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Actions and action effects

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Actions and action effects

Content

Foreword

NORSOK N-003:2017, was adopted as NORSOK Standard in January 2017.

[AC⟩ Corrigendum NORSOK N-003:2017/AC:2018 (indicated in the text by tags [AC⟩ ⟨AC]) was incorporated into NORSOK N-003:2016 in May 2018. ⟨AC]

NORSOK N-003:2017, replaces NORSOK N-003, rev. 2, September 2007.

NORSOK is an acronym for the competitive position of the Norwegian continental shelf and comprise petroleum industry standards in Norway. The collaboration initiative in 1993 between the authorities and the petroleum industry initiated the development of NORSOK standards.

Reducing the project execution time and developing and operating cost for petroleum installations on the Norwegian shelf was the target.

The intention for the Petroleum industry is to develop and use standards providing good technical and cost effective solutions to ensure that the petroleum resources are exploited and managed in the best possible way by the industry and the authorities. The industry will actively contribute to the development and use of international standards in the global market.

The NORSOK standards shall:

- bridge the gap based on experiences from the Norwegian continental shelf where the international standards are unsatisfactorily;
- replace oil company specifications where possible;
- be available as references for the authorities' regulations;
- be cost effective;
- promote the Norwegian sector as an attractive area for investments and activities.

Developing new NORSOK standards and regular maintenance of existing standards shall contribute to maintain the competitiveness both nationally and internationally for the Norwegian petroleum industry.

The NORSOK standards are developed by experts from the Norwegian petroleum industry and approved according to the consensus principles as laid down by the guidelines given in this NORSOK directive.

The NORSOK standards are owned by the Norwegian Oil and Gas Association, the Federation of Norwegian Industries and the Norwegian Shipowners´ Association. They are managed and published by Standards Norway.

The principal standard for offshore structures is NORSOK N-001, Structural design, which refers to ISO 19900, Petroleum and natural gas industries – General requirements for offshore structures.

Main changes in this edition compared to Edition 2 are:

- The inclusion of more complete description of cold climate actions from sea ice, icebergs, icing and snow.
- Updates of accidental actions due to ship collision risk.
- The standard now allows the use of hindcast data as basis for design.
- Air gap recommendations included.
- Adjustments are made to the recipe for wave action effects on jacket structures, including wave kinematics and wave crest.
- The standard now allows for the use of CFD methods in wave load calculation.
- Wave impacts actions are updated to include impacts from breaking waves, slamming on vertical structures, and wave impact in floater deck.
- Action combinations are updated e.g. to include cold climate actions in ULS and ALS. ALS in damaged condition is updated for actions uncorrelated with the accidental event.
- Sections on design waves, contour method, and long-term analysis are rewritten and updated according to present knowledge.
- The inclusion of a section on climate change.
- The preparation for the removal of NORSOK N-002 Collection of Metocean Data has resulted in the inclusion of some issues into this standard, e.g. recommended duration of metocean measurements. The major part of NORSOK N-002 is found in ISO 19901-1 Metocean Design.

– The informative Annexes A, B and C are new.

1 Scope

This NORSOK standard specifies general principles and guidelines for determination of characteristic actions and action effects for design, assessment and verification of structures. Reference is made to NORSOK N-001 (and NORSOK N-006) as to how this standard is to be used to achieve the desired safety level for new and existing structures.

This NORSOK standard is applicable to all types of offshore structures used in the petroleum activities, including bottom-founded structures as well as floating structures, including substructures, topside structures, vessel hulls, foundations, mooring systems, risers and subsea facilities.

This NORSOK standard is primarily written for the design of new facilities on the Norwegian continental shelf (including the continental shelf of Svalbard) as defined by NPD 16.06.2014, but the principles may also be applicable for other areas.

2 Normative references

The following standards include provisions and guidelines which, through reference in this text, constitute provisions and guidelines of this NORSOK standard. Latest issue of the references shall be used unless otherwise agreed. Other recognized standards may be used provided it can be shown that they meet the requirements of the referenced standards.

DNVGL-OS-C101, *Design of offshore steel structures, general – LRFD method, Ch.2, Sec. 2.4.3 Tank pressures (Ed 2015).*

NORSOK N-001, *Integrity of offshore structures*

FOR-2007-10-26-1181, *Forskrift om kontinentalsokkelflyging - ervervsmessig luftfart til og fra helikopterdekk på innretninger og fartøy til havs*

3 Terms, definitions and abbreviations

3.1 Terms and definitions

3.1.1
action
assembly of concentrated or distributed forces acting on a structure (direct actions), displacements or thermal effects imposed to the structure, or constrained in it, or environmental influences that may cause changes with time in the material properties or in the dimension of a structure (ISO 2394)

3.1.2
action effect
effects of actions (or action effects) is a result of action on a structural member (e.g. internal force, moment, stress, strain) or on the whole structure (e.g. deflection, rotation) (ISO 2394)

3.1.3
air gap
vertical distances measured from still water level (still water air gap) or top of wave crest (air gap) to bottom of steel of deck structure.

3.1.4
can
AC expression in the content of a document conveying expected or conceivable material, physical or causal outcome.

Note 1 to entry: Possibility and capability is expressed using the verbal forms specified in Table 6. AC

3.1.5
characteristic value
value specified preferably on statistical bases, so it can be considered to have a prescribed probability of not being exceeded (ISO 2394)

3.1.6
design (value of an) action
value of an action used in semi-probabilistic verification calibrated to the reliability target. In the partial factor method, the value is obtained by multiplying the representative (characteristic) value by the partial factor (ISO 2394)

3.1.7
design premises
set of project specific design data and functional requirements which are not specified or are left open in the general standard

3.1.8
epistemic uncertainty
uncertainties originating from lack of knowledge and understanding of the problem under consideration. Epistemic uncertainties can be reduced by e.g. collection of more data or further investigation regarding statistical models and action models

3.1.9
freeboard
vertical distances measured from still water level to main deck

3.1.10
inherent uncertainty
aleatory or inherent variability (randomness in nature) typically associated with the metocean parameters, the geometry of the structure, and the material properties

3.1.11
may (permission)
expression in the content of a document conveying consent or liberty (or opportunity) to do something

Note 1 to entry   Permissions are expressed using the verbal forms specified in ISO/IEC Directives Part 2 clause 7.4 Table 5.

3.1.12
metocean
meteorological and oceanographic conditions (including sea ice, icebergs and icing for cold climate areas) for the determination of actions and action effects for the design, construction and operation of offshore structures. Metocean actions are one type of environmental actions

3.1.13
most probable maximum
value of the maximum of a variable with the highest probability of occurring AC within a specified temporal period or spatial area AC

3.1.14
q-probability
annual probability of exceedance equal to q. For q less than 0,1 this corresponds approximately to a return period of 1/q years

3.1.15
sea state
condition of the sea during a period in which its statistical parameters are assumed to remain constant, see stationary condition

3.1.16
stationary condition
the duration of a stationary metocean event is 3 hours unless stated otherwise

3.1.17
shall (requirement)
expression in the content of a document conveying objectively verifiable criteria to be fulfilled and from which no deviation is permitted if compliance with the document is to be claimed

AC Note 1 to entry: Requirements are expressed using the verbal forms specified in ISO/IEC Directives, Part 2 clause 7.2 Table 3. AC

3.1.18
should (recommendation)

expression in the content of a document conveying AC a suggested possible choice or course of action deemed to be particularly suitable without necessarily mentioning or excluding others

Note 1 to entry: Recommendations are expressed using the verbal forms specified in ISO/IEC Directives, Part 2 clause 7.3 Table 4.

Note 2 to entry: In the negative form, a recommendation is the expression that a suggested possible choice or course of action is not preferred but it is not prohibited. AC

3.1.19
slender members or slender structures (geometrically)
structural members that are geometrically slender, i.e. the length to cross-sectional dimension ratio is large

Note 1 to entry: Slender members and slender structures are used in a variety of meanings, e.g. for determining the use of the drag term in the Morison formula. In this standard slenderness is, as the definition states, only used for geometrically slenderness.

3.1.20
target extreme value
extreme value corresponding to an annual probability of exceedance equal to q. For q less than 0,1 this corresponds approximately to a return period of 1/q years

3.1.21
verification
examination to confirm that an activity, a product or a service is in accordance with specified requirements

3.1.22
water level

mean water level (MWL):	arithmetic mean of all sea levels measured over a long period
highest astronomical tide (HAT):	level of high tide when all harmonic components causing the tides are in phase
lowest astronomical tide (LAT):	level of low tide when all harmonic components causing the tides are in phase
still water level (SWL):	abstract water level used in the calculation of elevations at which actions are applied, normally the highest possible water level in extreme weather (see Figure 1). If a low water level is governing (e.g. TLP tendons in slack), LAT can be taken as the still water level (SWL) in extreme weather conditions. In good weather, a water level lower than LAT is possible due to a negative storm surge caused by very high air pressure.

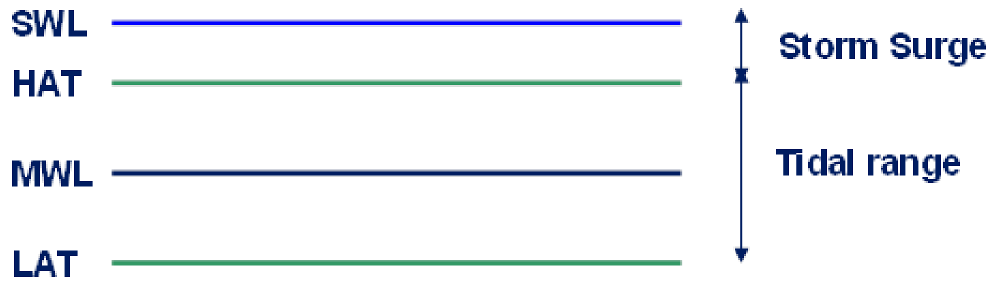


Figure 1 – Still water level

3.2 Abbreviations

ALS	accidental limit states
CQC	complete quadratic combination
DAF	dynamic amplification factor
DP	dynamic positioning
EOF	empirical orthogonal functions
FE	finite element
FLS	fatigue limit states
FPSO	floating production storage and offloading unit
GBS	gravity based structure
HAT	highest astronomical tide
HF	high frequency (higher than wave frequency)
ISO	International Organization for Standardization
KC	Keulegan-Carpenter number
LF	low frequency (lower than wave frequency)
MWL	mean water level
NMA	Norwegian Maritime Authority (Sjøfartsdirektoratet)
NFR	Norges Forskningsråd (The Research Council of Norway)
NS	Norsk Standard (Norwegian Standard)
OTM	overturning moment
P-Δ effect	second order effect of an axial force (P) due to a lateral displacement, Δ
POT	peak-over-threshold
SLS	serviceability limit states
SRSS	square root of sum of squares
SWL	still water level
TLP	tension leg platform
ULS	ultimate limit states
VIV	vortex induced vibrations
WCT	wind chill temperature
WF	wave frequency

4 Permanent actions

4.1 General

Permanent actions are actions that will not vary in magnitude, position or direction during the time period considered. Examples are:

a) weight of the structure;
b) weight of permanent ballast and equipment, including mooring systems and risers;
c) external hydrostatic pressure up to the mean water level;
d) pretension (static tension in initial position).

