0	15.08.2019	Final issue	HPO	ANE	SEJ	
Rev.	Publish date	Description	Made by	Checked by	Project appro.	Client appro.

Client

Statens vegvesen

Contractor

AAS-JAKOBSEN **COWI** Multiconsult
JOHS HOLT · AkerSolutions · entail · NGI · DISSING+WEITLING architecture a/s · mossmaritime

Contract no.:

18/91094

Document name:

Preferred solution, K12 – Appendix L

Design of cable stayed bridge and abutments

Document no.:		Rev.:	Pages:
SBJ–33–C5–AMC–22–RE–112		0	244

CONCEPT DEVELOPMENT, FLOATING BRIDGE E39 BJØRNAFJORDEN

Preferred solution, K12

Appendix L – Design of cable stayed bridge and abutments

CLIENT

Statens vegvesen

DATE: / REVISION: 15.08.2019 / 0
DOCUMENT CODE: SBJ-33-C5-AMC-22-RE-112

AAS-JAKOBSEN   COWI   Multiconsult

JOHS HOLT   AkerSolutions   entail   NGI   DISSING+WEITLING architecture a|s   mossmaritime

REPORT

PROJECT	Concept development, floating bridge E39 Bjørnafjorden	DOCUMENT CODE	SBJ-33-C5-AMC-22-RE-112
SUBJECT	Appendix L – Design of cable stayed bridge and abutments – K12	ACCESSIBILITY	Restricted
CLIENT	Statens vegvesen	PROJECT MANAGER	Svein Erik Jakobsen
CONTACT	Øyvind Kongsvik Nedrebø	PREPARED BY	Henrik Polk / Arne Munz / Tore B. Aas / Torben Forsberg / Petter Buckholm
		RESPONSIBLE UNIT	AMC

0	15.08.2019	Final issue	HPO / AMU / TBA / TFO / PBU	A. Nesteby	S. E. Jakobsen
REV.	DATE	DESCRIPTION	PREPARED BY	CHECKED BY	APPROVED BY

SUMMARY

Introduction: The cable stayed bridge is constituting the south portion of the Bjørnafjorden Bridge, "cantilevering" from Reksteren across the Svarvahelleholmen Island and connecting to the floating bridge at axis 3. The tower is positioned on the island, while the 380 m main span allows the navigation to pass under at a height of 45 m and a width of 250 m or more.

Concept for cable stayed bridge and south abutment: The bridge spans from the south abutment to the tower are 40 + 4 x 55 + 140 = 400 m. This span arrangement and the corresponding cable stay configuration (380 m main span/250 m side span/18 stays/4 fans) is the result of an evaluation of feasible cable stayed bridge options that fit with the general scheme of the floating bridge.

ELEVATION
A1= 1:1500
A3=1:3000
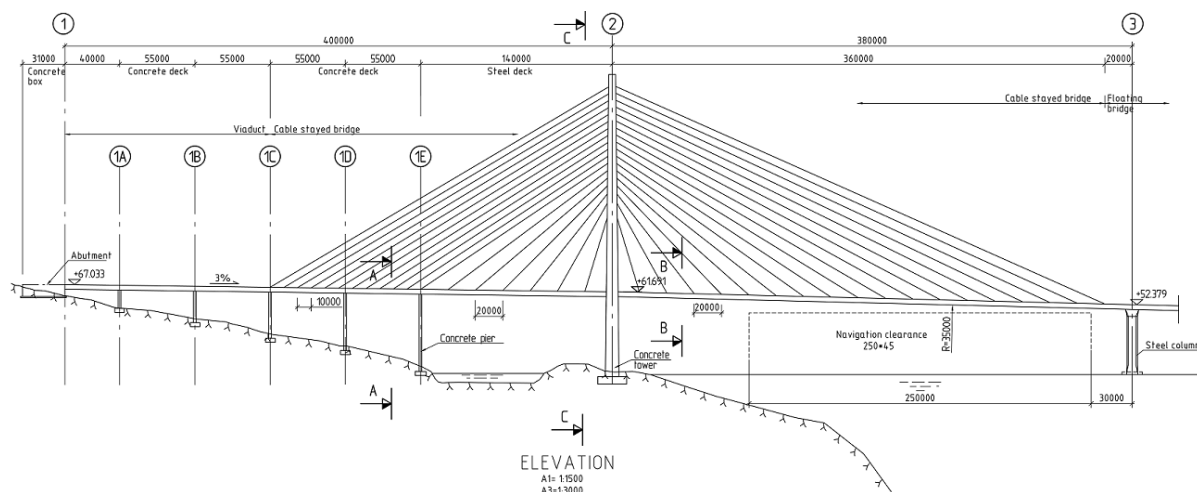
Of the 380 m long main span, 360 m belongs to the cable stayed bridge, whereas the remaining 20 m is part of the floating bridge. With the 140 m side span there is totally 500 m closed steel box bridge deck. The remainder of the bridge deck, 110m cable stayed side span and three viaducts spans, all together 260 m, is a closed concrete box structure. The tower, piers and abutment are all concrete structures. The bridge superstructure is integrated with the abutment and the piers, whereas bearings are supporting the steel deck vertically and laterally at the tower, allowing relative longitudinal movements of the deck. Thermal contraction/expansion of the viaduct spans results in flexure of the piers. There are no expansion joints in the bridge deck, neither at the abutment, at the tower nor at the intersection to the floating bridge. Axial compression forces in the bridge deck caused by stay inclinations are in balance about the tower, whereas other axial loads are transmitted through the cable stayed bridge and into the abutment. All in all, the cable stayed bridge consists of well-known standard solutions built before and therefore not adding significant risks to the project.

At the tower section of the bridge deck, the ULS strong axis bending moment becomes 2100 MNm. The ALS ship collisions to the pontoons or deck house impacts to the floating bridge deck structure lead to maximum moment of 3300 MNm. The ship collision impact and ULS load combinations lead to high lateral loads to the side span piers. The pier nearest to the tower is thus subjected to a high shear force and a consistent overturning moment.

Concept for north abutment: At the north end of the floating bridge, a huge concrete abutment ties the bridge to the Gulholmane Island. There is no expansion joint at this end either, and the structure is subjected to high axial loads in combination with very high bending moments due to transverse wind and wave loads to the floating bridge. The governing maximum ULS bending moment becomes 2400 MNm. The governing moment in the ALS ship collision situations becomes 5900 MNm. Both ship collisions to the pontoons or deck house impacts to the floating bridge deck structure are critical and the above results are based on refined ship collision analyses. To resist the ship collision forces transferred into the abutment the fill inside the abutment is iron ore.

Steel deck: The bridge deck is steel grade S420. The closed steel box is subjected to load effects comparable with those of the floating bridge. Therefore, it is natural that the design of the cable stayed steel deck follows the design concepts of the floating bridge deck of a closed steel box with a traditional orthotropic deck, circumferential skirt plates with trough/bulb stiffeners. Truss-type transverse diaphragms stabilise the longitudinal plates of the box girder, and tubes are slotted through the edges for cable stay anchorages. The standard (minimum) cross section properties will provide enough strength except at the tower region and when approaching the first floating bridge pier A3, where ULS and ALS ship collision loads require some reinforcement of the structure. The linear weight of the steel structure is in average 17.0 tonnes/m, totally 8500 tonnes.

Concrete deck: The closed 4-cell concrete deck that constitutes the three viaduct spans and the two stayed spans has ample reserves to cater for the various load effects, thus a moderate amount of posttensioning will be sufficient. The stay anchorage tubes are cast with the concrete and the passive end of the cables are sticking through the bottom.

Tower: The tower has a distinct A-shape which provides transverse strength and stiffness to the bridge and enables it to transfer very high horizontal forces to the ground. The ground quality is good, hence relatively small foundation slabs are required. The lower part of the concrete legs is very robust, and with the cross beam they constitute a very strong frame in transverse direction. With the present geometry, the frame can resist transverse load of 50 MN, while the actual load becomes 35 MN transmitted through the bearing on the side of the deck when a ship collides with the pontoon in axis 3. This load case together with the ULS load situations governs the lower part of the tower for loads in transverse direction. Wind on free standing tower in longitudinal direction governs the capacity in that direction. Otherwise, the various section verification will be straight forwards and moderate amounts of reinforcement will be required for the tower to resist the demands. The cross beam will have longitudinal posttensioning cables, and some vertical cables shall be cast in the inner flange of the lower leg for strengthening of the frame corner. The tower legs above deck level are slender and the compartment for the stay anchors are thus compact. However, they are large enough to allow for stay stressing equipment during construction. The stay anchors are fixed to internal steel boxes that work with the tower concrete structure by means of shear connectors. The steel boxes provide the necessary tie between the horizontal component of the stay force on either side of the leg, whereas the shear connectors transfer the vertical load component to the tower leg.

Cable stays: The stays are parallel multi-strand cables. The stays nearest the tower have 31 strands, and the longest stays have 67 strands. The stays are constructed by strand-by-strand tensioning. The strands are PE-sheathed individually and enclosed in HDPE-pipes. The cables have anchorages that fit with steel tubes integrated with the main structure at either end. The wire strength is 1860 MPa, and the total weight is approximately 1000 tonnes. The cable size is driven by ULS1 (permanent load dominant) and ULS2 (traffic load dominant) combinations, whereas ULS3 (wind and wave dominant) is slightly less demanding. The ship collision case is not critical. The outermost stays towards the first floating bridge pontoon have been fatigue checked without being found critical.

Piers: The piers are 10–11 m wide and 2–2.5 m thick and stand on simple rectangular foundations. The height varies in accordance with the topography, the tallest will be about 56 m high. The piers will be governed by both the ship collision effects and ULS3 combination. For the tallest pier in axis 1-E, wind on free standing pier is critical for the size of the foundation and for the reinforcement in bottom of the pier.

South abutment: With piers integrated with the bridge deck, the south abutment is not subjected to high forces from ship collision, as was the case for the previous phase concepts. The abutment can therefore be designed for the longitudinal axial force in the bridge deck. The force requires a gravity construction that can resist the longitudinal force by means of friction. The abutment is a cell concrete structure 30 m long and 28 m wide and a height of 9.5 m filled with gravel.

North abutment: The north abutment is highly determined by the large horizontal bending moments from ship impact (bridge girder strong axis moment). Similar high moments occur also for normal wind and wave loads, but not quite as severe. It has been found necessary to design a concrete cell structure which is large enough to contain enough ballast to prevent rotation by means of friction between the base and ground. The north abutment becomes 50 m long and 36 m wide and 10.0 m high. With the present high ship collision loads it is found beneficial to use iron ore as ballast material within the abutment. The bridge deck adjacent to the north abutment is widened and has increased strength compared with normal sections of the floating bridge. The joint between the abutment concrete structure and the steel deck is secured with longitudinal posttensioning cables.

Construction works: Some of the construction works for the cable stayed bridge is done in prefabrication yards, but most of the work is carried out at site. The cable stays, steel anchor boxes and deck sections are prefab elements that are brought to the construction site for installation. The in-situ works begins with establishment of the work site and access roads. The tower construction will be on the critical time path, so the works must start early by blasting and preparation for the in-situ concreting of the foundation slabs. After casting of slabs, the lower legs can be constructed by use of jump forms. Due to the inward inclination of legs, temporary propping will be required to prevent high bending moments. The abutment and piers are cast in-situ concrete works, whereas the concrete spans may be constructed either by means of in-situ casting on a moving scaffolding, or by span-by-span launching from the abutment. Upon arrival from the prefab yard, the steel deck elements lifted into place and attached to the stays at alternate side of the tower hoisted by first floating crane followed by deck mounted derricks, while balancing the horizontal forces at the tower. The construction of the two abutments begins with normal ground preparation works, followed by concrete wall construction and ballast filling works. For the north abutment a temporary cofferdam may be required at one end. The construction work is described in more details in Appendix N.

Alternative bridge deck layout: The selected concept doesn't necessarily show the fully optimal solution for the cable stayed bridge. An alternative concept has therefore been considered if longer side- and viaduct spans will be feasible utilising the natural strength of the 3.5 m deep concrete deck box structure, thereby save one pier and improve architecture with a shorter side span in combination with an equal stay spacing. The concrete deck might also be extended to the tower and further say 40 m into the main span with cost saving as a result. This option might be possible as the ALS ship collision forces have stabilised at a moment of approximately 3300 MNm, which the concrete box structure can easily resist. The development of the cable stayed bridge is presented below as "alternative layout".

ELEVATION
A1= 1:1500
A3=1:3000

TABLE OF CONTENTS

1   Cable stayed bridge

1.1   Concept layout - general arrangement

The cable stayed bridge is constituting the south portion of the Bjørnafjorden Bridge, "cantilevering" from Reksteren across the Svarvahelleholmen Island and connecting to the floating bridge at axis 3. The tower is positioned on the island, while the 380 m main span allows the navigation to pass under at a height of 45 m and a width of 250 m and even some 20 m more. The bridge spans from the south abutment to the tower are 40 + 4 x 55 + 140 = 400 m. This span arrangement and the corresponding cable stay configuration (380 m main span/250 m side span/18 stays/4 fans) is the result of an evaluation of feasible cable stayed bridge options that would fit with the general scheme of the floating bridge, see below Figure 1-1. This is the concept selected and incorporated in the overall floating bridge analyses, belonging FE-modelling, drawings, bill of quantities etc.
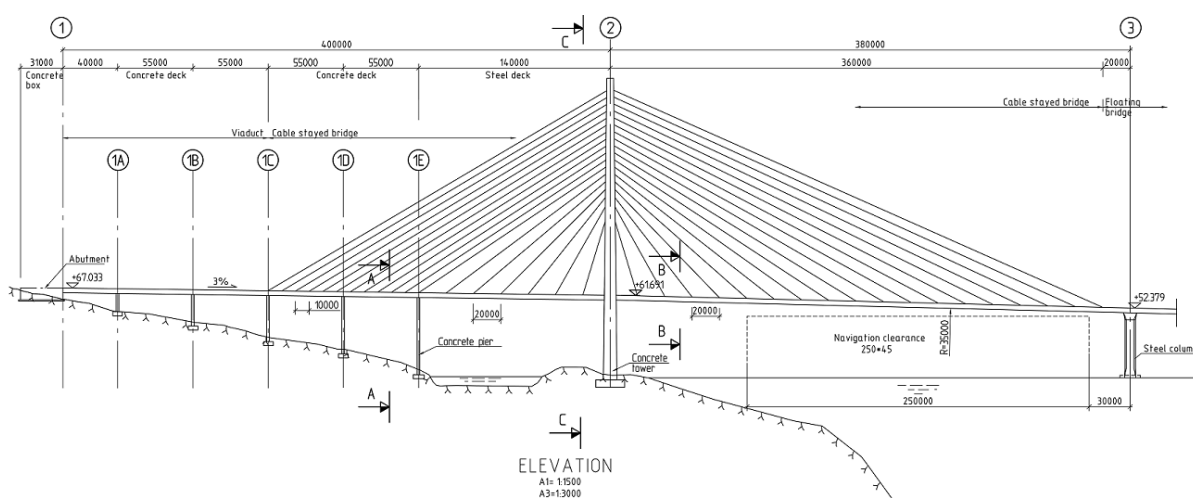

Figure 1-1 Cable stayed bridge – concept layout

The width of the bridge deck basically follows the shape of the floating bridge but is widened to give space for the cable stay anchorages. The stays anchorages are 28.0 m apart with a spacing along the bridge girder of 20.0 m for steel respectively 10.0 m for concrete. Along the tower the vertical spacing between the anchorages is chosen to 5.0 m.

The stayed span lengths lead to an asymmetrical bridge which becomes natural as piers can be erected on shore in the side span supporting the deck girder and moreover giving stiffness to the entire cable stayed bridge. The bridge deck is in principle divided into a steel portion above water, and concrete above land continuing with the concrete viaduct spans.

The cable stayed bridge is of a fully conventional type, and the most significant influence from the floating bridge is in general tidal variation and to some extent wave motion of pontoon at axis 3. These none common loading events leading to dynamic stay forces and bending of the steel deck have been examined carefully without giving significant impact to the design. The distance from the last stay cable to the first pontoon of the floating bridge is selected to 20 m but a parameter which can be furthermore fine-tuned later. The most severe impact from the floating bridge is for sure the ship collision to the bridge deck and the first pontoon of the floating bridge, which however is fully common design situations for cable stayed bridges.

The bridge deck and tower have a tight fit in transverse direction where the deck is supported on normal sliding bearings to the tower legs. The tower is a strong point being able to accommodate the

transverse loading from ship collision and normal ULS load situations, see furthermore the tower description in section 1.4. In vertical direction the deck is supported by normal bridge bearings on the tower cross beam and in the longitudinal direction the deck is free to move. In general, the aim has been to avoid bridge bearings and expansion joints and it has therefore been investigated if the bridge deck and tower structure could be monolithically connected. This showed however up not being feasible as temperature loading within the 400 m long concrete deck (side span and viaduct) will govern the tower design.

However, the viaduct is monolithic with the abutment and piers and further integrated with the concrete bridge deck. For ship collision the piers are therefore designed to accommodate the high transverse loads.

The entire cable stayed bridge has a straight horizontal alignment and a visualisation of the bridge can be seen below in Figure 1-2.
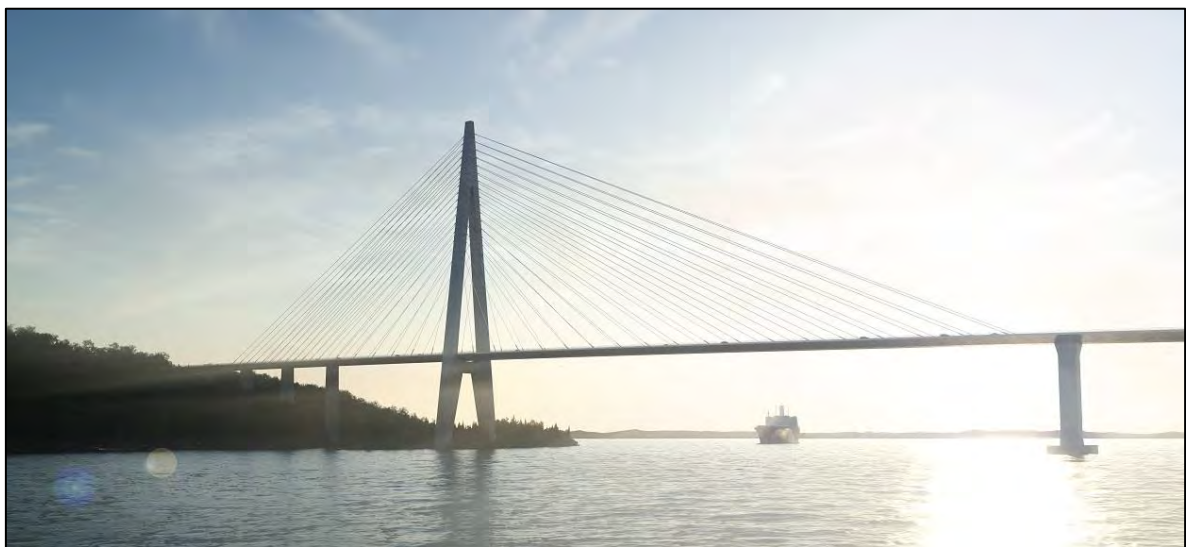

Figure 1-2 Cable stayed bridge - visualisation

1.2 Alternative layout of bridge deck

The above concept is not necessarily a fully optimised solution in all aspects and it does not fully reflect all architectural preferences. An alternative concept is therefore presented in Figure 1-3.
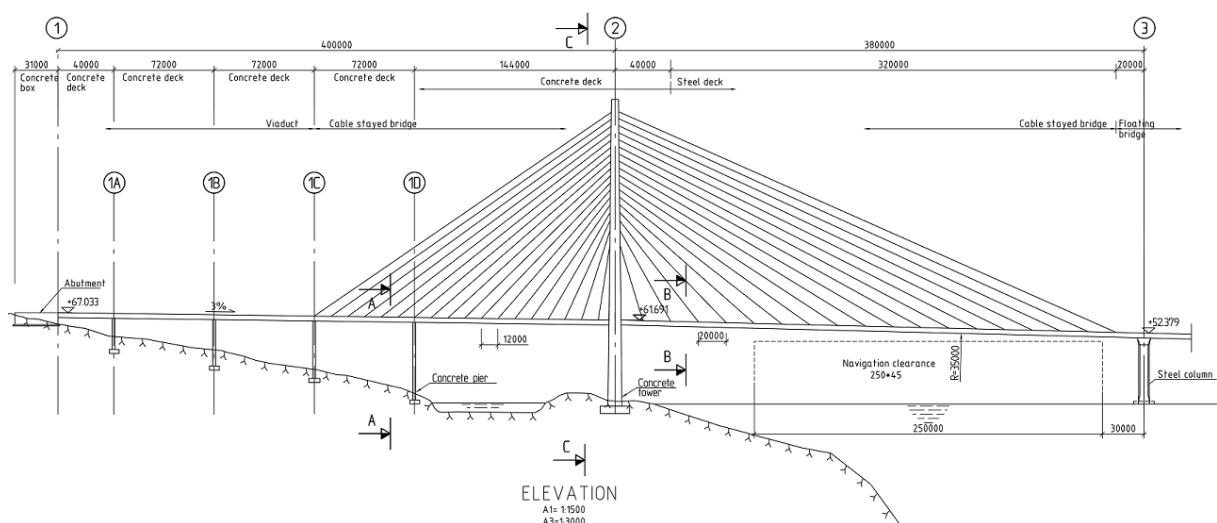

Figure 1-3 Cable stayed bridge – alternative layout of bridge deck

Here it has been considered if longer side- and viaduct spans up to 72 m might be feasible utilising the fully natural strength of the 3.5 m deep concrete deck box structure and thereby save one pier and give an architectural improvement to the project. The concrete box girder is furthermore extended to the entire side span and further say 40 m into the main span with the aim reducing the cost and improve the bridge appearance of having an equally stay spacing throughout the entire side span. The alternative layout perhaps becomes even more feasible now as the ALS ship collision forces at tower section tending to stabilise at a maximum bending moment of 3300 MNm.

1.3   Analyses

The analyses supporting the cable stayed bridge (and abutment) design are combinations from different analyses:

- global, dynamic wind and wave analyses of floating bridge (ORCAFLEX)

- global dynamic analyses of ship collision to floating bridge (LS-DYNA)

- permanent load, temperature and traffic (RM Bridge)

- a local dynamic wind analysis of critical construction stages for the cable stayed bridge (Novaframe analysis described in this appendix)

- local cable stayed bridge (SOFISTIK, described in this appendix)

The various results are found accurate enough to justify the technical feasibility and the quantities of the cable stayed bridge and abutments.

The design of the cable stayed bridge will predominantly be controlled by ULS (both construction stages and permanent stage) and ALS ship collision demands. However, SLS might also governs the design of some tower- and pier elements. At this stage, mainly the ULS and ALS cases have been verified, which for the ULS combinations are:

- ULS 1 permanent load dominant

- ULS 2 traffic load dominant

- ULS 3 environmental load dominant

- ULS wind analysis of different construction stages

1.4 Tower

The tower has a distinct A-shape that is architecturally preferred, and which is optimal seen in relation to the demanding ship collision loads being able to accommodate large forces in the transverse direction.

The tower is a concrete structure 220 m tall and A-shaped, meaning it has a high transverse stiffness and strength. The tower will also act strong for uneven loading of the cable planes in traffic and ship collision situations due to the interconnection of the legs at the top. The lower legs and cross beams are strong and forms a frame able to transfer large horizontal loads from ship impact to the ground. The form is technically sound and aligns well with the architect's preference, see Figure 1-4.

The lower legs taper towards the foundation, where the bending moments therefore become small, hence the leg sections need to be large where they meet the cross beams. Due to esthetical reason the tower foundations are selected fully buried into the ground.

The analyses show that ship collisions and ULS tower demands are almost equally important.

The concrete volumes at the cross beam connection to the leg are quite substantial and will occupy some space inside the tower. The frame connection requires transverse posttension from outside leg to outside leg through the cross beam, and likely also vertical posttensioning some depth down the inner flange of the lower leg.

Above the bridge deck, the tower legs are slender due to aesthetic reasons meaning that the compartment becomes compact. Moreover, the anchorages have been placed asymmetric in the legs making it possible to position all the stay anchorages along a straight line meeting each other at the tower top. At the location of steel boxes for the cable stay anchorages, there is only limited space for a stair to pass on one side of the asymmetrical placed stay anchorages, see
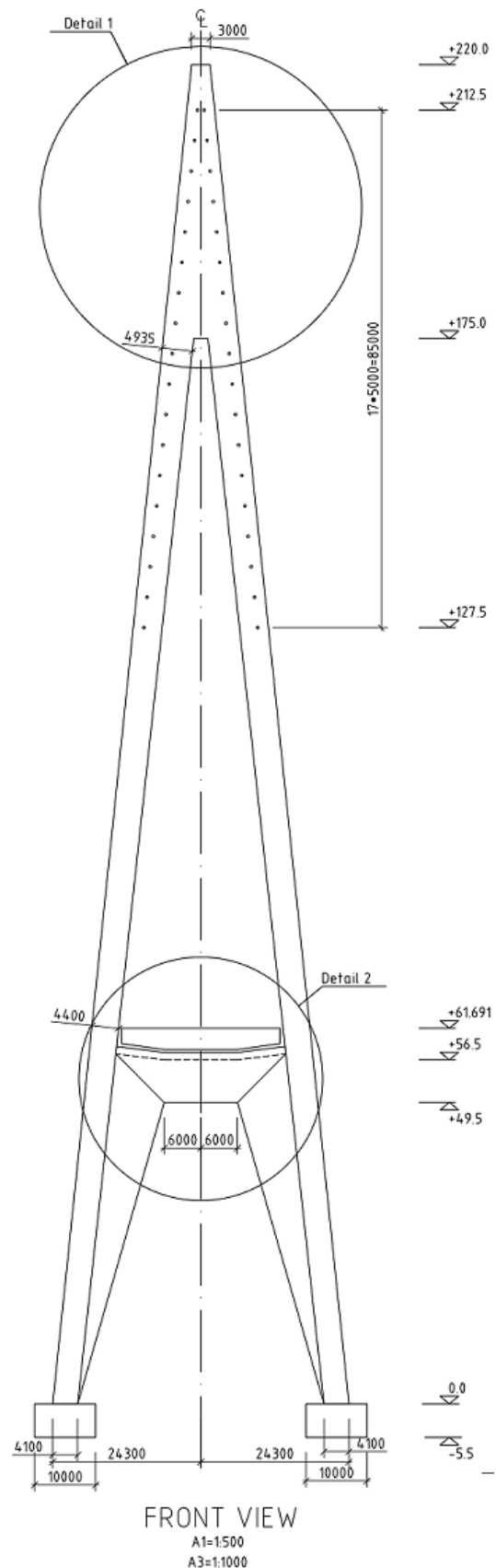Figure 1-5.

Figure 1-4 Tower – front view

Due to esthetical reason the tower stay anchorages are spaced every 5.0 m, which is more than normal, i.e. it will be easy to adopt the cable stay forces in the steel anchor structure and transfer it to the concrete by means of shear connectors. This will furthermore make the stay tensioning within the tower easier.
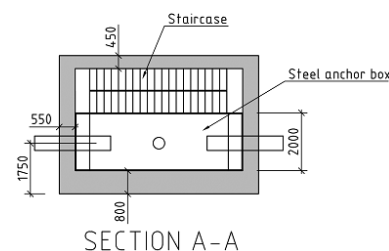

Figure 1-5 Tower – plan section through leg at stay anchorages

At the tower top, the two legs are merged and taper further towards the top. Here the space conditions will be constrained to keep the slender impression. Normal concrete strength C40 – C50 will suffice, and the reinforcement quantity will be normal, 150 – 200 kg/m³.

The concrete tower structure of the cable stayed bridge can be seen to be well documented standard products built before and therefore not adding significant risks to the project.

A visualisation of the cable stayed bridge can be seen in Figure 1-6 where the tower forms almost a lighthouse landmark.

Figure 1-6 Cable stayed bridge with the tower forming a light house landmark

1.5   Bridge deck

1.5.1   Bridge deck – steel

For the selected concept, the steel bridge deck starts 140 m from the tower into the side span and continues throughout the entire main span of 380 m until 20 m from the first pontoon pier A3, see also Figure 1-1. The length of the steel deck then becomes 500 m with an average total girder weight of 17.0 t/m. The steel deck is much lighter than the concrete deck and therefore a natural choice for the main span. In the side span the steel deck has been chosen going 140 m into the span due to high bending moments round strong axis from ship impact in the tower region and furthermore to span over the water to the first pier A1E.

An alternative layout of the bridge deck has already been mentioned and described briefly in section 1.2 considering extending the concrete girder to the entire length of the side span in the aim of optimising the design further. This option has become possible as the ship collision forces have decreased reaching a maximum level of 3300 MNm round strong axis at tower position.

A cross section of the bridge girder can be seen in Figure 1-7 where the steel girder is optimised for the cable stayed bridge having a reduced girder depth of 3.5 m instead of 4.0 m as for the floating bridge. The cross section is more aerodynamically shaped with wind noses and wider to accommodate the cable anchorages. A wider deck is also beneficial withstanding the large moments round strong axis from ALS ship collision as well as normal ULS. The outer vertical web plate from the floating bridge spaced 27.0 m can be found unchanged in the cross section layout for the cable stay bridge, now supporting the stay anchorages.
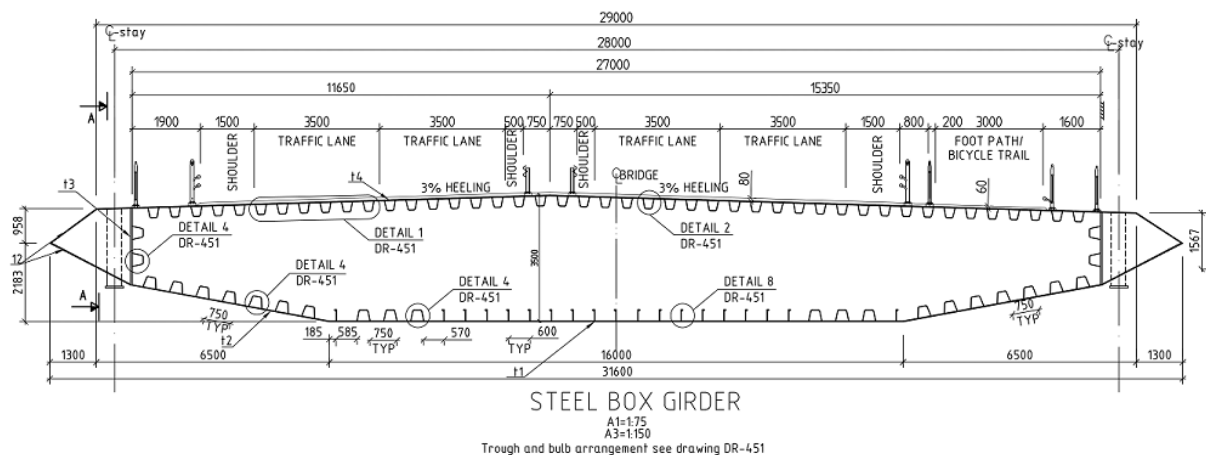

Figure 1-7 Steel bridge girder – cross section, longitudinal steel

The deck plate varies from a minimum thickness of 16 mm up to 20 mm when approaching the tower from both side- and main span and again when approaching the first floating bridge pier 3. Fatigue calculations carried out documents that a deck plate of min 16 mm will be necessary instead of the more conventional 14 mm due to truck loading. The trapezoidal stiffeners underneath the deck plate vary from minimum 8 to 10 mm, again to accommodate the fatigue requirements.

The bottom plate and the inclined web plates varies from a minimum thickness of 12 mm up to 20 mm near the tower and again when approaching the first floating bridge pier 3. For the bottom plate the principles using bulb stiffeners are chosen due to preference of the floating bridge.

The cross section layout is further described in section 3.3.1. and Figure 1-8 shows the layout of the transverse truss diaphragm with a spacing in the bridge line of 4.0 m. The transverse truss solution has been selected as it in general is lighter than a full plated diaphragm solution.
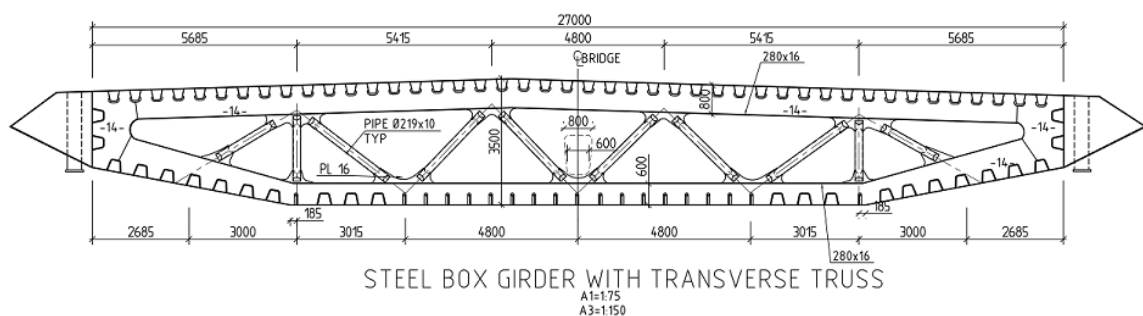

Figure 1-8 Steel bridge girder – layout of transverse truss diaphragm spaced 4.0 m

The deck constitutes an important part of the cable stayed bridge which is a well-documented standard product not adding a significant risk to the project. The steel girder quantity becomes 8600 t equal to an average total girder weight of 17.0 t/m. The steel bridge deck constitutes 40-45% of the total bridge cost.

1.5.2 Bridge deck – concrete

The concrete bridge deck is fully fixed in the abutment and to the steel deck. The total length for the concrete deck is 260 m. In the distance of 110 m closest to the tower, stay cables are anchored in the cross section. The distance between the stay cable planes is chosen to be 28.0 m. The width of the cross section is therefore selected to 29.5 m giving sufficient space for the stay cable anchorages.

It is possible to reduce the cross section width between the abutment and the cable stayed bridge. However, this will also be an aesthetically evaluation.

The cross section height is 3.5 m, somewhat optimal for a span length of 55 m and upwards. The thickness of the bottom slab is 280 mm while the top slab is 300 mm and web thicknesses are 450 mm. Diaphragms are used at each pier and at each stay cable anchor. The diaphragm thickness is 3000 mm at piers and 500 mm at stay cable anchorages. 24 prestressing tendons with breaking load 5300 kN are used above the piers and 12 prestressing tendons within the field. Normal reinforcement amount of 160 kg/m^3 concrete is used. A cross section of the box girder can be seen in Figure 1-9.

