Statens vegvesen

Ferry free E39 –Fjord crossings Bjørnafjorden	304624

0	15.08.19	Issued for use	OHHK/DDS	SAH	KH	
Rev.	Publish date	Description	Made by	Checked by	Project appro.	Client appro.

Client Statens vegvesen	
Contractor Norconsult DR. TECHN. OLAV OLSEN	Contract no.: 18/91094

Document name: Design of pontoons and columns		
Document no.: SBJ-33-C5-OON-22-RE-018	Rev.: 0	Pages: 61

CONCEPT DEVELOPMENT FLOATING BRIDGE E39 BJØRNAFJORDEN

DESIGN OF PONTOONS AND COLUMNS

Norconsult

DR. TECHN. OLAV OLSEN

Prodtex IFE Pure Logic HEYERDAHL ARKITEKTER AS H&BB MIKO MARINE AS BUKSÉR OG BERGING FORCE TECHNOLOGY SWERIM

REPORT

Project name:
CONCEPT DEVELOPMENT FLOATING BRIDGE E39 BJØRNAFJORDEN

Document name:
DESIGN OF PONTOONS AND COLUMNS

Project number:	5187772/12777
Document number:	SBJ-33-C5-OON-22-RE-018
Date:	15.08.2019
Revision:	0
Number of pages:	61
Prepared by:	Odd H. Holt Kristensen/Daniel Dimmen Sæther
Controlled by:	Stein Atle Haugerud
Approved by:	Kolbjørn Høyland

Summary

Finite element models of two typical pontoon and column geometries have been developed. Both linear and non-linear finite element models are created.

The ULS utilization and ALS utilization of both pontoon geometries studied have been found acceptable.

The long column analyzed will collapse due to ship impact. However, this is a preferred collapse mechanism. The upper part of the long column will act as a weak link. The bridge girder connection will be designed to carry higher loading than the upper part of the column. If a ship impact occurs, the destroyed pontoon and column structures can easily be replaced because the bridge girder remains intact.

The short column is highly utilized in the ALS condition. Permanent damage in the bridge girder intersection area cannot be ruled out if not a similar "stronger than column bridge girder connection" is introduced in the same fashion that will be present for the long column.

The FLS methodology that can be used for finding fatigue utilization based on shell modelling is described, but a fatigue shell model should be part of a complete global model analysis to obtain reliable fatigue results. Fatigue analyses will be carried out in the next phase of the project. Highest fatigue utilization is expected to be present in the column-pontoon intersection area which is the same area that show highest ULS stresses. The column-bridge girder intersection area may also have substantial fatigue damage, but this fatigue utilization will be documented in other reports.

Table of Content

1 INTRODUCTION

1.1 Current report

This report describes the design of the pontoons and columns. The pontoons and columns are designed as conventional plated steel ship type structures. The design service life is 100 years. The conventional design makes production possible worldwide including Norwegian shipyards.

The top of the columns is connected to the bridge girder, and the bottom of the columns is connected to the top of the pontoons.

There is one central column at each pontoon supporting the bridge girder. The column design is rectangular with rounded corners. Inside the column there are internal walls

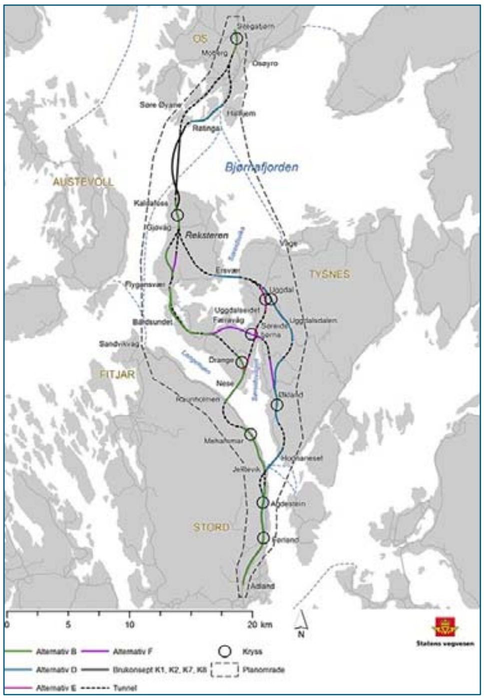
The pontoons are subdivided into compartments so that an accidental flooding of 2 compartments will not jeopardize the floating bridge integrity and post-accident behavior.

Both the pontoon and the column are designed to withstand ship impact. Permanent damage is allowed, but sufficient residual load carrying ALS capacity must be assured for both for the pontoon and the column. The pontoon is exposed to direct ship hit and will have the highest permanent damage.

Eight of the pontoons are anchored. The mooring lines goes via fairleads located in moonpools. Each anchored pontoon has two moonpools. The moonpools are located on each side of the column close to the pontoon center. The location of the moonpools, close to the pontoon center, reduces the risk of ship impact damaging the mooring lines.

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 LINEAR ANALYSIS

2.1 Geometry

The geometry used for finite element modelling and structural analyses in general is based on current drawing status. The geometry for both pontoons and columns is under continuous development.

2.2 Finite Element Models

2.2.1 General

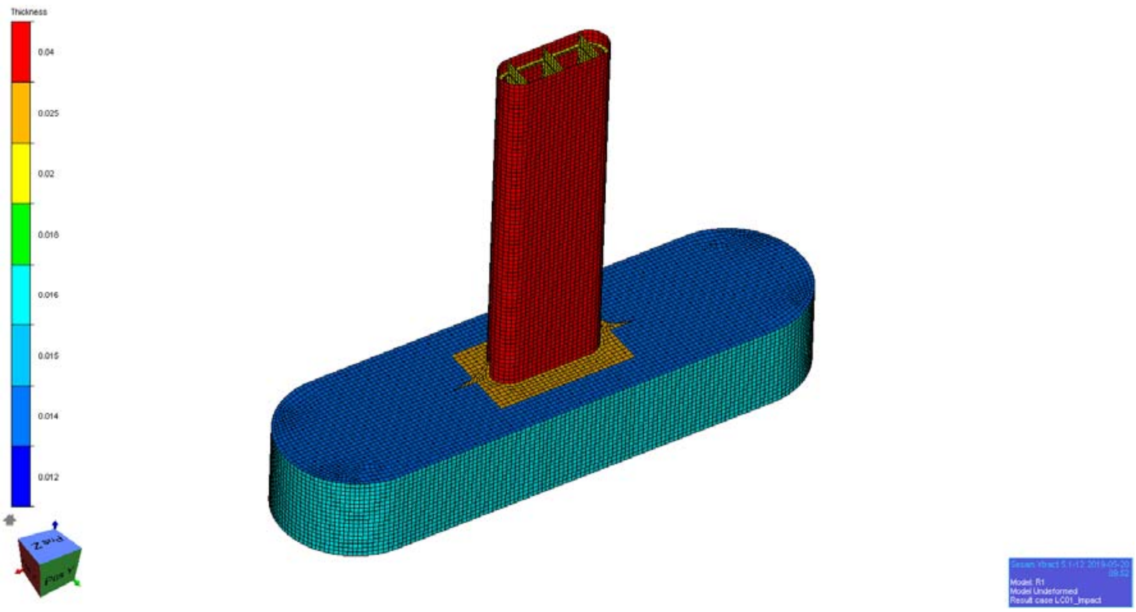
Linear and non-linear finite element models are developed. The linear finite element models are used for structural dimensioning and fatigue evaluations. Non-linear finite element models are mostly used for structural verification.

2.2.2 Sesam Sestra Models

Sesam Patran Pre is used for linear finite element modelling. Two finite element geometries are developed.

The first finite element model is for the pontoon and column at axis 3, which is closest to the navigation channel. The finite element model consists of a 58 meter long and 16 meter wide pontoon. The height of the pontoon is 9 meters with a draught of 5 meters.

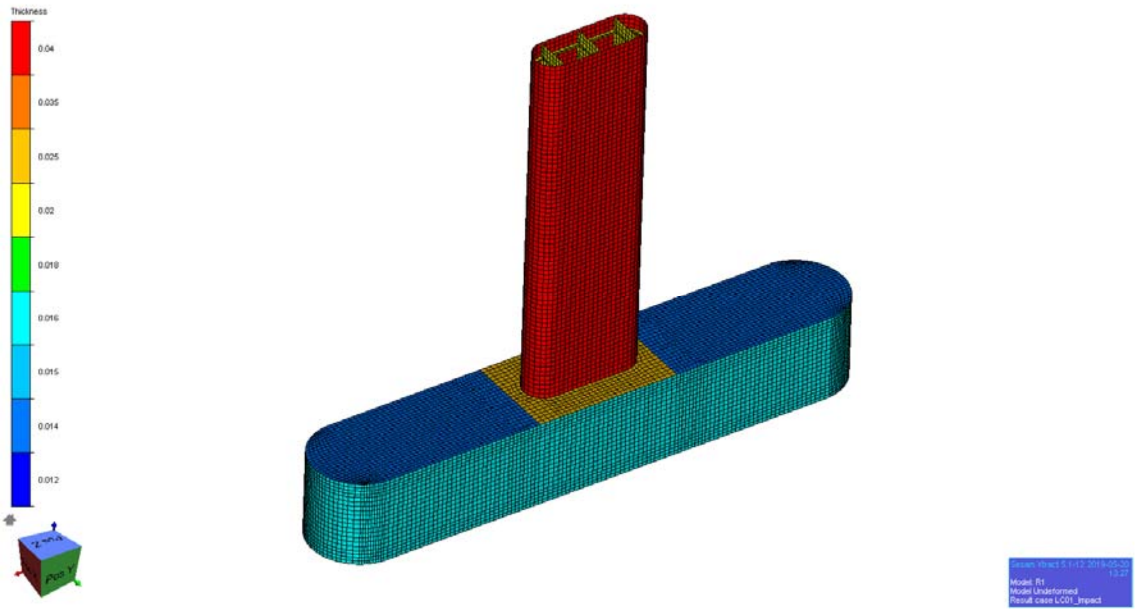
The column is rectangular in nature with half circular short edges. The rectangular part is 8 meter long and 4 meter wide. The radius of the short edge circles is 2 meters making the total length of the column equal to 12 meters. The height of the column from top of pontoon to underside of bridge girder is assumed 37.5 meters (39.3 m on latest drawing).



> *Figure 2-1 Large Sesam model (for axis 3) shown with shell thickness*

The second finite element model represents the large number of pontoons with short columns. The column shape and modelled column height is the same as for the first linear finite element model developed, but the column height used in a finite element run can be adjusted. The desired column height, for the moment 11 meters, can be chosen by keeping all nodes at this location completely fixed.

The pontoon is remodeled with a reduced breadth of 10 meters. The pontoon length is the same, 58 meters. The pontoon height of 10 meters and the pontoon draught of 5 meters are unaltered.

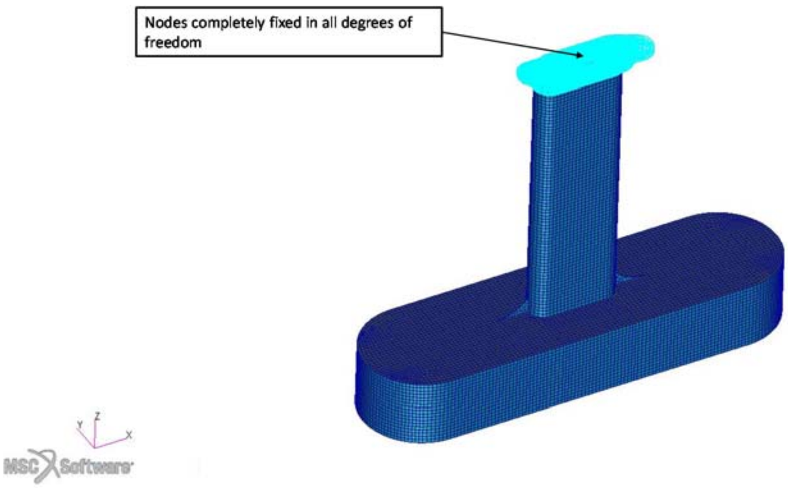


> *Figure 2-2 Sesam model for pontoons with short columns*

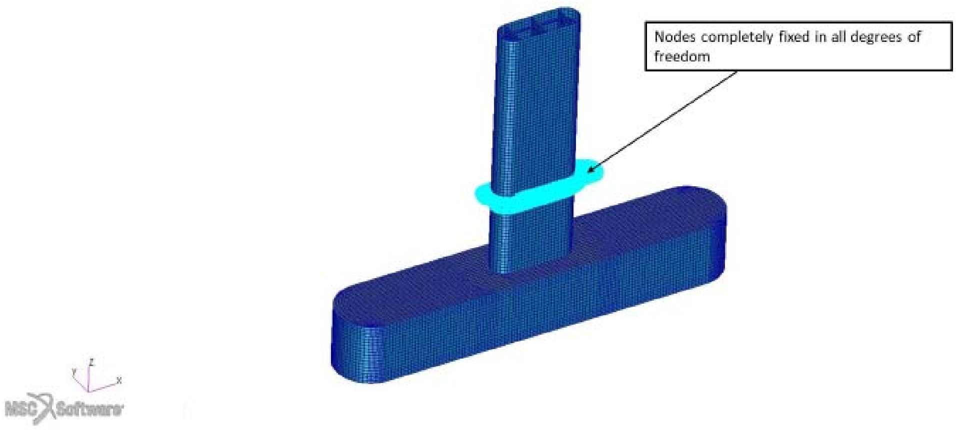
Due to adjusted bride girder span, currently about 120 meters, the breadth of both pontoons modelled will most probably be adjusted (i.e. increased). The latest drawings have a breadth of the pontoon at axis 3 equal to 17.0 meters and the breadth of the pontoon with short columns equal to 12 meters.

2.3 Boundary Conditions

The top of the columns is kept completely fixed. This is a simplification. The intersection area between the column and the bridge girder is studied in other reports.



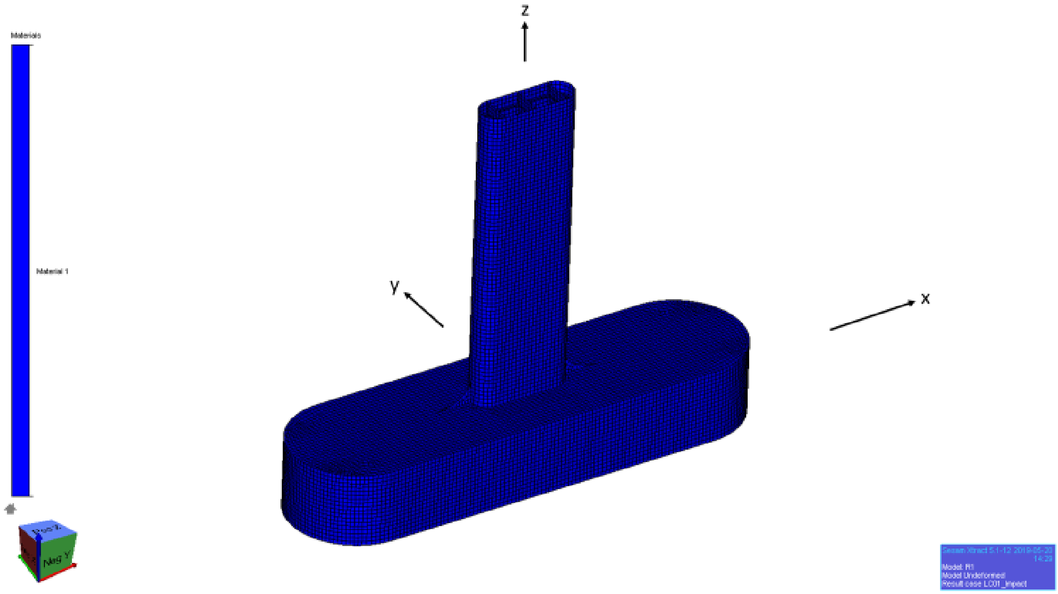
> *Figure 2-3 Boundary conditions shown for Sesam Sestra model with large pontoon and long column*



> *Figure 2-4 Boundary conditions shown for Sesam Sestra model with short column*

2.4 Coordinate System

A local co-ordinate system is used for the Sesam Sestra models. Local x-axis is positive forward of pontoon, y-axis is positive pontoon port and z-axis is positive upwards.



> *Figure 2-5 Local co-ordinate system for Sesam Sestra models.*

2.5 Material Definitions

For elastic analyses the elastic modulus is set equal to 210000 MPa. The density is 7850 kg/m^2. Poisson's number is 0.3. All values shall be in agreement with NS-EN 1993-1-1:2005, Chapter 3.

Yield stress is 420 MPa, which is the highest yield stress allowed (S420) according to Design Basis, Chapter 5.2.3.

3 NON-LINEAR ANALYSIS

3.1 Geometry

The geometry used for finite element modelling and structural analyses in general is based on current drawing status. The geometry for both pontoons and columns is under continuous development.

3.2 Finite Element Models

3.2.1 General

Non-linear finite element models are developed. The linear finite element models are used for structural dimensioning and fatigue evaluations. Non-linear finite element models are mostly used for structural verification and ALS.

3.2.2 Abaqus Models

Abaqus 2019 is used for non-linear finite element analyses.

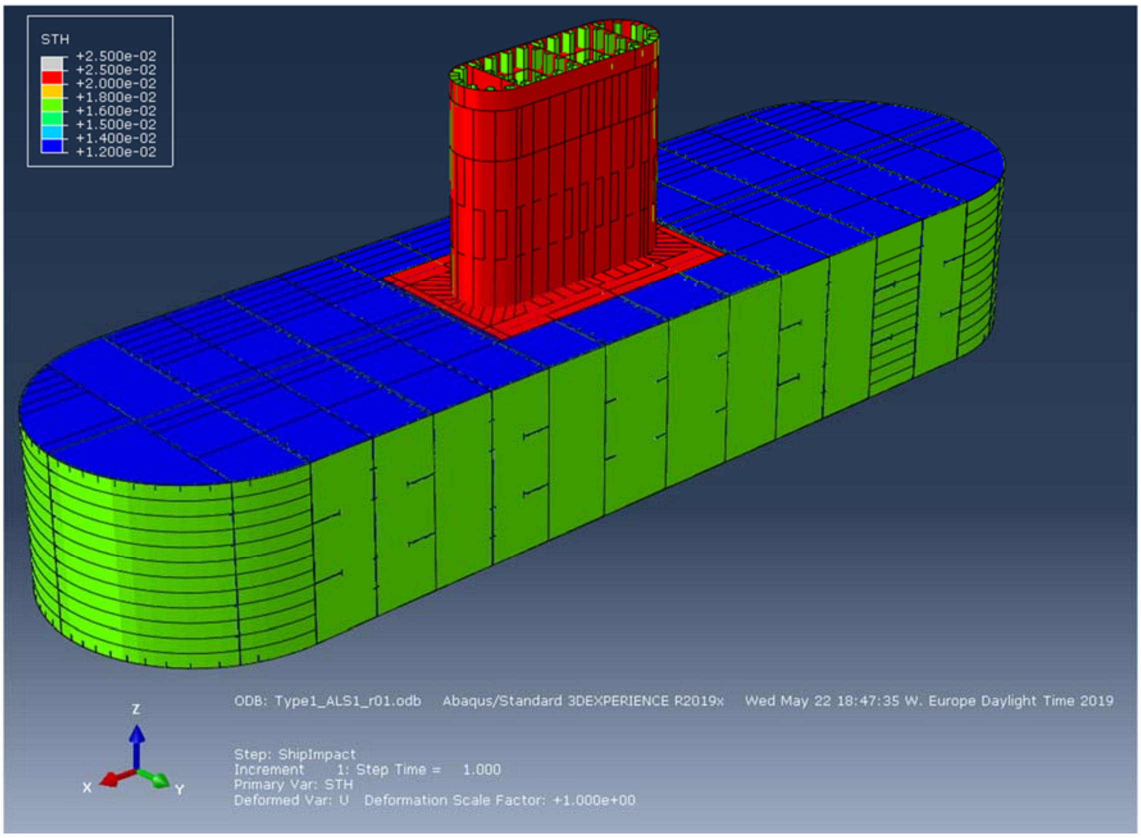
A finite element model for the pontoon at axis 3, which is closest to the navigation channel is used in the analyses. The finite element model consists of a 58 meter long and 16 meter wide pontoon. The height of the pontoon is 9 meters with a draught of 5 meters. Note that the pontoon width increased to 17 meters at the later stages of the project. The ULS analyses with different wave directions are updated to account for this, the ship impact ALS analyses are however not updated with the new pontoon width as it will have low impact on the results.

The columns are rectangular in nature with half circular short edges. The rectangular part is 8 meter long and 4 meter wide. The radius of the short edge circles is 2 meters making the total length of the column equal to 12 meters. The height of the column at axis 3 is 37.5 meters, while at the low bridge the height is 11 meters.

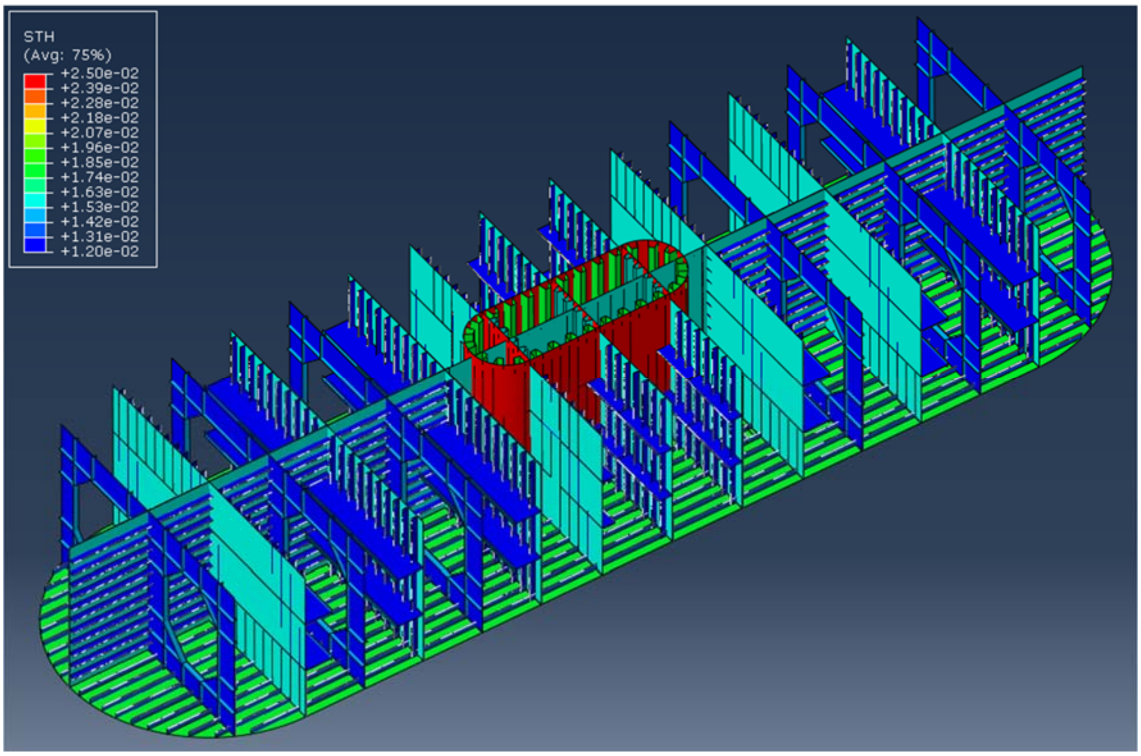
The pontoon and column are modelled as stiffened panels. In the pontoon, the outer walls, top- and bottom plate and internal bulkheads are stiffened with bulb-stiffeners in general, and additional T-girders for large panel spans (internal transverse bulkheads). In the column, the plates are stiffened with trapezoidal stiffeners, ring-frames and three internal bulkheads in the transverse direction. All plates and web of stiffeners are modelled as shells, while the flange of stiffeners and T-girders are modelled as "stringers" with proper flange characteristics (except for trapezoidal stiffeners, which is modelled in full).

The pontoon is modelled in several parts and tied at places sufficiently away from high stress areas. The interface between column and pontoon is modelled as single part to avoid tie-constraints in critical areas.

