Statens vegvesen

Ferry free E39 –Fjord crossings Bjørnafjorden | 304624

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K12 - SHIP IMPACT, GLOBAL ASSESSMENT

REPORT

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K12 - SHIP IMPACT, GLOBAL ASSESSMENT

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Prepared by:	Eivind Bjørhei
Controlled by:	Thanh Ngan Nguyen
Approved by:	Kolbjørn Høyland

Summary

The global response of the bridge due to ship impact has been studied in this report. The main focus is impacts between ship deckhouse and girder and between ship bulb and pontoon. Girder impacts all along the bridge length have been considered, orthogonal to the bridge girder from both directions. Pontoon impacts have been considered on all three pontoon types, at selected characteristic locations along the bridge. Three impacts are considered; head on (0-degrees) and centric and eccentric side impacts (90-degrees). Pontoon impact from a sideway drifting ship and submarine impact has also been discussed. Post impact the bridge must withstand 100-years environmental conditions.

The global ship impact analyses shows that the bridge will survive both a ship impact as given in the design basis and the following 100-years conditions.

A performed screening of girder impacts gives a maximum girder strong axis bending moment of almost 3000 MNm in the bridge "span", while it is 3750 MNm at the south end (near the cable stayed bridge) and 6600 MNm in the north end. This means the girder needs to be strengthened locally. The maximum elongation of anchor lines due to ship impact is 13,5 m, which is within the acceptable value. The robustness in general is quite good for the given ship impacts. The damage in girder due to girder impacts give small reductions of the moment capacities.

The results from the pontoon impacts are varying and very dependent on impact direction and type of ship. The minimum indentation from the design ship is 2,0 m while the maximum is 13,0 m. Impacts from a drifting ship is not expected to cause fracture in the pontoon, but it cannot be excluded. A submarine impact is not expected to cause fracture in the pontoon, but a direct hit on an anchor line could lead to loss of this.

For the head-on (0-degree) pontoon impacts the indentations are less than 10 m, which gives satisfactory behavior of the bridge post impact. Some green water on deck and overtopping of waves on pontoon must be expected during the post-impact 100-years environmental condition.

The 90-degree impacts could give large plastic displacements in the tall columns on the high bridge and deep indentations in the columns on the low bridge. These special cases cannot dissipate much more energy before loss of entire pontoons could be a case. Increased robustness can be solved by a more detailed design of these critical parts.

The design of the weak axis column-girder connection is critical. The column top should be designed to be weaker than the girder for weak axis bending moments, so this is where a plastic hinge will develop during a ship impact. Otherwise there will be large plastic deformations in the girder which is harder to repair/replace than the column. The column top must be strong enough to withstand the acting bending and torsion moments, ductile enough to dissipate the ship energy and weaker than the girder. This is the most critical detail due to ship impacts.

The ship impact energy is expected to be reduced in the next phase. This will give lower global response and lower damage/indentation of pontoons and girder. It might still give large forces in the column and girder, so these are details that still needs to be addressed.

Table of Content

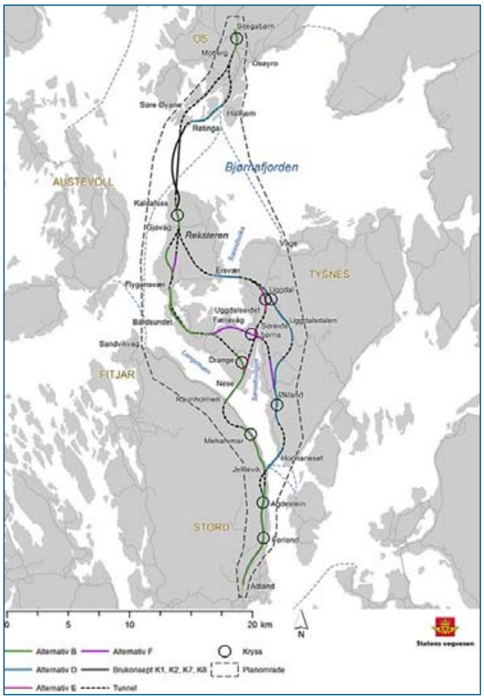
1 INTRODUCTION

1.1 Current report

This report describes the ship impact global response of the K12 – end-anchored floating bridge with mooring system over Bjørnafjorden.

1.2 Project context

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



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

1.3 Project team

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

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

harbour assistance, project work and offshore support from headquarter at Lysaker, Norway.

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

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

1.4 Project scope

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

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

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

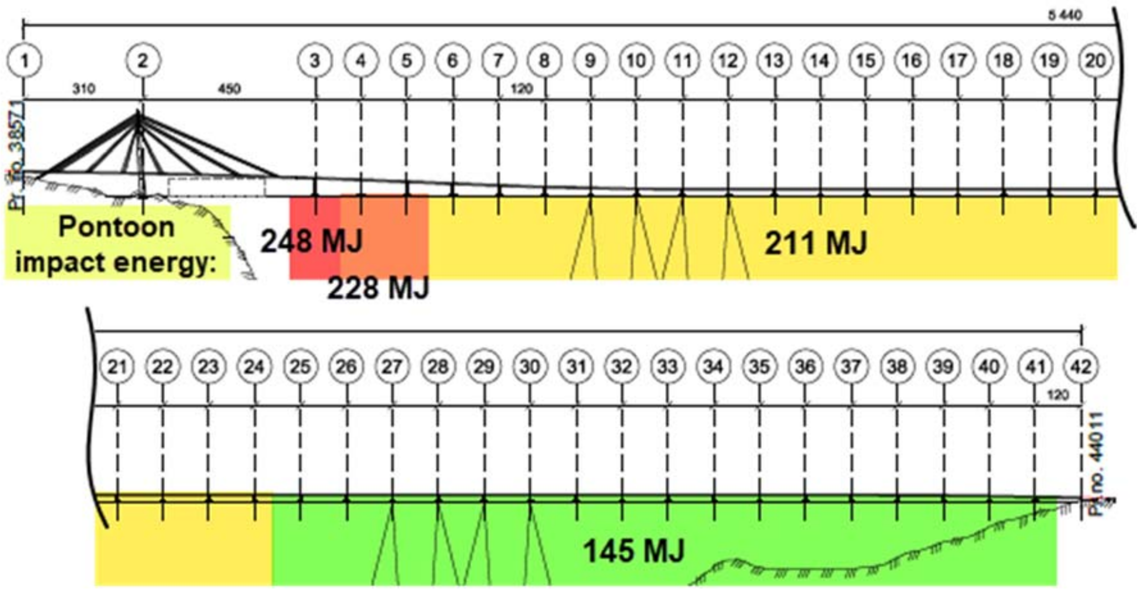
- Global analyses including sensitivity studies and validation of results
- Prediction of aerodynamic loads
- Prediction of hydrodynamic loads
- Ship impact analyses, investigation of local and global effects
- Fatigue analyses
- Design of structural elements
- Marine geotechnical evaluations
- Steel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

2 INTRODUCTION TO SIMULATIONS

2.1 General

This report deals with the case of a ship impact on the K12 – moored+end-anchored floating bridge over the Bjørnafjord. This report seeks to clarify how the structure responds to ship impact and how the bridge behaves post-impact. The response is evaluated both locally at the points of impact and globally for the whole structural behavior.

Axis numbering is shown in Figure 2-1. Pontoon axes are designated from 3 (pontoon at ship navigation channel) to axis 41 (pontoon at northern abutment).



> *Figure 2-1 Numbering of axis referring to pontoons and distribution of pontoon impact energy, see also Figure 3-2 for details. From drawing SBJ-33-C5-OON-22-DR-001.*

2.2 Design philosophy

Ship impacts are defined as accidental load conditions related to a recurrence period of 10 000 years. The Norwegian Public Roads Administration (NPRA) has in handbook N400 [1] set this as the limit where less likely events are disregarded.

In the Accidental Limit State (ALS) all loads are applied with partial load factors of 1.0, and it is allowed to utilize lower material safety factors than in Ultimate Limit State (ULS) and Serviceability Limit State (SLS). Local collapse is acceptable, provided the global stability can be maintained to prevent total collapse. The bridge must be able to sustain a post-impact phase according to NS-EN 1991-1-7-2006 [2]. Examples of such local collapse is filling of water in some pontoon compartments and evaluation of a plastic hinge in the column-girder connection.

Impact loads depend on the relationship between the incoming ships mass and velocity (total impact energy) and the system response. The system response is depending on the mass (m), the combined stiffness (k) of the structure and ship, and the system damping (c). Simplified it can be described with the equation of motion below, where the impact load F varies over time.

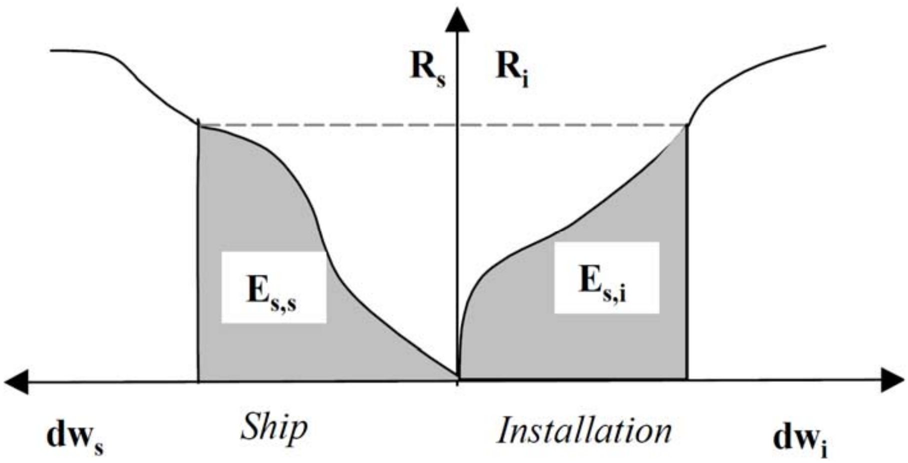
$$m\ddot{x} + c\dot{x} + kx = F(t)$$

The dynamic response from the impact energy depend on ship stiffness and stiffness and mass of the structure. To ensure a ductile design the analysis considers the differences in stiffness. This is done by transferring the energy through the following steps:

1. Ship bow-pontoon/deckhouse-girder impact. Represented by a force-indentation curve, based on local analysis.
2. Bridge structure. Represented by global FE-model.

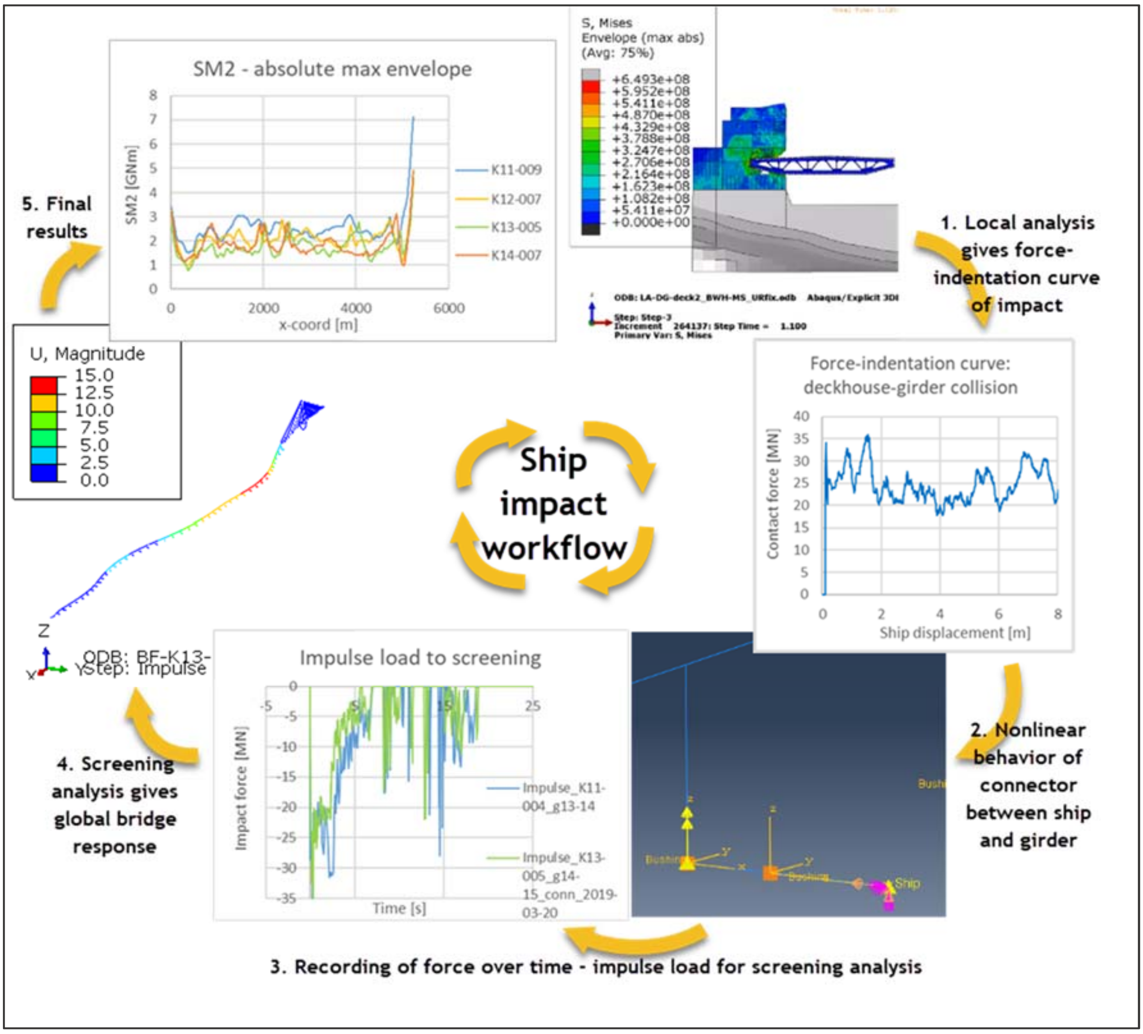
For the pontoon side impacts (90 deg, girder longitudinal direction) there has also been performed local analysis giving moment-rotation-curves for bending and torsion in columns, as the section forces for some impacts are larger than the elastic capacities.

By combining the stiffness and mass in different parts of the system in one model, we obtain a realistic energy distribution. For the connection between ship and pontoon this can be illustrated with the graph in Figure 2-2. The graph shows that the mobilized resistance is equal in the two systems, and that this balance, together with the force-indentation relations, give the corresponding deformations and energy absorption in each part of the system.



> *Figure 2-2 Force equilibrium based on force-indention curves.*

Figure 2-3 shows an overview of the workflow used for the ship impact analysis. The figures and graphs inside are for illustration purpose only.