A wide box section type is chosen due to the following reasons:

- The cross section shape is similar to the cross section layout within the main span

- Forces from the stay cables can easily be incorporated in the cross section

- The bridge deck is subjected to high bending moment about strong axis from ship collision which becomes favourable having large concrete areas close to the outer edges of the cross section

- Small reinforcement amount is needed in the transverse direction due to the cross section shape

Construction of the bridge girder for the selected concept can e.g. be done by:

- Use of MSS (movable scaffolding system, spanning from pier to pier) for concrete cross section typical max 70-75 m between piers/temporary supports

- Incremental launching (ILM) with typical max 50-55 m between piers/temporary supports

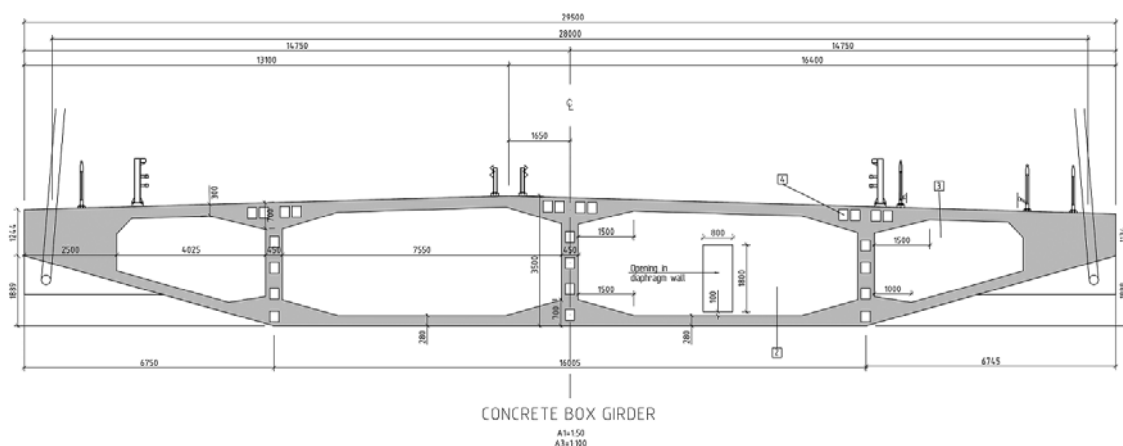- Concrete cast in place with scaffolding supported on ground

Figure 1-9 Concrete box girder - cross section

The concrete girder of the cable stayed bridge can be seen to consist of well documented standard products not adding significant risks to the project.

1.5.3   Bridge girder joint

The joint between the steel bridge girder and the concrete girder is located 135 m from the tower near the first side span pier. Here the strong axis bending moment due to ship collision is reduced to 50% of the maximum moment at the tower or less and the connection by means of post-tensioning cables and high-strength bars will then be less demanding.

The joint will be based on the same principles as used for the Pont de Normandie (France). The posttensioning cables will be anchored a certain distance from the joint inside the steel box and be cast in the concrete deck. Along the circumference of the steel box a number of high strength bars will be installed at tight spacings.

The connection could be established in two steps. First the steel box and the concrete deck is brought near to each other and partially stressed. In this locked situation a stich joint concrete (grout) is cast ensuring the complete connection of the two bridge deck types. After curing of the stich joint, the cables and bars are fully tensioned.

The bridge girder joint of the cable stayed bridge is not a standard solution within cable stayed bridges but is however well-known technology from other large cable stayed bridges as the Pont de Normandie in France and the Russky Bridge in Russia, both well known within the Joint Venture. The joint will therefore not add any significant risks to the project.

1.6   Cable stays

The cable stays of the bridge are multi-strand type with 31–67 no of strands, each 15.7 mm dia. having an ultimate breaking strength of 1860 MPa. The 31 strand cable thus has a breaking strength of 8.6 MN, and the 67 strand cable 18.6 MN.

The stays are arranged in 4 fans, each having 18 cables, the shortest nearest to the tower are subjected to the lowest loads, whereas the flattest outermost cables have the highest loads. The cables outbalance each other horizontally at the tower anchorage, however minor deviations are acceptable, and forces may be slightly redistributed. Hence, the cables can be grouped and totally 6 different cable sizes are presented here.

The cable stays stressing ends are anchored in the steel boxes in the tower legs making the tensioning process easy, whereas their passive ends are anchored at the bottom end of steel tubes sticking trough the verges of the bridge deck.

The cable sizes are determined by the maximum ULS load that may occur. ULS1 (permanent load dominant), the ULS2 (traffic load dominant) and ULS3 (wind/wave dominant) lead almost to the same loads, whereas ship collision not becomes critical. All load effects are comparable with those of other cable stayed bridges, except wave load effects to the longest stays nearest to axis 3. In appendix I also the cable capacity for fatigue is checked and found to be very high. The main contributors to fatigue are traffic and tide. Traffic contributes most to fatigue near the tower while tide contributes most to fatigue in the northern bridge end in vicinity of the first floating bridge pontoon in axis 3.

The stays are documented by a mix øf analyses results. New results from global floating bridge analyses (Orcaflex/LS-DYNA) and RM-Bridge have been combined, and a separate SOFISTIK model of the cable stayed bridge has been used for comparison and check.

The load effects in each cable are tabulated in the stay cable drawing. Further documentation is enclosed in section 3.5.

The longest stays will require a damping system to be installed as also indicated on the drawings. Additional damping might be necessary for the longest stays if snow is expected to accumulate on the stays. For reference, the Øresund Bridge has additional damping installed safeguarding against stay oscillations due to possible snow accumulation. The above topic is well known by the Joint Venture.

The stays constitute an important part of the cable stayed bridge, however, they are well-documented standard products that are not supposed to add a significant risk to the project. The total quantity is 1000 tonnes approximately and constitutes about 15 % of the total cable stayed bridge cost. The construction is standard strand-by-strand installation.

1.7 Side span piers

The concrete piers are rectangular, 10–11 m wide and 2.0–2.5 m thick. The pier foundations are also rectangular with the below shown sizes all buried into the ground. The height varies in accordance with the terrain, and the tallest pier is about 56 m. The piers will be governed by the ship collision effects (ALS) and forces from the load combination ULS3. For the longest pier 1-E the load situation with wind on free standing pier is slightly governing for the foundation size and the reinforcement in the bottom of the pier. Since the heights are much shorter for the other piers (1-D is approximately 42 m high), the construction stages seem not to be governing for them.

The pier foundations will be cast directly on rock or blasted rock with a concrete scaling. All the foundations are placed above water but buried into the ground due to esthetical reason. The piers can be cast e.g. with jump- or slip form.

The foundation sizes:

1-A, 1-B, 1-C   WidthxDepthxHeight: 7x15x3 m

1-D   WidthxDepthxHeight: 7x18x3 m

1-E   WidthxDepthxHeight: 8x18x3 m

1.8 Ship collision

Ship collision is a governing load situation and an important factor for the cable stayed bridge design. The K11 alternative in the previous phase had the bridge deck and tower disintegrated transversely, with the purpose to distribute the elastic response as wide as possible. For this phase, a tight fit between bridge deck and tower has been selected. Thereby, the ship collision effect is more direct leading to high loads, but as the tower have such high strength, it becomes an appropriate design methodology.

The A-tower has a very robust lower frame that initially designed resisting a load of 50 MN, but actual load is found to be 35 MN. This force between the horizontal bearing on the side of the bridge deck and the inner side of the tower is slightly affected by the stiffness of the side-span piers.

The A-shape and wide distance between the tower leg foundations makes it very stable against ship collision, and the tower as such is far from being overloaded or overturned due to ship impact.

The strong axis bending moment of the bridge deck at the tower is 3300 MNm and is the most dominant ship collision effect. The effect to all the structures can more or less relate to this high moment. With the rigid approach, the south abutment is no longer subjected to severe moments as in the previous phase of the project.

For ship collision to the pontoon axis 3 the bridge deck also experiences large bending moments at this end (and further into the floating bridge).

Torsion and shear effects are small, however, the shear between the tower and the nearest on-shore pier is high.

Ship collision is not critical for the stays and the tower legs above bridge deck.

1.9 Tidal variation of axis 3 support

The nearness of the axis 3 support subjected to vertical tidal movements has been a concern. The pontoon stiffness is however much higher than the cable stiffness, hence the cable stayed bridge will follow the deformations of the pontoon, basically.

However, the effect in terms of weak axis moments in the bridge deck, and additional cable loads for the outermost flattest cables, have shown to be only moderate. The cable forces will be rather immune for such design changes. This shows that the proposed system with 20 m between the no 18 stay anchorage and axis 3, and the inclination angle of 22^0 seems a feasible and appropriate solution.

Even an extension of the distance between the outermost cable and the first pontoon support is not foreseen becoming a challenge. It has been evaluated to increase the distance by further 20 m without any notable impact to the design.

The outermost stay has furthermore been examined for fatigue due to tidal and wave motions showing sufficient capacity, please refer to Appendix I.

1.10 Construction

The construction of the cable stayed bridge contain working processes that are well known and straight forward. The bridge is situated on-shore except that a temporary bridge or embankment fill is required to provide access to Svarvahelleholmen. Due to limited space on the island, probably floating barges around the tower foot will be required for concrete works.

The works start with access roads and ground works for the south abutment, tower foundation and side span piers, with access roads between these sites. After preparation of the ground, the foundations can be casted. Jump formwork is then used for construction of the lower tower legs and piers. Temporary strutting between the tower legs is required due to their inclination.

After completion of piers, span-by-span construction of the concrete side spans can begin. An overhung or underhung moving scaffolding system can be used, and the scaffolding system is shifted along the bridge span by span, with in situ concrete works. The bridge deck is cast monolithic to the pier tops (no bearings). The bridge structure is longitudinally post-tensioned as necessary for the construction stage. Full tensioning can await completion of the concrete works.

To reduce thermal constraint, it may be necessary to cast the stich joint connection to the abutment at a late stage after creep and shrinkage effects, and at neutral temperature.

The tower cross beam is cast in-situ supported by temporary scaffolding (truss) spanning between the legs. Hereafter the tower legs are cast by climbing formwork, alternatively by slip forming. Steel boxes for the stay anchorages are placed one by one while the concrete progress towards the top. Where the two legs join at level +175, the jump-form is shifted to a full-width form, used to the very top of the tower at +220.

Once constructed, the steel bridge "tower segment" is shifted in between the tower legs from the main span side. Then balanced erection of steel deck segments, 20 m long, is carried out. The water

between the Svarvahelleholmen and Reksteren allows the side-span segments to be lifted off the barges also to this side, i.e. no steel element need to be transported on the ground.

When a steel segment has been lifted by the derrick crane and the joint is partly welded, the stays are installed. The cable is formed by individual strands that are pulled from the top and anchored at the bottom. When the stays are complete, and the steel segment joint is welded, the derrick crane can be moved further out ready for the next section. The segment installation continues until the segment towards the concrete spans is in place. The joint will be with in-situ stich concrete or grout, followed by tensioning of the longitudinal cables and bolts going through the joint.

The cycle of erecting steel segments is estimated to be in the range of 10-14 calendar days. It can however be done in parallel both sides for the first 2*7 segments.

The above erection schedule implies that the cable stayed bridge shall stay the winter fully erected without connection to the floating bridge. This situation is analysed and found not to be critical.

The bridge finishing works, pavement, barriers, bearings etc. are finally installed. A more detailed description of the different construction steps is given in appendix N Construction and marine operations.

2 Abutments

2.1 General arrangement

The bridge is without expansion joints, and the bridge girder is fixed to the abutments both in south and north. This solution induces large forces to the abutments, and especially in north where the abutment was found to need about 2000 m² area, 52 m x 38 m, and a height of about 10 m to contain the necessary amount of ballast to achieve a weight of 60.000 ton. The weight/area is based on stability by weight only and a coefficient of friction 1.0 for concrete poured on cleaned rock surface.

In north the steel girder for the floating bridge is widened from 27 m to 36 m to suit the abutment, and large prestressing tendons are used in the steel/concrete connection.

In south the concrete approach bridge continues as the upper part of the abutment and is casted monolithically to the walls.

2.2 Analyses

The governing loading on the abutments is bending moment about the vertical axis, coming from environmental loads or ship impact. Below is summarised the max/min loading N is axial load in the bridge, positive sign is tension. Mz moment about the vertical axis, and My moment about the horizontal axis, positive sign is tension upper face. Units MN and MNm.

Table 2-1 Abutment north and south – design loads for ULS and ship collision

	Nmax	Nmin	Mzmax	Mzmin	Mymax	Mymin
Ship imp. north	74	-77	**5876**	-5838	341	152
ULS 3 north	63	-55	**2439**	-2680	1196	-370
Ship imp. south	74	-75	606	-770	17	-15
ULS 3 south	69	-83	24	-24	128	71

Torsional moment and shear forces are not shown as they have a minor influence on the design.

2.3 North abutment

At the north abutment the road level is at elevation 11.6 m, and the area is chosen such that the entire abutment is above the water level. Ship impact governs the size of the abutment. With an abutment size of 50 m x 36 m, footprint 52 m x 38 m, the necessary height with iron ore ballast is around 10.0 m. Rock anchors can decrease the area or height but is not used at this stage.

A reduction of the ship impact forces will reduce the footprint/height, or olivine/gravel can be used as ballast instead.

The abutment is a large box structure with longitudinal walls lining up with the longitudinal bulkheads in the steel floating bridge. 150 nos. longitudinal prestressing tendons secure the connection to the bridge.
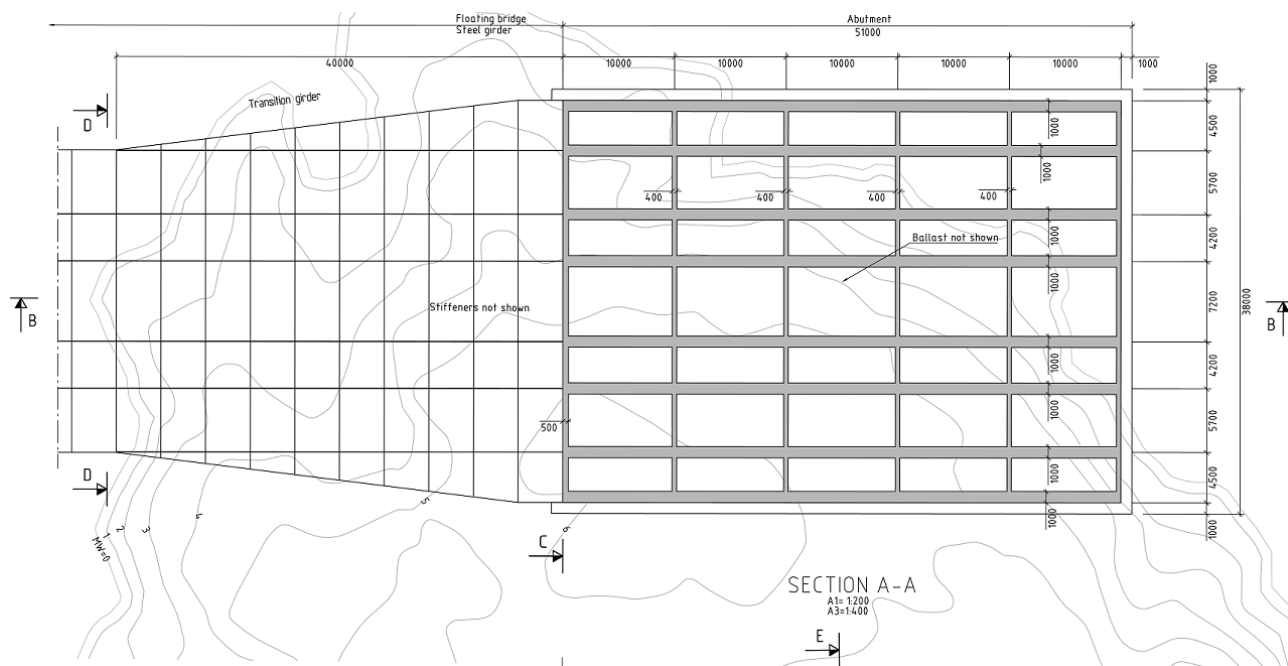
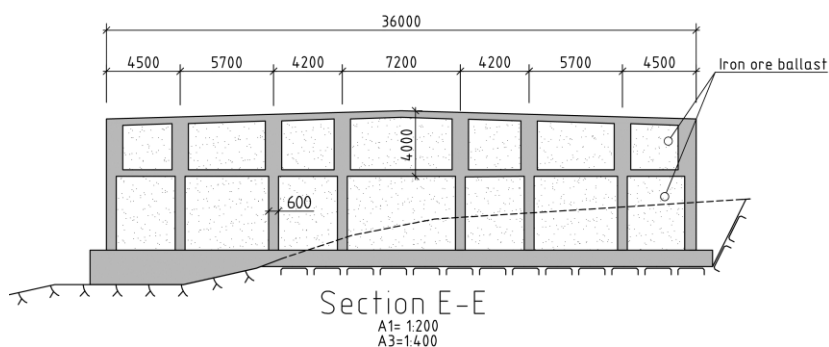*Figure 2-1 North abutment - horizontal section*

Figure 2-2 North abutment - transverse section

The position of the abutment can be shifted somewhat towards south and a shortening of the floating bridge can be achieved. Also, the fill concrete shown to the left can be avoided.

Below is shown an illustration of the northern part of the floating bridge ending at the north abutment and the transition into the tunnel.

Figure 2-3 Illustration of the northern part of the floating bridge, abutment and tunnel entrance

2.4 Bridge deck joint

For the north abutment the strong axis bending moment is very high. The joint shall transfer these high moments, combined with other load effects, for example axial tension. The solution is to prestress the joint by large cables with a force large enough to avoid tension in the joint in the SLS limit state. Standard cables for bridges and offshore will be used. It appears that the SLS limit state is governing for the number/size of cables. These cables shall be grouted, alternatively unbonded with the possibility of later replacement.

It is proposed to place one cable inside between each longitudinal stiffener in the steel girder, i.e. distance between 600 mm, all around the perimeter and along the webs. That means a total number of 150 cables are installed. A typical cable will have a breaking load of 5300 kN, (19 strands).

The steel box girder will end in a special cross frame, transition module, where the forces from the cables can be transferred to the longitudinal plates/stiffeners in the steel girder.

2.5 Approach bridge

Continuation north of the north abutment via an approach bridge has not been dealt with in this phase of the project.

2.6 South abutment

The south abutment needs a length of 30 m with a width equal the bridge width of 28 m. 9.5 m height is chosen as it fits well with the terrain and gravel ballast can be used. On this side the abutment is connected to a concrete bridge to which it will have a monolithic connection.
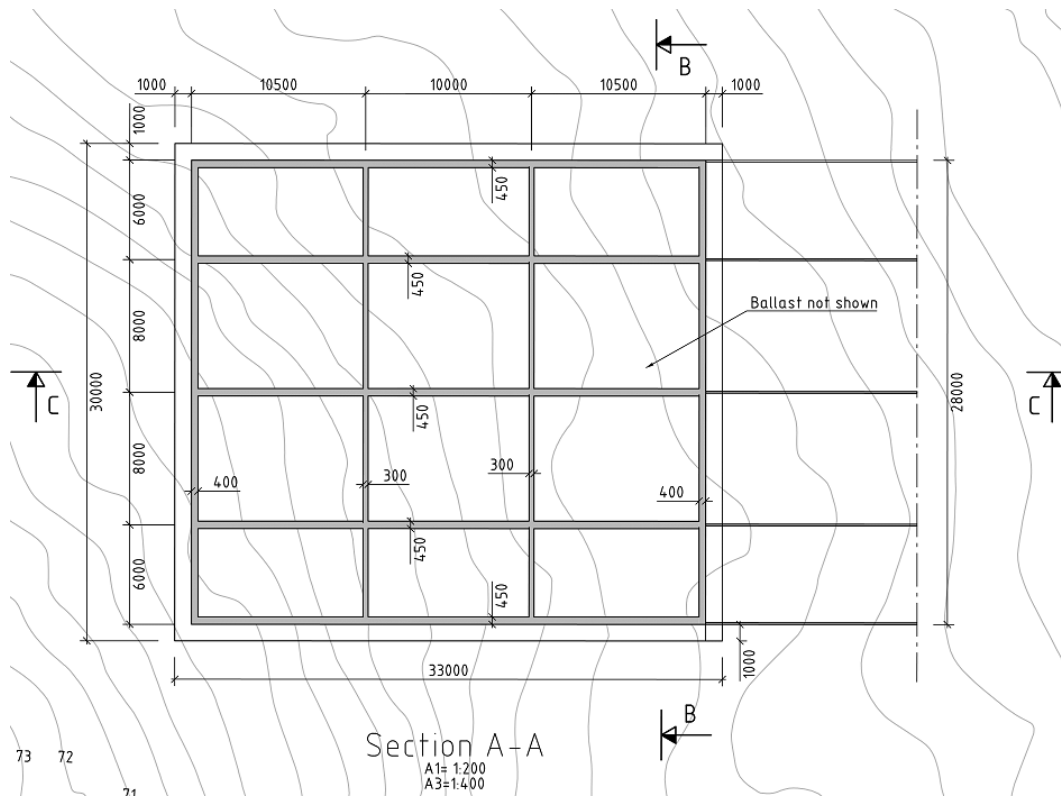
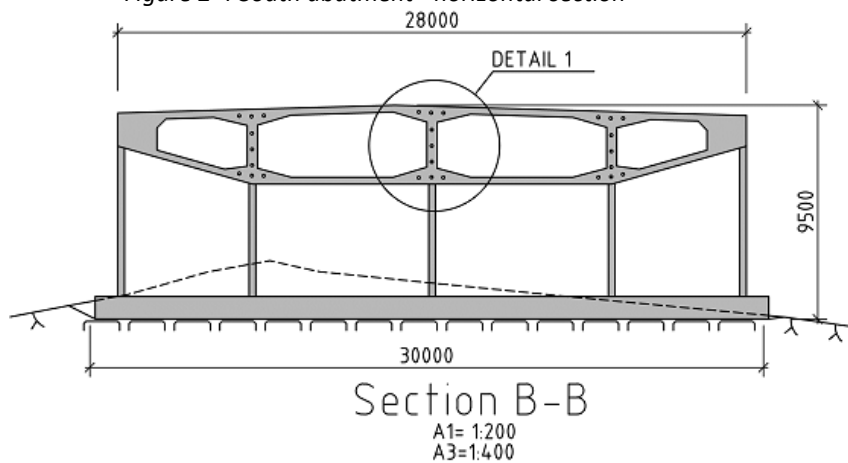*Figure 2-4 South abutment - horizontal section*

Figure 2-5 South abutment - transverse section

2.7　Ship collision

The ship collision is a very demanding load effect for the north abutment. The collision to the floating bridge leads to high strong axis bending moments, for which the gravity structure and connection shall be designed. The ship collision loads have varied throughout the project period and more refined analyses now show a strong bending moment of 5900 MNm. The fixity of the bridge enhances the bending moments compared with those of the floating bridge in general. The widening of the bridge deck over some length near the abutment leads to sufficient increase in capacity, however, the increased stiffness will again give increased bending moments.

Introduction of a Vessel Traffic Management System in the area is being considered which probably will result in less forces from ship impact.

2.8   Construction

Both abutments shall be built at solid rock, and it is expected to find it right below the ground surface. All work will be above water. In south about 2500 m^3 rock shall be blasted for levelling off the bottom slab, and the double volume in north. A minimum 1 m thick bottom slab will be casted directly on rock forming the base for the abutment. The construction work should be quite straightforward, but there are large areas, 7000 m² and 14000 m^2 formwork in south and north respectively and concrete volumes of 4000 m^3 and 9000 m³. Both abutments shall be filled with solid ballast, in south 4000 m^3 gravel and in north 11000 m^3 iron ore.

In south the concrete bridge side span will be cast monolithically to the abutment. In north the floating bridge steel girder shall be attached to the abutment by 150 nos. prestressing tendons. The ducts will be placed in the walls and slabs in the abutment, and the structures casted until about 1.0 m from the position for the steel bridge. When the steel girder is in position the gap will be cast and the strands installed for anchorage in the steel bridge. The tendons will be stressed from the north side of the abutment.

3   Cable stayed bridge calculations

3.1   Introduction

The analyses that support the cable stayed bridge (and abutment) design are combinations from different analyses:

• Global, dynamic wind and wave analyses of floating bridge (ORCAFLEX)

• Global dynamic analyses of ship collision to floating bridge (LS-DYNA)

• Permanent load, wind and traffic (RM Bridge)

• A local dynamic wind analysis of critical construction stages for the cable stayed bridge (Novaframe)

• Local cable stayed bridge (SOFISTIK) used for verification

The design checks for permanent stage for the different structural elements are based on forces from the global analysis. These analyses are documented in separate Appendices, primarily F, G and J. The load combinations used are envelope based. For dynamic loads such as dynamic wind and waves, separate time history analyses are done for wind and waves for a one hour storm (one seed). The time history of the results, e.g. section forces, are then added linearly, and the maximum section force/moment combined with the other five coexisting section forces/moments at the time step is defining the envelope. This methodology is accurate enough to justify the technical feasibility and estimate the quantities.

The structural design capacities are checked for the following design forces:

- ULS forces during service: Maximum concurrent time history forces and moments from environmental load realisations load combined with static loads. In ULS2 traffic load is the dominating variable load combined with 1 year return period environmental loads. In ULS3 100 year return period environmental loads are the dominating variable load ("ULS2_ULS3" in capacity diagrams).

- ALS Ship collision: Maximum concurrent time history forces and moments from the different ship impact simulations for deck house and pontoon impacts combined with permanent static loads ("ALS-ship-deckh_pont" in capacity diagrams)

- ULS constructions stages: Frequency domain wind results for constructions stages a, b and c load combined with permanent static loads. ("CS_abc" in capacity diagrams)

The designed cross sections are generally chosen as equal to those in the global analysis models.

In the subchapters below, the design of critical structural components and sections is outlined. For more detailed documentation of the design of concrete sections, refence is made Enclosure L1 for plots, detailed listings and comparative plots for different load actions and bridge concepts. The following six concrete sections are evaluated:

Table 3-1 FE-model, structural parts and concrete sections

No	Structural part	Section	No. of sections
1	Lower tower legs	Connection to foundation	2 (east and west)
2	Lower tower legs	Connection to cross beam	2 (east and west)
3	Tower cross beam	Connection to tower legs	2 (east and west)
4	Side piers	Connection to foundation	5 (5 piers)
5	Girder concrete side span	Mid spans	5 (between all side piers)
6	Girder concrete side span	Supports	5 (at all side piers)

The design verification of the steel box girder is done within the global FE-model itself by postprocessing routines using the same principles as for the floating bridge. The design verification is done using two different cross section types H1 and H2 with different plate thicknesses of the skin plates. General stresses are reported at corner points of the box girder along the box girder with 10.0 m spacing. The design verification is carried out using the reduced width method within Eurocodes.

3.2 Tower

The tower design will be driven by wind loads on the free-standing tower during construction, ship collision forces and maximum ULS forces during service.

Wind along the bridge on the free standing tower in construction stage is governing for the reinforcement in lower parts of the tower. The normal force is low for the tower in this stage and therefore have a small contribution to the capacity. (Compression force to a certain extend increase the bending capacity)

The ship collision leads to a very high transversal force between the bridge deck and the tower legs. Still, ULS demands and ship collision demands are almost equal.

For the tower legs from deck level and upwards, ULS loadings will be governing. For the dimensions of the tower leg, quite conventional, these loads will not be very critical to the leg sections.

The steel anchor boxes will be subjected to the splitting cable forces, and they shall provide a tie between the pair of cables. Shear connectors shall transfer horizontal forces not being in equilibrium to the concrete walls. Based on comparison with similar concepts of other cable stayed bridges, it is relatively easy to design these steel elements for the local loads that are determined by the size of the cables.

3.2.1 Tower design

Capacity diagrams for bending-axial interaction are plotted for three critical tower sections below. A more detailed documentation is given in Enclosure L1.

The capacity diagram for the leg sections shows that moment about transverse axis is most critical compared to the moment around longitudinal axis. The most critical load is wind on free standing tower in construction stage.

Minimum reinforcement for the tower is according to Eurocode simplified 1.2% of concrete area with the used concrete and reinforcement quality. The reinforcement amounts in the diagrams are lower or not much higher than the minimum reinforcement.

For the cross beam both wind in construction stage, ULS and loads from ship impact, ALS are governing. Moment about the horizontal axis is governing.
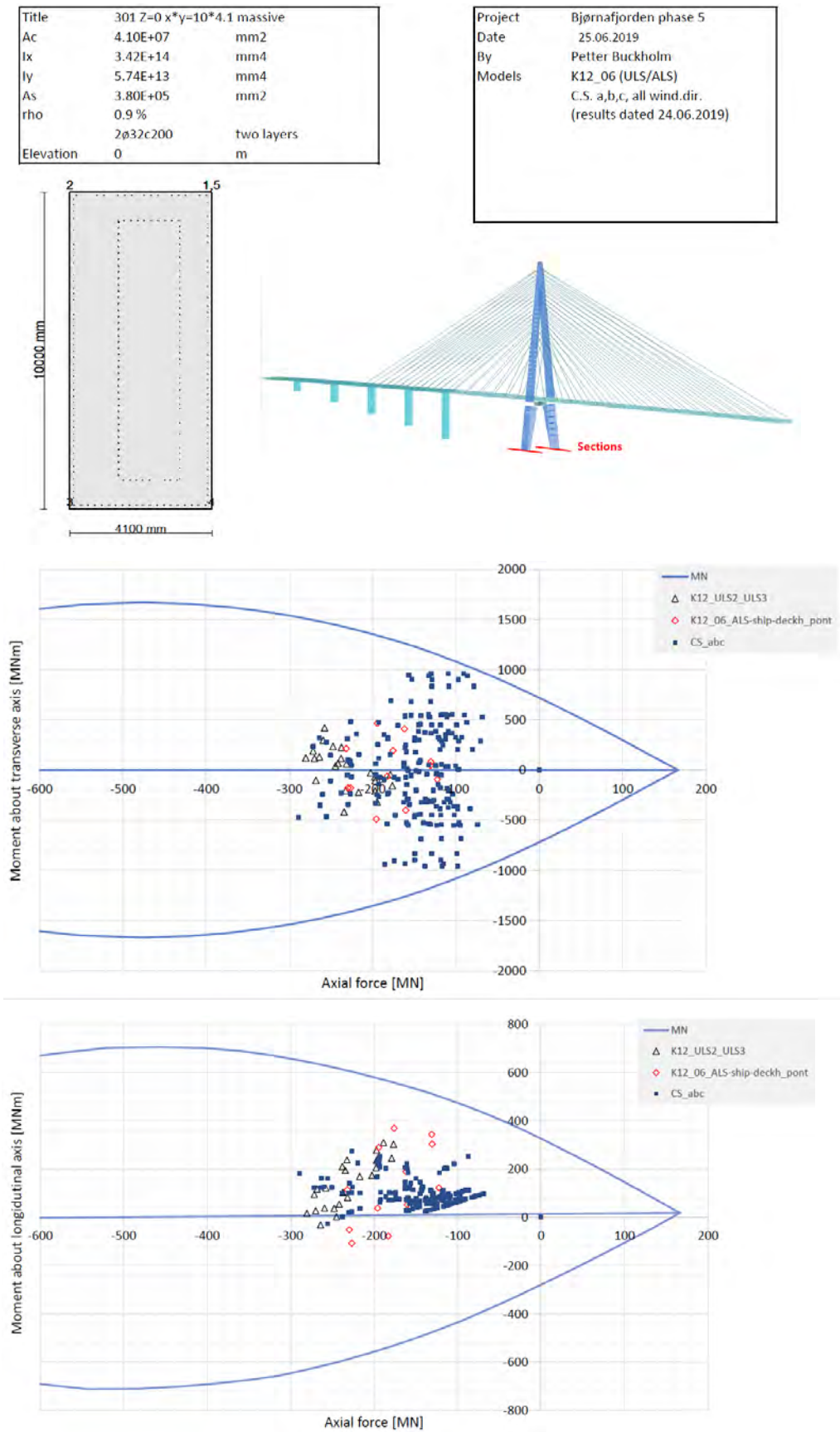
Tower legs at foundation interface

Title	301 Z=0 x*y=10*4.1 massive	
Ac	4.10E+07	mm2
Ix	3.42E+14	mm4
Iy	5.74E+13	mm4
As	3.80E+05	mm2
rho	0.9 %	
	2ø32c200	two layers
Elevation	0	m

| Project	Bjørnafjorden phase 5
Date	25.06.2019
By	Petter Buckholm
Models	K12_06 (ULS/ALS)
	C.S. a,b,c, all wind.dir.
	(results dated 24.06.2019)

Figure 3-1: Capacity diagrams. Tower legs at foundation interface. Triangle – ULS2 & ULS3, red diamond – ALS ship collision, blue square – erection situations

Tower legs at cross beam interface

Title	309 Z=52 x*y=8.82*4.28	
Ac	3.52E+07	mm2
Ix	2.98E+14	mm4
Iy	6.01E+14	mm4
As	5.84E+05	mm2
rho	1.7 %	
	2ø32c200	two layers
Elevation	52	m

