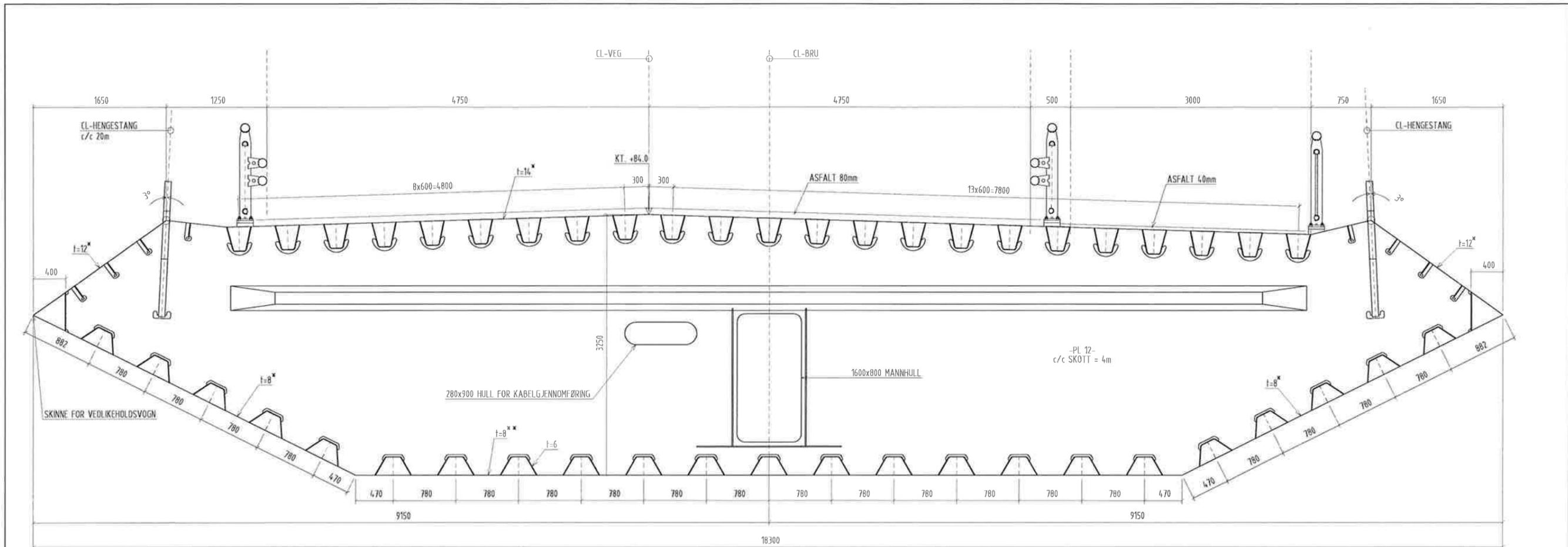
© 2011 JAKOBSEN, Mulighetsstudie for kryssing av Sognefjorden, Tegningsnummer 11258-K023.dwg - CLH - 28.11.12 - 16:27:28 - Referanser: jh-holten.dwg, snitt.dwg, plan.dwg





A OPPRISS
1:250

PROSJEKTERINGSGRUPPEN:				
AAS-JAKOBSEN		Johns.Molt as		COWI
				SKANSKA
Revisjon	Revisjonen gjelder	Utarb	Kontor	Godkjent
				Rev dato
Saksnr:		Tegningsdato		
		28.11.2012		
Statenes vegvesen		Bestiller		
		BYV Region vest		
SOGNEFJORDEN		Prosjekt av		
MULIGHETSSTUDIJE FLYTEBRU		AAS-JAKOBSEN		
FORANKRING HOVEDKABEL		PROJ-nummer		
		Aktivnummer		
		Byggenummer		
		Helseikk A1		
Utarbeid av	Kontrollert av	Godkjent av	Konsulentnr	Tegningsnummer /
SEJ	RM	SEJ	11258	revisjonskrets
				K031



TYPISK TVERRSNITT
1:25

- * 150mm til hver side av akse 3 og 4, l=20mm
- ** 150mm til hver side av akse 3 og 4, l=14mm