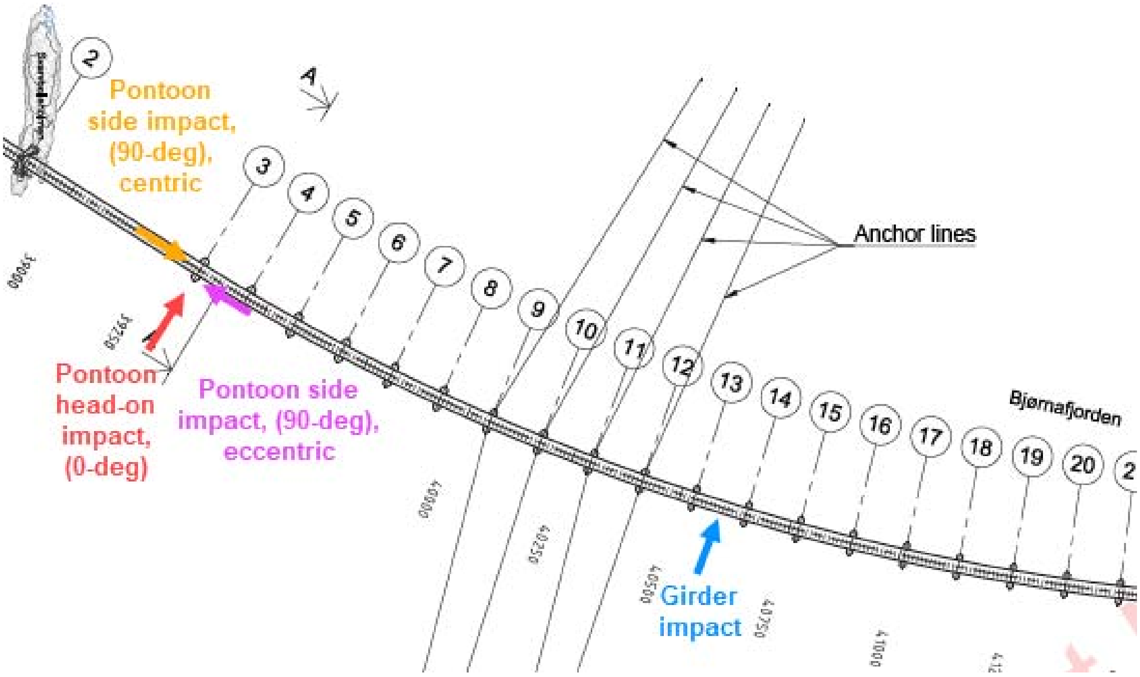
> *Figure 2-3 Ship impact workflow. Step 1: Local analyses as in report [3] and [4]. Step 2: Global analyses with spring-mass-system. Step 3-4: Screening analyses. Step 5: Post-processing of results and evaluations.*

3 IMPACT SCENARIOS

3.1 Design basis

Impact scenarios are based on the specified cases in the Design basis [5]:

- Bow collisions with bridge pontoons (centric and eccentric), all possible impact angles
- Deckhouse collision with bridge girder
- Sideway collisions (against the pontoons longitudinal walls)
- Submarine impact



> *Figure 3-1 Ship impact illustration. Examples of the main impacts studied in this report.*

The main spans in the bridge are 120 m. The design basis gives distribution energies for spans of 100 m and 125 m. These tables are for practical purposes equal. The table for 125 m span has been used as the basis, and the impact energy for the southern half of the low bridge has been expanded to axis 24 (from axis 23 in the 125 m design basis table).

There is also a possibility to reduce the girder impact energies on the northern half of the bridge, but as this increases the pontoon impact energies this is not further considered at this stage.

The chosen distribution of impact energies is shown in Figure 3-2. Mainly it is the larger impact energies that are studied in detail, as these are the critical for the concept evaluation and robustness.

CC 120 m Element	Displacement [tonne]	Velocity [m/s]	LOA [m]	Incl addmass [tonne]	Energy incl addmass [MJ]
Bridge girder	19084	6.2	200	20 038	**385**
Pontoon, Axis 3	14565	5.7	140	15 293	**248**
Pontoon, Axis 4-5	13878	5.6	130	14 572	**228**
Pontoon, Axis 6-24	13259	5.5	130	13 922	**211**
Pontoon, Axis 25-41	10649	5.1	120	11 181	**145**

> *Figure 3-2 Distribution of impact energies, given in design basis and modified for the 120 m span.*

3.1.1 Expected reduction of impact energies

According to the client, the impact energies are expected to be reduced for the Bjørnafjorden crossing due to stricter control of the ship traffic in the area. This is not included in the analyses done in this report, but it is discussed. It is also a part of the evaluation of critical details. Areas where we at this stage have small margins are expected to be less critical at the next stage.

In this report however, the eventual reduction of impact energies is considered as increase of robustness.

3.2 Impact characteristics

The impact characteristics are given by the design basis. Simplified, the girder impacts are governing for the input to the global design, while the pontoon impacts are governing for the design of the pontoons, columns and the girder-column connection.

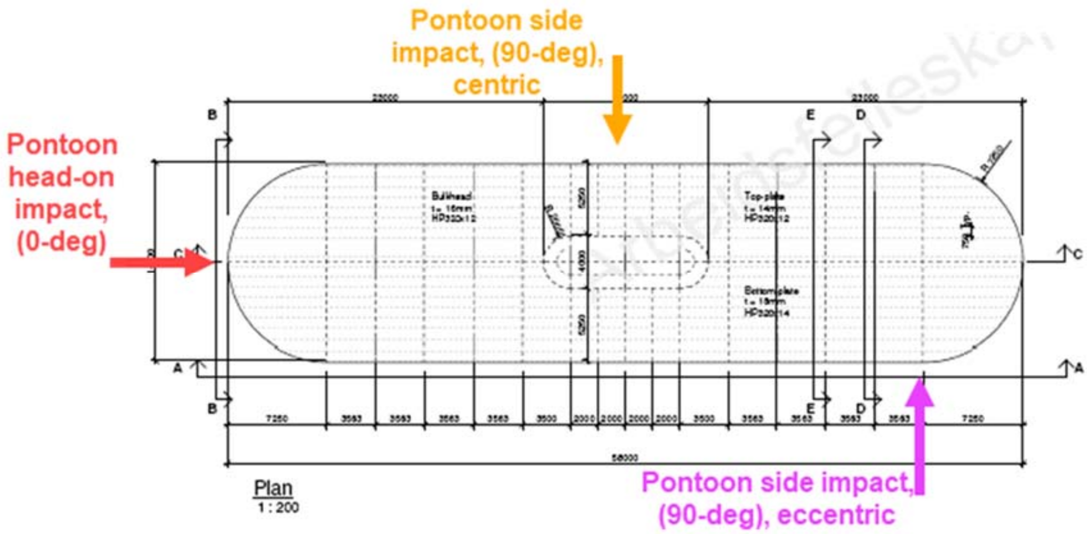
3.2.1 Bow collision with pontoon

Mainly two impact characteristics have been considered in this phase – impact on the tallest column – axis 3, and the high energy pontoon impacts on the axis 6-24. This gives the governing forces from ship impact for the following construction parts:

- Axis 3 impact:
    - Maximum tension in the cable stayed bridge tendons
    - The largest weak axis bending moment in both column and girder
    - The largest torsion moment of girder
- High-energy low bridge impact:
    - Largest indentation in pontoon (90 deg centric impact, low bridge)
    - Largest torsion in column (90 deg eccentric impact, low bridge)
    - Largest force in anchor line (head-on impact, axis 9-12)

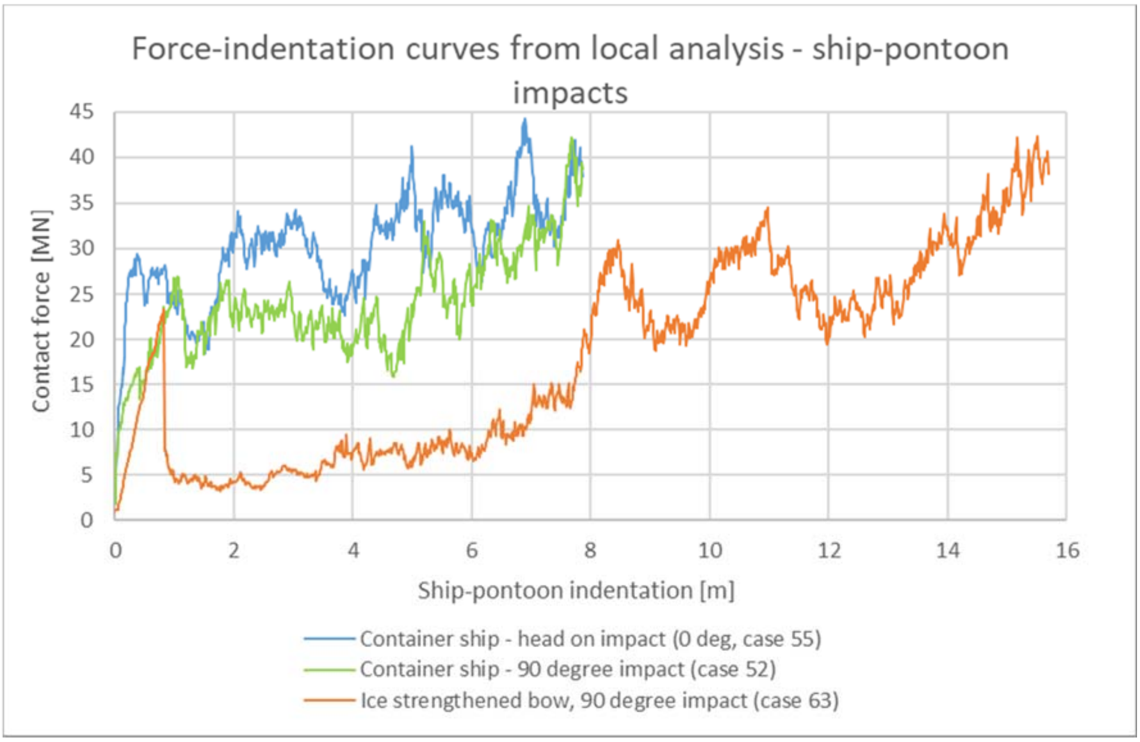
The impacts are given in Figure 3-2 and gives impact energy of 248 MJ for the axis 3 pontoon and 211 MJ for the axis 6-24.

The location of the considered impact points on the pontoon is shown in Figure 3-3.



> *Figure 3-3 Illustration of the three pontoon impacts considered in this study.*

The bow impact with pontoon and also column is thoroughly studied in the report SBJ-33-C5-OON-22-RE-015-K12 - Ship impact, Pontoons and columns [3]. The results here are from a 16 m wide pontoon. As the pontoon sizes have been changed throughout the design phase, this is no longer an actual pontoon on the K12 bridge. There has been performed a lot of sensitivity studies on the local analysis, and the pontoon size is not expected to have a large impact on the force-indentation curve. See local analysis report [3] for more details. The remaining pontoon properties, such as mass and water plane stiffnesses, are updated to the latest design. The force-indentation curve used for the pontoon impact analyses is given in Figure 3-4.



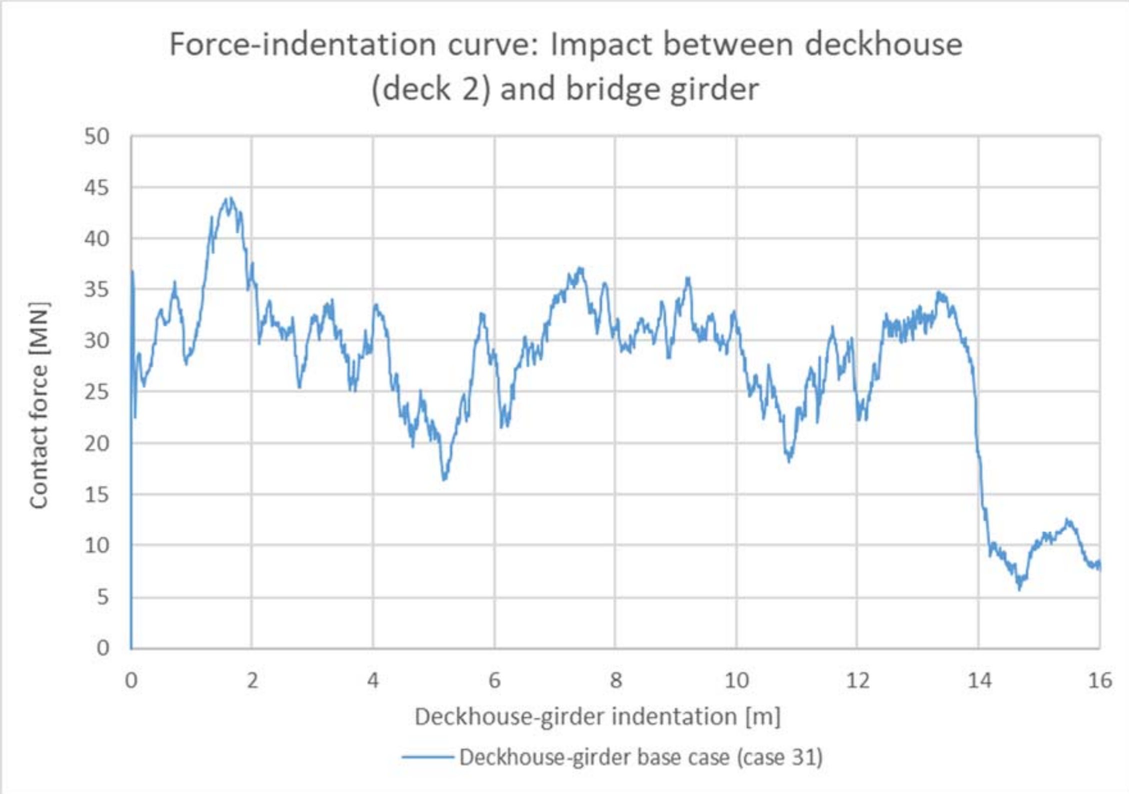
> *Figure 3-4 Force-indentation curve for ship-pontoon impacts. Head-on impacts for container ship and 90-degree impact from both container ship and ice-strengthened bow. Obtained from local analysis and used for all pontoon impacts in this report.*

The container bow impact is used for the global analyses, as it transfers the most energy into the bridge. For the evaluation of maximum damage of pontoon compartments, a 90-degree (along bridge girder) impact from an ice-strengthened bow is governing. A centric impact on the low bridge gives maximum indentation and therefore maximum local damage of pontoon.

3.2.2 Deckhouse collision with bridge girder

The impact is given in Figure 3-2 and gives impact energy of 385 MJ, which can strike anywhere on the bridge, in any possible direction. In the analysis these ship impacts are applied midspan (between pontoons), orthogonal to the bridge girder.

The deckhouse collision with bridge girder is thoroughly studied in the report SBJ-33-C5-OON-22-RE-016-K12 - Ship impact, Bridge girder [4]. A direct result obtained from these studies is the force indentation curve from this impact, which is used in the global analysis to evaluate the global response.



> *Figure 3-5 Force-indentation curve for deckhouse-girder impact. Obtained from local analysis and used for all girder impacts in this report.*

3.2.3 Sideway collision with pontoon

Sideway collision against the pontoons longitudinal walls is discussed in this section and elaborated with results from the local analysis and considerations in Appendix C.1.

The Design Basis states an impact velocity of 2 m/s for the axis 3 pontoon, 1 m/s for the remaining pontoons. Assumed that it is the largest ship that is drifting into the pontoon, the ship mass is 19 084 tonnes. Including 40 % added mass for sideway ship impact, the total kinetic energy of the axis 3 impact is 53 MJ, see Figure 3-6.

CC 120 m Element	Displacement [tonne]	Velocity [m/s]	LOA [m]	Incl addmass [tonne]	Energy incl addmass [MJ]
Pontoon axis 3	19084	2.0	200	26 718	**53**
Pontoon axis 4-41	19084	1.0	200	26 718	**13**

> *Figure 3-6 Distribution of impact energies of sideway collisions, given in design basis and modified for the 120 m span.*

Given the same force-indentation curve as for the 90-degree ship-pontoon impact, 53 MJ corresponds to 2,6 m indentation, see Appendix C.1. This very conservative approach gives a maximum loss of two compartments which is less than the worst case from bulb-pontoon impacts. Hence this is not a governing load case in the accidental limit state.

There has been done some evaluations on Appendix C.1 on the more likely outcome of a sideway collision, as the behavior in a broad-side collision is expected to be a lot stiffer than the bow collision, with a larger impact area. The most likely scenario in a "clean" (directly) sideway impact is that the pontoon survives the impact without fractures. It is likely that the ship will take more of the damage itself due to a weaker construction on the side than in the bulb. In a less clean impact with a larger impact angle between the ship and the pontoon the impact area will be smaller, and it is possible that there will be fractures in the pontoon, and possibly water filling of one or two compartments. This totally depends on the geometry and impact angles and is hard to predict.

For the axis 4-41 impacts of 13 MJ the maximum indentation in a conservative approach is 0.8 m. This means the 13 MJ-impact is not expected to cause any fracture (and related water filling of compartments) in the pontoons.

3.2.4 Submarine impact

The case of a submarine impact is defined in design basis and summarized in Figure 3-7.

