Statens vegvesen

Fergefri E39 – Kryssing av Bjørnafjorden

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Design Basis
Bjørnafjorden floating bridges

Dokumentnr.:	Rev.:	Sider:
SBJ-32-C4-SVV-90-BA-001	0	40

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1 INTRODUCTORY PROVISIONS

1.1 Application

This revision of design basis shall serve as basis for the final concept development phase for the Bjørnafjorden floating bridge, project phase: 01.11.2018 – 31.08.2019.

In case of conflicting rules, the specific rules as given in this document will govern over general rules.

1.2 Definitions and abbreviations

Terms used in the design basis have the following definitions:

Floating bridge
A floating structure, designed for traffic loads directly applied on to floaters or on a separately constructed carriageway, which may have fixed or floating supports between the abutments.

Mooring system
Arrangement of cables that is connecting a bridge structure to the seabed.

Splash zone:
External surface that is periodically in contact with seawater.

Freeboard
The vertical distance from the water level to the buoyancy body's lateral surface.

Service Life
The service life of the structure estimated from its completion date.

Green sea
High sea wave that flows onto the pontoon deck and causes a significant portion of the deck area temporary to be fully submerged in the sea water.

LAT
Lowest Astronomical Tide.

MLW
Mean Low Water.

MSL
Mean Sea Level

MHW
Mean High Water.

HAT
Highest Astronomical Tide.

Service life
The service life of the structure estimated from its completion date.

NPRA
Norwegian Public Road Administration.

EC

Eurocode

AADT
Total number of vehicles passing a fixed point (both directions) during a year divided by 365

HDPE
High density polyethylene.

ISO
International Organization for Standardization

DNV GL
Det Norske Veritas Germanischer Lloyd

SLS
Serviceability Limit State

ULS
Ultimate Limit State

EQU
Loss of static equilibrium of the structure or any part of it considered as a rigid body.

STR
Internal failure or excessive deformation of the structure or structural members

GEO
Failure or excessive deformation of the ground where the strengths of soil or rock are significant in providing resistance

FAT
Fatigue failure of the structure or structural members

ALS
Accidental Limit State

DFF
Design Fatigue Factor

MBL
Minimum breaking load (characteristic breaking strength of considered component)

2 INTRODUCTION

2.1 General

A bridge will replace todays ferry connection between Halhjem and Sandvikvåg in Bjørnafjorden. The bridge will be a part of a larger project to make E39 continuous, without ferries, from Kristiansand to Trondheim. Replacing the ferries with bridges will significantly reduce the travelling time and will have large positive socioeconomic effects for the regions.

Bjørnafjorden is located about 30km south of Bergen. The crossing is planned from a small island, with the name Svarvahella at Rekstern (in the south) to Søre – Øyane (in the north). The distance is about 4.8km.

This design basis is valid for floating bridge over Bjørnafjorden.

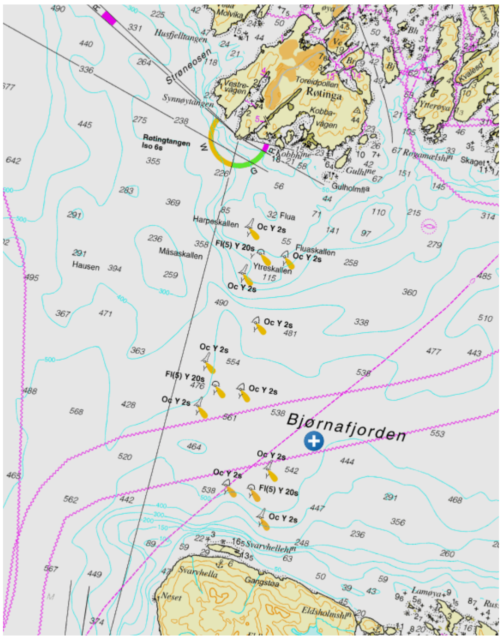


Figure 2-1 Bjørnafjorden basin

All structural elements that supports the road line between the abutment in the south and the rock tunnel in the north shall be covered in design.

2.2 Geographic coordinate system

The geographic coordinate system is EUREF 89 NTM Sone 5. Bathymetry maps is given with reference to NN2000 (not LAT). All maps, geographic information and geographic drawings shall be according to this coordinate system.

MLW, MSL, MHW and HAT is given in the metocean design basis with reference to LAT. In Bergen the NN2000 is defined at +97 cm above LAT. The transfer coefficient Bergen - Bjørnafjorden is 0.81, which implies that the applicable distance from LAT is +78 cm.

3 DESIGN PRINCIPLES

3.1 General

The bridge design shall be in accordance with relevant design rules in Eurocodes (EC) and NPRA publication N400 and other rules and regulations by the Norwegian Public Road Administration. Additional rules defined by the project, not regulated by the EC or N400 is defined in this document.

3.2 Design method

The design shall be based on the limit state method. The definitions of limit state categories are given in NS-EN 1990.

- SLS – Serviceability limit state
- ULS – Ultimate limit state
    - EQU – Loss of static equilibrium
    - STR – Internal failure or excessive deformation
    - GEO – Failure or excessive deformation in the ground
    - FAT – Fatigue failure
- ALS – Accidental limit state

In addition the mooring system shall be verified in:

- MBL – Minimum breaking load

MBL is a limit state that shall be documented for the mooring system. MBL is defined as the characteristic breaking strength of the considered mooring component. The MBL for the support structure and mooring equipment's shall be documented.

3.3 Consequence, reliability, control and inspection class

The bridge is categorized as consequence class CC3 and reliability class RC3 in accordance with NS-EN 1990 Annex B [1]. Control level DSL3 (extended supervision,) and Inspection level IL3 (extended inspection during execution) shall be applied.

Particular members of the structure may be categorised as consequence class CC2 (Medium) and consequently reliability class RC2. For these members Design Supervision Level 3 (DSL3) and Inspection Level 2 (IL2) shall be applied.

A FMECA (Failure mode, effects, and criticality analysis) should be conducted as a part of the design, in order to ensure an identification of all failure modes of the bridge. All failure modes should be analysed.

The identified failure modes shall be evaluated using the accidental limit state method.

3.4 Design life

The design life of the structure is 100 years.

Components in the structure that has a design service life, less than 100 years, shall be replaceable. The replacement of such components shall have minimum disturbance on road and maritime traffic, so that the bridge on average is open 99,5 % of the time, considering all events.

3.5 Total risk acceptance

A TRA (total risk analysis) regarding safety of people should be conducted as a part of the design, in order to ensure that the overall risk level is acceptable. Risks that should be considered in a TRA should as a minimum include the following risk:

- Traffic accidents
- Fires & explosions
- Ship collisions
- Landslides

- Earthquake
- Aircraft crash

4 FUNCTIONAL CRITERIA

4.1 Roadway

The partitioning of bridge deck due to traffic related functions is shown in Figure 5-1.

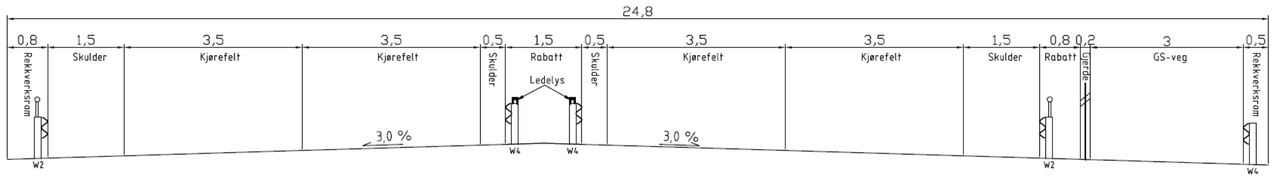


Figure 4-1 Bridge deck sectioning principles

4.2 Guard rails

Guard rails are type W2 and W4 s shown in Figure 5-1, ref. [2].

4.3 Minimum clearance for ship traffic

4.3.1 General

The fairway norm by the Norwegian Coastal Administration [3] and project specifications for the fairway in Bjørnafjorden shall be used.

4.3.2 Minimum vertical clearance:

The vertical clearance is defined as the lowest distance from sea surface to the underside of the bridge girder.

The distance for vertical clearance shall account for the following effects:
- Largest combined deformations in the SLS condition (in-frequently occurring).
- The highest astronomical tide (HAT), if pontoon is restrained.