Tege. nr.	Rev.
11258-K041	-

MERKNADER

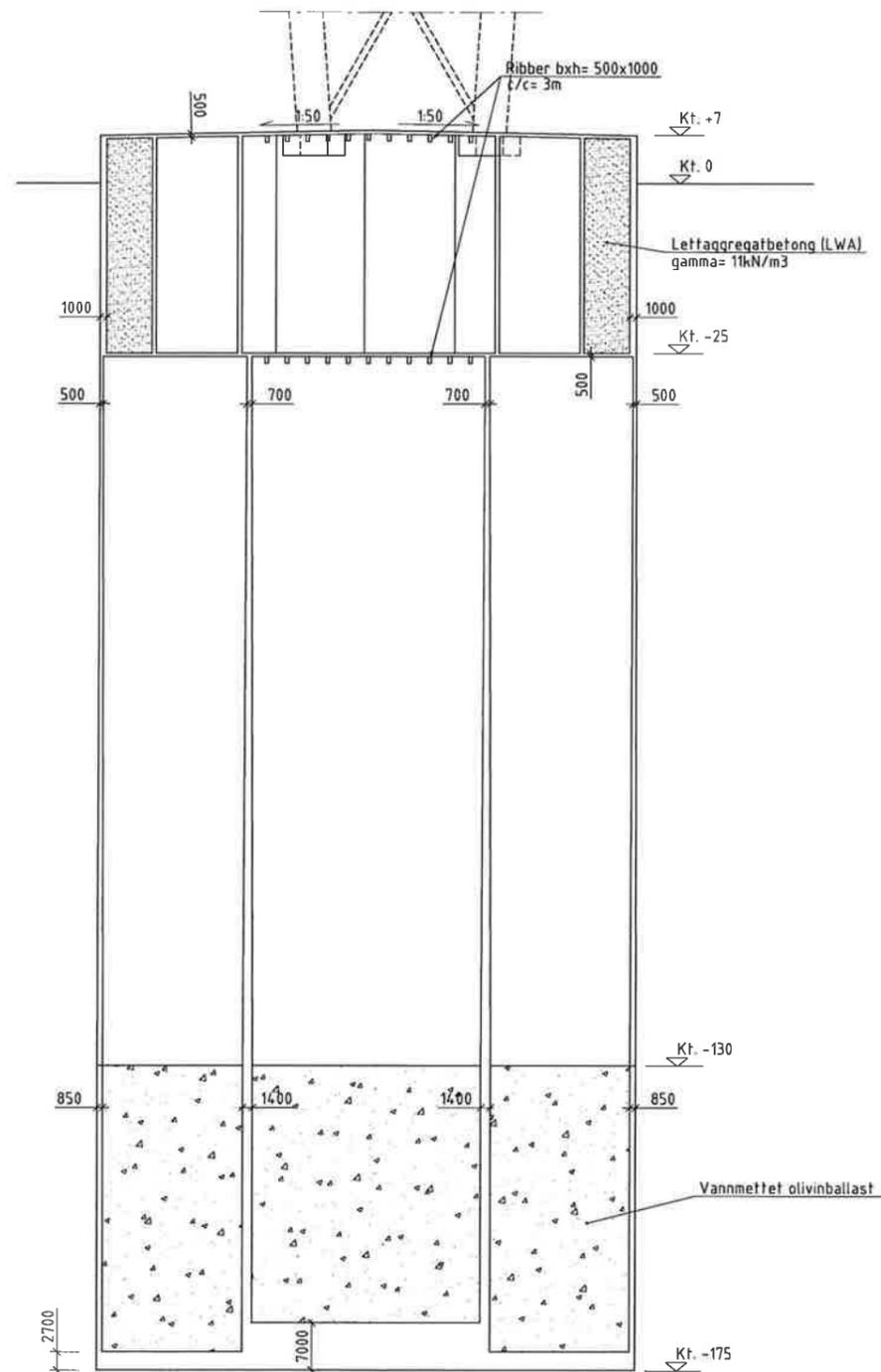
Stålsort generelt:
S355 N/NL
150m til hver side av akse 3 og 4 S420 N/NL.