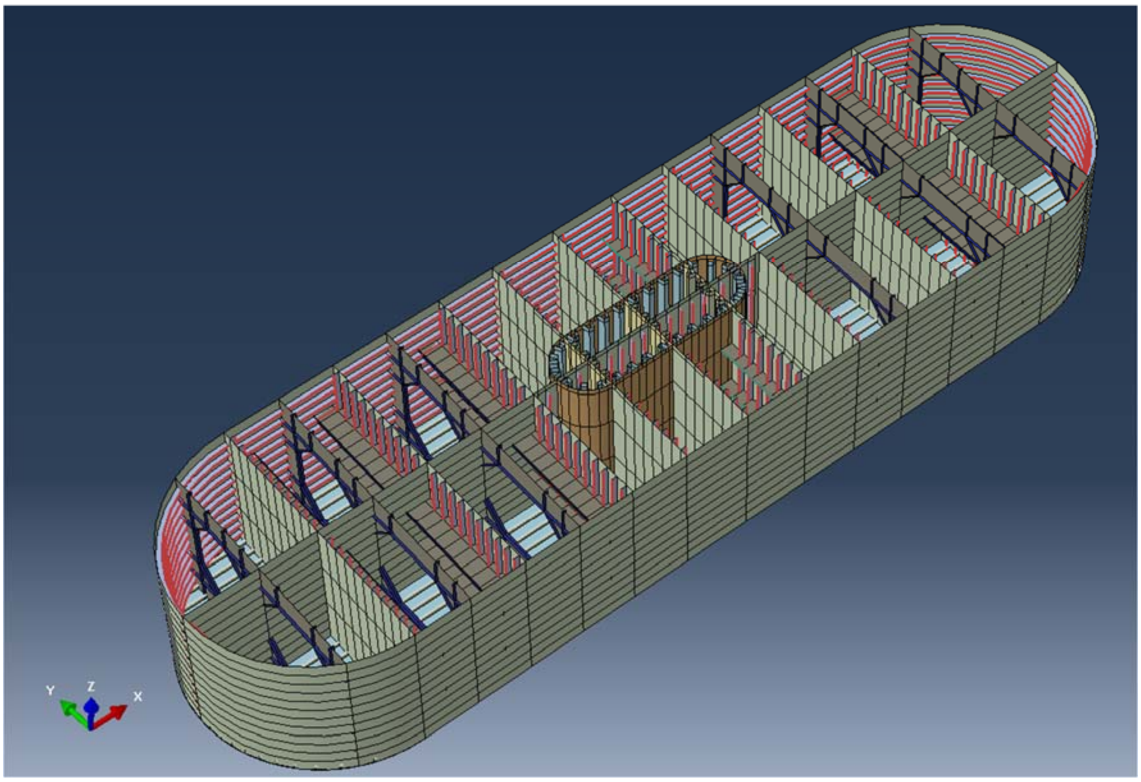
The pontoon and column are modelled with 8-node shell elements with reduced integration, with mesh size ranging from approx. 1.5m x 0.75 m (at pontoon outer surfaces) to TxT at column/pontoon interface (where T is the plate thickness).



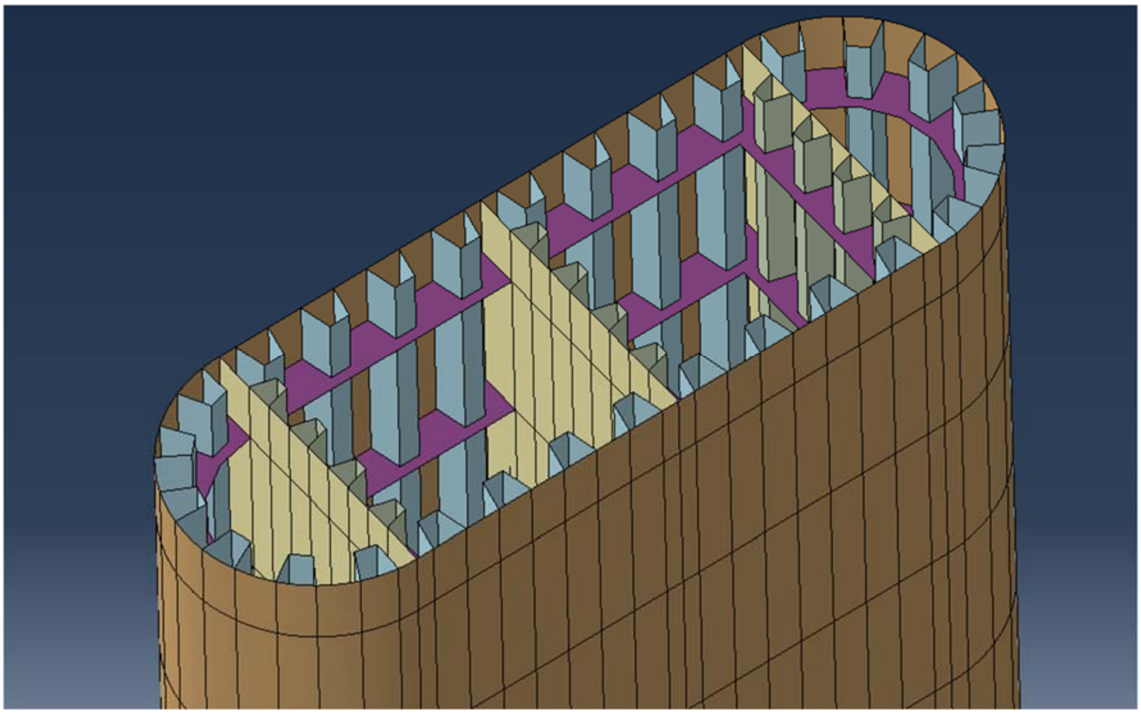
> *Figure 3-1 Thickness for pontoon and column*



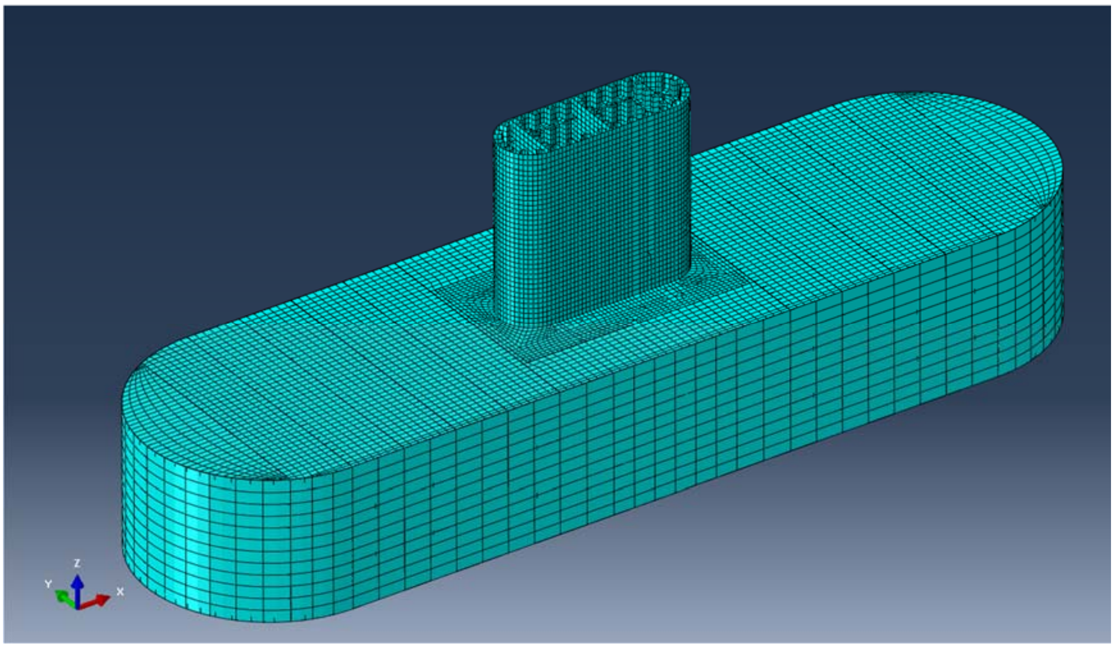
> *Figure 3-2 Thickness internal bulkheads and bottom plate*



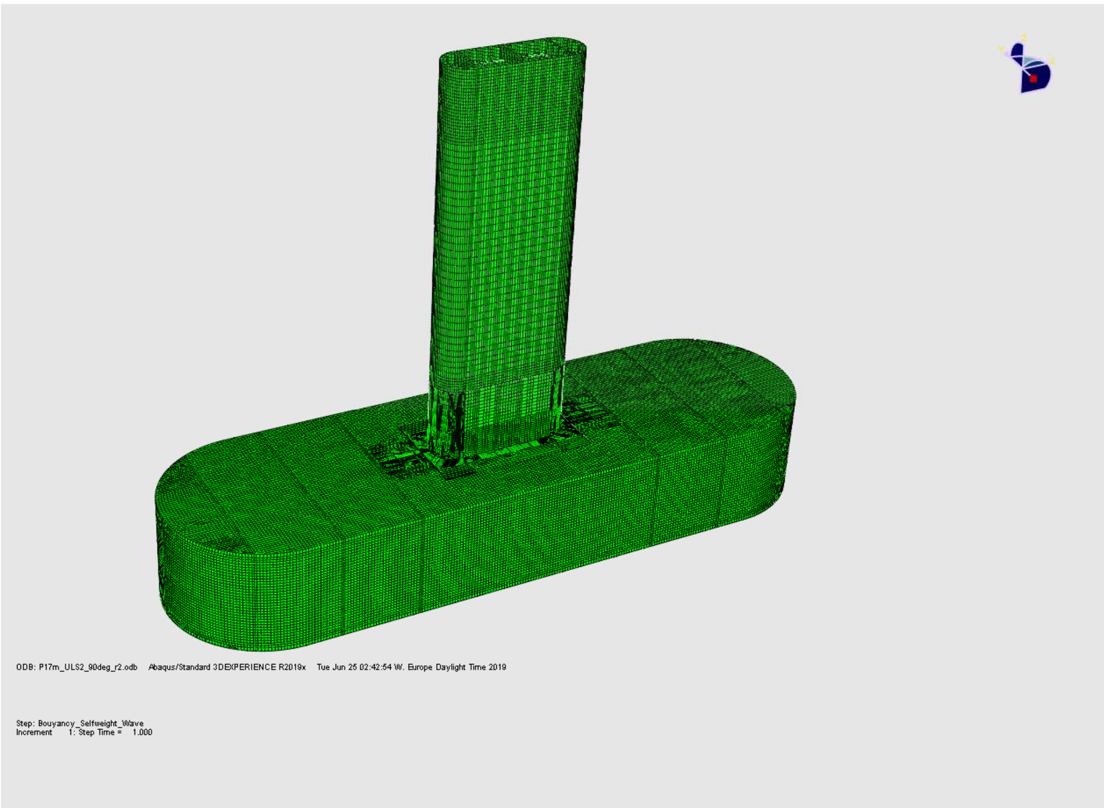
> *Figure 3-3 Internal compartments and bulkheads/frames*



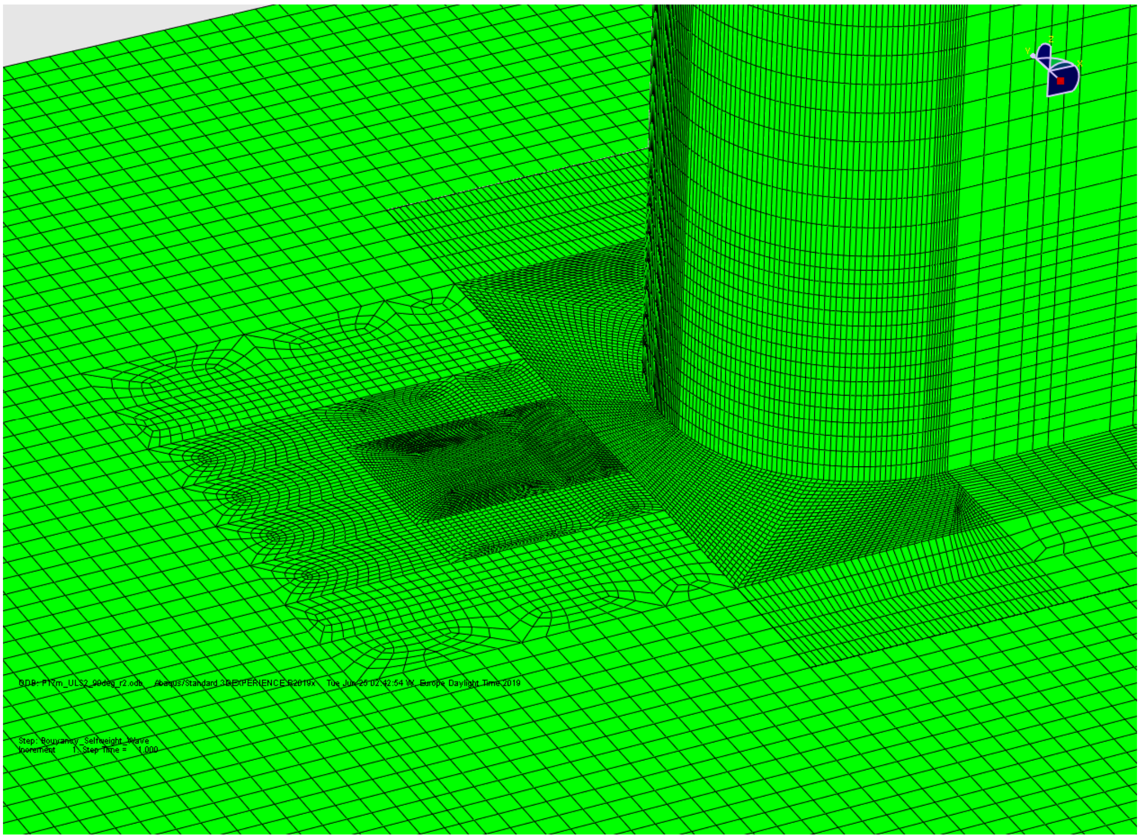
> *Figure 3-4 Column design, internal stiffeners and bulkheads*



> *Figure 3-5 Pontoon and column mesh for ship impact ALS analyses. Mesh is refined in interface between column and pontoon. The pontoon is modelled in several parts and tied at places sufficiently away from high stress areas. The interface between column and pontoon is modelled as single part to avoid tie-constraints in critical areas.*



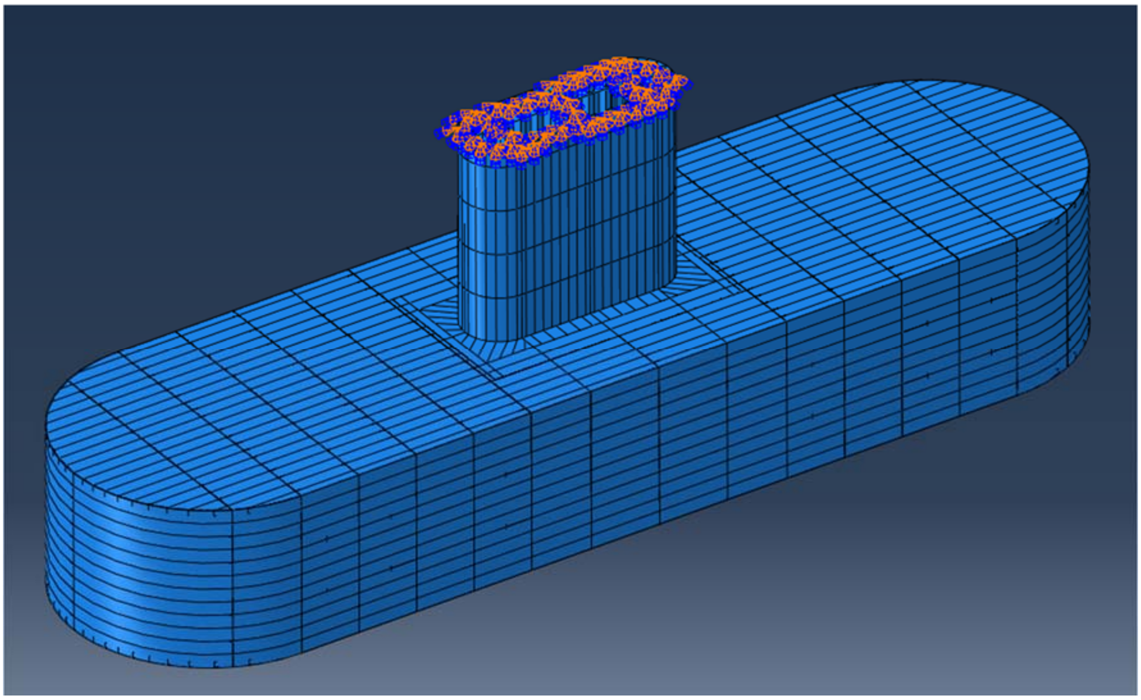
> *Figure 3-6 Pontoon and column mesh for ULS analyses. Mesh is refined in interface between column and pontoon. The pontoon, column and interface column-pontoon is modelled as separate parts and tied together with tie constraints. The interface between column and pontoon is modelled as single part to avoid tie-constraints in critical areas.*



> *Figure 3-7 Local refinement of mesh at interface between pontoon and column for FLS and ULS analyses. T x T mesh at peak stress areas for stress concentration evaluation.*

3.3 Boundary Conditions

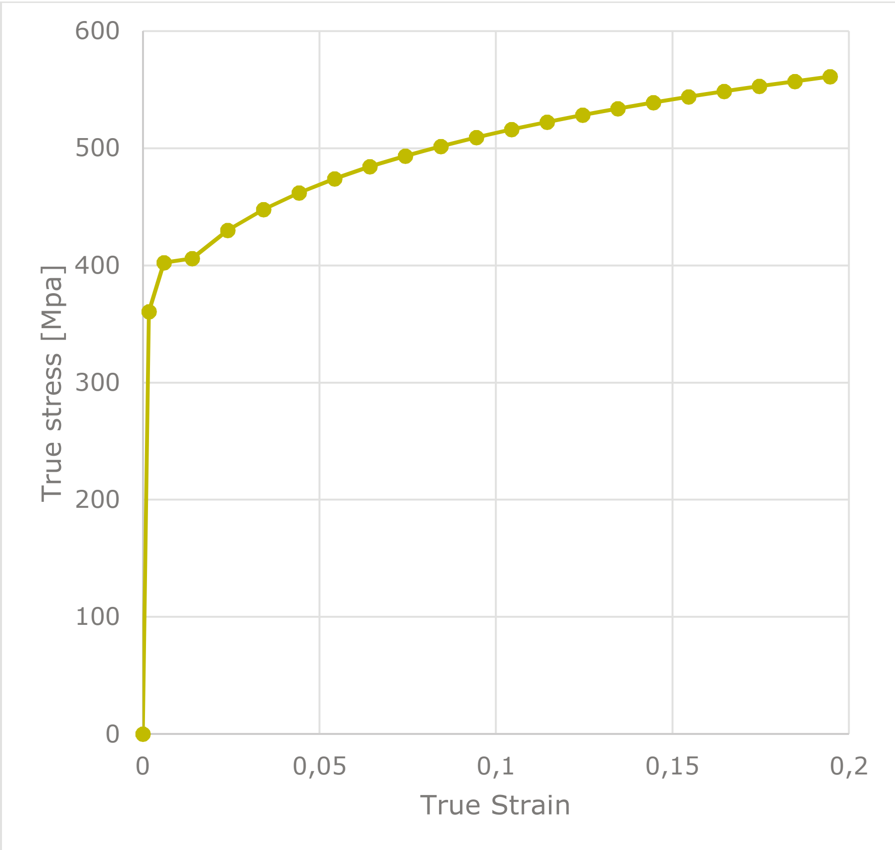
The top of the columns is kept completely fixed. This is a simplification. The intersection area between the column and the bridge girder is studied in other reports.



> *Figure 3-8 Boundary conditions shown for Abaqus model with short column*

3.4 Material Definitions

For non-linear analyses, the material curves for steel is obtained from [1]. The elastic modulus is set equal to 210000 MPa. The density is 7850 kg/m^2. Poisson's number is 0.3. All plates, T-girders and ring-frames are made with S420 steel, while the bulb-stiffeners are made with S355. Different material curves for different thickness ranges are implemented. A typical material curve is shown below.



> *Figure 3-9 Material curve for S420, 16 mm < t ≤ 40 mm*

4 LOADING

4.1 General

The pontoon and columns will be designed for ULS, FLS and ALS. Most parts of the column and some parts of the pontoon have ship impact as the governing design load. For the pontoon ship impact will be important in the interaction area between the pontoon and the lower part of the column.

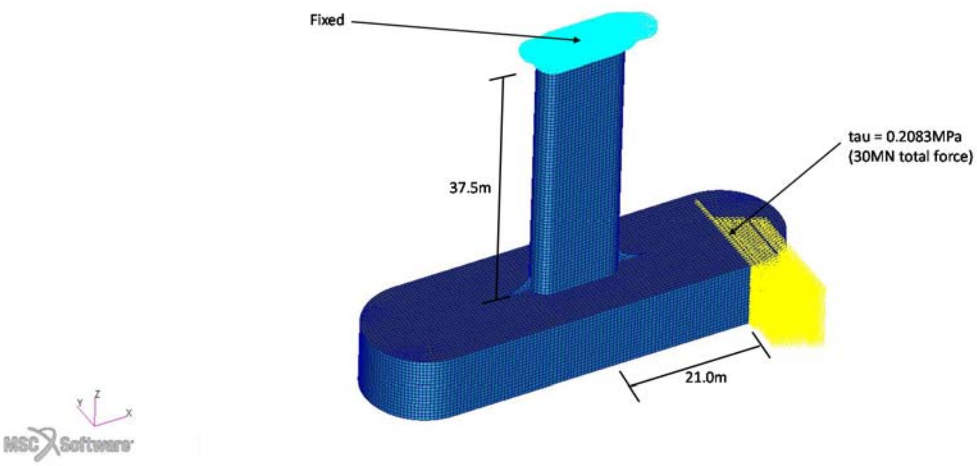
Dynamic loading is put on as quasi-static loading without taking dynamic amplification (including effects from added mass) into account. As a first approach, column section forces occurring in the analyses are compared with column sectional forces from the global model.

4.2 ALS

4.2.1 Ship Impact

Ship impact is handled by applying a ship impact force equal to 30MN on the pontoon. The ship impact force can have different directions and locations. One of the critical ship impact forces will be ship impact close to the pontoon end perpendicular to the pontoon longitudinal direction. Such a ship impact scenario will induce a large torsional moment in the column, a large bending moment at the top of the column and a constant transverse shear force in the column.

The ship impact force is applied by giving the outermost pontoon wall with full breadth, located 21 meters in front of pontoon center, a shear stress equal to 0.2083MPa. With an area of 144m^2 the total transverse ship impact force becomes 30MN. For the smaller pontoon the shear stress applied to the wall located 21 meters in front of pontoon center is 0.3333MPa.



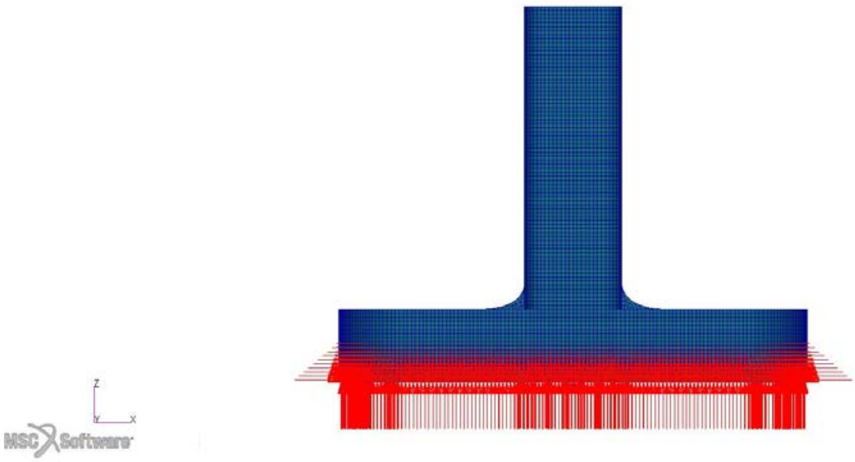
> *Figure 4-1 Application of ship impact force for finite element model with large pontoon and highest column (axis 3).*

4.2.2 Static Water Pressure

The static water pressure is calculated based on the pontoon draught, equal to -5.0m.

$$p_{static} = -\rho g z = -1025\frac{kg}{m^3}\cdot 9.81\frac{m}{s^2}\cdot(-5m) = 50276\frac{N}{m^2}$$

The load factor for static ALS water pressure is 1.0. The static water pressure is applied with linear variation starting from zero at waterline. The static ALS water pressure at the pontoon bottom becomes 50276N/m^2.



> *Figure 4-2 Application of static ALS water pressure for finite element model with large pontoon and highest column (axis 3)*

4.2.3 Gravity

Gravity is introduced with a load factor of 1.0 for ALS.

$$g_{ALS} = 1.0g = 1.0\cdot 9.81\frac{m}{s^2} = 9.81\frac{m}{s^2}$$

4.2.4 Tank Pressure

The tank pressure will be equal to the filling height of the tanks. Preliminary tank pressure height is set equal to the top of the pontoons.