The minimum vertical clearance is defined as:
- 45m in the mainspan (ship navigation channel).
- 11.5m in the sidespans.

4.3.3 Minimum horizontal clearance:

Horizontal navigational clearance is defined as the free width for ship passage and shall not be less than:
- 250m in the mainspan (ship navigation channel).
- No requirements in the side spans.

4.3.4 Minimum keel clearance

Design keel clearance in the navigation channel shall be according to regulations given in Farledsnormalen Ch.3 by the Norwegian Costal Administration [3]. The minimum water depth in the navigation channel shall be minimum 16.5m including safety distance to keel. The distance shall be measured relative to LAT.

4.4 Safety systems for navigation

The bridge shall be equipped with signs for ship and aeronautical navigation, see 12.7.5 in N400 [4]. When the bridge is installed, the Norwegian Coastal Administration will operate a ship surveillance system (VTS) for the Bjørnafjord area.

4.5 Pontoons

4.5.1 Splash zone

To determine the total splash zone height for the pontoons the following effects shall be accounted for:

- Splash zone height from waves, calculated according to DNV GL-OS-C101 see chapter 2, section 9 and clause 2.2 [5], equations in 2.2.6 and 2.2.7 shall be used.
- The effects calculated from traffic loading in the quasi-permanent SLS condition (Ψ_2=0.2).
- An addition of ΔH=30 cm to take the transitions zone between splash zone protection and marine environment coating into account.
- The stiffness from the mooring system shall be considered where relevant for the pontoons.

4.5.2 Contingency and ballast

To account for deviation in the permanent action during the building phase, a structural contingency shall be included in the pontoon. Designer shall evaluate the contingency fraction for each structural element. The contingency shall be based on acceptable deviations during construction and uncertainties in the applied permanent loading.

The pontoons shall be designed for water ballast for both the temporary and permanent situation.

4.5.3 Water detectors and inspection hatches

All pontoon compartments shall be equipped with minimum two water detectors. Inspection and testing of detector systems shall be part of the maintenance program.

All pontoon compartments shall be available for inspection.

The pontoon compartments shall be accessible through watertight hatches. The hatches shall be designed in such way, that these will be closed mechanically, when not in use. The compartments shall be available from the top plate.

4.5.4 Bilge pump systems

Permanent bilge pump systems shall not be installed as an integrated part of the pontoons. Provision shall be made for easy installation of mobile pumping system, if a pontoon is subjected to an un-intendent water leakage.

4.6 Instrumentation

There shall be installed two different instrumentation systems:

- Monitoring of the bridge response, such as velocities, accelerations and deformations.
- Monitoring of the mooring tension, reference is made to [6].

4.6.1 Inspection, operation and maintenance

It shall be developed a program for inspection, operation and maintenance. The program shall be planned in a way that has systematically focus on detecting mechanisms that differ from intended design.

The design of the bridge shall provide safe and easy access (within arm's length of all components) for inspection and maintenance of all relevant structures, equipment and systems.

The design of the bridge shall allow routine inspection and maintenance to be carried out with minimum disturbance to road traffic (One lane can be closed during a short period and in general respecting that the bridge on average shall be open for partial or full traffic at least 99.5% of the time considering all events).

Bridge components that expectedly require maintenance shall be easy to maintain with minimal disturbance to road- as well as maritime traffic.

4.6.2 Design against intended attacks

A risk analysis focusing on intended attacks should be carried out as a part of the design. The design of the bridge should take into account that intended attacks could be conducted, and the goal is to have the design itself to reduce the probability of an attacker’s success given an attack. Areas that should be restricted for public should be planned in time, allowing for solutions that are well functioning and maintainable. Likewise, for areas vulnerable for fire and high temperature, e.g. as a result of use of explosives and/ or intended fires, fire protection should be considered.

5 MATERIALS AND IMPLEMENTATION

5.1 Concrete structures

5.1.1 General

Handbook R762 [7], N400 [4] and the Eurocode series apply in design. There is given some additions to the existing rules and codes in the clauses below.

5.1.2 Concrete cover requirements

The concrete cover shall be in accordance to 7.4.2 in [4].

5.1.3 Concrete aggregate and quality

The concrete, its aggregates and workmanship shall be in accordance to the requirements in Handbook R762 [7], N400 [4] with necessary adjustments according to NS-EN 1992-1 [8] and NS-EN 1992-2 [9]

Minimum concrete grade shall be C45.

5.1.4 Concrete material factors

Material factors shall be used in accordance to NS-EN 1992-1-1:2004+NA: 2008, table NA.2.1N.

5.1.5 Concrete structure properties

Concrete properties shall be used according to NS-EN 1992-1-1:2004+NA: 2008.

5.1.6 Reinforcement quality

Rebar quality shall be of B500NC according to NS 3576-3, [10] and NS-EN 10080, [11].

5.1.7 Reinforcement placement

All cross sections shall have sufficient minimum reinforcement to ensure controlled cracking.

All panels shall have double-sided reinforcement

The minimum centre distance for rebar placement shall be no less than 150mm. For external walls, the minimum rebar dimension shall be no less than 16mm. For internal walls, the minimum dimension shall be no less than 12mm [4].

5.1.8 Prestressing reinforcement

Prestressing steel and its components shall satisfy the requirements of prEN 10138 [12].

Prestressing cable anchors shall be cast with normal concrete cover requirements.

In general, all prestressing ducts shall be grouted, cables that are scheduled to be replaced during the service life of the bridge shall not be grouted. Protective measures for corrosion will in these cases be specified and approved.

5.2 Steel structures

5.2.1 General

Handbook R762, N400 and the Eurocode series apply in design. There is given some additions to the existing rules and codes in the clauses below.

5.2.2 Steel structure material factors

Material factors shall be used in accordance to relevant section in NS-EN 1993:

5.2.3 Normal/construction steel properties

Steel type and maximum thicknesses shall comply with the requirements in NS-EN-1993-1 [13] and NS-EN-1993-2 [14].

For construction steel, the maximum grade shall be limited to S420, unless other agreement with the client.

5.2.4 Corrosion protection

Steel surfaces shall have corrosion protection to preserve the steel structure. Maintenance intervals shall be planned during design.

The steel surfaces exposed to air shall be protected with coating systems, and for inner surfaces of box girder and steel pylons, corrosion protection is ensured using dehumidification system and light zinc-rich primer, according to N400 [4].

Permanently submerged steel surfaces shall be protected by a passive galvanic cathodic protection systems (i.e. sacrificial anodes).

All steel surfaces in tidal and splash zone (see 4.5.1) shall be protected by using super duplex steel or dedicated special coating systems, resulting in a zero need of repair during the service life of the bridge.

Enclosed surfaces unavailable for inspection and surface treatments, such as the inside of pipes, steel hollow sections etc. shall be airtight and the airtightness ensured by pressure tests.

Enclosed surfaces available for inspection and surface treatments, such as the steel box girders and columns etc. shall be watertight. If internal corrosion protection is ensured by low internal humidity, the structure shall be airtight. Doors, hatches and other openings shall be equipped with gaskets and closing devices that ensure the airtightness. Valves (or something similar) must be utilized in order to cancel out differences in pressure between the inside and outside of the airtight structure.

Railing fixes, embedded details and other minor steel parts shall in general be acid proof.

5.2.5 Cable systems

5.2.5.1 Stay cables and tension bars

Material factors for stay cables and tensions bars are defined in NS-EN 1993-1-11 [15] NA.6.

$$\gamma_R = 1.2$$

5.2.5.2 Stay cables

Cables with parallel strands or spiral stands (locked coil) can be used for the high bridge. The design of tension components shall comply with the requirements of NS-EN 1993-1-11 [15].

Material properties
Stay cables shall be of the type; Group C according to Table 1.1, see [15] comprising bundles of parallel wire strands, anchored with wedges.

Properties (in accordance with EN 10138-3: Strands) shall be adopted:

Corrosion protection
The cable stays will be comprised of galvanised, grease, PE coated strands contained within a HDPE outer pipe. THE HDPE outer pipe is assumed to be of the standard type with respect to diameter.

6 DETERMINATION OF LOADS

6.1 General

The loads are divided into categories based on their nature and the likelihood of their occurrence:

- Permanent loads (G)
- Variable loads (Q)
- Accidental loads (A)

The classification of individual loads are shown in the following chapters. Load designations are given with a symbol for the main group as well as a symbol for type of load.