BENEVNELSER
Alle mål gis i mm. Alle koter gis i m.
Ufførelse skal være i samsvar med Statens vegvesens
prosesskode 2 - hovedprosess 8
Toleranser: nøyaktighetsklasse B, prosesskode -2

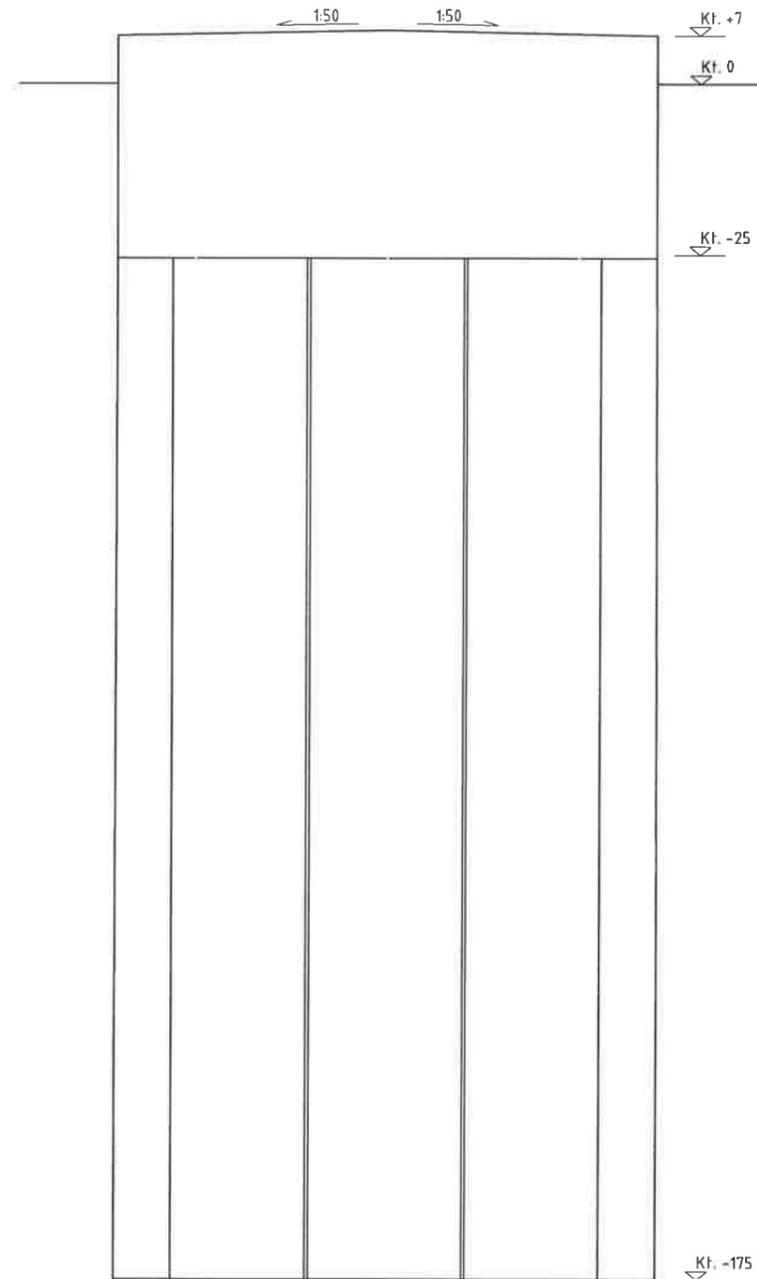
HENVISNINGER



PROSJEKTERINGSGRUPPEN:				
AAS-JAKOBSEN		Johs. Holt AS		SKANSKA
Revisjon	Revisjonen gjelder	Utdr.	Kontr.	Godkjent
-	-	-	-	-
Statens vegvesen Mulighetsstudie for kryssing av Sognefjorden Parsell Sognefjorden Sognefjorden Flytebru Avstivningsbærer Form				
Tegningsdato		28.11.2012		
Bestiller		XX		
Prosjekt nr.		XX		
Prosjekt av		Johs. Holt AS		
PROJ-nummer		XXX		
Arbeidsnummer		XXX		
Byggesaksnummer		XXX		
Målestokk A1		Som vist		
Utdr. av	Kontrollert av	Godkjent av	Konsulentarkiv	Tegningsnummer / revisjonsstokk
CLH	GJE	PNL	1154 / 30120011	11258-K041