State	Displacement [tonne]	Velocity [m/s]	LOA [m]	Energy [MJ]
Surfaced	1450	3	x	**7**
Submerged	1830	5	x	**23**

> *Figure 3-7 Impact energies from submarine as given in design basis.*

As the submerged mass is larger than the surfaced displacement the numbers are assumed to include added mass. These energies are less than the pontoon impact from ships, hence they are not governing load cases. Compared to the sideway impact discussed in the previous section, the surfaced submarine impact is not expected to cause pontoon fracture.

If a submerged submarine hits an anchoring line it will probably fail. The bridge is designed for loss of two anchor lines, so this is neither a governing load case. For documentation of the bridges behavior with loss of two anchor lines, see "SBJ-33-C5-OON-22-RE-021-K12 - Design of mooring and anchoring" [6].

3.3 Loads and load combinations

Ship impact event is treated as an Accidental limit state according to Eurocode 1990:2002+A1:2005+NA:2016 [7].

There are two relevant load combinations in the design basis for this report, which is the ship impact and the post-impact 100-years storm, see Figure 3-6.

Load combinations in ALS		Stage a				Stage b (damaged condition)		
		Earthquake	Abnormal environmental loads	Fire and explosion	Ship impact	Pontoon filled with water	Lost mooring cable	Lost cable stay
		Ψ_2	Ψ_2	Ψ_2	Ψ_2	Ψ_2	Ψ_2	Ψ_2
Permanent loads								
Permanent loads	G- EQ_K	1.0	1.0	1.0	1.0	1.0	1.0	1.0
Variable loads								
Traffic loads	Q_Trf_K	0.5	0	0.5	0.5	0	0	0
Temperature loads	Q-$Temp_K$	0	0	0	0	0	0	0
Other loads	Q_K	0	0	0	0	0	0	0
Environmental loads (100yr)	Q-$E_{K(100)}$	0	0	0	0	1.0	1.0	1.0
Accident loads								
Earthquake	A-EarthQ	1.0	0	0	0	0	0	0
Environmental loads (10.000yr)	Q-$E_{K(10.000)}$	0	1.0	0	0	0	0	0
Ship impact	A-Coll	0	0	0	1.0	0	0	0
Pontoon filled with water	A-Flood	0	0	0	0	1.0	0	0
Lost mooring cable	A-Morfail	0	0	0	0	0	1.0	0
Lost stay cable	A-SCab	0	0	0	0	0	0	1.0
Fire and explosion	A-Fire	0	0	1.0	0	0	0	0

Table 9 Load combinations in the accident limit state

> *Figure 3-8 Load combination table from design basis, where load combinations handled in this report is highlighted.*

Traffic loads are to be included with a partial load factor of 0.5 to the ship impact, see Figure 3-8. For the post impact 100-years storm there is no traffic load included. The traffic loads are small compared to the self-weight and the forces from ship collisions and are disregarded for most of the analysis. Verifications of this have been done with one case, section 6.5 and Appendix E.

3.3.1 Characteristic loads

Self-weight is applied to all construction elements with a gravitational acceleration of 9.81 m/s^2.

Traffic loads are described in section 4.1.2 in "SBJ-32-C5-OON-22-RE-003-A Analysis method" [8], where reduction factors are included. In the ship impact analysis, the traffic loads are simplified to increased mass in the bridge girder. The characteristic line load from traffic is 30.4 kN/m, see table 4-2 in the OONO Analysis methods report [8]. The static effects of the traffic loads are neglected at this stage. For detailed description of how the loads are applied in the simulation model, see section 4.3.4.

3.3.2 Impact

The impact loads are given by the impact characteristics and the impact energies as given in the design basis, see Figure 3-2.

3.3.3 Post impact

The post impact load is described in the verification of global analysis report [9]. This is a 100-years environmental load. In this report, the analysis results done for the SLS-state in load combination 23 is used as input for bridge response, as the load factors are the same for the SLS- and ALS-state – 1,0 for both self-weight and environmental loads.

It has not been performed separate time-domain analysis fort the post impact state as it is not considered necessary.

4 SIMULATION MODEL

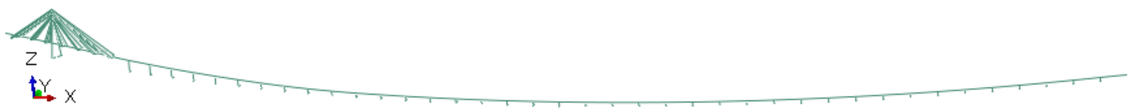
The ship impact analyses have been performed using the finite element software Abaqus [10]. For the ship impact analyses, the implicit solver is used.

The Abaqus model is built in the same way as the global 3D-float model, based on the same input. For details regarding geometry and boundary conditions see the structural response analysis report [11]. The basics of the model are explained in this chapter, so are the details that differ from the global model in 3D-float.

4.1 Geometry

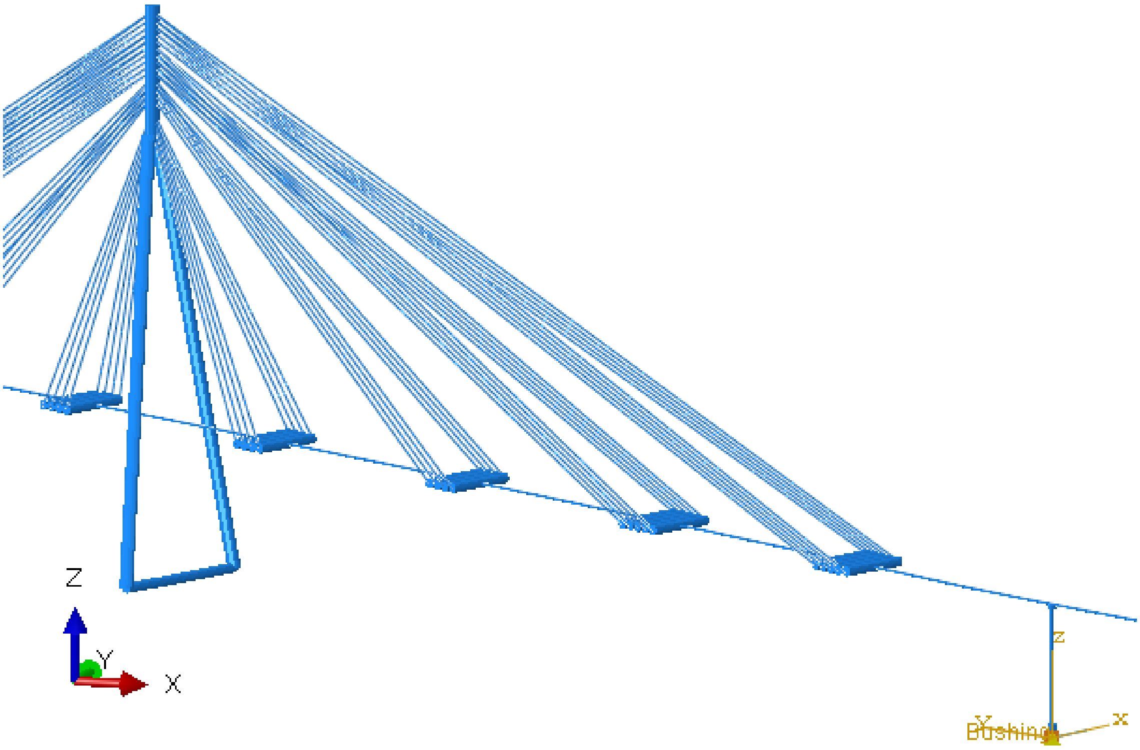
The geometry, section properties and material properties have been imported directly to Abaqus based on the same input as the global analysis of the bridge in 3D float. Hence, the geometry is in principle identical to the one used for simulation of (non-impact) dynamic and static load effects. Specific terminology for the Abaqus analysis is explained in Appendix A.

The FE-model geometry consists of wires only, which means the only applicable elements are beam and truss elements, see Figure 4-1.



> *Figure 4-1 Global FE-model of the Bjørnafjorden bridge, used for ship impact analysis.*

The different bridge elements (bridge girder, columns, cables and more) are rigidly connected unless else is specified. All connections transfers 6 degrees of freedom (U1, U2, U3, UR1, UR2, UR3 / X, Y, Z, RX, RY, RZ).



> *Figure 4-2 Global FE-model showing area around the navigation channel. Beam profiles for tower, girder and pontoon columns are rendered.*

4.1.1 Elements

Except for the cables the elements are of the type B31 which are 2-node 3-dimensional beam elements with a linear geometric order (uses linear shape functions for the approximations between integration point and the element ends). The cable elements are of the type B31H. B31H are the same elements as B31 but with two additional variables related to the axial force and transverse shear force.

The element size of the cables is set to a large number such that one cable is one element only, which improves the computational behavior. This means the geometric stiffness of the cable is neglected, but as the cables are tensioned the overall behavior is quite good. This is the same way as the cables are represented in the global design models in 3D-float and also the global verification model from Abaqus.

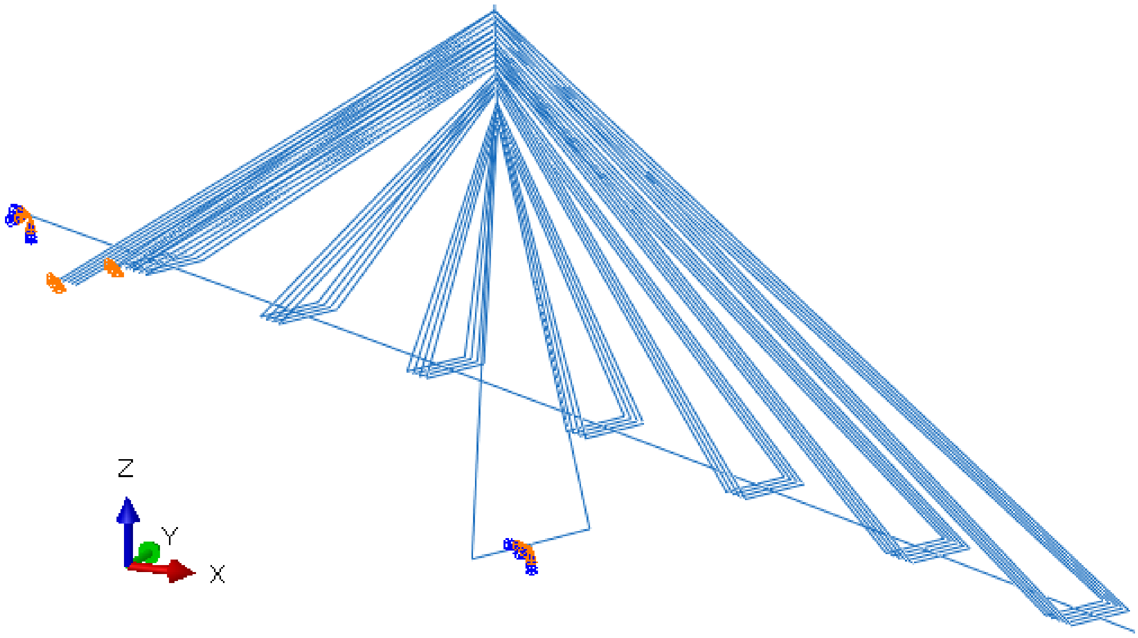
For the rest of the model the global element size is approximately 10 m, meaning all the structural elements are parted into calculation elements of approximately 10 m.

4.1.2 Boundary conditions

The "global" boundary conditions are the same as for the global model, see the structural response analysis report [11]:

- Bridge girder is fastened at abutments: fixed for translations, strong axis rotations (about vertical axis) and torsion moments, but free to weak-axis rotations (about transverse axis).
- Fixed bridge tower
- Cable group nearest southern abutment fastened to ground

Boundary conditions on cable bridge in simulation model is shown in Figure 4-3.



> *Figure 4-3 Boundary conditions on cable stayed bridge, global FE-model. The bridge girder is fixed towards the southern abutment, and first cable group from south abutment is pinned to the ground. The tower is fixed to "the ground".*

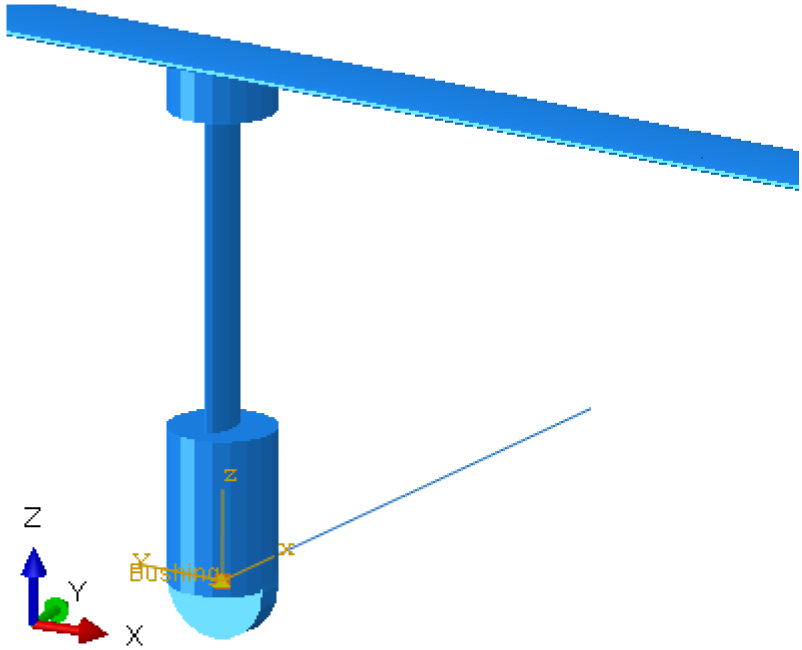
4.1.3 Pontoons

The pontoons are not included physically in the model, but their hydrostatic characteristics are represented. These are implemented using connector elements (see Appendix A.2) with elastic behavior and damping. The connectors are applied at water level and describe a linear stiffness for vertical motions and for rotations about horizontal axis (longitudinal and transverse). The pontoon structural masses are applied in the buoyancy center and includes rotational inertias.

Viscous damping on the pontoons is included in the horizontal degrees of freedom (U1, U2), as a function of the horizontal velocity. The drag factors are based on CFD-analysis, see the hydrodynamic optimization report [12]:

- 0.3 in the longitudinal direction
- 0.6 in the transverse direction

Added mass is applied in the same point as the water plane stiffness. Added mass is conservatively set to infinite frequency values. The added mass is specified for all six degrees of freedom.



> Figure 4-4 General pontoon represented with a connector "fixed to ground" with vertical and rotational stiffness. The remaining degrees of freedom are free to move/rotate. Structural mass and added mass are applied as point masses/inertias in the same point as the connector.

4.2 Materials

The materials used in the model are the same as used in the global analysis in 3D float. In general, the materials are elastic meaning the modulus of elasticity is the only material property of importance. For detailed material properties, see the structural response analysis report [11].

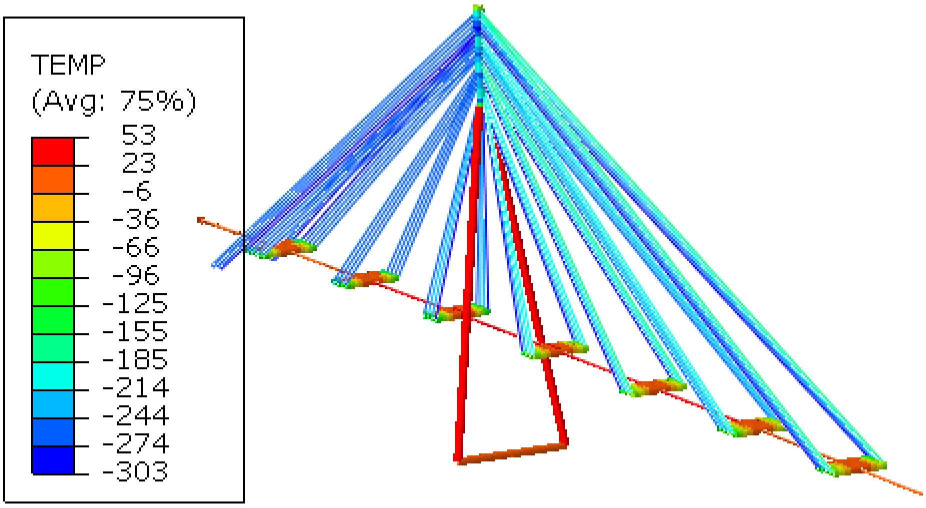
4.3 Cable tensioning and structural loads

4.3.1 Gravity

Gravity is applied to the whole model with a gravitational acceleration of 9.81 m/s^2.