6.2 Permanent loads (G)

6.2.1 General

Loads classified as permanent is described in N400 chapter 13.12.2.

Deformation loads are treated as permanent loads in accordance to the Eurocodes.

<u>Permanent loads (G)</u>

• Self-weight	G-W
• Super self-weight	G-Add
• Permanent water head (buoyancy)	G-B
• Marine fouling	G-Mfoul
• Permanent ballast	G-S
• Stay cable forces	G-Cab
• Pretension of anchoring system	G-Mor

<u>Deformation loads (G)</u>

• Shrinkage, creep and relaxation	G-D
• Pretension of tendons	G-P

6.2.1.1 Self-weight (G-W)

The following loads for self-weight shall be used:

- Structural steel: 77kN/m^3
- Normal concrete (reinforced): 26kN/m^3

Weight of mooring- and stay cables shall be included as described from supplier.

6.2.1.2 Super self-weight (G-Add)

Road surface weight is defined in 5.2.2.2 in N400 [4]:

Weight of equipment and outfitting such as railings ect shall be as described from supplier.

6.2.1.3 Permanent water head (buoyancy) (G-B)

The water density variations shall be according to MetOcean Design basis Rev 0 [16].

6.2.1.4 Marine fouling (G-Mfoul)

Thickness variations and densities are defined in MetOcean Design basis Rev 0 [16].

6.2.1.5 Permanent ballast (G-S)

Water ballast shall be assumed for both the temporary and the permanent situation.

Water ballast shall account for the contingency defined in 4.5.2.

6.2.1.6 Stay cable forces (G-Cab)

Applies to prestressing forces in cables of the main bridge that are included in the equilibrium group G-EQ.

6.2.1.7 Pretension of anchoring system (G-Mor)

Pretension in the mooring system shall be included in the equilibrium group G-EQ.

Deformation loads (G)

6.2.1.8 Shrinkage, creep and relaxation (G-D)

Creep and shrinkage shall be applied in accordance with NS-EN 1992-1-1, 2.3.2.2, 3.1.4 and 5.8.4 [17].
Relaxation is applied in accordance with NS-EN 1992-1-1, 3.3.2 and 5.10.6 [17].

6.2.1.9 Pretension of tendons (G-P)

Applies to pretension tendons in concrete structures, effects of friction and anchor loss in tendon shall be included.

6.3 Variable loads - Q

6.3.1 General

Variable operational loads are loads associated to the expected use of the structure, and include:

<u>Variable loads (Q)</u>

- Traffic loads — Q-Trf
- Temperature variations — Q-Temp
- Water level variations — Q-Tide
- Wave loads — Q-Wave

- Wind loads Q-Wind
- Current loads Q-Cur
- Slamming loads Q-Slam

6.3.1.1 Traffic loads (Q-Trf)

SLS traffic loads:
The SLS condition and the evaluation of motion limitations shall be evaluated against the loads given in "*Forskrift for trafikklast på bruer ferjekaier og andre bærende konstruksjoner i det offentlige vegnettet",* [18].

ULS traffic loads:
The structure shall be designed (capacity checked) against the loads given in "*Forskrift for trafikklast på bruer ferjekaier og andre bærende konstruksjoner i det offentlige vegnettet",* [18].

FAT traffic loads
Traffic running on bridges produces stress cycles that leads to fatigue damage. The traffic load model that shall be used for fatigue verification is FLM4 in NS-EN 1991-2:2003+NA:2010 [19]. The load model is a set of five "equivalent" lorries. Each lorry represents a percentage of the heavy traffic crossing the bridge and are divided into fractions, representing long distance, medium distance and short distance traffic volume. The model is a function of N_{obs} and N which are the numbers of heavy traffic lorries crossing the slow lanes and fast lanes each year respectively.

Traffic category definition
Traffic category 2 shall be used for fatigue verification. The traffic category represents motorways with medium flowrates of lorry's. The yearly number of lorry's in each of the North- and southern outer slow lanes is N_{obs}=0.5E6, which provides the following distribution of traffic volume:

North direction
- N_{obs}=0.5E6
- N=0.10x0.5E6=0.5E5

South direction
- N_{obs}=0.5E6
- N=0.10x0.5E6=0.5E5

Traffic categories		N_{obs} per year and per slow lane
1	Roads and motorways with 2 or more lanes per direction with high flow rates of lorries	$2{,}0 \times 10^6$
2	Roads and motorways with medium flow rates of lorries	$0{,}5 \times 10^6$
3	Main roads with low flow rates of lorries	$0{,}125 \times 10^6$
4	Local roads with low flow rates of lorries	$0{,}05 \times 10^6$

Figure 6-1 Table 4.5(n) from NS-EN 1991-2 – Traffic category definition

Traffic type definition
The fatigue damage caused by different distributions of axel spacing's and the corresponding axel loads shall be calculated assuming traffic type 4 (Long distance) shown on Figure 6-2.

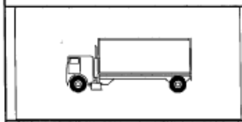
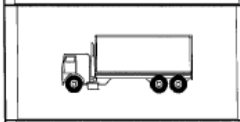
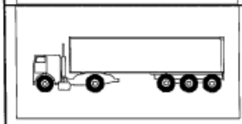
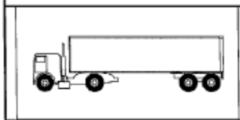
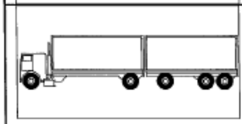
VEHICLE TYPE			TRAFFIC TYPE			
1	2	3	4	5	6	7
			Long distance	Medium distance	Local traffic	
LORRY	Axle spacing (m)	Equivalent axle loads (kN)	Lorry percentage	Lorry percentage	Lorry percentage	Wheel type
	4,5	70 130	20,0	40,0	80,0	A B
	4,20 1,30	70 120 120	5,0	10,0	5,0	A B B
	3,20 5,20 1,30 1,30	70 150 90 90 90	50,0	30,0	5,0	A B C C C
	3,40 6,00 1,80	70 140 90 90	15,0	15,0	5,0	A B B B
	4,80 3,60 4,40 1,30	70 130 90 80 80	10,0	5,0	5,0	A B C C C

Figure 6-2 Table 4.7 from NS--EN 1991-2 – Traffic type definition

The combined fatigue damage caused by environmental- and traffic loads is further described in 8.13.

6.3.1.2 Temperature variations (Q-Temp)

The effect of temperature variations shall be accounted for in the analysis. Further description of temperature variations are stated in MetOcean Design basis Rev 0 [16].

6.3.1.3 Water level variations (Q-Tide)

Water level variation shall be accounted for in the analysis as described in MetOcean Design basis Rev 0 [16].

The assumption on rise of future sea level shall be according to MetOcean Design basis Rev 0 [16]. The effects can be encompassed by use of permanent ballast according to Ch. 6.2.1.5.

6.3.1.4 Wave loads (Q-Wave)

Description of wave elevation spectrum and directional spreading for generation of wave elevation time series is stated in MetOcean Design basis Rev 0 [16]. Wave load calculation on buoyancy elements shall reflect both linear and non-linear loads that are important for determining the structural response.

6.3.1.5 Wind loads (Q-Wind)

Wind velocities, turbulence intensity’s and other parameters necessary to generate wind field series are stated in the MetOcean Design basis Rev 0 [16].

6.3.1.6 Current loads (Q-Cur)

The effect of current shall be accounted for in the analysis. Current velocity's is stated in the MetOcean Design basis Rev 0 [16].

6.3.1.7 Slamming loads (Q-Slam)

Slamming effects shall be accounted for, if relevant according to DNVGL-RP-C205 Ch.8 [20].

6.4 Accidental loads - A

6.4.1 General

The accidental loads are described in 13.12.3.5 in N400.

The following is described under 5.6.2 in N400.

"*Prevalence and consequences of accidental loads relates generally to a certain level of risk. In so far accident loads can be determined by probability calculations, the likelihood of an incident that's being disregarded not exceed* 10^{-4} *per year, to the extent the accidental load can be determined based on probability calculations.*"

The accumulated probability of occurrence is assumed when evaluating the incident to the return period 10^{-4}.