4.2.5 Compartment Flooding

Compartment flooding, created by ship impact or other causes, will affect the buoyancy of the pontoon. The draught will be altered to compensate for the loss of buoyancy.

4.3 ULS

4.3.1 General

The ULS loading consists of a permanent static water pressure, gravity and dynamic loading. The dynamic loading is the effect of waves, wind, current, traffic loads and structural response from the entire bridge structure.

As a first approximation a quasi-static approach is used. Permanent static water pressure is combined with dynamic wave pressure caused by a 100-year ULS design wave.

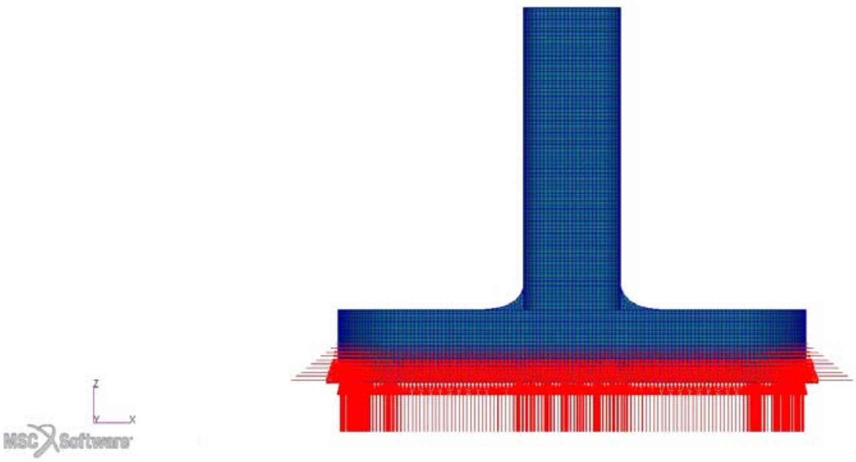
4.3.2 Static Water Pressure

The static water pressure is calculated based on the pontoon draught, equal to -5.0m.

$$p_{static} = -\rho g z = -1025\frac{kg}{m^3}\cdot 9.81\frac{m}{s^2}\cdot(-5m) = 50276\frac{N}{m^2}$$

The load factor for static ULS water pressure is set to 1.2. The static water pressure is applied with linear variation starting from zero at waterline. The static ULS water pressure at the pontoon bottom becomes:

$$p_{static,ULS} = 1.2p_{static} = 1.2\cdot 50276\frac{N}{m^2} = 60331.5\frac{N}{m^2}$$



> *Figure 4-3 Application of static ULS water pressure for finite element model with large pontoon and highest column (axis 3)*

4.3.3 Dynamic Loading

Elementary Airy wave theory is used for calculating the hydrodynamic water pressure. A sinusoidal 100-year design wave is found based on MetOcean Design Basis, Table 1.

H_s = 2.1 m
T_p = 5.5 s

$$H_{max} = 2.12\cdot H_s = 2.12 * 2.1m = 4.452m$$

$$T_{max} = 1.1 \cdot T_p = 1.1 \cdot 5.5s = 6.05s$$

$$\lambda = \frac{gT_{max}^2}{2\pi} = \frac{9.81\frac{m}{s^2} \cdot (6.05s)^2}{2\pi} = 57.15m$$

$$k = \frac{2\pi}{\lambda} = \frac{2\pi}{57.15m} = 0.110m^{-1}$$

The dynamic water pressure at depth z is given by the following formula:

$$p = \frac{1}{2}\rho g H_{max} e^{-kz} cos\left(kx - \frac{2\pi}{T_{max}} \cdot t\right)$$

$$p = 22382.9865\frac{N}{m^2}e^{-0.109946138m^{-1}z}\cos(0.109946138m^{-1}x - 1.038543026s^{-1}t)$$

$$p = 22383\frac{N}{m^2}e^{-0.10995m^{-1}z}\cos(0.10995m^{-1}x - 1.0385s^{-1}t)$$

The load factor for dynamic ULS water pressure is set to 1.6. The dynamic ULS water pressure becomes:

$$p = 35813\frac{N}{m^2}e^{-0.10995m^{-1}z}\cos(0.10995m^{-1}x - 1.0385s^{-1}t)$$

The maximum dynamic water pressure is 35813 N/m^2 at surface elevation and 20558 N/m^2 at 5 meters dept (pontoon bottom). At pontoon bottom the static ULS water pressure is approximately 3 times the maximum dynamic water pressure.

The dynamic 100-year design wave will have a wave length that is approximately equal to the pontoon length. The total dynamic vertical force will be approximately equal to zero if it is integrated over the entire pontoon length.

Maximum quasi-static pontoon moment will occur with wave crest or wave bottom at pontoon middle point. With wave bottom at pontoon middle point the equation for the dynamic water pressure becomes:

$$p = -35813\frac{N}{m^2}e^{-0.10995m^{-1}z}\cos(0.10995m^{-1}x)$$

The waves may also be inclined. Equivalent expressions for 30 degrees, 45 degrees, 60 degrees and 90 degrees inclination become:

$$p_{30deg} = -35813\frac{N}{m^2}e^{-0.10995m^{-1}z}\cos\big(0.10995m^{-1}(0.866025404x + 0.5y)\big)$$

$$p_{45deg} = -35813\frac{N}{m^2}e^{-0.10995m^{-1}z}\cos\big(0.10995m^{-1}(0.707106781x + 0.707106781y)\big)$$

$$p_{60deg} = -35813\frac{N}{m^2}e^{-0.10995m^{-1}z}\cos\big(0.10995m^{-1}(0.5x + 0.866025404y)\big)$$

$$p_{90deg} = -35813\frac{N}{m^2}e^{-0.10995m^{-1}z}\cos(0.10995m^{-1}y)$$

Maximum quasi-static column moment will occur with wave crest or wave bottom at ¼ of the pontoon length and the corresponding opposite wave bottom or wave crest at ¾ of the pontoon length. With wave bottom at ¼ of the pontoon length and wave crest at ¾ of the pontoon length the dynamic water pressure can be written:

$$p = 35813\frac{N}{m^2}e^{-0.10995m^{-1}z}\sin(0.10995m^{-1}x)$$

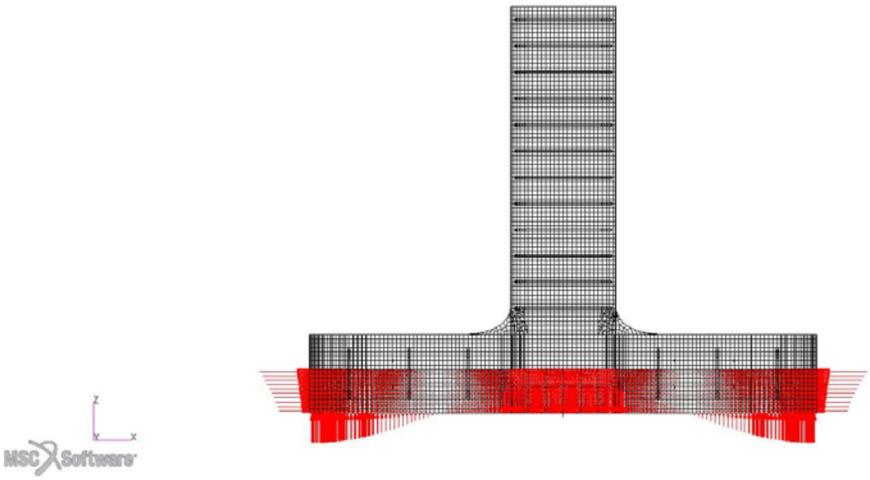
Equivalent expressions for 30 degrees, 45 degrees, 60 degrees and 90 degrees inclination become:

$$p_{30deg} = 35813\frac{N}{m^2}e^{-0.10995m^{-1}z}\sin\left(0.10995m^{-1}(0.866025404x + 0.5y)\right)$$

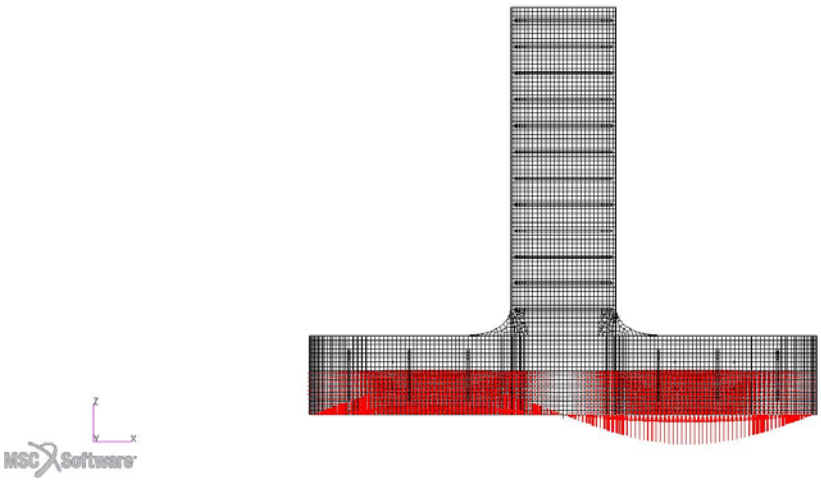
$$p_{45deg} = 35813\frac{N}{m^2}e^{-0.10995m^{-1}z}\sin\left(0.10995m^{-1}(0.707106781x + 0.707106781y)\right)$$

$$p_{60deg} = 35813\frac{N}{m^2}e^{-0.10995m^{-1}z}\sin\left(0.10995m^{-1}(0.5x + 0.866025404y)\right)$$

$$p_{90deg} = 35813\frac{N}{m^2}e^{-0.10995m^{-1}z}\sin(0.10995m^{-1}y)$$



> *Figure 4-4 ULS dynamic water pressure application with wave bottom at pontoon middle point*



> *Figure 4-5 ULS dynamic water pressure application with wave bottom at ¼ of the pontoon length and wave crest at ¾ of the pontoon length*

4.3.4 Gravity

Gravity is introduced with a load factor of 1.2 for ULS.

$$g_{ULS} = 1.2g = 1.2 \cdot 9.81\frac{m}{s^2} = 11.772\frac{m}{s^2}$$

4.3.5 Tank Pressure

The tank pressure will be equal to the filling height of the tanks. Preliminary tank pressure height is set equal to the top of the pontoons.

4.4 FLS

4.4.1 General

The fatigue evaluation consists of an evaluation of first principal stress ranges for the dynamic part of the loading.

There are several levels possible for fatigue evaluation. Preliminary global model fatigue evaluations show low fatigue damage in the column-pontoon intersection area. Highest ULS-stresses are observed in this area and fatigue damage is believed to be highest in this area.

Further fatigue damage evaluations will be carried out in a later phase.

5 RESULTS LINEAR ANALYSIS

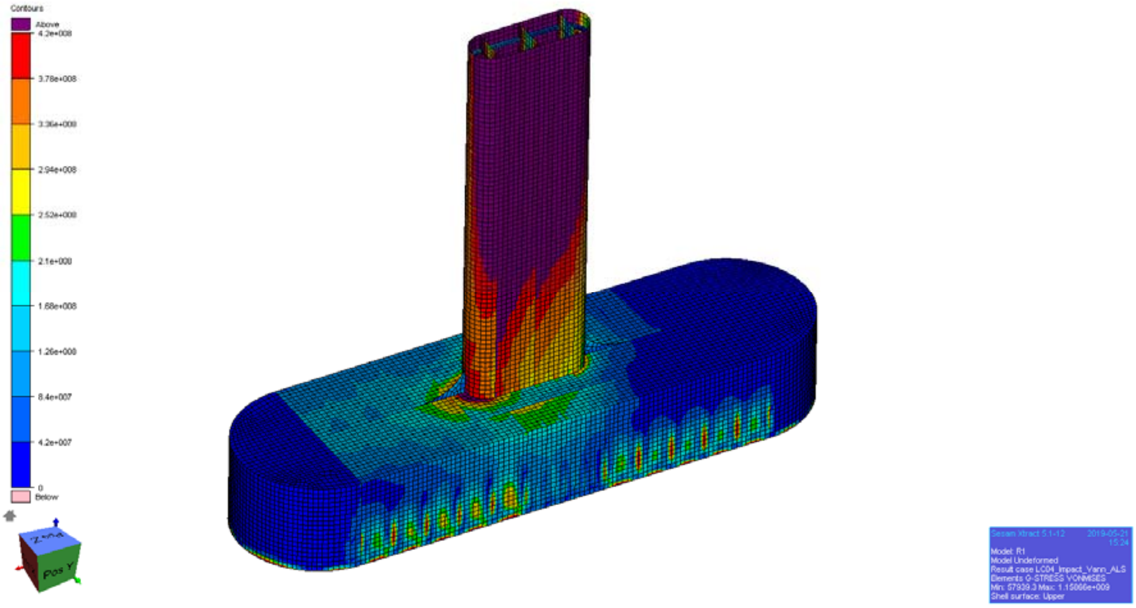
5.1 General

Stiffeners are not included in the FE-models used for linear analyses in Sestra.

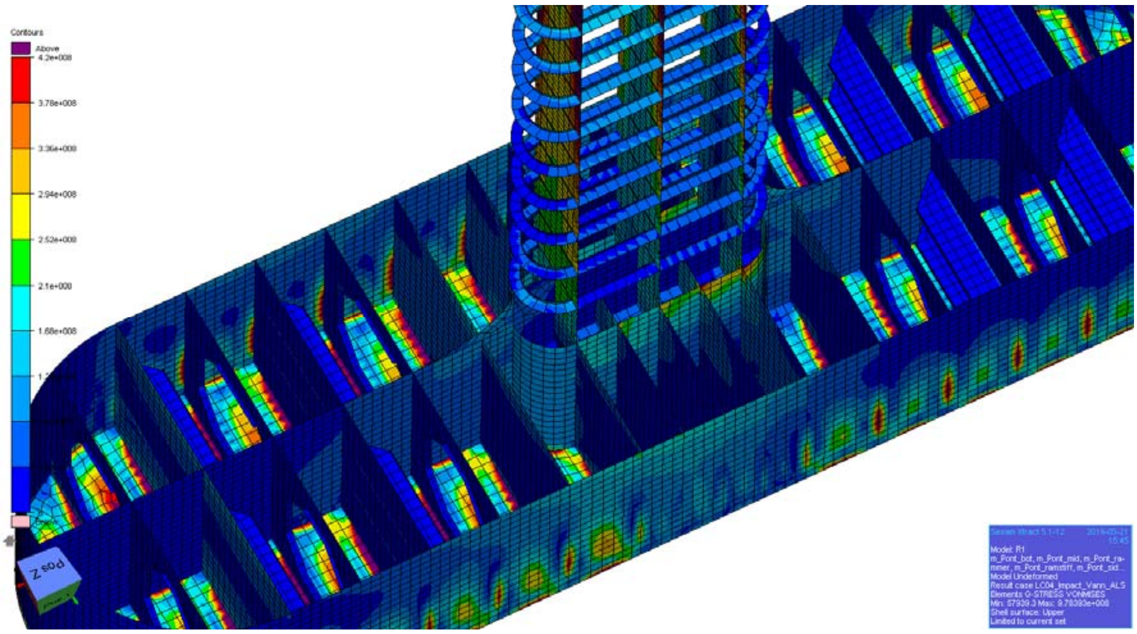
5.2 Large Pontoon with High Column

5.2.1 ALS Results

The ship impact load is combined with static water pressure and gravity.



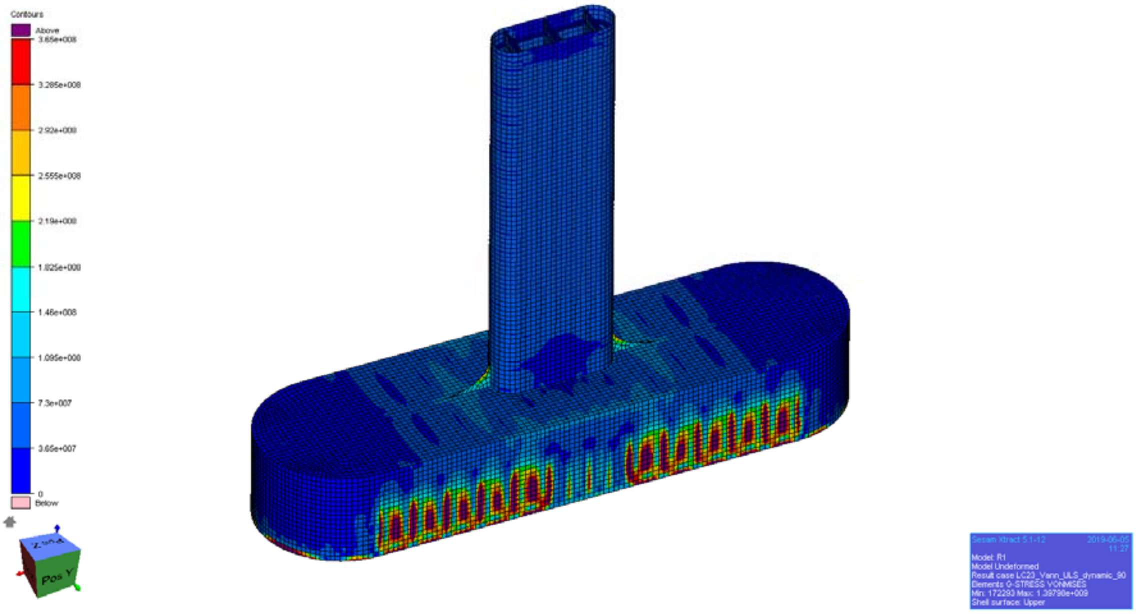
> *Figure 5-1 von Mises stress*



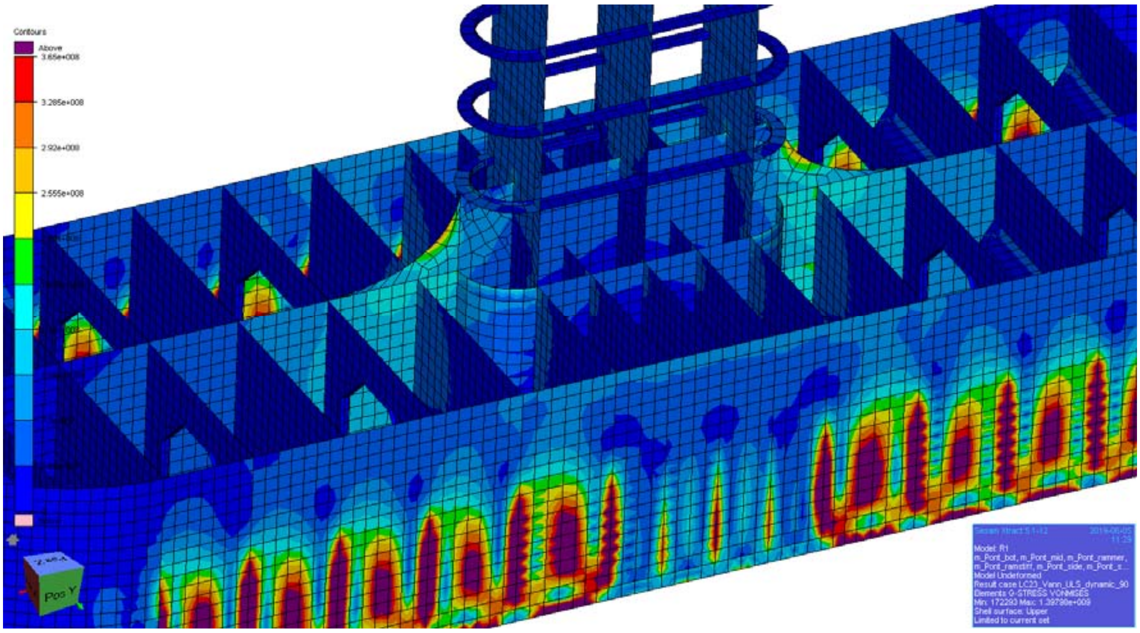
> *Figure 5-2 von Mises stress, column shell and pontoon top removed*

5.2.2 ULS Results, maximum Pontoon Moment

The load combination consists of static water pressure, dynamic water pressure and gravity.



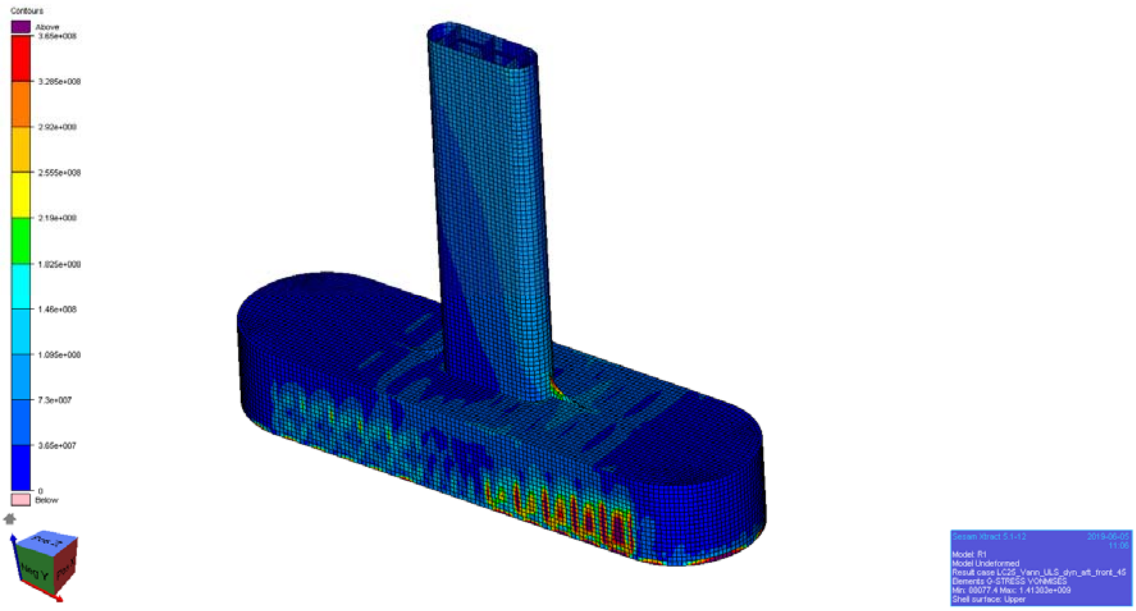
> *Figure 5-3 von Mises stress*



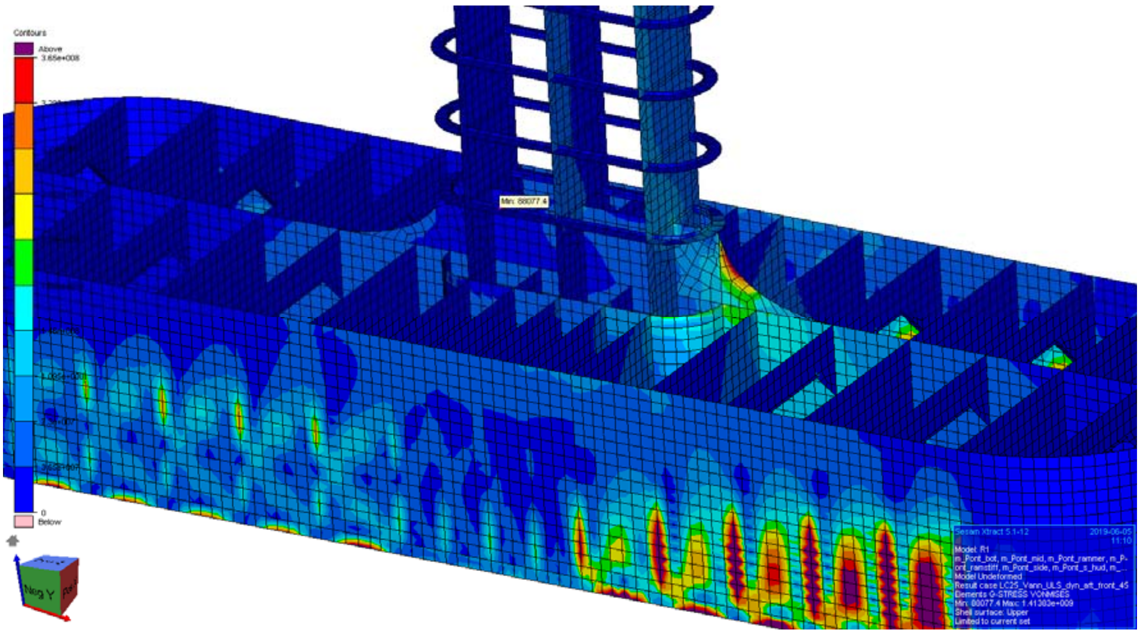
> *Figure 5-4 von Mises stress, column shell and pontoon top removed*

5.2.3 ULS Results, maximum Column Moment

The load combination consists of static water pressure, dynamic water pressure and gravity.