Characteristic permanent actions are defined by the expected value.

4.2 Hydrostatic pressure difference

Structural components subjected to high counteracting hydrostatic pressures, e.g. external walls in fixed concrete structures with in the order of 100 m and higher differential water pressure height, should be designed taking into account the possible uncertainties due to possible variation in:

– level and density;
– dimension tolerances;
– measuring inaccuracies;

and other uncertainties affecting the pressure difference.

Unless documented otherwise by detailed analyses of operations, the minimum pressure difference for the ultimate limit states should be at least equal to the smallest of one tenth of the maximum pressure and 0,1 MPa.

5 Variable functional actions

5.1 General

Variable actions originate from normal operation of the structure and vary in position, magnitude and direction during the period considered. They include AC⟩ actions ⟨AC from

a) persons,
b) helicopters,
c) lifeboats,
d) cranes,
e) tank pressures and weights,
f) stored goods,
g) modules and structural parts that can be removed,
h) weight, pressure and temperature of gas and liquid in process plants,
i) local pressure and global effect of variable ballast,
j) installation and drilling operations.

Assumptions regarding variable actions shall be reflected in the operational manual, and complied with in operation. Possible deviations from the assumed value due to operational errors or mechanical failures or damages shall be treated as accidental actions, see Clause 9.

Characteristic values of variable functional actions are defined as a specified value not to be exceeded.

5.2 Crane actions

Crane actions shall be determined with due account of dynamic effects of crane and, if applicable, the motions of the facility.

Fatigue calculations shall be carried out based on expected frequency of crane usage, the magnitude of actions, dynamic effects from wind, loading and discharging of ships and, if applicable, from motions of the facility.

5.3 Deck area actions

Variable actions on deck areas of the topside structure shall be stated in the design premises and reflected in the structural load plan drawing or the operational manual and shall be complied with in operation.

Variable actions on deck areas of the topside shall as a minimum be based on Table 1 unless larger actions are specified in the design premises. The distributed actions are set to depend on local/global aspects of the deck structure to take into account the spatial variability of the actions (e.g. local peaks in lay-down area need to be accounted for in local design, but can be distributed for design of primary steel). The following notations are used:

Local design:	design of deck plates, stiffeners and local effects on deck beams and columns;
Primary design:	design of deck beams, beam-columns and complete modules;
Global design:	design of global loadbearing structure such as topside main structure, substructure and foundation (e.g. jacket, hull, piles, mooring and anchors).

Table 1 – Minimum variable actions in deck areas

Area	**Local structure design** See 1,2,3 and 5		**Primary structure design**	**Global structure design** See 4
	Distribution action, p_d (kN/m^2)	**Point action,** P (kN)	**Apply factor given below to distributed action,** p_d (A = the action area in m^2)	**Apply factor given below to distributed action,** p_d
Storage areas, See 6	p_d	$1{,}5 \cdot p_d$	1,0	1,0
Laydown areas, See 6	p_d	$1{,}5 \cdot p_d$	min(1,0;(0,5 + 3/$A^{0,5}$))	min(1,0; (0.5 + 3/$A^{0,5}$))
Lifeboat platforms	9,0	9,0	1,0	may be ignored
Area between equipment	5,0	5,0	min(1,0; (0,5 + 3/$A^{0,5}$))	may be ignored
Walkways, staircases and platforms, crew spaces	4,0	4,0	min(1,0; (0,5 + 3/$A^{0,5}$))	may be ignored
Walkways and stair-cases for inspection and repair only	3,0	3,0	min(1,0; (0,5 + 3/$A^{0,5}$))	may be ignored
Areas not exposed to other functional actions	2,5	2,5	1,0	may be ignored
Roofs, accessible for inspection and repair only	1,0	2,0	1,0	may be ignored

1. Wheel actions to be added to distributed actions, where relevant. Wheel actions can normally be considered acting on an area of 300 mm x 300 mm.
2. Point actions to be applied on an area 100 mm x 100 mm, and at the most severe position, but not added to wheel actions or distributed actions.
3. p_d is to be evaluated for each case. Storage areas for cement, wet or dry mud should be the maximum of 13 kN/m^2 and ρgH, where H is the storage height in m. Laydown areas not normally to be designed for less than 15 kN/m^2.
4. Global action cases should be established based on specified values of the variable action combinations, complying with the limiting global criteria to the structure. For buoyant structures these criteria are established by requirements to the floating position in still water and intact and damage stability requirements, as documented in the operational manual, considering variable actions on the deck and in tanks.
5. Transport routes and laydown areas shall be designed for a forklift truck with minimum capacity of 1500 kg with corresponding wheel loads of 1750 kg. The capacity of laydown areas and transport routes shall be described in the Material Handling Philosophy.
6. Storage areas are not normally in use for daily crane operations, but used for temporary equipment- and service-containers etc, that may be manned. Laydown areas are areas used for daily crane-operations, but not for permanent storage. Laydown areas are normally provided with flexible barriers.

For actions on floors in accommodation and office sections, ISO 2103 shall be used.

Guard rails should be designated for the following actions:

- A horizontal line action of 1,5 kN/m acting on the handrail.
- A point action of 1,0 kN acting in worst location and worst direction (horizontal or vertical). The point action does not act together with the horizontal line action.
- Actions from possible attachments shall be established for each case.
- Actions on guard rails in areas with cargo handling should be determined with due account of relevant operational conditions.

Unless specified otherwise the following design-energy shall form the basis for design of a typical barrier on a laydown area subject to rough handling:

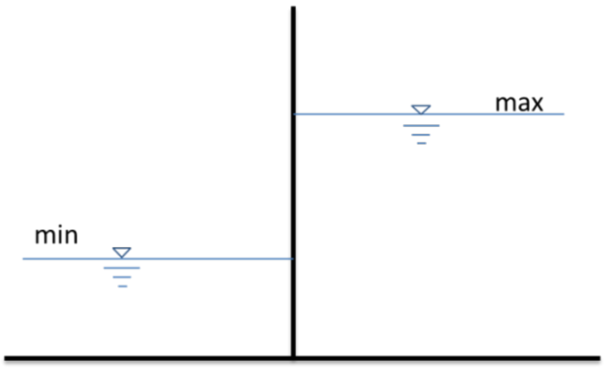
- General requirements:
  - Top-rail shall be designed for an impact energy of 400 J. Impact from above (lowering hook-load) shall be documented.
  - The barriers shall be designed with continuity so that loads sliding along one section of the barrier-structure will not be stopped towards next section.
- Fixed installations:
  - External sea side barriers, to stop swinging and rotating loads: Impact energy from swinging loads: 20 kJ;
  - Internal barriers, to stop stabilized loads: 9 kJ.
- Floating installations:
  - External sea side barriers, to stop swinging and rotating loads: Impact energy from swinging loads: 36 kJ;
  - Internal barriers, to stop stabilized loads: 20 kJ.

5.4 Tank pressures and tank weight

5.4.1 Hydrostatic pressures

The structure shall be designed to resist the maximum hydrostatic pressure of the heaviest filling in tanks that may occur during fabrication, installation and operation. The hydrostatic pressures in tanks shall be determined as described in Ch.2, Sec. 2.4.3 Tank pressures DNVGL-OS-C101.

For internal tank walls the most unfavourable combination of tank pressures on each side of a tank wall should be applied, e.g. the maximum and minimum pressure on each side, see Figure 2.



Figur 2 – Pressures on internal tank walls

For external surfaces facing the sea the relevant condition of external pressure should be considered, including hydrostatic and hydrodynamic pressures both 1) inside in tank and 2) outside in the sea, in such a way that the most conservative pressure difference is chosen as basis for the design actions.

5.4.2 Ballast

Tanks, pipes, etc. shall be designed to resist local pressures as described in 5.4.1.

The structure should be designed to resist the maximum uneven distribution of fluid consumables and ballast in tanks that may occur during fabrication, installation and operation.

Pressure actions that may occur during emptying of water or oil filled structural parts for condition monitoring; maintenance or repair shall be evaluated.

5.5 Variable actions in temporary phases

Lifting actions are imposed on the facility or parts thereof during fabrication and installation phases. Such actions should be determined with due consideration of weight and weight growth, shift of centre of gravity, dynamic effects as well as fabrication and sling tolerances.

Guidance on actions and action effects during marine operations may be found in [AC) DNV-RP-N103 (AC] and ISO 19901-6.

6 Metocean actions

6.1 General

6.1.1 Metocean data for design and operation

Metocean actions are one type of environmental action. Metocean actions originate from various metocean processes or metocean events and their actions and action effects shall be assessed for operations and structures. They will typically be associated with very large inherent randomness both with regard to occurrence rates and to magnitude. In the context of this standard, metocean actions include actions from:

- waves;
- wind;
- current;
- snow and icing;
- sea ice and icebergs;
- varying water level;
- air temperature and sea temperature.

Assessments of actions may depend on the particular application, i.e. whether activity is control against:

- ultimate and accidental limit states;
- fatigue limit states;
- serviceability limit states.

The assessment and need for metocean actions may also differ by the type of operation under consideration, e.g. design of a permanent offshore structure versus planning of a temporary operation within a given season.

The parameters describing the metocean conditions shall be based on observations or reliable hindcast data from, or in the vicinity of, the relevant location and on general knowledge about the metocean conditions in the area. When hindcast data is used, such data shall be calibrated against measured data of good quality. For the Norwegian continental shelf, the hindcast database NORA10 (Reistad et al., 2011) is recommended.

For combination of metocean actions, Table 7 shall be used in lack of more detailed data giving joint occurrence probability.

More accurate estimation of design actions can be based on joint occurrence of e.g. wave, wind and current conditions. The latter approach can be utilized provided the length of simultaneous data is sufficiently long. The duration of the database with simultaneous observations shall be sufficiently long to capture the probability of occurrence for all combinations of importance regarding predictions of actions and action effects. Regarding wind and waves this is assumed fulfilled by using a high quality and verified hindcast database (e.g. NORA10). Regarding a joint modelling involving current, availability of simultaneous data will in general be limited. Collection of joint data is then necessary. The time period of simultaneous wave and current data should be at least 3 years if a joint occurrence approach involving current is to be applied, see also 6.2.3. A model for associated current in extreme wave and wind events shall be applied in combination with the data. The model shall account for the expected change in correlation of current and waves from low and moderate sea states (significant wave height less than 10 m) to sea states of severity of importance regarding prediction of extremes (significant wave height larger than 14 m).

For time-limited operations, it is sufficient to consider the relevant months when determining actions, see i.e. DNVGL-ST-N001

6.1.2 Possible consequences of climate changes

Climate changes may lead to change in metocean conditions compared to present basis for structural design and assessment. For permanent facilities with a planned service life of more than 50 years, climate change shall be accounted for.

Some additional information can be found in the commentary, see A.2.