Accidental loads are loads imposed to the structure due to incorrect operation or extraordinary situations such as:

<u>Accidental loads (A)</u>

• Ship impact	A-Coll
• Filling of pontoon compartments	A-Flood
• Failure in mooring system	A-Morfail
• Failure of stay cables	A-SCab
• Underwater landslides	A-Slide
• Earthquake	A-EarthQ
• Abnormal environmental loads	A-Abnor
• Fire and explosion	A-Fire&Exp

6.4.1.1 Ship impact (A-Coll)

The concept shall be designed according to accidental limit state requirement in N400 13.12.4.5 [4]. The capacity must be evaluated for impact and post-impact considerations, with load factors set to 1.0 and material factors according to relevant Eurocodes. FE-simulation of ship impact shall be based on a validated material model and fracture criterion. Mesh-sensitivity effects should also be taken into account. Characteristic material properties shall be used in the analysis, which typically means that lower 5th percentile shall be used when lower limit for strength is critical, and the 95th percentile shall be used when upper limit for the strength is critical.

Impact events for the floating bridge includes:
- Bow collisions with bridge pontoons (perpendicular to bridge line)
- Bow collisions with bridge pontoons (parallel to bridge line)
- Deckhouse collision with bridge girder
- Sideway collisions (against the pontoons longitudinal walls)

To account for added mass of the striking ship:
- 5 % of the ship mass shall be assumed for longitudinal (surge) impact.
- 40 % shall be assumed for sideway (sway) impact [21].

Local damage may be acceptable, but requires evaluation of the bridge for post-impact considerations (NS-EN 1991-1-7-2006, 3.2, [22]). Post-impact denotes a limit state for a damaged condition of the bridge. The limit state must be according to N400, which states that the environmental return period must be 100 years if not documented otherwise. A damaged condition may involve the following and more:

- Filling of pontoon compartments
- Mooring line failure
- Local plastic damage of bridge girder and columns

In general, the (ship impact) load variation with time shall be taken into account using the prescribed design vessels.

6.4.1.1.1 Distribution of design ship and impact energies

For both the end-anchored bridge concepts (K11 and K12), and the straight floating bridge concepts (K13 and K14), distributions of the required design ship and collision energies are given in Table 1 and

Table 2, respectively. Note that the provided distributions of design ship and collision energies are based on the present K7 and K8 bridge concepts. The distributions are sensitive with respect to both the ship traffic, distance between pontoons, and bridge geometry. Consequently, the effect of variations in impact energy and impact force must be evaluated with sensitivity studies. If the number of pontoons are changed during design (or other parameters that will affect the possibility of a collision), the distribution of impact energies shall be clarified with the client.

CC 100m	**Displacement**	**Velocity**	**LOA**
Element	[tonne]	[m/s]	[m]
Bridge girder	19084	6.2	200
Pontoon, Axis 3	14933	6.2	140
Pontoon, Axis 4-5	14933	6.2	140
Pontoon, Axis 6-28	14565	5.1	140
Pontoon, Axis 29-43	11473	5.1	125

Table 1 Distribution of impact energies for end-anchored bridge concepts K11 and K12 (based on the present K7 concept).

CC 125m Element	Displacement [tonne]	Velocity [m/s]	LOA [m]
Bridge girder	19084	6.2	200
Pontoon, Axis 3	14565	5.1	140
Pontoon, Axis 4-5	13878	5.1	130
Pontoon, Axis 6-28	13259	5.1	130
Pontoon, Axis 29-43	10649	5.1	120

Table 2 Distribution of impact energies for straight floating bridge concepts K13 and K14 (based on the present K8 concept).

6.4.1.1.2 Collision with bridge pontoons

The required bridge capacity for the case of ship impact shall be met for all possible impact directions and impact points. Typically, this means that both impacts perpendicular to the bridge axis and parallel to the bridge axis shall be examined. Furthermore, there may be a possibility of collision against the connecting columns depending on the design of the connections between pontoons and bridge girder.

Impact velocity of 2 m/s shall be used for sideways impact against pontoon in axis 3 (longitudinal direction of the bridge). For the other pontoons, an impact velocity of 1 m/s shall be used.

Added mass and damping for the pontoons shall be accounted for through the dynamic response analysis.

The client can provide FE-models of a container ship bow and an ice-strengthened bow. Use of other FE-models of ship bow than these provided models requires approval from the client. The container ship bow is obtained from a ship with an overall length of 166.62 m, a breadth of 27.4 m, a depth of 13.2 m and a scantling draught of 9.6 m. The FE-model of ice-strengthened bow is obtained from a passenger ship with an overall length of 223.8 m and a breadth of 35 m. With a ductile (or shared) energy design, extensive damage in the pontoons can occur. Consequently, an ice—reinforced bulb may result in less impact area as compared with conventional bulb, and will thus give larger indentation. Conventional bulb can give larger impact area and thus a larger impact force. The bridge must be able to handle both conventional bulb and ice-reinforced bulb. Force-indentation curves from simulation of ship bow colliding with a pontoon are presented in [23].

For the floating bridge concepts, a robust design shall be established by ensuring that local energy dissipation takes place through plastic deformations in the pontoons.

6.4.1.1.3 Deckhouse collision with bridge girder

The load indentation curves showed below are obtained from numerical simulations of deckhouse collision with bridge girder for the end-anchored and side-anchored floating bridge concepts developed in 2017. More details are found in [24]. A FE-model of the deckhouse can be provided by the client. Due to uncertainties in geometry and material properties of the deckhouse, a sensitivity study must be performed in order to evaluate the corresponding effect on the bridge response. Other structures such as containers and cranes can also be considered in the analysis.

6.4.1.1.4 Submarine impact

In lieu of better founded input the consequence of an impact from the New Norwegian submarine class shall be investigated. The following displacement and velocity shall be assumed:

- Surfaced: displacement of 1450t and velocity 3m/s.
- Submerged: displacement of 1830t and velocity 5 m/s.

6.4.1.2 Filling of pontoon compartments (A-Flood)

Unintended filling of a pontoon includes flooding of one or two neighbouring compartments.

Most unfavourable compartments shall be assumed, flooding does not need to be related to ship impact, and filling of the outer pontoon cells.

6.4.1.3 Failure in mooring system (A-Morfail)

Failure of mooring lines shall be documented in accordance to the requirements given in Mooring- and anchor design [6].

6.4.1.4 Failure of stay cables (A-Scab)

The bridge shall be controlled according to 13.2.5 in N400, with regard to failure in stay cables.

In addition the structure shall be evaluated for post-impact considerations in this damaged condition with a 100-year environmental loading applied to the structure.

6.4.1.5 Underwater landslides (A-Slide)

Underwater landslides shall be accounted for as described in Design Basis - Geotechnical design [25].

6.4.1.6 Earthquake (A-EarthQ)

Response from earthquake shall be calculated according to specification given in Design Basis - Geotechnical design [25].

6.4.1.7 Abnormal environmental loading (10.000-years) (A-Abnor)

Description of wave elevation spectrum and directional spreading for generation of wave elevation time series for a 10.000 year environmental condition is stated in MetOcean Design basis Rev 0 [16].

6.4.1.8 Fire and explosion (A-Fire&Exp)

During lifetime, several fires will occur on the bridge, due to traffic accidents etc. The severity of these fires are uncertain, as both pool fires and jet fires are possible. Explosions has a lower probability than fires, but should be considered as a part of the design.

Accidental loads for fire (A-Fire) and explosions (A-Exp) are described in a risk analysis from Multiconsult with reference SBJ-91-C4-MUL-23-RE-001, [26].

7 COMBINATION OF LOADS

7.1 Equilibrium group for permanent loads

All permanent loads are combined in an equilibrium group, denoted G-EQ, which is combined with other loads.

7.2 Combination of environmental loads

Combination of environmental loads shall be according to 13.12.3.1 in [4].

Designer shall evaluate necessary length of simulation time and document that the chosen length is sufficient. Metocean design basis facilitates both simulation times for 1- and 3 hour simulations.

If the omission of one or more environmental loads may give larger response values (eg current causes increased damping of wave response) this situation should be used for the design checks.

Temperature load is not a part of the environmental load group and shall be combined with reference to a 50-year return period with other loads, according to NS-EN 1991-1-5:2003+NA:2008.