Project	Bjørnafjorden phase 5
Date	25.06.2019
By	Petter Buckholm
Models	K12_06 (ULS/ALS)
	C.S. a,b,c, all wind.dir.
	(results dated 24.06.2019)
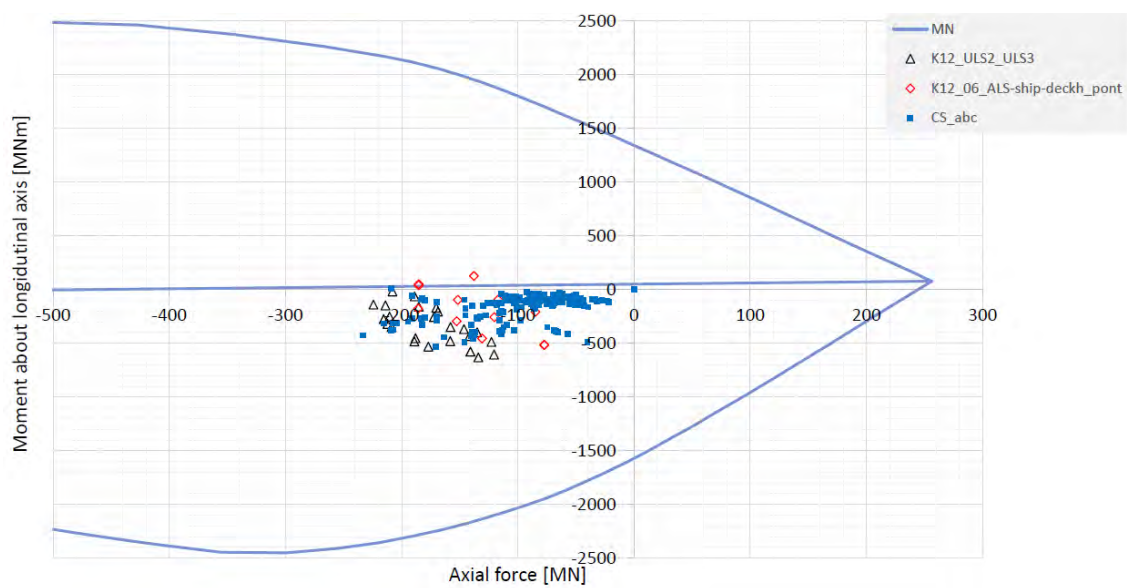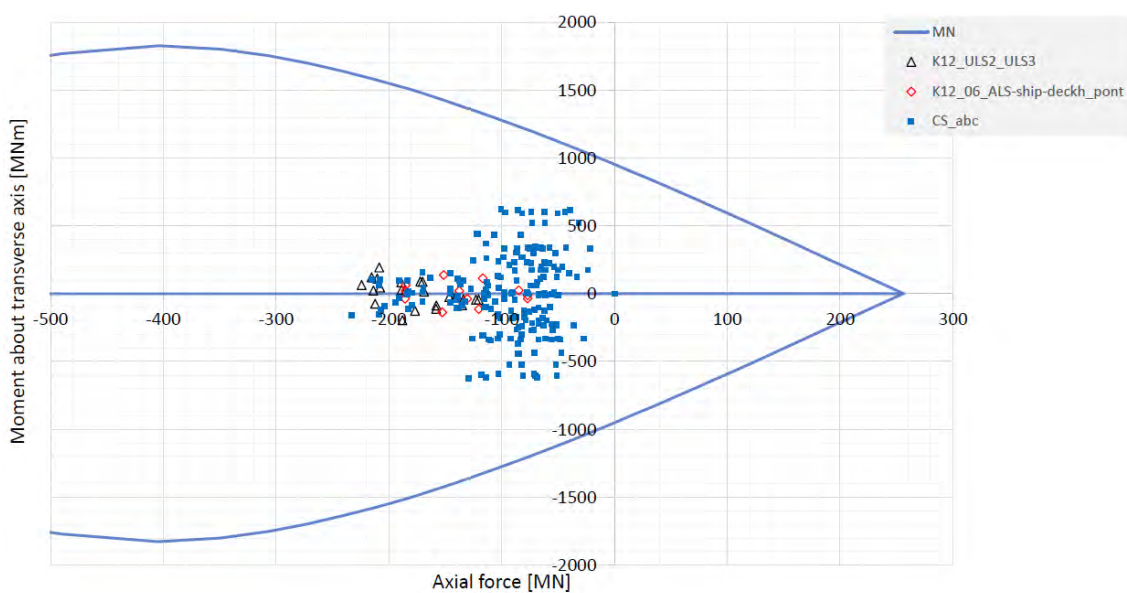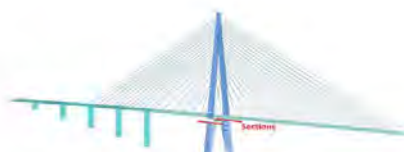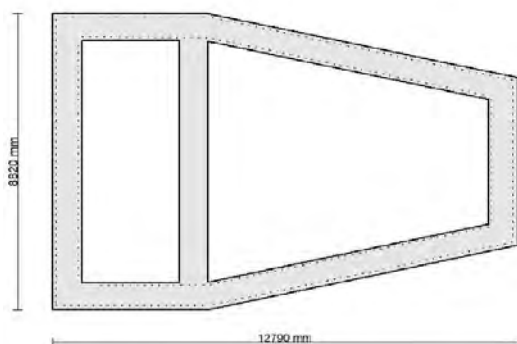
Figure 3-2: Capacity diagrams. Tower legs at cross beam interface. Triangle – ULS2 & ULS3, red diamond – ALS ship collision, blue square – erection situations

Tower cross beam at tower leg interface

Title	New section		Analyzed 5x5
Ac	3.14E+07	mm2	1.34E+07
Ix	1.55E+14	mm4	4.10E+13
Iy	1.31E+14	mm4	4.10E+13
As	4.07E+05	mm2	341001
rho	1.3 %		2.5 %
	2ø32c200		
	two layers top and bottom		
Elevation	0	m	

Project	Bjørnafjorden phase 5
Date	25.06.2019
By	Petter Buckholm
Models	K12_06 (ULS/ALS)
	C.S. a,b,c, all wind.dir.
	(results dated 24.06.2019)
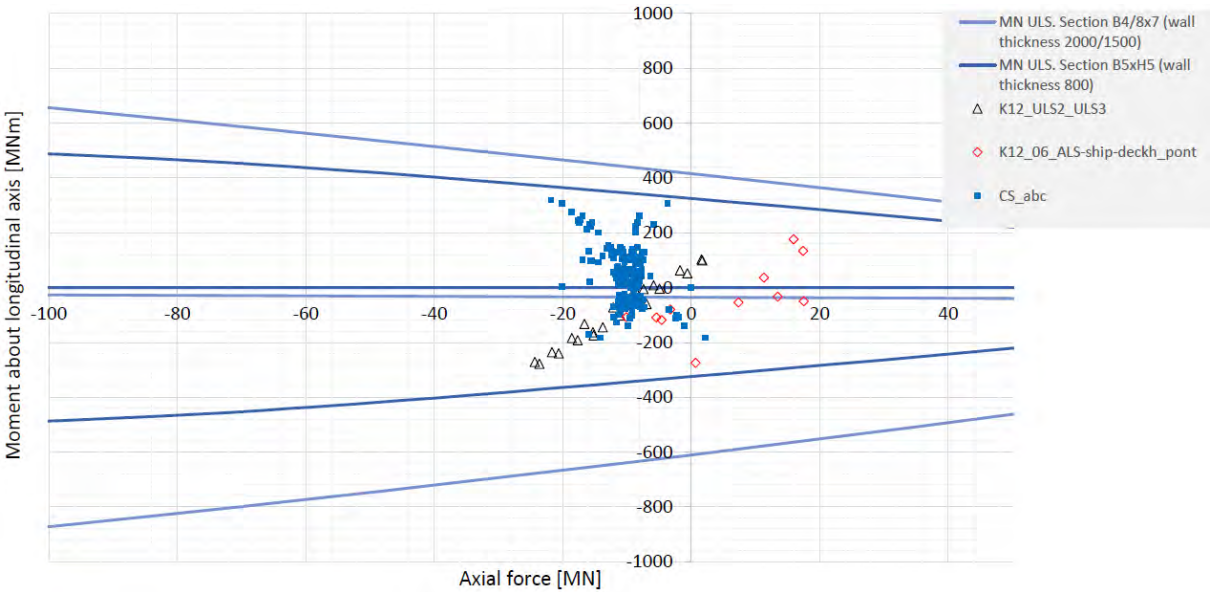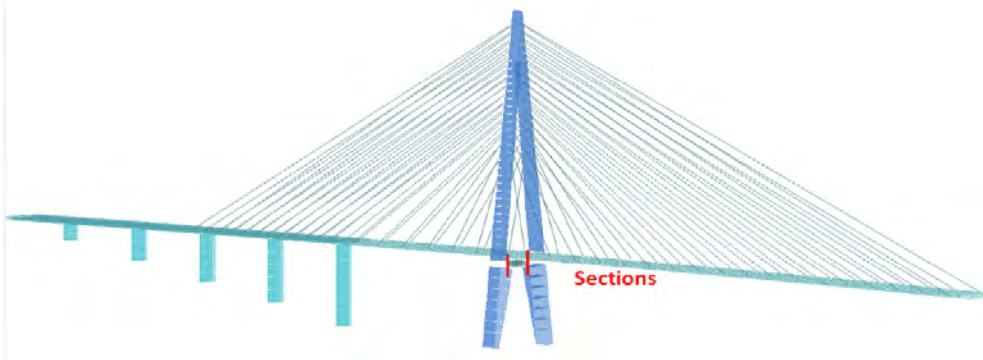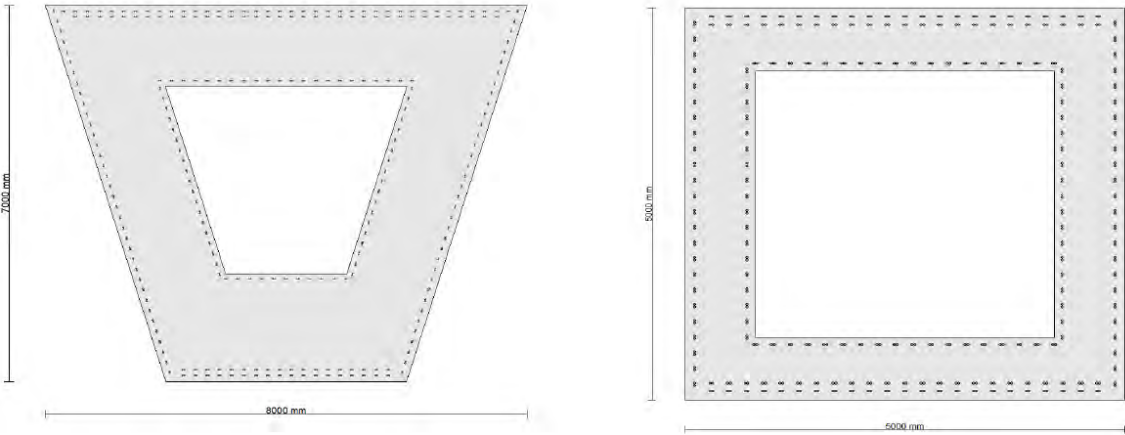
Figure 3-3: Capacity diagrams. Tower cross beam at tower leg interface. Triangle – ULS2 & ULS3, red diamond – ALS ship collision, blue square – erection situations

3.2.2 Tower foundation design

The foundation sizes are governed by wind forces on free standing tower during construction. The chosen sizes are 21x10x5.5. The height of 5.5 m is chosen so the horizontal reinforcement amounts in the cross section shall be moderate.

Following stability checks are performed for the critical load, wind during construction:

- Ground pressure is checked acc. to N400 11.2.3. Should be < 10MPa
- Gliding is checked acc. to N400 11.2.5. Torsional moment is not considered
- Overturning is checked acc. to N400 11.2.1
- Eccentricities also checked in SLS acc. to N400 11.2.2

The checks show a maximum ground pressure of 7 MPa in ULS to be the most critical check. For design checks in permanent stage ground pressure in ULS/ALS and indirectly overturning are checked.

The results show that in permanent stage, ship collision forces in ALS is more critical than the forces from ULS3 (wind and wave dominant) with a maximum ground pressure of 3.1 MPa, i.e. much lower than the ground pressure in construction stage.

Pressure less than 10 MPa on rock or on blasted rock are in most cases acceptable.

More detailed documentation is given in Enclosure L2.

3.3 Bridge deck

3.3.1 Bridge deck – steel

Layout

In general, the below FE-analyses of the steel box girder are based on the geometrical properties stated in the following excel files:

- Cross section type H1 used for the global FE-analyses, refer excel file K12_07_designers_format, sheet "input H1"

- Cross section type H2 used for the global FE-analyses, refer excel file K12_07_designers_format, sheet "input H2"

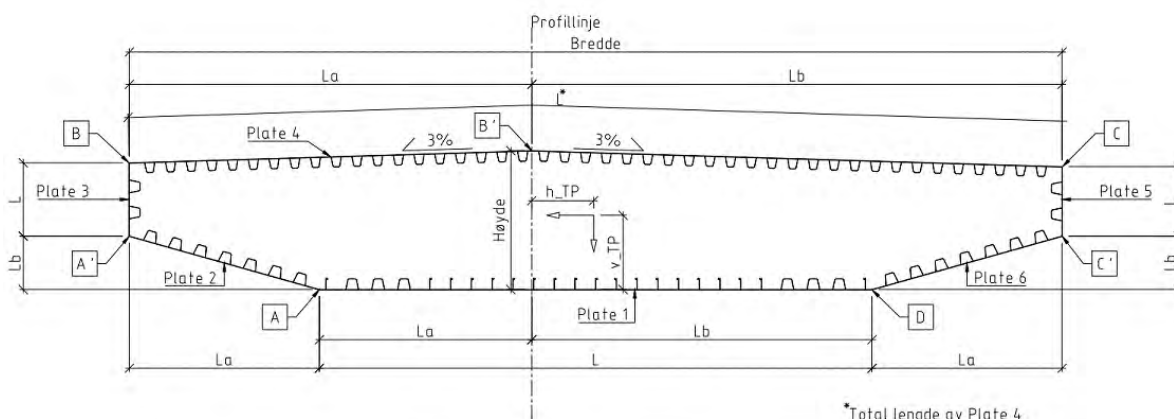The geometrical properties for the two cross section types H1 & H2 are shown in the below figures.

Figure 3-4 Cross section of box girder utilised in the global FE-analyses inclusive definition of thicknesses, lengths and stress points (A, A', B, B' C, C', D)

Section	K12_H1_rev02		Cable stayed bridge section H1 - consept K12	Revision date:			05-06-2019 @ 12:30	
	K12_H1_02			Last revised by:				
Bredde	27	[m]	fra innside plate 3 til innside plate 5			Total bredde	27.037	
Høyde	3.5	[m]	fra OK plate 1 til OK plate 4			Total høyde	3.512	
	Plate 1		**Plate 2**	**Plate 3**	**Plate 4**		**Plate 5**	**Plate 6**
Thickness [mm]	12		12	25	16		12	12
La [m]	6.150		5.500	-	11.650		-	5.500
Lb [m]	9.850		1.540	-	15.350		-	1.540
L [m]	16.000		5.712	1.595	13.268		1.484	5.712
Trapzoidal type	K12_H1-B		K12_H1-IB	K12_H1-VW	K12_H1-T		K12_H1-VW	K12_H1-IB
Trapezoidal thickness [mm]	6		6	10	8		10	6
Trapezoidal c/c [m]	0.750		0.750	Kun én stiver	0.600		Kun én stiver	0.750
Bulb type	K12_H1-B_b							
Width h [mm]	280							
Thickness t [mm]	11							
Bulb radius r [mm]	12							
Bulb height c [mm]	40							
Bulb width [mm]	42							
Bulb c/c [m]	0.6							

Figure 3-5 Cross section type H1, input for global FE-analyses - refer K12_07_designers_format sheet "input H1"

Tverrsnittsdata, kassetverrsnitt:			
Areal:		1.297	m2
Tyngdepunkt, fra OK:		1.463	m
Tyngdepunkt, fra UK:		2.049	m
Torsjonsmotstand*:		6.629	m4
Skjærareal vertikalt*:		0.0232	m2
Skjærareal transvers*:		0.7109	m2
2. Arealmoment, svak akse***		2.534	m4
2. Arealmoment, svak akse****		2.534	m4
2. Arealmoment, svak akse*		2.534	m4
2. Arealmoment, sterk akse*		89.531	m4
Treghetsmoment**		1348.672	tm2/m
Masse Areal:		10.18	t/m
Masse tverrrstål:		2.00	t/m
Sum stål:		12.18	t/m
Masse rekkverk:		0.71	t/m
Masse asfalt:		4.62	t/m
Sum masse:		17.52	t/m
Contingency:		1.48	t/m
Masse brukt i analyse		19.00	t/m
	*basert på fullt tverrsnitt inkl. stivere (IDEA CSS)		
	** inkl asfalt, rekkverk etc.		
	***basert på tverrsnitt redusert for shear lag i bruksgrense-/utmattingstilstand		
Bøyemotstander	**** basert på tverrsnitt redusert for shear lag i bruddgrensetilstand		

Figure 3-6 Cross section type H1, input of weight and cross section properties for global FE-analyses - refer K12_07_designers_format sheet, "input H1"

From above can be seen that for box girder type H1 the cross section area of the longitudinal steel becomes 1.297 m² equal to 10.2 t/m. For the cable stayed bridge more transverse steel is required compared to the floating bridge as the diaphragms at the stay anchorages shall be stronger. The transverse steel is therefore estimated to 20% of the longitudinal steel becoming 2.0 t/m. Steel for stay anchorages, full plated diaphragm at the tower location and longitudinal diaphragms in line with the bearings in vicinity of the tower, steel noses each side of the deck girder and steel for the transition to the concrete box girder is estimated all in all to be approximately 4 t/m for cross section H1 and the below mentioned H2.

Section	K12_H2_rev02		Cable stayed bridge section H2 - consept K12	Revision date:		06-06-2019 @ 13:30
	K12_H2_02			Last revised by:		
Bredde	27 [m]		fra innside plate 3 til innside plate 5		Total bredde	27.06
Høyde	3.5 [m]		fra OK plate 1 til OK plate 4		Total høyde	3.52
	Plate 1	**Plate 2**	**Plate 3**	**Plate 4**	**Plate 5**	**Plate 6**
Thickness [mm]	20	20	30	20	30	20
La [m]	6.150	5.500	-	11.650	-	5.500
Lb [m]	9.850	1.540	-	15.350	-	1.540
L [m]	16.000	5.712	1.595	13.672	1.484	5.500
Trapzoidal type	K12_H2-B	K12_H2-IB	K12_H2-VW	K12_H2-T	K12_H2-VW	K12_H2-IB
Trapezoidal thickness [mm]	10	10	12	10	12	10
Trapezoidal c/c [m]	0.750	0.750	Kun én stiver	0.600	Kun én stiver	0.750
Bulb type	K12_H2-B_b					
Width h [mm]	320					
Thickness t [mm]	12					
Bulb radius r [mm]	14					
Bulb height c [mm]	46					
Bulb width [mm]	48.6					
Bulb c/c [m]	0.6					

Figure 3-7 Cross section type H2, input for global FE-analyses - refer K12_07_designers_format sheet "input H2"

Tverrsnittsdata, kassetverrsnitt:		
Areal:	1.797	m2
Tyngdepunkt, fra OK:	1.633	m
Tyngdepunkt, fra UK:	1.887	m
Torsjonsmotstand*:	9.663	m4
Skjærareal vertikalt*:	0.0342	m2
Skjærareal transvers*:	1.0057	m2
2. Arealmoment, svak akse***	3.640	m4
2. Arealmoment, svak akse****	3.640	m4
2. Arealmoment, svak akse*	3.694	m4
2. Arealmoment, sterk akse*	123.340	m4
Treghetsmoment**	1343.222	tm2/m
Masse Areal:	14.10	t/m
Masse tverrrstål:	2.00	t/m
Sum stål:	16.10	t/m
Masse rekkverk:	0.71	t/m
Masse asfalt:	4.62	t/m
Sum masse:	21.44	t/m
Contingency:	-2.44	t/m
Masse brukt i analyse	19.00	t/m
	*basert på fullt tverrsnitt inkl. stivere (IDEA CSS)	
	** inkl asfalt, rekkverk etc.	
	***basert på tverrsnitt redusert for shear lag i bruksgrense-/utmattingstilstand	
Bøyemotstander	**** basert på tverrsnitt redusert for shear lag i bruddgrensetilstand	

Figure 3-8 Cross section type H2, input of weight and cross section properties for global FE-analyses - refer K12_07_designers_format sheet "input H2"

From above can be seen that for box girder type H2 the cross section area of the longitudinal steel becomes 1.797 m² equal to 14.1 t/m. For the cable stayed bridge more transverse steel is required compared to the floating bridge as the diaphragms at the stay anchorages shall be stronger. The transverse steel is maintained at 2.0 t/m. Remaining steel for anchorages, wind noses etc, please refer to cross section type H1.

Section types

The different section types used within the global FE-model can be seen in the below figure, refer K12_07_PROD_load_combinations_bridge_direct_expected_max, dated 25. June 2019.
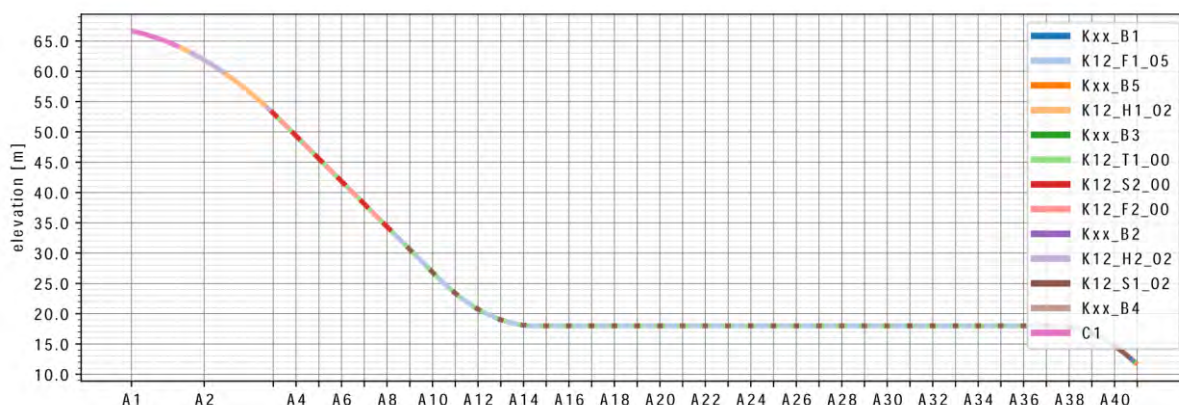
Figure 3-9 Section types used in global FE-analyse - ████ K12_H1_02, ████ K12_H2_02

The chainage for the different section types of the steel girder utilised in the global FE-model can be seen below and are found in the document K12_07_designers_format. For the cable stayed bridge cross section type H1 and H2 is utilised as follows:

Cable stayed bridge, side span: H1, chainage 38795 – 38850, length 55.0 m

                                                               H2, chainage 38850 – 38930, length 80.0 m

Cable stayed bridge, main span: H2, chainage 38930 – 39040, length 110.0 m

                                                               H1, chainage 39040 – 39270, length 230.0 m

                                                               H2, chainage 39270 – 39295, length 25.0 m

From above, the total length of steel box girder for the cable stayed bridge becomes 495 m. Please note that there can be small variation from above input into the global FE-models and the outcome shown on the drawings.

Design assumptions and verification

In the global FE-model stresses are taken out in corner points of the box girder, refer Figure 3-4.

The design verification is done within the global model by postprocessing routines using the same principles as for the floating bridge. In this section a summary will be presented showing important section forces, von Mises stresses and for the ULS design verification the reduced width method has been used in accordance with Eurocode, referred to as "method 2". Design verification will be done for the ULS design situations as well as the ALS design situations (ship collision).

Design verification of ULS2 & ULS3 load situations

The below figures have been taken from SBJ-33-C5-AMC-90-RE-107_0 Appendix G Global Analyses – Response as well as the results from K12_07_PROD_load_combinations_direct, dated 25. June 2019.

For the ULS load situations, the material factor is γ_m =1.1, which for steel S420 with mostly thin plates gives f_{yd} = 420/1.1 = 382 MPa. This value has been used as the limit in the von Mises plots shown for the ULS2 (traffic load dominant) and ULS3 load situations (environmental load dominant).

The below 6 figures present and overview/envelopes of the section forces for ULS2 and ULS3 load situations. The steel deck of the cable stayed bridge is located within position A1 to A3.
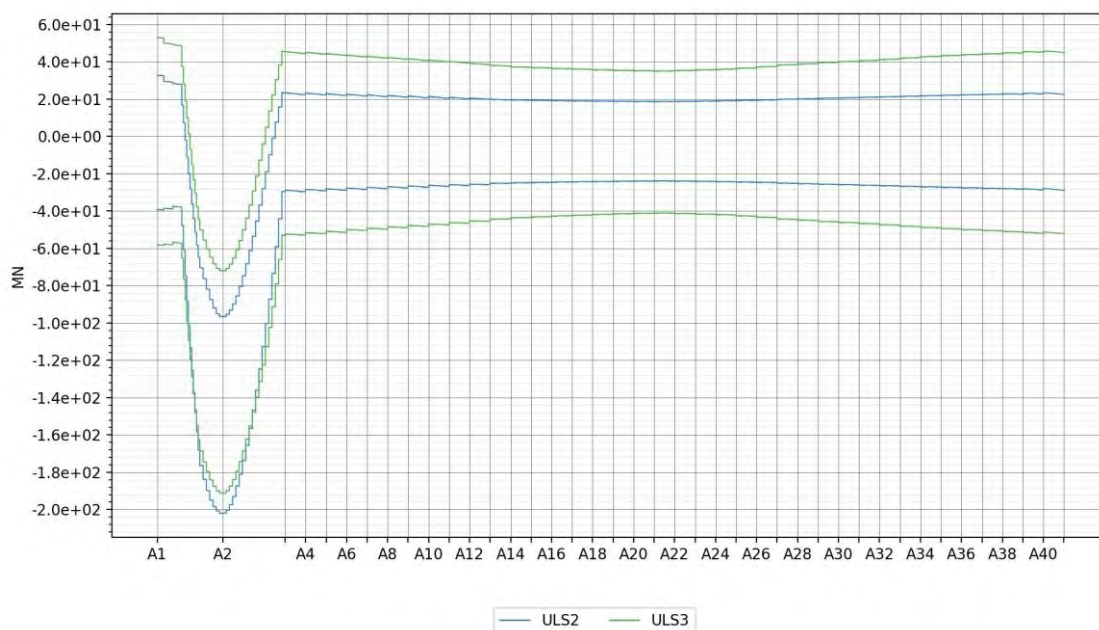
Figure 3-10 Bridge girder axial force – ULS. Note the additional compression force within the cable stayed bridge due to its nature
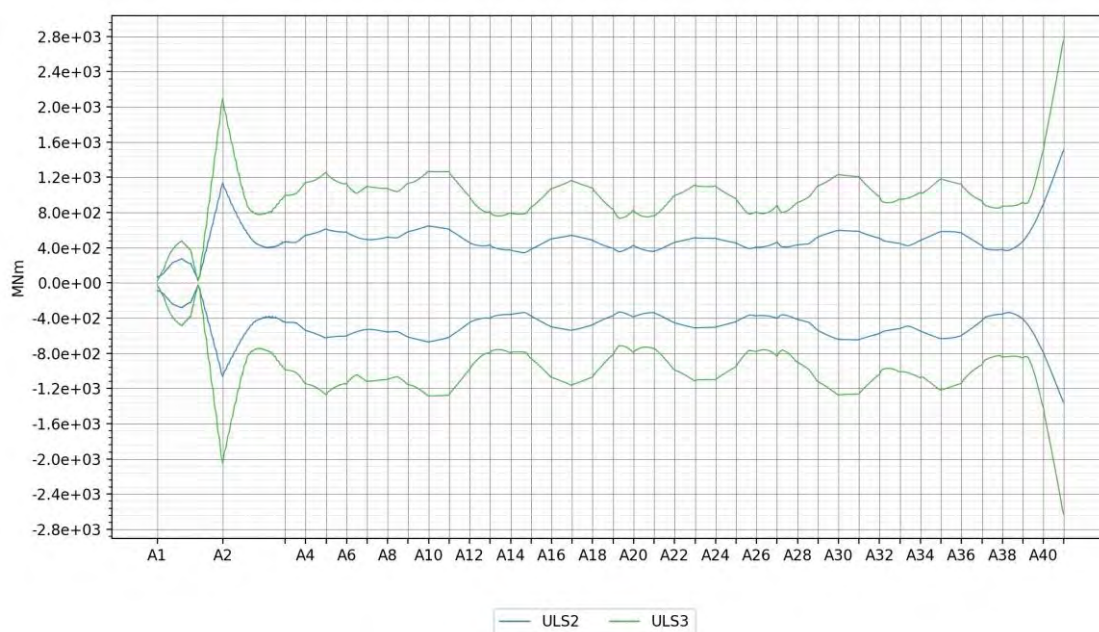
Figure 3-11 Bridge girder bending moment about strong axis – ULS

From the above figure is seen that maximum bending moment round strong axis becomes max/min 2100 MNm for the ULS3 load situations and only 1100 MNm for the ULS2 load situations. It is furthermore the general impression that ULS3 is worse than ULS2.
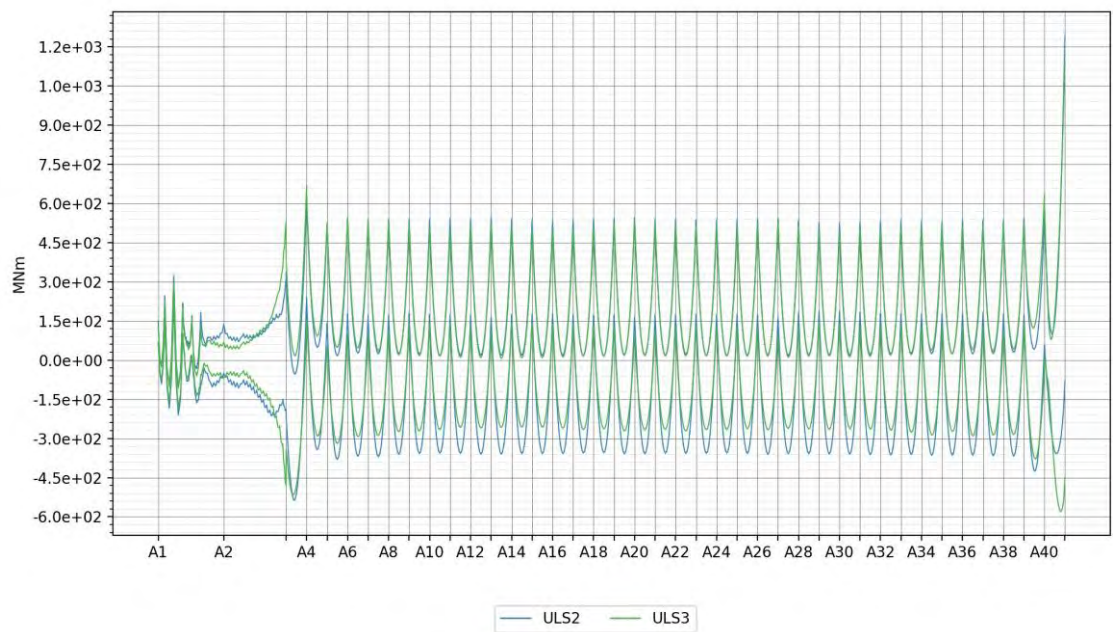
Figure 3-12 Bridge girder bending moment about weak axis – ULS
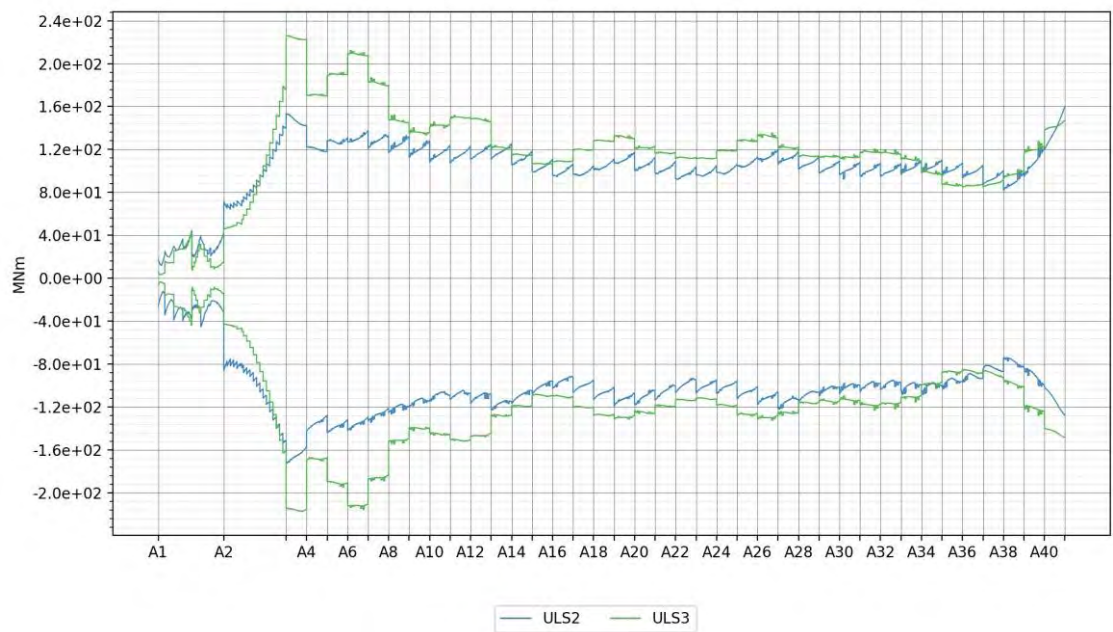
Figure 3-13 Bridge girder torsional moment – ULS
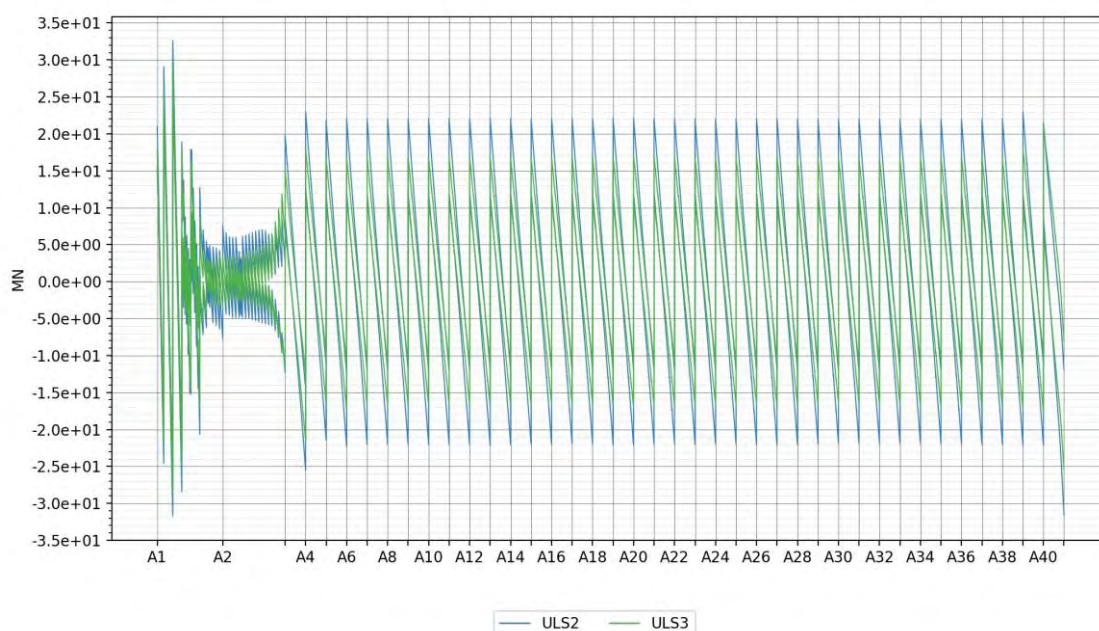
Figure 3-14 Bridge girder vertical shear force – ULS
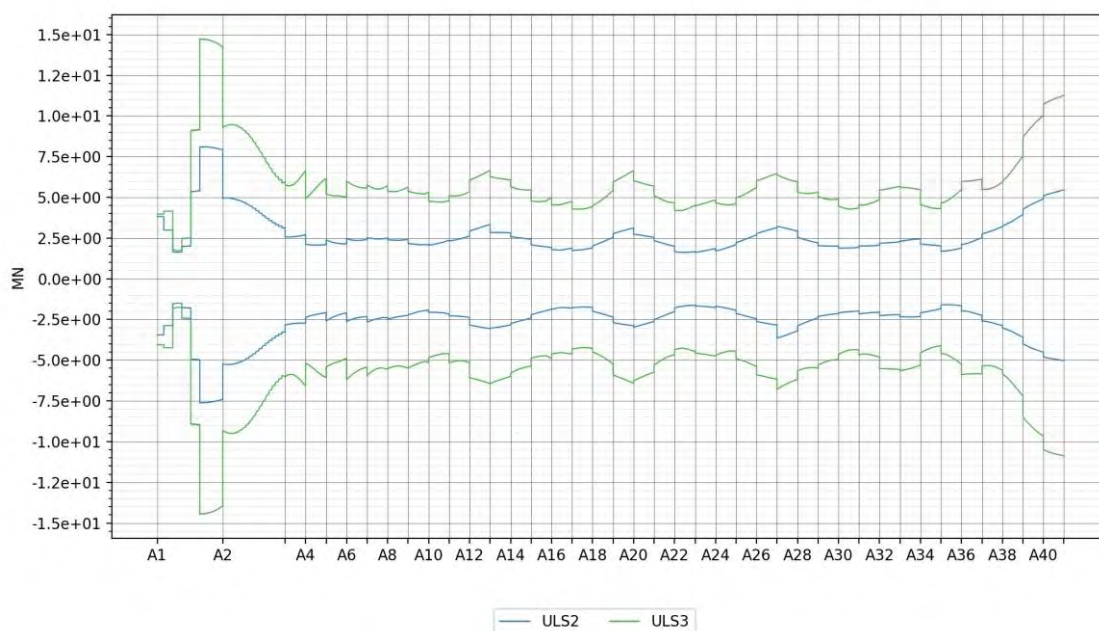
Figure 3-15 Bridge girder transverse shear force – ULS

In the below 3 figures a summary is presented of the max von Mises stresses obtained for load situations ULS2 and ULS3. For the cable stayed bridge it can been seen that in general the ULS3 load situations (environmental load dominant) is more severe than the ULS2 (traffic load dominant).