SNITT A-A
1:500
Kfr. tegn. K102

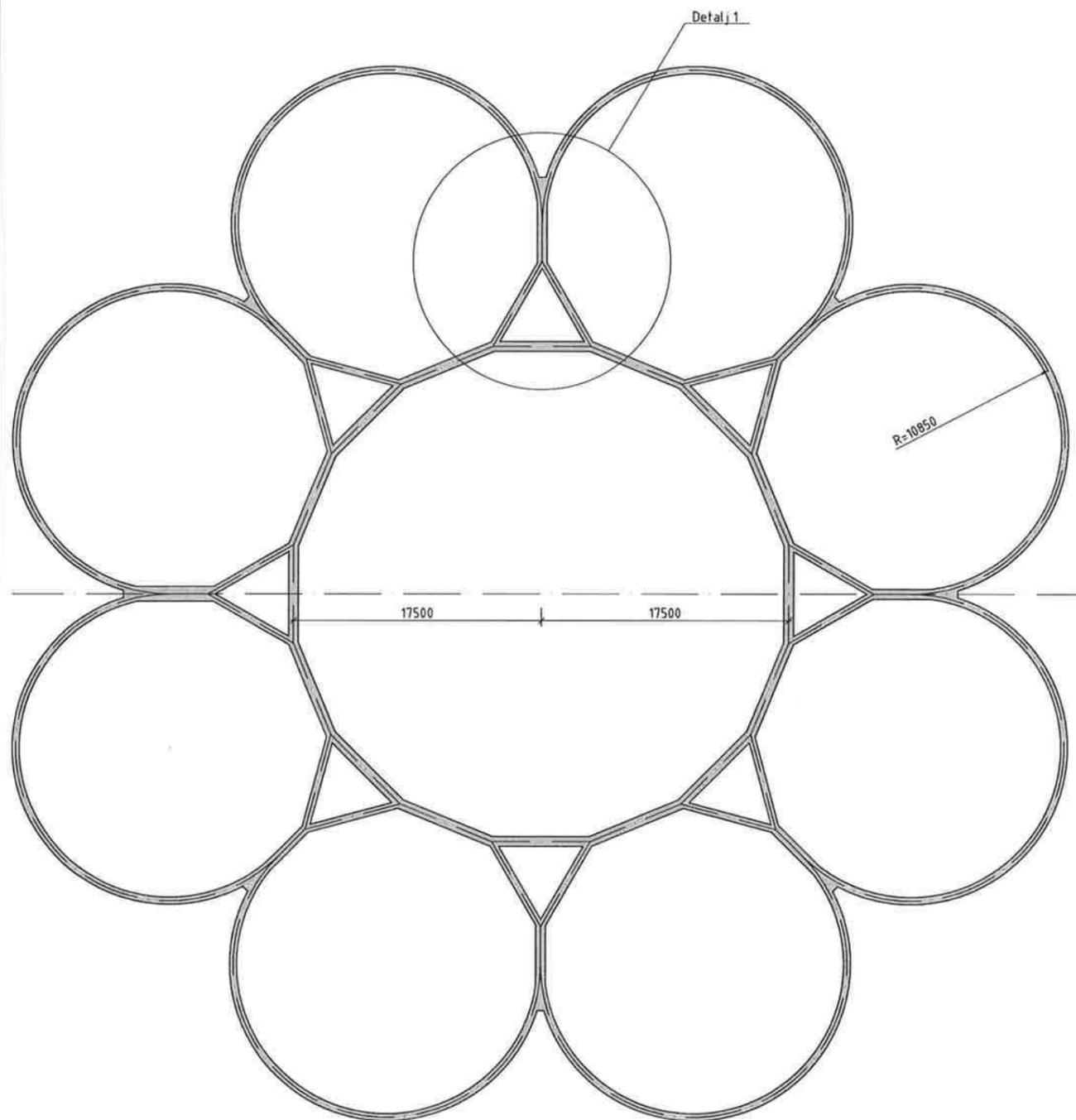


OPPRISS B-B
1:500
Kfr. tegn. K102

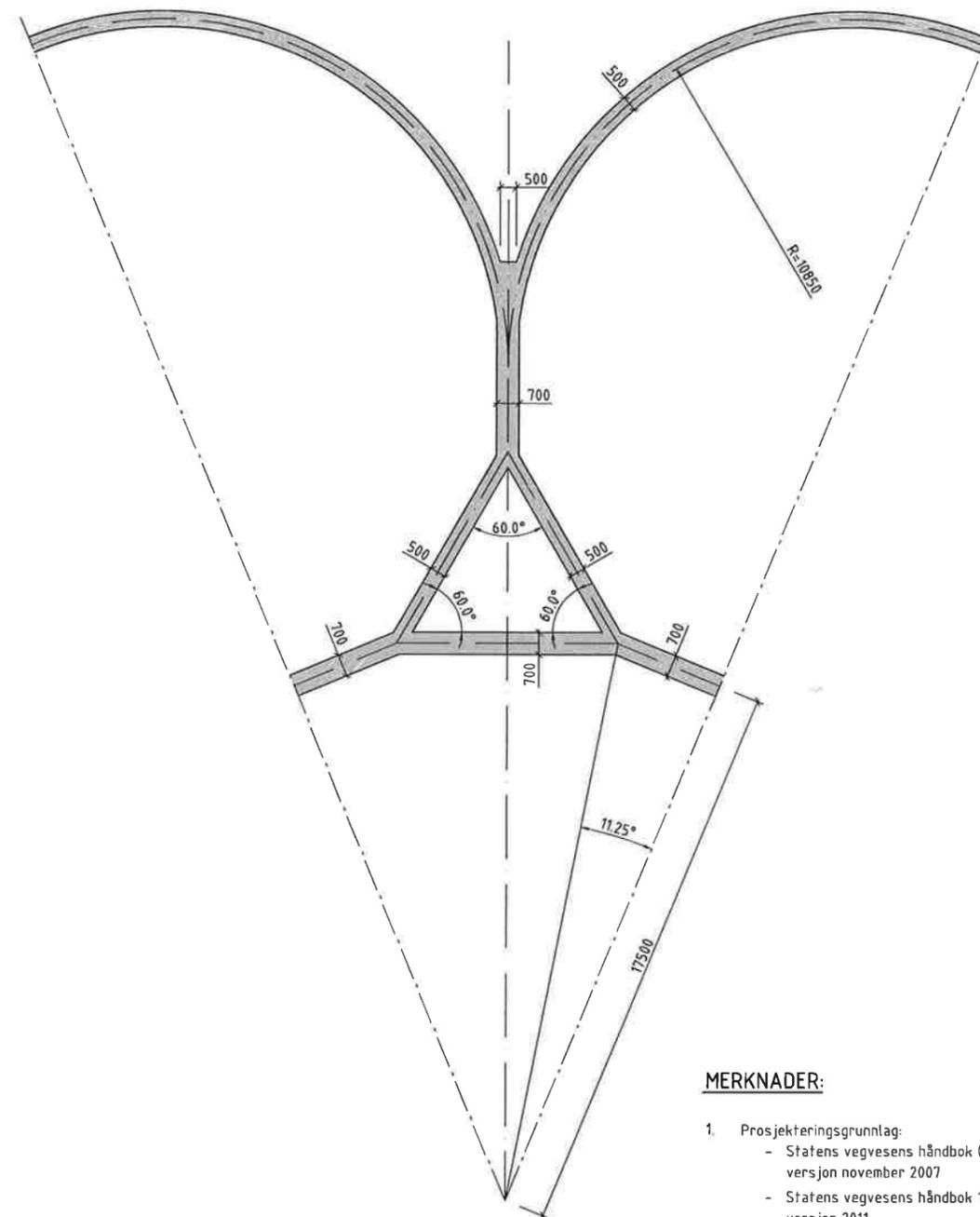
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1. Prosjekteringsgrunnlag:
 - Statens vegvesens håndbok 026 Prosesskode 2, versjon november 2007
 - Statens vegvesens håndbok 185 Bruprosjektering versjon 2011
 - Tilhørende norske standarder
2. Utførelse i samsvar med Prosesskode 2 (2007)
3. Kontrollklasse - utvidet kontroll, NS 3465
4. Alle mål i mm. Koter og koordinater i m
5. Betong: B45 SV-40 iht. Prosesskode 2 (2007)
6. Armering: B500 NC, NS3576, del 3