The combination of the various environmental load components to form characteristic loads for different return periods shall be taken from the table below.

Return period (Years)	**Wind**	**Waves**		**Current**	**Sea level**	
		Wind sea	Swell*		Astronomical	Surge
1	1	1	1	1	HAT	1
10	10	10	10	10	HAT	10
100	100	100	100	100	HAT	100
10000	10000	10000	10000	10000	MEAN	10000

Table 3 Environmental load combinations

*The swell which shall be combined with wind-sea is dependent on the storm direction, this is further described in the MetOcean Design basis Rev 0 [16].

If low water is governing, the water level corresponding to LAT shall be used.

7.3 Combination of environmental loads with other loads

The combination of different characteristic environmental load groups with other loads shall be according to NS-EN 1990:2002+A1:2005+NA:2016 [1] and is further described in the chapters below.

7.3.1 Serviceability limit state - SLS

Response in the serviceability limit state shall be determined by the load combinations given in accordance to table NA.A2.6 in NS-EN 1990:2002+A1:2005+NA:2016 [1].

SLS - Characteristic

Serviceability limit state (Characteristic) shall be used to determine bearing displacements ect.

13.12.3.1 in [4] states that environmental loads with a return period of 100-year shall be used in the ultimate- and accidental limit state. Bearing displacement and girder clearance ect, shall in principle be controlled with 50-year environmental condition, which is not consistent. Hence, the SLS condition will also be calculated based on the response from a 100-year environmental situation.

Response from environmental loads with traffic shall be calculated based on 1-year return period.

Ψ_0 is combination factor in accordance to table NA.A2.1 in NS-EN 1990:2002+A1:2005+NA:2016 [1].

The table below shows the principles for combining loads in the characteristic serviceability limit state.

Combination factors in the characteristic SLS condition							
Dominant loads		G-EQ$_K$	Q-Trf$_K$	Q-Temp$_K$	Q-E$_{env(1y)}$ w/traffic	Q-E$_{env(100year)}$ No traffic	Q$_K$
		Ψ_0	Ψ_0	Ψ_0	Ψ_0	Ψ_0	Ψ_0
Permanent loads							
Permanent loads	G-EQ$_K$	1.0	1.0	1.0	1.0	1.0	1.0
Variable loads							
Traffic loads	Q-Trf$_K$	0.7	1.0	0.7	0.7	-	0.7
Temperature loads	Q-Temp$_K$	0.7	0.7	1.0	0.7	0.7	0.7
Environmental loads with traffic	Q-E$_{K(1year)}$	0.7	0.7	0.7	1.0	-	0.7
Environmental loads without traffic	Q-E$_{K(100year)}$	-	-	-	-	1.0	-
Other loads	Q$_K$	0.7	0.7	0.7	0.7	0.7	1.0

Tabell 4 Combination factors in the characteristic SLS condition

SLS – In-frequent

The in-frequent combination shall be used for evaluation of minimum vertical navigation clearance.

Response for the in-frequent combination shall be based on environmental loads with a 50-year return period.

Ψ_1 / $\Psi_{1,infq}$ are combination factors in accordance to table NA.A2.1 in NS-EN1990:2002+A1:2005+NA:2016.

The in-frequently occurring condition shall be use for control of the compression zone height and cracking, when traffic and environmental loads occur simultaneously.

The table below shows the principles for combining loads at the in-frequent occurring serviceability limit.

Combination factors in the in-frequent occurring SLS condition					
Dominant loads		Q-Trf$_K$	Q-Temp$_K$	Q-E$_{env(50\text{-}year)}$	Q$_K$
		$\Psi_1 / \Psi_{1,infq}$	$\Psi_1 / \Psi_{1,infq}$	$\Psi_1 / \Psi_{1,infq}$	$\Psi_1 / \Psi_{1,infq}$
Permanent loads					
Permanent loads	G-EQ$_K$	1.0	1.0	1.0	1.0
Variable loads					
Traffic loads	Q-Trf$_K$	0.8	0.7	0.7	0.7
Temperature loads	Q-Temp$_K$	0.6	0.8	0.6	0.6
Environmental loads	Q-E$_{K(50\text{-}year)}$	0.6	0.6	0.8	0.6
Other loads	Q$_K$	0.6	0.6	0.6	0.8

Table 5 Combination factors in the in-frequent occurring SLS condition

SLS-quasi-permanent

Initial imperfections for the girder to account for deformations caused by permanent loading shall be calculated as described in 3.6.1 in [4], (Ψ_2=0, for variable loads).

Combination factors in the quasi-permanent SLS condition						
Dominant loads		Q-Trf$_K$	Q-Temp$_K$	Q-E$_{env(50\text{-year})}$	Q$_K$
		Ψ_2	Ψ_2	Ψ_2	Ψ_2
Permanent loads					
Permanent loads	G-EQ$_K$	1.0	1.0	1.0	1.0
Variable loads					
Traffic loads	Q-Trf$_K$	0.2/0.5	0.2/0.5	0.2/0.5	0.2/0.5
Temperature loads	Q-Temp$_K$	0/0.5	0/0.5	0/0.5	0/0.5
Environmental loads	Q-E$_{K(50\text{-year})}$	0/0.5	0/0.5	0/0.5	0/0.5
Other loads	Q$_K$	0/0.5	0/0.5	0/0.5	0/0.5

Table 6 Combination factors in the Quasi-permanent SLS condition

7.3.2 Ultimate limit state - ULS

From requirements in [4] section 13.12.3.1, the characteristic response of a floating bridge in ULS shall be defined based on an environmental event with a return period of 100 years, in this event the bridge shall be assumed closed for traffic.

Characteristic response from environmental and traffic loading shall be evaluated with an environmental event with a return period of 1 year.

Load combinations shall be based on combination factors in NS-EN 1990 [1] on the following manner.

Ultimate limit state – EQU

Global stability shall be checked in accordance to 13.12.4.2 in [4].

The ultimate limit state - EQU Shall be established for load combinations according to equation 6.10 in Table NA.A2.4 (A) NS-EN 1990:2002+A1:2005+NA:2016.

Ψ_0 is combination factor in accordance to table NA.A2.1 in NS-EN 1990:2002+A1:2005+NA:2016.

Load and combination factors in ULS (comb A) - EQU							
Dominant loads		G- EQ$_K$	Q-Trf$_K$	Q-Temp$_K$	Q-E$_{env(1y)}$ w/traffic	Q-E$_{env(100y)}$ No traffic	Q$_K$
		γ x Ψ_0	γ x Ψ_0	γ x Ψ_0	γ x Ψ_0	γ x Ψ_0	γ x Ψ_0
Permanent load							
Permanent load [1)]	G- EQ$_K$	1.0/0.9	1.0/0.9	1.0/0.9	1.0/0.9	1.0/0.9	1.0/0.9
Variable loads							
Traffic loads	Q-Trf$_K$	0.95	1.35	0.95	0.95	-	0.95
Temperature loads	Q-Temp$_K$	0.84	0.84	1.2	0.84	0.84	0.84
Environmental loads with traffic	Q-E$_{K(1y)}$	1.12	1.12	1.12	1.6	-	1.12
Environmental loads without traffic	Q-E$_{K(100y)}$	-	-	-	-	1.6	-
Other loads	Q$_K$	1.05	1.05	1.05	1.05	1.05	1.5

Table 7 Load and combination factors in ULS (comb A)

Ultimate limit state – STR

Capacity verification in ordinary ultimate limit state (STR) shall be according to 13.12.4.3 in [4].

The ultimate limit state - STR shall be established for load combinations according to equation 6.10a and 6.10b in Table NA.A2.4 (B) NS-EN 1990:2002+A1:2005+NA:2016

γ is load factor in accordance to table NA.A2.4(B) in NS-EN 1990:2002+A1:2005+NA:2016.

Ψ_0 is combination factor in accordance to table NA.A2.1 in NS-EN 1990:2002+A1:2005+NA:2016.

The table below shows the principles for combining loads for the characteristic values in the ultimate limit state (STR).