6.1.3 Determination of characteristic metocean actions

6.1.3.1 General

The metocean actions shall be determined with estimated annual probabilities of exceedance. Characteristic actions for the design of structures in the in-place condition shall be defined so that the annual exceedance probabilities of the action effects correspond to 10^{-2} (ULS/SLS) and 10^{-4} (ALS). For FLS, the expected action history shall be used. The statistical analysis of measured data or simulated data should make use of different statistical methods to evaluate the sensitivity of the result. The validation of distributions with respect to data should be tested by means of recognised methods.

A necessity for an adequate prediction of metocean actions and action effects for design requires an adequate description of the metocean conditions. The particular needs will depend on which limit states are considered and may also vary from platform concept to platform concept. Modelling of metocean conditions for the purpose of predicting actions and action effects for the various applications is found in 6.2.

The analysis of the data shall be based on the longest possible time period for the relevant area. In the case of short time series, the statistical uncertainty should be accounted for when determining design values.

6.1.3.2 Definition of characteristic actions – ULS and ALS limit states

Characteristic metocean actions (and action effects as described in Clause 11) to be used in ULS and ALS limit states are defined in terms of their annual exceedance probability q. When estimating the characteristic value one shall account properly for all inherent uncertainty associated with the randomness of the underlying phenomenon. If one or more sources of epistemic uncertainty have a variability comparable to the inherent uncertainty, one should also account for this variability. See 3.1 for definition of inherent and epistemic uncertainty and A.3 for further discussion.

A consistent estimation of the characteristic value corresponding to a given exceedance probability requires AC⟩ ⟨AC a combination of short-term and long-term analyses. This is the case both for estimating extremes of metocean parameters and for extremes of actions and corresponding action effects.

The long-term development of a metocean action is a non-stationary stochastic process. In the analysis it is modelled as a series of piecewise short-term stationary conditions. The stationary conditions are characterized by a set of metocean parameters z_i. Examples of such parameters can be significant wave height, spectral peak period, wind speed, current speed and associated directions. Within a stationary condition, all these parameters are assumed to be constant. The duration of a stationary condition shall be taken as 3 hours.

Within a given stationary short-term condition, the value of an action variable X is given by a probability distribution which is conditional on the metocean parameters, $F_{X|Z_i}(x|z_i)$. For a general case, the major challenge is to establish an accurate model for the conditional short-term variability.

6.1.3.3 Estimation of characteristic actions

For fixed structures behaving quasi-statically, q-probability hydrodynamic actions can be estimated using the design wave method. See 6.2.1.3 and 6.3.2.3 for further specifications of the design wave method. In particular, if design wave characteristics are established conditionally with respect to directions, corrections suggested in 6.1.4 apply.

For other structures, q-probability actions and action effects should if possible be based on long-term action or action effect analyses. The resulting annual exceedance probability is estimated by a weighted sum of the contribution from each short-term metocean condition. The weights are the probability of the particular conditions.

Two essentially different approaches for the long-term analyses are recommended:

- all short-term conditions approach;
- storm event approach.

These methods are described in more detail in A.4.

In addition, the metocean contour method (an approximate method using only short-term analysis) is recommended for nonlinear problems.

Metocean contour method

If the problem under consideration is of a very nonlinear nature, an extensive model test program may be necessary to model the short-term variability for all important metocean conditions. In particular, this will be the case if the problem is of an on-off nature. For such cases, a simplified and approximate approach may be useful – in particular for early phase concept evaluations. A method suggested for this purpose is the metocean contour method. In its present formulation, it is a simplified approach for the all short-term conditions approach.

The method is described in more detail in A.4, but the main steps are listed below:

1) Establish q-probability contours of the metocean characteristics, e.g. significant wave height and spectral peak period.
2) Identify the worst metocean condition along the contour for the variable under consideration.
3) For this sea state, determine the distribution function for the 3-hour extreme value for the variable under consideration.
4) Estimate the q-probability value from the α-percentile of this distribution. For ULS α = 0,9 is recommended, while for ALS α = 0,95 should be used.

The values given above are expected to be slightly conservative if the coefficient of variation does not exceed 0,20 for the 3-hour extreme value of the variable under consideration. If the coefficient of variation exceeds 0,20, the adequacy of the contour method can be questioned and studies shall be performed in order to establish the appropriate values of α.

6.1.4 Determination of effects of directional metocean conditions

When using directional metocean criteria for obtaining characteristic actions or action effects, it shall be verified that they fulfil requirements regarding target annual exceedance probabilities. If omni-directional extremes are used for all sectors, no correction effects apply.

The directional metocean condition actually used shall result in actions fulfilling overall requirements regarding annual exceedance probabilities q. The most accurate approach is to perform a full long-term analysis, i.e. the exceedance probability is estimated for each direction and the resulting probability is taken as a weighted sum of the directional failure probabilities. The weights are the directional probability of incoming metocean conditions. Such an analysis will show that for some sectors, the design metocean condition need to be artificially adjusted in order to give adequate design actions when a long-term analysis is not carried out.

A characteristic directional wave height can be calculated as the wave height corresponding to an exceedance probability of $q/(0{,}5\ N_{DS})$, where N_{DS} is the number of directional sectors (e.g. for 12 sectors, the AC 10^{-2} AC extreme value for each direction corresponds to the 600 year return period). The characteristic wave height for a given sector is taken as the minimum of calculated equivalent characteristic wave height and the omni-directional characteristic wave height. In principle, this correction method also applies to wind and current. This approach can be used for SLS, ULS and ALS as found appropriate. The method is an approximate method. If more accurate results are needed a full long-term analysis should be carried out.

6.2 Modelling of metocean conditions

6.2.1 Wave conditions

6.2.1.1 General

The long-term non-stationary wave conditions can for practical applications be modelled as a sequence of piecewise stationary wave conditions. The duration of stationary conditions is taken to be 3 hours. The short-term stationary conditions are in a statistical sense fully described by the wave spectrum, $S(\omega)$, and the mean direction of wave propagation, θ. Possible models for the wave spectrum are suggested in 6.2.1.5.

The wave spectra can be written as the product of a frequency spectrum and a spreading function. The frequency spectra used on the Norwegian Continental Shelf are parameterized in terms of significant wave height, H_S, and spectral period, T_P. Accordingly, the long-term modelling of wave conditions is given by the joint probability density function of H_S, T_P and θ. Probabilistic modelling of these quantities are given in 6.2.1.6.

A sufficient amount of available data is crucial for long-term modelling of wave conditions. Good quality hindcast wave databases are recommended. The main advantage of hindcast data is that they typically cover a much longer time period than available wave measurements. The Norwegian hindcast database, NORA10, covers the years from 1957 to present. For preliminary considerations, values are given in A.5.

If the wave observations at the given or adjacent locations are limited, further wave data should be recorded according to the requirements of ISO 19901-1 concerning metocean data. The purpose being to validate the hindcast database at the target location.

Omni-directional metocean conditions can be modelled by pooling data from all directions. Alternatively, the metocean conditions can be described for N_{DS} directional sectors, each of width $360/N_{DS}$. $N_{DS} = 12$ is most commonly used, but $N_{DS} = 8$ or 16 may AC AC also be used. A simplified estimation of directional extremes can be done by the procedure in 6.1.4. A more accurate estimation of extremes requires that the conditional distribution of e.g. Hs and Tp for the included sectors are available. In addition to the modelling of each sector, a directional modelling also requires that the probability for the various sectors is given.

6.2.1.2 Wave theories

Irregular sea analyses

Provided the action effect under consideration is not sensitive to neither the local wave crest height nor the kinematics close to the crests, use of linear wave theory will most often give sufficient accuracy. This means that a Gaussian wave surface process can be assumed. As far as this is considered accurate, the action effect analyses can conveniently be done in frequency domain, provided structural system is of a linear nature.

If the action effect under consideration is sensitive to kinematics close to the wave crests (e.g. drag dominated structures), a second order random process shall be adopted for the surface process. Modelling of the consistent second order surface process and corresponding kinematics are detailed in DNV-RP-C205. For a second order surface process, kinematics based on Wheeler stretching may be applied for early phase design evaluations. For final design Wheeler stretching is not recommended for estimation of extreme hydrodynamic actions of drag dominated structures since this method underestimates the kinematics at still water level – in particular for steep waves. A comparison of second order kinematic models, Wheeler stretching and laboratory measurements are shown the commentary section, see AC A.6 AC.

Special consideration should be made for crest kinematics in sea states with near breaking waves or breaking waves.

Actions on flexible risers and mooring lines attached to large volume floating structures can be calculated based on linear wave theory with due considerations to wave kinematics above SWL and the effect of disturbed kinematics due to large volume wave diffraction. Consistency between floater motions and wave kinematics model used for calculating actions on attached slender structures should be ensured.

Regular wave analysis

For fixed structure members with cross-sectional dimensions much less than the wavelength ($^D/_\lambda < 0{,}2$, a regular Stokes 5th order wave (Fenton, 1985) shall be used for ULS and ALS analysis. For floaters linear sinusoidal waves can be used provided the characteristic wave height is calibrated against irregular sea analyses.

Water depth limitation

The validity of regular wave theories in shallow water is determined by the Ursell number $U_r = \frac{H}{k^2 d^3}$ where H is wave height, k is wave number and d is the water depth. Airy and Stokes wave theory (second-order and higher) is valid if $U_r < 2/3$. The same criterion applies to second order random waves where the Ursell number is defined in terms of significant wave height H_S and wave number k_p corresponding to peak spectral period T_P (Stansberg, 2011).

6.2.1.3 Regular design waves

A design wave is an engineering abstraction in terms of a regular wave used for the calculation of quasi-static characteristic actions for structural design of an offshore structure.

Most often a periodic wave with suitable characteristics is used (e.g. height H, period T, crest elevation C, and direction). The wave characteristics are determined to give the same action(s) and action effect(s) as could have been obtained by a long-term analysis. The choice of a design wave depends on

- the design purpose(s) considered (e.g. ultimate strength or fatigue limit states),
- the wave environment (e.g. deep or shallow water),
- the geometry of the structure, (e.g. due to phase, certain wavelengths may be governing regarding extreme actions),
- the type of action(s) or action effect(s) pursued (e.g. to which extent structure is affected by dynamics).

For assessment of ultimate strength, different combinations of wave periods, wave heights and directions at the same probability level q (e.g. 10^{-2} or 10^{-4}) shall be considered in order to arrive at the most unfavourable values for the different action effects. The wave height and wave crest shall be determined using long-term analysis, see 6.1.3.

6.2.1.4 Non-periodic waves

As an alternative to periodic regular wave theories, representative waves from a random sea derived with non-periodic wave theories such as “New-wave” theory may be used. See ISO 19901-1 for further recommendations.

6.2.1.5 Design irregular wave conditions

If a regular wave is not applicable and a full long-term analysis is out of reach due to problem complexity, the use of a subset of stationary irregular wave conditions represent an approximate tool. Stationary metocean conditions are in this context a 3-hour sea state described by constant wave characteristics. The subset of sea states should be selected along the q – probability contour for the wave characteristics if the aim is to estimate actions or actions effects corresponding to an annual exceedance probability of q. One implementation of the design irregular sea state is the metocean contour method. This method is briefly introduced in 6.1.3.3 and is also discussed in some detail in A.4. The sea state input for this method is the q-probability contours for H_S and T_P.