> *Figure 5-5 von Mises stress*

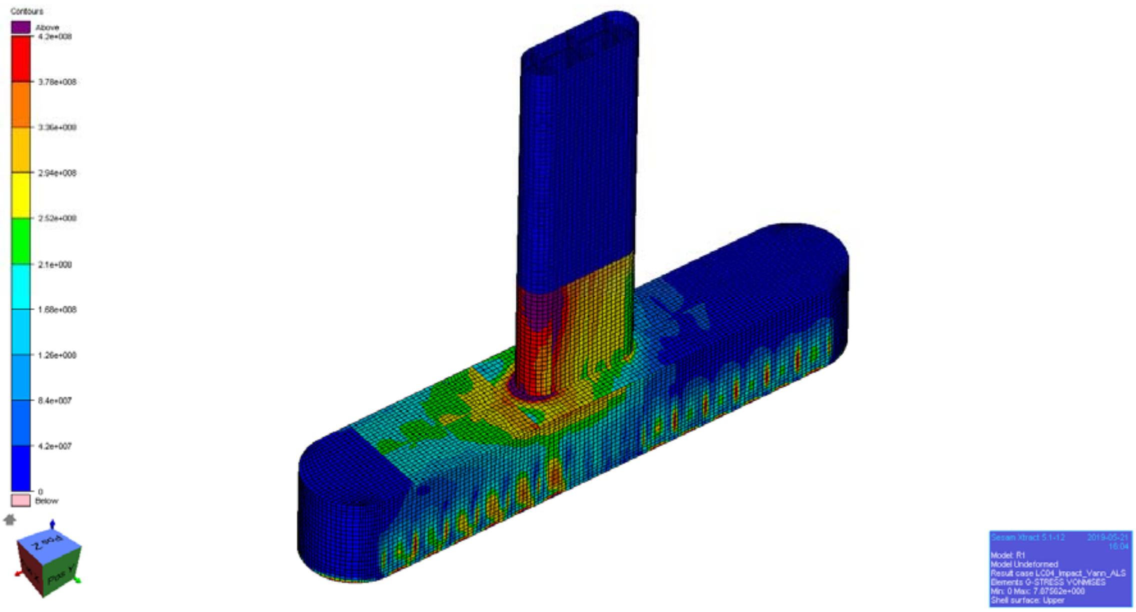


> *Figure 5-6 von Mises stress, column shell and pontoon top removed*

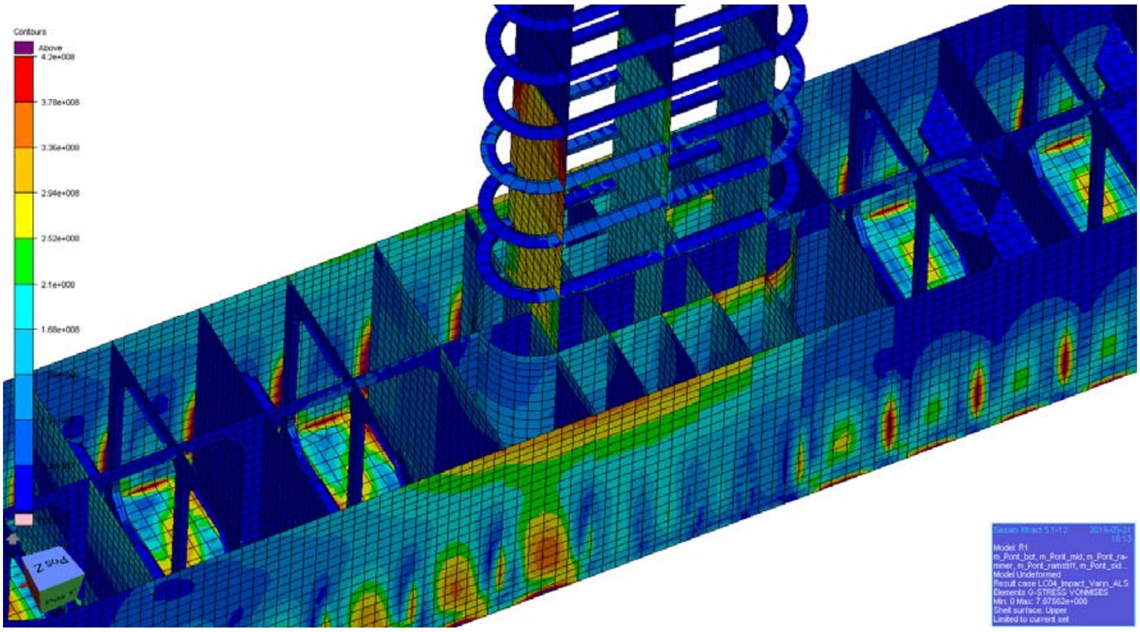
5.3 Small Pontoon with Low Column

5.3.1 ALS Results

The ship impact load is combined with static water pressure and gravity.



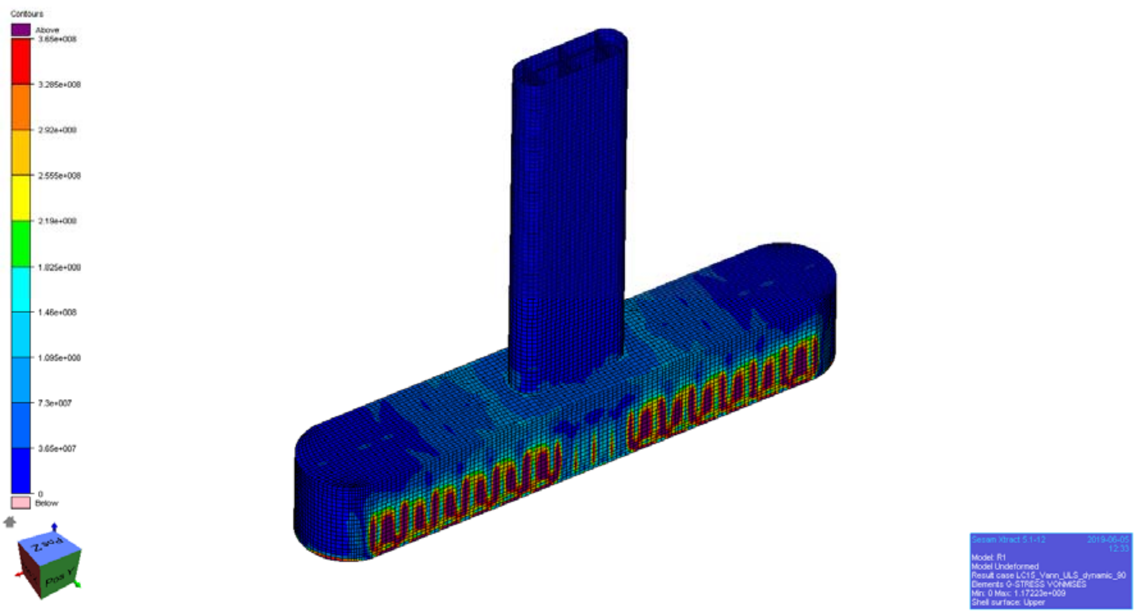
> *Figure 5-7 von Mises stress*



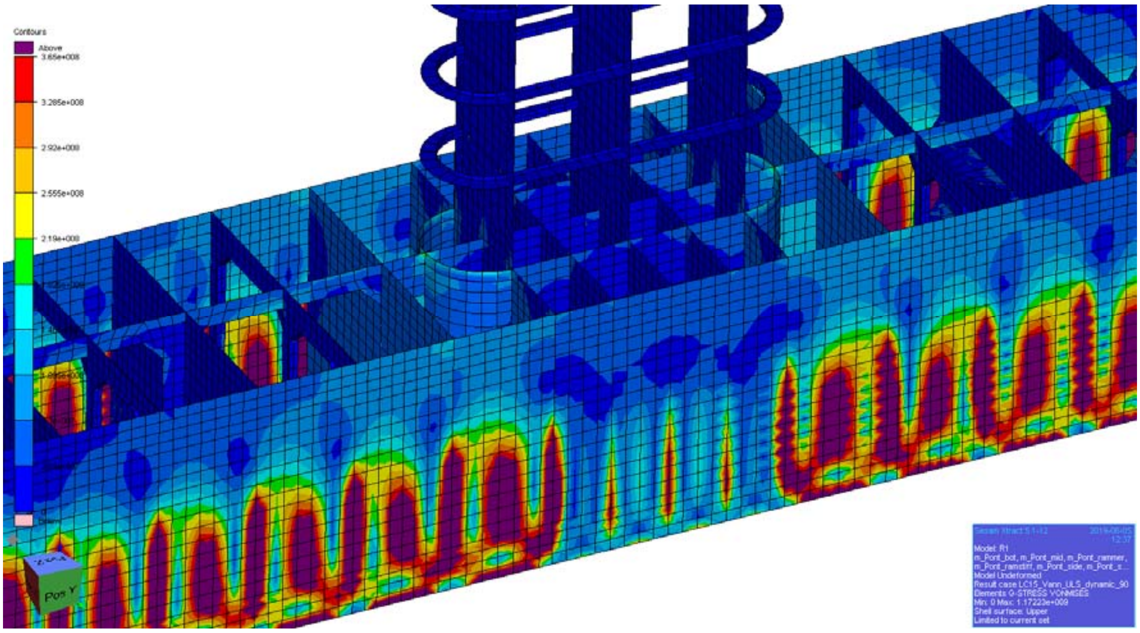
> *Figure 5-8 von Mises stress, column shell and pontoon top removed*

5.3.2 ULS Results, maximum Pontoon Moment

The load combination consists of static water pressure, dynamic water pressure and gravity.



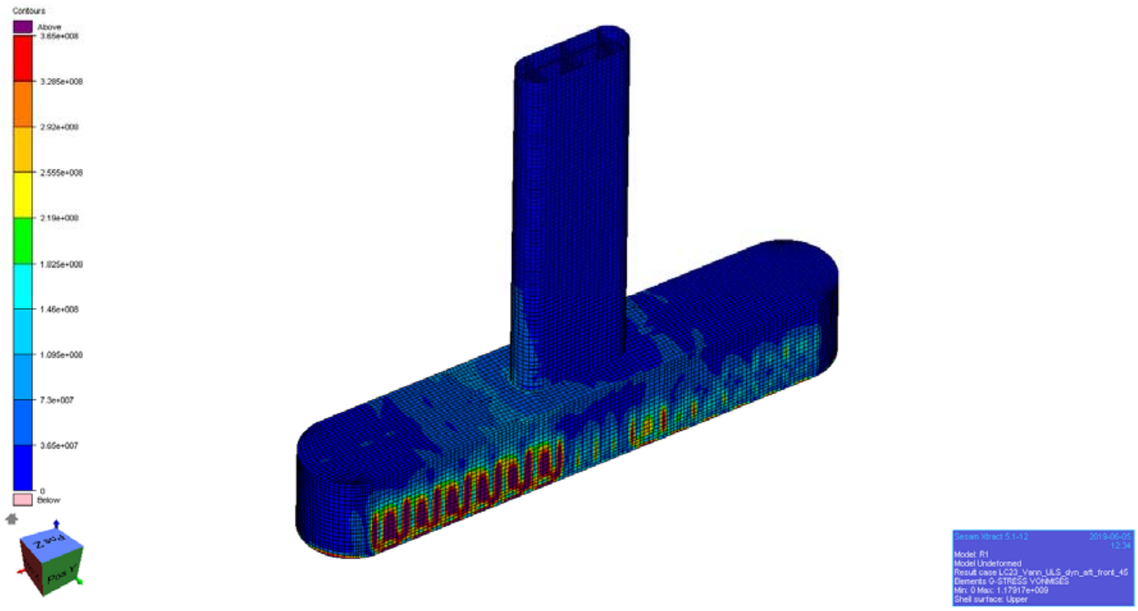
> *Figure 5-9 von Mises stress*



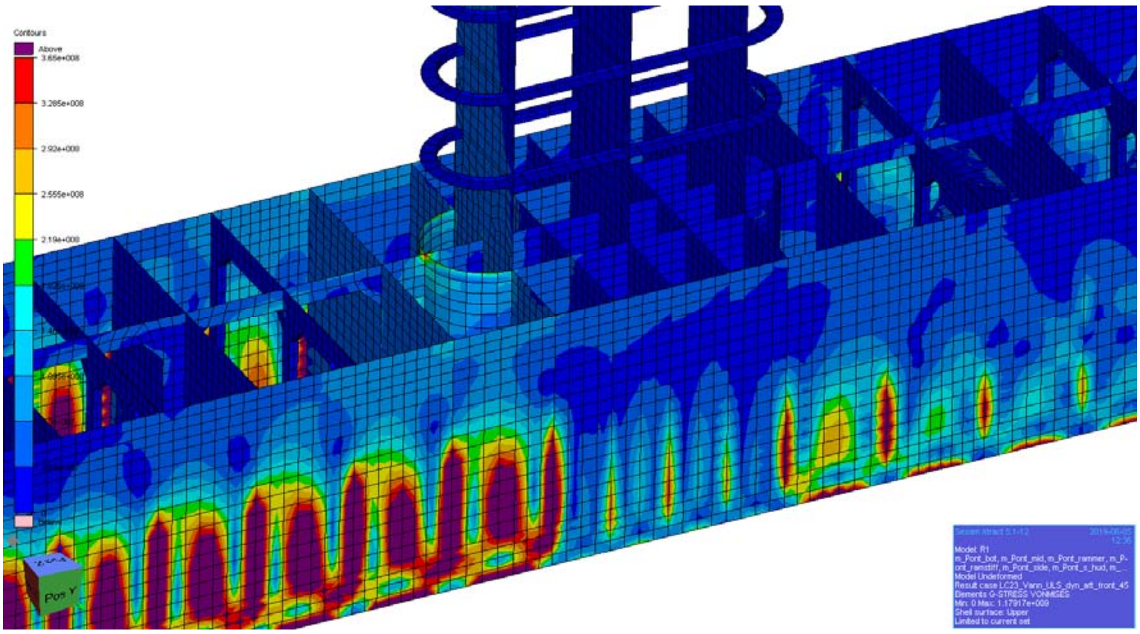
> *Figure 5-10 von Mises stress, column shell and pontoon top removed*

5.3.3 ULS Results, maximum Column Moment

The load combination consists of static water pressure, dynamic water pressure and gravity.



> *Figure 5-11 von Mises stress*



> *Figure 5-12 von Mises stress, column shell and pontoon top removed*

5.4 Discussion

Linear analyses are used for dimensioning. The linear analyses indicate that the connection between the column and the pontoon will withstand the loading for both the ULS condition and the ALS condition.

For the long column the stresses at the top of the column become too high. Plastic collapse will probably take place before impact load becomes equal to the design value of 30 MN.

The short column will also have very high stresses, with highest vales in the top region. Plastic utilization will be found in the non-linear analyses.

For the model with large pontoon and long column two knee-plates are introduced where the pontoon longitudinal bulkhead crosses the column skin. For the ALS and ULS condition non-linear analyses will most probably show that these knee-plates are not necessary. However, they will increase the fatigue life in this region. Later FLS evaluations will show if they are necessary.

6 RESULTS NON-LINEAR ANALYSIS

6.1 General

Same pontoon type (width 16 meters, length 58 meters) is used for both tall and short column non-linear analyses. As the major stresses from ship impact (except local stresses at impact zone) are in the interfacing part between the column and pontoon due to torsion, the width of the pontoon is of less importance and should not affect the results in any major way. It is considered acceptable to lose up to four compartments in flooding from ship impact, so the focus of these analyses is to verify the integrity of the overall strength of the pontoon-column interface and the column itself. Hence the ship impact load is treated as a uniform shear stress at the outermost bulkhead (situated 21 meters from the center of the column), equal to 0.2083 MPa which totals to 30 MN. The forces and moments in the column will thus be a shear force of 30 MN, together with a torsional moment of 630 MNm and weak axis bending moment with maximum value of 465 MNm for the short column and 1260 MNm for the tall column (in ship impact report, SBJ-33-C5-OON-22-RE-013, more accurate forces and moments are available).

Static water pressure and structural self-weight is applied with safety factor 1.0 for the ALS analyses. The FE analyses are checked without internal water pressure from ballast tanks.

ULS analyses for all wave directions are performed in Abaqus for largest pontoon (17 meter width) with refined mesh and bulkhead and plate stiffeners added. The analyses are run with linear-elastic material properties, but with non-linear geometry properties (i.e. 2nd order theory) to capture any local buckling behavior.

6.2 Large Pontoon with tall column

6.2.1 ULS Results

The two waves described in 4.3.3 are checked for ULS, at angles 0, 15, 30, 45, 60, 75 and 90 degrees to the pontoon longitudinal axis.

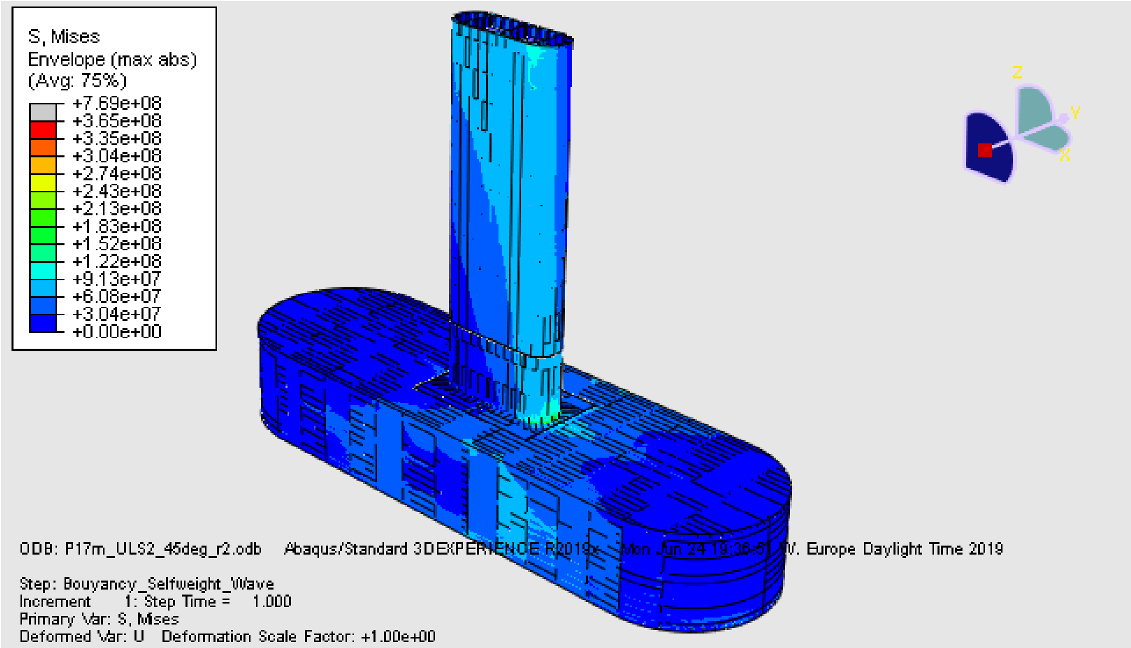
The ULS dynamic water pressure applied with wave bottom at ¼ of the pontoon length and wave crest at ¾ of the pontoon length, at an angle of 45 degrees to the pontoon longitudinal axis is shown to give the highest utilization.

All stresses are below the Mises yield criteria and no permanent deformation occur in ULS except for one local peak between pontoon longitudinal bulkhead and column intersection at top of pontoon. This peak stress will be reduced with added knee-plate mentioned in linear elastic analyses chapter. Elsewhere in the pontoon the stresses are far below yield limit. The knee-plate may also be necessary from fatigue life calculations.

In addition, the transversal bulkheads are checked with internal water pressure in a separate Stipla check with conservative stresses from the analyses (i.e. peak plate end stresses assumed uniformly over the plate length/height).

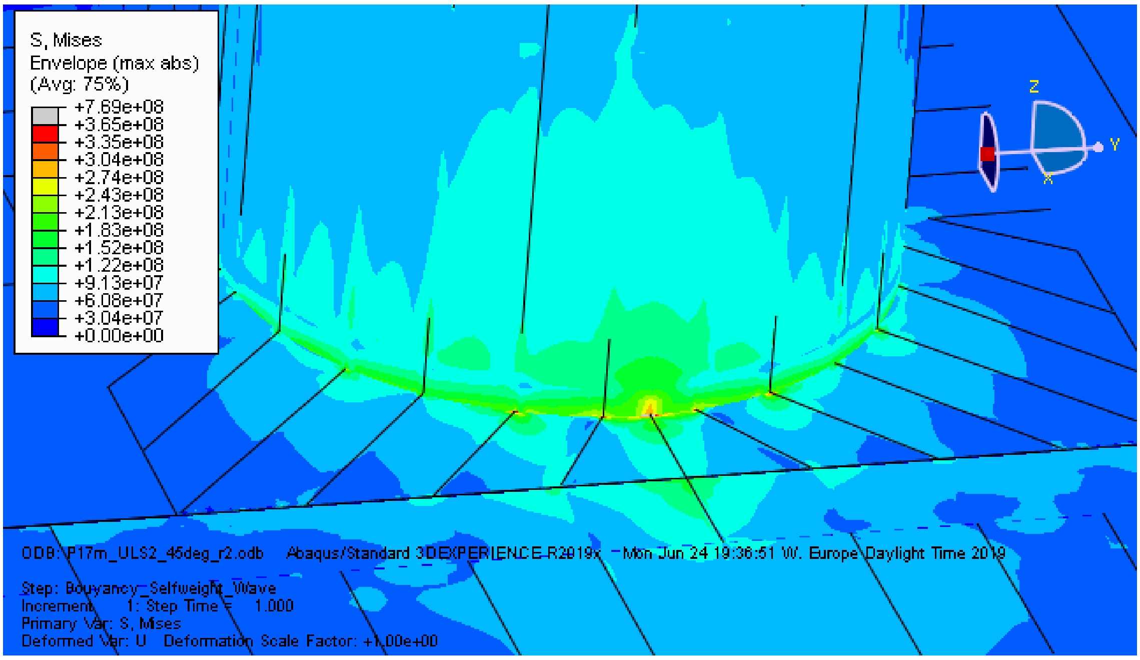
The maximum allowable yield stress for plates with steel grade S420 is 420 MPa / 1.15 = 365 MPa.

The maximum allowable yield stress for bulbflats with steel grade S355 is 355 MPa / 1.15 = 309 MPa.

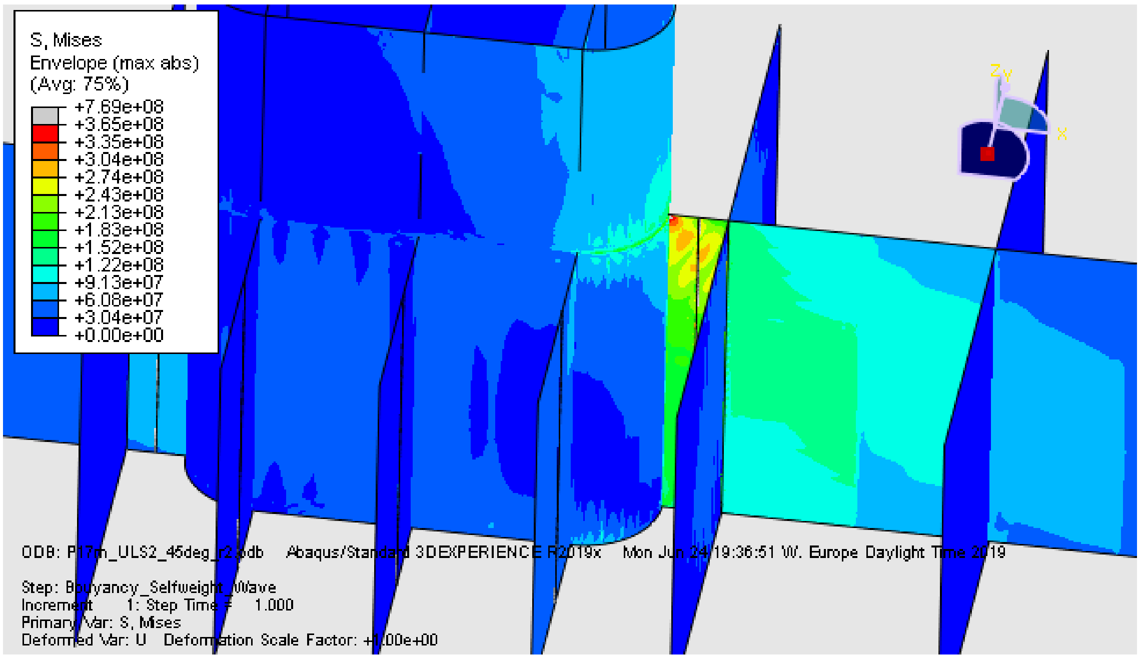


> *Figure 6-1 Mises stress at pontoon and column skin*

Overall Mises stress in pontoon and column for large pontoon (17 m width) is shown in Figure 6-1 to Figure 6-4 for the worst wave condition. Note that largest stress in legend is from tied mesh-constrain at the longitudinal bulkhead in the pontoon and not real stress. The tied mesh constrain stresses are shown in Figure 6-5 and Figure 6-6. The mentioned peak stress in column pontoon intersection is shown in Figure 6-2 and Figure 6-4.