Load and combination factors in ULS (comb B) - STR							
Dominant loads		G- EQ$_K$	Q-Trf$_K$	Q-Temp$_K$	Q-E$_{env(1y)}$ w/traffic	Q-E$_{env(100y)}$ No traffic	Q$_K$
		γ x Ψ_0	γ x Ψ_0	γ x Ψ_0	γ x Ψ_0	γ x Ψ_0	γ x Ψ_0
Permanent load							
Permanent load [1)]	G- EQ$_K$	1.35/1.0	1.2/1.0	1.2/1.0	1.2/1.0	1.2/1.0	1.2/1.0
Variable loads							
Traffic loads	Q-Trf$_K$	0.95	1.35	0.95	0.95	-	0.95
Temperature loads	Q-Temp$_K$	0.84	0.84	1.2	0.84	0.84	0.84
Environmental loads with traffic	Q-E$_{K(1y)}$	1.12	1.12	1.12	1.6	-	1.12
Environmental loads without traffic	Q-E$_{K(100y)}$	-	-	-	-	1.6	-
Other loads	Q$_K$	1.05	1.05	1.05	1.05	1.05	1.5

Table 8 Load and combination factors in the ultimate limit state (comb B)

Ultimate limit state – GEO

Shall be in accordance to Design Basis - Geotechnical design [25] and Design basis - Mooring and anchor Rev 0 [6].

Ultimate limit state – FAT

This relates to the different fatigue contributions from waves, swell, wind, traffic and tide. There is no need to use a combinations method if all contributions are included in the same analysis, that would give an accurate stress history, which in turn will give an accurate prediction of combined fatigue damage from respective components. However, if contributions are calculated separately, these shall be combined with the following procedure. The procedure has been established by DNVGL, more details can be found in [27].

The formula given below to combine fatigue damage from wind & waves with traffic and tide. It assumes that a long-term stress range distribution has been established for environmental action, which can be derived from frequency domain analysis or from several time domain analysis. The annual fatigue damage is presented as:

$$\mathrm{D}_{yrl} = f_t \sum_{i=1}^{5} p_i \sum_{j=1}^{k} \frac{1}{a} n_j \left(\Delta\sigma_{wj} + \Delta\sigma_i + \Delta\sigma_{tide}\right)^m + \sum_{i=1}^{5} (f_i - f_t \cdot p_i) \sum_{j=1}^{k} \frac{1}{a} n_j \left(\Delta\sigma_{wj} + \Delta\sigma_i\right)^m + \left(1 - \sum_{i=1}^{5} f_i\right) \sum_{j=1}^{k} \frac{1}{a} n_j \left(\Delta\sigma_{wj}\right)^m$$

Where:
f_t – fraction of tidal cycles relative to the number of environmental cycles
p_i – fraction of lorry type i relative to the total number of different lorry types
f_i – fraction of lorries of type i relative to number of environmental cycles
n_j – annual number of cycles in stress block j
a – intercept of the design S-N curve with the log N axis
m – negative inverse slope of the S-N curve
k – number of stress blocks
$\Delta\sigma_{wj}$ – stress range at hot spot due to environmental action in in block j
$\Delta\sigma_i$ – stress range at hotspot due to lorry type i

The fatigue damage contribution from tide is calculated with an equivalent stress range. The equivalent stress range should be calculated by using a long-term distribution of tide. Then the equivalent stress range can then be calculated with the following expression.

$$\Delta\sigma_{tide} = \left(\frac{\sum_{j=1}^{k} \left(\Delta\sigma_{tide\ j}\right)^m \cdot n_j}{\sum_{j=1}^{k} n_j} \right)^{1/m}$$

k – number of stress blocks
$\Delta\sigma_{tide\ j}$ – stress range in block j due to tidal variation
n_j – number of cycles in stress block j
m – negative inverse slope of S-N curve, 3.0 as it is assumed that the stress range due to tidal variation should be combined with the left part of the S-N curve.

There can be more than one stress cycle at a hotspot from a passing lorry, especially for lorries with several axles. If that is the case, the stress cycle with the largest stress range shall be used in the expression for annual fatigue, as presented above (as $\Delta\sigma_i$). Any remaining stress cycles shall not be neglected, but the fatigue damage from these cycles can be calculated without addition of the stress ranges from environmental action. Using the Palmgren-Miner rule the fatigue damage from these cycles should finally be added to the accumulated fatigue damage.

As stated previously, this procedure is based on a long-term distribution of stress ranges. It is possible to split the expression up, calculate and combine fatigue damage from individual sea states, with respective stress range histograms before summing all sea states in a year, giving a yearly fatigue damage. Splitting the calculation up into individual sea states can give the designer a better understanding of what environmental contributions are important with respect to fatigue damage of the structure. Splitting this expression up is considered an equally viable option for fatigue calculations.

This procedure is based on that fatigue from environmental action and traffic are calculated separately, if the long-term distribution of stress ranges are calculated with a combined stress time history of environmental action and traffic, the following expression can be used to account for the combined effect including the fatigue damage contribution from tide.

$$D_{yrl} = f_t \sum_{j=1}^{k} \frac{1}{a} n_j \left(\Delta\sigma_{wtj} + \Delta\sigma_{tide}\right)^m + (1 - f_t) \sum_{j=1}^{k} \frac{1}{a} n_j \left(\Delta\sigma_{wtj}\right)^m$$

Where:
$\Delta\sigma_{wtj}$ – stress range at hot spot in block j, established with combined stress time series from environmental action and traffic.

n_j – annual number of cycles in stress block j, from long-term distribution of combined environmental action and traffic.

7.3.3 Accidental limit state - ALS

The accidental limit state shall be verified in accordance to 13.12.4.5 in [4].

The accident limit state shall be verified through two stages, a and b, with load factors as given in the table below.

a: The structure in a permanent situation is subjected to an accident load. The purpose is to control the magnitude of local damage for such an action.

b: The structure in damaged condition. A damaged condition can be local damage as stated in a, or any other more explicitly defined local damage.

Design values for loads in the accident state are in accordance to Table NA.A2.5 in NS-EN 1990:2002+A1:2005+NA:2016.

Characteristic response from abnormal environmental loads shall be calculated based on a 10.000 year return period.

Characteristic response in a damaged structure shall be calculated based on a 100-year return period.

Minimum two mooring lines shall be assumed lost during 100-year storm condition; possible transient effects shall be evaluated.

Ψ_2 is a combination factor in accordance to Table NA.A2.1 in NS-EN 1990:2002+A1:2005+NA:2016.

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

Table 9 Load combinations in the accident limit state

8 DESIGN CHECK

8.1 General

The response calculations for the structure shall be according to 13.12.3.4 in [4].

The structure shall be checked in the following limit states to verify the structural integrity and the degradation performance:

- Serviceability limit state (SLS)
- Ultimate limit state (ULS)
  - EQU
  - STR
  - GEO
  - FAT
- Accident limit state (ALS)

The response shall be determined by using recognized methods that take into account the variation of loads in time and space, the response of the structure, the relevant environmental and soil conditions, as well as the limit state that is being controlled.

The response shall be verified by linearized models, which gives intuitive understanding of the loading situation.

8.2 Characteristic response from environmental loading

The characteristic response due to environmental loading should be determined based on a long-term response analysis. In lieu of available data supporting such analyses the characteristic response shall be determined based on the most critical short term storm state of 1 hour duration. The longterm characteristic responses shall then be taken as the following fractiles from the extreme value distribution of the short term response:

In ULS: the 90% fractile
In ALS: the 95% fractile
In SLS: the 50% fractile

provided that the Coefficient of Variation of the maxima does not exceed 0,20.

Here, the short term storm states refer to the annual probability of occurrence of 10-2, 10-4 and 0,63, respectively.

It shall be documented that the number of realizations are sufficient.

8.3 Design response from environmental loading

If non-linear effects gives non-linear design values using load factors on the response, this is to be investigated and measures taken.

8.4 Geometric deviations and fabrication tolerances

ULS/ALS
Geometric deviations and fabrication tolerances shall be included in the calculations with their most unfavourable tolerance limits in situations where it can have especially unfavourable effects on the structure's safety. Geometric deviations shall be accounted for as described in the Eurocode system.

FAT

In fatigue verification, geometric deviations shall be handled according to DNVGL-RP-C203.

8.5 Stay cables and mooring lines

Non-linear behaviour in stay-cables and mooring lines shall be accounted for, unless a linear analysis can be proven sufficient.