Other implementations of the design irregular wave conditions can be used if it is validated that the annual exceedance probability of selected extreme action or action effect is not larger than the target exceedance probability q.

The design sea conditions are described by a wave spectrum. Wave spectra should represent wind-induced waves and swell, if relevant. Sea-states comprising unidirectional wind-waves and swell should be represented by recognized double-peaked spectra, e.g. the frequency spectrum proposed by

Torsethaugen (2004)[1] . The JONSWAP spectrum (Hasselmann et al., 1973) may be used to describe pure wind sea conditions. JONSWAP spectrum can also be used as an approximate way of characterizing swell seas. In case of a very pronounced swell system (i.e. swell system originating from a remote storm event), high values for the JONSWAP peak enhancement factor should be used, i.e. γ = 10. See i.e. DNV-RP-C205 for the mathematical expressions for the wave spectra.

If windsea conditions and swell conditions with different mean directions are critical, due account of such conditions shall be made. Hindcast databases such as NORA10 or NEXT may be useful for such cases, since they provide simultaneously occurring wind sea and swell.

The short-crestedness of wind-induced waves may be described by a spreading function:

$$D(\theta - \theta_m) = C \cos^n(\theta - \theta_m) \qquad \text{for} \ -\frac{\pi}{2} \leq \theta - \theta_m \leq \frac{\pi}{2} \tag{1}$$

where θ_m is the mean wave direction and C is given by:

$$C = \frac{\Gamma(1+n/2)}{\sqrt{\pi}\Gamma(1/2+n/2)} \tag{2}$$

where $\Gamma(\ )$ is the Gamma function.

As n increases the directional spreading decreases, hence the sea state becomes more long-crested.

For cases where long-crested sea is conservative, it is recommended to use long-crested sea for the original design work. If short-crested sea is introduced in connection with estimating extremes, the exponent, n, shall not be taken lower than 10 without a more detailed documentation. Swell sea should be taken as long-crested. For fatigue assessment, where low and moderate sea states are governing the fatigue accumulation, n could be taken as the most unfavourable between 2 and 6. See also AC A.7 AC. The spreading function given above or formulations equivalent to this form are expected to be of sufficient accuracy for estimation of actions and action effects. For modelling wave evolution in time and space, more accurate spreading functions should be used, see i.e. ISO 19901-1. No directional spreading formulations can be used for shallow water. Refraction analysis is recommended.

When calculating the change in the soil's resistance during cyclic actions, the wave conditions shall be described by means of the variation of the sea state over an extensive period of time. The build-up as well as the tail-off phase of the storm shall be taken into account. Unless more accurate data are available, the storm development indicated in Figure 3 may be used.

[1] Both versions of the double-peaked spectrum as described in Torsethaugen (2004) and Torsethaugen (1996) may be used, but Torsethaugen (1996) contains errors and the formulations in DNV-RP-C205 is recommended to be used.

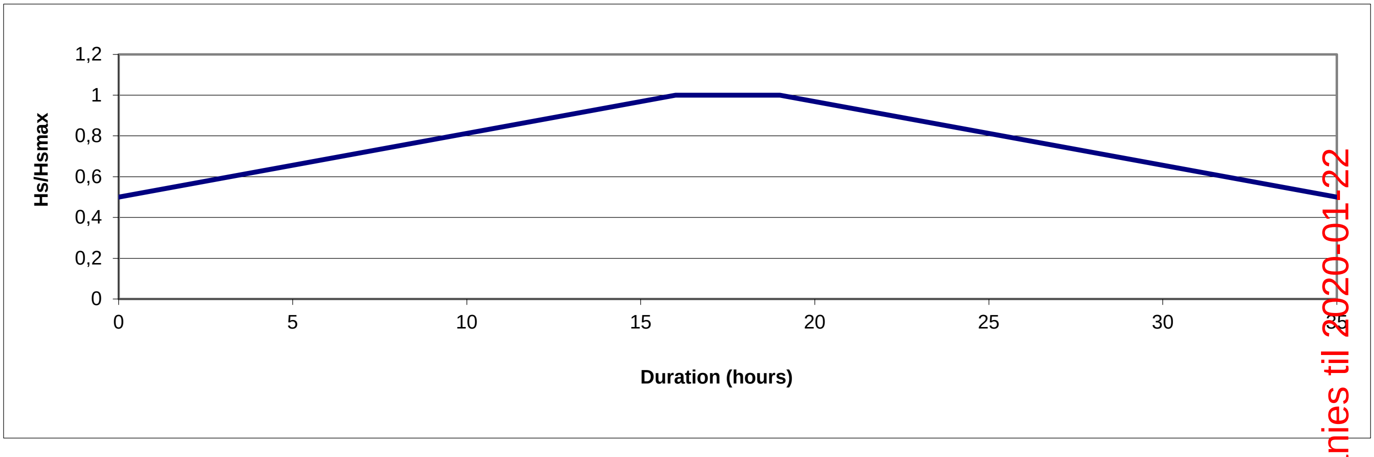


Figure 3 – Storm development for evaluation of the degradation of soil's resistance during cyclic action. The peak level is obtained for a sea-state with duration of 3 h.

Selection of surface process and associated wave theory should be made in view of underlying problem in agreement with guidance in 6.2.1.2.

Regarding statistical modelling of waves in a short-term sea state, the following probabilistic models are recommended:

– Crest height: Forristall crest height distribution. This is a Weibull model parameterized in terms of the Ursell number and the mean sea state steepness, i.e. in agreement with a second-order surface process. Expressions for parameters are available both for long-crested and short-crested sea, see e.g. DNV-RP-C205.
– Wave height: Næss or Forristall wave height distribution, see DNV-RP-C205.
– Wave period: The conditional distribution of wave period given the wave height can be modelled by a Gaussian distribution. Both mean and standard deviation depends on the wave height. For wave height higher than the significant wave height of the sea state, the mean period can be approximated by $0.9\,T_P$, where T_P is the spectral peak period of the sea state. Standard deviation for the period of high waves is indicated in DNV-RP-C205.

6.2.1.6 Long-term modelling of wave conditions

All sea state approach

A short-term wave condition is characterized by the significant wave height, spectral peak period and mean direction of wave propagation. For an all sea states approach regarding long-term analysis of actions and action effects, the joint distribution of these variables are required. Usually the mean direction of propagation is pooled into a limited number of wave sectors. The long-term distribution of mean direction of wave propagation, θ, is modelled by a probability mass function, $p_\Theta(\theta)$, estimated directly from data. For each sector, the long-term variablility of wave conditions are described by the joint probability density function of significant wave height, H_S, and spectral peak period, T_P:

$$f_{H_S T_p|\Theta}(h,t|\,\theta) = f_{H_S}(h|\theta) f_{T_p|H_S\Theta}(t|h,\theta) \quad (3)$$

The following distribution functions are recommended:

– H_S: A 3-parameter Weibull distribution. The parameters should be estimated using method of moments. If the frequency of very low sea states ($H_s \leq 1{,}5$ m) is of main concern, a log –normal distribution is expected to give better fit. However, the latter should not be used for H_s > 5 m.
– T_P given H_S: A log-normal distribution is generally found to give a good fit with a possible exception for ⟦AC⟩ ⟨AC⟧ low sea states.

For more details on joint distributions of sea state characteristics, reference is made to DNV-RP-C205.

Omni-directional modelling is done in a similar way by pooling data from the various direction sectors.

As the distribution function of significant wave height is available, the q-probability value for H_S is given by the value corresponding to an exceedance probability of $\frac{q}{n_{3h}}$, where n_{3h} is the expected number of 3-hour events per year for the sector under consideration. For an omni-directional analysis, $n_{3h} = 2920$.

Storm event approach – peak over threshold (POT) method

If a long-term analysis is to be based on the storm event approach, see Annex A for details, the wave input is time histories of the wave characteristics for all storms exceeding the threshold. Data representing several decades should be available before the storm event approach is utilized for final design.

For modelling of long-term variability of storm peaks, a 3-parameter Weibull model is recommended for storm maximum significant wave height. An alternative model can be the generalized Pareto distribution, but one should be careful if model suggests a too low upper bound for H_S. The conditional distribution of the associated spectral peak period can be modelled by a log-normal distribution.

As the distribution function for storm maximum significant wave height is available, the q-probability extreme value is estimated by the value exceeded by the probability q/n_s, where n_s is the expected number of storms per year for the sector under consideration. (For an ideal case with data for many decades and with no outliers, the extreme value obtained for the storm event approach is slightly lower than the value obtained for the all sea state approach.)

6.2.1.7 Long-term distribution of wave height or wave crest height

The long-term distribution of wave height and wave crest height can be found using methods in 6.1.3 and in A.3. Recommendations regarding short-term distributions are given in 6.2.1.5.

6.2.2 Current conditions

6.2.2.1 General

Availability of representative current measurements is required for final design.

The dominating current phenomena on the Norwegian continental shelf that should be included are:

- wind driven current;
- tidal current;
- Norwegian Coastal Current;
- North Atlantic Current;
- inflow of polar water;
- eddies in the Norwegian coastal current from Lista to Stadt;
- eddies in the North Atlantic Current;
- possible internal waves of the shelf ridge.

Hindcast predictions of wind-induced current, as well as theoretical models and other information about the tidal and coastal current may be used for early assessments. If the current velocity is of significant importance to the design and existing current data are scarce and uncertain, current velocity measurements should be carried out at the location in question.

The current velocity at the location of a facility shall be established on the basis of available measurements at the actual and/or adjacent locations. Hindcast current data may be an alternative if it is available. However, this requires that a thorough validation is made of the hindcast current data prior to using it for final design of a current sensitive problem. Current measurements will also be associated with uncertainties. For a current sensitive problem, the uncertainties of the measured current shall be quantified.

The minimum duration of measurements shall be two years, unless hindcast current for calibration exists, or nearby measurements are available. In such cases the minimum duration should be one year.

If simultaneous occurrence of current and e.g. wave data is to be utilized, a longer duration of current data is required. The data period should at least cover 3 years and the values used to determine characteristic actions should be based on a conservative estimate considering the variability from year to year.

6.2.2.2 Current measurements and design currents

In most cases a joint occurrence consideration is out of reach. Current conditions at a given location is therefore in most cases characterized by a profile obtained by predicting current speed corresponding to a given annual exceedance probability for each level with a sufficient amount of current measurements. The number of depth levels should be sufficient to capture current phenomena versus depth. As a minimum 5–8 levels seems reasonable depending on water depth (3 in upper 50 m, 1 at seabed and the rest equally distributed in between).

Prior to establishing current statistics for the purpose of estimating extremes, the deterministic tidal current should be removed from the measurements if it represents a significant part of observed current. The remaining current, the residual part, should be treated statistically. Current extremes at a given depth may be found by fitting a 3-parameter Weibull model to the residual current. The predicted extreme residual current speed should be added to the tidal current in order to represent design speeds.