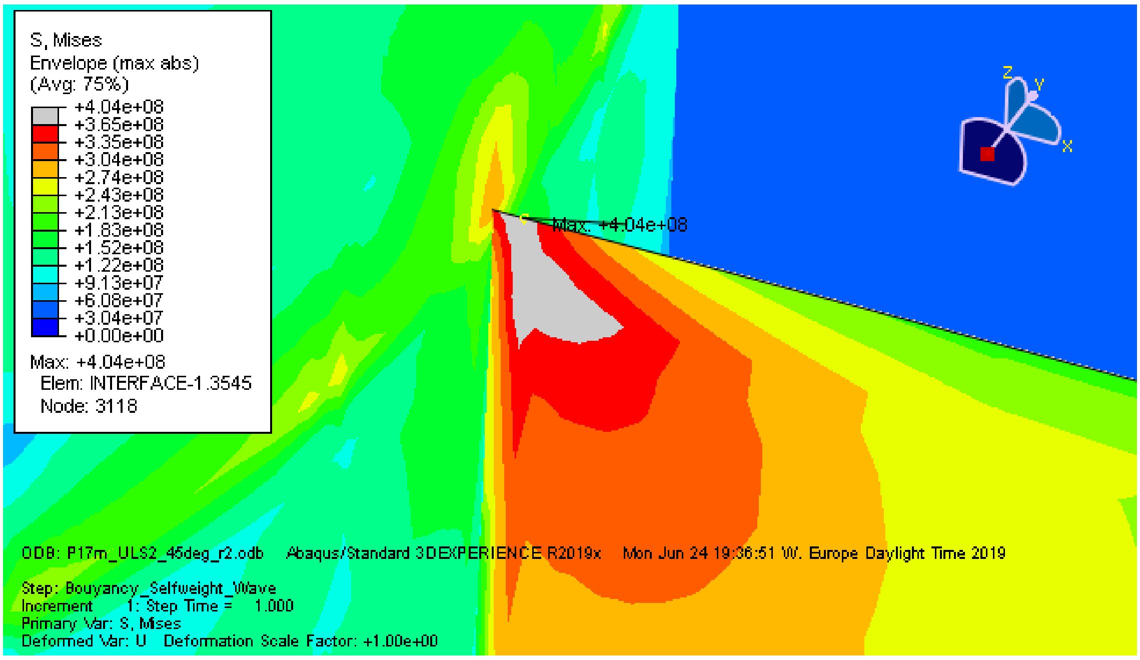


> *Figure 6-2 Mises stress at column-pontoon interface. Note that largest stress in legend is from tied mesh-constrain at the longitudinal bulkhead in the pontoon and not real stress.*

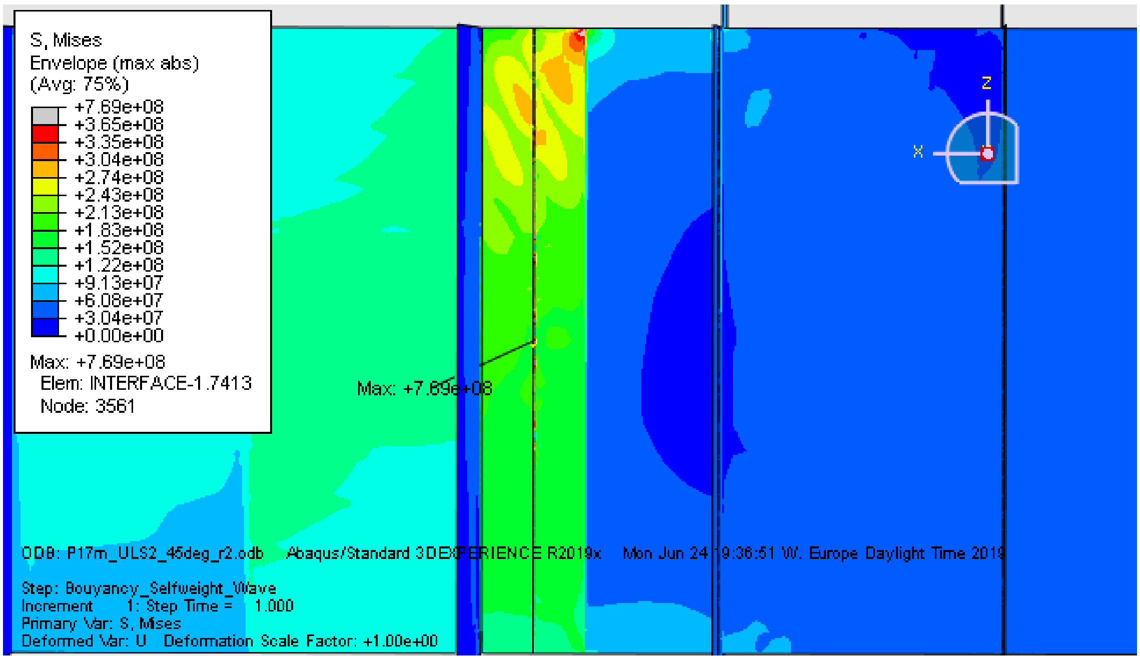


> *Figure 6-3 Mises stress at pontoon bulkheads*

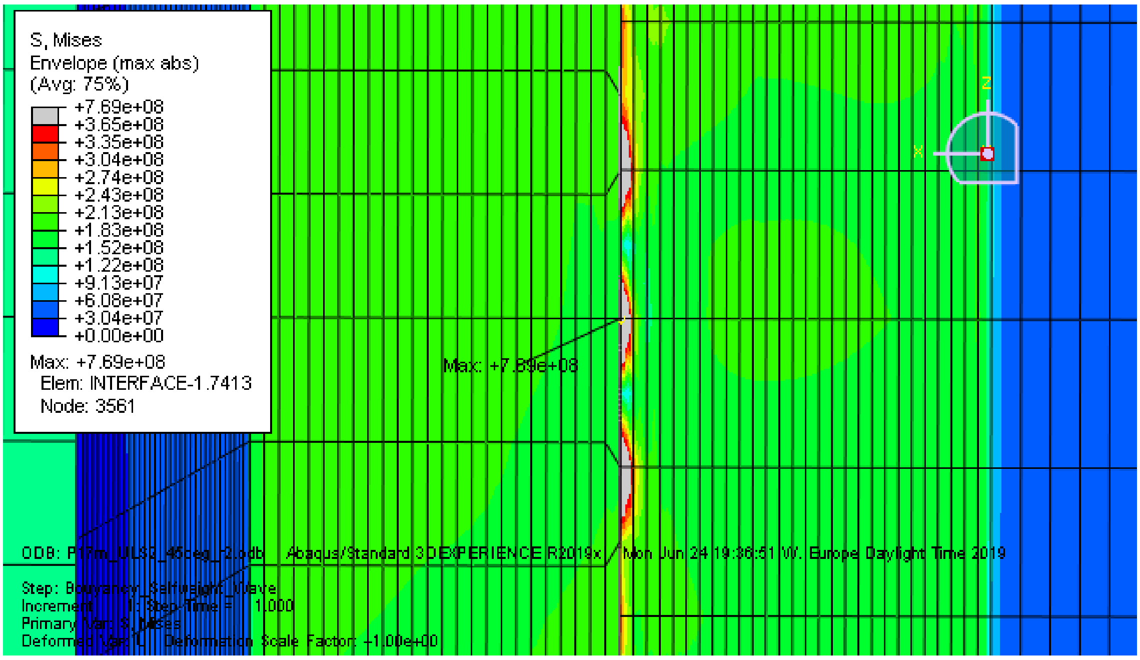
Additional stress plots for all wave directions are found in Appendix A.



> *Figure 6-4 Maximum Mises stress at intersection between column and pontoon longitudinal bulkhead.*



> *Figure 6-5 Tie constrain at bulkhead*



> *Figure 6-6 Tie-constrain close-up*

As seen from the plots of the tie-constrains, the stresses in the bulkhead tied together are about 150-200 MPa, except for the elements closest to the tie, which is prone to give “stress-noise” in the node elements closest to the tied nodes.

> *Table 6-1 Largest reactions forces and moments taken from local ULS models compared with global analyses ULS response*

Reaction Force/Moment	Value	% of global analysis response*
Rx	+/- 4.5 MN	78 %
Ry	+/- 12.5 MN	124 %
Rz	- 37 MN	94 %
RMx	+/- 460 MNm	130 %
RMy	+/- 145 MNm	89 %
RMz	+/- 117 MNm	152 %

*) At column top; a percentage above 100% means that the response in local analyses is overestimated, a value below 100% indicates that the response in local analyses is underestimated with the applied ULS static loads.

The reactions forces and moments from the local ULS models are compared with the ULS response at top of column from the global analyses (for K12 model 27). The table above show the largest values compared with the envelope values from the global analyses. As seen from the table the applied static ULS load underestimates response in pontoon longitudinal direction by 11-22%, but overestimates the response in pontoon transversal direction by 24-30% and for column torsional moment by 52%.

Bending moment of the pontoon itself is the main contribution to the stresses in the pontoon column interface, so the difference in column shear force of 22% or 1.3 MN (from 5.8 MN to 4.5 MN) results only in additional sigma-X stress equal to less than 10 MPa at the top of the longitudinal bulkhead, if the bulkhead transfers this added load alone.

6.2.2 ULS Results – Stipla check of transverse bulkhead for 17meter wide pontoon

DNVRPG Girder check based on DNV-RP-C201/OS-C101 Version 2.2 Copyright (C) 2004-2014 StruProg AB	Project: Bjørnafjorden Fase 5 Identification: Innvending bulkhead m/vanntrykk fra fu ll ballast 9m for 17m pongtong	Page: 1/1 Date: 24.05.2019 Time: 10:11
File: c:\abaqus_workspace\bf fase 5\results\uls\stipla\indre bulkhead med vanntrykk 9m for 17m pongtong og uls-2.drpg		

Material/Safety Format:

Plate/girder:			General/General	
Yield stress	fyp/fyg	=	420/420	MPa
Youngs modulus	E	=	2.10E+5	MPa
Material Factor:	gm	=	1.15	
Allowable Usage Factor:	UF	=	1.00	

General:

Buckling length	Lk	=	8500	mm
Mom fact - Field	km2	=	8.0	
Sniped girder				

Geometry:

Girder spacing	L1	=	4500	mm
	L2	=	4500	mm
Girder span	Lg	=	8500	mm
Length of panel	Lp	=	9000	mm
Lat tors buckl length	Lt	=	2125	mm
Stiffener spacing:	s	=	750	mm
Plate thickness	t	=	16.0	mm

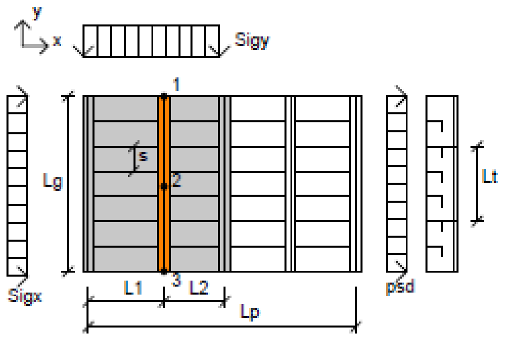
Stiffener: BF 320x12.0

Stiffener continuos through girder (Eq 8.4)

Stress/Force:

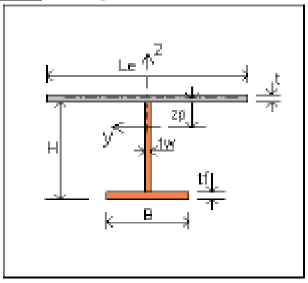
Sigx1	=	-10.0	MPa
Sigx3	=	-10.0	MPa
Sigy	=	-10.0	MPa
Tau	=	29.4	MPa
psd	=	0.122	MPa (On plate side)

Stiffened plate:



Stiffened plate effective against Sigy-stress (Method 1 ch 8.4.2)

Girder: Built-up: T 1500x300x12.0x15.0



Local buckling of web taken into account according to Eurocode 3/NS3472

Girder property:

H	=	1500	mm
B	=	300	mm
tw	=	12.0	mm
tf	=	15.0	mm
A	=	22320	mm2
g	=	175.2	kg/m
Iy	=	5.296E+9	mm4
Iz	=	3.375E+7	mm4

Girder incl. eff. plate:

zp	=	215.3	mm (elastic)
zp	=	2.5	mm (plastic)
Ae	=	8.133E+4	mm2
Ie	=	1.911E+10	mm4
Plate in compression:			
Wep	=	6.924E+7	mm3
Weg	=	1.458E+7	mm3
Plate in tension:			
Wep	=	9.456E+7	mm3
Weg	=	1.040E+7	mm3

Webclass: 1 M - Pl in compr.
Webclass: 4 M - Pl in tens.
Webclass: 4 N - Axial force
Flangeclass: 3

<u>GIRDER BUCKLING CONTROL:</u> (2 = field g = girder, p = plate)
Le = 4448.4 / 3400.0 mm (buckling/bending, ref ch 8.4.2) Sigxsd = -10.0 MPa p0 = 0.001 MPa z* = 215.3 mm
UF2g=Nsd/NksRd-2*NSd/NRd+(MSd+NSd*z)/(MstRd*(1-Nsd/Ne)) =
720.0/28525.5-2*720.0/29703.5+(5006.9+720.0*0.215)/(5325.8*(1-720.0/548283.5)) = 0.95 < 1.00 (Eq 7.59)
UF2p=Nsd/NkpRd+(MSd+NSd*z)/(MpRd*(1-Nsd/Ne)) = 720.0/28909.0+(5006.9+720.0*0.215)/(25289.3*(1-720.0/548283.5)) = 0.23 < 1.00 (Eq 7.60)
Shear control Vsd/Vrd = 2337.1/3757.5 = 0.62 < 1.00 (Ch 7.8)
Recommended maximum distance between tripping brackets to avoid lateral torsional buckling = 2422 mm (Eq 8.31)

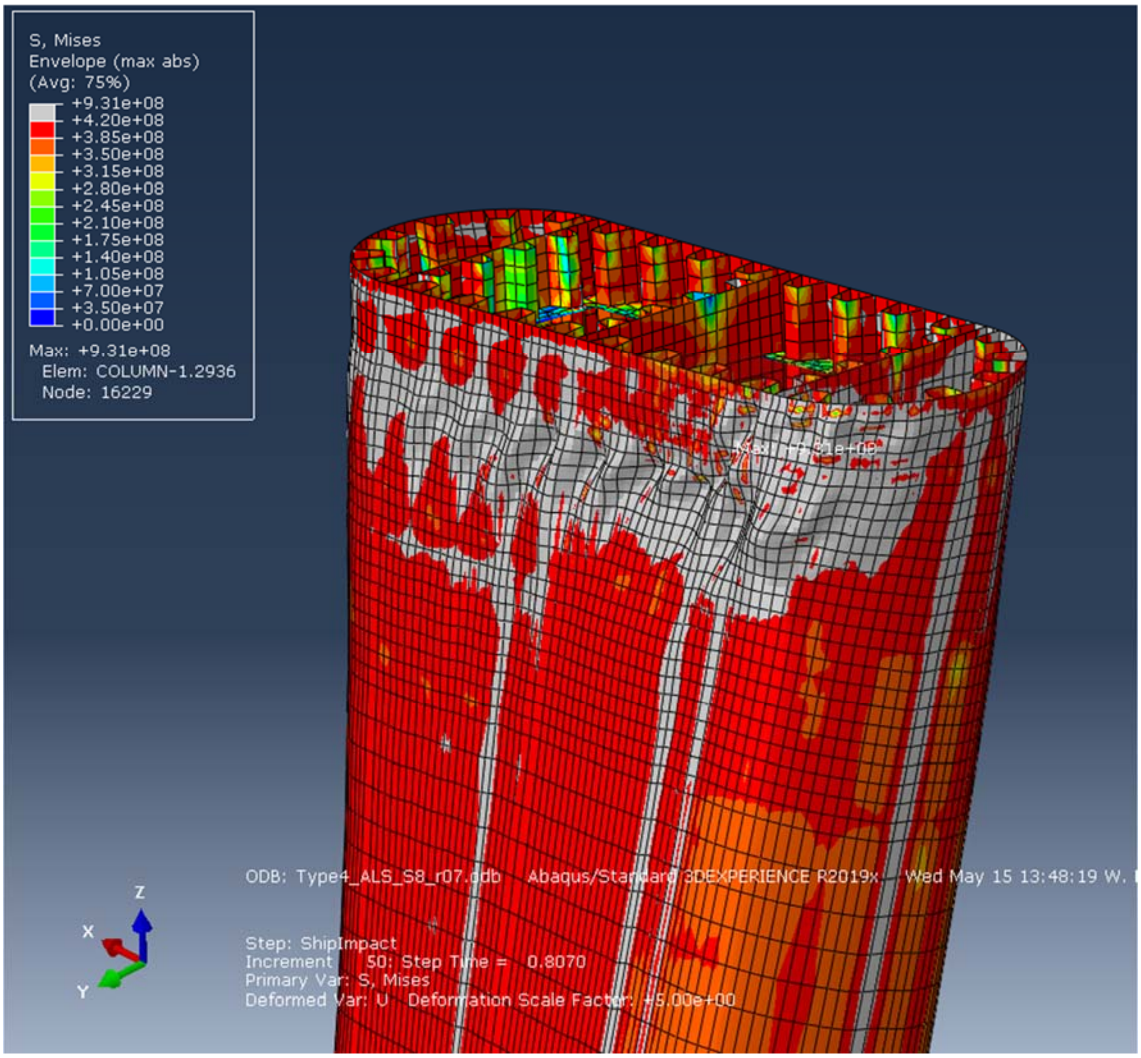
<u>GIRDER YIELD CHECK:</u> (check at point 2, plate(p) and girder(g)). Effective width Le = 2870.0 , ref DNV OS C101, sec 5, G400
Point 2p: UF = Sigjd/fyd = 102.6/365.2 = 0.28 < 1.00
Point 2g: UF = Sigy/fyd = 334.1/365.2 = 0.91 < 1.00

<u>GIRDER SECTION MODULUS CHECK (DNV-OS-C101, sec 5, G 600):</u> (check at point 2, plate(p) and girder(g))
Effective width Le = 2870.0 mm calculated according to DNV OS C101, sec 5, G400, Np>5
Point 2p: Sigjd = 51.9 MPa UF = Zs/Wp = 1.585E+7/5.937E+7 = 0.27 < 1.00
Point 2g: Sigyd = 10.0 MPa UF = Zs/Ws = 1.398E+7/1.443E+7 = 0.97 < 1.00

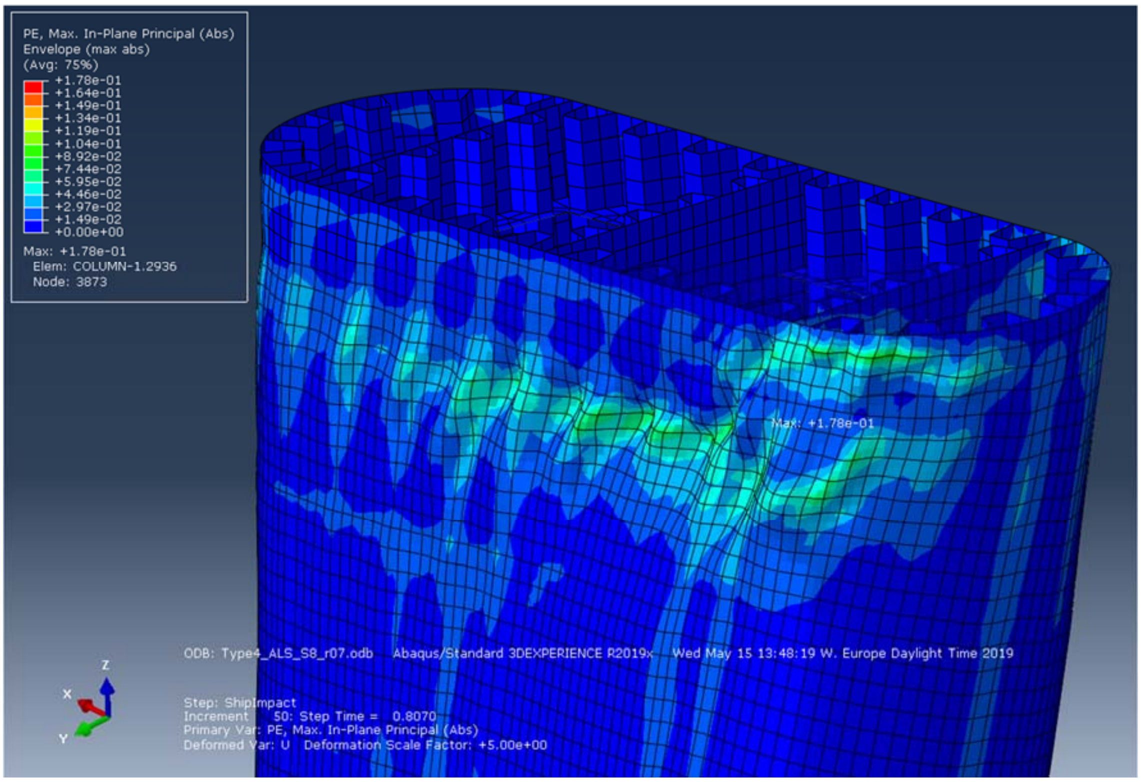
<u>GIRDER WEB AREA: (DNV-OS-C101, sec 5, G 603):</u>
Web area at support: tw/t = 8.62/12.0 = 0.72 < 1.00

6.2.3 ALS Results

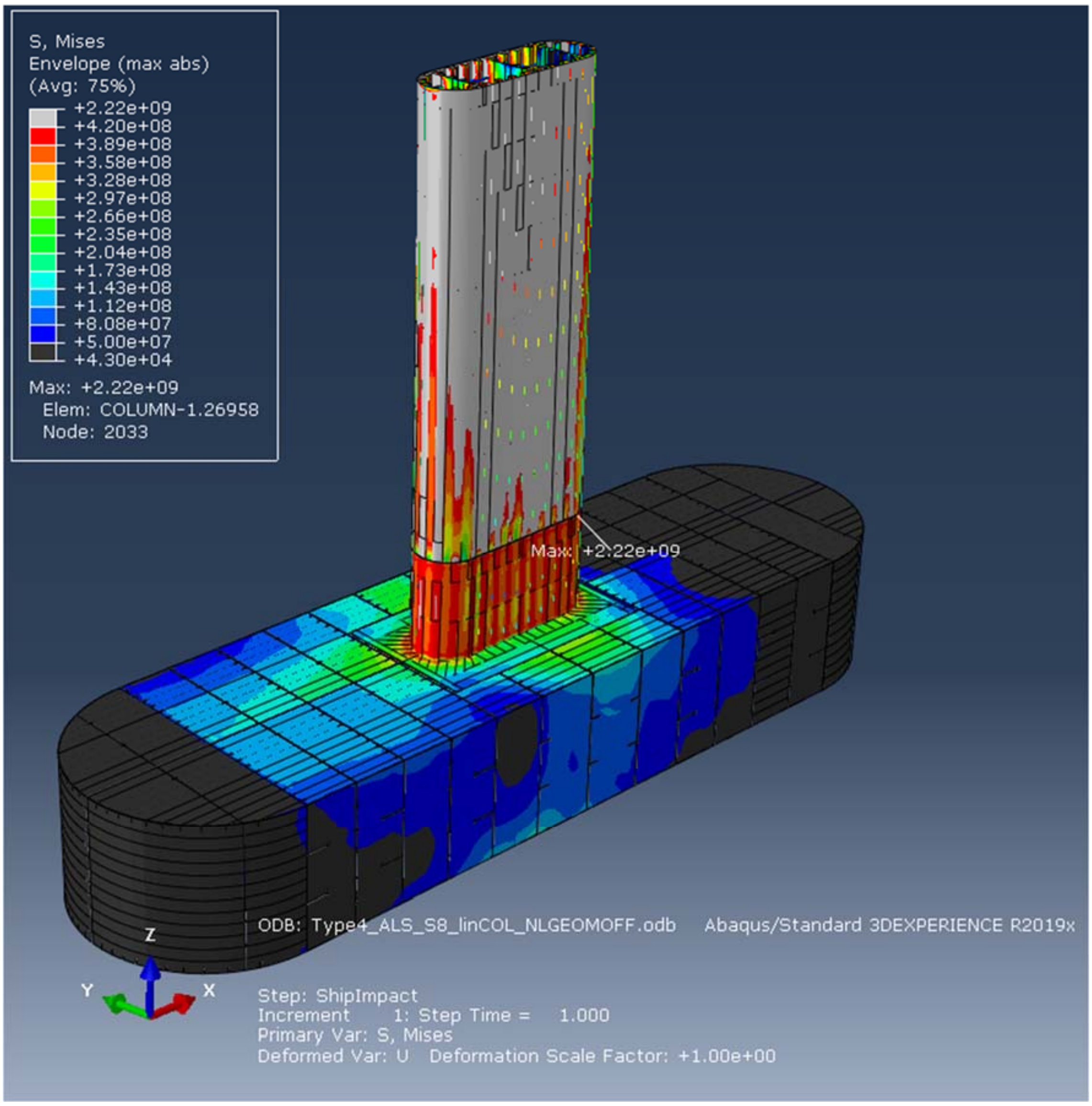
Two analyses are performed for the large pontoon with tall column. First, a fully non-linear analysis was performed for the ship impact at the large pontoon with the tall column. At 75-80% of the ship impact load, extensive yielding and subsequent local buckling is seen at Figure 6-7 and Figure 6-8. Due to large rotations, the non-linear analysis performed with statically applied impact force solved implicitly, the analysis did not manage to complete any further. To avoid time-consuming dynamic analyses to solve the problem, the upper part of the column was treated as linear-elastic and local buckling prevented, so that the analyses was managed to run completely. As the plastic hinge is already verified in other dynamic ship-impact analyses, and the establishment of such a hinge is shown to occur here, it was deemed sufficient to model the column such that the analysis could complete as the focus is primarily on the interface between column and pontoon. Figure 6-9 and Figure 6-10 show the stresses and plastic strains in the lower part of the column modelled with non-linear material.