8.6 Restoring coefficients on buoyant elements

Non-linear behaviour for roll and pitch movements shall be accounted for unless restoring coefficients can be proven within a linear range of behaviour through analysis.

8.7 Shear lag effects in beam elements

Stiffness reduction due to shear lag shall be accounted for in the global dynamic beam analysis. Effective cross-section are desirable.

It shall be documented how the effects from shear deformations are taken into account for the different types of response analysis (if use of traditionally Euler-Bernulli formulation is assumed). If the effects is neglected, it shall be documented that the vibrating modes contributing to response is outside the range were shear deformations have effect on the stiffness.

8.8 Permanent loads

Deviations from intendent permanent loading will affect the restoring coefficients as well the mass distribution of the system. Designer shall evaluate mass distribution and stability reduction in context with the contingency and verify that response is acceptable in combination with response from other loads.

8.9 Traffic loads and wind interaction

Traffic loads will affect the mass distribution and restoring coefficients of the system, especially in the high bridge. These effects shall be investigated in combination with environmental loads when relevant.

The effect of the presence of traffic on the wind coefficients and the resulting wind load shall be investigated.

8.10 Interaction on turbulence between wind and waves

The effect of the wave surface on the turbulence level and the dynamic load on bridge girder shall be investigated.

8.11 Wind induced vortex shedding

Wind induced vortex shedding and possible vortex induced vibrations shall be investigated.

8.12 Sensitivity study on swell response

A sensitivity study on the response from swell waves shall be performed to investigate the structural robustness. The sensitivity study shall reflect variations in both T_p and $H_{s.}$

8.13 Fatigue

The long-term fatigue damage the structure is subjected to should be represented by a set of discrete variable loading conditions. These conditions shall represent the combined fatigue damage generated by traffic and environmental loading, taken into account the possibility of occurrence and the relevant environmental state.

For each state, the damage shall be determined by the fatigue damage accumulation method (Palmgren-Minor rule), where the fatigue damage is expressed by:

$$d_i = \sum_{i=1}^{k} \frac{n_i}{N_i} = \sum_{i=1}^{k} \frac{1}{a} n_i (\Delta\sigma_i)^m$$

It shall be documented that the number of stress blocks are sufficient.

The total accumulated long-term fatigue damage shall then be evaluated against design life:

$$D_{1year} = \sum_{i=1}^{k} d_i \, p_i \, N_{tot}$$

$$\frac{1}{D_{1year}} \geq DFF \; DL$$

Fatigue damage in temporary phases shall also be considered.

Where:

- $D_{1\text{-year}}$ = Accumulated fatigue damage from variable loads, swell, wind-sea, wind, current, tidal variations and traffic over 1-year.
- $d_i =$ Fatigue damage to component arising in state i.
- k = number of stress blocks in histogram
- $N_i =$ number of cycles to failure at constant stress range $\Delta\sigma_i$.
- $n_i =$ number of cycles in stress block i.
- $\Delta\sigma_i$ = stress range at stress block i.
- m = negative inverse slope on the S-N curve.
- $a =$ intercept of the design S-N curve with the log N axis.
- N_{tot} = Total number of occurrences during 1-year.
- DFF = Design fatigue factor.
- DL = Design life for relevant component.
- p_i = Probability of occurrence for relevant state.

Fatigue detail analysis

The fatigue detail analyses should be based on methodologies depending on what is found to be most efficient for the relevant structural detail. The corresponding S-N curve, depending on the joint classification for the detail shall be taken from DNVGL-RP-C203 [28].

Stress concentrations from misalignment of welds shall be taken into account by stress concentration factors (SCF) from DNVGL-RP-C203 [28]. Critical local details shall be investigated by Hot spot analysis, and documented SCFs from these analyses shall be used in the fatigue damage calculation.

Design Fatigue factors
The level of safety shall correspond to NS-EN 1993-1-9 NA 3(7). Using the Design Fatigue Factor (DFF) method the following DFF factors shall be used to prove fatigue safety corresponding to safe life verification method.

- DFF=2.5 for low consequence of failure [1), 2)]
- DFF=10 for high consequence of failure [3)]

These values together with the use of the S-N curves in DNVGL-RP-C203 gives the same safety level as by using the S-N curves in EN 1993-1-9 together with factors (1.35 and 2.0) on stress range.

Note 1) This DFF should be applied to details which can be inspected and repaired without need for closure of the bridge.
Note 2) For details that can be inspected and repaired, but where the fatigue capacity can be increased without addition of significant cost, a DFF equal 5.0 is recommended.
Note 3) This DFF should be applied to details with significant consequence of a failure and that is difficult to inspect and repair

The resulting accumulated yearly fatigue damage D_{yrl}, can then be compared against the fatigue life requirement of 100 years in the following way.

$$\text{Fatigue life} = \frac{1}{D_{yrl} \times DFF} > 100 \text{ years}$$

Or equivalently

$$\text{Unfactored fatigue life} = \frac{1}{D_{yrl}} > (100 \text{ years}) \times DFF$$

8.13.1 Structural damping

The following values shall be used for structural damping in fraction of critical damping, based on the logarithmic decrements given in NS-EN 1991-1-4 Table F.2 [29]:

- steel: $\zeta = 0.005$

- concrete, uncracked: $\zeta = 0.008$

- concrete, cracked: $\zeta = 0.016$

- stay cables (parallel strands): $\zeta = 0.001$

- stay cables (locked coil): $\zeta = 0.003$

Where: $\zeta = \frac{1}{\sqrt{1+(\frac{2\pi}{\delta})^2}}$

9 DESIGN CRITERIA

9.1 Stability

The stability shall be verified in ULS (EQU) according to 13.12.4.2 in [4].

The change of mass and aerodynamic coefficients for the girder, due to the presence of traffic shall be accounted for in the analysis when evaluating the 1-year condition.

Sensitivity studies of the robustness of the structure when freeboard is temporarily lost shall be conducted.

For construction parts that do not follow the rise of the tide, the freeboard shall be positive and measured from the highest water level for a tide with a 100-year return period.

9.2 Static motion limitations

Floating bridges shall be designed in such way that they are comfortable to drive on in normal conditions. Deflection and motion criteria's shall be used to ensure this.

Motion limitation	Load scenario	Maximum motion
Vertical deformation from traffic loads	0.7xtraffic	$u_y \leq 1.5m$
Rotation about bridge axis from eccentric traffic loading	0.7xtraffic	$\theta_x \leq 1.0$ deg
Rotation about bridge axis from static wind load	1-year static wind	$\theta_x \leq 0.5$ deg

Table 10 Motion limitation

9.3 Accelerations - comfort

Limitations for accelerations shall be established based on driver comfort.
The driver of a vehicle on the bridge may be subjected to vertical and lateral accelerations as well as rotational accelerations in roll and pitch. Overall Vibration Total Value (OVTV) shall be used to assess the combined exposure to accelerations from these contributions. The definition of OVTV are taken from ISO 2631-1 [30], which presents a general ride comfort evaluation framework. The formula for OVTV is presented below:

$$OVTV = \sqrt{k_{vs}^2 RMS_{vs}^2 + k_{ls}^2 RMS_{ls}^2 + k_{ps}^2 RMS_{ps}^2 + k_{rs}^2 RMS_{rs}^s + k_{vb}^2 RMS_{vb}^2 + k_{lb}^2 RMS_{lb}^2 + k_{vf}^2 RMS_{vf}^2 + k_{lf}^2 RMS_{lf}^2}$$

Where:

Multiplication factor	**Value**	**Location**	**Direction**
k_{vs}	1.00	Seat	Vertical
k_{ls}	1.00	Seat	Lateral
k_{ps}	0.40 (m/rad)	Seat	Pitching
k_{rs}	0.63 (m/rad)	Seat	Rolling
k_{vb}	0.40	Backrest	Vertical
k_{lb}	0.50	Backrest	Lateral
k_{vf}	0.40	Floor	Vertical
k_{lf}	0.25	Floor	Lateral

Table 11 Multiplication factors

The ISO framework provides in total 12 components, the longitudinal accelerations are excluded under the assumption of constant driving velocity.