It can be seen that the von mises stresses when considering global section properties stay at or are below the allowable stress level of f_{yd} = 420/1.1 = 382 MPa.
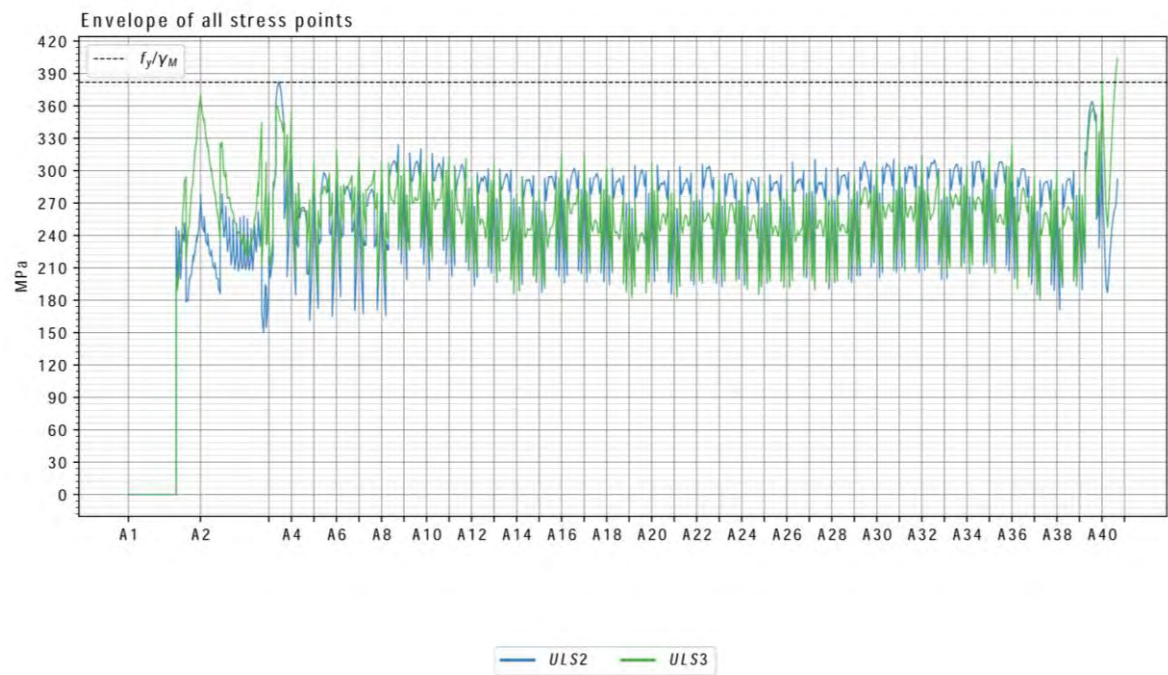
Figure 3-16 Max von Mises stress for ULS2 and ULS3 in all stress points
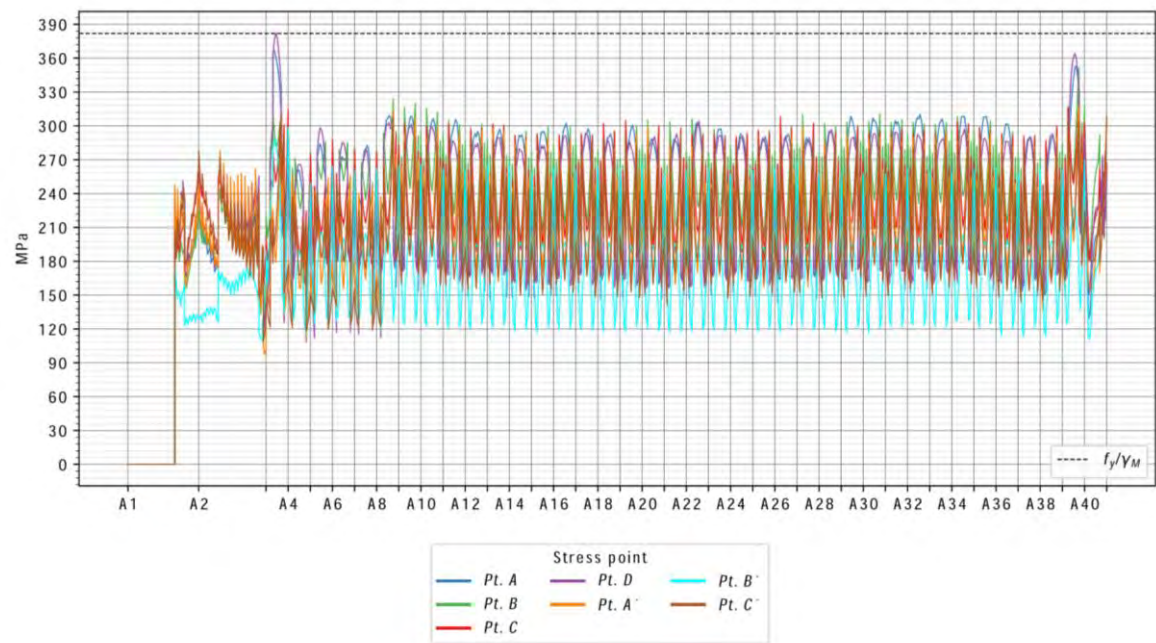
Figure 3-17 Max von Mises stress for ULS2 in stress points
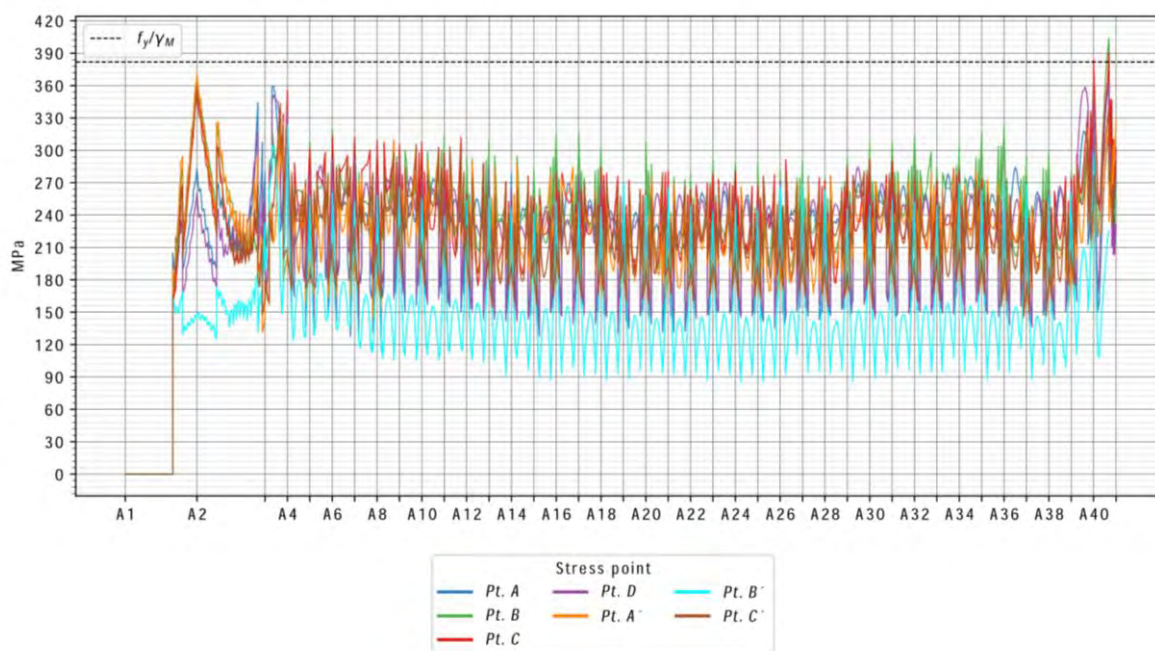
Figure 3-18 Max von Mises stress for ULS3 in stress points

The 3 figures below present the design verification of the cable stayed bridge deck within the ULS load situations. In general, it is seen that ULS3 load situations govern the design and the utilisation ratio is below 1.0 unless 3 small peaks – one at the tower locations, another at the intersection between cross section type H1/H2 and the third when approaching the first pontoon of the floating bridge. All peaks of over utilisation are small and found acceptable at this level of design. Steel from other locations can be moved to these areas bringing the utilisation under 1.0 without increasing the total steel deck quantity.
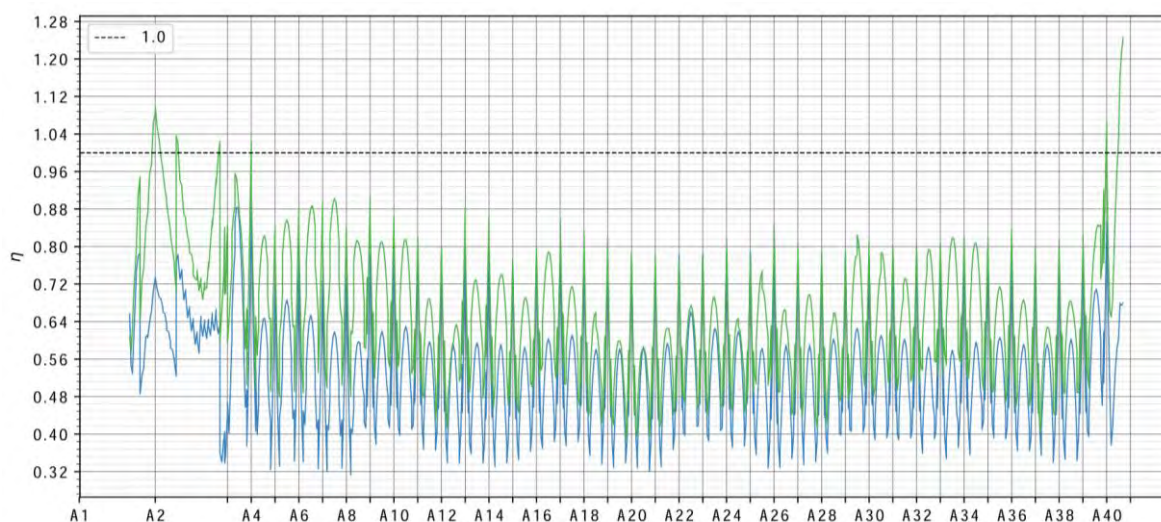
Figure 3-19 Utilisation envelopes using capacity check "method 2" – ULS2 (blue), ULS3 (green)
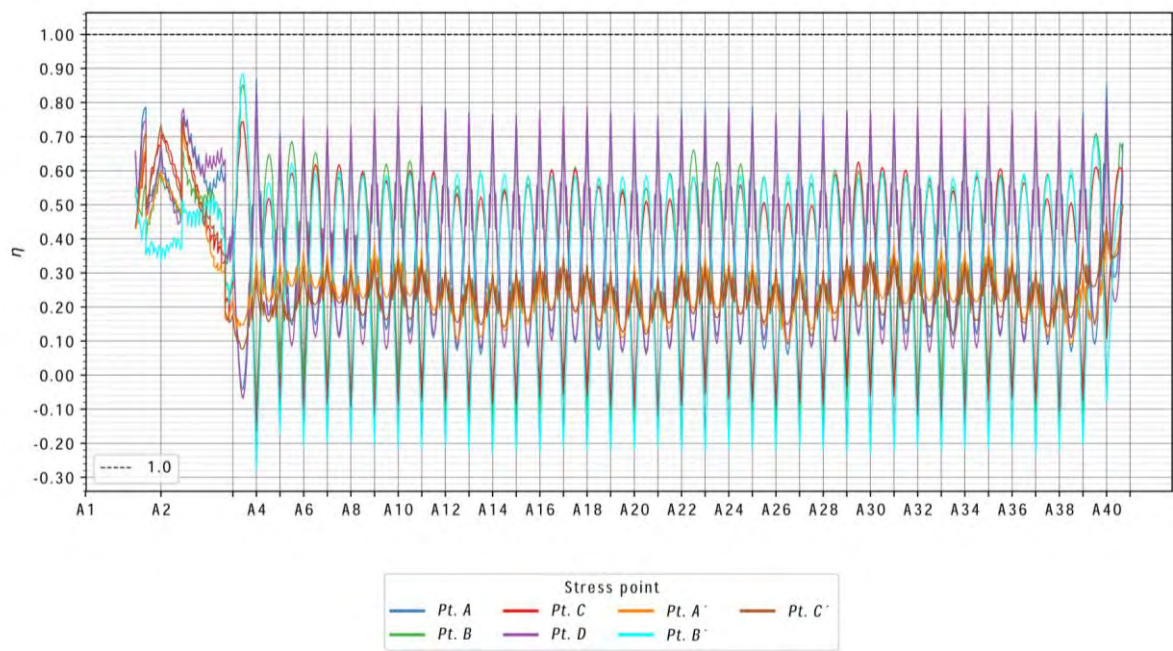
Figure 3-20 ULS2 - utilisation envelopes using capacity check "method 2"
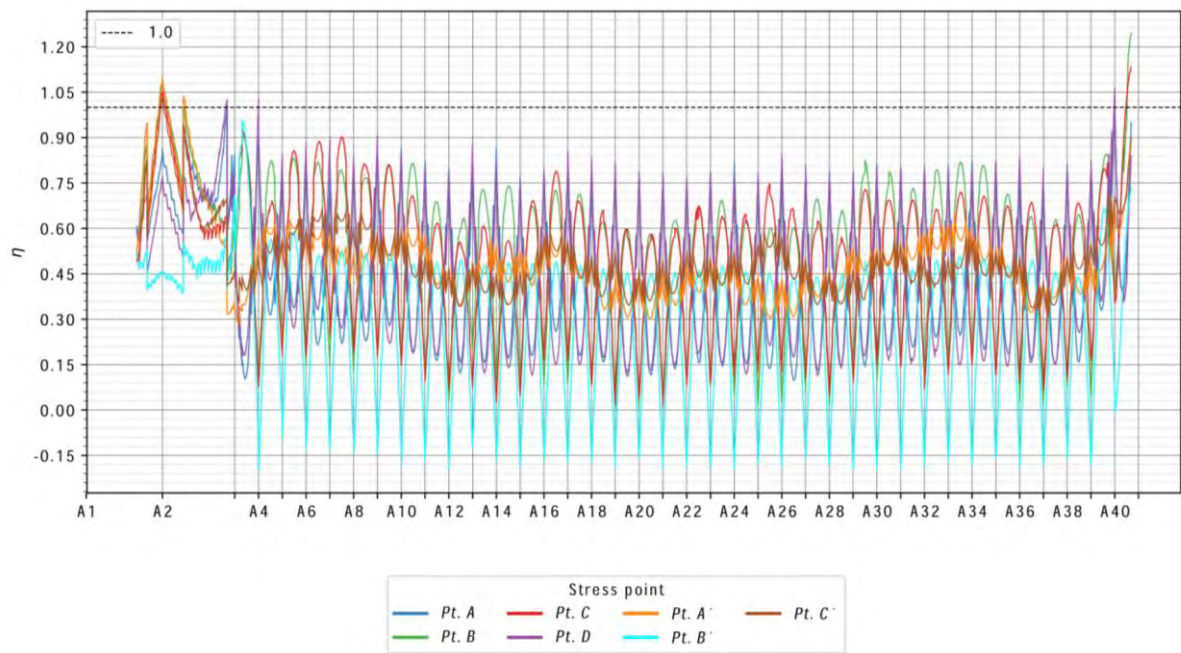
Figure 3-21 ULS3 - utilisation envelopes using capacity check "method 2"

Design verification of ALS ship collision

The below figures have been taken from SBJ-33-C5-AMC-27-RE-110_0 Appendix J Ship collision and ShipCollision_K12_06_revised, dated 26 June 2019, where the section forces are presented for ship collision towards the pontoons and deck house collision towards the box girder. For the cable stayed bridge, the overall dominant section force is the transverse shear force and the bending moment round strong axis as presented below.

For the ALS the material factor γ_m =1.0. Steel S420 with mostly thin plates gives f_{yd} = 420 MPa.

The below 4 figures present shear force and bending moment round strong axis for all ship collision analyses carried out for "collision towards pontoon" and "collision towards bridge girder". The steel deck of the cable stayed bridge is located within position A1-E to A3.
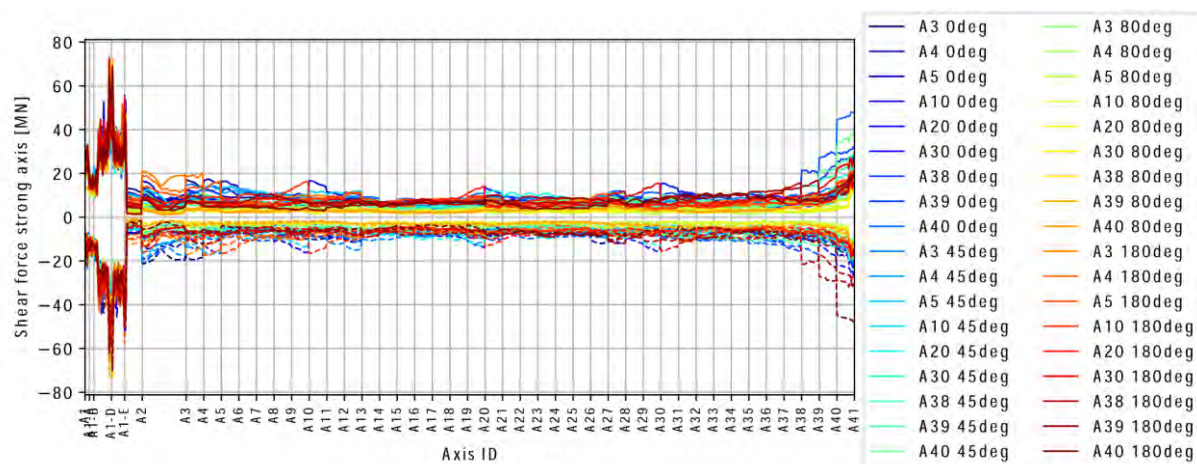
Figure 3-22 Ship collision towards pontoon - shear force strong axis for all analyses
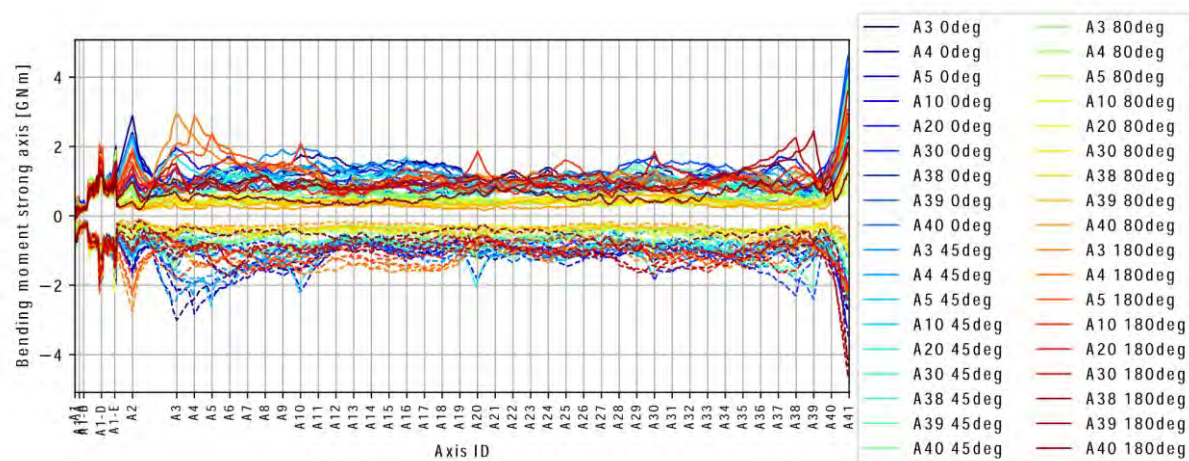
Figure 3-23 Ship collision towards pontoon – bending moment round strong axis for all analyses
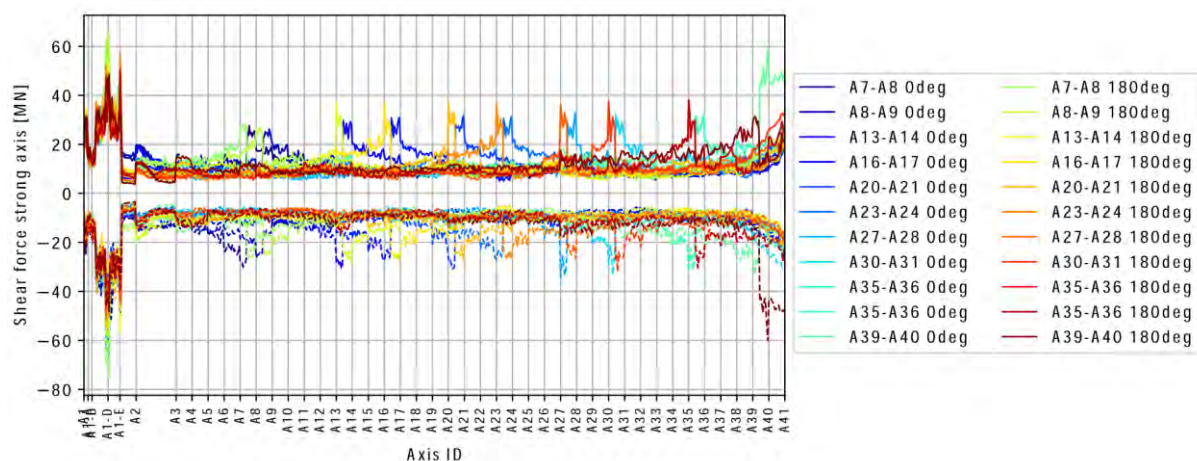
Figure 3-24 Ship collision towards bridge girder – shear force strong axis for all analyses
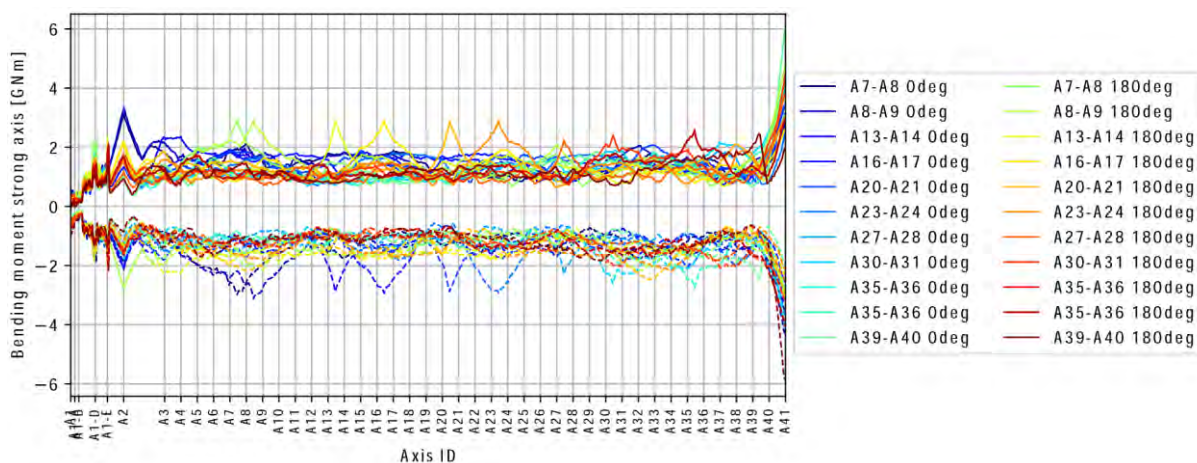
Figure 3-25 Ship collision towards bridge girder – bending moment round strong axis for all analyses

To present a design verification of all the above analyses carried out becomes too comprehensive here and therefore a summary will be presented in the 5 figures presented below where all results are envelopes and summaries of the ship collision analyses carried out.
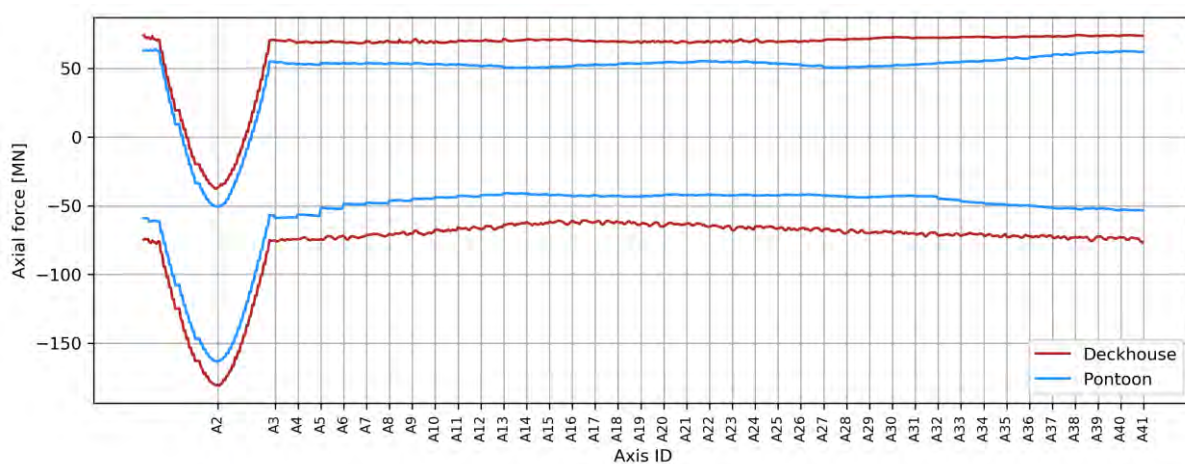

Figure 3-26 Axial force envelopes for the bridge girder, note the additional compression force within the cable stayed bridge due to its nature
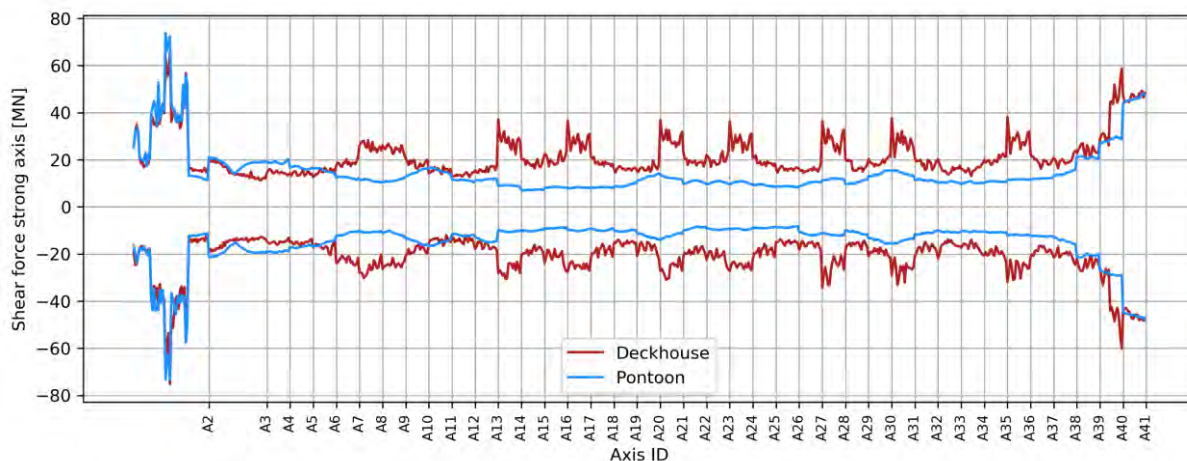
Figure 3-27 Strong-axis shear force envelopes for the bridge girder
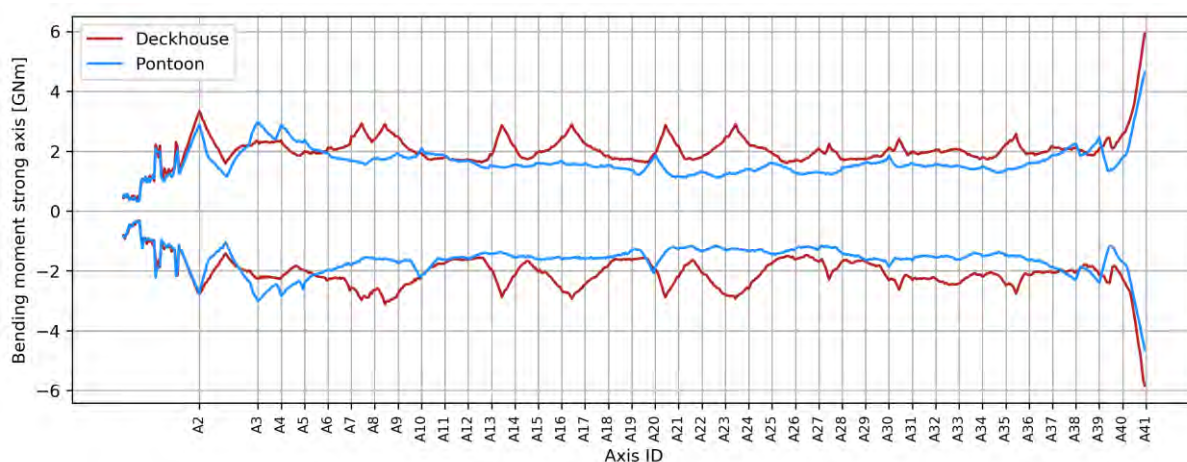
Figure 3-28 Strong-axis bending moment envelopes for the bridge girder

From the above figure can be seen that deck house collision in general is worse that the pontoon collision, which is also applicable for the cable stayed bridge. The maximum bending moment round strong axis becomes 3300 MNm for deck house collision and 2800 MNm for pontoon collision.
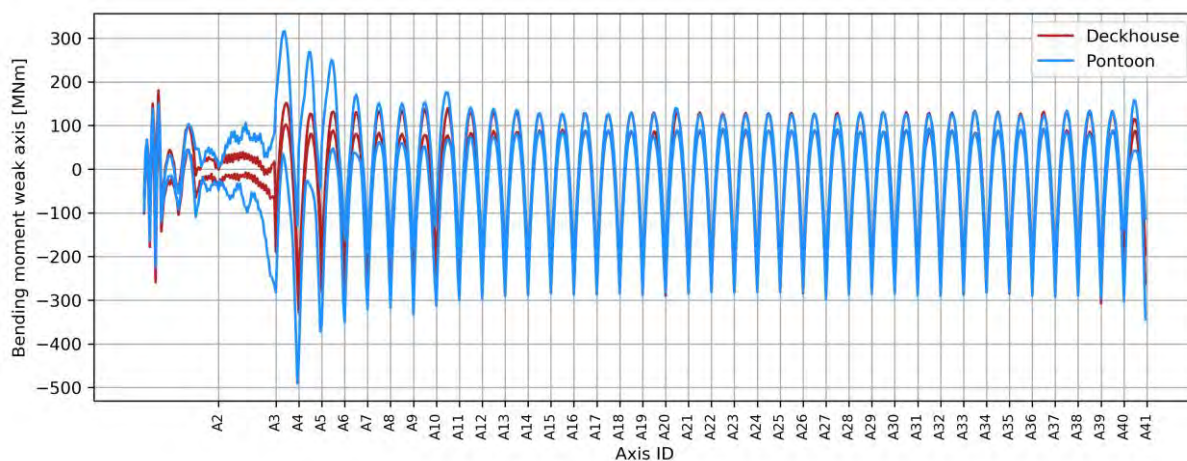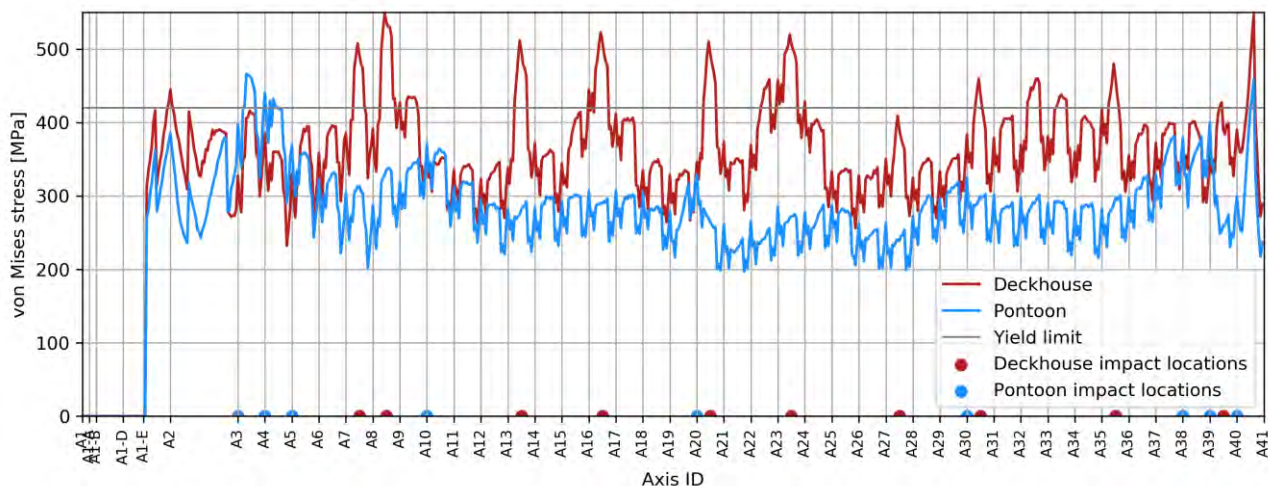

Figure 3-29 Weak-axis bending moment envelopes for the bridge girder

Figure 3-30 Envelopes of von Mises stress for the K12_06 bridge. Lines represent stresses, dots show the simulated impact locations

The envelope of von mises stresses in the bridge girder are shown in Figure 3-30. The response is almost within the elastic range for the cable stayed bridge, see A1-E to A3 which is judged to fully acceptable for the design verification of the ALS load situations at this stage of the project. Please furthermore note that a full design verification using the reduced width method has not implemented for the ALS design situations. However, based on results from the ULS analyses, it can be expected that the utilisation will increase by approximately 10% when considering the full design verification using reduced width method (method 2). For the ALS ship collision this over utilisation is however found acceptable at this stage of the design.