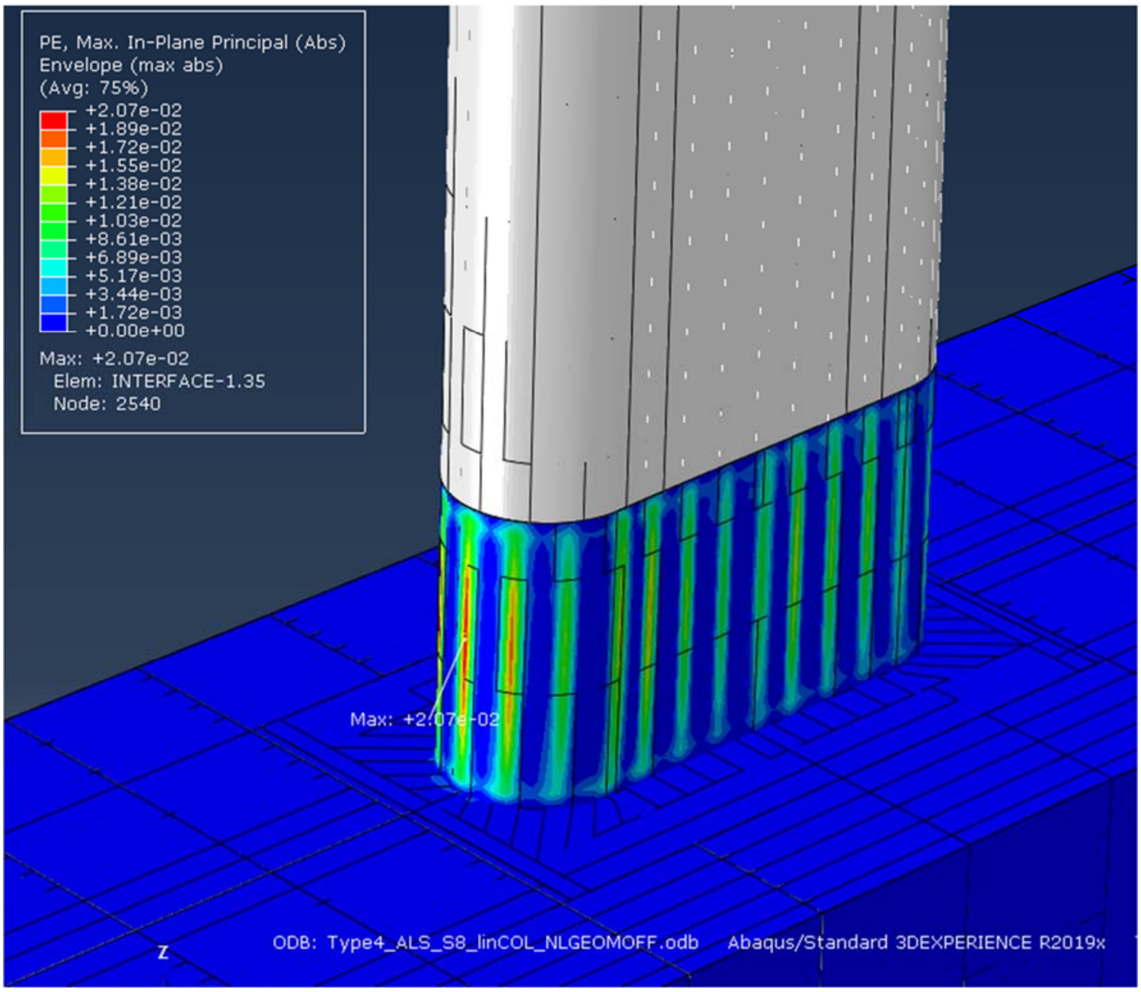
> *Figure 6-7 Yielding and local buckling at 80% ship impact load, which shows the establishment of a plastic moment hinge at the top of the column.*



> *Figure 6-8 Plastic strain at column top at 80% of ship impact load.*



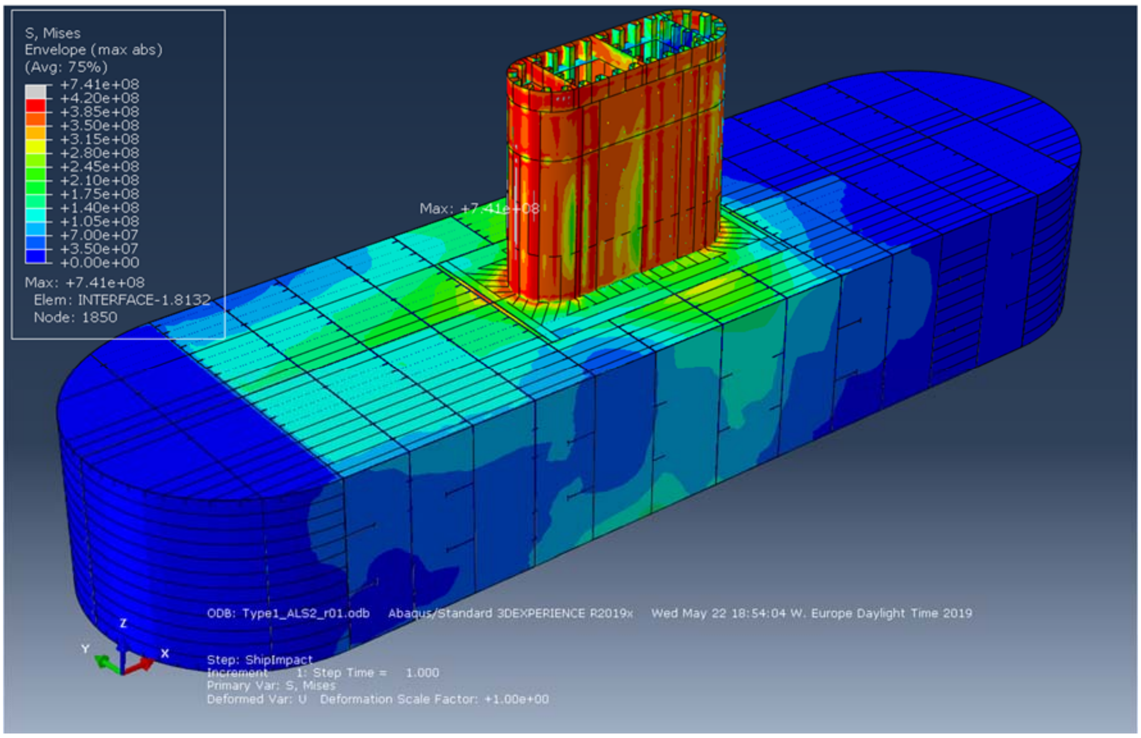
> *Figure 6-9 Mises stress at 100% impact load, with linear elastic material in column (except for 6.25 meters closest to pontoon) and NLGEOM turned off, i.e. no local buckling to occur.*



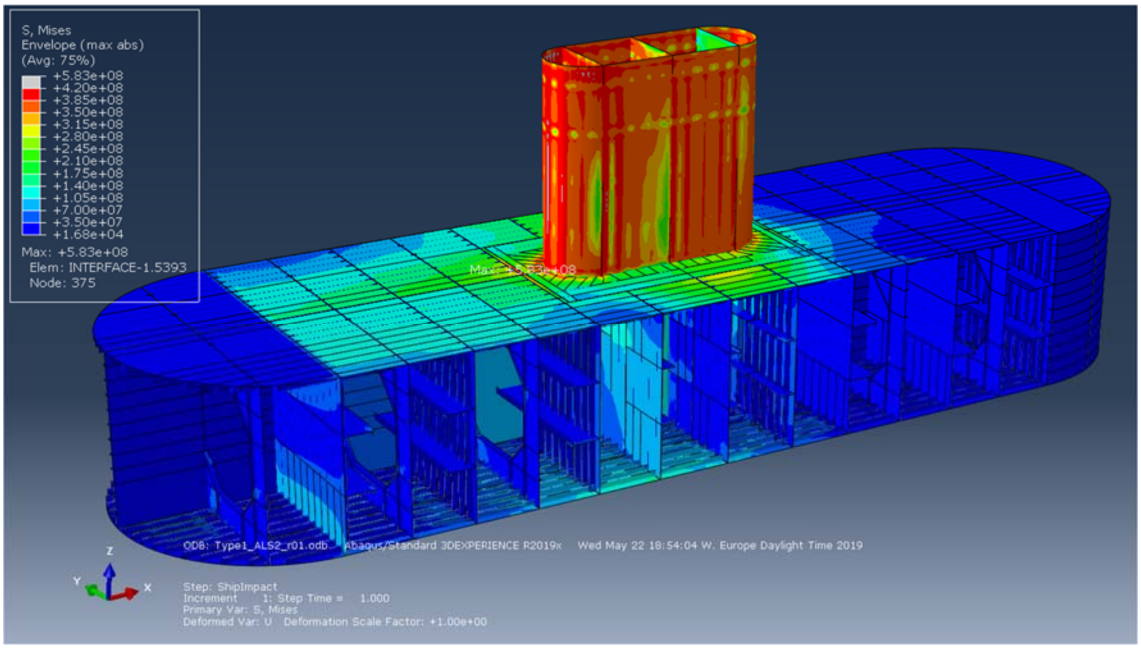
> *Figure 6-10 Plastic strain in column part with non-linear material at 100% impact load. Maximum 2.1% due to combined torsion and shear force from eccentric impact load.*

6.3 Large Pontoon with short column

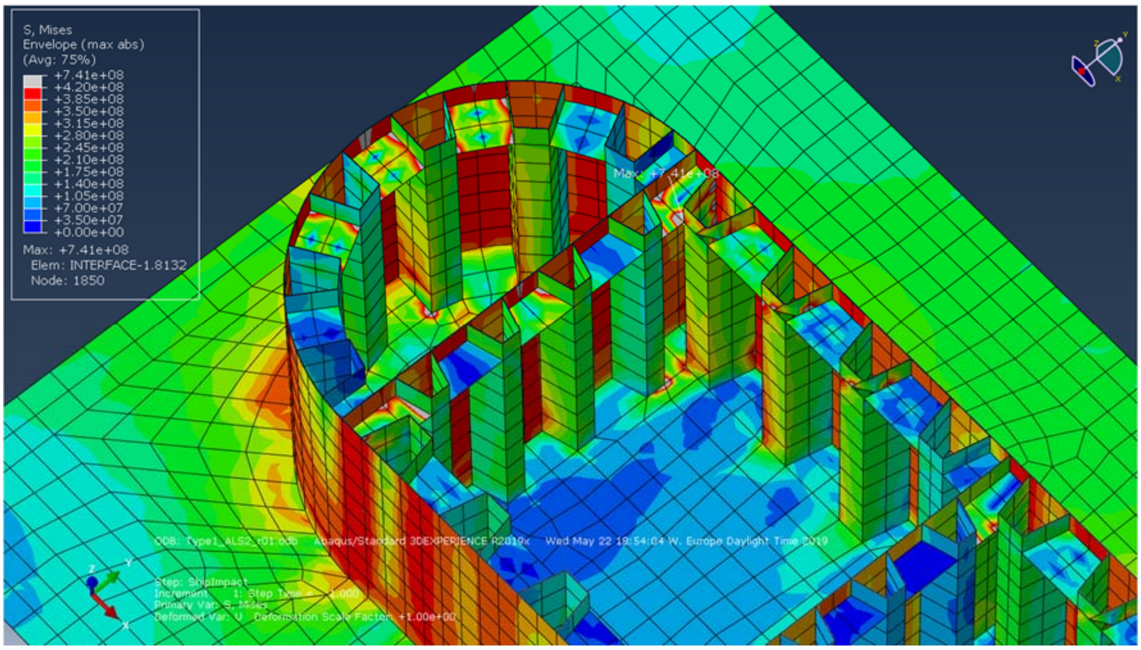
6.3.1 ALS Results



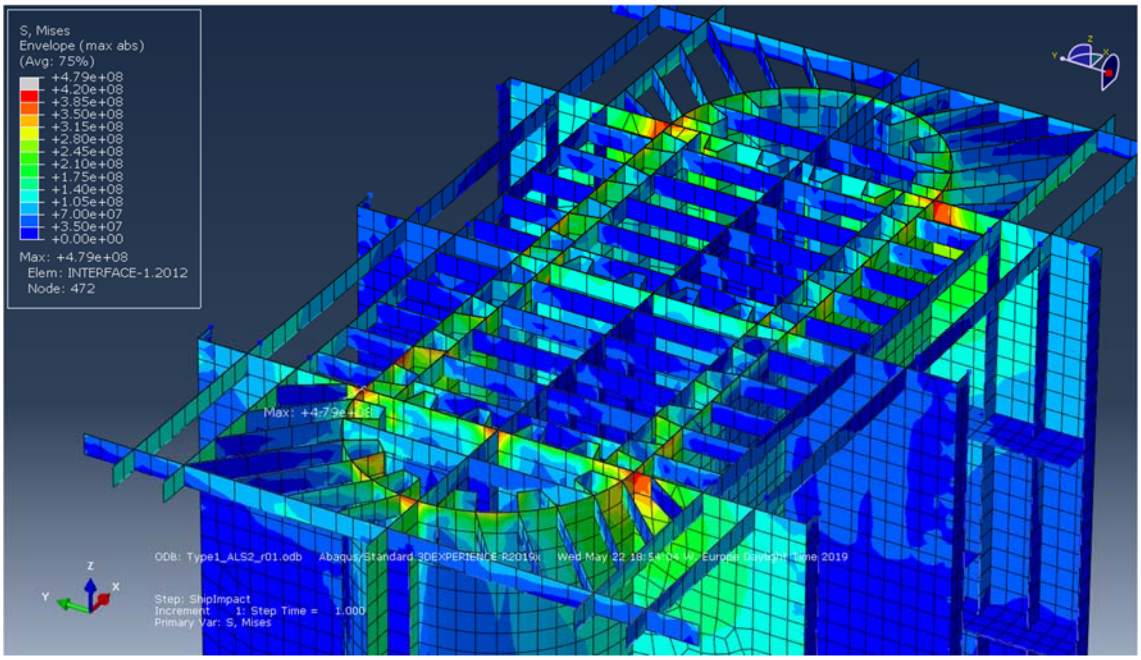
> *Figure 6-11 Von Mises stress, short column. Yield in column skin to torsion from eccentric ship impact*



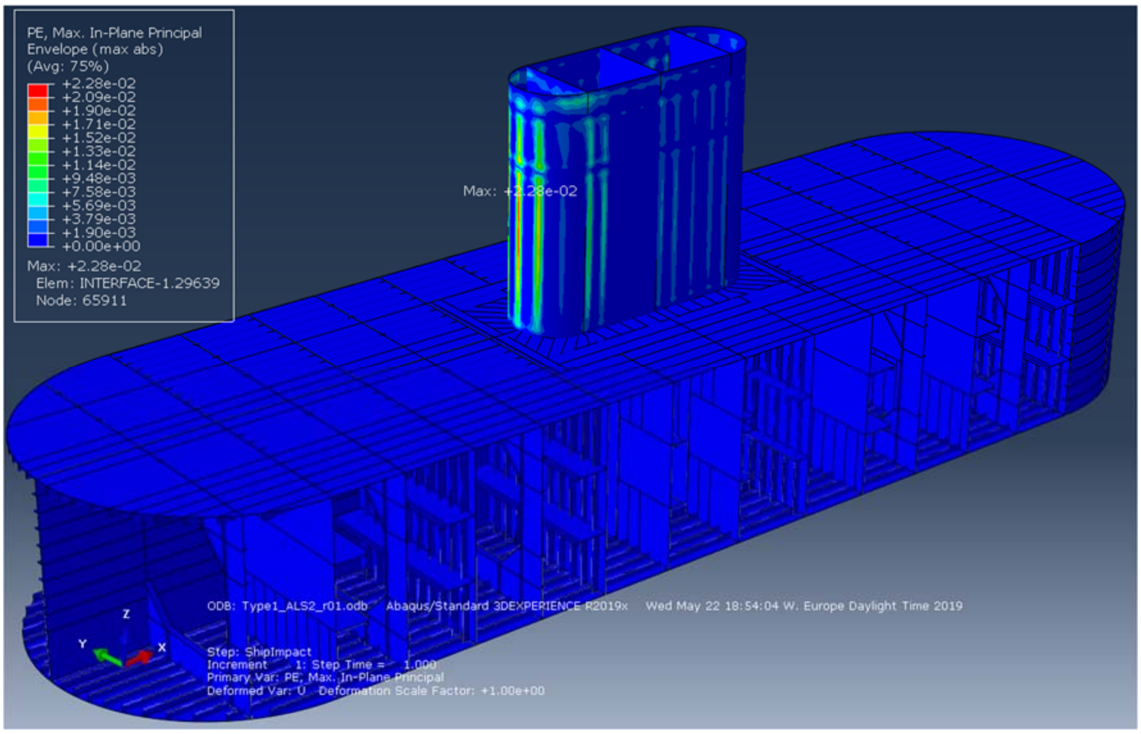
> *Figure 6-12 Von mises stress, short column (outer walls removed)*



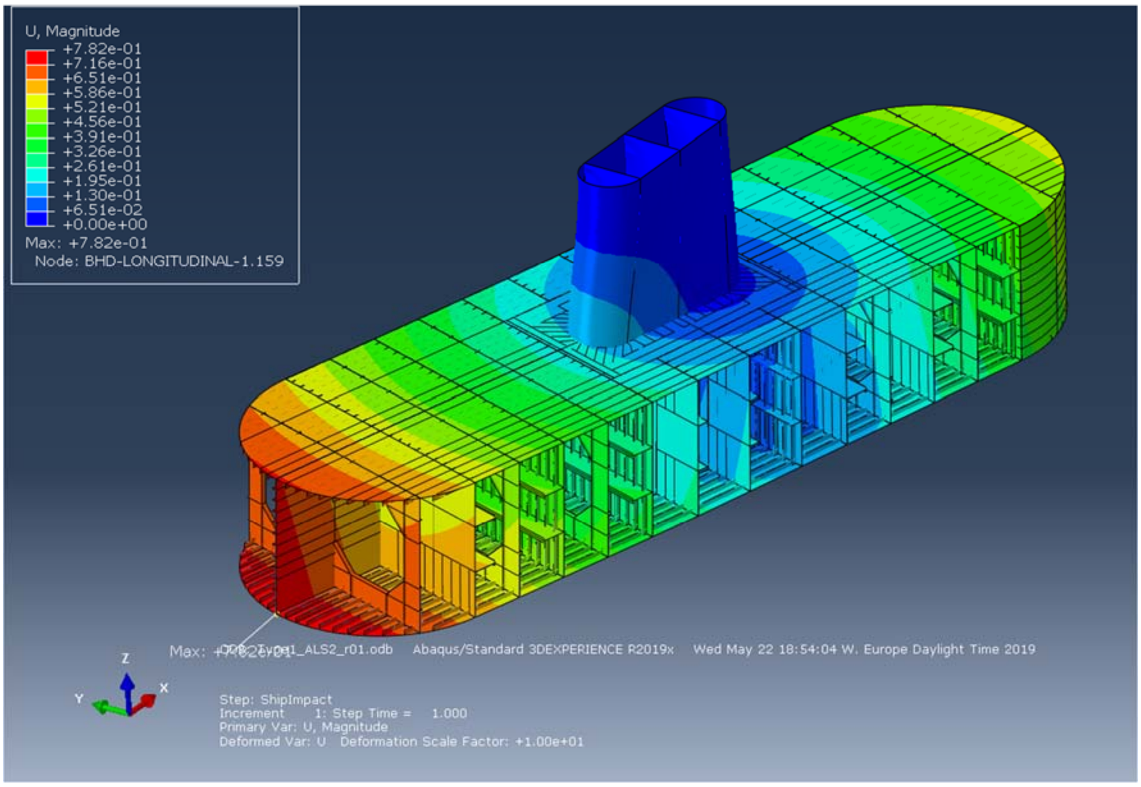
> *Figure 6-13 Von Mises stress local yield*



> *Figure 6-14 Hotspot stresses at bulkheads towards column (below pontoon top plate)*



> *Figure 6-15 Plastic strain, maximum 2.3% at column skin due to torsion from eccentric ship impact*



> *Figure 6-16 Deformation of pontoon from eccentric ship impact (0.8 m at pontoon end). Note that the stiff boundary conditions will likely underestimate this type of deformation.*

6.4 Discussion

In ULS, the wave which is shown to give the highest utilization is a wave with wave bottom at ¼ of the pontoon length and wave crest at ¾ of the pontoon length, at an angle of 45 degrees to the pontoon longitudinal axis. The wave load model applied as static pressure show some discrepancies with the response in column top from global analyses, but the 1.3 MN difference in column shear force at column top results only in a minor stress increase in pontoon-column interface. The difference in column top bending moment is more than accounted for with the increase in shear force. Hence the difference between global response and local response at pontoon-column interface level is negligible.

All stresses in ULS are below the Mises yield criteria and no permanent deformation occur in except for one local peak between pontoon longitudinal bulkhead and column intersection at top of pontoon. This peak stress will be reduced with added knee-plate mentioned in linear elastic analyses chapter. Elsewhere in the pontoon the stresses are far below yield limit. The knee-plate may also be necessary from fatigue life calculations.

For ship impact ALS analysis, the eccentric impact lead to a combination of shear force, torsion and bending moment in the column. The torsional moment in the column is enough to cause yielding in the current column design. For the short column, the maximum permanent strain is about 2% which is acceptable in an ALS case, but the top of the column towards the bridge girder should be strengthened such that the permanent deformations are sufficiently small to be able to dismantle the column from the bridge girder.

For the tall column, at 75-80% of the ship impact load, extensive yielding and subsequent local buckling is seen at column top. Due to large rotations, a regular non-linear static analysis is not able to complete, and the problem must be analyzed dynamically. As the plastic hinge is already verified in other dynamic ship-impact analyses, and the establishment of such a hinge is shown to occur here, it was deemed sufficient to model the column such that the analysis could complete as the focus is primarily on the interface between column and pontoon. The analysis shows similar behavior for at the interface between column and pontoon for the tall column as it did for the short column, with a permanent deformation at about 2% plastic strain at the lower part of the column.

The permanent strains in the pontoon overall due to ship impact (except the impact zone itself) are negligible.

7 BUCKLING CHECK

7.1 General

When running non-linear analyses buckling will be checked implicitly. However, initial imperfections are either not present or may be larger than modelled in the finite element analyses.

7.2 Stipla Check

Separate buckling checks, and other capacity checks will be carried out in software Stipla in next phase of the project.