If current is combined with wind and waves according to Table 7, and wind and waves are dominating the action process, current measurements should be smoothed such that the measurements represents 1-hour average values. q-probability events of current can then be taken as the q-probability 1-hour current which is assumed to last for the full 3-hour design weather condition. If current is the governing process, (i.e. ULS case 2 or ALS case 2 in Table 7), the q-probability current should be taken as the worst 10-minute current during $1/q$ years.

If a simultaneous description of wind, waves and current is used for a long-term response prediction performing analysis for all 3-hour weather events or selected storm events (POT approach), the default choice should be to use 1-hour current speed assumed to be valid for a 3-hour period. For such an application there is no reason to remove tidal components. If current show large fluctuations around the 1-hour mean and current is considered very important, one should find ways to account for these fluctuations.

A conservative q-probability profile for design is obtained by using linear interpolation between q-probability values for depths with data. Profiles may be omni-directional or established for the various direction sectors.

The profile described above is in most cases conservative due to lack of full correlation between current speeds at different depth levels. If a sufficient amount of data is available Turkstra Models, Winterstein et al. (2009) or EOF profiles, Kleiven (2002), Forristall and Cooper (1997) may be considered.

For early phase design assessments or non-critical marine operations in areas where no accurate measured data or documented numerical studies are available, some guidance is given in A.8.

When calculating erosion, it shall be taken into consideration that the structure may change the local current velocity.

6.2.2.3 Effect on current of adjacent structures

The current velocity in the vicinity of a large volume platform (within half the diameter or main dimension of the object) will be increased, and shall be considered.

The current velocity in the vicinity of a space frame or lattice type structure may be reduced from the specified «free stream» value by blockage. The blockage factor for steady current can be estimated as:

$$\left[1 + \sum_i \frac{C_{di} D_i}{4 D_p}\right]^{-1} \tag{4}$$

but not less than 0,7. C_{di} is the drag coefficient, D_i the diameter of member i and D_p the width of the cluster of members.

In absence of a detailed evaluation, a blockage factor of 0,9 and 0,85 can be used for jackets with 3 legs and more than three legs, respectively, for static behaviour. Further details are given in ISO 19902. For structures with significant change in geometry with depth, the blockage factor should be estimated for different depth levels.

6.2.3 Modelling simultaneous occurrence of metocean conditions

Unless the simultaneous wind, waves and currents are accounted for by joint probability occurrence model, Table 7 should be used.

An important condition for establishing reliable joint probabilistic models is a sufficient amount of simultaneous data. Using high quality and verified hindcast databases, this is fulfilled for wind and waves.

If current shall be included in joint modelling, simultaneous data should be used. The data should be site specific or documented to be representative for the area in question. Furthermore, the dataset should have sufficient duration in order to ensure that the long-term variability in storminess, seasonality and inter-annual variability are encountered for. At least 3 years of simultaneous data (in accordance with 6.2.2.1) are recommended to evaluate any reduction in the current speeds relative to Table 7. This is acceptable for indicating the effect of including simultaneous current history instead of target extreme profile.

Some guidance on how to include more than 3 parameters in the joint modelling is given in A.9.

6.3 Hydrodynamic actions

6.3.1 General

When numerical predictions are subjected to significant uncertainties, theoretical calculations shall be supported by model tests or observations of existing structures or by a combination of such tests and observations. Guidance regarding model testing can be found in 11.1.8.

Full-scale measurements may be used to update the action and action effects predictions of the relevant structure. Such measurements may be applied to reduce uncertainties associated with actions and action effects, which are difficult to simulate in model scale. However, reliable full-scale measurements require sufficient instrumentation and accurate logging of all important metocean processes and action effects, see 11.1.9.

Proof tests of the structure may be necessary to confirm assumptions made in the design. Hence, inclining tests of buoyant structures should be carried out to demonstrate the location of the centre of gravity.

6.3.2 Hydrodynamic actions on fixed structures

6.3.2.1 General

This sub-clause focuses on the action on fixed platforms. In an action-effect analysis of a fixed platform, the platform may behave dynamically and inertia and damping forces are mobilised. In such a case, relative velocity of structure and fluid particles may be considered, but in general, the absolute fluid particle speed should be used for calculating actions on fixed structures.

6.3.2.2 Wave and current effect

Waves and current should be considered when calculating hydrodynamic actions. In combination with waves, the current velocity profile should be stretched to the local water surface.

The hydrodynamic action on structural members with a ratio between wavelength (λ) and characteristic dimension (D) of a member $\lambda/D > 5$ should be determined by Morison's formula using a particle velocity obtained by vector addition of wave and current induced particle velocities. The local normal component of the resulting velocity vector towards the member shall be used in assessing the actions.

For large volume structures, with $\lambda/D < 5$, wave diffraction theory should be considered when deriving resultant actions. Wave-current interaction effects should be considered by detailed analysis for e.g.:

- slowly varying drift forces;
- air gap;
- higher order actions;
- slamming actions.

6.3.2.3 Tubular structural elements

Hydrodynamic actions on unshielded cylinders should be calculated by Morison's formula for framed structures consisting of slender tubular members with diameter to wavelength ratio less than 1/5. Force coefficients C_M(inertia) and C_D (drag) in Morison's formula depend on surface roughness, Reynolds number $Re = uD/\nu$ and Keulegan-Carpenter number $KC = u_{\max}T/D$ where:

- $u_{\max}$ is the maximum horizontal particle velocity at the depth of the tubular member considered.
- T is the wave period.
- $\nu = \mu/\rho$ is the kinematic viscosity, μ is the dynamic viscosity and ρ is the density of water.
- D is the diameter of the cylinder.

For global action analysis of jacket type of structures in ULS and ALS, $u_{\max}$ shall be taken as the maximum horizontal particle velocity at mean sea level under the wave crest determined by Stokes 5th order theory (Heideman and Weaver, 1992). It is assumed that this approach is valid for both regular and irregular waves.

For structures with small motions (e.g. conventional jackets), actions may be determined by design waves. Design wave actions shall be calculated from Stokes 5th order or Stream function wave kinematics, in accordance with 6.2.1.2 and 6.2.1.3. Design waves should be chosen based on maximum crest height and corresponding mean period. The design wave characterized by wave height and period is found by iteration of the wave height to meet the required design crest. An alternative method is to redefine the water level to align top of crest with the required design crest. Wave kinematics factor on the wave particle velocity should not be taken less than 0,95 for North Sea conditions. The kinematics factor is introduced in the regular wave approach to account for the directional spreading of the waves and irregularity in real sea states.

Generally, when applying Morison's formula the relative velocity between the structure and the water particles should be used. However, when the mean motion amplitude is smaller than the diameter, the relative motion between the structure and the water particles shall be neglected as it may overestimate damping. For further details, see DNV-RP-C205.

The action effects for bottom-fixed structures consisting of slender members exposed to significant wave induced dynamics should be assessed by means of time-domain simulations. It is recommended that the surface elevation process is modelled as a second order process with its corresponding second order kinematics with no kinematic reduction factor.

The external action vector $\boldsymbol{F}$ is obtained by integrating the following action per unit length over each member of the structure according to the Morison's formula:

$$d\boldsymbol{F}_N = C_D \frac{1}{2}\rho D|\boldsymbol{U}_{RN}|\boldsymbol{U}_{RN} + C_M \rho \, \pi \, \frac{D^2}{4}\dot{\boldsymbol{U}}_N \tag{5}$$

and decomposing the action in its Cartesian components. $\boldsymbol{U}_{RN}$ and $\dot{\boldsymbol{U}}_N$ are the relative velocity, and acceleration normal to the member, respectively.

It should be noted that the Morison's formula ignores lift and slamming. Further, local actions close to intersection with free surface and close to member ends and member joints are not well represented by Morison's formula.

For flow conditions characterised by large Keulegan-Carpenter number (KC > 60 for C_D and KC > 20 for C_M), the following force coefficients should be used for tubular members:

C_D = 0,65 and C_M = 1,6 for smooth members;

C_D = 1,05 and C_M = 1,2 for rough members.

Force coefficients for non-circular cross sections are given in DNV-RP-C205. Drag coefficients for cross-sections with sharp corners can be taken as independent of roughness.

Extreme wave (ULS / ALS) conditions will normally imply large KC values. The surface of the member is considered smooth for AC $k/D < 10^{-4}$ AC and rough for $k/D > 10^{-2}$ where k [m] is the surface roughness, see e.g. DNV-RP-C205 Section 6.7.1.3. For intermediate roughness, linear (in $\log k/D$) interpolation can be used.

For low to moderate KC numbers C_D and C_M shall be modified by the wake effect. For flow conditions with $KC < 60$ the drag coefficient C_D is multiplied by a wake amplification factor $\psi(KC)$. The mass coefficient C_M for low to moderate KC numbers is shown in Figure 4. For more details, reference is made to DNV-RP-C205 Section 6.7.

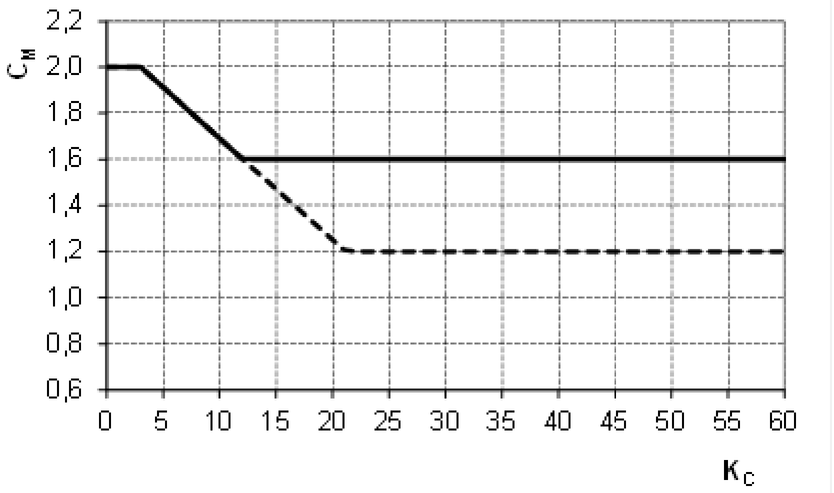


Figure 4 – Mass coefficient C_M as function of KC number for smooth (solid line) and rough (dotted line) tubular members

In fatigue analyses (low KC-number) a mass coefficient of $C_M = 2{,}0$ shall be used for tubular members. C_D depends on the sea state level, as parameterized by KC, and should be chosen in accordance with DNV-RP-C205 Section 6.7. Fatigue analyses can normally be conducted without considering current. The wave kinematics factor and conductor-shielding factor should be taken to be 1,0. For fatigue assessment a Gaussian sea surface would normally be sufficiently accurate.

Members shall be considered smooth at the installation stage. During operation members 2 m above MSL shall be considered smooth, see 6.11.1.

Wave actions on conductors/risers may contribute to the global actions on structures. If conductors/risers are closely spaced the actions on them may be modified as compared to actions on individual components, due to hydrodynamic shielding. Guidance on the shielding factor for drag actions may be found in ISO 19902. For closely spaced cylinders, spacing less than one diameter, an increase in the added mass (and inertia actions) should be accounted for. This effect may increase with amplitude of oscillation.