3.3.2   Bridge deck – concrete

Capacity diagrams for bending-axial interaction are plotted for design forces at the side span support axes below. The reinforcement and prestressing amounts are chosen so that capacities are fulfilled for the different sections and load situations. It is possible, without getting any high reinforcement amounts or congestion problems, to increase the capacity considerably. More detailed documentation is given in enclosure L1.

The capacity diagram for the section shows that moment about strong axis is most critical compared to the moment about weak axis. The most critical load is ship impact in ALS.
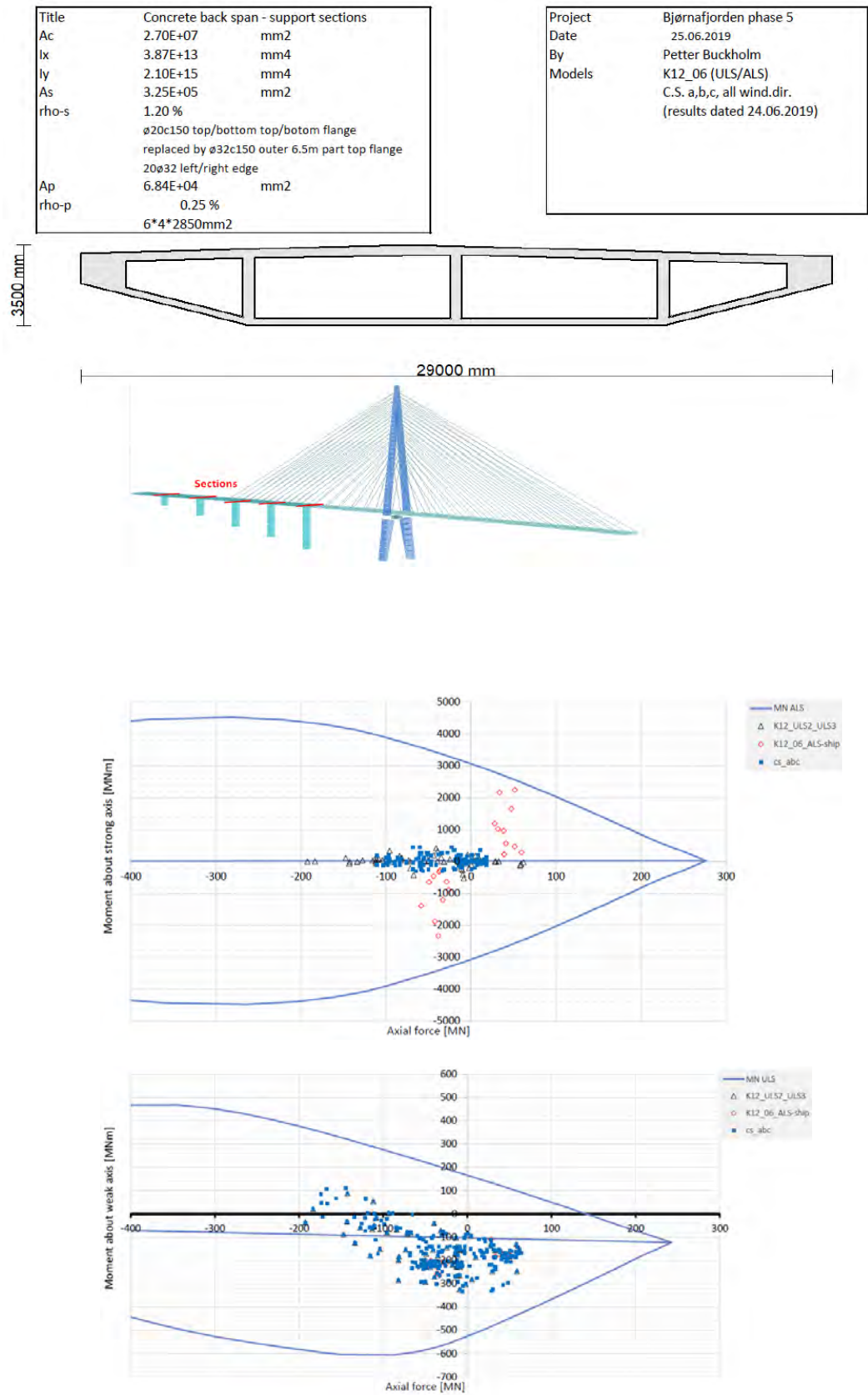
Title	Concrete back span - support sections		Project	Bjørnafjorden phase 5
Ac	2.70E+07	mm2	Date	25.06.2019
Ix	3.87E+13	mm4	By	Petter Buckholm
Iy	2.10E+15	mm4	Models	K12_06 (ULS/ALS)
As	3.25E+05	mm2		C.S. a,b,c, all wind.dir.
rho-s	1.20 %			(results dated 24.06.2019)
	ø20c150 top/bottom top/botom flange			
	replaced by ø32c150 outer 6.5m part top flange			
	20ø32 left/right edge			
Ap	6.84E+04	mm2		
rho-p	0.25 %			
	6*4*2850mm2			

Figure 3-31: Capacity diagrams. Concrete bridge deck at side span support axes. Triangle – ULS2 & ULS3, red diamond – ALS ship collision, blue square – erection situations

3.3.3   Bridge deck joint

The joint between the steel bridge deck and the concrete deck will have to transfer the ULS sectional forces, as well as ship collision. The calculation below only considers the ship collision effect, i.e. a high strong axis bending moment. The coexisting axial force N is 90 MN (compression, beneficial to the joint).

The bending moment of 1500 MNm (currently this is assumed to be on the safe side) leads to edge stresses of 210 MPa. The normal force 64 MPa. With 24 nos 19C15 tendons stressed at 4 MN each, all together 96 MN, the joint will open up somewhat as indicated below. The remaining part will be in compression, and the maximum steel stress will be 418 MPa.
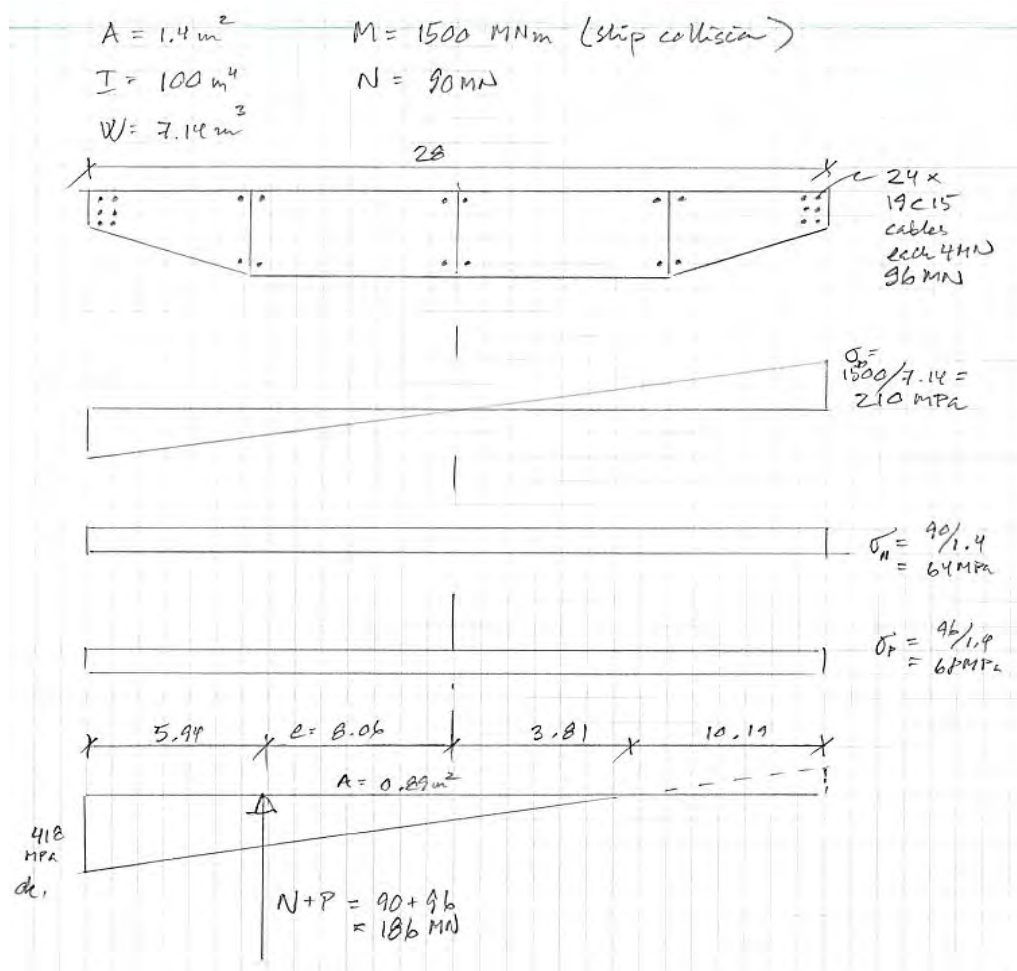
Figure 3-32 Bridge deck joint. Ship collision calculation

3.4 Side span piers

3.4.1 Pier design

Capacity diagrams for bending-axial interaction are plotted for the piers at the foundation interface below. More detailed documentation is given in Enclosure L1.

The pier sizes in both the ULS service and ALS ship impact analyses is a rectangular section of 8.0 m wide and 1.5 m thick. For analysis of wind loads during construction, updated sizes of piers are used, rectangular section 10.5 m wide and 2.1 m thick. The pier stiffness is important for the load distribution and thus to ensure consistency, the designed sections in the capacity diagrams are chosen according to the analysed section described above. Uncracked stiffnesses are used in the analysis being somewhat conservative. Design capacity diagrams are used to document that the capacities are sufficient.

Wind on free standing piers in the construction stage is defined as phase A1 (more details in Enclosure L3), and the envelope section forces in this phase are shown separately in the capacity diagrams. The most critical load combination for the longest pier (pier 1-E) is wind in the longitudinal direction (210°) in phase A1 resulting in a transverse bending moment of approximately 140 MNm at the foundation interface. For pier 1E at the foundation interface the reinforcement demand is slightly higher than for the section showed in the capacity diagrams. For the other piers the reinforcement amount in the capacity diagrams will suffice. Generally, the bending moment about the transverse axis in phase A1 will be more critical than the bending moment about the longitudinal axis.

Minimum reinforcement for the pier is according to Eurocode simplified 1.2% of concrete area with the used concrete and reinforcement quality. The reinforcement amounts in the diagrams are similar to the minimum reinforcement amount.

Title	Back column x*y=1.5*8 mass.	
Ac	1.20E+07	mm2
Ix	2.25E+12	mm4
Iy	6.40E+13	mm4
As	1.83E+05	mm2
rho	1.5 %	
	2ø32c200	3 layers short edge
Title	Back column x*y=2.1*10.5 mass.	
Ac	2.21E+07	mm2
Ix	8.10E+12	mm4
Iy	2.03E+14	mm4
As	2.54E+05	mm2
rho	1.2 %	
	2ø32c200	3 layers short edge

Project	Bjørnafjorden phase 5
Date	25.06.2019
By	Petter Buckholm
Models	K12_06 (ULS/ALS)
	C.S. a,b,c, all wind.dir.
	(results dated 24.06.2019)
	For CS calculations, analyses
	are based on sect 10.5m*2.1m

Figure 3-33 Capacity diagrams. Side span piers. In service ULS/ALS. Analysed/designed with 8x1.5 sections. Triangle – ULS2 & ULS3, black diamond – ALS ship collision
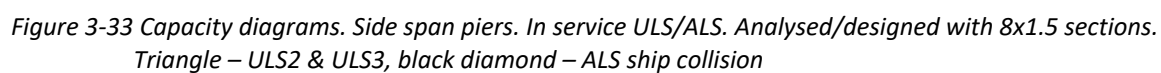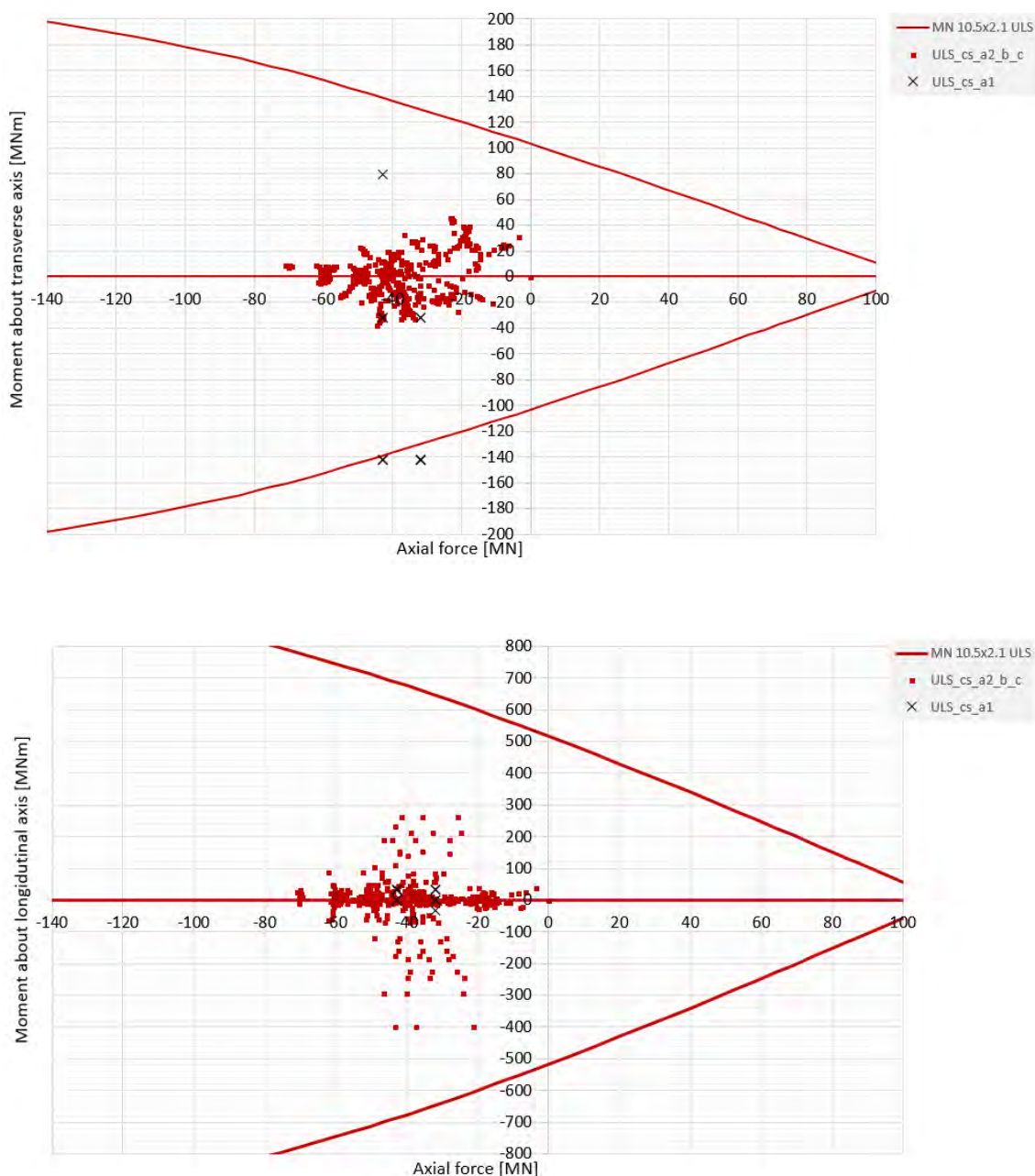
Figure 3-34 Capacity diagrams. Side span piers. Constr. stages ULS

3.4.2 Pier foundation design

The foundation sizes are checked in ULS and ALS ship collision with calculation of the ground pressure and indirectly check of the overturning moment. This check ensures a reasonable sizing of the foundations. The height of 3.0 m is chosen and will give moderate reinforcement amounts in the foundations. The following formulas are used in the calculations:

$e_l = M_{trans}/N$

$e_n = M_{long}/N$

$A = (Width - 2e_l)*(depth - 2e_n)$

$\Sigma(sigma) = N/A$

N, M_{trans}, M_{long}: Forces and moments at bottom of foundation.

The results show that ULS 3 (wind and wave dominant) is the critical load combination, the ship collision forces become less critical.

The calculated ground pressure is low for all foundations. Pressures less than 10 MPa on rock or on blasted rock are in most cases acceptable. The calculations are enclosed in enclosure L2. For axis 1-E wind loads on free standing pier during construction is governing for the foundation size. The ground pressure is calculated to 3.6 MPa

Figure 3-35 Table of ground pressure stresses from service state

Axis	ULS 3 Sigma max	PLS Sigma max
1-A	-0.7	-0.8
1-B	-0.9	-0.9
1-C	-1.0	-1.0
1-D	-1.2	-0.9
1-E	-1.1	-0.4

Unit:Mpa

3.5 Cable stays

The various key data and load effects are summarised below. The total steel mass in all stay cables is approximately 1000 tons. The load effects presented on drawing-DR-105 vary slightly from the values presented in this document since the drawing was based on earlier analysis revision. The permanent cable tension loads are based on an earlier revision of the permanent weight of the bridge girder. It has been checked, although not incorporated in the results below, that this does not change the total steel cable quantity. The results are quite as expected and comply well with general experience from cable stayed bridge design. The permanent load is dominant, followed by traffic load, whereas wind and wave loads are of relatively little influence. The most noticeable effect for wind and wave is the wave load effects to the stays nearest to axis 3, and their side span stays.

Table 3-2: Cable data

		Main span																	
Cable no		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
Length	m	69	81	97	114	132	150	170	189	209	229	249	269	289	309	329	350	370	390
No of strands	nos	31	31	31	37	43	43	55	55	55	55	61	61	67	67	67	67	67	67
Breaking load	MN	8,6	8,6	8,6	10,3	12,0	12,0	15,3	15,3	15,3	15,3	17,0	17,0	18,7	18,7	18,7	18,7	18,7	18,7
Linear weight	kg/m	36,50	36,50	36,50	43,57	50,63	50,63	64,76	64,76	64,76	64,76	71,83	71,83	78,89	78,89	78,89	78,89	78,89	78,89
Total weight*	T	2,5	3,0	3,5	5,0	6,7	7,6	11,0	12,2	13,5	14,8	17,9	19,3	22,8	24,4	26,0	27,6	29,2	30,8
HDPE tube	mm	160	160	160	180	200	200	200	200	200	200	225	225	250	250	250	250	250	250
Anchor tube deck	mm	244	244	244	273	323	323	323	323	323	323	355	355	406	406	406	406	406	406
Anchor tube tower	mm	323	323	323	355	406	406	419	419	419	419	419	419	508	508	508	508	508	508
Permanent load	MN	2,0	2,2	2,5	2,7	3,0	3,2	3,4	3,6	3,8	4,1	4,4	4,7	4,9	5,2	5,2	5,2	5,2	5,2

		Side span																	
Cable no		18	17	16	15	14	13	12	11	10	9	8	7	6	5	4	3	2	1
Length	m	292	281	270	258	247	236	225	214	203	192	181	170	150	132	114	97	81	69
No of strands	nos	61	61	61	61	61	61	55	55	55	55	43	43	43	37	37	31	31	31
Breaking load	MN	17,0	17,0	17,0	17,0	17,0	17,0	15,3	15,3	15,3	15,3	12,0	12,0	12,0	10,3	10,3	8,6	8,6	8,6
Linear weight	kg/m	71,83	71,83	71,83	71,83	71,83	71,83	64,76	64,76	64,76	64,76	50,63	50,63	50,63	43,57	43,57	36,50	36,50	36,50
Total weight*	T	21,0	20,2	19,4	18,5	17,7	17,0	14,6	13,9	13,1	12,4	9,2	8,6	7,6	5,8	5,0	3,5	3,0	2,5
HDPE tube	mm	225	225	225	225	225	225	200	200	200	200	200	200	200	180	180	160	160	160
Anchor tube deck	mm	355	355	355	355	355	355	323	323	323	323	323	323	323	273	273	244	244	244
Anchor tube tower	mm	419	419	419	419	419	419	419	419	419	419	406	406	406	355	355	323	323	323
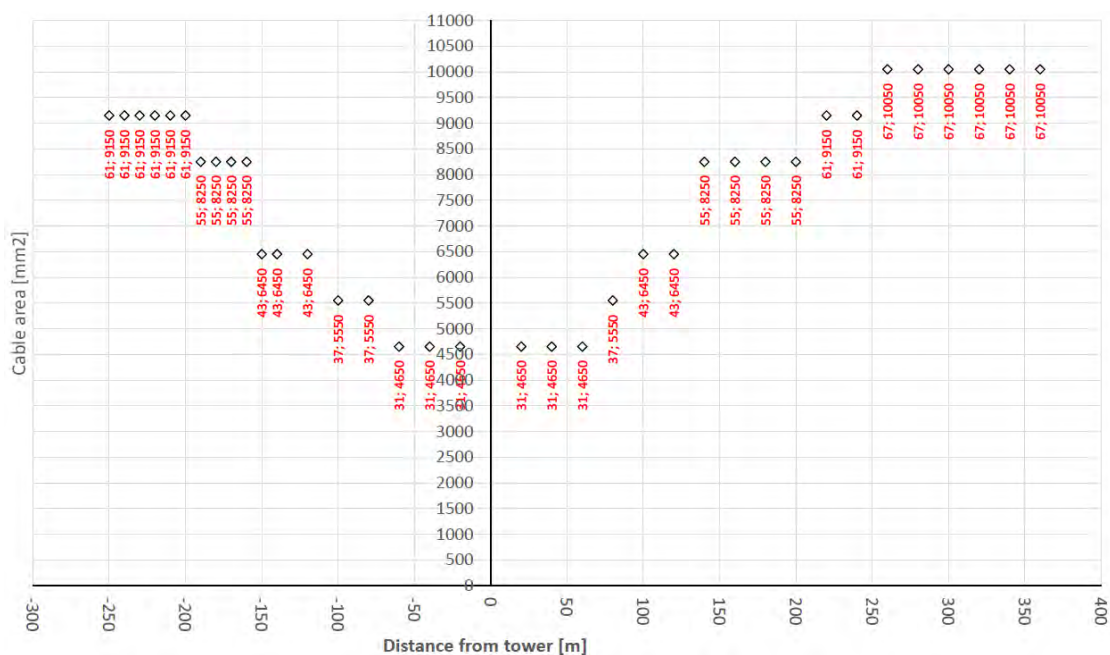
Figure 3-36 Stay cables. Number of strands and corresponding steel area
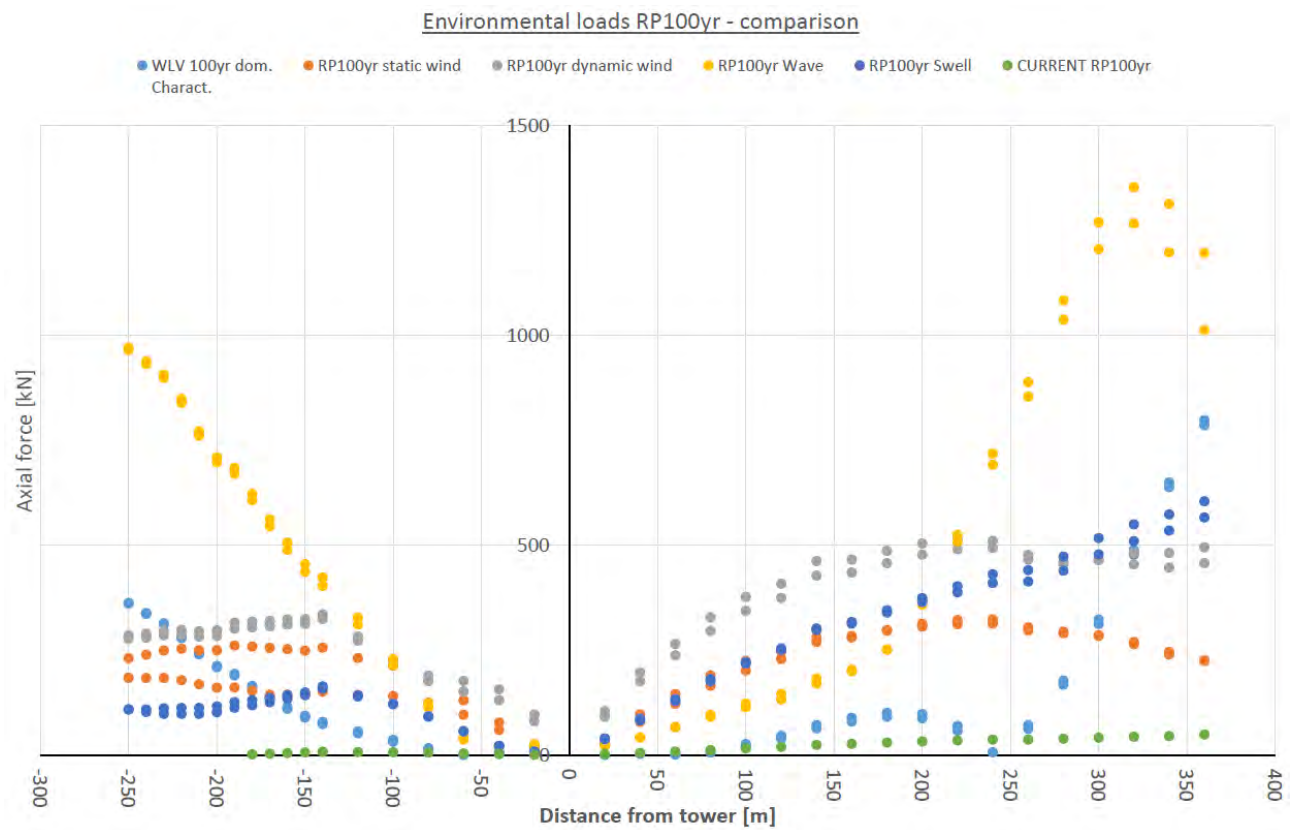
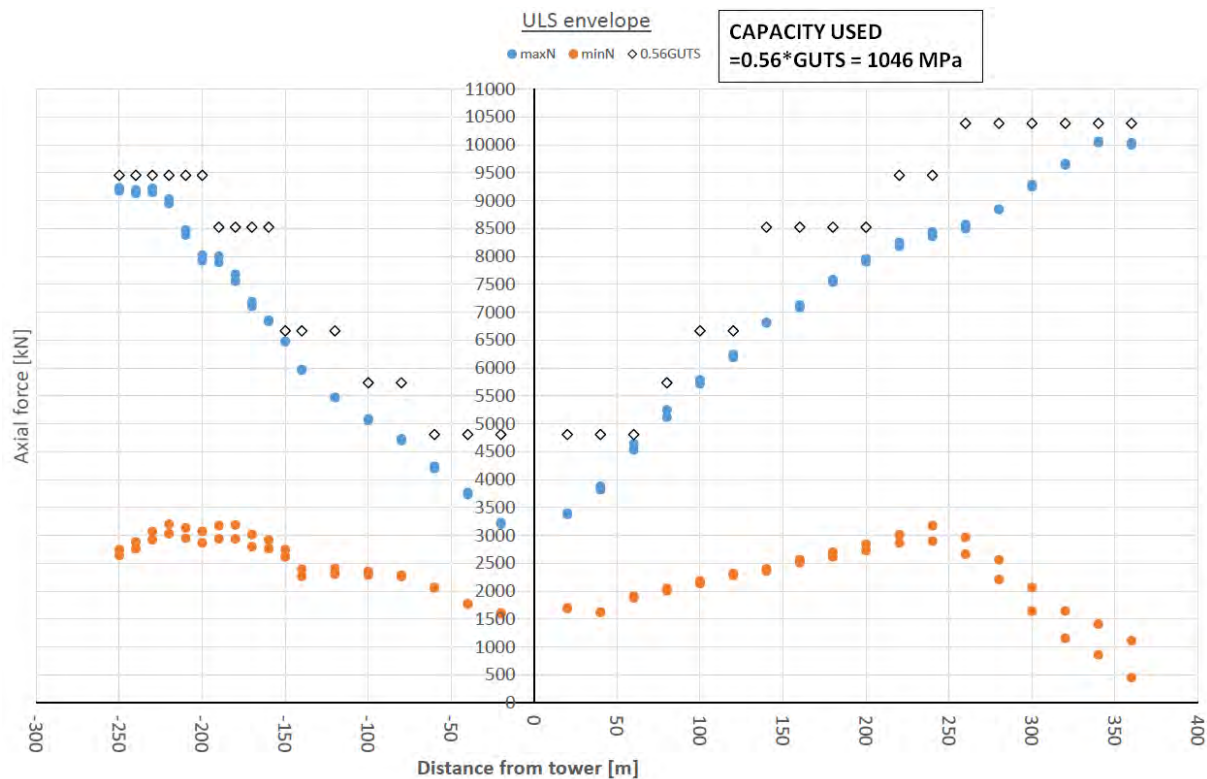*Figure 3-37 Stay cables. K12. Comparison environmental loads RP100yr*

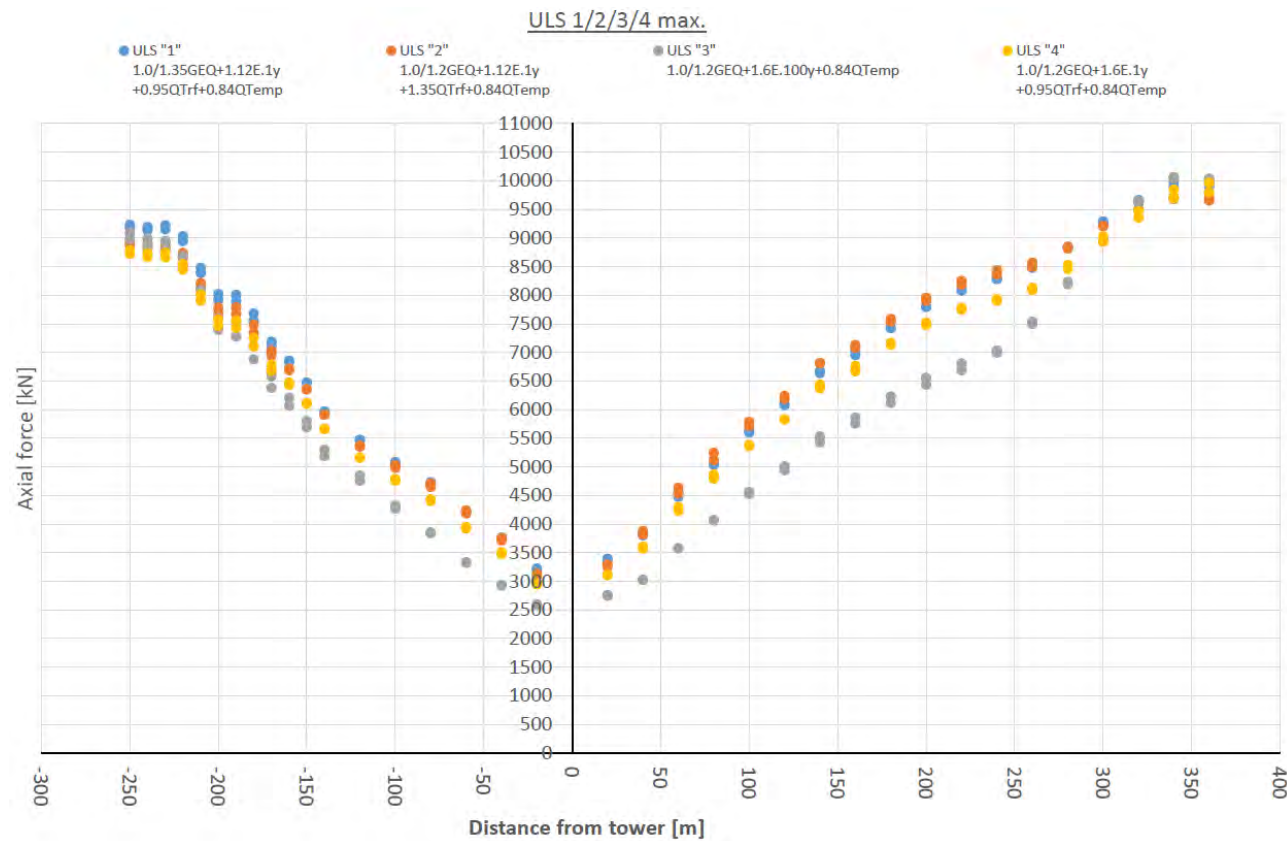*Figure 3-38 Stay cables. Max and min ULS forces and capacity*