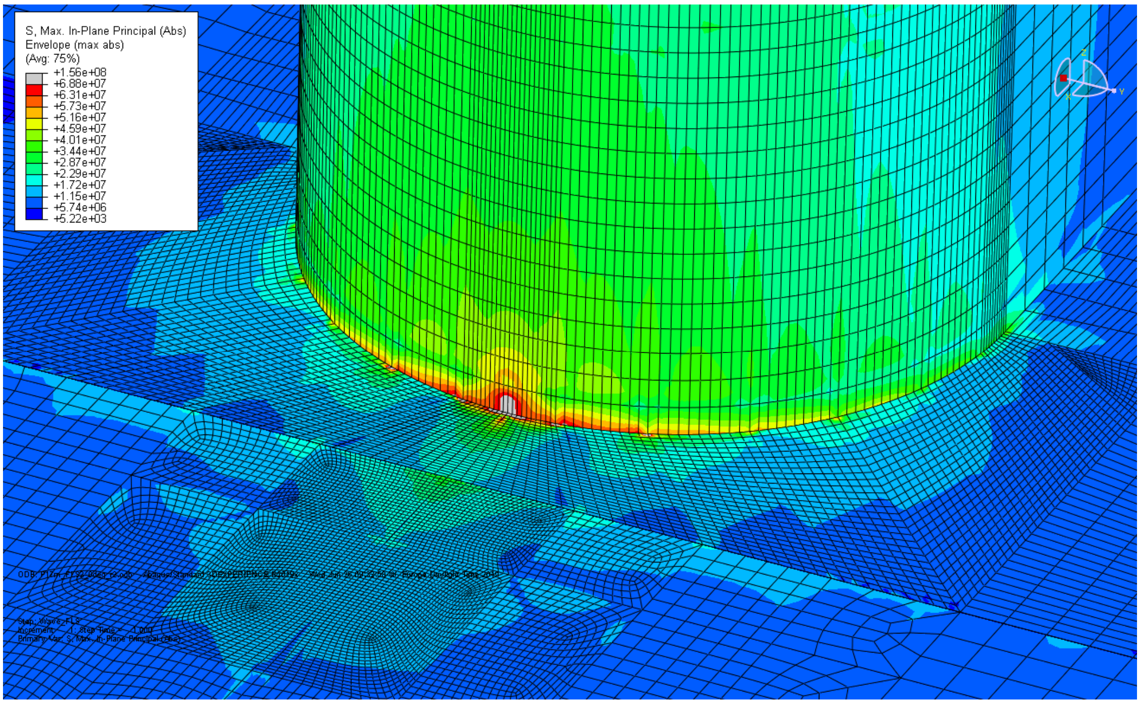
8 FLS LARGE PONTOON

The principal stresses are checked for two loading scenarios based on the unfactored dynamic wave pressure described in 4.3.3. A dynamic water pressure is applied with wave bottom at ¼ of the pontoon length and wave crest at ¾ of the pontoon length, at an angle of 0 and 45 degrees to the pontoon longitudinal axis (the same which gave the highest utilization in ULS). The principal stresses shown are limited to two S-N curve stress limits, with Weibull shape parameter h=1.2 and utilization factor η=0.2 (which gives a reduction factor 0.717).

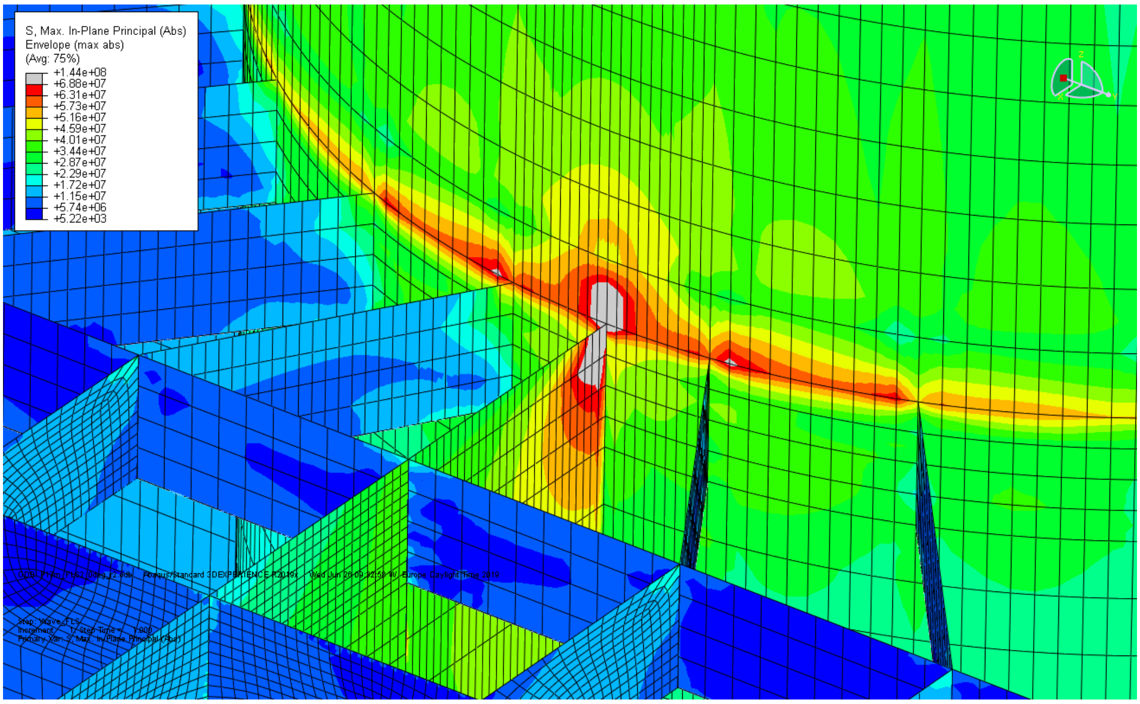
For curve D and T this yields an allowable extreme stress range equal to 191.9 MPa * 0.717 = 137.6 MPa (principal stress limited to half = 68.8 MPa)

For curve F3 this yields an allowable extreme stress range equal to 119.3 MPa * 0.717 = 85.5 MPa (principal stress limited to half = 42.8 MPa)

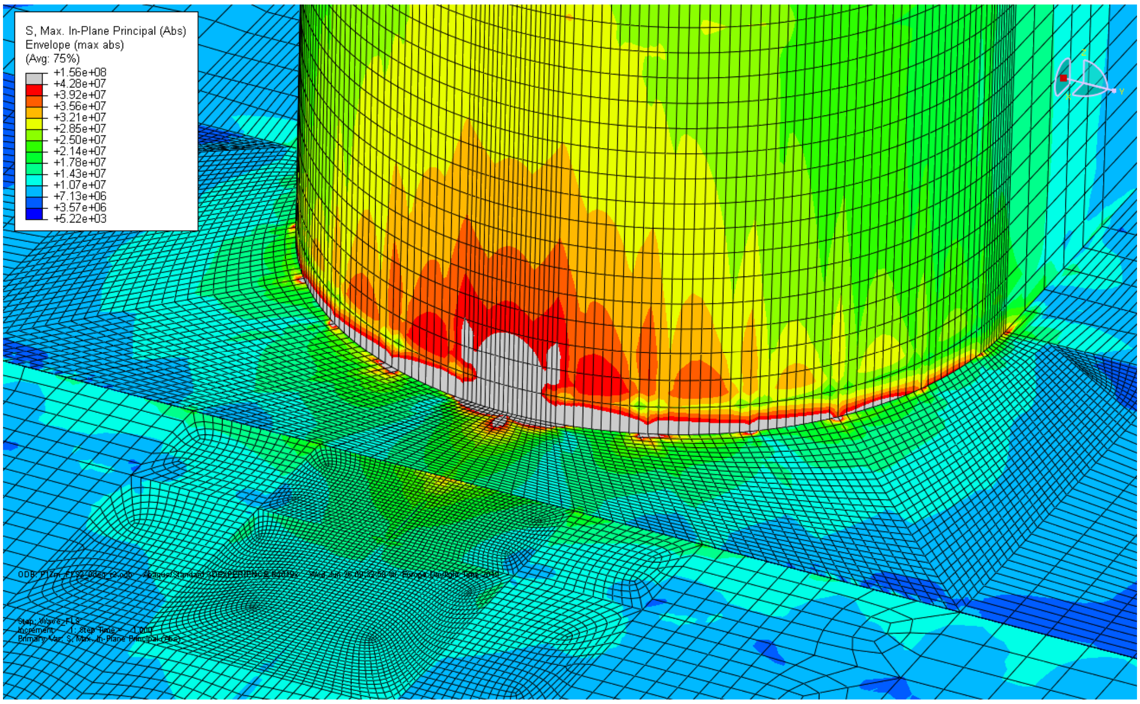
Only one area is subjected to high stress concentrations in the pontoon, namely the intersection between the pontoon top-plate and -longitudinal bulkhead, and the column. The following figures shows the principal stresses limited to the allowable extreme stresses for the curves calculated above for the two wave directions.



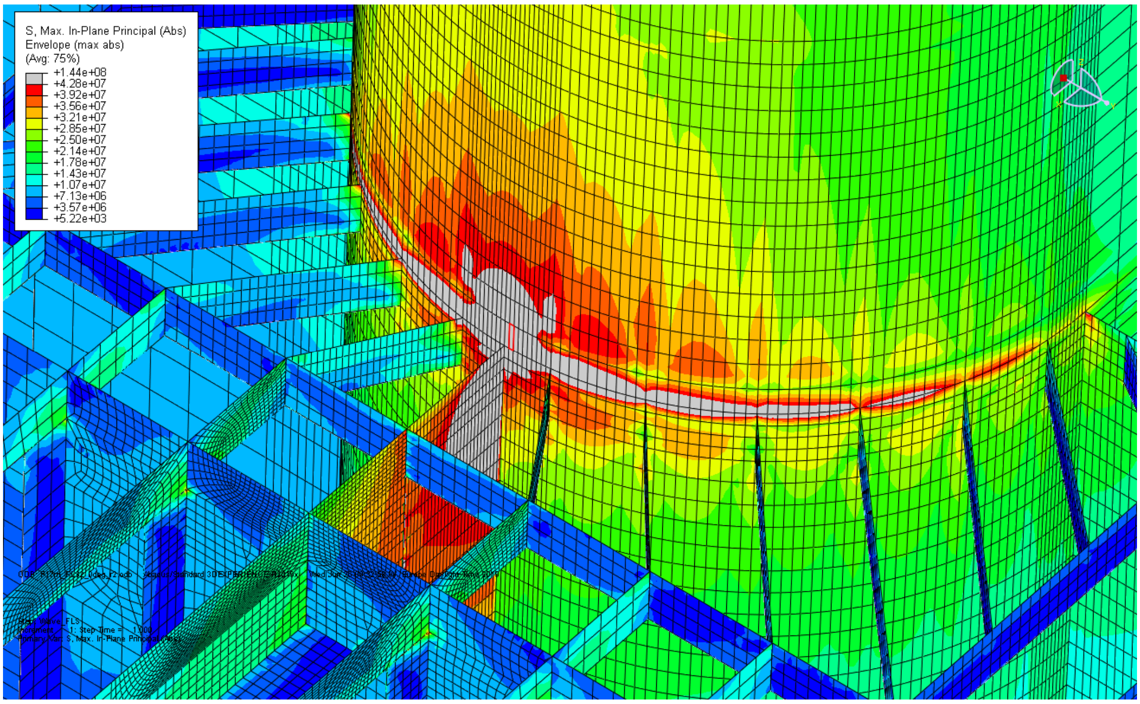
> *Figure 8-1 Principal stress for wave direction 0 degrees, limited to 68.8 MPa (D and T curve). Top of pontoon plate towards column.*



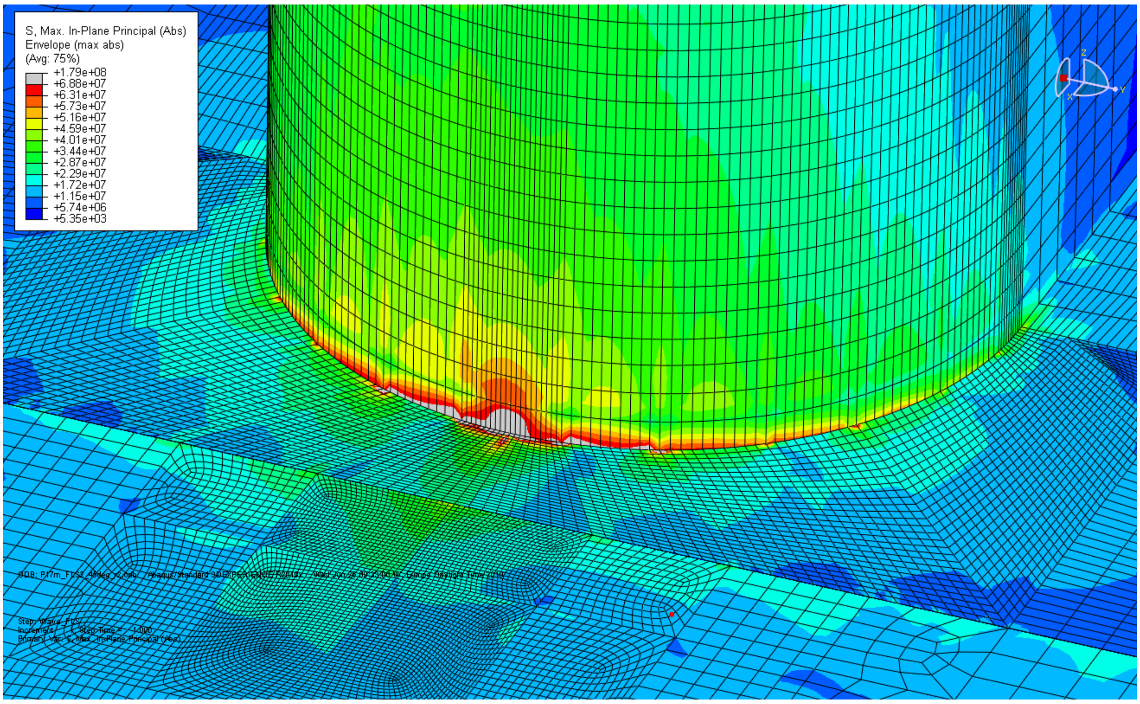
> *Figure 8-2 Principal stress for wave direction 0 degrees, limited to 68.8 MPa (D and T curve). Top of pontoon plate towards column, top plate removed.*



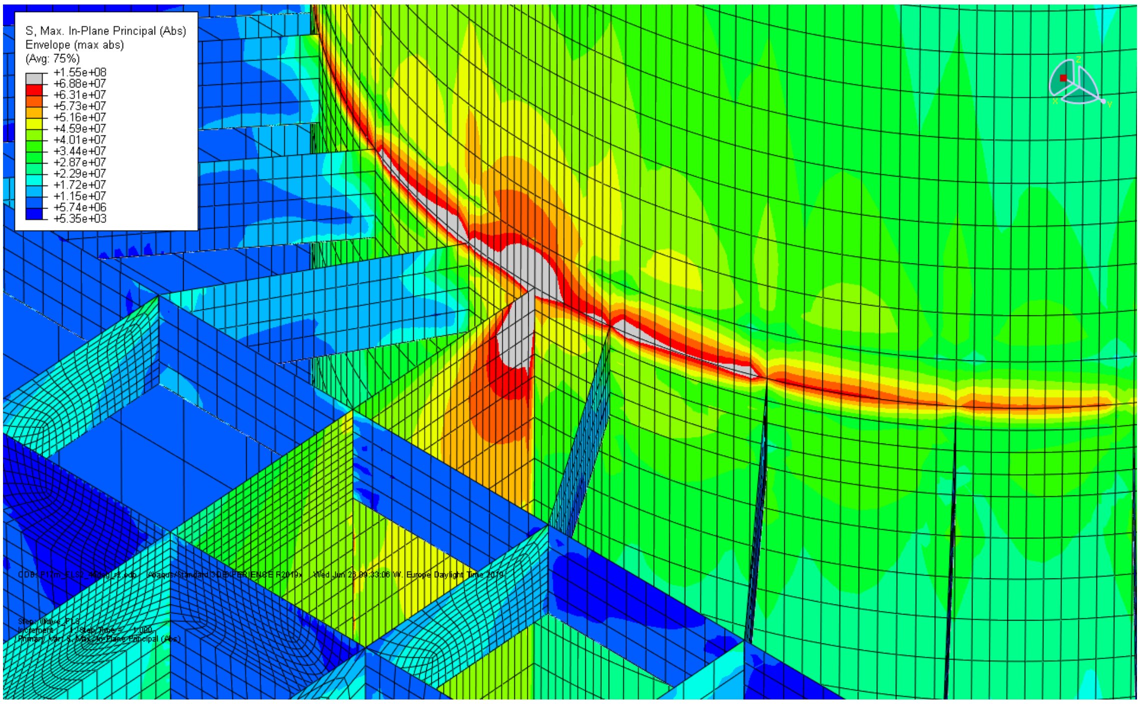
> *Figure 8-3 Principal stress for wave direction 0 degrees, limited to 42.8 MPa (F3 curve). Top of pontoon plate towards column.*



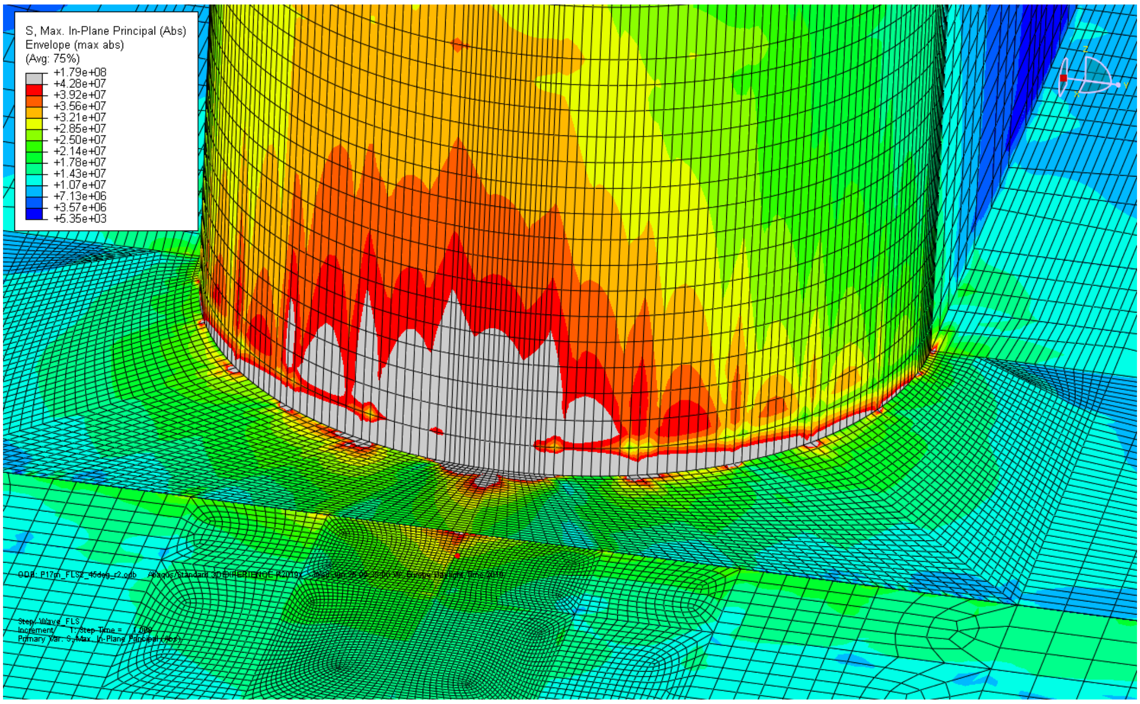
> *Figure 8-4 Principal stress for wave direction 0 degrees, limited to 42.8 MPa (F3 curve). Top of pontoon plate towards column, top plate removed.*



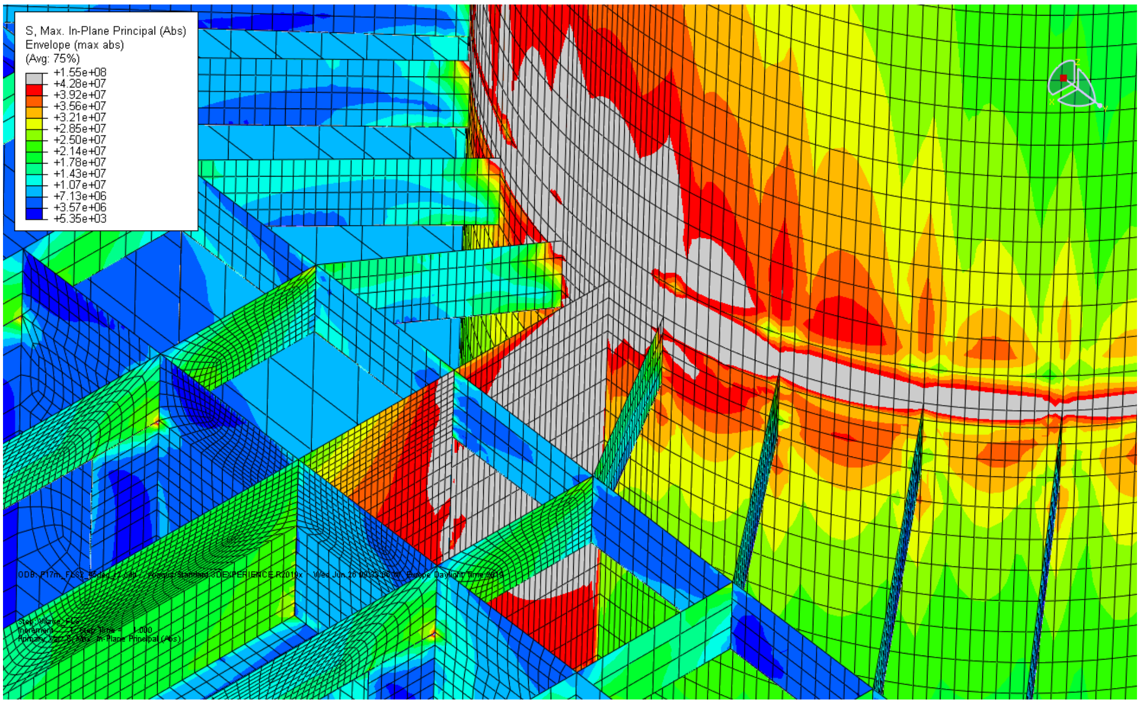
> *Figure 8-5 Principal stress for wave direction 45 degrees, limited to 68.8 MPa (D and T curve). Top of pontoon plate towards column.*



> *Figure 8-6 Principal stress for wave direction 45 degrees, limited to 68.8 MPa (D and T curve). Top of pontoon plate towards column, top plate removed.*



> *Figure 8-7 Principal stress for wave direction 45 degrees, limited to 42.8 MPa (F3 curve). Top of pontoon plate towards column.*



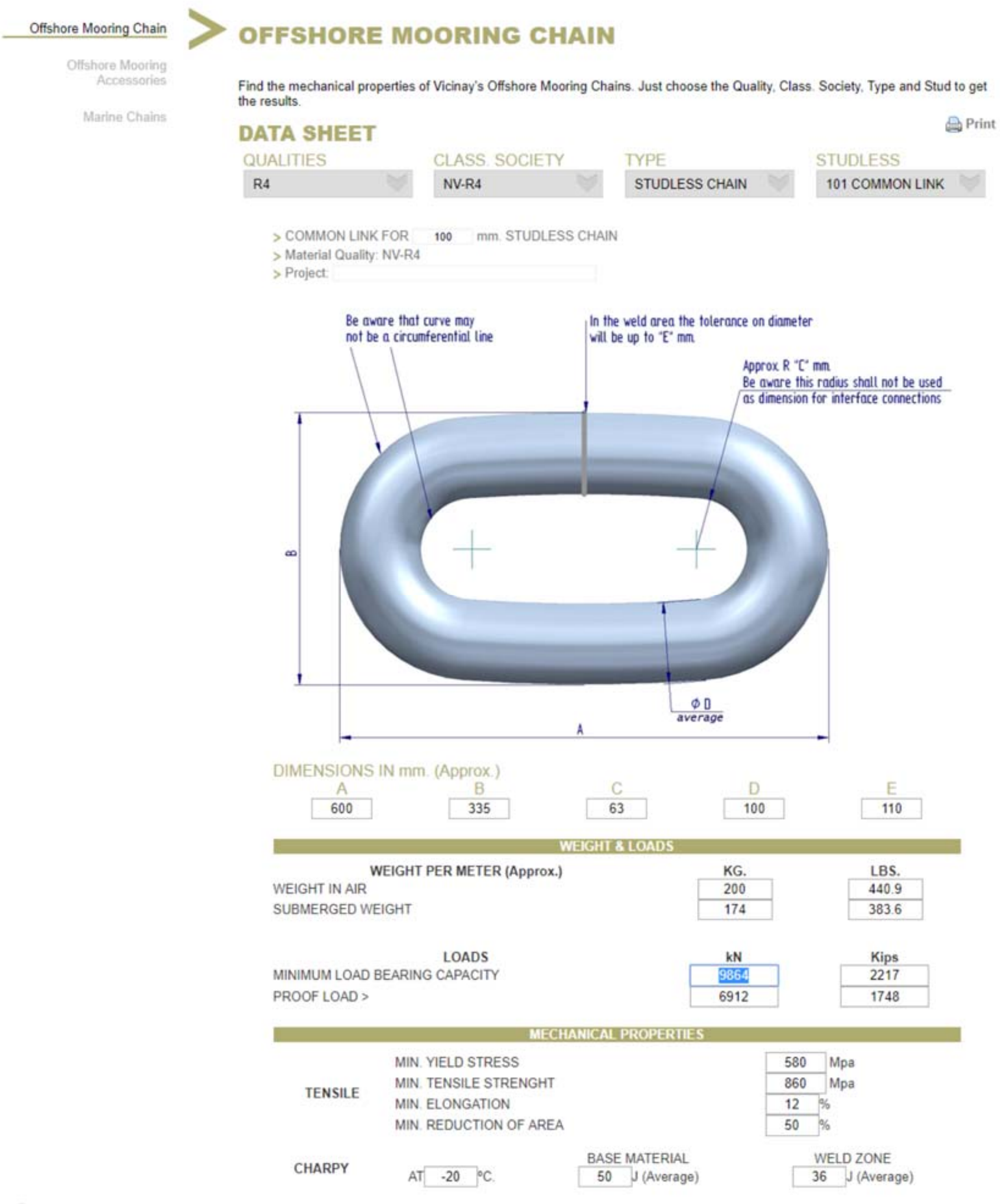
> *Figure 8-8 Principal stress for wave direction 45 degrees, limited to 42.8 MPa (F3 curve). Top of pontoon plate towards column, top plate removed.*

9 FAIRLEAD

9.1 General

A moonpool area is established for the large pontoon linear finite element model. Based on drawing SBJ-33-C5-OON-22-DR-123, Pontoons and Columns, Type 2A Anchor – Structural Arrangement, a fairlead-like shell structure is modelled inside the moonpool area. The fairlead and a corresponding area at the pontoon top has been given loads corresponding to the MBL-load for the mooring line.