PROSJEKTERINGSGRUPPEN:					
AAS-JAKOBSEN		Johns. Holt AS		COWI SKANSKA	
Revisjon	Revisjonen gir/de	Utlarb	Kont	Godkjent	Rev. dato
Statens vegvesen				Regningsdato	27.09.2012
Parsell				Bestiller	
MULIGHETSSTUDIE SOGNEFJORDEN, FLYTEBRU				Prosjekt for	
Panglong				Prosjekt av	COWI AS
Oppriss og vertikalsnitt				PROF. nummer	
				Arkivnummer	
				Byggesaksnummer	
				Skisse AT	1500, 1100 (A1)
Utlarb av	Kontrollert av	Godkjent av	Konsulentarkiv	Revisjonsnummer	K101
JOMA	BJAK	ESU	137916	revisjon/utlark	



PLANSNITT Kt. -25,5
1:200

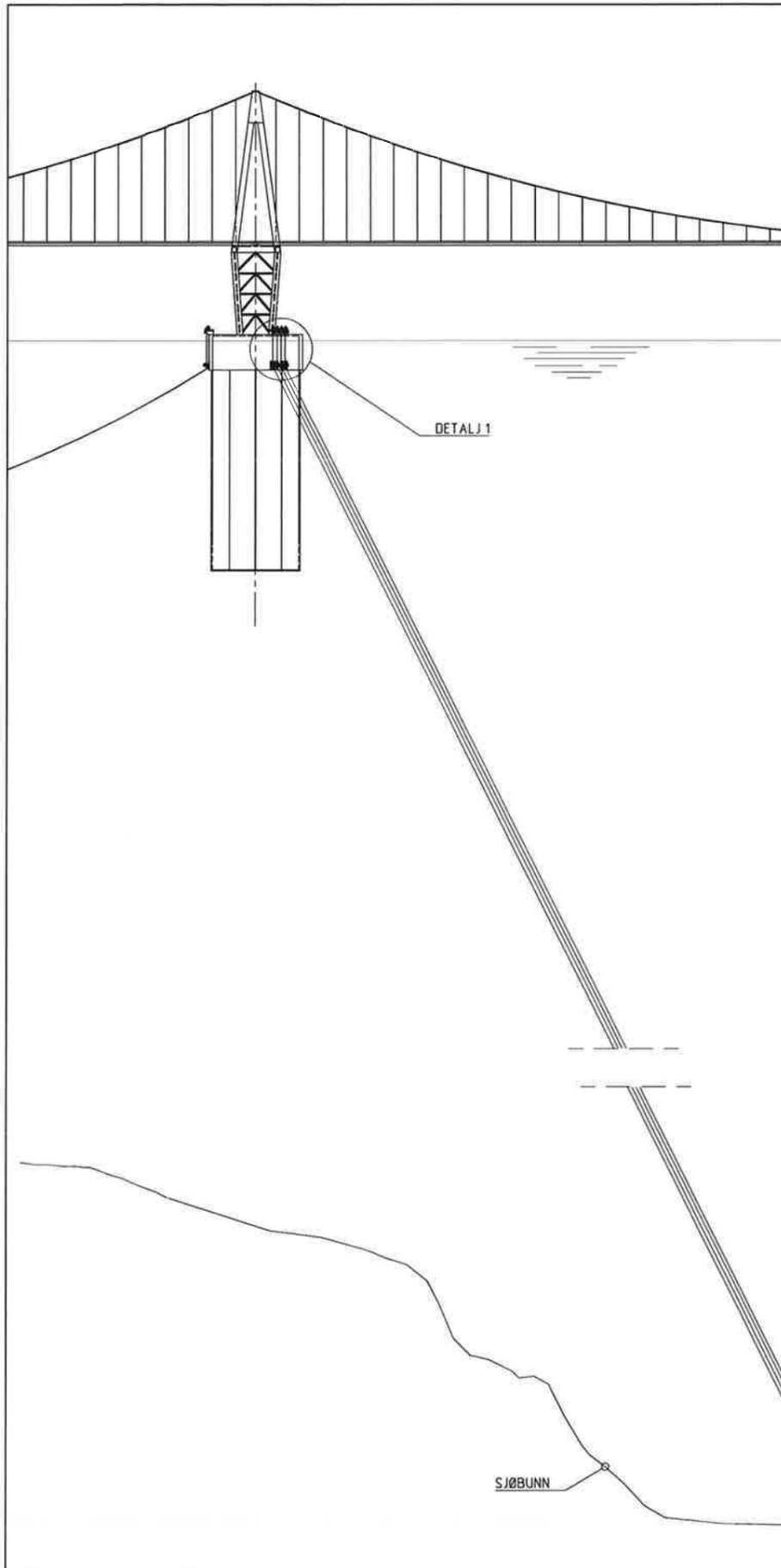


DETALJ 1
1:100

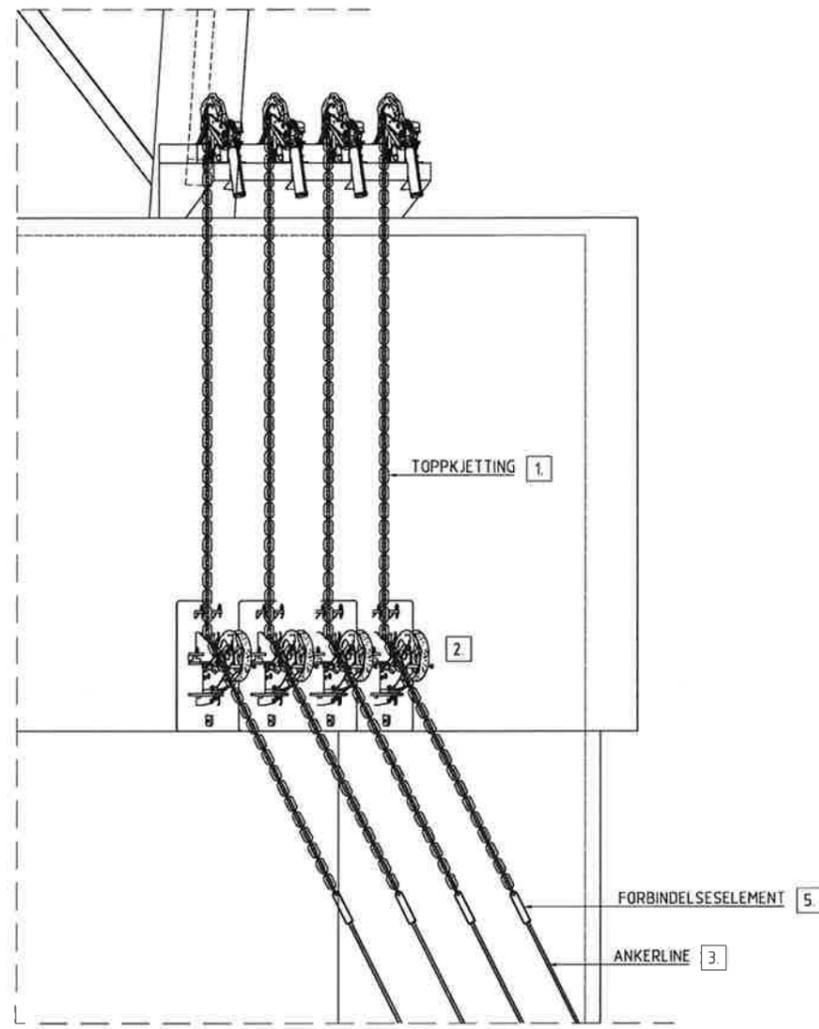
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 - Statens vegvesens håndbok 026 Prosesskode 2, versjon november 2007
 - Statens vegvesens håndbok 185 Bruprosjektering versjon 2011
 - Tilhørende norske standarder
2. Utførelse i samsvar med Prosesskode 2 (2007)
3. Kontrollklasse - utvidet kontroll, NS 3465
4. Alle mål i mm. Koter og koordinater i m
5. Betong: B45 SV-40 iht. Prosesskode 2 (2007)
6. Armering: B500 NC, NS3576, del 3