4.3.2 Cable tensioning

To obtain the correct geometry when gravity is applied, the cable tensioning needs to be adjusted. This is done in the Abaqus model used for verifications of the global analysis [9] and applied to this model. The temperatures used is shown below in Figure 4-5. The cables are "post tensioned" by lowering the cable temperature. This gives a compression strain of the girder and tower which is compensated for by increasing the temperature.



> *Figure 4-5 Post tensioning of cables and elongation of girder and tower with temperatures [K].*

4.3.3 Pontoon buoyancy

The pontoon buoyancies are applied as point loads in the buoyancy centers. The buoyancies are retrieved from the same model as the cable pre-tensioning input – the Abaqus model used for verifications of the global analysis [9].

4.3.4 Traffic loads

Traffic loads are mostly neglected in the ship impact analysis, as they do not affect the local behavior of the model. Nor do traffic affect the global motions of the bridge in a significant way, as the traffic load is 30 kN/m, while the girder self-weight is close to 200 kN/m. With a participation factor of 0.5 for traffic loads the traffic load increases the girder mass with less than 8 %.

There has been performed a couple of analysis with traffic on the bridge, see Appendix E. Here, the traffic load is included as increased girder density in order to affect the dynamic behavior. To apply maximum eccentricity there is added a torsional line moment along the girder.

4.4 Ship impact setup

The ship impact analysis is performed on a stabilized model with gravity, tensioning of cables and pontoon buoyancy applied. There is a static step in the beginning of the analysis to obtain this stabilized model, before the implicit dynamic ship impact steps in the time domain.

4.4.1 Ship impact on pontoons

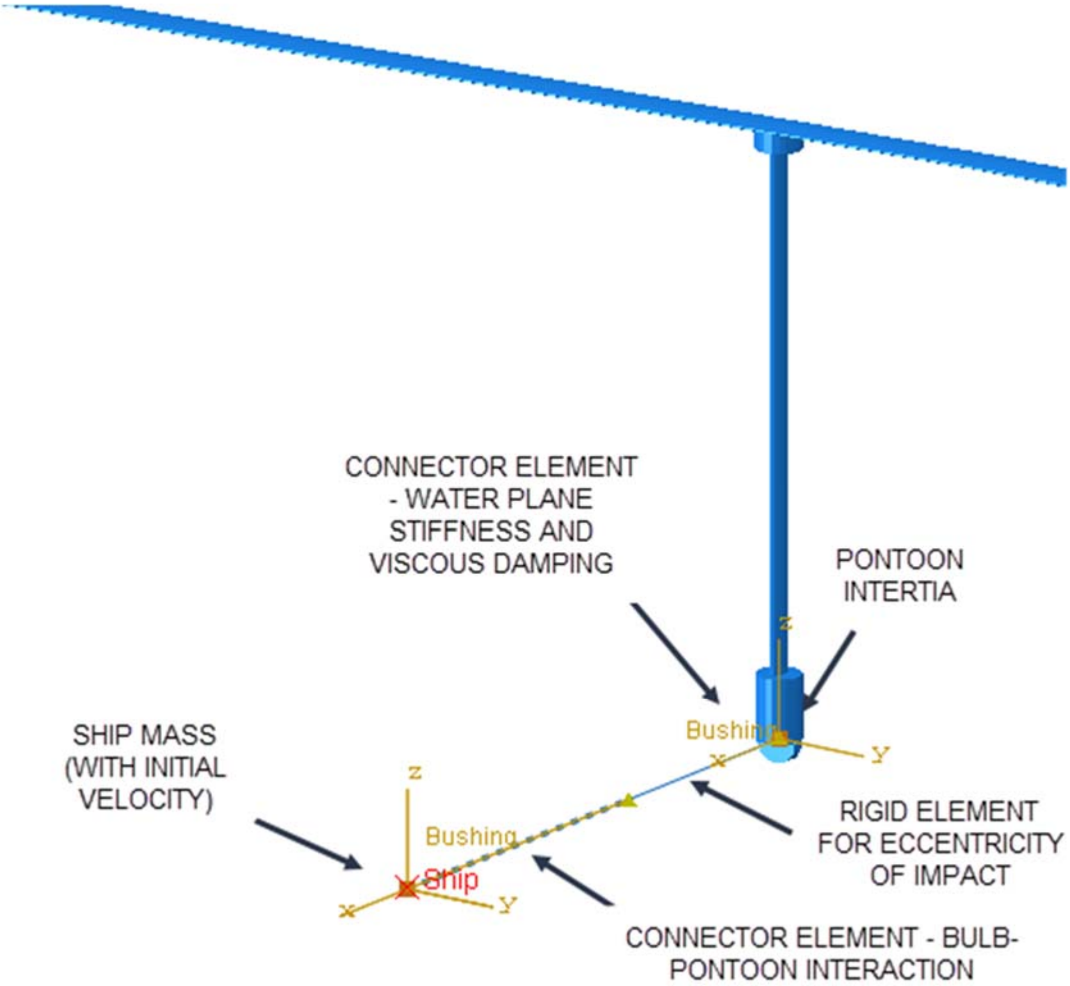
The ship impact analysis is set up using a point mass describing the ship and a connector element. The connector element represents the force-indentation between the ship and the pontoon. The "ship" is set up with an initial speed in the impact direction and allowed to move in the horizontal plane only, see Figure 4-6. Between the ship and the pontoon there is a connector element representing the deformation of the ship bow and the pontoon wall as given from the local analysis. The connector element has an inelastic behavior in the impact direction, according to the force-indentation curve described in section 3.2.1. The elastic part of the compression behavior is set to a large number, as the results from the local analysis includes both linear and plastic deformations. The pontoon deformation connector is elastic in the transverse direction and for separation of ship and pontoon, both with low stiffnesses. The transverse stiffness is set to 1000 times the tensional stiffness, to see if the ship changes direction due to deformations in the column and pontoon. There is no connection for vertical motions, allowing the pontoon center point to move independently of the ship in the vertical direction.

During the impact event and response, the ship is restricted from vertical and rotational movement, and is moving in the horizontal plane only. The kinetic energy in of the ship mass is transferred to the connector system until the ship is stopped and sent back by the strain energy accumulated in the bridge during the impact. The connector elements have a very low spring stiffness for separation of the ship and pontoon, allowing the ship to "float away". The ship impact setup is shown in Figure 4-6.

Note that the distances in Figure 4-6 are only for visual representation and that the true force-indentation characteristics are given as properties in the connector elements, see Table 4-1.

> *Table 4-1 Ship-pontoon connector properties*

Degree of freedom	Property	Stiffness
U1- - axial compression, elastic part	Elastic, stiff	10 GN/m
U1- - axial compression, plastic part	Plastic	As in Figure 3-4
U1+ - axial tension/elongation	Elastic, soft	0.1 N/m
U2 +/- - transverse motion	Linear elastic	100 N/m
U3 – vertical motion	None	-
UR1/UR2/UR3 – rotational DOFs	None	-



> *Figure 4-6 Ship impact setup for a head-on (0-deg) ship impact on pontoon in axis 3 (for an older model version). Note that the length of the connector is only for visual representation.*

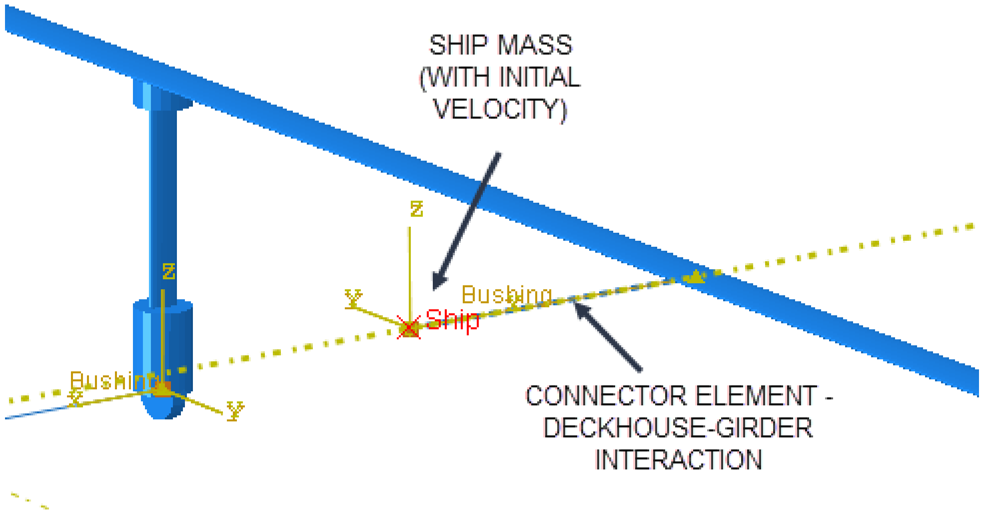
The mass of the ship is placed in the reference point "Ship", while the inertia-properties of the pontoon placed in the buoyancy center. The rigid element has a length equal to the distance from pontoon center to the transition between straight and curved pontoon wall. As all the pontoons are 58 m long, this is 58/2 m minus half of the pontoon width – respectively 23 m, 21.75 m and 20.25 m for the 12 m, 14.5 m and 17.5 m wide pontoons. As the center of the impact from the container ship (center bulb) is approximately at the buoyancy center of the pontoon (2,5 m below water plane), the rigid element is horizontal.

4.4.2 Ship impact on bridge girder

Impact directly on the bridge girder is modelled with a single connector that takes deckhouse and girder deformation into account.

1. The ship is modelled as a point mass with mass and velocity consistent with the impact energy.
2. Deckhouse-girder indentation is modelled with a connector element using force-indentation curve from local analysis.

Figure 4-7 shows graphically how the point mass, the connector element and the bridge girder are connected. The figure show both the model rendered displaying beam element profiles (above) and wire frame model to show relevant element connections (below).



> *Figure 4-7 Ship impact setup for impact on bridge girder. Note that the length of the connector is only for visual representation. Above with beam profile rendering, below without.*

4.5 Uncertainties and assumptions

The following simplicities/assumptions are done in the global modelling:

- The cables are modelled as one element per cable, so the geometric stiffness due to deflections of the cables is neglected.
- The water plane stiffnesses are linear, so the pontoon (local) surge stiffness is underestimated for large rotations, as in the pontoon ship impact. At the same time, it is not updated during the impact, so both heave and surge stiffnesses are overestimated after the pontoon damage.
- The ship only transfers large forces in the impact direction, no transverse forces due to pontoon translations/rotations.
- The head-on (0-degree) and 90-degree impact are considered adequate for evaluation the pontoon impacts.
- In the pontoon impact the container bow is used for the structural response and capacity controls of the column and girder, while the ice-strengthened bow is used only for evaluation of maximum indentation of pontoon for a post-impact evaluation.

5 SCREENING OF IMPACT SCENARIOS

There has been performed screening analysis of girder impacts to investigate the global behavior of the bridge. The screening analysis consists of 22 impacts on 11 locations, so impact from both west and east are considered. The chosen cases are assumed to be representative for all girder impacts, as they include critical points along the girder such as high bridge close to navigation channel, both anchor groups, center of span between anchor groups/abutments and critical points for the fixation of north end.

The response presented in this chapter is the response that is regarded as relevant for the global design:

- Bridge girder strong axis bending moment
- Maximum displacement of anchor point (K12-K14) – gives maximum elongation of anchor line
- Horizontal displacement of bridge girder at bridge tower, orthogonal to bridge girder.

Important assumptions for the screening analysis:

- Traffic loads are neglected – these are considered to not change the response between the concepts. See chapter 6.5.
- For screening analysis, there has only been considered impacts to the bridge girder, as this is the impact with most energy and will transfer the most energy to global girder motions.

The connector analysis gets the ship impact force from a mass-spring system that requires a force equilibrium between the ship and pontoon at each step. This is a quite fast analysis in pure calculation time, but it is time demanding to do the modelling in Abaqus CAE. This is the reason for doing the screening analysis with impulse loads, which is easier to mass produce by programming. See verification of this in section 5.4.

5.1 Screening setup

The 11 impact points are chosen at critical points along the bridge and are considered representative for all possible girder impacts. The impact points are distributed along the bridge length and marked “imp->” on Figure 5-1. The performed connector analysis (mass-spring system) that gives the impulse loads are marked ”Conn->”.

Bridge-axis	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22
Connector impact			Conn->					Conn->									Conn->			
Impulse impact	imp->		imp->					Imp->									Imp->			
Bridge-axis	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40	41	North_end
Connector impact				Conn->													Conn->			
Impulse impact				Imp-	Imp-	Imp->							Imp->			Imp-	Imp-	Imp->		

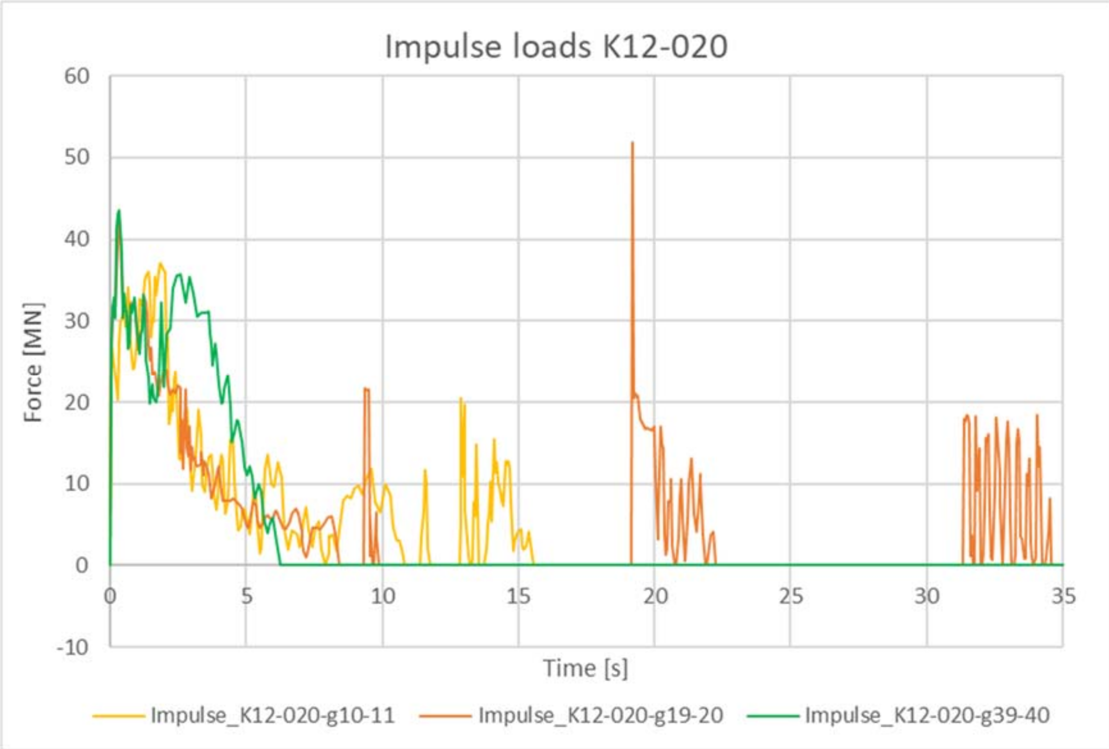
> *Figure 5-1 Screening analysis overview. Green cells are axis with anchored pontoons.*

The relationship between impact point and impulse load is shown in Table 5-1.