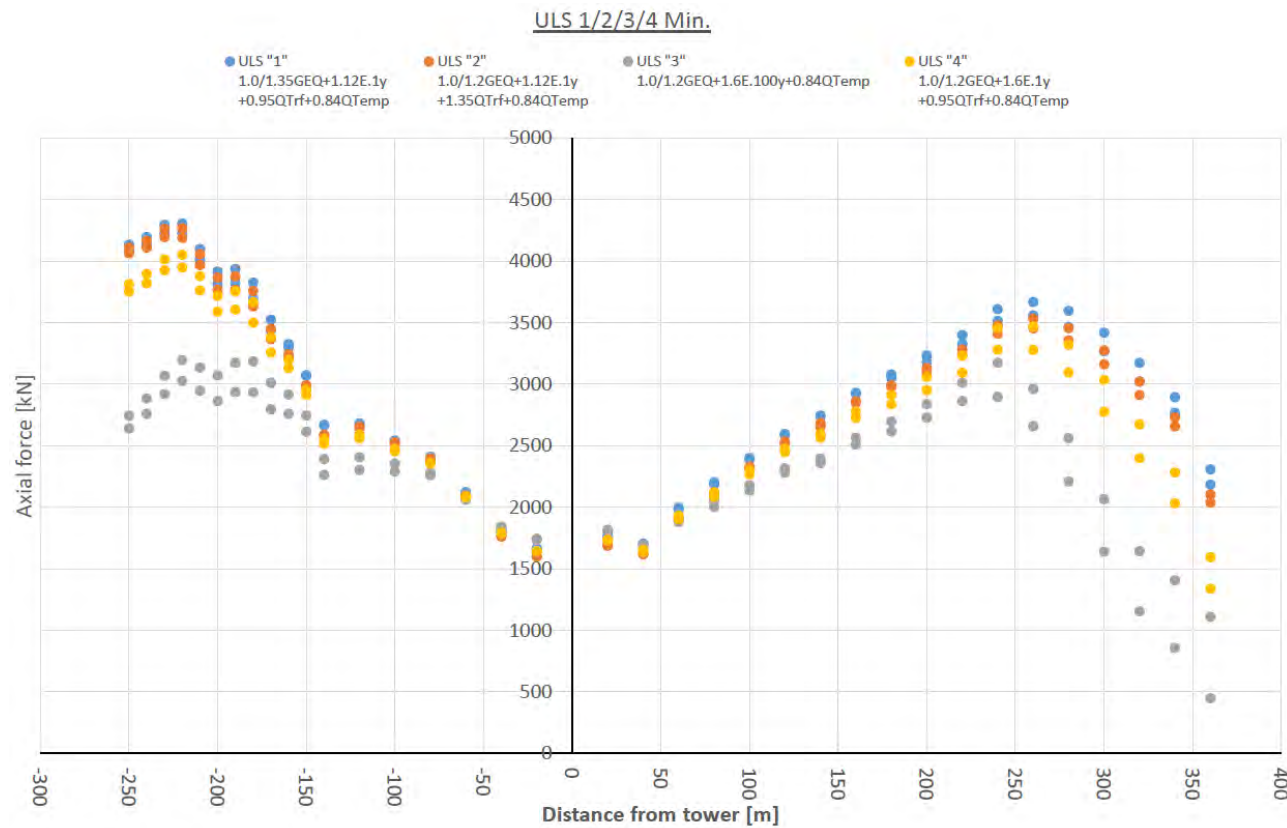*Figure 3-39 Stay cables. K12. Max ULS forces*
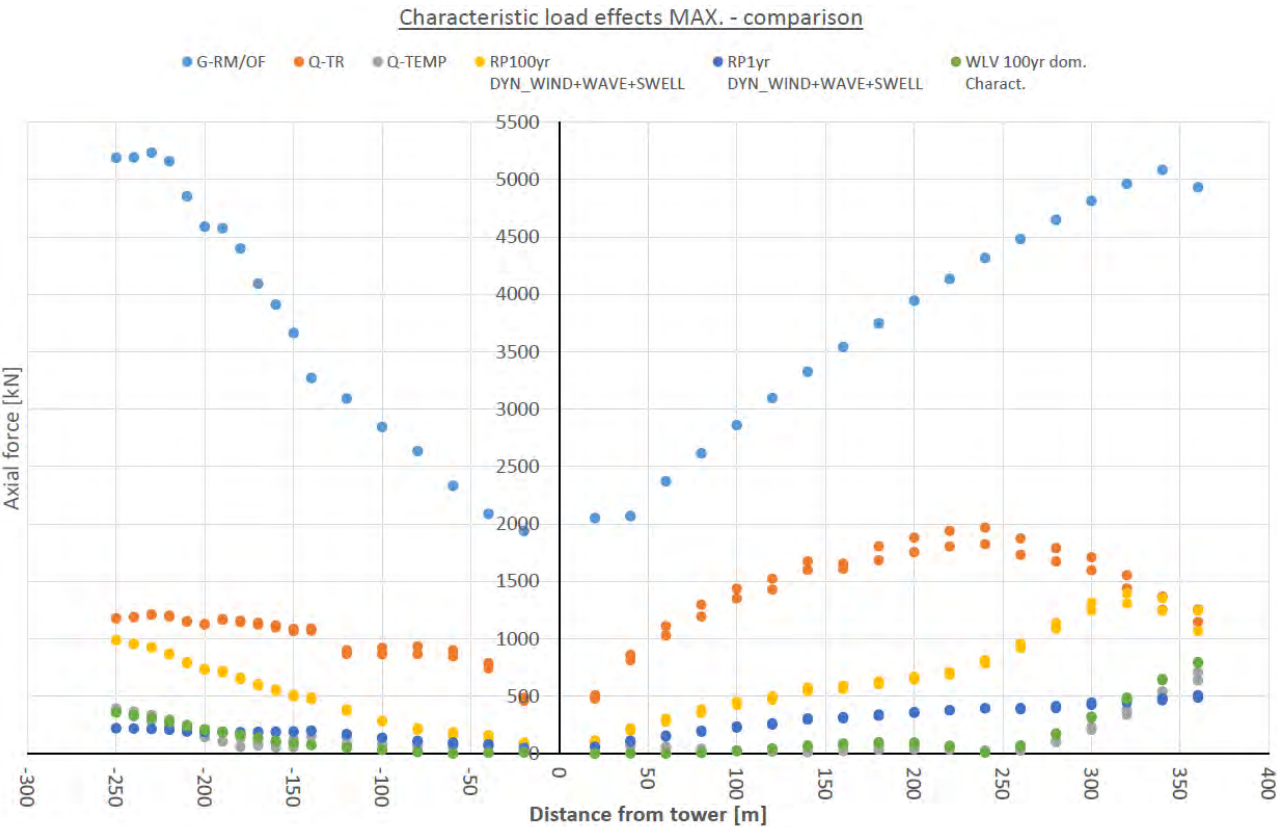

Figure 3-40 Stay cables. K12. Min ULS forces

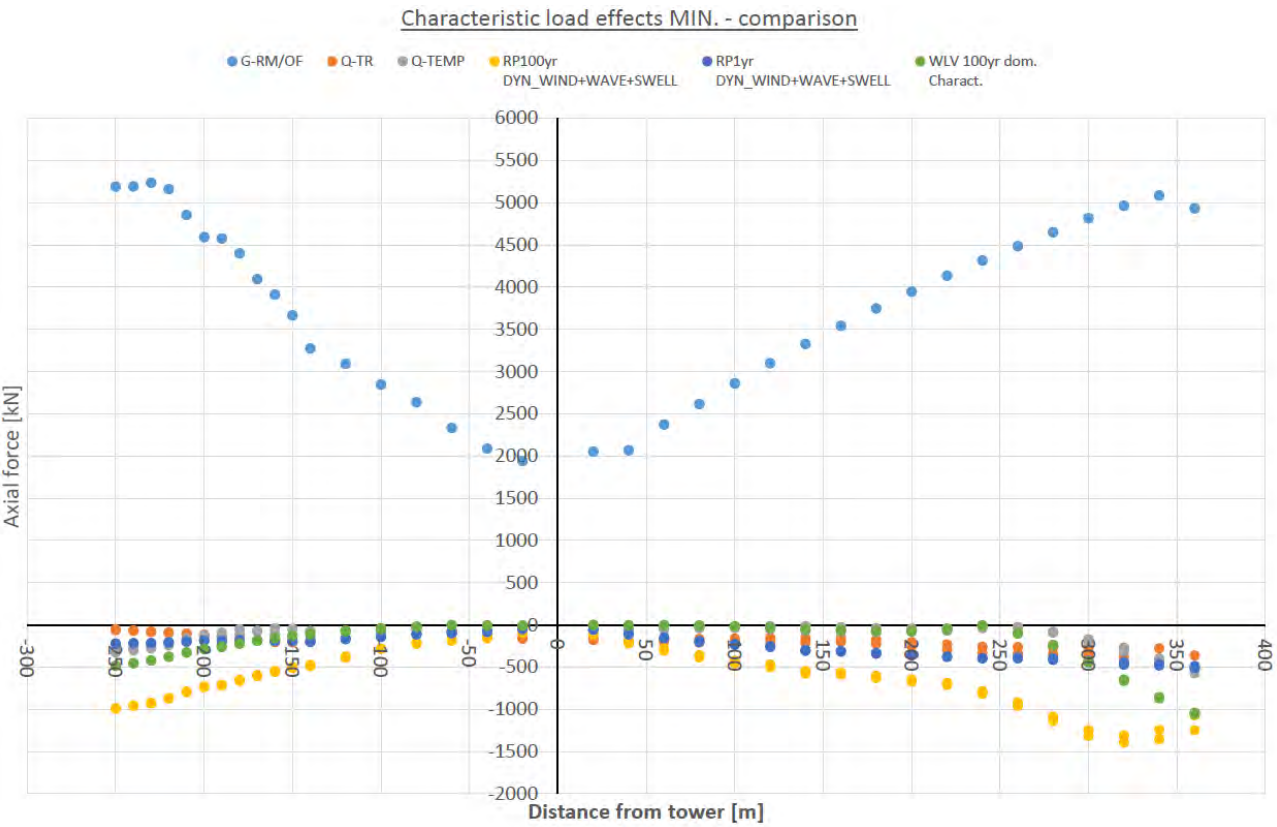*Figure 3-41 Stay cables. K12. Max characteristic load group forces*

Figure 3-42 Stay cables. K12. Min characteristic load group forces

4   Abutment calculations

4.1   Introduction

The section includes calculations for both abutments, however, only a brief extract of south abutment documentation. The north abutment is subjected to much higher load than the south abutment, that is sheltered by the tower and piers which reduces the strong axis moment.

4.2   North abutment

At this stage it is chosen to design an abutment which are stabilised by self weight only.

The necessary weight is decided by using a friction coefficient of 1.0 for the resulting shear from horizontal loads and moment about the vertical axis, (twisting moment), on an effective area, (length/width reduced with twice the eccentricity in each direction). The bottom plate is casted to scaled rock.

The stability is checked for both ULS and ALS ship impact. The ULS combination that gives the highest demand is ULS 3 (wind and wave dominant).

The joint between the floating bridge steel girder and the abutment concrete walls are prestressed to zero tension in the SLS condition.

Below is seen the max/min Mz moment, (about the vertical axis), for the north abutment, with the belonging other forces. Positive N and My, negative Vz, are unfavourable for the abutment. For Mz and Vy are used the absolute values.

Ship coll	N	Mz	My	T	Vz	Vy
Mz+	51	**5876**	346	64	-13	48
Mz-	-50	-5838	252	-92	-16	-44

ULS 3	N	Mz	My	T	Vz	Vy
Mz+	-30	**2393**	-8	77	10	-13
Mz-	25	-2381	596	-16	-11	-19

Units: meter and MN

Figure 4-1 North abutment - ULS3 and ALS ship collision, max/min M_z with belonging section forces

To find the necessary weight for the abutment the forces are transformed to the centre of the bottom plate, and the eccentricities and the effective area is calculated. Then the torsional modulus can be calculated, (Wt = (0.5*B.eff^2)x(Leff-Beff/3), and the shear stresses from torsion and lateral load can be found). At last the weight necessary is found. Below is shown the calculations.

B	L	Fx0	Mz0	My0	Mx0	Fz0	Fy0	Mx	My	Mz
m	m	MN	MNm	MNm	MNm	MN	MN			
38	52	51	5876	346	64	-13	48	537	1174	7124

ex	ey	Beff	Leff	Wt	tau-t	tau-f	Weight	Volume	hball	H=hball+2.6 m
m	m	m	m	m3	Mpa	MPa	MN	ballast	m	inkl. 1m slab.
2.0	0.9	36.2	48.0	23483	0.30	0.04	596	10149	7.3	10.0

(Mx=IMx0I+IFyoI*h, My=My0+Fx0*h-Fz0*L/2, Mz=IMz0I+IFy0I*L/2, h= abutment height)

The extrapolation from the end node of the floating bridge to the centre of the abutment may be a conservative assumption.

In above calculations iron ore with density 35 kN/m^3 is used. Abutment concrete weight is 241 MN.

The Vessel Traffic Management System under consideration may reduce the risk for ship collision and the ship impact forces. This in combination with a more no conservative approach for stability calculations than used above, may bring down the abutment dimensions and weight requirement.

The SLS condition with no tension in the joint is governing for prestressing between the steel girder and the abutment.

Tendons c/c 600 mm both in plates and bulkheads give space for about 170 tendons.

It is chosen to have 54 tendons 6-19 in the webs and 48 in the bottom slab. In top slab 48 tendons 6-22 for partly counteracting the permanent My moment. (6-19 means 19 strands 0.6 in diameter)

Total compression from prestressing is 536 MN after losses.
Below is shown the steel stresses in the joint for the SLS combinations.

SLS	N	Mz	My	sig P	sig N	sig Mz	sig My	sum
N+	39	-228	426	-141	10	8	70	-52
N-	-34	183	-51	-141	-9	7	8	-135
Mz+	4	1524	-80	-141	1	57	13	-70
Mz-	19	-1675	451	-141	5	62	74	0
My+	8	-428	698	-141	2	16	114	-8
My-	3	572	-281	-141	1	21	46	-73

For ship collision the utilization for the most stressed tendon is 0.90, (1339 MPa*1.1/1640 MPa), with additional strain from the load 0.14 %, (total strain 0.74 %).
The ALS capacity is determined by the concrete. At Mz=8100 MNm, (1.4 x collision load), the concrete utilization is 1.0, and the tendon strain 1.1 %, (limit 2.4 %).

DETAIL 1, DR-202
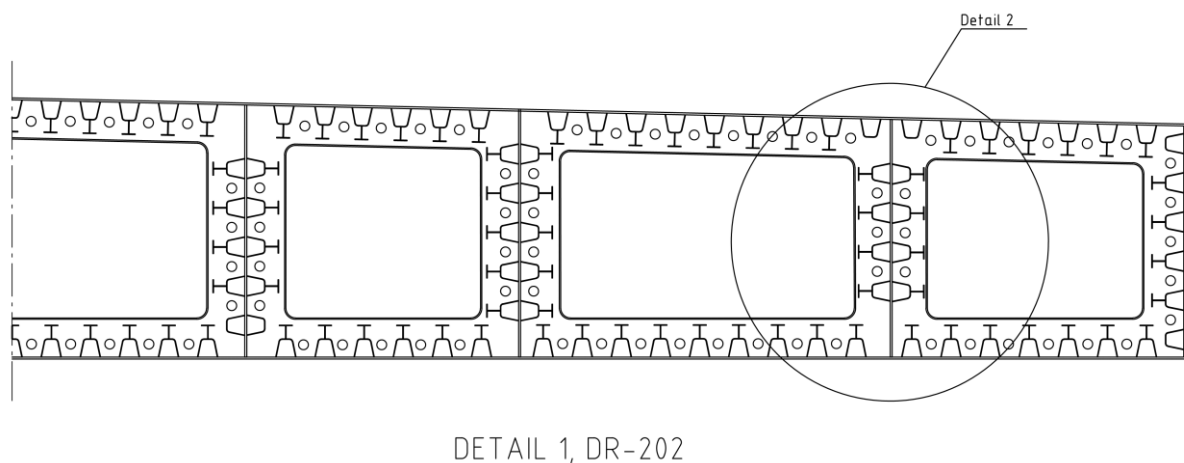
Figure 4-2 Joint floating bridge steel girder/concrete abutment
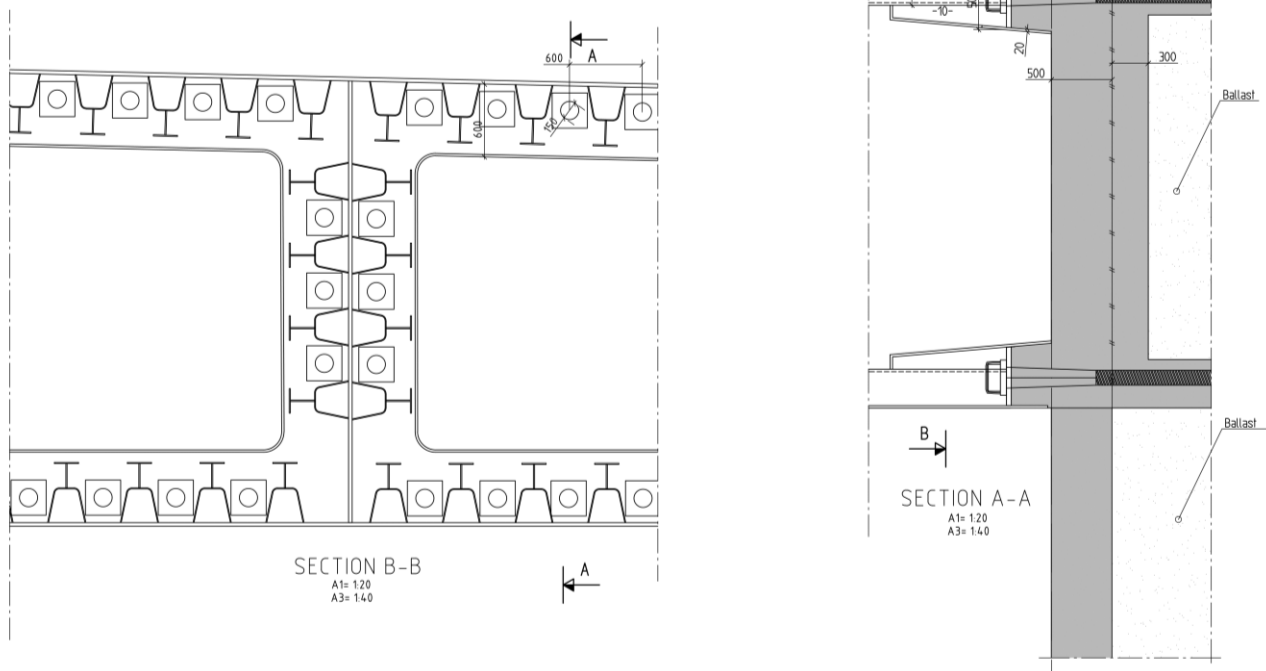
Figure 4-3 Details, joint floating bridge steel girder/concrete abutment

4.3   South abutment

The forces for the south abutment are much less than for the north one.

Ship impact south, (permanent loads not included)

ALS	N	Mz	My
N+	74	156	6
N-	-75	-362	-12
Mz+	44	606	16
Mz-	-39	-770	-10
My+	22	352	19
My-	-28	-462	-22

ULS-3	N	Mz	My
N+	51	9	88
N-	-82	-7	114
Mz+	-10	24	111
Mz-	-23	-19	93
My+	1	13	128
My-	-1	-10	72

From above loads the abutment could have been about 3 meters shorter than the length drawn. However, the loads are for the case that piers in the side span are fixed to the bridge which very effective damping out the effect from loads on the floating bridge. The height is suited to the terrain and the width determined by the bridge. From this it is chosen to keep the dimensions.

5   Wind analyses of construction stages

Wind analyses of critical construction stages of the cable stayed bridge is performed with NovaFrame.

The following four stages are considered critical and evaluated:
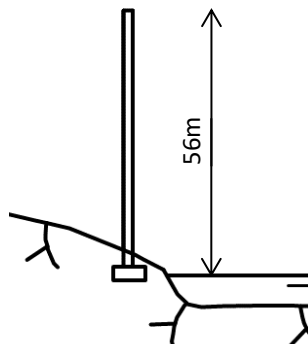
Figure 5-1 Stage A1. *The last stage of casting the pier in axis 1E*

The axis 1E pier with a total height of 56m casted. Climbing formwork at the top.
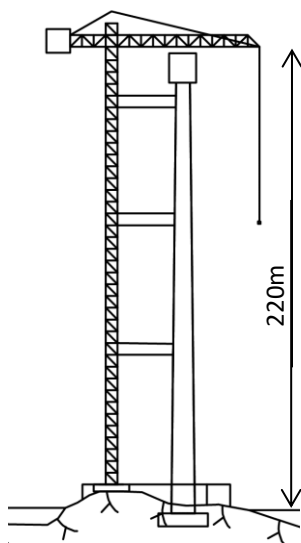
Figure 5-2 *Stage A2: The last stage of casting the tower in axis 2*

Tower fully casted up to elevation z=220m. Climbing formwork at the tower top. Crane and lift mounted to the tower.
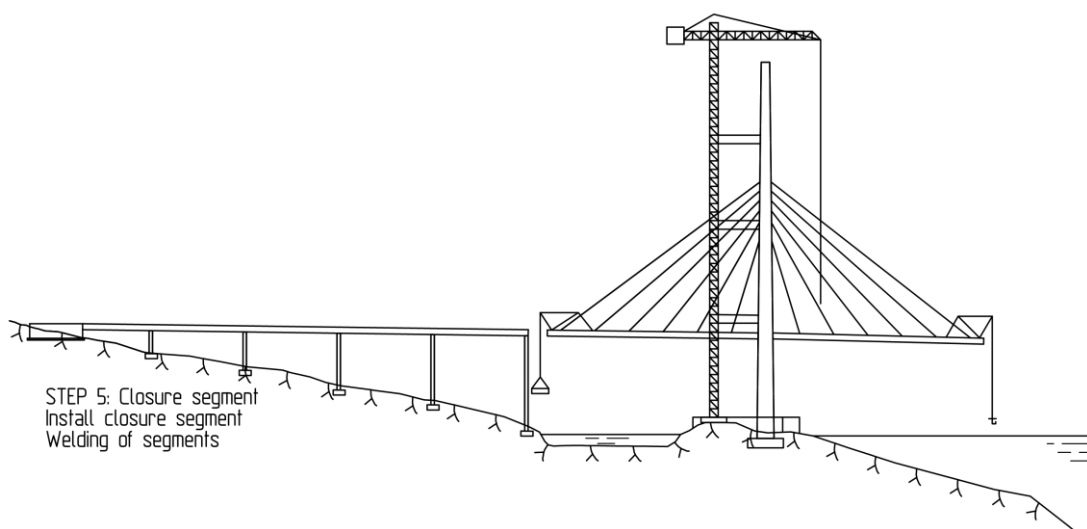
Figure 5-3 Stage B Cantilevered situation of cable stayed bridge before closing at the side span

Derrick cranes at the end of both 130m cantilevers. Crane and lift mounted to the tower is also assumed, but this is not shown in the figure above.
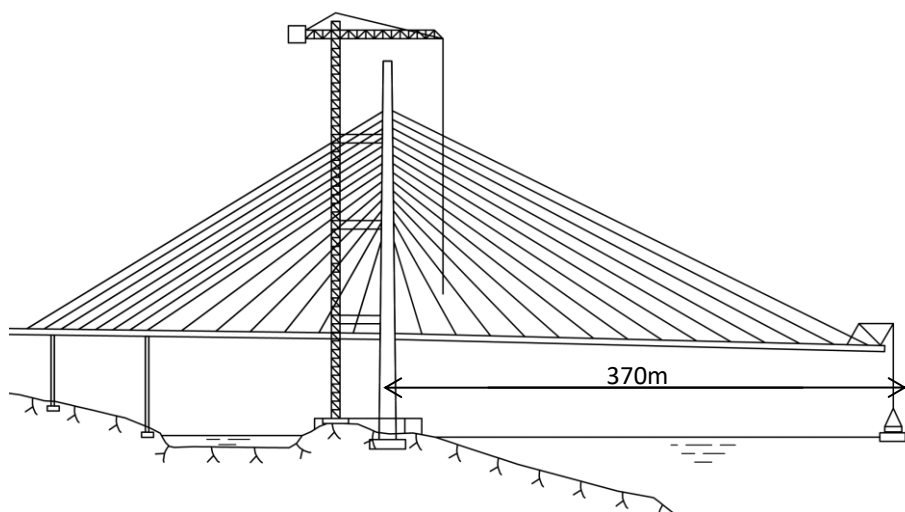
Figure 5-4 Stage C. Cantilevered bridge girder in main span before closing in axis 3

Main span cantilever 370m. No special equipment included. Instead it is chosen to include 100 years return period wind loads due to the assumption that this stage may last for a considerably longer period of time compared to the other stages. 10 years return period is used in general for construction stages.

Included impacts on these stages are:

- Dead loads and pre-stressing of stay cables (included in a simplified manner)

- Static wind loads

- Dynamic buffeting wind loads

All wind directions are analysed with the P-Delta effect taken into account. Deformation loads and secondary forces from previous construction stages are neglected. Imperfections are also neglected.

However, it is chosen to include an unbalance of permanent load by increasing the self weight of the bridge girder in main span by 5 % for erected situations.

Effect of cracking is included by performing additional analyses with a reduced E-modulus, $E_{cracked}$ = 0.4 x 30 000 MPa, for all tower elements in stages A2, B and C, and for all side span pier elements in stage A1. Stability checks with cracked stiffness are performed for concept verification. However, section capacity checks with cracked stiffness are considered a matter of detailed reinforcement design. Additionally, it is seen that the cracked stiffness of 40 % will not have a significant impact on the total forces due to the significant increased structural damping. Reference is made to the stability checks. Thus, cracked stiffness is not considered for capacity checks in this phase.

Loads and load factors are in accordance with the Design basis.

Enclosure L3 describes the models and analyses in detail, and section forces, both ULS max/min and characteristic forces, are presented. Capacity and stability checks due to forces resulting from this Enclosure are included in Enclosure L1 and Enclosure L2 respectively.

A summary of the most important results is presented in the following.

Tower: Capacity checks of critical sections and foundation stability are performed. Constructions stages are governing for the tower.

Stay cables: Constructions stages will not be governing for the cables as the maximum ULS force in the longest cable in main span is ca. 8 MN.

Side span piers: Capacity checks of critical sections and foundation stability are performed. Constructions stages are governing for the axis 1E pier.

Bridge girder (steel): Maximum bending moment about strong axis is 1700 MNm at axis 2 for stage C, and maximum bending moment about weak axis is 55 MNm. Thus, construction stages are not governing for the steel bridge girder.

Bridge girder (concrete): Capacity checks of critical sections are performed, but the construction stages are not governing.

Table 5-1 Construction stage summary – values shown are not showing interdependent forces

Construction stages - summary

Phase	Limit state	Criteria	Criteria2	Axial [MN]	BMTA [MNm]	BMLA [MNm]
Tower leg west	Phase a	ULS	Max	-70	385	106
			Min	-160	-958	24
	Phase b	ULS	Max	-90	552	127
			Min	-186	-943	28
	Phase c	ULS	Max	-104	276	243
			Min	-289	-473	-28

Leg at bottom

Phase	Limit state	Criteria	Criteria2	Axial [MN]	BMTA [MNm]	BMLA [MNm]
Tower leg west	Phase a	ULS	Max	-24	269	-23
			Min	-103	-604	-126
	Phase b	ULS	Max	-43	369	-44
			Min	-130	-625	-173
	Phase c	ULS	Max	-57	65	11
			Min	-233	-157	-459

Leg at crossbeam

6   Verification calculations

The section includes verification calculations and analyses, which work as control and input for the global analysis models.

6.1   Local analyses

In this subchapter, two local analysis models of the free standing tower (FST) and free standing high bridge (FSHB) are outlined with mode shapes, buckling modes, permanent loads and simplified tidal load and static ship impact analyses. These two analysis models also work as control and input for the global analysis models.

The total permanent load of the bridge is deck is assumed as 19 ton/m = 186.3 kN/m for the steel part and 79.1 ton/m = 776 kN/m for the concrete part. The analysed bridge girder properties are A=1.48/27.95 m^2, I-weak=3.35/40.50 m^4 and I-strong=101/2138 m^4 for the steel and concrete part respectively. The analysed side span piers are connected monolithically to the bridge girder and foundation and have a rectangular section 8m*1.5 m. The piers are modelled with lengths 11, 21, 32, 42 and 54 m.

The cross sections of the tower are modelled along the centre of gravity with linear interpolation of cross section properties between the nodes. The tower section properties are shown below:
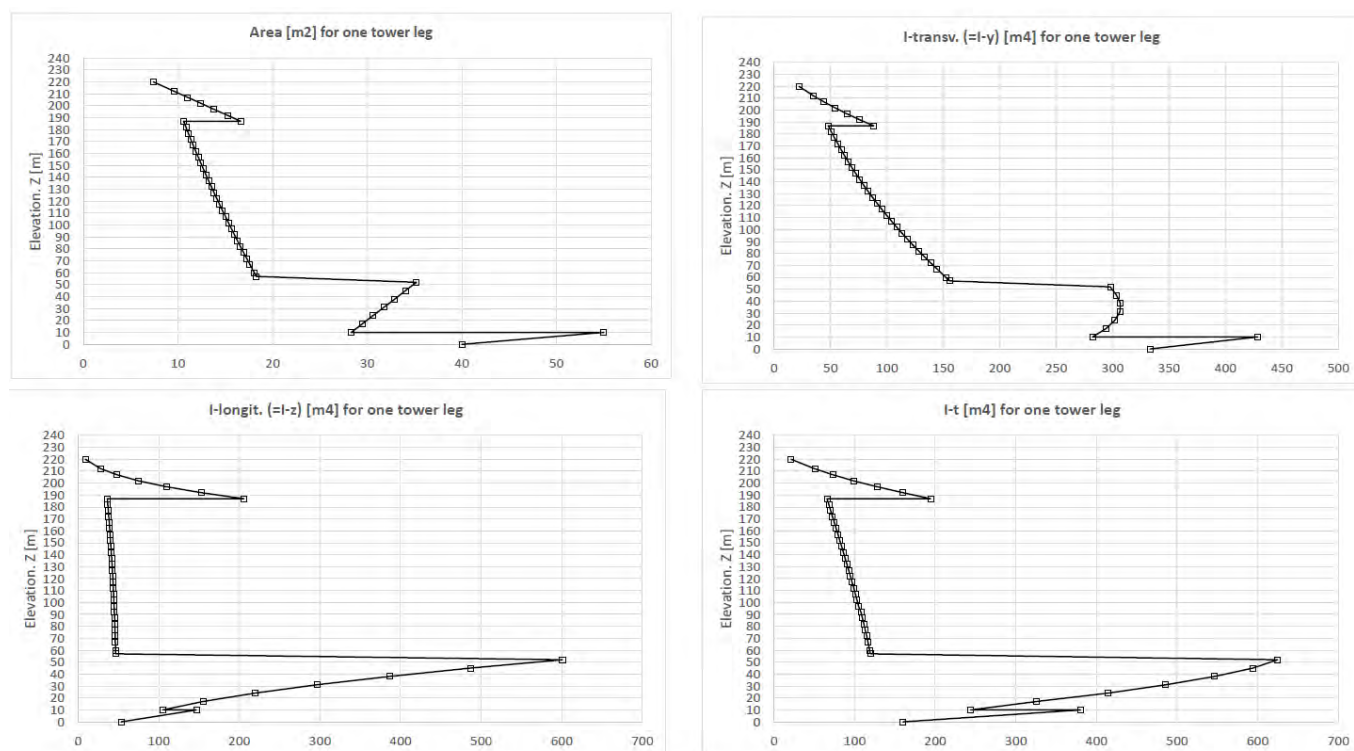
Figure 6-1 Tower - cross section properties, for one leg where applicable

The discontinuity regions where the lower and upper tower legs meet the cross beam are modelled with kinematic constraints.
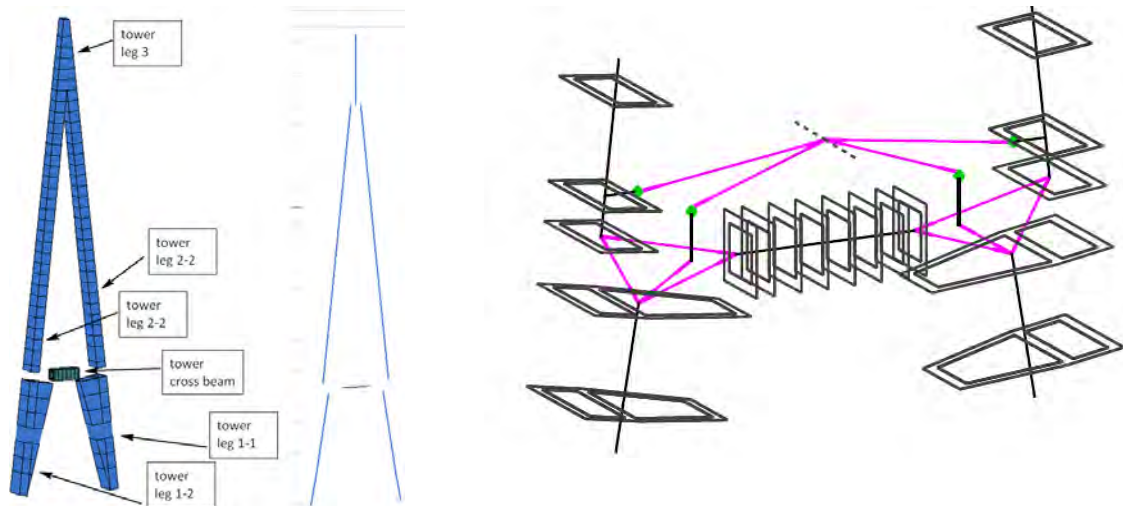
Figure 6-2 Tower - discontinuity region model

6.1.1   Free standing tower model

Permanent loads
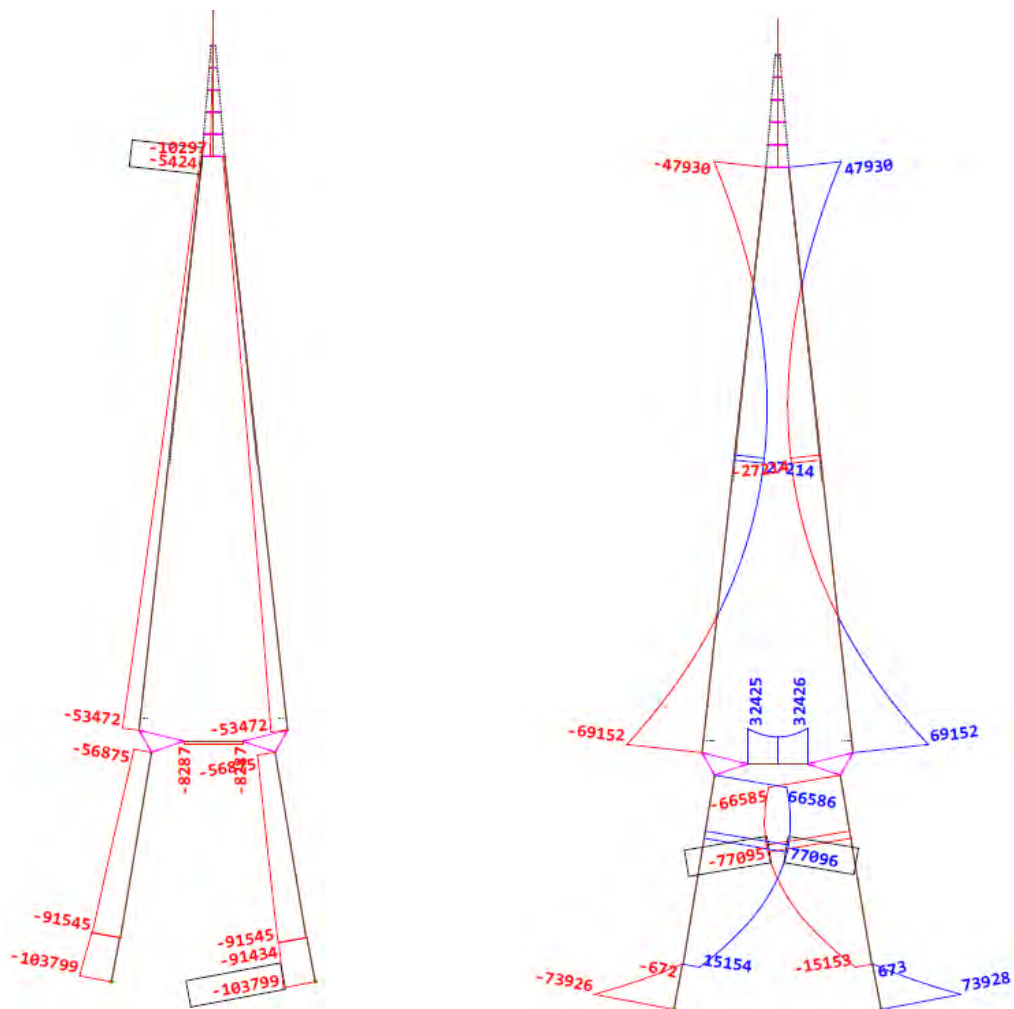
Figure 6-3 FST. Permanent loads. Left: Axial force [kN]. Right: Bending moment [kNm]
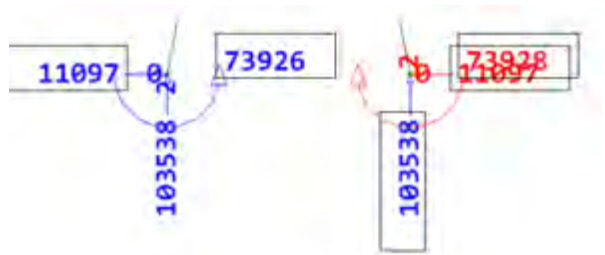
Figure 6-4 FST. Permanent loads. Reactions [kN, kNm]

Mode shapes

Mode 1. f=0.21Hz. X-sway.