RMS	Description
RMS_{vs}	RMS of vertical acceleration of seat
RMS_{ls}	RMS of lateral acceleration of seat
RMS_{ps}	RMS of pitch acceleration of seat
RMS_{rs}	RMS of roll acceleration of seat
RMS_{vb}	RMS of vertical acceleration of backrest
RMS_{lb}	RMS of lateral acceleration of backrest
RMS_{vf}	RMS of vertical acceleration of floor
RMS_{lf}	RMS of lateral acceleration of floor

Table 12 Standard deviation of acceleration components

The accelerations in back-rest, seat and floor shall be assumed equal.
As the human body are more sensitive to acceleration in certain frequency ranges, the RMS values used in the OVTV expression shall be frequency weighted. The frequency weighting shall be done in accordance with weighting factors taken from ISO 2631-1, dependent on relevant degrees of freedom.

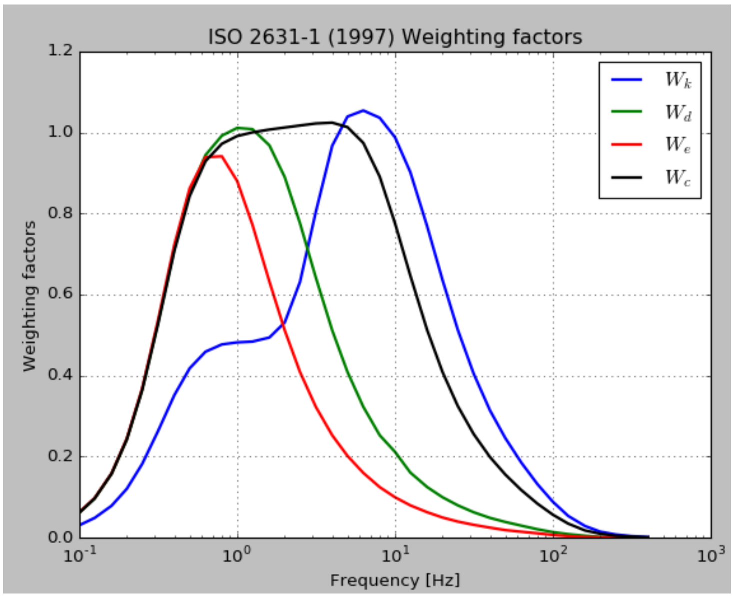


Figure 9-1 Frequency weighting functions from ISO 2631-1 (1997)

The table below shows which weighing function should be used for relevant degrees of freedom.

DOFs	Weighting function
Vertical acceleration of seat	W_k
Lateral acceleration of seat	W_d
Pitch acceleration of seat	W_e
Roll acceleration of seat	W_e
Vertical acceleration of backrest	W_d
Lateral acceleration of backrest	W_d
Vertical acceleration of floor	W_k
Lateral acceleration of floor	W_k

Table 13 Component description of frequency weighting functions

9.3.1 Vehicle models

Depending on the frequency distribution of the response of the bridge girder, it might be necessary to establish a model that takes into account the stiffness, mass and damping characteristic of representative vehicles. A model with transverse, lateral, roll and pitch DOFs are necessary to solve the problem, along with respective parameters for relevant vehicle classes. There does exist theoretical models that can be implemented in python/matlab to calculate transfer functions or alternatively perform the entire calculation in frequency domain. As can be seen in the frequency weighting functions from ISO 2631-1 it is primarily response frequencies higher than 1 Hz which is governing with respect to driving comfort. This is strongly related to the range one will typically find the first vertical eigenmode of a vehicle, which usually is the lowest eigenfrequency of the system. If it can be shown that the bridge response will not contribute to any significant dynamic amplification of vehicle response, one can investigate accelerations with regards to driving comfort without the use of a vehicle model. In such a case vehicle accelerations can be taken as accelerations of the bridge girder that a vehicle with forward speed will encounter.

The vehicle properties (mass, stiffness and damping distributions) shall applied with agreement from client.

9.3.2 Response contributions

The following effects should be considered in order to establish vehicle response:

9.3.2.1 Dynamic response of the bridge

The dynamic motion of the bridge girder that needs to be considered in this context, is the motions of the girder that a moving vehicle will encounter as it is crossing the bridge with a given speed. Since the vehicle is moving across the bridge, it is not sufficient to evaluate the accelerations at fixed points along the bridge. Which means results from dynamic analysis needs to be processed before one can establish load spectrums. It can be done by drawing a start time for a vehicle randomly, and then calculate the position of the vehicle on the bridge at given times. Then interpolating the deformations to the actual location at each time step, which consequently results in a timeseries of road surface elevation for a given vehicle. This should be repeated randomly for several vehicles, in order to get reasonable estimates for the load spectrum.

9.3.2.2 Wind action on vehicles

Wind actions on vehicles should calculated directly from the wind velocity’s series, which is input to the global response model. Vehicle wind coefficients shall be implemented in agreement with client.

9.3.3 OVTV Limits

The table below lists the subjective indication related to specific OVTV [30].

OVTV value [m/s^2]	**Subjective indication**
0.000 – 0.315	Not uncomfortable
0.315 – 0.630	A little uncomfortable
0.500 – 1.000	Fairly uncomfortable
0.800 – 1.600	Uncomfortable
1.250 – 2.500	Very uncomfortable
2.000 – ∞	Extremely uncomfortable

Table 14 Acceleration thresholds according to ISO 2631-1

9.3.4 Acceptance criteria

Accelerations are considered acceptable if one can keep the bridge open for traffic for all vehicle classes with a reduced speed limit of 70 km/h during a 1-year environmental event, the OVTV should be limited to $0.315 m/s^2$.

If this is not fulfilled, an uptime assessments for the relevant vehicle classes shall be documented, where the necessary reduced speed limit during the year, shall be reflected.

9.4 Boundary conditions at abutments

Chapter 13.12.1.1 in N400 restricts the abutments to have hinges for driving velocities larger than 70km/h. For velocities less than 70km/h the following rotations at the hinges are allowed:

- Tidal variation θ_h=2.5%
- Tidal variation and wave response θ_h=3.5%
- SLS – Characteristic θ_v=3.0%

If hinges are introduced, this shall be under the assumption of local speed reduction in the areas of the abutments. Hinges and speed reduction shall be in agreement with the client.

9.5 Concrete structures

9.5.1 General

Concrete structures shall be designed in accordance to NS-EN 1992-1 [8] and NS-EN 1992-2 [9] .

9.5.2 Crack widths

Crack widths shall be checked in the in-frequent occurring SLS condition.

9.6 Steel structures

9.6.1 General

Steel structures shall be designed in accordance to NS-EN-1993.

9.6.2 Structural components specially subjected to fatigue

Structural components specially subjected to fatigue shall be available for inspection.

9.6.3 Water runoff on pontoon top plate

The pontoon top plate shall have an angle of 3% to the horizontal plane to ensure an appropriate water runoff.

10 BEARINGS AND EXPANSION JOINTS

10.1 Bearings

10.1.1 General

The bearings shall have a service life of 100 years, unless parts of the bearings are replaceable and can be changed during the bridge service life. Such operations shall be planned and facilitated in the design, to ensure a cost efficient replacement and low interruption in the normal use of the bridge.

Piers, abutments and superstructures shall be prepared for jacking equipment if parts of the bearings are planned to be changed during the service life.

10.1.2 Design

Maximum forces and displacements are determined in the ultimate and serviceability limit states. Calculated values shall not exceed the capacity guaranteed by the supplier.

It shall be ensured that the joint/bearing structure's displacement and rotational capacity is adequate for the applied calculation model for checking of ultimate limit state.

Degradation mechanisms of moveable parts shall be documented. Lifetime- and replacement analysis of bearings shall be documented through design.

10.2 Expansion joints

10.2.1 General

The expansion joint shall allow for snow ploughing, and shall be dampened to avoid unnecessary noise.

Expansion joints shall not be placed at the bottom of sag-curves.

Water runoff systems shall be included beneath the expansion joint, to make sure that water does not run down on underlying structures.

Expansion joints shall be easily accessible. The expansion joint's wearing parts shall be possible to disassemble for one driving lane at a time. Fasteners shall be resistant in contact with seawater and easy to detach when being replaced.

10.2.2 Design

Expansion joint displacement and rotation shall not exceed the upper deformations values given by the supplier.

In the SLS (characteristic), the distance between joint edges or slats that can be in contact with the wheels will not exceed 80 mm (N400 12.5.4).

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