> *Table 5-1 Screening analysis setup*

Analysis name	Description	Impact point – between axis	Impulse load
BF-K12-020_g3-4(pos/neg)_imp	High bridge	3-4	Impulse_K12-020-g5-6
BF-K12-020_g5-6(pos/neg)_imp	High bridge	5-6	Impulse_K12-020-g5-6
BF-K12-020_g10-11(pos/neg)_imp	Center anchor group high bridge	10-11	Impulse_K12-020-g10-11
BF-K12-020_g19-20(pos/neg)_imp	Center bridge	19-20	Impulse_K12-020-g19-20
BF-K12-020_g26-27(pos/neg)_imp	Outside anchor group low bridge	26-27	Impulse_K12-020-g26-27
BF-K12-020_g27-28(pos/neg)_imp	Anchor group low bridge	27-28	Impulse_K12-020-g26-27
BF-K12-020_g28-29(pos/neg)_imp	Anchor group low bridge	28-29	Impulse_K12-020-g10-11
BF-K12-020_g35-36(pos/neg)_imp	¾-point low bridge	35-36	Impulse_K12-020-g19-20
BF-K12-020_g38-39(pos/neg)_imp	Towards north 1	38-39	Impulse_K12-020-g39-40
BF-K12-020_g39-40(pos/neg)_imp	Towards north 2	39-40	Impulse_K12-020-g39-40
BF-K12-020_g40-41(pos/neg)_imp	Towards north 3	40-41	Impulse_K12-020-g39-40

Representative impulse loads have been added to the bridge in several impact points in order to record the maximum response of the bridge. See a selection of relevant impulse loads in Figure 5-2 below and in Appendix C-C.2.



> *Figure 5-2 Relevant impulse loads on bridge K12.*

5.2 Response from screening analysis

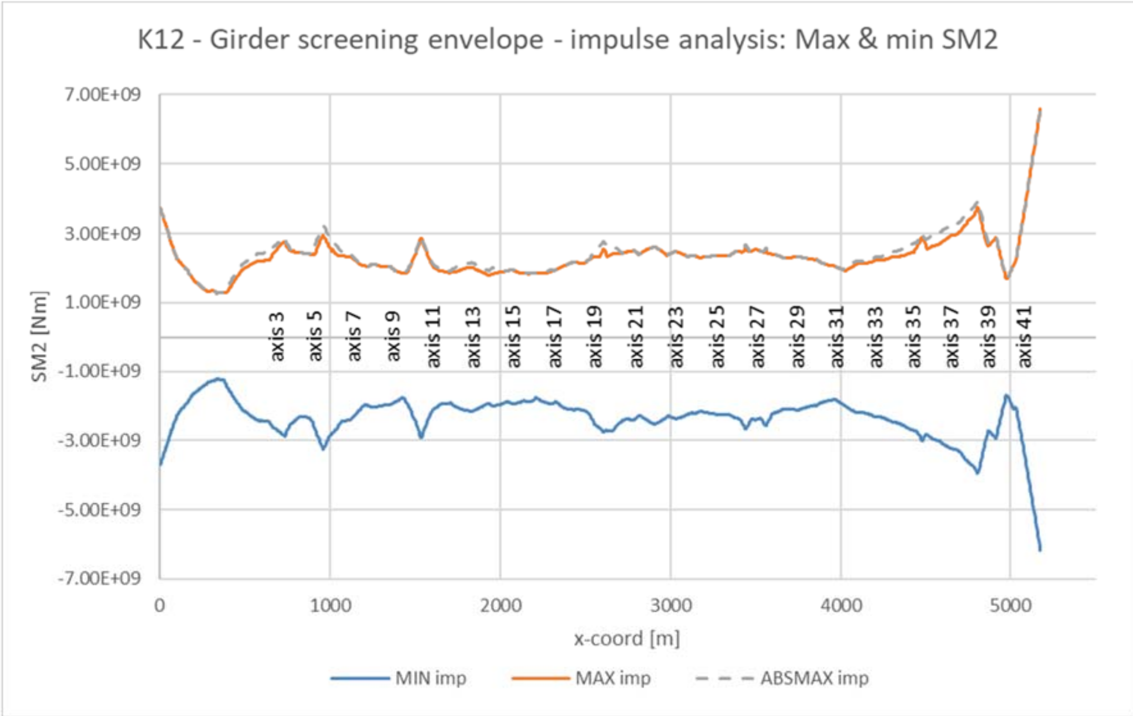
The response presented in this report is the response that is regarded as relevant when comparing the different bridge concepts. These are

- Bridge girder strong axis bending moment
- Maximum displacement of anchor point (K12-K14) – gives maximum elongation of anchor line
- Horizontal displacement of bridge girder at bridge tower, orthogonal to bridge girder.

Important assumptions:

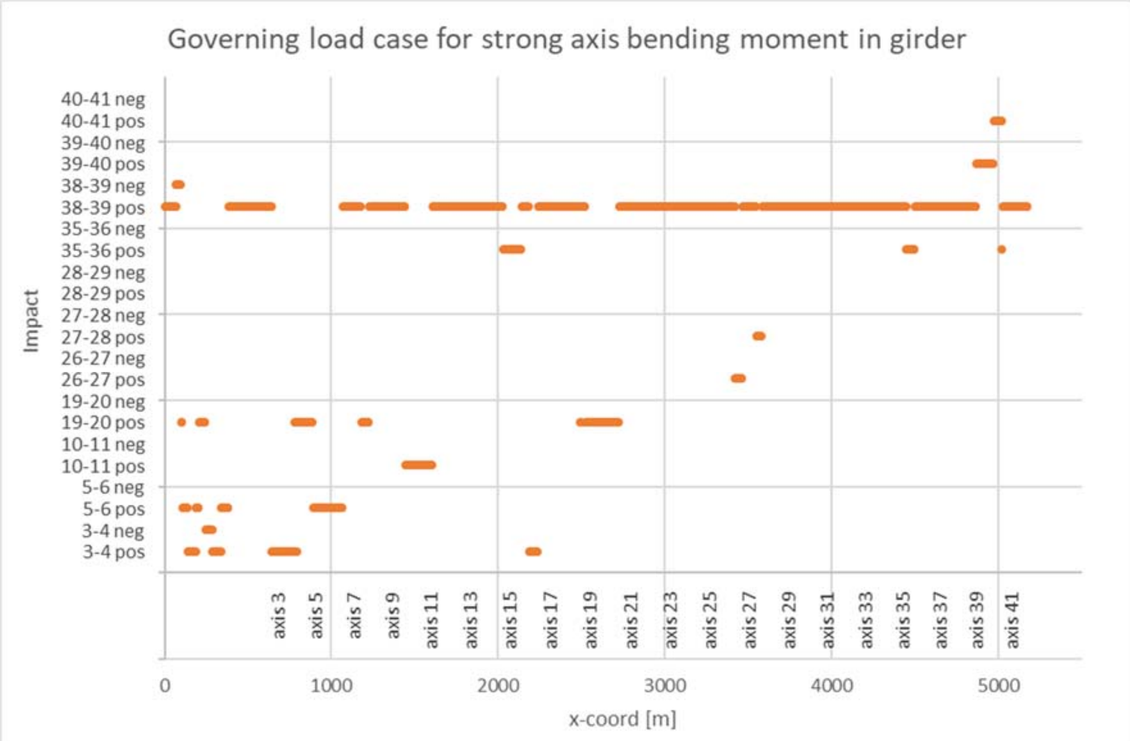
- Traffic loads are neglected – the effect of traffic loads is described in section 6.5.
- For screening analysis, there has only been considered impacts to the bridge girder, as this is the impact with most energy and will transfer the most energy to global girder motions.

Results from the screening analysis are presented below. The strong axis bending moments is quite equal along the whole girder – except from the fixpoints at the two bridge ends, where the northern end naturally gives highest strong axis bending moments.



> *Figure 5-3 Max and min envelopes for strong axis bending moments (SM2), [Nm]*

All the moment diagrams are shown in Appendix C-C.2. The governing impacts for these bending moments are shown in Figure 5-4.



> *Figure 5-4 Governing impacts along girder for strong axis bending moment in girder, based on screening impulse analysis.*

5.3 Input to design calculations

Relevant input to design calculations from the screening analysis are the strong axis bending moments, the horizontal displacement of the girder at the land tower and the elongation of the anchor lines due to ship impact. The bending moments are shown in section 5.2 and the horizontal displacement at land tower and elongation of mooring lines are shown in Appendix C-C.2.

The highlighted results are shown in Table 5-2.

> *Table 5-2 Bridge girder responses from screening analysis*

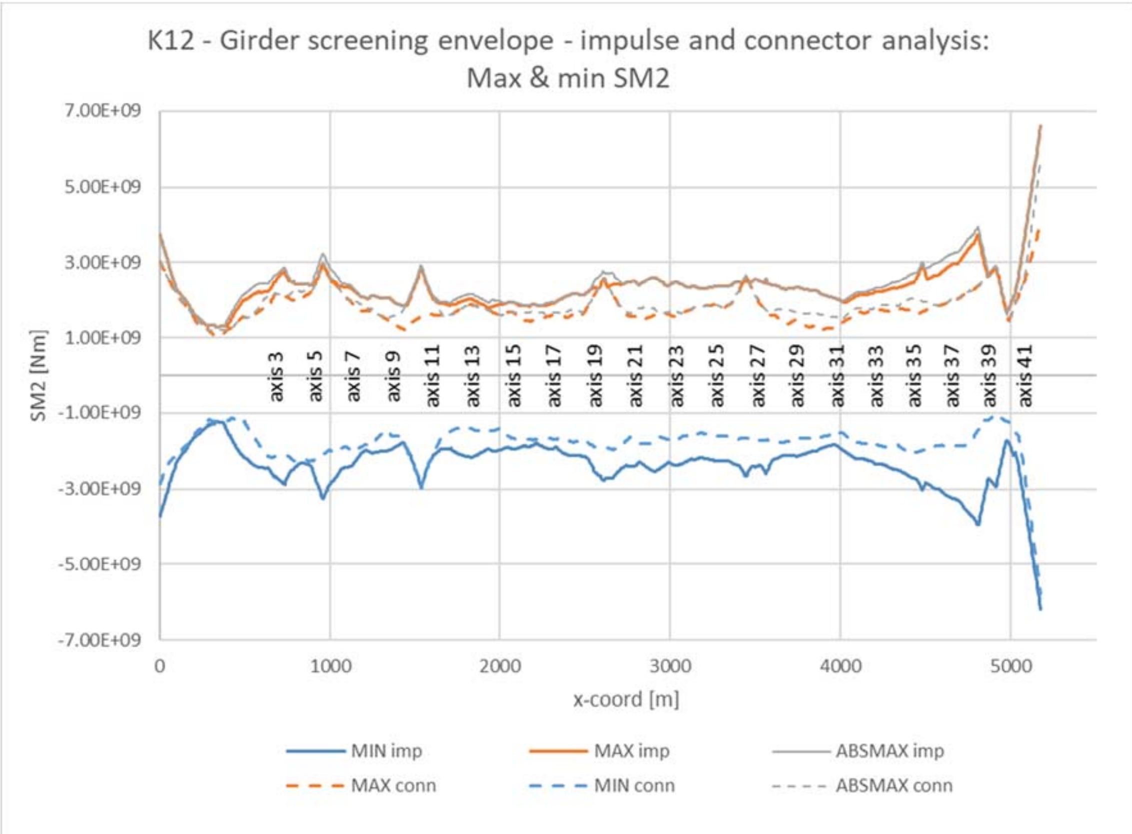
Measure parameter	Maximum response
Strong axis bending moment in south end	3,74 GNm
Strong axis bending moment in "span", between abutments	2,95 GNm
Strong axis bending moment in north end	6,60 GNm
Maximum displacement anchor line and position anchor line	13,25 m – pontoon axis 30
Maximum horizontal displacement of girder at bridge tower, orthogonal to girder.	3,85 m – impact between axis 38-39

Input to design calculations includes:

- Cable and land tower forces to design of cable stayed bridge (transferred as pure data), report [13].
- Strong axis girder bending moment in north end to girder design. See report [14].
- Anchor line elongation to the mooring line and anchor design. See report [6].

5.4 Verification of impulse analysis

Results from the impulse analysis have been compared to the connector analysis with the mass-spring system. Figure 5-5 shows the envelope of max and min girder strong axis bending moment. As there are more impact scenarios in the impulse analysis the moment here is generally larger, but at the corresponding points the moments are almost identical.



> *Figure 5-5 Comparison between screening results from impulse analysis (solid lines) and from connector analysis (dashed lines) – strong axis bending moment in girder, SM2 [Nm].*

In Figure 5-7 and Figure 5-8 the same impacts have been performed with a connector analysis, a load impulse based on the connector analysis and a load impulse based on a similar impact on another location. The compared impacts are the axis 10-11-impact and the 26-27-impact, which are both impacts near anchored pontoons, but the 10-11-impact is in the center of the anchor group near the high bridge, and the 26-27-impact is situated right outside the anchor group on the low bridge. The impulse loads are similar but not equal, see Appendix C-C.2.

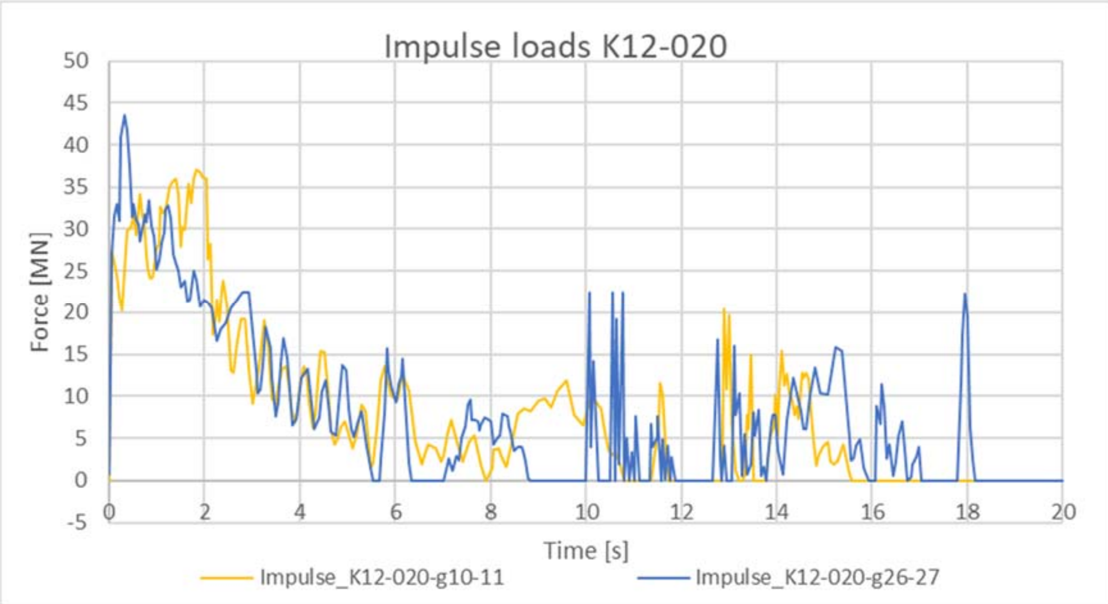


Figure 5-6 Compared impulse loads on impacts g10-11 and g26-27

The absolute maximum strong axis bending moments in girder based on the three load approaches are almost equal, see Figure 5-7 and Figure 5-8.