9.2 Loading and Boundary Conditions



Offshore Mooring Chain
Offshore Mooring Accessories
Marine Chains

OFFSHORE MOORING CHAIN

Find the mechanical properties of Vicinay's Offshore Mooring Chains. Just choose the Quality, Class. Society, Type and Stud to get the results.

DATA SHEET

Print

QUALITIES	CLASS. SOCIETY	TYPE	STUDLESS
R4	NV-R4	STUDLESS CHAIN	101 COMMON LINK

> COMMON LINK FOR 100 mm. STUDLESS CHAIN
> Material Quality: NV-R4
> Project:

DIMENSIONS IN mm. (Approx.)

A	B	C	D	E
600	335	63	100	110

WEIGHT & LOADS

WEIGHT PER METER (Approx.)	KG.	LBS.
WEIGHT IN AIR	200	440.9
SUBMERGED WEIGHT	174	383.6

LOADS	kN	Kips
MINIMUM LOAD BEARING CAPACITY	9864	2217
PROOF LOAD >	6912	1748

MECHANICAL PROPERTIES

TENSILE	MIN. YIELD STRESS	580	Mpa
	MIN. TENSILE STRENGHT	860	Mpa
	MIN. ELONGATION	12	%
	MIN. REDUCTION OF AREA	50	%

CHARPY	AT	BASE MATERIAL	WELD ZONE
	-20 °C.	50 J (Average)	36 J (Average)

> *Figure 9-1 Mooring chain used*

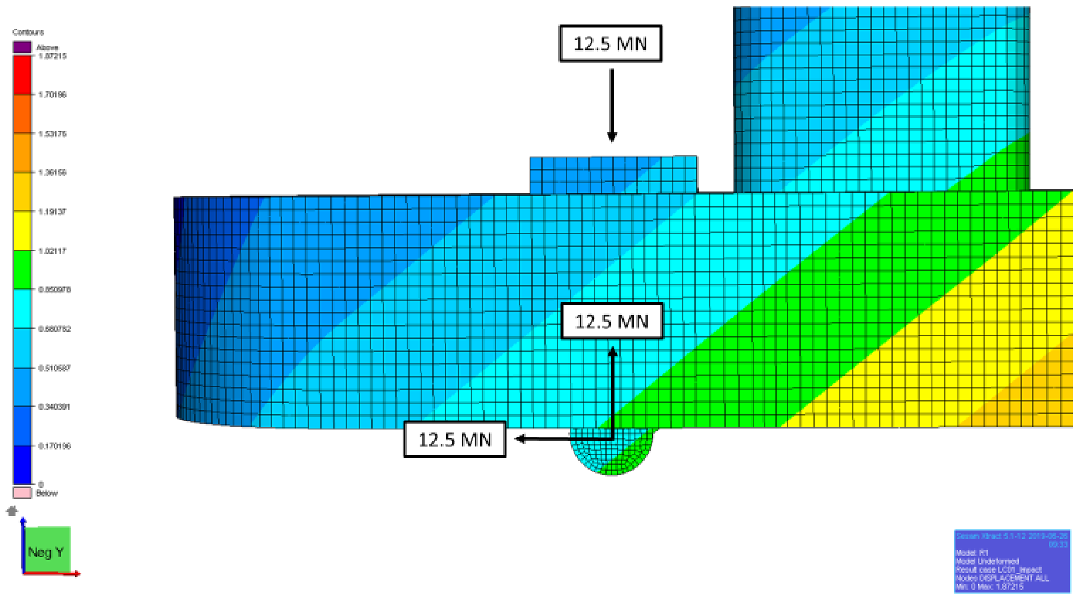
The minimum breaking load for the mooring line is set to 10 MN (i.e. 9864 kN). With a load factor of 1.25 (DNVGL-RP-C103, Chapter 6.1.2 [2]), the dimensioning anchor line breaking load becomes:

F_l = 1.25*10 MN=12.5 MN

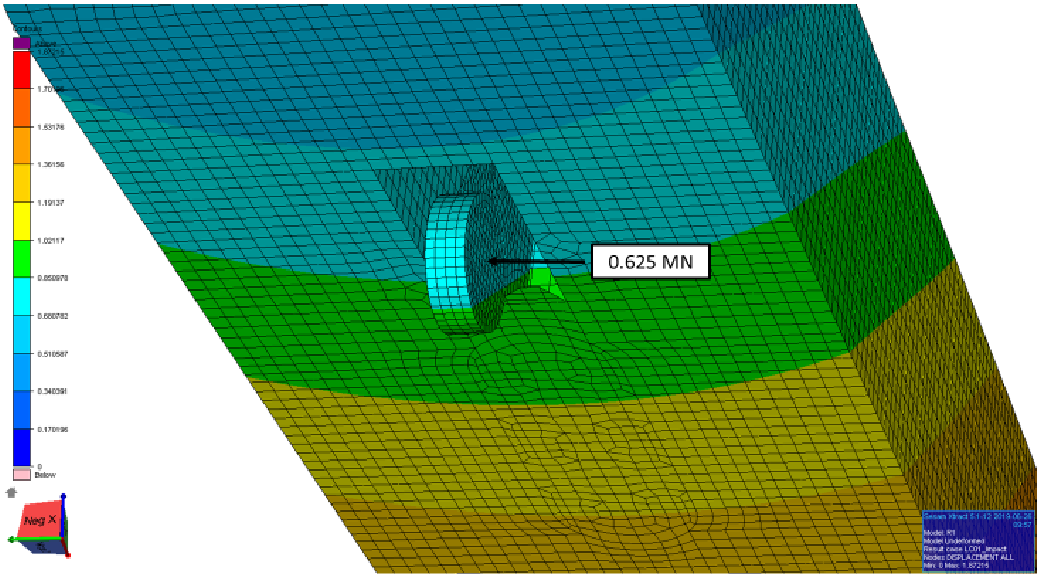
The most unfavorable loading direction for the mooring chain is used, horizontal direction or zero degrees inclination. The inclination is expected to be between 5 degrees and 32 degrees.

The transverse loading is set to 5 percent of the dimensioning anchor line breaking load:

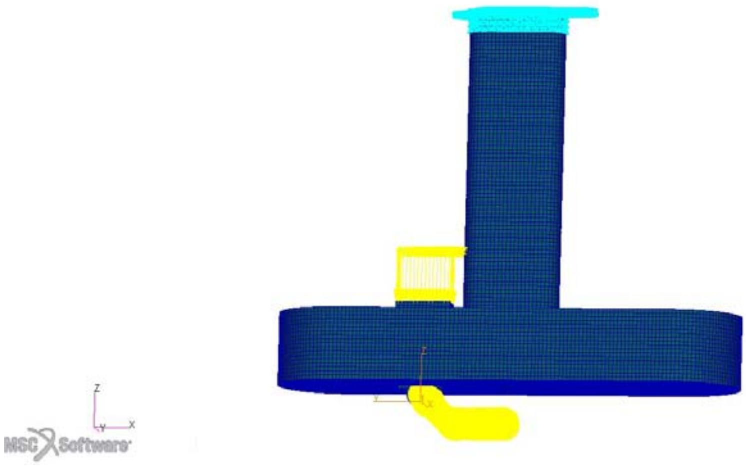
F_t = 0.05*F_l = 0.05*12.5 MN =0.625 MN



> *Figure 9-2 Load application, in-plane loading*



> Figure 9-3 Load application, transverse loading



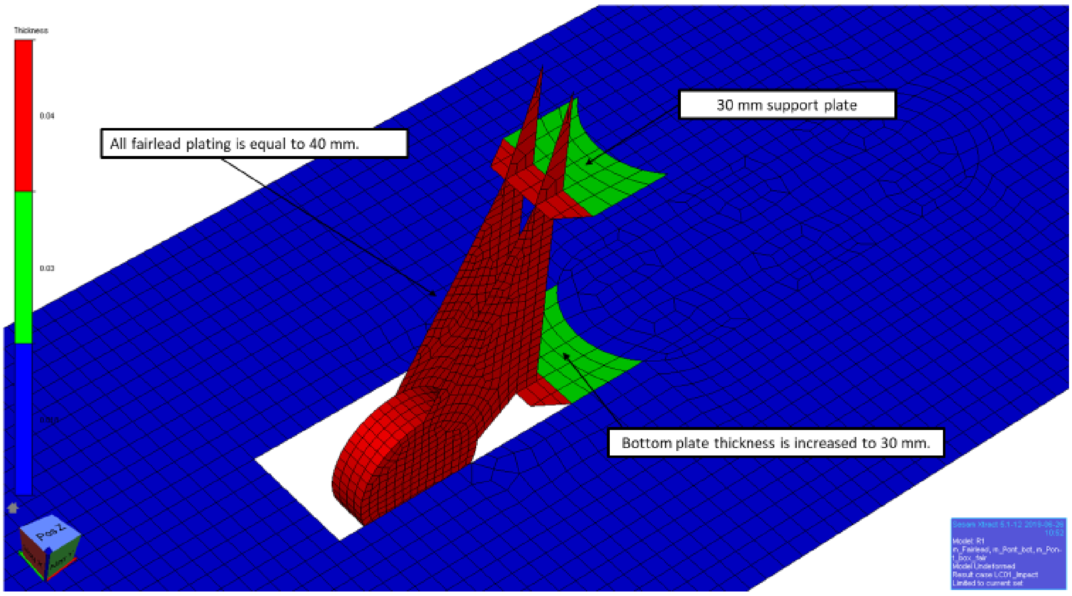
> Figure 9-4 Loading and boundary conditions, as put on in Sesam Patran Pre

The boundary condition is the same as for the original analysis. The top of the column is kept completely fixed.

9.3 Fairlead Shell Thickness and Structural Reinforcement

A 30 mm support plate behind the upper transverse fairlead plate is inserted, and the thickness of the pontoon bottom plate is increased to 30 mm in a local area behind the lower transverse fairlead plate.

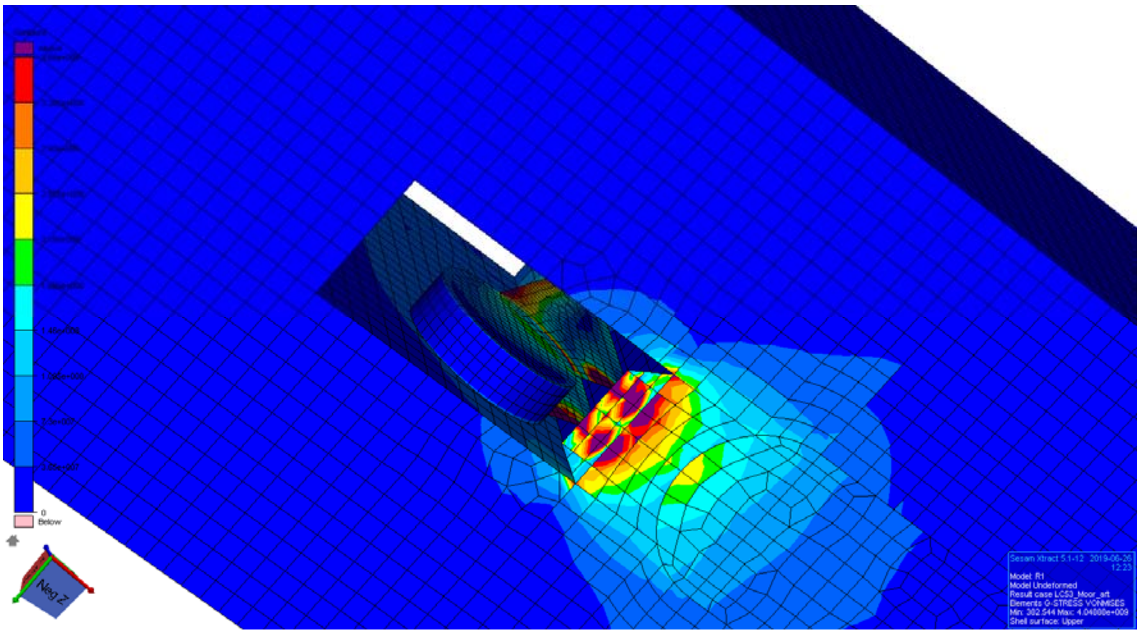
All fairlead plates have been given a thickness of 40mm.



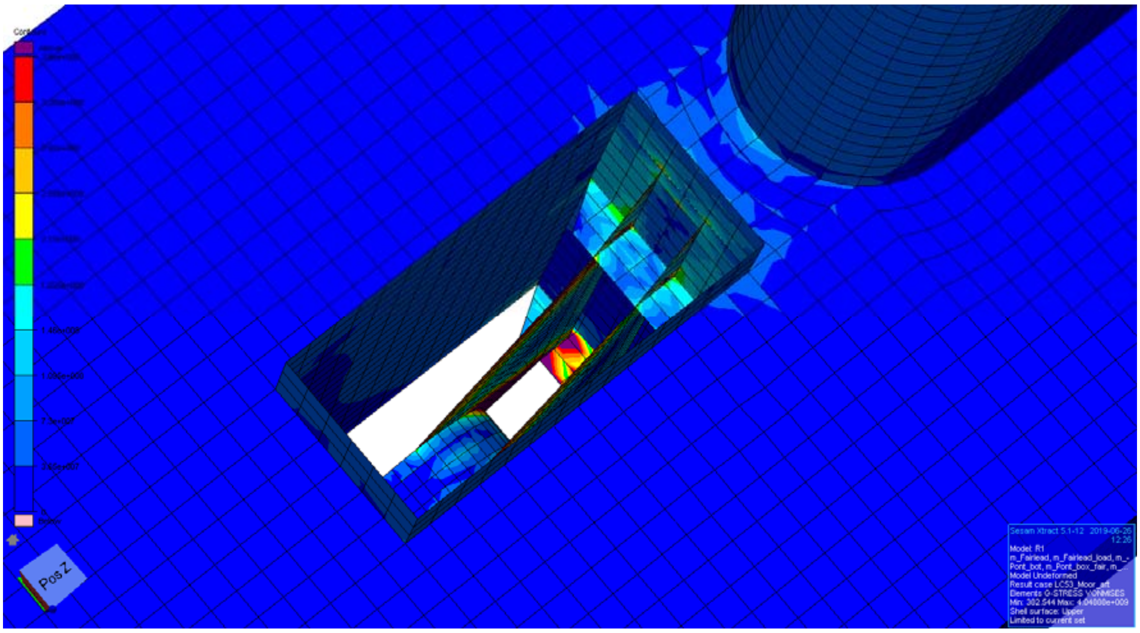
> *Figure 9-5 Thickness of fairlead plating and structural reinforcement in the fairlead intersection area*

9.4 Results

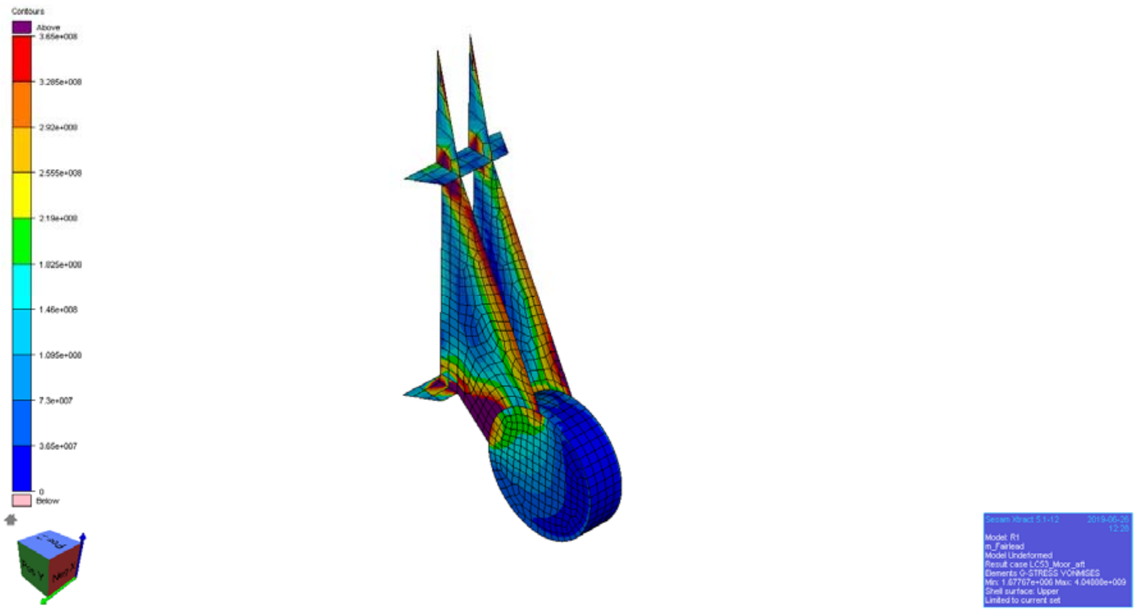
Both the fairlead itself and the surrounding pontoon structure have local linear stresses above the 365 MPa stress limit.



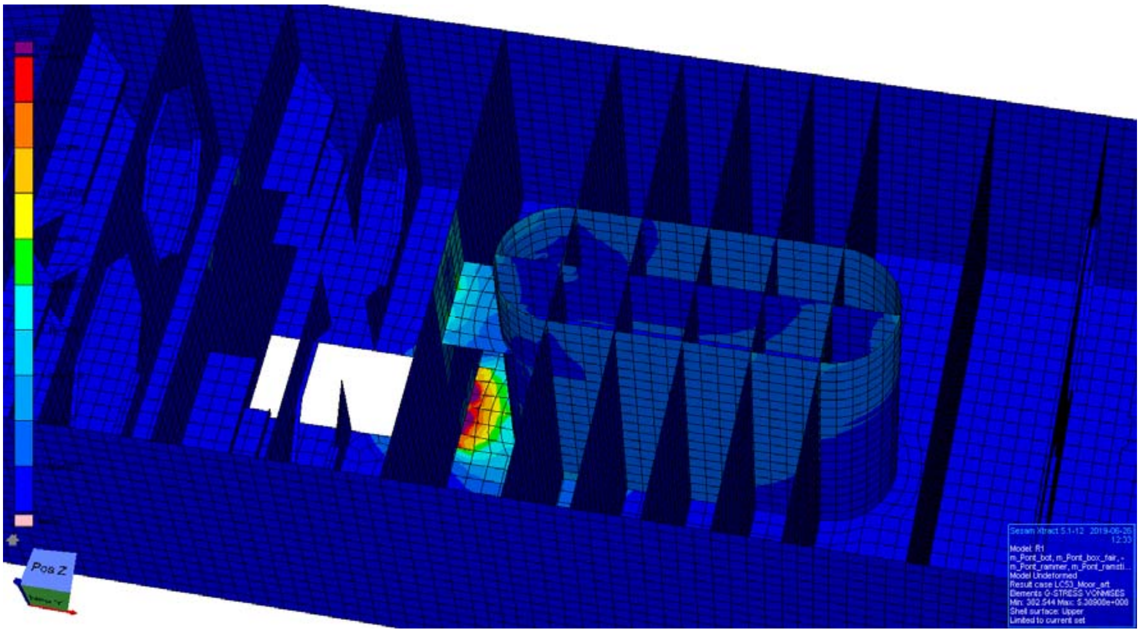
> *Figure 9-6 Finite element model shown with von Mises stresses, bottom view*



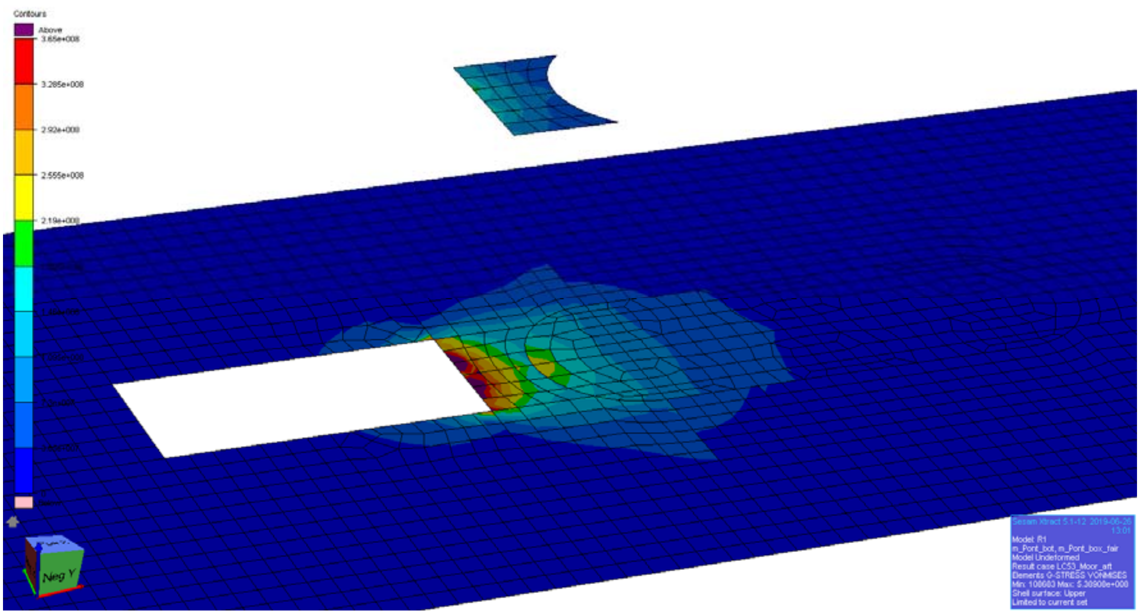
> Figure 9-7 Finite element model shown with von Mises stresses, top view



> Figure 9-8 Fairlead, von Mises stresses



> Figure 9-9 Critical parts of pontoon, von Mises stresses



> Figure 9-10 von Mises stresses in support plate and pontoon bottom

The fairlead is assumed to be designed for the anchor chain used and is not further evaluated.

When it comes to the pontoon structure, local reinforcements will be necessary in the interaction areas with the fairlead. For the ULS condition these local reinforcements are assumed possible.

Fatigue evaluations must be carried out in a later phase of the project.

10 REFERENCES

[1] DNVGL-RP-C208, Determination of structural capacity by non-linear finite element analysis methods, September 2016.

[2] DNV-RP-C103, Recommended Practice, Column-Stabilised Units, April 2012.

[3] Håndbok N400 , «Bruprosjektering,» Statens vegvesen Vegdirektoratet, 2015.

[4] SBJ-32-C4-SVV-90-BA-001, «Design Basis Bjørnafjorden floating bridges,» Statens Vegvesen, 2018.

[5] NS-EN 1993-1-1:2005+A1:2014+NA:2015, «Eurocode 3: Design of steel structures - Part 1-1: General rules and rules for buildings,» Standard Norge, 2005.

[6] SBJ-01-C4-SVV-01-BA-001., Statens Vegvesen, MetOcean Design Basis, 2018.