Shielding reduction factors should not be applied without documentation.

6.3.2.4 Large volume structures

Wave actions on fixed large volume structures should be calculated on the basis of wave diffraction theory using boundary-element (BEM) or Computational Fluid Dynamics (CFD).

The results from BEM should be carefully checked for surface-piercing bodies to ensure that irregular frequencies are avoided, see DNV-RP-C205.

Convergence of the results from BEM and CFD should be checked and documented.

The effect of small gaps, e.g. between two vessels (gap/draft<0,5) or vessels and sea bottom (gap/width<0,5), shall be assessed carefully.

Calculations of wave actions for novel structural shapes should be checked by model tests. This includes structures with considerable flare at water line.

6.3.3 Hydrodynamic actions on floating structures

6.3.3.1 General

Actions on structures with significant motions involve excitation actions as well as inertia, damping and restoring actions as formulated by the dynamic equation of motion.

6.3.3.2 Slender structural members

The formulation of actions in terms of relative velocity is uncertain for facilities with small motion. The method may in such cases give too high damping. Hydrodynamic damping should therefore be taken into consideration through an equivalent viscous damping, and the particle velocity should be used instead of relative motion. Guidance on applicability of relative velocity formulation is given in DNV-RP-C205.

When using the relative velocity formulation for structures with large motion in a combined wave and current condition, actions should be obtained for zero as well as maximum current velocity. Depending on whether the drag action is primarily damping or excitation action, low (for damping) and high (for excitation), value of AC⟩ C_D ⟨AC should be applied for a conservative estimate of the action. For damping a maximum C_D of 0,4 is recommended.

6.3.3.3 Large volume structures

For large volume structures with significant motion, a radiation wave system generated by the moving structure should be added to the incident and scattered wave system.

The radiation wave system should be determined by diffraction analysis (see 6.3.3.5) and gives rise to added mass and wave damping actions.

In addition, viscous actions on the hull, a possible mooring system or thrusters and risers should be considered.

Viscous actions due to wave- and low-frequency motions may be calculated separately by a relative velocity formulation and superimposed or calculated simultaneously, using appropriate phase for the actions, as well as hydrodynamic coefficients, as explained in 11.3.5.

6.3.3.4 Freeboard exceedance and green water

Green water is the overtopping by water of the part of a structure facing the seas in severe wave condition. For a ship, the phenomenon depends on bow shape, design free board, flare, breakwaters and other protective structures, as well as drainage arrangements, and weathervaning procedures. Green water may enter a ship in the bow as well as mid-ship and aft. Significant amounts of green water will have influence on the design of the vessel deck, accommodation superstructure, equipment and layout. It should be noted that for large amounts of green water, the ship motions will also be affected.

Freeboard exceedance and green water action effects shall be considered when applicable, see further requirements in 11.6.4.

6.3.3.5 Wave enhancement

The wave enhancement and modification of the kinematics caused by the structure may be estimated by linear radiation and diffraction theory. The fact that the wave crest is higher than given by linear theory shall be accounted for.

Model tests should be performed if the theoretical methods cannot predict the effect of substructure with sufficient confidence due to nonlinearities related to steep waves, and wave-current interaction. This situation may arise in connection with caissons of gravity structures and shallow pontoons of floating platforms, strong interaction between large columns, non-vertical sides near the water plane, and other features.

Wave motion in moon pool or centre well of ship and caisson vessels (spar buoys) should be particularly accounted for.

6.3.3.6 Sloshing actions in tanks

Motion induced dynamic pressure effects in partially filled tanks, especially with large horizontal dimensions, should be considered. Sloshing represents a dynamic magnification of internal pressure effects beyond the static pressure. Sloshing occurs if the natural periods of the fluid in tanks and of the vessel motions are close to each other. Possible impact effects should be accounted for in this connection.

Sloshing effects depend upon tank dimensions and filling levels, structural arrangement inside the tank and vessel motion characteristics.

For design action effects due to sloshing, see NORSOK N-004, L.4.5.2.

6.3.4 Hybrid structures

Wave diffraction solutions do not include viscous actions. When body members are relatively slender or have sharp edges, viscous effects may be important and viscous effects should be added to the diffraction forces determined.

Wave actions on structures composed of large volume parts and slender members may be computed by a combination of wave diffraction theory and Morison's formula. Parts of the structure may be modelled both by boundary elements to represent the potential hydrodynamic actions and beams to represent the viscous drag actions. The modifications of velocities and accelerations as well as surface elevation due to the large volume parts should however be accounted for when using Morison's formula.

If properly calibrated or validated, simplified conservative methods may be used to calculate actions.

6.3.5 Hydrodynamic action for all type of structures

6.3.5.1 Effect of adjacent structure

If a structure is located adjacent to another structure and one or both of them are large volume structures, there will be an interaction between the actions on the two structures.

A diffraction analysis may then be required for the total problem. Model tests should be considered if the analysis involves uncertainties, e.g. associated with extreme actions situations.

6.3.5.2 Nonlinear wave actions

Higher order potential flow wave diffraction methods as applicable for large volume structures and nonlinear finite amplitude wave kinematics used in conjunction with Morison's formula on slender structures, cause mean, sum-frequency and difference frequency actions in irregular waves. Such actions may cause significant response, and hence action effects, if resonance occurs.

The higher order actions should be determined by a consistent theory relevant for the structure considered and validated by model tests.

Difference (low) frequency wave actions associated with nonlinear wave-body interaction may excite lightly damped resonant modes of motion with natural frequency below the wave frequency range. This may be important for the rigid body motions of floaters and their positioning systems. Both horizontal and vertical modes of motion may be excited. Such motions are also referred to as slowly varying drift motions. Special attention should be paid to wave-current interaction effects and viscous effects on the low frequency wave actions. Due to the uncertainty associated with calculations of slowly varying drift motions model tests are required to reduce the uncertainty when the response is significant. Special care should be taken as the damping may be over-estimated in model tests due to scale effects. Dynamic calculations should be done for both AC⟩ ⟨AC model scale and full scale in order to estimate the scale effect.

Sum (high) frequency wave actions may be important for the response of GBS structures and jack-ups in their elastic modes with natural frequencies above the wave frequency range, for tension-leg platforms in their restrained modes and for ship shaped structures in their elastic modes. Sum frequency actions may cause springing response, which may be of importance especially for the fatigue limit states.

Actions from steep, high waves on structures extending above the still water level may cause nonlinear transient actions. Structural responses to these actions may be dynamically amplified and cause substantially increased response. Such response is also referred to as ringing. Nonlinear transient actions may be important for structures consisting of large diameter shafts and having a natural period under 8 sec. Actions causing ringing can be estimated by nonlinear potential theory or Computational Fluid Dynamics. One method that predicts ringing actions reasonably well, is presented by Johannessen

(2011). However, available analysis methods are generally only amenable to screening analysis of the ringing problem. The phenomenon is best quantified by model tests. Actions from wave impact/slamming from steep or breaking waves on GBS structures or waves impacting jacket topside may cause dynamically amplified resonant response. It is then generally difficult to distinguish impact/slamming from higher order wave diffraction effects.

With regard to structures that have been optimised with respect to minimal linear wave actions, experience has shown that nonlinear effects may be dominant. Motion analyses for new types of structures where the results cannot be checked against previous experience, should be checked against model tests.

6.3.5.3 Wave slamming

Slamming of horizontal or near horizontal surfaces that enter the water or are hit by a wave should be assessed. The corresponding action should be investigated with due consideration to rate of change of fluid momentum, viscous actions, and buoyancy actions. The effect of elasticity of the structure should be considered. Action effects from wave slamming are covered in 11.6.

Members in splash zone

Horizontal members in the splash zone are susceptible to actions caused by wave slamming when the member is being submerged. For slender members the slamming action per unit length may be estimated by $F = \frac{1}{2}\rho D C_{sl} u^2$, where u is the relative velocity between the water surface and the member normal to the member surface. C_{sl} is an appropriate slamming coefficient. For smooth cylindrical members the coefficient shall not be taken less than 3,0. For a flat plate a coefficient of 6,0 may be used to estimate the average pressure over the plate. An even larger coefficient is applicable for more local pressures. When the slamming force is determined by velocity squared, due consideration of a consistent choice of coefficient and area of contact should be made. The method established by Ridley (1982) can be used as a conservative to estimate splash-zone wave actions on inclined slender tubular members. DNV-RP-C-205 may be used for more detailed analysis.

Appropriate consideration shall be given to the possibility of slamming events occurring to underwater structural arrangements as a result of lack of depth of submersion.

Wave in deck

Wave actions on decks consisting of plated and tubular structural components comprise inertia, slamming, drag and buoyancy actions. Both the water entry and exit phase should be duly modelled. For further details on global wave-in-deck analysis, see 11.6.3.

Slamming on ship-shaped facilities

Slamming on a ship may occur in the forward bottom, bow flare, accommodation structure in the fore ship as well as exposed parts of the aft structure and turret. Slamming effects depend upon the following features of the vessel:

– draught;
– hull geometry;
– superstructure arrangement;
– vessel speed;
– heading.

Slamming actions may contribute to local effects as well as enhancement of hull girder bending moment and shear force. Vibrations in the hull may be induced by bow flare, bottom or stern slamming. This transient action effect is called whipping and may reduce the fatigue life of the vessel, and cause discomfort for the crew.

In lack of more accurate information bottom and bow flare slamming may be determined according NORSOK N-004, (L.4.5.4.)

Run-up effect

The possible effect of the up-welling (run-up) caused by a large, steep wave passing a large diameter vertical cylinder or columns with flat areas should be considered in designing the lower deck structure.

Local run-up (vertical jet flows) can cause local impacts at the column deck intersections. Model tests or advanced nonlinear numerical tools are recommended for estimating the run-up flows and impact forces.

While the principal effect of slamming and up-welling is local, possible global effects should also be assessed.

Slamming caused by breaking waves

Breaking waves occur in deep water as well as in shallow water. Spilling breakers are most common, but plunging breakers may also occur. The horizontal particle velocity under such waves may exceed the phase velocity by at least 20 %, see DNV-RP-C205, 8.8.1.2.

Vertical surfaces, especially on large-volume structures, can be exposed to actions from breaking or near breaking waves. The structure shall be checked against these actions during the design phase both with respect to ULS and ALS.

Numerical predictions of these actions will be associated with large uncertainties. The reason for the large uncertainties of these actions are uncertainties in water particle speed at impact, the area exposed to the impact, the slamming coefficient and the time history of the impact event. Hydro-elasticity may be of importance for some structures. Guidance regarding estimation of impact actions can be found in DNV-RP-C205.

Methods used for numerical prediction of slamming actions should be validated, e.g. by high quality model tests.

6.4 Wind actions

6.4.1 General

Wind acts on the topsides and that portion of the structure that is above the water, as well as upon any equipment, deckhouses, bridges, flare-booms, and derricks that are located on the topsides. As the wind speed varies with elevation, the height of the component shall be taken into account. For description of wind conditions, see 6.4.2.