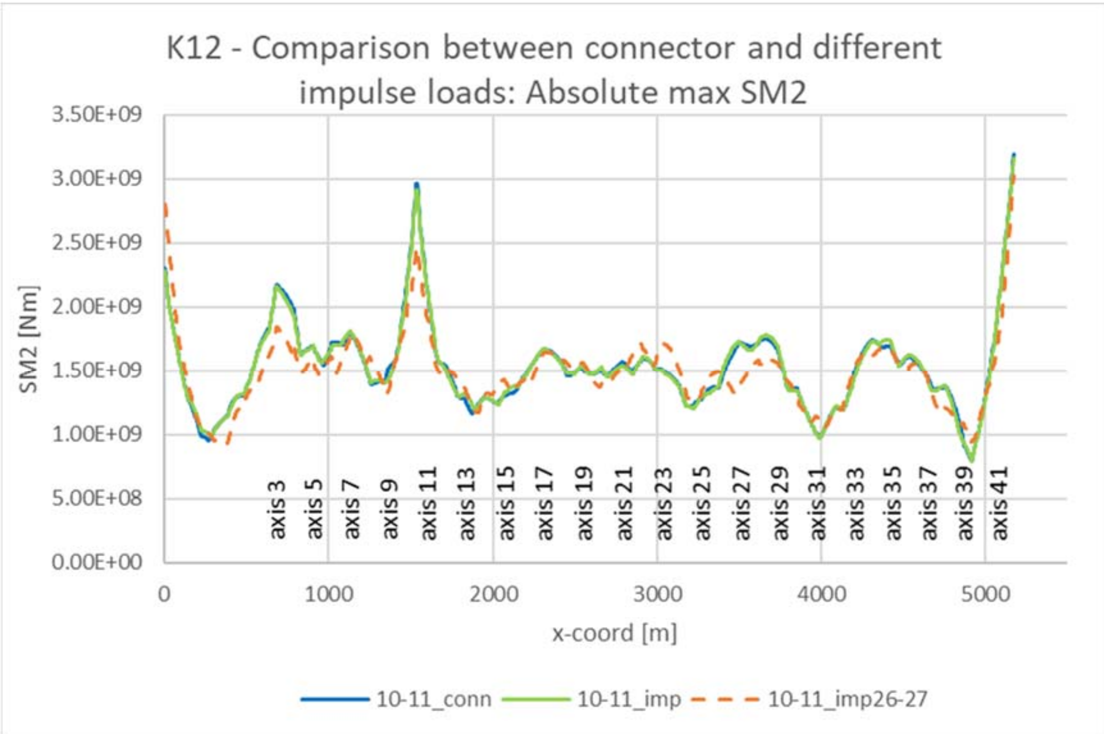
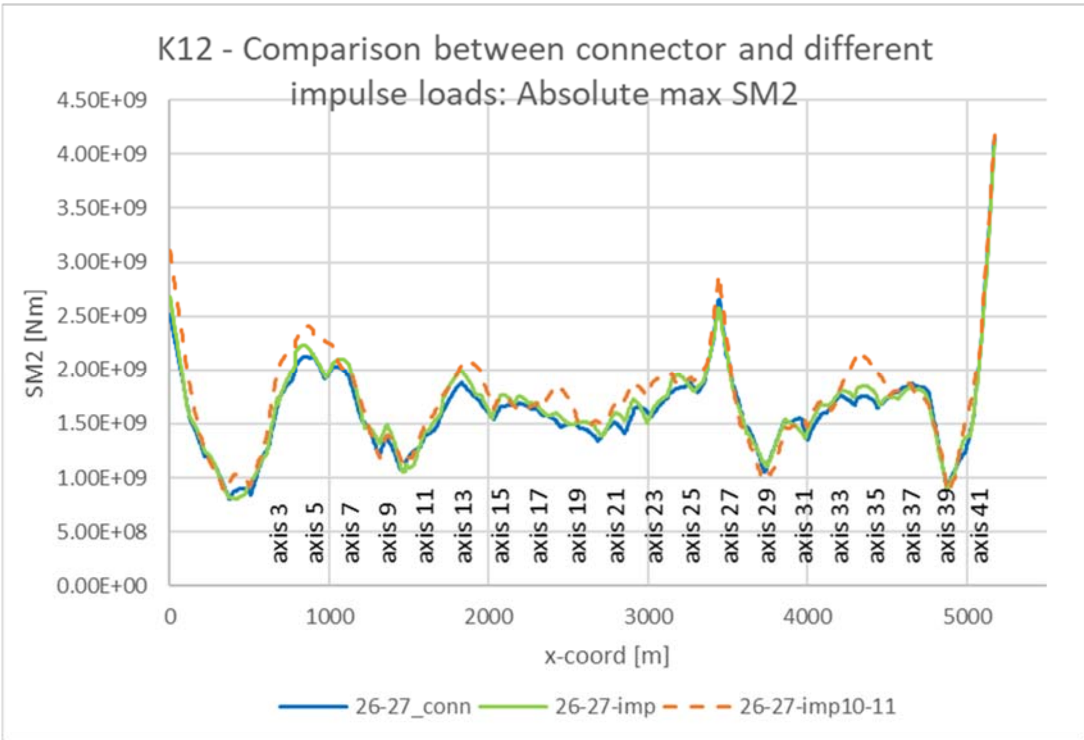


Figure 5-7 Comparison between screening result from connector analysis g10-11, impulse analysis based on connector analysis g10-11 and impulse load based on connector analysis from similar impact g26-27 – absolute max strong axis bending moment in girder, SM2 [Nm].



> *Figure 5-8 Comparison between screening result from connector analysis g26-27, impulse analysis based on connector analysis g26-27 and impulse load based on connector analysis from similar impact g10-11 – absolute max strong axis bending moment in girder, SM2 [Nm].*

The results from equal impulse load are very similar, as the global motions of the bridge are slow – the reaction time for the bridge is much longer than the impact time. First eigenmode period is close to one minute, while most of the impact energy is transferred the first 10 seconds.

Using impulse loads instead of spring-mass system connector analysis for screening and global response is considered as a satisfactory analysis method.

6 SIMULATIONS

6.1 Response parameters

See chapter 5 for global response of bridge, such as girder bending moments. In this chapter the more specific ship impact results are presented – indentation between ship and girder/pontoon, distribution of impact energy between plastic dissipation and global bridge motion etc.

6.2 Ship impact on pontoons and columns

Bow collision with pontoon is handled in the local impact report, SBJ-33-C5-OON-22-RE-015-K12 - Ship impact, Pontoons and columns, [3]. The local analysis gives a force-indentation curve used for further evaluations and as input to the global analysis. These are shown in Figure 3-4.

For ship-pontoon impact evaluations three pontoons have been chosen for detailed studies:

1) Axis 3 – large pontoon with the tallest column
2) Axis 12 – anchored pontoon (medium size) with quite large impact energy
3) Axis 20 – small pontoon at the center of the bridge

The three pontoons have been evaluated for three different impacts:

a) Head on impact – impact at pontoon end, orthogonal to bridge girder
b) 90-degree centric impact – impact at pontoon center, impact direction alongside bridge girder
c) 90-degree eccentric impact – impact eccentric on pontoon (at transition between straight long side and curved end), impact direction alongside bridge girder

The impact points are illustrated on Figure 3-3.

These three impacts are considered sufficient for evaluation of ship impacts at pontoons. Impacts at different locations or from different angles will be covered by these three cases.

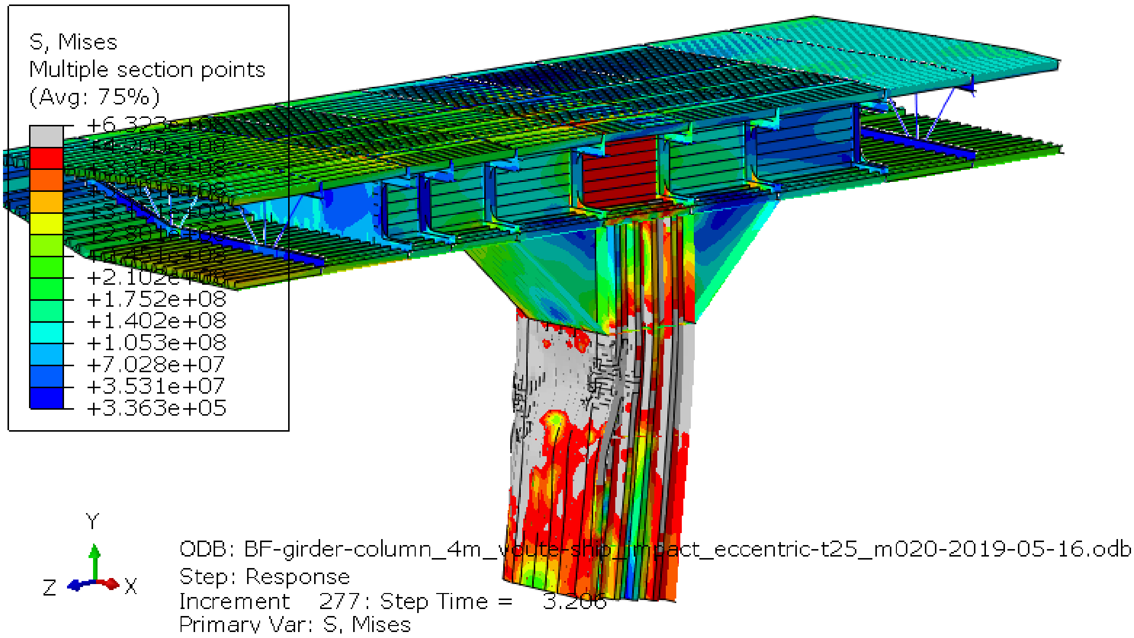
6.2.1 Plastic hinge in column for 90-degree pontoon impact

The bridge design is sensitive to the strength of the column, especially the connection between the column and the girder. For the 90-degree pontoon impact on the high bridge, a shear force in the bottom of the column of 30-35 MN leads to bending moments in the column top of 13-1500 MNm. As the weak axis elastic capacity of the "normal" bridge girder is about 650 MNm, the 90-degree ship-impact will lead to local plastic deformations in the girder if the column is not made weaker. For the repair of the bridge, it is easier to change a column than replace a part of the girder. Therefore, the column needs to be designed weaker than the girder. This is solved by reinforcing the girder locally and to design the column to withstand the given ship impact, but not more. In this way, the ship impact damage is limited to the column top in addition to the pontoon.

The 90-degree pontoon impacts lead to high section forces in the columns, both in bending and torsion. The tall columns are both highly utilized for bending and torsion, while the short columns are highly utilized for torsion. There has been performed FE-analysis of this detail to ensure a good design of this connection. These analyses are governing for the design of the column top and girder reinforcement above columns. See Appendix F for model description and results.

Tall columns

The tall columns need to be accurately designed: The column weak axis bending resistance must be lower than the girder weak axis bending resistance to make sure there is limited damage in the girder at the impact. At the same time, it must be strong and ductile enough to withstand the impact and the following post impact state. To make sure the girder behaves elastic, there is placed a voute on the top of the high bridge columns to avoid stress concentrations in the girder. This ensures a plastic hinge in the columns, below the voute.



> *Figure 6-1 Plastic hinge in tall column (axis 3), local model. Local reinforcement of girder and a voute in the column top makes sure most plastic deformations takes place in the column. SeeAppendix F for details.*

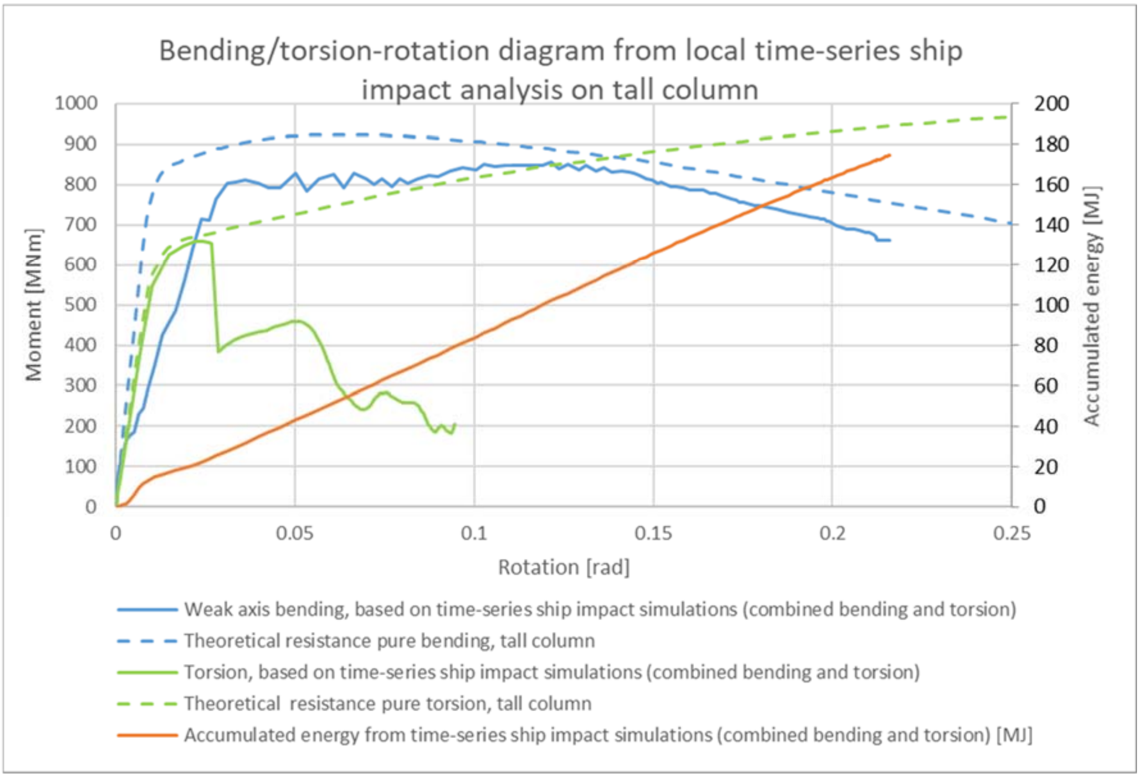
Short columns

For the low bridge columns, the situation is a bit different as the shorter column leads to a lower bending moment at the top of the column, and the bridge girder weak axis capacity is no longer governing for the design of the bending resistance of the column. As the column is stiffer both for weak axis bending and torsion, the damage will mainly happen between the ship and the pontoon, and the shear force and torsion in the short columns will be higher than for the high bridge long columns. Hence, the short columns need to be stronger than the high columns, especially for torsion moments.

Implementation in global analysis

The results from the local analysis presented in Appendix F are implemented into the global analysis if needed. This means if the elastic section forces in the column top exceeds the elastic resistance, the column top is replaced by a plastic hinge. This hinge is a connector element with M-phi-diagrams for weak axis bending and torsion, obtained from the analysis in Appendix F. This is needed only for the 90-degree impacts. In axis 3 it is needed both for centric and eccentric impact, while for the axis 12 and axis 20-impacts it is only needed for the eccentric impact.

Figure 6-2 shows moment-rotation relationships from local analyses on the column plastic hinge in the tall columns. The solid lines are used as input to the global analyses and are recorded from a ship impact time domain analysis on the local model, see Appendix F for details. There has also been performed isolated "push-over"-analysis for pure bending and pure torsion which gives the upper limits, shown as dashed lines. As the interaction between bending and torsion is important for the behavior, the properties based on results from the combined torsion and bending local analysis is implemented into the global model as a plastic hinge.



> *Figure 6-2 Bending/torsion-rotation diagrams from the column plastic hinge evaluations.*

Optimization of column crown design

In Appendix F.1-F.4 there has been performed local analysis of the column in order to obtain a good column design. The reinforcements in the girder above the column, the column crown, are here oversized. To get a better design of the column crown, analyses has been performed in Appendix F.5-F.7. These analyses are the basis for the girder reinforcements above columns shown on drawings.

6.2.2 Results from ship impacts on pontoons

Ship impacts on pontoons are governing for the column design, especially the tall columns. They are also governing for the design of the girder stiffeners and bulkheads above columns.

Extended results from ship impact on pontoons are shown in Appendix C.4. The main results are presented here.

The ship impact characteristics are given by the design basis. The loads are presented in Table 6-1. See also section 3.2.