Mode 2. f=0.58Hz. Y-sway

Mode 4. f=1.22Hz. Z-rotation
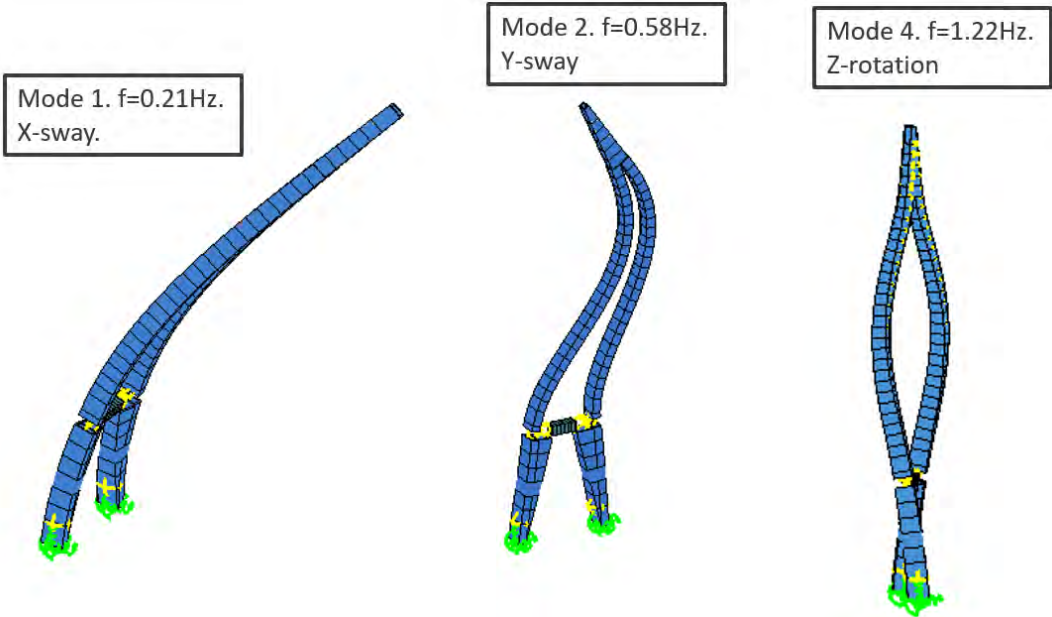
Figure 6-5 FST. Mode shapes and natural frequencies

Buckling modes

Mode 1. f=21.3 X-sway

Mode 2. f=67.7 Y-sway.
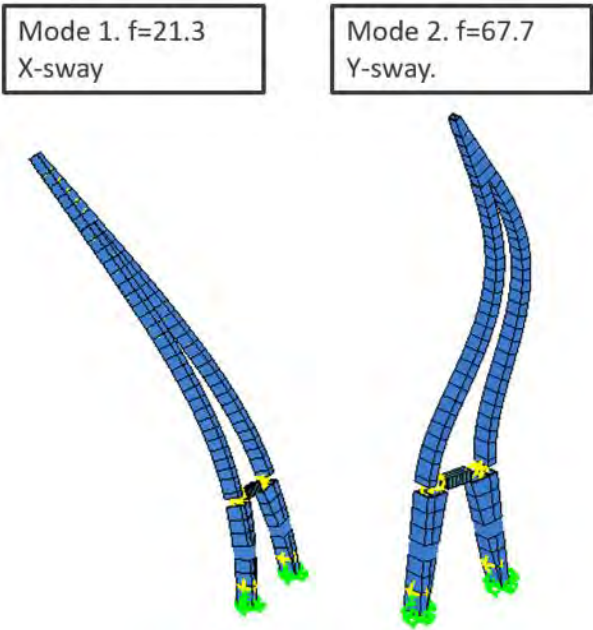
Figure 6-6 FST. Buckling modes and critical factors (N$_{cr}$/N$_{Ed}$)

6.1.2 Free standing high bridge

In this model it is assumed that the stiffness and loads of all components are activated in one step.
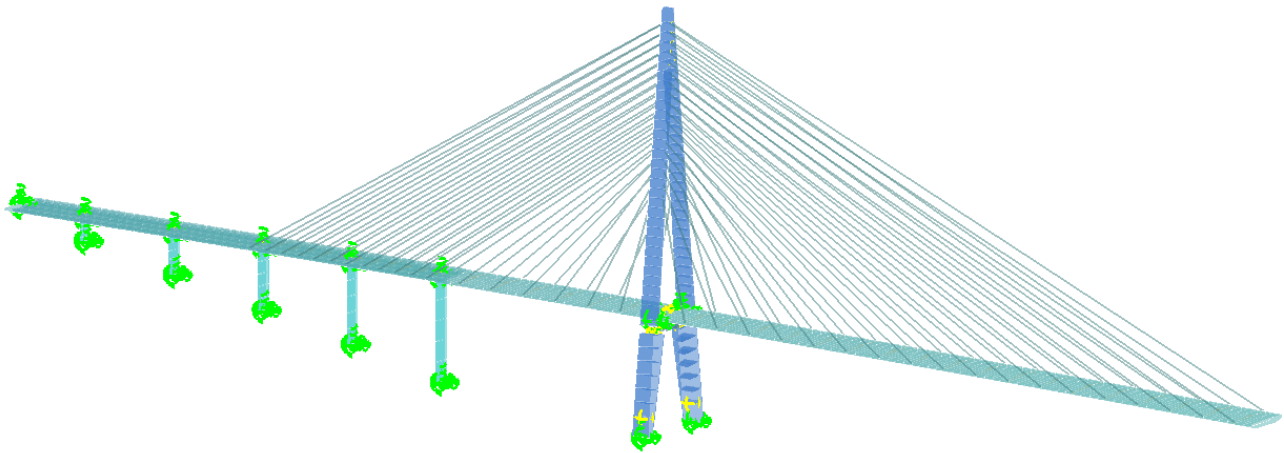
Figure 6-7 FSHB. Local FE-model

Permanent loads
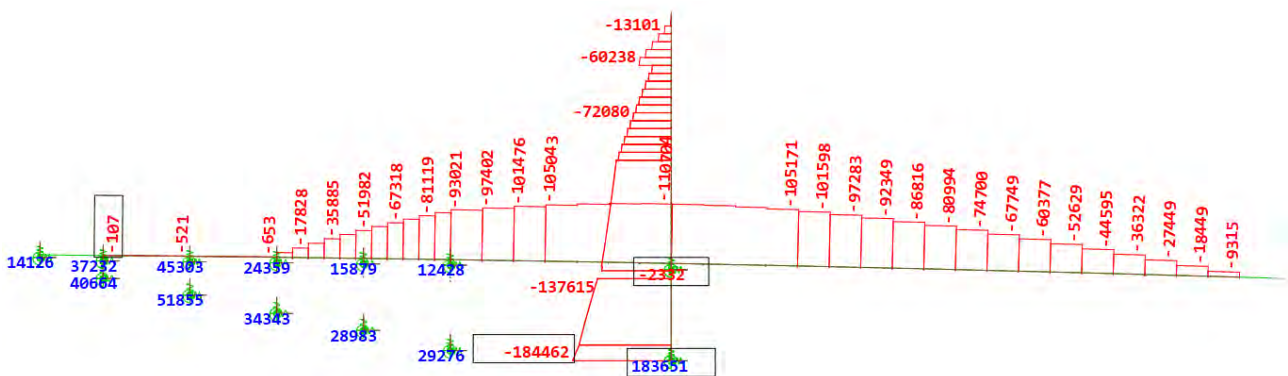
Figure 6-8 FSHB. Permanent loads. Axial forces and vertical bearing and reaction forces [kN]
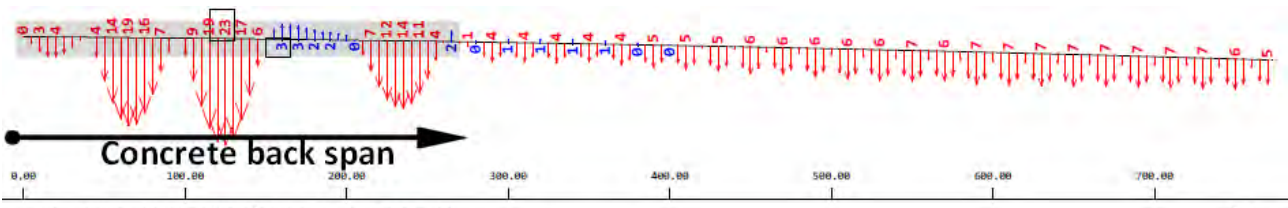
Figure 6-9 FSHB. Permanent loads. Vertical displacement bridge girder [mm]
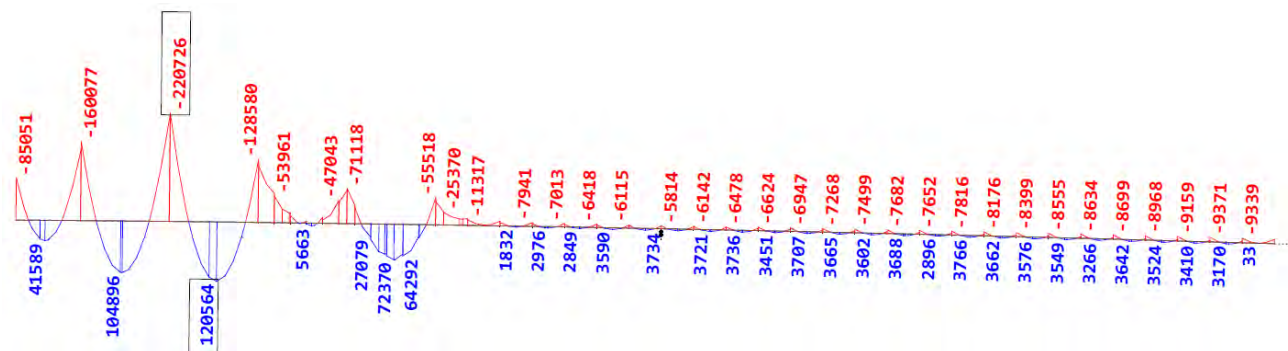
Figure 6-10 FSHB. Permanent loads. Weak axis bending moment [kNm]
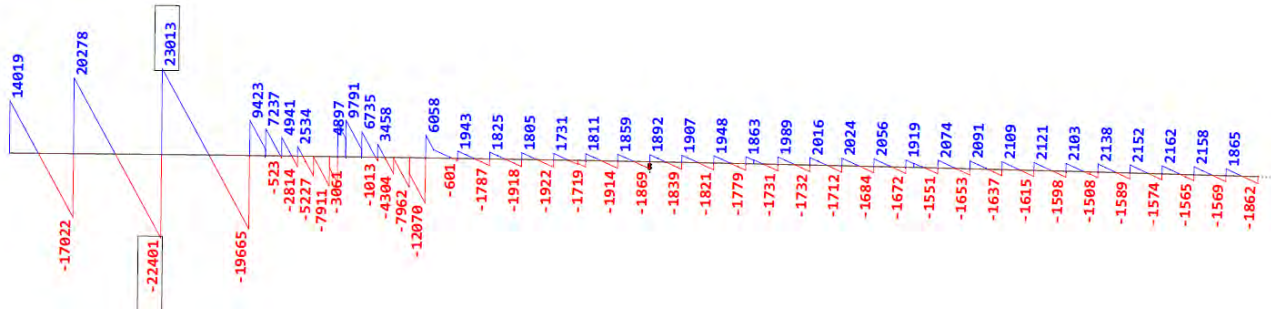
Figure 6-11 FSHB. Permanent loads. Vertical shear force [kN]
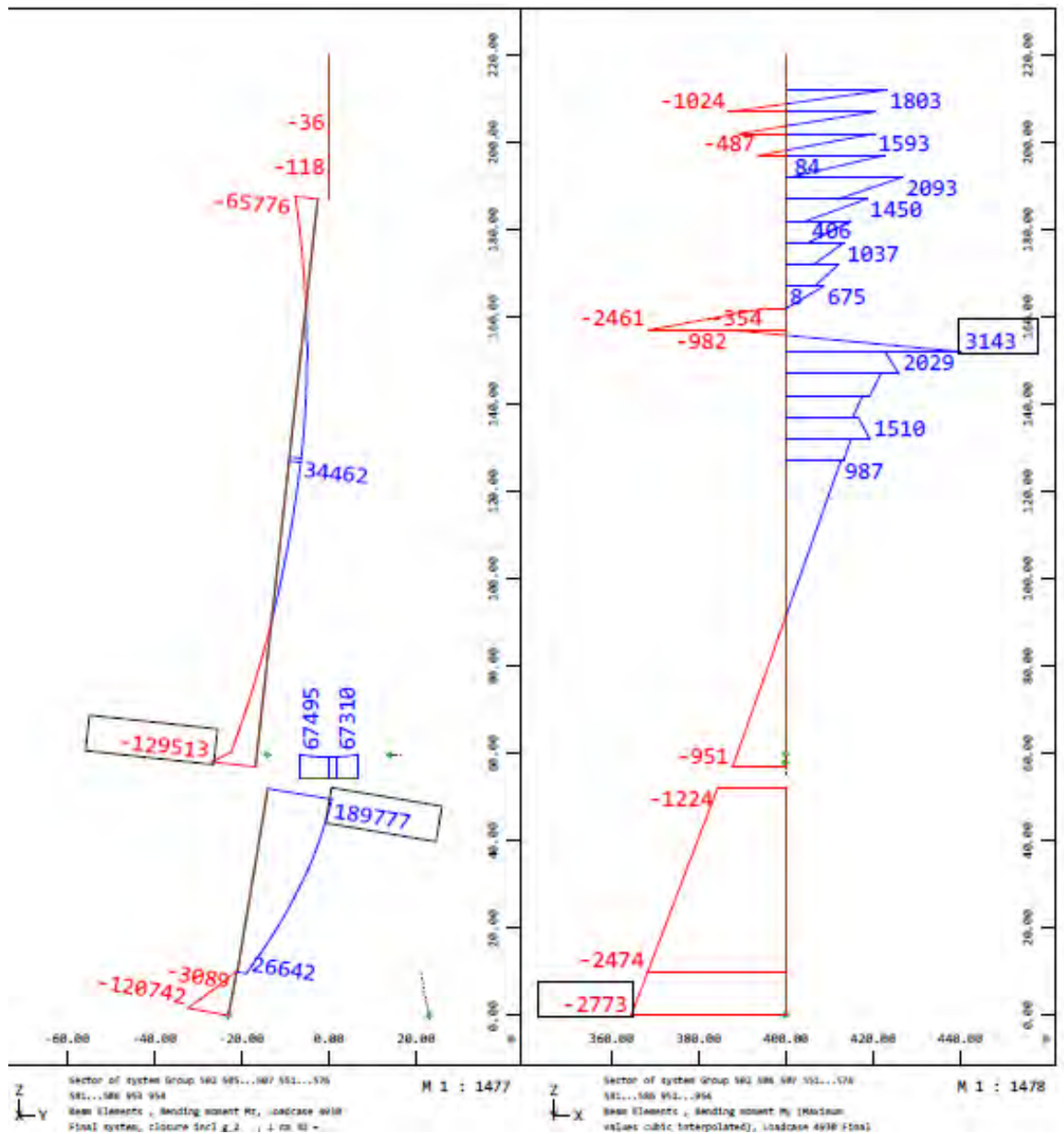
Figure 6-12 FSHB. Permanent loads. Tower moments [kNm]
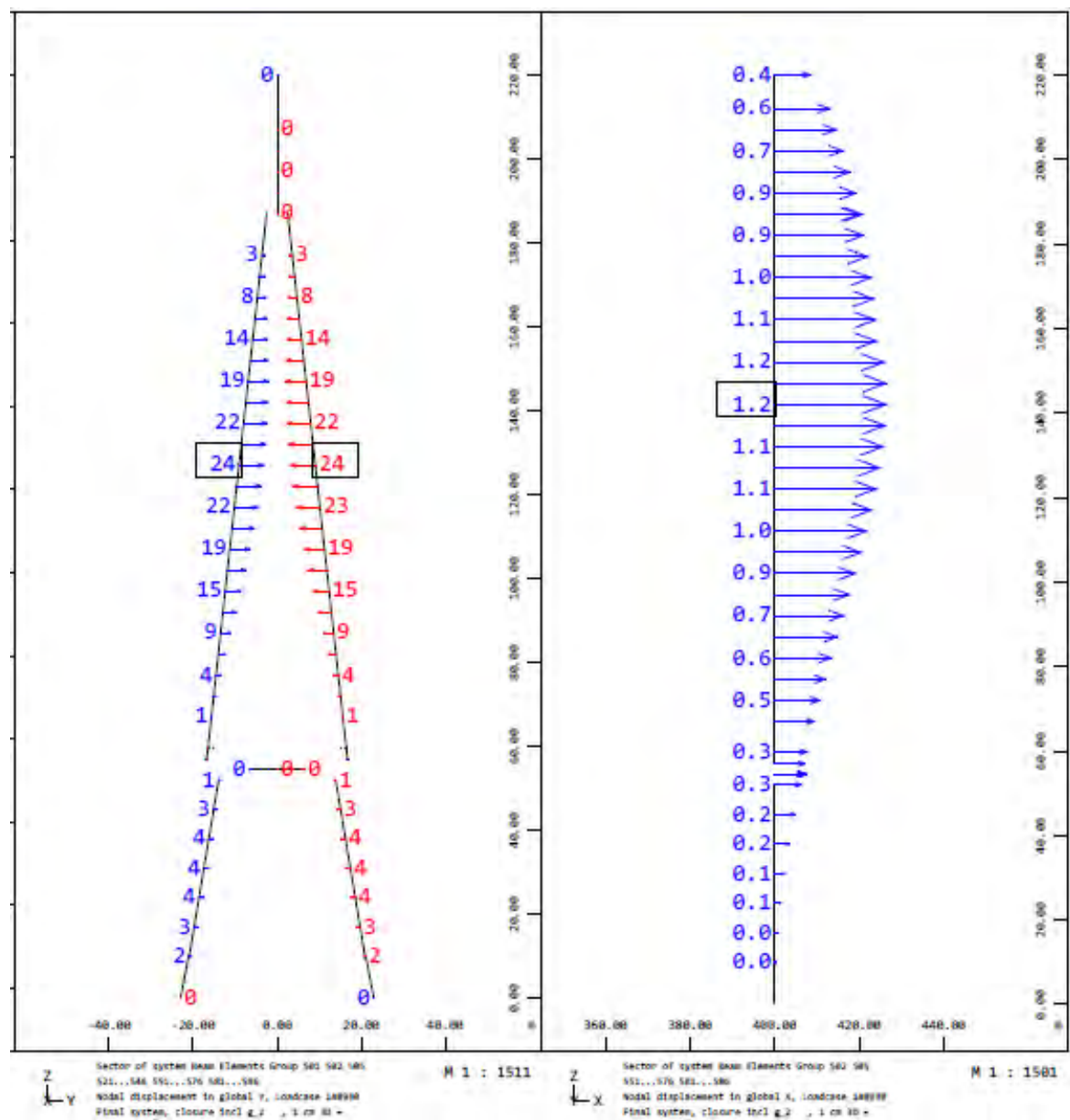
Figure 6-13 FSHB. Permanent loads. Tower horizontal displacements [mm]
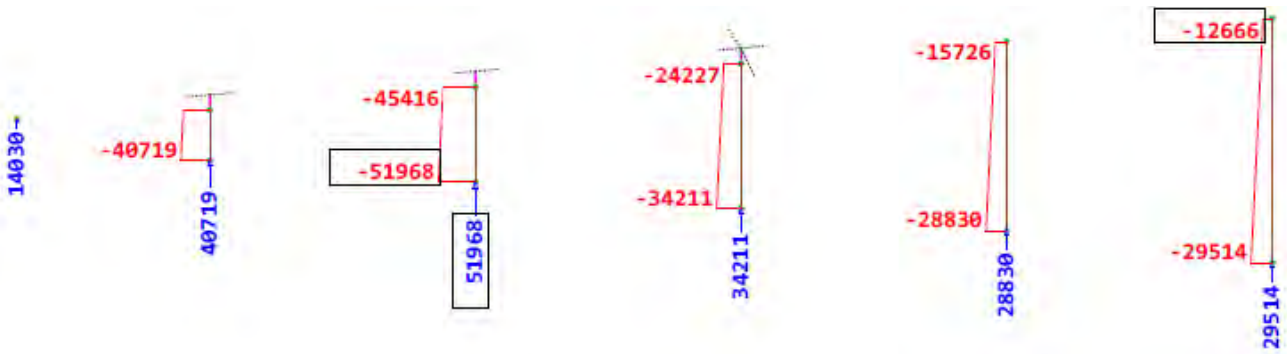
Figure 6-14 FSHB. Permanent loads. Side span piers and abutment. Axial force and reactions [kN]

Figure 6-15 FSHB. Permanent loads. Side span piers and abutment. Bending moments [kNm]

Tide simulation

A unit load which gives 1 m vertical displacement at the tip of the main span cantilever is applied to evaluate how tidal loads affect the high bridge.

The maximum stay cable force in the main span (outer cable, 18) is +-1300 kN

The maximum stay cable force in the side span (outer cable, 18) is +-531 kN
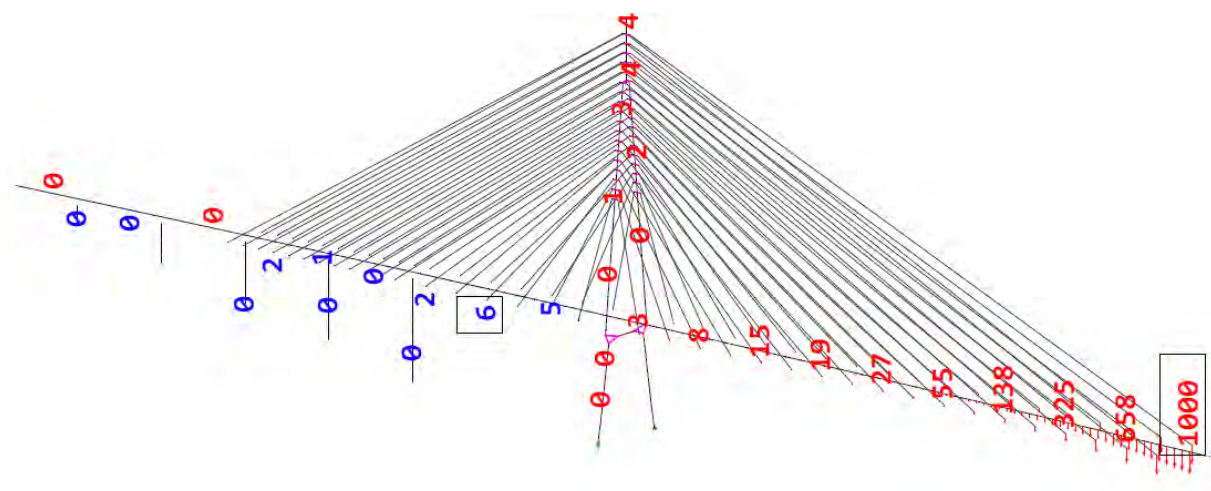
Figure 6-16 FSHB. Tide simulation. Vertical displacement [mm]
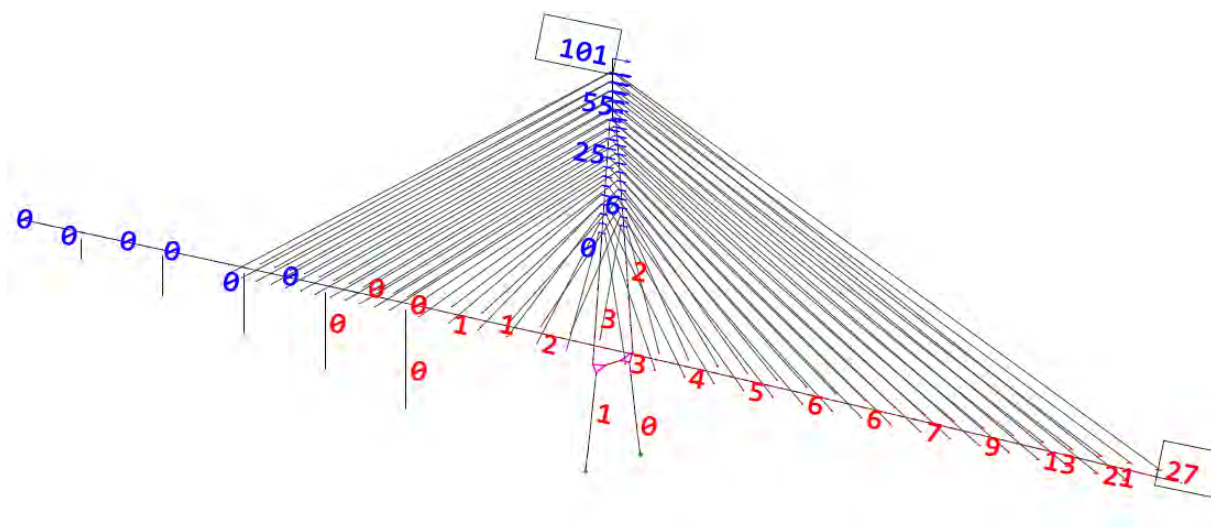
Figure 6-17 FSHB. Tide simulation. Longitudinal displacement [mm]
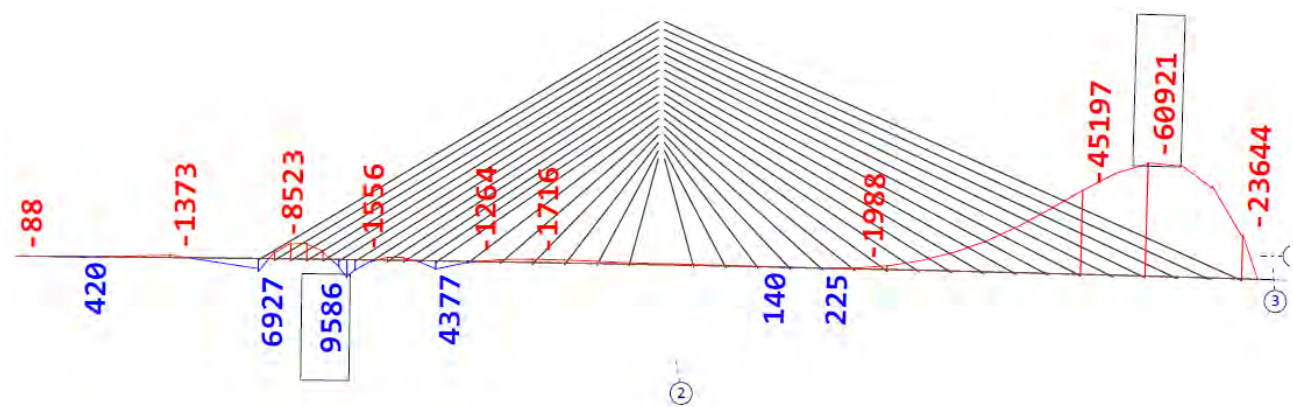
Figure 6-18 FSHB. Tide simulation. Weak axis bending moment [kNm]
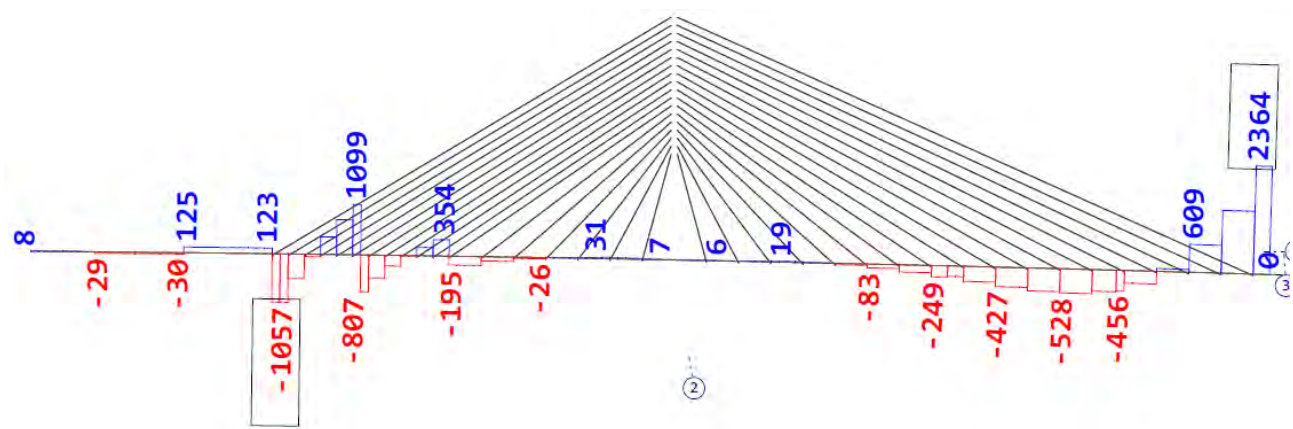
Figure 6-19 FSHB. Tide simulation. Vertical shear force [kN]
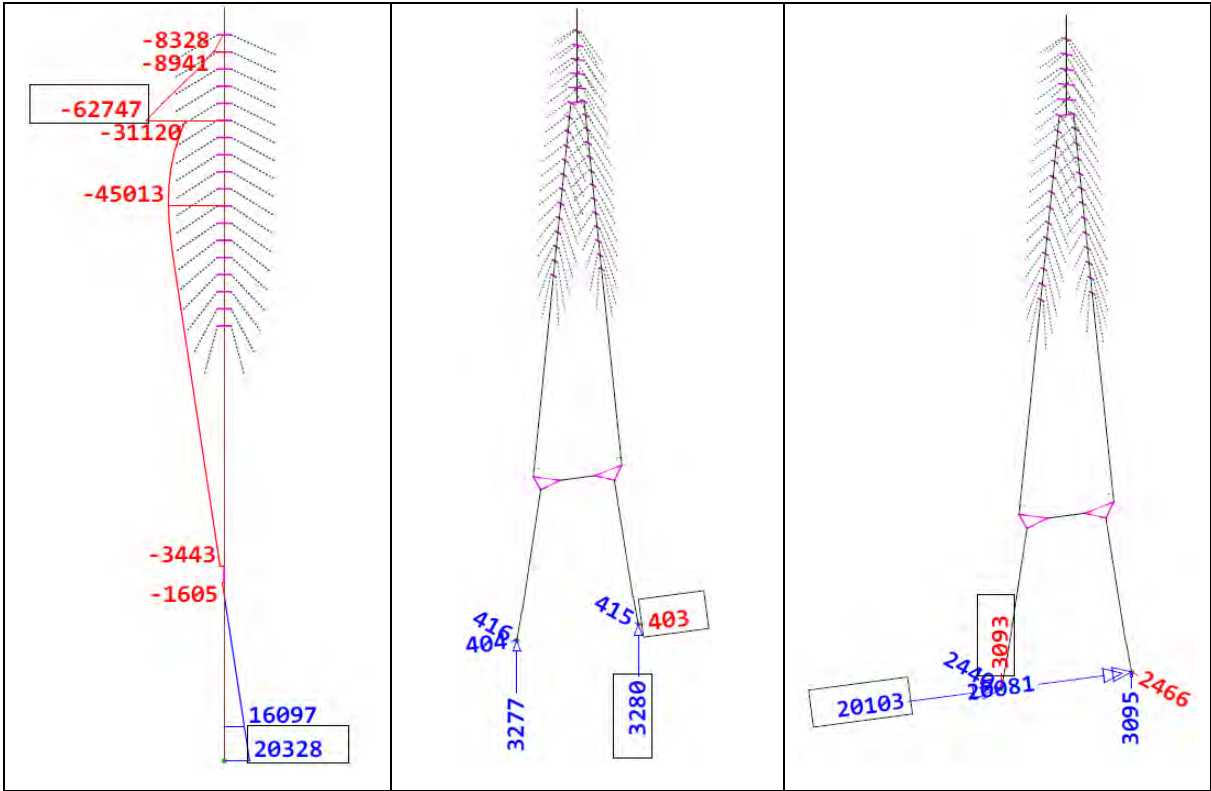
Figure 6-20 FSHB. Tide simulation. Tower. Left: Transverse bending moment [kNm]. Mid: Force reactions [kN]. Right: Moment reactions [kNm]

Static ship impact simulation

A strong axis bending moment of 3000 MNm and a transverse force of 20 MN are applied at the tip of the main span cantilever to evaluate how ship impact forces are distributed to the high bridge, assuming static behaviour. This is a rough simplification and shall thus not be mistaken as design forces. Nevertheless, it gives a good understanding of the stiffness distribution of the high bridge and is used for sensitivity study for side span pier section types, concrete cracking and potential use of bearings.

The results below are the result of non-linear analysis including permanent loads. The non-linear effects are geometric non-linearity and the two transverse tower bearings only carrying compression. There is no material non-linearity in the model. All stiffness is calculated using the nominal E-modulus.