Wind actions will depend on how the structure respond to the wind.

- Wind actions on structures or structural components that are not dynamically sensitive to wind may be calculated by considering the wind action as static, see 6.4.5.
- Wind actions on structures that are dynamically sensitive to wind shall be calculated by considering the wind as a dynamic action, see 6.4.6. Examples of such structures are structures with a natural frequency less than 5 Hz (e.g. high towers, flare booms, tension-leg structures, compliant and catenary anchored structures).

Regarding wind action effects, including VIV, see 6.4.7 and 11.6.5.

6.4.2 Wind conditions

The wind velocity at the location of the facility shall be established on the basis of available hindcast data or measurements at the actual and/or adjacent locations. If the wind velocity is of significant importance to the design and existing wind data are scarce and uncertain, wind velocity measurements should be carried out at the location in question.

High quality and verified hindcast data may be a useful source of wind data for design and operations providing mean wind (speed and direction) for every 3-hour at various heights for the whole Norwegian Continental shelf. The database NORA10 covers from 1958 until present. Extreme wind speed contours and annual average wind speed contours at the Norwegian Continental Shelf are given in AC A.10 AC.

For high wind speed there are some concerns that NORA10 underestimate the actual mean wind conditions. Therefore, NORA10 output wind speed in Figure A.7 above 15 m/s is slightly increased according to the following formula:

$$U_{Cor} = U + 0,2 \cdot (U - 15) \tag{6}$$

where U is original output from NORA10. Similar correction may be relevant for other hindcast databases.

Characteristic values of the wind velocity should be determined with due account of the inherent uncertainties.

6.4.3 Description of wind conditions

For a short-term condition, the wind may be described by an average wind velocity and a superimposed fluctuating wind gust with a mean value equal to zero, as well as a mean direction.

Unless a more detailed assessment is made, the average wind velocity at 10 m above sea level with an annual probability of exceedance of 10^{-2} can be chosen as U_0 = 36 m/s (1-hour average) for the whole continental shelf. The characteristic value with an annual probability of exceedance of 10^{-4} can be chosen U_0 = 41 m/s (1-hour average).

The characteristic wind velocity $u(z,t)$ [m/s] at a height z [m] above sea level and corresponding averaging time period t less than or equal to $t_0 = 3600$ s may be calculated as

$$u(z,t) = U(z)(1 - 0{,}41\, I_u\,(z)\ \ln(t/t_0)) \tag{7}$$

where the 1-hour mean wind speed U(z) [m/s] is given by:

$$U(z) = U_0\left[1 + C\ \ln\left(\frac{z}{z_0}\right)\right] \tag{8}$$

where z_0=10 m, $C = 5{,}73 \cdot 10^{-2}\,(1 + 0{,}15\,U_0)^{0{,}5}$, U_0 [m/s] is the 1-hour mean wind speed at 10 m and the turbulence intensity factor $I_u(z)$ is given by:

$$I_u(z) = 0{,}06[1 + 0{,}043U_0]\left(\frac{z}{z_0}\right)^{-0{,}22} \tag{9}$$

For moderate and strong wind speeds and neutral conditions, turbulent wind shall in general be described by the Frøya wind spectrum, see Andersen and Løvseth (2006):

$$S(f) = 320\frac{\left(\frac{U_0}{10}\right)^2\left(\frac{z}{z_0}\right)^{0{,}45}}{(1+\tilde{f})^{\frac{5}{3n}}} \tag{10}$$

where $n = 0{,}468$

$$\tilde{f} = 172f\left(\frac{z}{z_0}\right)^{2/3}\left(\frac{U_0}{10}\right)^{-0{,}75} \tag{11}$$

where

$S(f)$ [AC] [m²/s] [AC] is the spectral density at frequency f [Hz]

z [m] is the height above sea level

U_0 [m/s] is the 1-hour mean wind speed at 10 m above sea level.

The wind profile description in Equations (7)–(9) and the spectral description in Equations (10) and (11) are valid both for moderate and strong (extreme) wind speed conditions. However, for moderate conditions (U_0 < 15 – 20 m/s) and non-neutral stability conditions both the wind profile and the wind spectrum may deviate significantly from the above neutral descriptions. For the non-neutral wind profile reference is made to Plate (1982) and for the wind spectrum to Andersen and Løvseth (2006).

The squared correlation between the spectral densities, see Equation (10), of the longitudinal wind speed fluctuations of frequency [AC] f [AC] between two points is described in terms of the two-point coherence spectrum, [AC] $\mathrm{Coh}(f)$. [AC]

The recommended coherence spectrum between two points $P_1(x_1, y_1, z_1)$ and $P_2(x_2, y_2, z_2)$ with:

x_1 and x_2 along-wind positions

y_1 and y_2 across-wind positions

z_1 and z_2 elevations

are given by:

$$\mathrm{Coh}(f) = \exp\left[-\frac{1}{U_0}\left[\sum_{i=1}^{3}(A_i^2)\right]^{1/2}\right] \tag{12}$$

where

$$A_i = \alpha_i \cdot f^{r_i} \cdot \Delta_i^{q_i} \cdot \left(\frac{z_g}{z_r}\right)^{-p_i} \tag{13}$$

$$z_g = (z_1 \cdot z_2)^{1/2} \tag{14}$$

where the coefficients α_i, p_i, q_i, r_i and the separations Δ_i i are given in Table 2.

Table 2 – Coefficients and separation for the 3-D (i = 1,2,3) coherence spectrum. Separations are given by absolute values.

i	Δ_i	q_i	p_i	r_i	α_i
1	$\lvert x_2 - x_1\rvert$	1,00	0,4	0,92	2,9
2	$\lvert y_2 - y_1\rvert$	1,00	0,4	0,92	45,0
3	$\lvert z_2 - z_1\rvert$	1,25	0,5	0,85	13,0

Equations (12) – (14) require that [AC⟩ f ⟨AC] is in Hertz [Hz], [AC⟩ A_i ⟨AC] in metres per second [m/s] and Δ_i in metres [m].

6.4.4 Extreme wind speed

Extreme wind speed can be estimated fitting a 3-parameter Weibull model to the 1-hour hindcast data from 10 m above sea level. The all short-term condition approach or a peak-over-threshold approach (see 6.1.3.3) can be used for predicting $\boldsymbol{q}$-probability values for the 1-hour mean wind speed. Sensitivity to choice of method shall be assessed.

Extremes for various heights (i.e. profiles) and averaging time can be calculated using Equations (7) – (9).

6.4.5 Mean wind actions

For structures and structural parts where the maximum dimension is less than 50 m, 3 s wind gusts shall be used when calculating static wind actions.

For structures and structural parts where the maximum length is greater than 50 m, the averaging period for the mean wind may be increased to 15 s.

When design actions due to wind need to be combined with extreme actions due to waves and current [AC⟩ and hydrodynamic actions are dominating, ⟨AC] the mean wind speed averaged over a 1 min period may be used. A longer averaging period may be used if properly documented.

The mean wind action [AC⟩ F ⟨AC] on a structural member or surface, acting normal to the member axes or surface, shall be calculated by [AC⟩ equation 15 ⟨AC] (see Figure 5):

$$F = \frac{1}{2}\,\rho C_D A_m\, U_m^2\, \sin\alpha$$ AC (15) AC

where

ρ	is the mass density of air
C_D	is the drag coefficient[2]
A_m	is the area of the member or surface area normal to the direction of the wind
U_m	is the mean wind speed
α	is the angle between the direction of the wind and the axis of the exposed member or surface

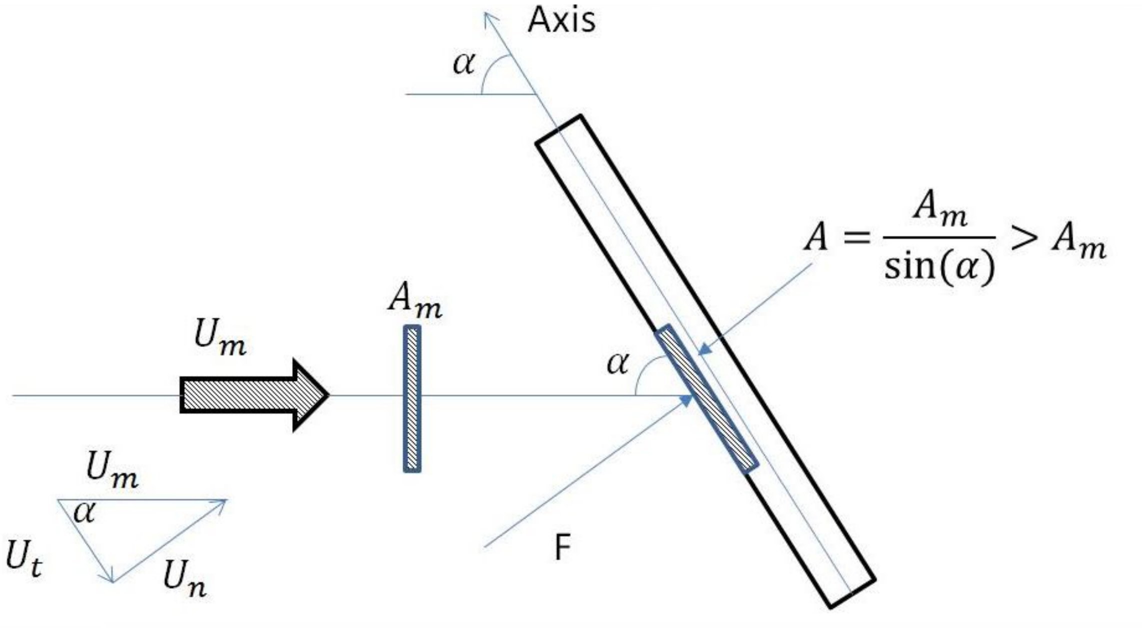


Figure 5 – Definition of axes, wind direction and area normal to the direction of the force

The drag coefficient depends on the flow conditions round the structure. The flow is dependent on the turbulence of the incoming flow as well as the surface character of the structure. For smooth circular tubular structures the drag coefficient can be taken as C_D = 0,65 for high Reynolds number flow conditions (Re > 500 000). The effect of Reynolds number and the effect of roughness on C_D are given in DNV-RP-C205.

In the case of tubular structures covered with ice, C_D = 1,2 shall be used for all Reynolds numbers.

Further information about drag coefficients, C_D may be found in DNV-RP-C205 and NS-EN-1991-1-4.

A simplified method for determining wind forces on ships is given by OCIMF (1994).

Global wind action on topsides and connecting bridges can be conservatively estimated by a simplified geometry using the silhouette box geometry and corresponding drag coefficient.

Wind pressures and resultant actions shall be determined from wind tunnel tests on representative model when wind actions are crucial. Wind actions on structures calculated by CFD shall be calibrated towards wind tunnel tests.

When calculating wind actions, increase in dimensions, weight and change in the shape and surface roughness due to icing of the structure shall be taken into account, see 6.6 and 6.7.

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[2] Drag coefficients for wind action are often referred to as shape coefficients C_S.