> *Table 6-1 Ship impact characteristics pontoon impacts axis 3, 12 and 20.*

Impact characterstic	Axis 3	Axis 12	Axis 20
Ship mass (incl. 5 % added mass) [tonne]	15 293	13 922	13 922
Ship initial velocity [m/s]	5,7	5,5	5,5

Ship initial kinetic energy [MJ]	248	228	228

The main results from the ship-pontoon impacts are presented in Table 6-2. The three force-displacement curves from Figure 3-4 are used as input. This means the same force-indentation curve is used for both 0-deg and 90-deg ice bow impacts, which probably underestimates the contact force (and overestimates the indentation) in the 0-deg ice bow impact.

> *Table 6-2 Main results from impacts on pontoons axis 3, 12 and 20.*

Parameter	Impact ship and direction	Pontoon axis 3	Pontoon axis 12	Pontoon axis 20
Maximum indentation between ship and pontoon [m]	0-deg, container	2,0	2,5	2,3
	0-deg, ice bow	8,0	9,5	9,0
	90-deg centric, container	3,5	-	8,0
	90-deg centric, ice bow	8,5	-	13
	90-deg eccentric, container	2,8	7,0	7,5
Maximum force between ship and pontoon [MN]	0-deg, container	32	33	33
	0-deg, ice bow	21	30	30
	90-deg centric, container	25	-	40
	90-deg centric, ice bow	28	-	33
	90-deg eccentric, container	25	33	40
Plastic dissipation (energy) between ship and pontoon [MJ]	0-deg, container	50	65	60
	0-deg, ice bow	70	95	85
	90-deg centric, container	70	-	185
	90-deg centric, ice bow	90	-	200
	90-deg eccentric, container	55	170	180
Plastic dissipation (energy) in column top plastic hinge [MJ]	0-deg, container	-	-	-
	0-deg, ice bow	-	-	-
	90-deg centric, container	170	-	-
	90-deg centric, ice bow	150	-	-
	90-deg eccentric, container	185	10	-
Maximum elongation of anchor line [m]	0-deg, container	11	8	7
	0-deg, ice bow	11	7,5	6,5

Extended results are shown in Appendix C.4 as time series. Noticeable results from the ship pontoon impacts are listed below:

- In the centric 90-degree impact on axis 3 with the tall column, 240 of 248 MJ, or 97 % of the initial kinetic energy is dissipated locally in the pontoon and column. Most in the column – which means this is a very critical detail for the bridge design. The plastic displacement of the pontoon center due to weak axis rotation in the column top is 10 m. This gives an extra second order moment from the buoyancy load in the post-impact state.

- Maximum indentation between ship and pontoon is 8,5 m or more on all three pontoon types, which means that water ingress in (maximum) 4 pontoon compartments must be expected for all pontoon types.
- The maximum elongation of anchor line is as expected (due to less energy) less than for the girder impact.
- The maximum indentation is a 90-degree centric impact on the small pontoon in axis 20. The total indentation is 13 m, which means the bulb could penetrate all the way through the pontoon which has a width of 12 m. See force-indentation curve in Figure 6-3 and damage in ship and pontoon on Figure 6-4.

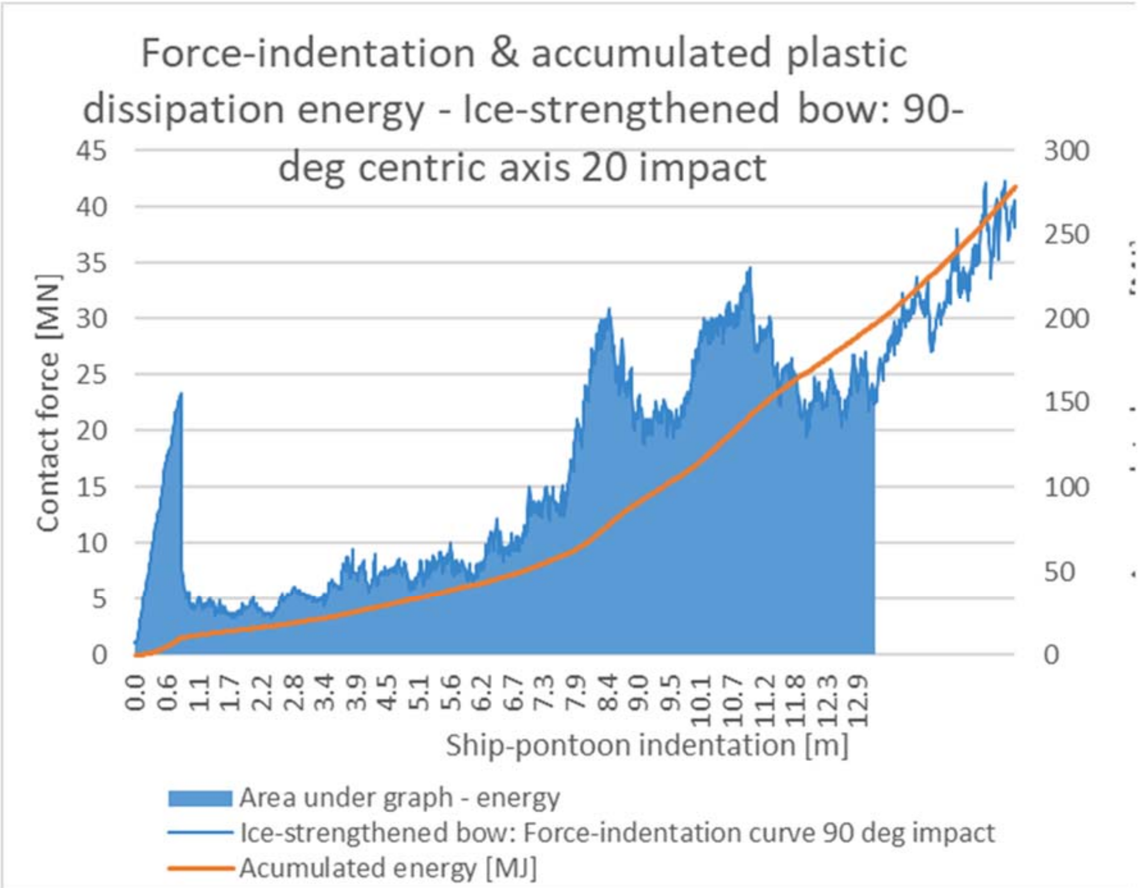
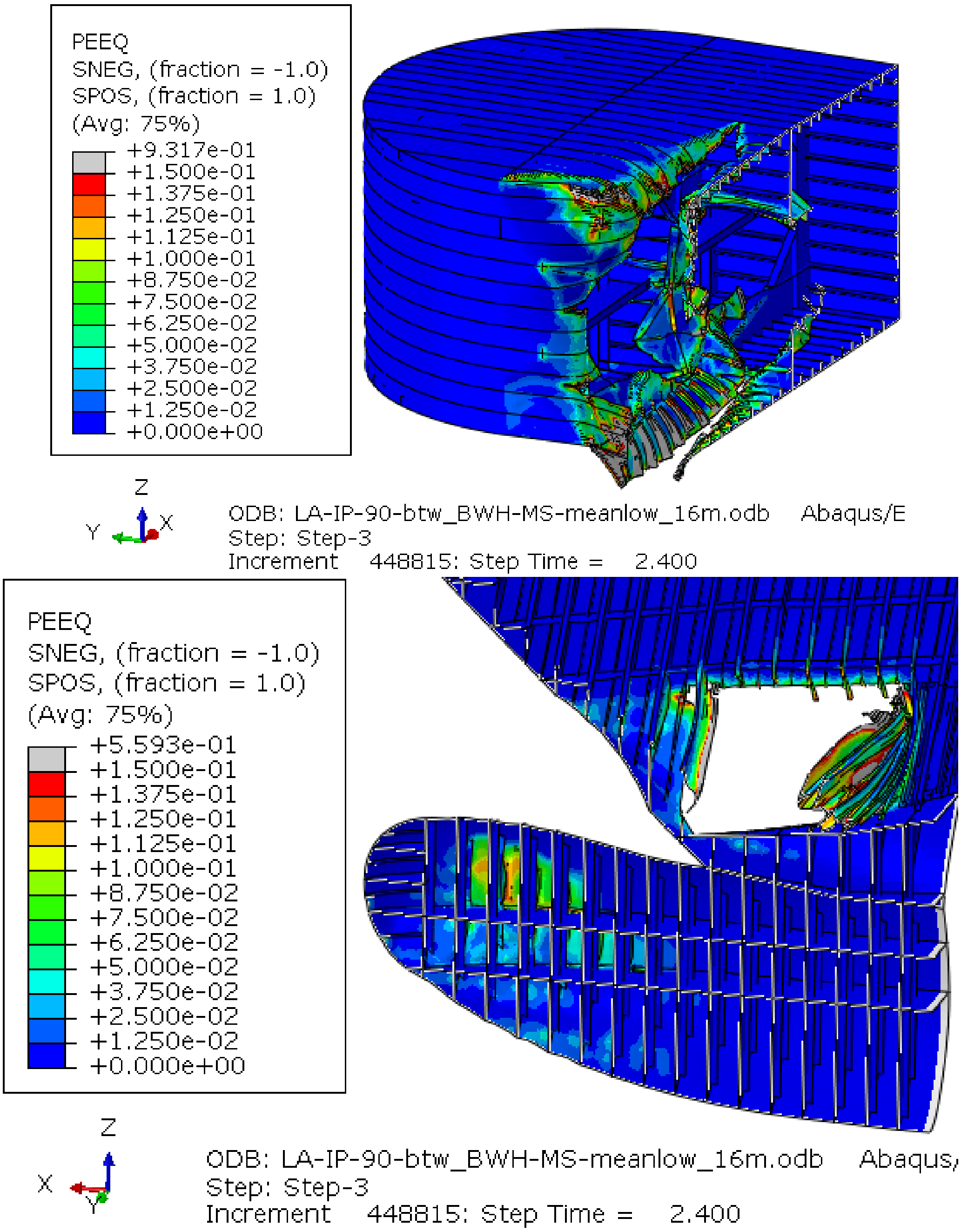


Figure 6-3 Force-indentation curve for pontoon impact with ice strengthened bulb. Total indentation is 13 m and the dissipated plastic energy is 200 MJ.



> *Figure 6-4 Damaged ship and pontoon at 13 m indentation between ship and pontoon. Taken from local analysis as described in local analysis report [3]. This pontoon is 16 m wide, while the axis 20 pontoon with the 13 m indentation is 12 m wide. This impact could penetrate the entire pontoon.*

6.2.3 Ship impact on columns

The ship impact on column is not evaluated in this report, as it will not be governing for neither global design nor the connection between column and girder. The column is stiffer than the ship bow, which leads to low force-indentation curves. The bulb-pontoon-impact from the container ship is governing for the girder-column connection.

See the local impact report, SBJ-33-C5-OON-22-RE-015-K12 - Ship impact, Pontoons and columns [3], for details.

6.3 Ship impact on bridge girder

All girder impacts have been placed midspan between the axis. The global response of the bridge is presented in the screening analysis in chapter 5, only the results related to the impact itself is presented here. This is the total indentation of the impact in meters, and the amount of energy transferred to local damage as plastic dissipation in the connector between girder and ship. The total amount of energy in the girder impact is 385 MJ, see Figure 3-2. The remaining energy is mainly transferred into elastic strain and kinetic energy in the bridge. For evaluation of the damage and distribution of indentation and energy between the deckhouse and girder, see SBJ-33-C5-OON-22-RE-013 – K12 – Ship impact, Bridge girder [4].

> *Table 6-3 Maximum indentation between deckhouse and girder along bridge*

Impact between axis	Description	Indentation [m]	Plastic dissipation energy [MJ]
5-6	Ramp, near cable bridge	5,9	178
10-11	Center of first anchor group	5,6	157
19-20	Center of bridge	5,3	160
26-27	Right outside second anchor group	5,4	164
39-40	Towards north, gives large bending moment in northern abutment	8,0	241
41-north end	Close to abutment north, stiffest impact	12,9	383

6.4 Post impact capacity

According to design basis the bridge must withstand a 100-years storm post impact in an accidental limit state, see section 3.3. Five main concerns a ship impact could lead to is investigated in this section:

1) Local damage in bridge girder
2) Local damage in pontoon, leading to water filling of pontoon compartments
3) Local damage in tall columns due to pontoon impact
4) Local damage of column due to direct hit from a ship bow
5) Loss of anchor lines

These five points are discussed in the following sections.

6.4.1 1) Local damage in bridge girder

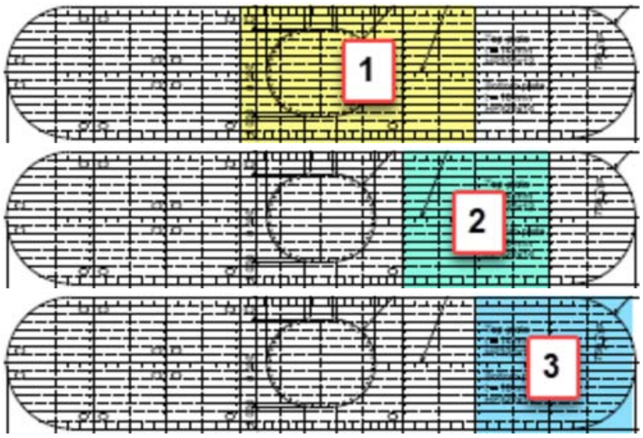
The residual capacity of the bridge girder is analyzed in the "Ship impact, bridge girder" report [4]. This shows a residual capacity of at least 80 % for both weak axis and strong axis bending moments in the girder even at 16 m indentation between deckhouse and girder. As the 100-years environmental loading case also is a ULS-case, the reductions of load- and material factors from ULS to ALS itself reduces the need of strength with a factor of 1/1,6 = 63 %. It can be concluded without further investigations the girder can withstand a 100-years ALS storm post impact.

6.4.2 2) Local damage of pontoon

There has been performed considerations of the post impact properties of the pontoons. The given scenario is 8,5-13 m indentation between ship and pontoon, see Table 6-2, which theoretically could lead to water filling of four compartments.

The considerations done are simplified and conservative.

There has been considered three different impact scenarios for all three pontoons, see Figure 6-5.



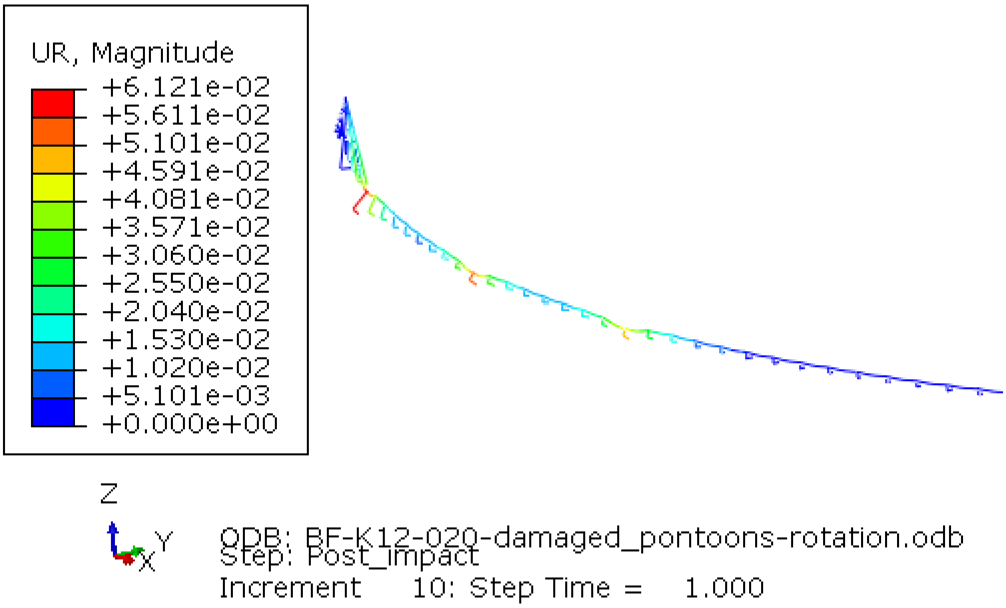
> *Figure 6-5 Considered impact scenarios on pontoons.*

The three scenarios give different effects. Scenario 1 gives the largest draft, while scenario 3 gives the largest rotation.