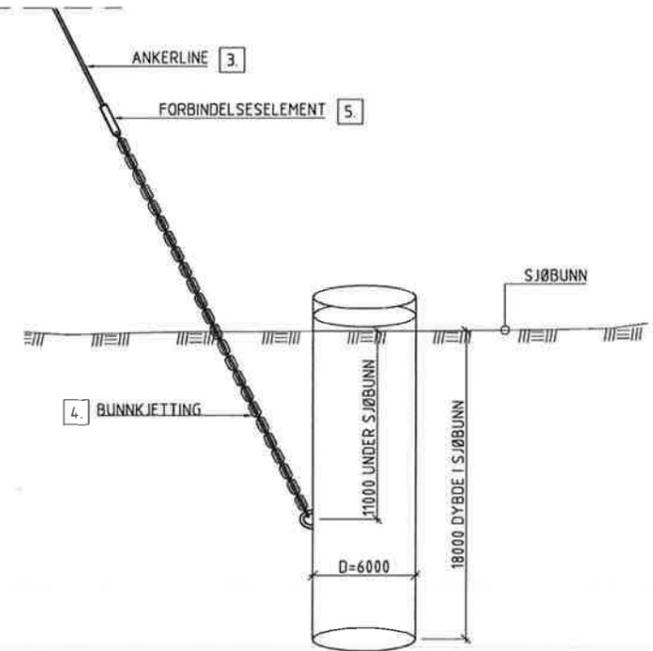
PROSJEKTERINGSGRUPPEN:					
AAS-JAKOBSEN		Johs. Holt as		COWI SKANSKA	
Revisjon	Revisjonen gjelder	Utlarb	Kontr	Godkjent	Rev dato
				Tegningsdato	27.09.2012
				Bestiller	
				Prosjekt for	
				Prosjekt av	COWI AS
Par tittel MULIGHETSSTUDIE SOGNEFJORDEN, FLYTEBRU					
Pongfong Plansnitt kt. -25,5				PROJ-nummer	
				Arkivnummer	
				Eggetekstnummer	
				Skisse	1:200, 1:100 (A1)
Utarbeidet av	Kontrollert av	Godkjent av	Konsulentarkiv	Eggetekstnummer/	revisjonsbokstav
JHMA	BJAK	ESU	137916		K103



A OPPRISS
1:2500



1 DETALJ
1:200



2 DETALJ
1:200

- BEMERKNINGER**
1. TOPPKJETTING: R4S DIAMETER 175mm "STUDLESS CHAIN" BRUDDLAST 27930kN (20mm KORROSJONSTILLEGG)
 2. "FARILEAD" TILPASSET KJETTINGDIMENSJON.
 3. ANKERLINE: "SHEATED SPIRAL STRAND" 153mm, AREAL 1384,6mm², BRUDDLAST 22070kN.
 4. BUNNKJETTING: R4S DIAMETER 152mm "STUDLESS CHAIN" BRUDDLAST 22363kN
 5. FORBINDELSELEMENT TILPASSET DIMENSJON PÅ ANKERLINE OG KJETTING.

PROSIEKTERINGSGRUPPEN:					
AAS-JAKOBSEN		Johs. Holt AS		COWI	SKANSKA
Revisjon	Revisjonen gjelder	Utlarb	Kontr	Godkjert	Rev. dato
		Saksnr	Tegningsdato	28.11.2012	
Statens vegvesen		Beskriv	Produkt for	SYV Region vest	
SOGNEFJORDEN		Produert av			
MULIGHETSSTUDE FLYTEBRU		AAS-JAKOBSEN			
FORANKRINGSSYSTEM		PROF-nummer			
OVERSIKT		Aktidnummer			
		Byggeskisse			
		Høvelokke AS	SDM VSI		
Utlarbeid av	Kontrollert av	Godkjert av	Konsulentnr	Tegningsnummer /	revisjonsnotat
RML	SEJ	SEJ	11258	K-301	