The maximum stay cable force increase/decrease with respect to the permanent load is +/-730 kN (approximately the same in both side and front span).
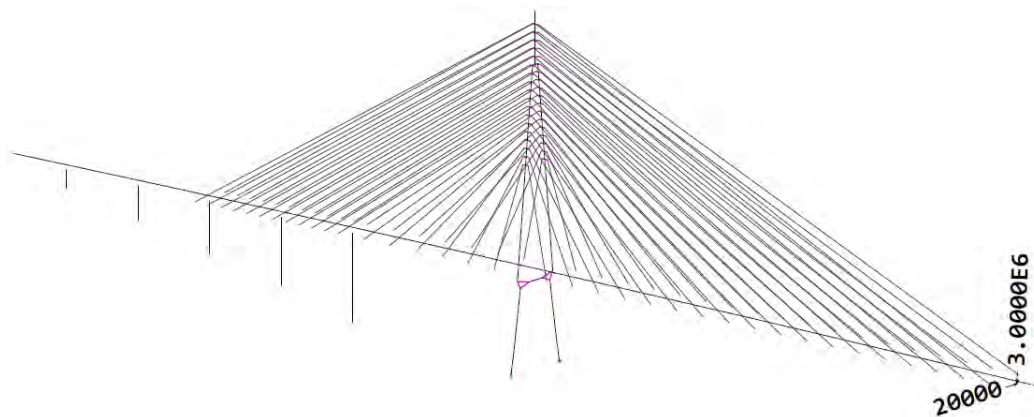

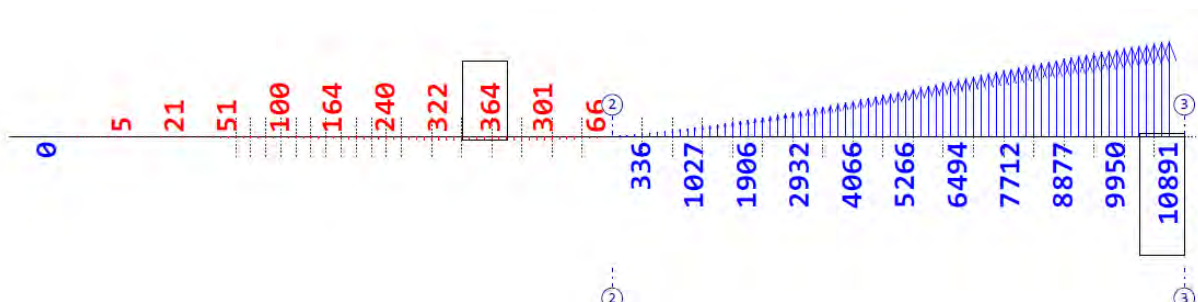*Figure 6-21 FSHB. Ship impact simulation. Load application [kN, kNm]*

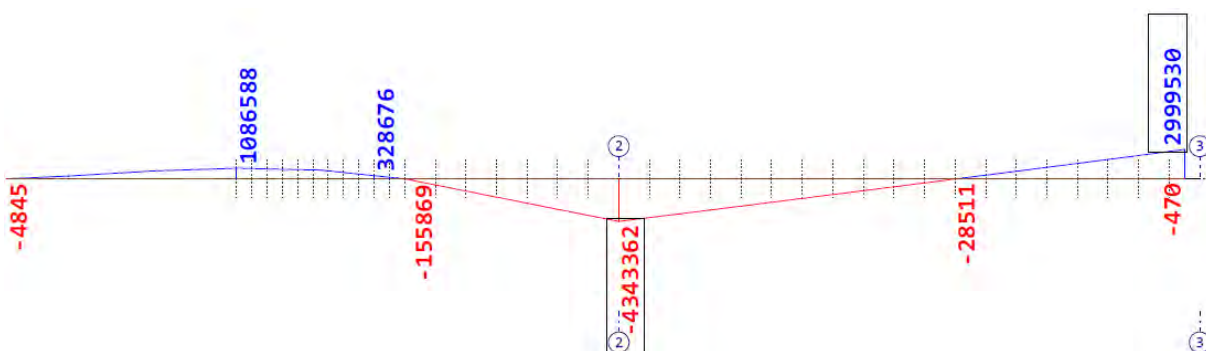*Figure 6-22 FSHB. Ship impact simulation. Transverse displacement [mm]*

Figure 6-23: FSHB. Ship impact simulation. Strong axis bending moment [kNm] Results not updated to reflect final global analyses which shows bending moment at tower of max 3300 MNm
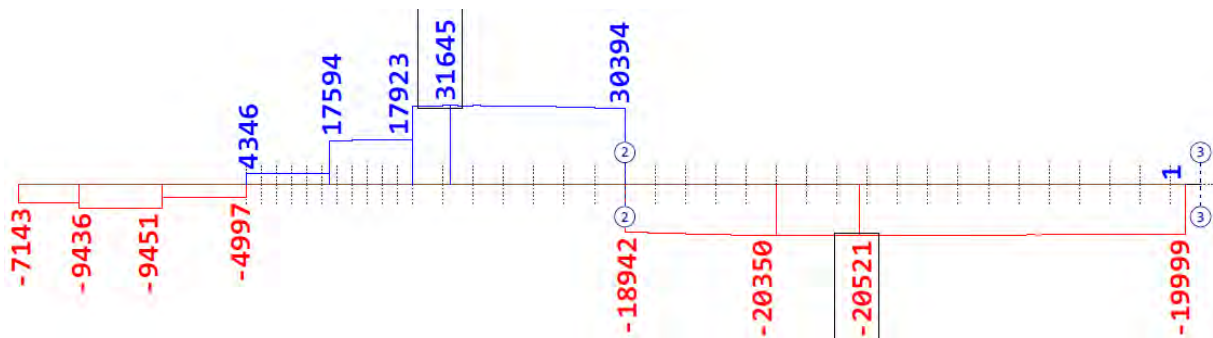
Figure 6-24 FSHB. Ship impact simulation. Transverse shear force [kN] Results not updated to reflect final global analyses which shows shear force at tower of 15 MN against side span & 21 MN against main span
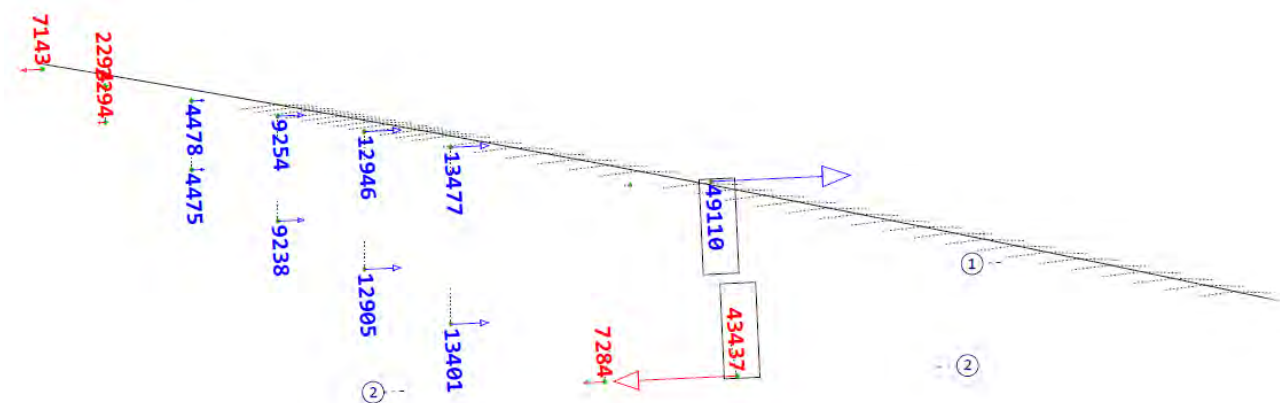

Figure 6-25 FSHB. Ship impact simulation. Transverse bearing and reaction forces [kN]
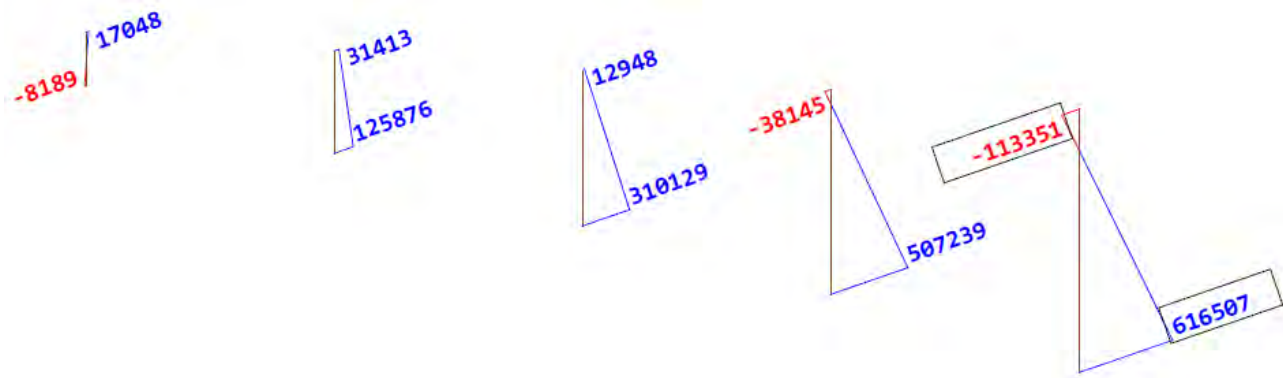

Figure 6-26 FSHB. Ship impact simulation. Side span piers. Longitudinal axis bending moment [kNm]

Figure 6-27 FSHB. Ship impact simulation. Side span piers. Axial force including permanent loads [kN]
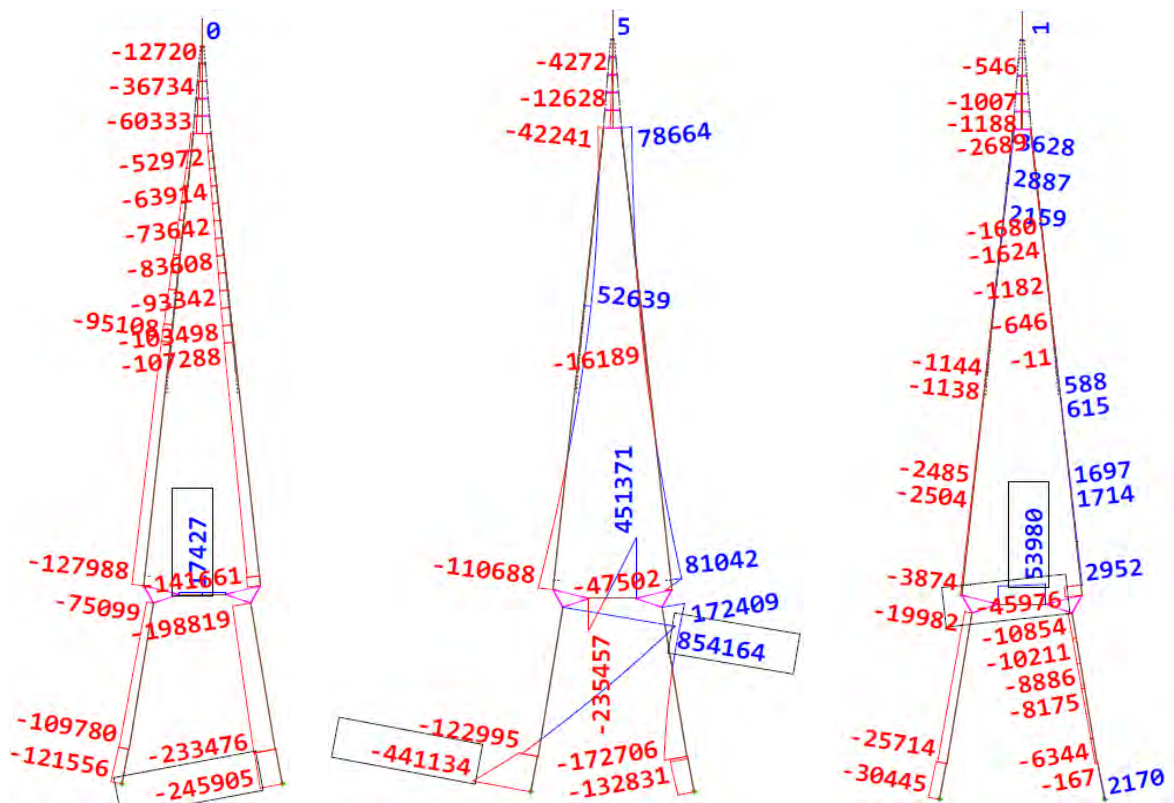
Figure 6-28 FSHB. Ship impact simulation. Tower. Including permanent loads. Left: Axial force [kN]. Mid: Longitudinal axis bending moment [kNm]. Right: Transverse shear force [kN]
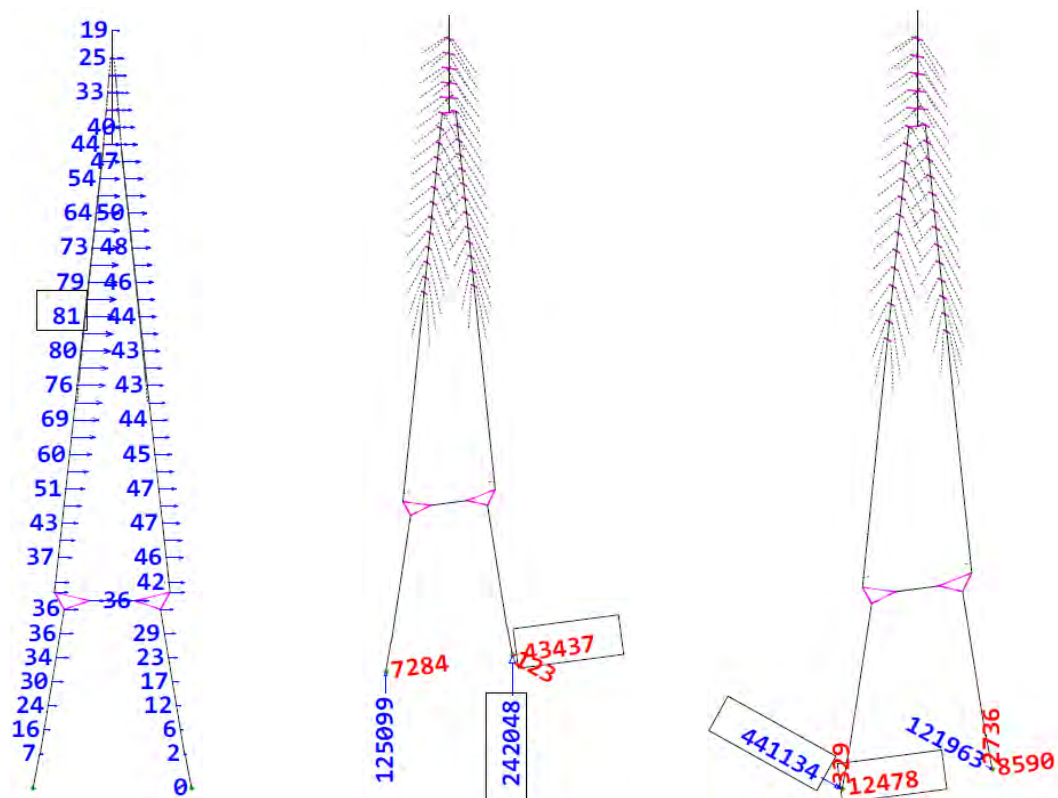

Figure 6-29 FSHB. Ship impact simulation. Tower. Including permanent loads. Left: Transverse displacement [mm]. Mid: Reactions forces [kN]. Right: Reactions moments [kNm]
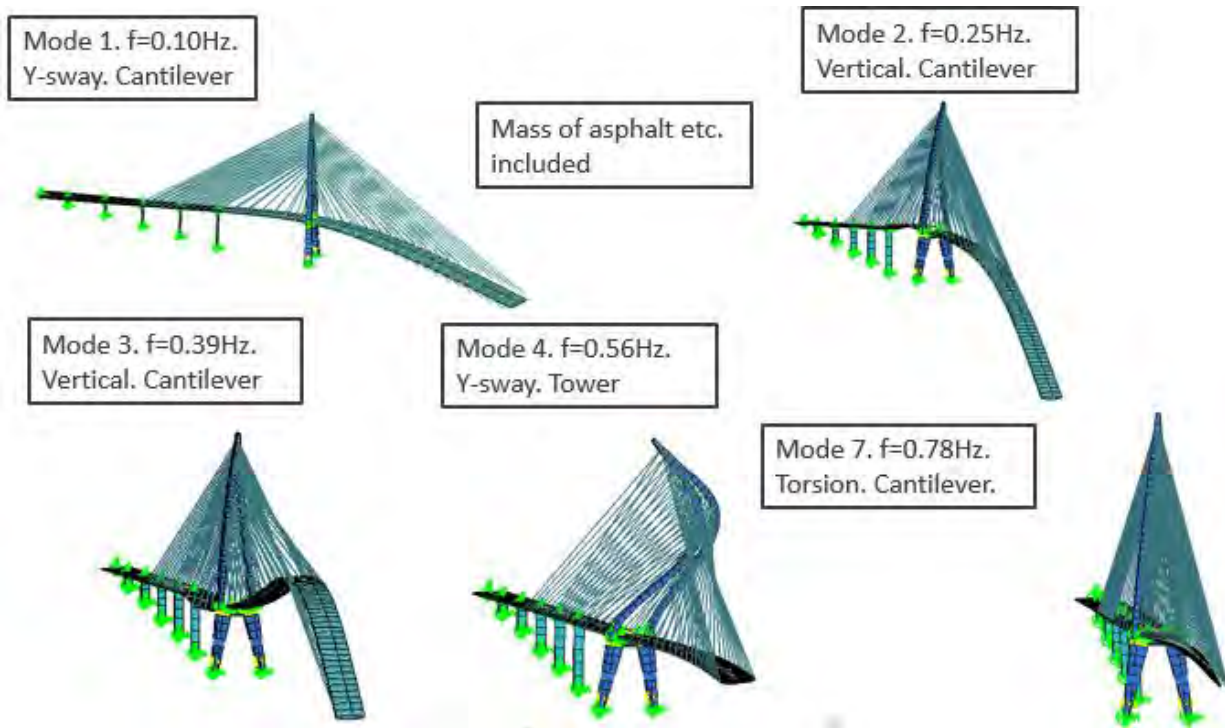
Mode shapes

Mode 1. f=0.10Hz.
Y-sway. Cantilever

Mass of asphalt etc.
included

Mode 2. f=0.25Hz.
Vertical. Cantilever

Mode 3. f=0.39Hz.
Vertical. Cantilever

Mode 4. f=0.56Hz.
Y-sway. Tower

Mode 7. f=0.78Hz.
Torsion. Cantilever.

Figure 6-30 FSHB. Mode shapes and natural frequencies

Buckling modes

Mode 1. f=9.6.
Vertical. Cantilever

Mode 2-20. Vertical.
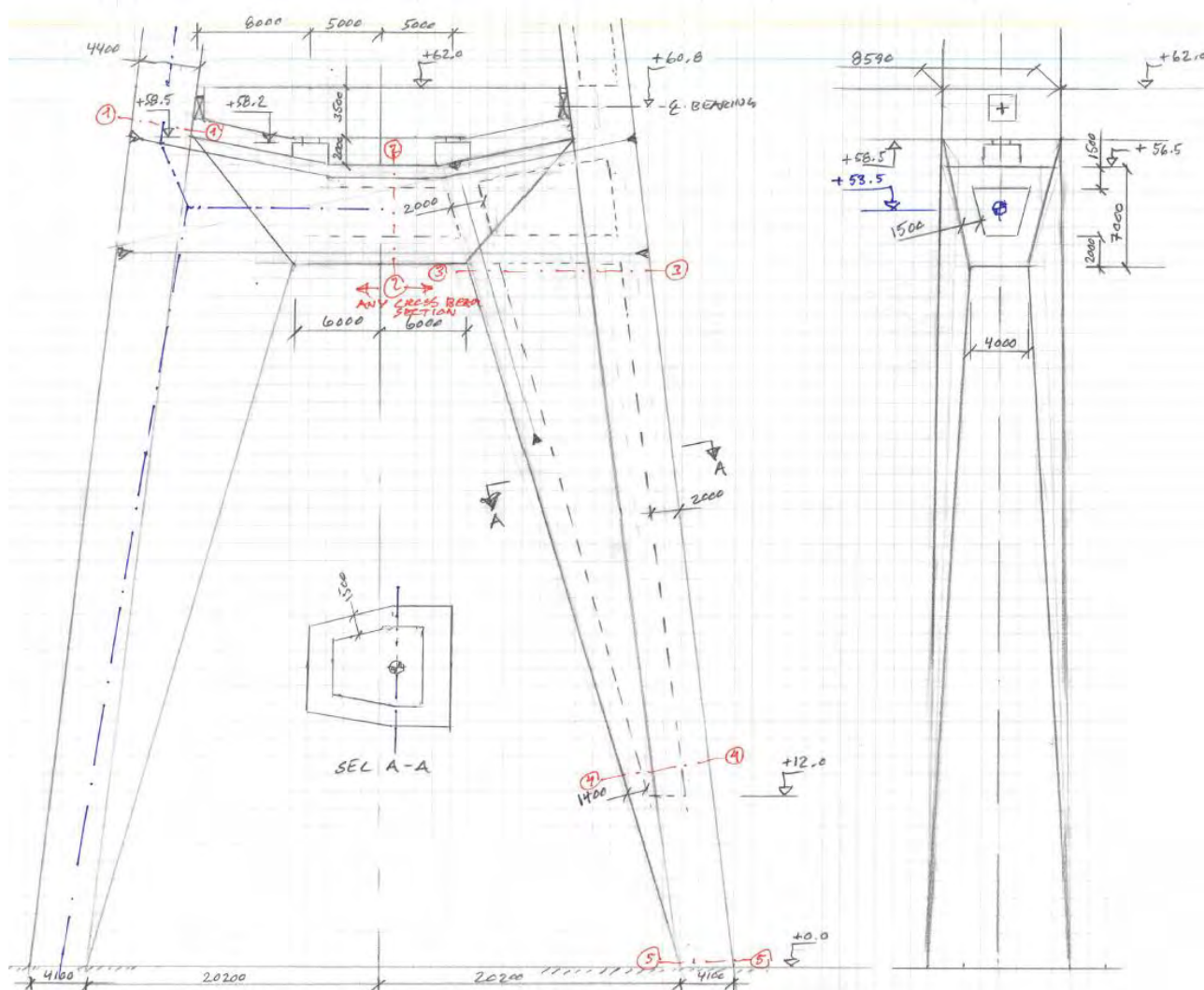F(mode20)=15.

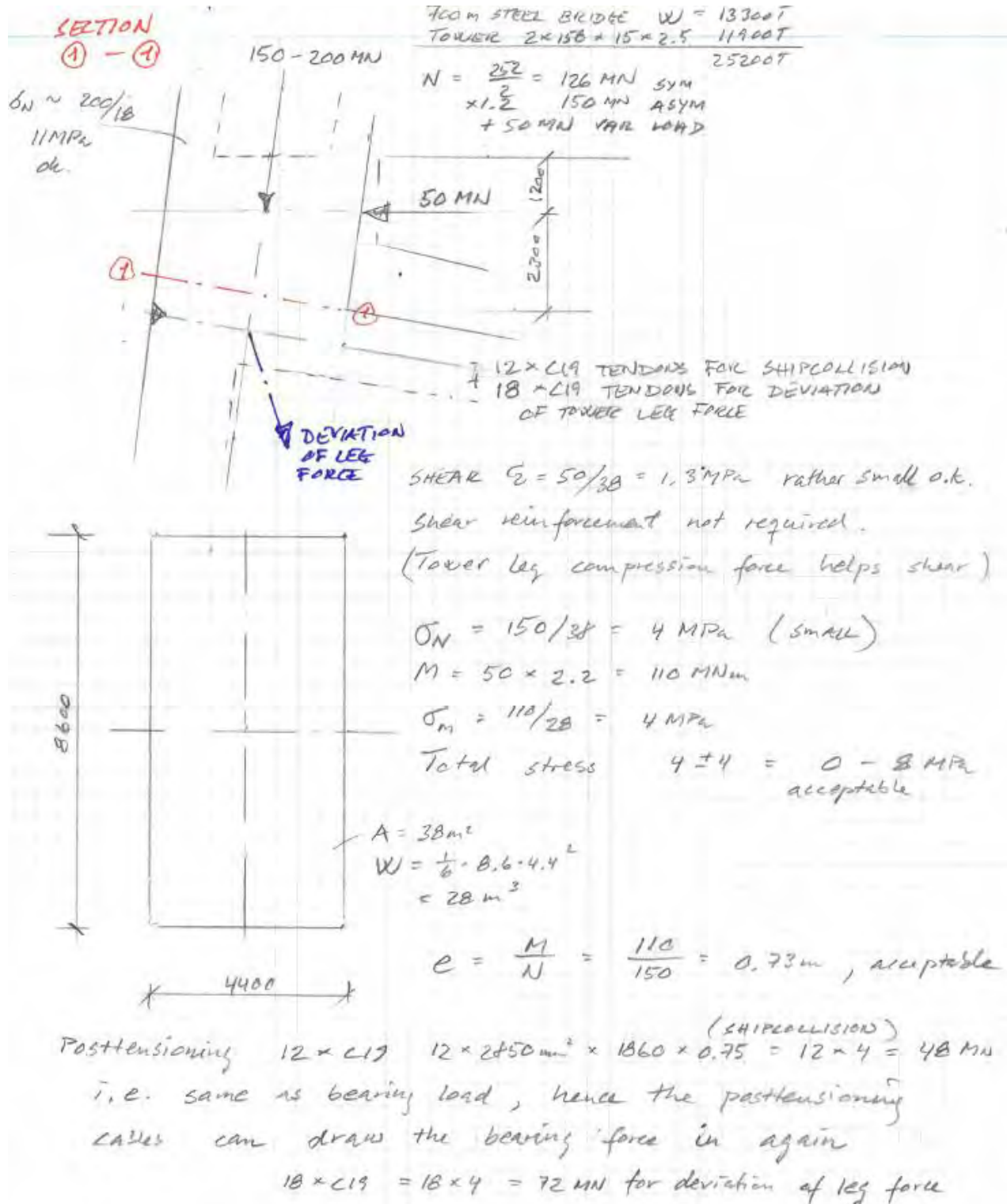*Figure 6-31 FSHB. Buckling modes and critical factors (N_{cr}/N_{Ed})*

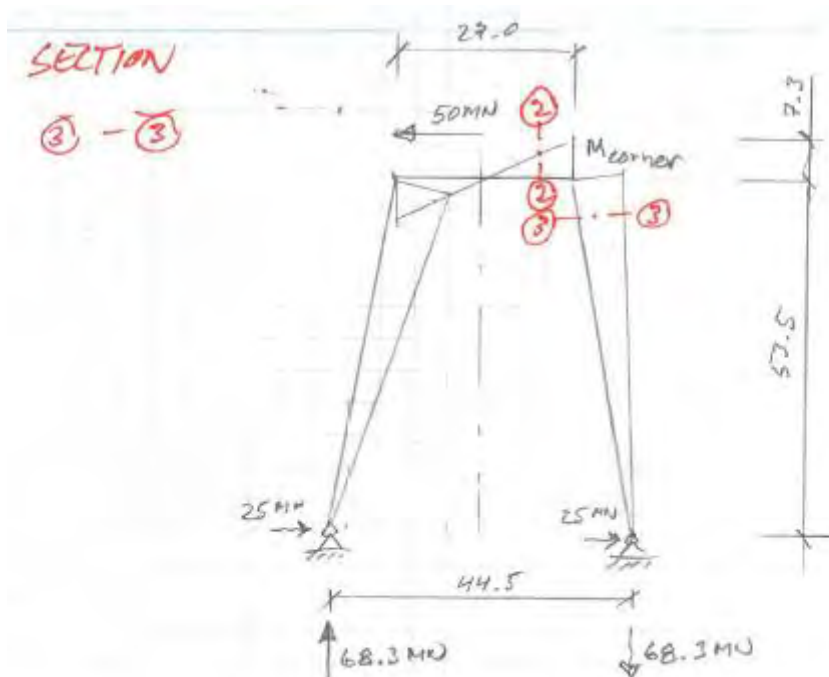### 6.2   Tower design – hand calculation of ship collision

The following hand calculation was done as a preliminary evaluation of the tower and is now included as verification calculation. A comparison between the hand calculations and typical section design in chapter 3 show that the hand calculations are conservative.

The ship collision case is a high demanding effect for the lower frame. The following is a rough calculation based on the assumption that the ship collision leads to a bearing force of 50 MN. The final results for K12 give a bearing force of approximately 35 MN. This chapter is only used as a verification, and thus the hand calculation is not updated. Other load effects (permanent) are approximate estimates.

The frame is checked at 5 sections as indicated below.

SECTION
① – ①   150 – 200 MN

$\delta_N \sim 200/18$
11MPa
ok.

700 m STEEL BRIDGE W = 13300 T
TOWER 2×150 × 15 × 2.5 11900 T
—————
25200 T

$N = \frac{252}{2} = 126$ MN SYM
×1.2   150 MN ASYM
+ 50 MN VAR LOAD

50 MN

12 × C19 TENDONS FOR SHIPCOLLISION
18 × C19 TENDONS FOR DEVIATION
OF TOWER LEG FORCE

DEVIATION
OF LEG
FORCE

SHEAR $\tau_e = 50/38 = 1.3$ MPa rather small o.k.
Shear reinforcement not required.
(Tower leg compression force helps shear)

$\sigma_N = 150/38 = 4$ MPa (small)
$M = 50 × 2.2 = 110$ MNm
$\sigma_m = \frac{110}{28} = 4$ MPa
Total stress $4 \pm 4 = 0 - 8$ MPa
                              acceptable

$A = 38$ m²
$W = \frac{1}{6} \cdot 8.6 \cdot 4.4^2$
$= 28$ m³

$e = \frac{M}{N} = \frac{110}{150} = 0.73$ m , acceptable

8600

4400

Posttensioning 12 × C19   12 × 2850 mm² × 1860 × 0.75 $\overset{(SHIPCOLLISION)}{=}$ 12 × 4 = 48 MN
i.e. same as bearing load , hence the posttensioning
cables can draw the bearing force in again
18 × C19 = 18 × 4 = 72 MN for deviation of leg force

SECTION

③ — ③

M_{corner} = 25 × 53.5 − 68.3 × 8.75
1338 − 598 = 740 MNm

section 3   M ~ 700 MNm (shipcollision)

N = 160 MN   min

e = 4.4 m , acceptable

12 C 19
12 × 4 = 48 MN
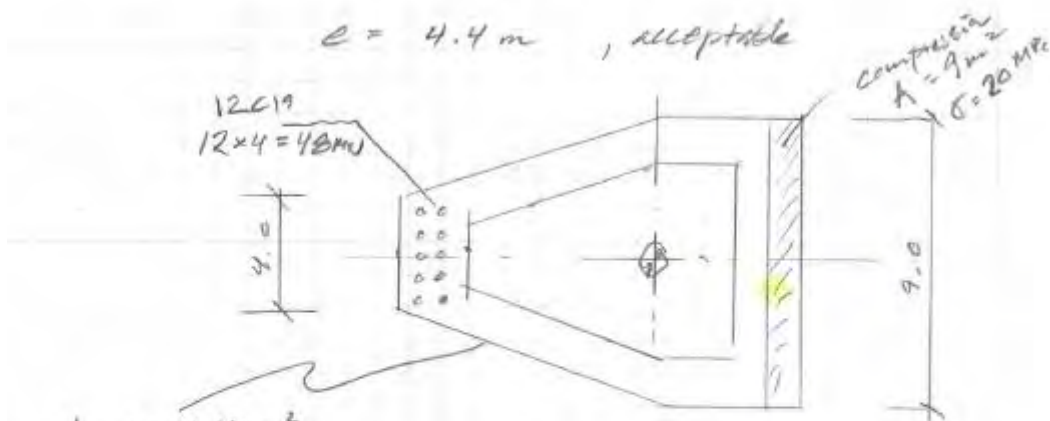
A total ~ 40 m²
N = 250 MN
σ = 6 MPa
normal situation
(no ship collision)

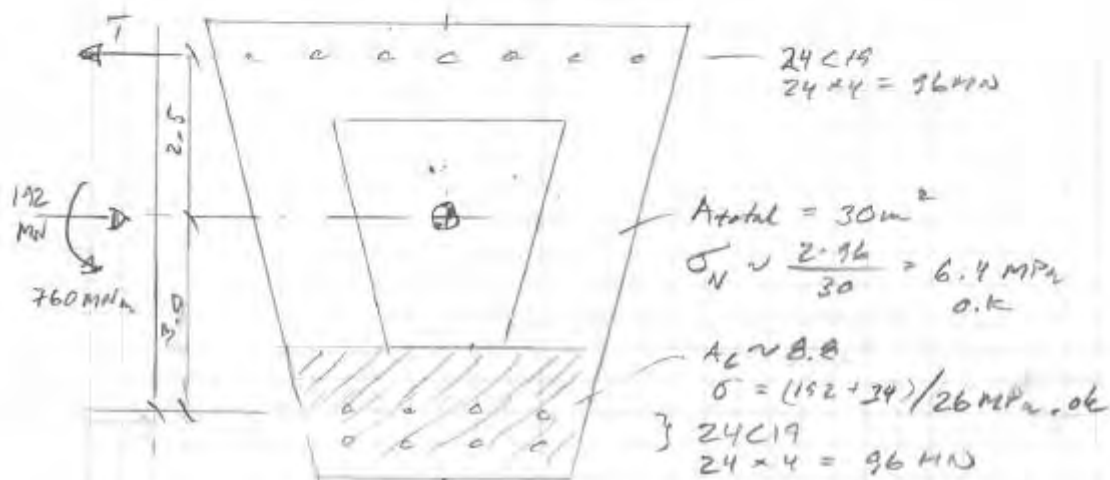Tension T = (700 − 160 · 3.8)/10.8 ~ 10 MN < 48 MN
o.k.

SECTION ②-②

Gradual zone, say, $\frac{2}{3}$ of section ③ M

$\sim \frac{2}{3} \times 700 = 462$ MNm ship collision

Permanent negative moment due to

leg force $150 \times 2 = 300$ MNm

Check for $M = 760$ MNm

$24 \, C19$
$24 \times 4 = 96$ MN

$A_{total} = 30 \, m^2$

$\sigma_N \sim \frac{2 \cdot 96}{30} = 6.4$ MPa o.k

$A_c \sim 8.8$

$\sigma = (152 + 34)/26$ MPa . ok

$24 \, C19$
$24 \times 4 = 96$ MN

$$T = \frac{760 - 192 \times 2.0}{5.5} = 34 \text{ MN}$$

to be taken by normal reinforcement

$2\% \times A \sim 0.02 \times 10.5 = 0.21 \, m^2$

$\sigma_s = 34/0.21 \div 162$ MPa, ok.

$$Shear = \frac{2 \times M_{corner}}{27} = \frac{2 \times 740}{27} = 55 \text{ MN}$$

$\tau = \frac{55}{30} = 1.8$ MPa high.

Webs shall have (high) shear reinforcement.
Beam model conservative, real situation
better in regards to shear.
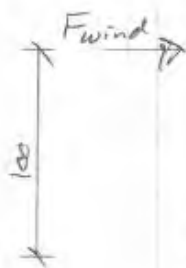Posttensioning 192 MN will be able to transfer
much shear!

Section ⑤ – ⑤

Weight of lower part of tower (level 0 – level +62)

$\simeq 120$ MN

Max reaction normal situations

$N = 200 + \dfrac{120}{2} = 260$ MN   per one leg

If uniform (no moment unrealistic) $\sigma = \dfrac{260}{4.1 \cdot 10} = 6.3$ MPa o.k.

Free standing tower

Can also take various moments.

With ship collision $+ \dfrac{68.7}{4.1 \cdot 10} = 1.7$ MPa additional, no problem.

$F_{wind} \rightarrow$

$N = 120$ MN per leg   240 MN total

$M = 7.3 \times 100 = 730$ MNm

$A_{wind} \sim 2 \times 4.5 \times 220 + 2 \times 50 \times 5 + 27 \cdot 7$

$1980 + 500 + 190 = 2670$ m^2

$C_D = 1.7$

$v_{gust} = 50$ m/s $p = 1.6$ kN/m^2

$F_{wind} = 2670 \times 1.6 \cdot 1.7 = 7.3$ MN

$e = \dfrac{M}{N} = \dfrac{730}{240} = 3.0$ m , seems reasonable.

(Foundation slab will be 16 m long)

Section ④ – ④

$A_{concrete} \sim 32$ m^2

$\sigma = (260 + 68)/32 = 10$ MPa, ok.

7   Enclosures