Changed pontoon properties due to water filling of pontoon compartments

The increased draft due to water filling of pontoon compartments is calculated using the global FE-model and iterating on draft and buoyancy. Simplified, the largest loss of draft area (scenario 1) is combined with the largest loss of pitch rotational stiffness (scenario 3). When the new draft and buoyancy is found, the corresponding bending moment from the eccentricity of the buoyancy load is applied on the global model.

The water plane stiffness of three damaged pontoons are changed in the model: axis 3, 12 and 20. An increased draft means that nearby pontoons carries more of the girder weight, so this needs to be recalculated a few times to get consistent results. The buoyancy in the global FE-model is iterated until it is corresponding with the draft. Static results from the post-impact state is shown in Figure 6-6.



> *Figure 6-6 Post impact static state, pontoons in axis 3, 12 and 20 (will not happen at the same time). Scale factor 15 in plot.*

Damaged pontoon properties, displacement of buoyancy center and static rotations post impact are calculated in Appendix D. The results are presented in detail in Appendix D.1.

Highlights from the considerations are presented in Table 6-4.

> *Table 6-4 Damaged pontoon properties*

Pontoon property	Pontoons high bridge	Pontoons ramp and anchors	Pontoons low bridge
Pontoon width [m]	17,0	14,5	12,0
Maximum change in static draft due to reduced buoyancy and vertical water plane stiffness (centric impact) [m]	1,31	1,19	1,15
Minimum remaining water plane vertical stiffness (centric impact, compared to undamaged state)	60 %	60 %	61 %
Minimum remaining strong axis rotation stiffness (eccentric impact, compared to undamaged state)	57 %	57 %	57 %
Maximum buoyancy eccentricity moment [MNm]	292,7	250,0	209,7
Maximum static rotation (local pitch) of pontoon centers, results from FE-model - see Appendix D section D.2 [deg]	3,5	3,1	2,6
Maximum static change in draft at damaged pontoon end, draft + rotation [m]	3,1	2,7	2,5

For the large pontoons the static increased draft of pontoon end is 3,1 m. The freeboard is 4 m, so in a 100-years environmental case it must be expected green water on the pontoon deck and overtopping of waves. Both strong axis bending moment capacity of the columns and torsion capacity of the girder are very high and the construction elements are low utilized, so this is not considered critical for the bridge. See evaluations of this in Appendix D.2.

6.4.3 3) Local damage in columns due to pontoon impact

The local damage in the column needs to be combined with the results from 2) as they will both affect the column post-impact section forces. Sideway ship impact (90-deg) on pontoons leads to large indentations of pontoons and on the high bridge it also gives large plastic weak-axis rotations of column top. These cases have been investigated more closely in this section and in Appendix D.

Loss of buoyancy leads to increased draft and increased loading from current, swell and wind-generated waves. For the tall columns, plastic rotations of the column top leads to permanent displacements of pontoon center (and buoyancy center), which gives second order moments in the column from the heave (buoyancy) load. These are also the columns with largest weak axis bending moment from 100-years environmental case.

As a conservative approach for the tall columns the maximum indentation in the column is combined with the maximum plastic rotations of the column top.

Increased environmental loads due to increased draft

Due to increased draft the environmental loads on the pontoon will increase:

- Heave forces are kept the same, as this is a quite fast motion and probably much faster than the rate of changing the water level inside the pontoon. The vertical water plane stiffness is considered unchanged for the load evaluation.
- Forces from current, sway and wind-generated waves are increased with the increased draft. These forces decrease with the water depth but are conservatively uniformly distributed on the pontoon in these evaluations.

Total increased weak axis bending moments in columns

The weak axis bending moment of the columns has been calculated in Appendix D.3. The highlights are presented here.

> *Table 6-5 Highlighted section forces in columns during 100-years environmental case. See Appendix D for details.*

Column section force during 100-years environmental loading	Column axis 3	Column axis 12	Column axis 20
Axial force, damaged state (same as for undamaged state) [MN]	33,4	25,2	25,2
Weak axis bending moment at column top, undamaged state [MNm]	227	30,6	47,8
Post impact environmental scale factor for shear forces (due to increased draft)	1,26	1,24	1,23
Weak axis bending moment at column top, damaged state (incl scale factor) [MNm]	286,9	37,8	58,8
Maximum weak axis bending moment due to eccentric buoyancy load from plastic rotation in column "hinge" [MNm][1]	334	25,2	25,2
Total weak axis bending moment at column top [MNm]	620,9	63,0	84,0

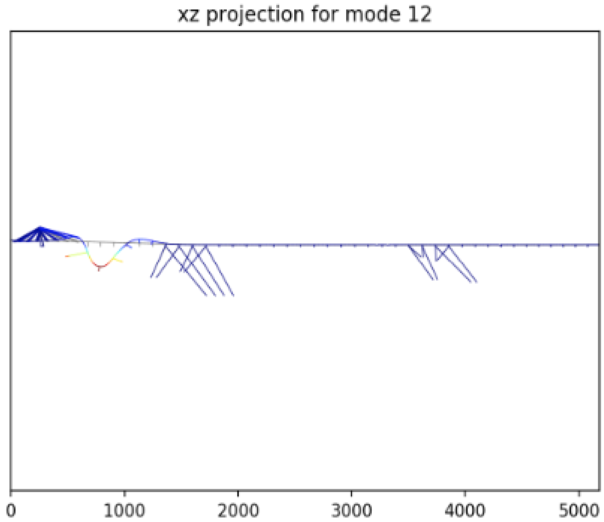
The weak axis bending resistance is approximately 800 MNm, see pushover analysis in Appendix F. This means the columns remains within the elastic area in the 100-years environmental loading.

As the weak axis second order moment from the buoyancy force is larger than the moment from environmental loads, the post-impact state is depending on the column design. If the weak axis capacity and ductility of the column is designed too low, the second order moment from the buoyancy could be higher than the bending moment capacity.

Change in dynamic behavior due to increased pontoon mass

The change of pontoon mass in the axis 3 pontoon due to filling of water will change the modal properties of the bridge. The modes 12 and 13 are pendulum modes where mainly the high bridge is participating, see mode 12 in Figure 6-7. The same goes for change of mass in pontoons in axis 4 and 5. The pendulum modes are mainly trigged by the wind-sea. Sway gives little response on the bridge.

[1] The eccentricity of the buoyancy load for the axis 3 column is based on the results from the pontoon impact analysis, see section 6.2.2. For the short columns in axis 12 and 20 it is set to 1 m. See Appendix D for details.



> *Figure 6-7 Mode 12 – pendulum mode of high bridge columns (model K12-020). From interactive and structural response analyses [11].*

The wind sea has eigen periods lower than 5,5 second, see "Design basis MetOcean_rev_1", [15], while the pendulum modes have eigen periods above 7 seconds in an undamaged state. An increase of the time period for the pendulum modes will not lead to an increase in the loads due to dynamic effects, rather a decrease.

The bridge response is not expected to be affected by increased mass due to water filling of the pontoons in the high bridge. For the low bridge the weak axis bending moments in the columns are very low for the 100-years environmental condition, see plot in Appendix D.3, so they can handle a large increase in bending moment.

6.4.4 4) Local damage of column due to direct hit

The direct column hit is investigated in the local analysis of pontoons and columns report [3]. The column is both stronger and stiffer than the bow of the investigated container ship, so the real translation of forces will be between the bulb and the pontoon. Point 4) is not governing for the column design.

6.4.5 5) Loss of anchor lines

The anchor lines can take 15-20 m elongation, see report "Design of mooring and anchoring" [6]. Maximum elongation of anchor line due to ship impact is 13,5 m, see Table 5-2 in the screening analysis section. The lines are fixed under the pontoon, but as the largest indentation from the ship-pontoon analysis are more than half the pontoon width, there must be expected loss of at least one anchor line.

The design basis gives loss of two anchor lines as a load case, so this is handled by the "Design of mooring and anchoring" report [6].

In the case of loss of one anchor line the strong axis bending moment of the column will be slightly increased. Maximum extra bending moment from anchor line is 20 m x 0.1 MN/m x 50 m = 100 MNm, which is neglectable when comparing with the elastic capacity of about 1500 MNm. In the post impact environmental 100-years condition the columns are in general low utilized for strong axis bending, see results on interactive or the structural response report [11].

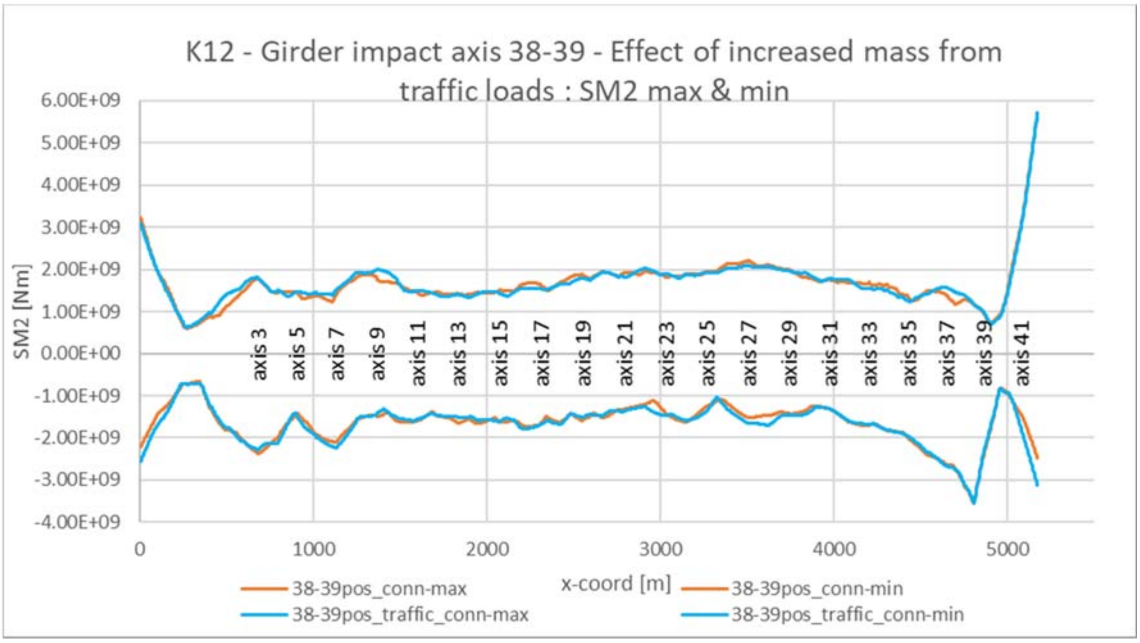
6.5 Traffic loads on bridge

The combination of ship impact and traffic loads on the bridge has not been evaluated in detail in this report. Globally the traffic loads mainly affect the girder weak axis bending moments and the cable bridge, while the ship impact affects the girder strong axis bending moment, the column and the pontoons.

6.5.1 Global response sensitivity to traffic loads

There has been performed a simplified global analysis of the girder impact between axis 38 and 39 as this is the governing ship impact strong axis load case for a large part of the girder (see screening analysis in chapter 5). Only the traffic mass has been included as increased girder density in order to see the effect of the changed bridge response. See Appendix E for details.

The results from the two cases are very similar, see girder strong axis bending moments in Figure 6-1. The deckhouse-girder-indentation is also very similar; just below 6 m for the base case and just above 6 m for the traffic model. This is as expected as the increased girder mass gives more resistance as the girder needs more energy to be set into motion.



> *Figure 6-8 Effect of traffic loads on bridge. Comparison between two girder impacts between axis 38 and 39 with different girder densities. Max and min strong axis bending moment in girder, SM2 [Nm].*

Traffic loads on bridge is not expected to change the bridge behavior from ship impact.

6.5.2 Local effects from traffic loads

Local effects from traffic loads has not been evaluated at this stage. Concentrated traffic due to traffic jams will give a different global response in the bridge at a girder impact, but this is not expected to give large deviations.

After a ship impact the bridge will be closed for traffic if there are large damages. Scenarios with unfavorable traffic placements together with ship impact is not regarded likely.

7 SUMMARY AND CONCLUSIONS

The global response of the bridge due to ship impact has been studied in this report. The main focus is impacts between ship deckhouse and girder and between ship bulb and pontoon. Girder impacts all along the bridge length have been considered, orthogonal to the bridge girder from both directions. Pontoon impacts have been considered on all three pontoon types, at selected characteristic locations along the bridge. Three impacts are considered; head on (0-degrees) and centric and eccentric side impacts (90-degrees). Pontoon impact from a sideway drifting ship and submarine impact has also been discussed. Post impact the bridge must withstand 100-years environmental conditions.

The global ship impact analyses shows that the bridge will survive both a ship impact as given in the design basis and the following 100-years conditions.

A performed screening of girder impacts gives a maximum girder strong axis bending moment of almost 3000 MNm in the bridge "span", while it is 3750 MNm at the south end (near the cable stayed bridge) and 6600 MNm in the north end. This means the girder needs to be strengthened locally. The maximum elongation of anchor lines due to ship impact is 13,5 m, which is within the acceptable value. The robustness in general is quite good for the given ship impacts. The damage in girder due to girder impacts give small reductions of the moment capacities.

The results from the pontoon impacts are varying and very dependent on impact direction and type of ship. The minimum indentation from the design ship is 2,0 m while the maximum is 13,0 m. Impacts from a drifting ship is not expected to cause fracture in the pontoon, but it cannot be excluded. A submarine impact is not expected to cause fracture in the pontoon, but a direct hit on an anchor line could lead to loss of this.

For the head-on (0-degrees) pontoon impacts the indentations are less than 10 m, which gives satisfactory behavior of the bridge post impact. Some green water on deck and overtopping of waves on pontoon must be expected during the post-impact 100-years environmental condition.

The 90-degree impacts could give large plastic displacements in the tall columns on the high bridge and deep indentations in the columns on the low bridge. These special cases cannot dissipate much more energy before loss of entire pontoons could be a case. Increased robustness can be solved by a more detailed design of these critical parts.

7.1 Discussion

The design of the weak axis column-girder connection is critical. The column top should be designed to be weaker than the girder for weak axis bending moments, so this is where a plastic hinge will develop during a ship impact. Otherwise there will be large plastic deformations in the girder which is harder to repair/replace than the column. The column top must be strong enough to withstand the acting bending and torsion moments, ductile enough to dissipate the ship energy and weaker than the girder. This is the most critical detail due to ship impacts.

The column-girder design evaluated in this project phase shows ability to dissipate large amounts of energy in plastic deformations. The design should be optimized in the next project phase in order to obtain an even better performing "fracture mechanism" of the column.

The ship impact energy is expected to be reduced in the next phase. This will give lower global response and lower damage/indentation of pontoons and girder. It might still give large forces in the column and girder, so these are details that still needs to be addressed.

7.2 Further work

7.2.1 The girder-column connection in the high bridge

The girder-column connection on the high bridge is critical for the ability of the bridge to withstand a post-impact state and at the same time be repaired in a convenient way.

Therefore, the tall columns need to be accurately designed: The weak axis bending resistance must be lower than the girder weak axis bending resistance to make sure there is limited damage in the girder at the impact. At the same time it must be strong and ductile enough to withstand the impact and the following post impact state. This is critical in the design phase of the columns and needs to be addressed.

7.2.2 The girder-column connection in the low bridge

The girder-column connection in the low bridge is subjected to larger shear forces and torsion moments as the shorter columns gives a stiffer response in a ship impact. The plastic dissipation in the pontoon impacts on the low bridge mainly occurs between the ship and pontoon, therefore the girder-column connection can be reinforced without affecting the response too much. The girder-column connection for the short columns is more a design case, but the design needs to be developed further in the next phase.

The boundary between the long and short columns also needs to be located in order to obtain a correct design for each column in the ramp between the low and high bridge.

7.2.3 The effect of traffic loads combined with ship impact

As the ship impact mostly is governing for the local design of the pontoons and the columns while the traffic loads are governing for the girder design, there has not been performed detailed studies with the combination. This should be done according to design basis in the next project phase.

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