6.4.6 Fluctuating actions

Structures that are sensitive to wind gusts shall be calculated by considering the wind action as a dynamic action in accordance with DNV-RP-C205.

The total wind speed is the sum of a mean velocity (based on a 1-hour period) and a fluctuating component $u(t)$. The wind force on a structure with structural response velocities which are negligible compared to the wind velocity range can be calculated by:

$$F = \frac{1}{2}\rho C_D A[U_m + u(t)]^2 \approx \frac{1}{2}\rho C_D A\,[U_m^2 + 2U_m u(t)] \tag{16}$$

This means that the fluctuating wind force is linear in the fluctuating velocity.

Low-frequency wind forces are normally determined from the Frøya wind energy spectrum; see Equations (10) and (11). The Harris wind spectrum may be considered when action effects in structures such as flare towers, which are sensitive to the high frequency excitation are to be calculated, see DNV-RP-C205.

For structures of large size, the spatial variation of the fluctuating wind could be incorporated in the analysis by accounting for the coherence, see 6.4.3 and DNV-RP-C205.

6.4.7 Action effects from wind

The effects of dynamic wind actions may be determined by a frequency domain analysis. An alternative is a modal formulation. The modal action effects may be combined with the SRSS method if the modes are not too close to each other. In case of modes having periods close to each other, the CQC method can be applied.

The extreme action effect due to wind can be determined by:

$$S = S_S + \alpha \cdot \sigma_S \tag{17}$$

where

S_S is the static response due to the mean wind

σ_S is the standard deviation of the dynamic structural action effect

α is wind response peak factor

The effect of dynamic wind actions may also be determined by a properly verified time-domain analysis.

6.5 Actions from snow

6.5.1 Action

Snow accretion shall be assessed in relation to actions on offshore structures and in relation to safety critical systems such as lifeboats, escape routes, ventilation, communication systems, etc.

Snow actions shall be determined based on local conditions (temperature; wind; structure, precipitation). [AC) Values corresponding to annual (AC] probabilities of exceedance of 10^{-2} and 10^{-4} shall be considered for ULS and ALS, respectively. In relation to safety critical systems, any impact on performance thereof shall be assessed. Further guidance related to actions from snow will be available in ISO 35106.

The joint effect of snow and icing shall be considered.

6.5.2 Mitigations and mitigation effects

The operator shall develop design principles and operation plan for safe management of situations with snow accumulations (e.g. avoiding unacceptable snow accumulations in critical areas of the facility, heating, mechanical snow removal, and/or production shutdown). The design principles may include design for normal operations even under extreme and abnormal snow accumulations. Operational measures (e.g. fixed heating arrangement, portable equipment) can be taken into account in order to reduce design actions from snow or adverse snow effects. Expected performance and reliability of operational measures shall be documented.

6.5.3 Estimation of snow actions

Actions from accumulated snow may be estimated by:

1) locally recorded snow data
2) recorded local precipitation together with information on air temperature (snow if T_a less than +1C).
3) precipitation from hindcast data together with information on air temperature (snow if Ta less than +1C). When hindcast data is used, such data shall be calibrated against measured data of good quality. For the Norwegian continental shelf, the hindcast database NORA10 (Reistad et al., 2011) is recommended.
4) Values in Table 3.

In northern parts of the Norwegian shelf such as in the Barents Sea, it is very unlikely that sufficiently long recordings of snow or precipitation will be available for accumulation assessments. Alternatives 3 or 4 above are therefore recommended.

Given that a time series of accumulated snow has been prepared, extreme estimates for snow accumulations (snow masses exceeded with 10^{-2} and 10^{-4} annual probabilities) can be estimated by the "Annual maximum approach". This is done by identifying from the time series max accumulated snow mass each winter, fit a probability distribution to the annual maximum values and extrapolate accumulations to 10^{-2} and 10^{-4} action levels. It is important that the duration of the time series is sufficiently long to provide the required amount of data. A duration of 10 years is considered a minimum while 30–40 year long data sets are recommended. Note that when such an approach is used, it is recommended to define a year as the period from July to June.

For coastal municipalities and 10 km out from the shoreline tabulated values for extreme snow actions as given in NS-EN 1991-1-3 and Norwegian National Annex may be applied.

If recorded precipitation is used as data basis, it shall be taken into account that the recorded amount of precipitation can be underestimated in snow events. If recorded time series from coastal stations such as Fruholmen, Bjørnøya and Hopen is used, the amount of precipitation shall be increased with a factor of 1,4 unless use of other correction factors can be justified (Førland, 1996).

If reduction of snow actions due to melting or wind drift can be quantified and documented, accumulated amount of snow to be used for design will decrease. High snow rates at high latitudes (North of 70N) are correlated with severe (but not extreme) winds. Despite that winds will contribute to reduction in global snow accumulation, winds may still locally cause increased snow accumulations.

In general, it is recommended that structure specific calculations using e.g. principles in Computational Fluid Dynamics (CFD) are applied when considering local snow accumulations. The shape factors given in NS-EN 1991-1-3 and Norwegian National Annex shall be used unless more detailed analysis is carried out.

AC Additional guidance on snow accretion and density is given in A.11. AC

Table 3 – Snow actions on NCS estimated from selected grid points of the NORA-10 database and the estimation method 3) above

Zone	Location	Annual probability of exceedance 0,63 [kPa]	Annual probability of exceedance 10^{-2} [kPa]	Annual probability of exceedance 10^{-4} [kPa]
Zone 1	NCS south of 66º latitude	0,15 [1)]	0,35 [1)]	0,60 [1)]
Zone 2	NCS between 66º and 70º latitude	0,20 [1)]	0,50 [1)]	0,85 [1)]
Zone 3	NCS north of 70º latitude	0,35 [2)]	0,80[2)]	1,25 [2)]

1) Based on the assumption that snow is completely removed each second day. For unmanned facilities, higher values should be used.

2) Based on accumulation of snow over 1 week.

6.6 Actions from sea spray icing

6.6.1 Actions

Sea spray icing shall be assessed in relation to actions on structures and in relation to safety critical systems such as lifeboats, escape routes, ventilation, communication systems, etc.

AC〉 Icing values corresponding to 〈AC annual probabilities of exceedance of 10^{-2} and 10^{-4} shall be considered for ULS and ALS, respectively. In relation to safety critical systems, any impact on performance thereof shall be assessed. General information and further requirements related to actions from sea spray icing will be given in ISO 35106.

When calculating wave, current and wind actions, account shall be taken of increases in dimensions and changes in the shape and surface roughness of the structure as a result of accumulated ice from sea spray which covers the whole circumference of the element.

6.6.2 Mitigations and mitigation effects

Accumulations of ice from sea spray icing may be reduced by operational measures such as heating, mechanical removal, use of “anti-icing coatings” or operational procedures (e.g. vaning to reduce amount of sea spray). If ULS and/or ALS actions from sea spray icing are reduced due to operational measures, the operator shall document the performance thereof.

6.6.3 Marine ice accumulation

Estimates of sea spray icing shall take all relevant parameters and processes into account. To estimate icing accumulations corresponding to 10^{-2} and 10^{-4} annual exceedance probability level, state of the art numerical models shall be applied. Such models should incorporate the main processes and include the effects of parameters listed in AC〉 A.12. 〈AC

When calculating wave, current and wind actions, account shall be taken of increases in dimensions and changes in the shape and surface roughness of the structure as a result of accumulated ice from sea spray which covers the whole circumference of the element.

Uneven distribution of ice shall be considered, primarily for buoyancy stabilized structures, by assuming that the ice accretion is distributed in the most unfavourably way.

6.7 Actions from atmospheric icing

For classifications of atmospheric icing, see AC〉 A.13. 〈AC

6.7.1 Actions

Atmospheric icing shall be assessed in relation to actions on structures and in relation to safety critical systems such as lifeboats, escape routes, ventilation, communication systems, etc.

When calculating wind actions, increase in dimensions, weight and change in the shape and surface roughness of the structure shall be taken into account. It shall be assumed that the accumulated icing from the precipitation occurs for all surfaces facing upwards or against the wind. For tubular structures it may be assumed that ice covers half the circumference. VIV and other aero-elastic effects (such as galloping) should be considered.

6.7.2 Mitigations and mitigation effects

A method of avoiding or reducing damage from falling ice is the use of shielding structures. In case of accumulated ice, the potential for falling ice shall be taken into account by providing protection for personnel as well as for equipment (where relevant).

6.7.3 Atmospheric icing accumulation

Actions from atmospheric icing should be determined based on local conditions (temperature in sea and air, wind, sea state, precipitation, humidity, size/shape/location of structural elements).

Information and guidance related including atmospheric icing in design is provided in ISO 12494. There are, however, differences related to atmospheric icing offshore and on land structures, which should be taken into consideration.

In the absence of a more detailed assessment, a nominal ice thickness of 10 mm caused by precipitation icing can be considered as a value for 10^{-2} exceedance probability south of 70° north. This thickness can be assumed constant from a height of 5 m above sea level to the top of the facilities. The frequency and severity of atmospheric icing is expected to increase at higher latitudes. North of 70° north on the Norwegian Continental Shelf a nominal value for thickness of the accumulated icing caused by precipitation may be selected as 20 mm. If the nominal values are applied, an ice density of 900 kg/m^3 shall be used. If extreme ice thickness is based on a detailed assessment, 500 kg/m^3 shall be used for wet snow icing while 900 kg/m^3 shall be used for freezing rain.

All types of atmospheric icing shall be considered in relation to SLS criteria and winterization issues (e.g. falling ice).

6.8 Sea ice actions

6.8.1 General

Details on sea ice characteristics and definitions are found in ISO 19906, and WMO's Ice Nomenclature (WMO no. 259), and will be available in ISO 35106.

AC〉 Additional guidance on sea ice actions is found in A.14. 〈AC

6.8.2 Actions

Sea ice is a relevant source for actions on structures in regions such as Skagerak, in the northern and western parts of the Norwegian Sea and in the Barents Sea.

Annual probability of ice intrusions in the region of operation shall be assessed. In lack of site specific assessments, the occurrence of sea ice with annual probability of exceedance of 10^{-2} and 10^{-4} in the Barents Sea shown in Figure 6 should be applied. The approach applied to establish the extreme sea ice extent contours is presented in Eik et al. (2013).

Structures and operations in regions where sea ice presence is more frequent than AC〉 10^{-4} 〈AC shall take the effects of sea ice into account. Operators should be aware of that the main source for information on ice coverage, ice charts, often defines the transition from "ice covered waters" to "open water" at 10 % ice concentration (WMO no. 259 [94]). The practical implication of this is that there potentially can be presence of individual ice floes also in the waters south of the estimated sea ice extent. Operators should take this into account when planning operations close to ice covered waters.

North of 74°N the possibility of multi-year ice actions shall be considered.

The design of offshore structures shall include considerations of global actions related to the overall integrity of the structure, foundation and stationkeeping system and local actions for specific components or portions of the structure. ISO 19906 shall be applied for the design of structures against sea ice actions unless more accurate data and methods can be documented.

Particular emphasis shall be given to identify all possible structural failure modes, which can be caused by ice-structure interactions.

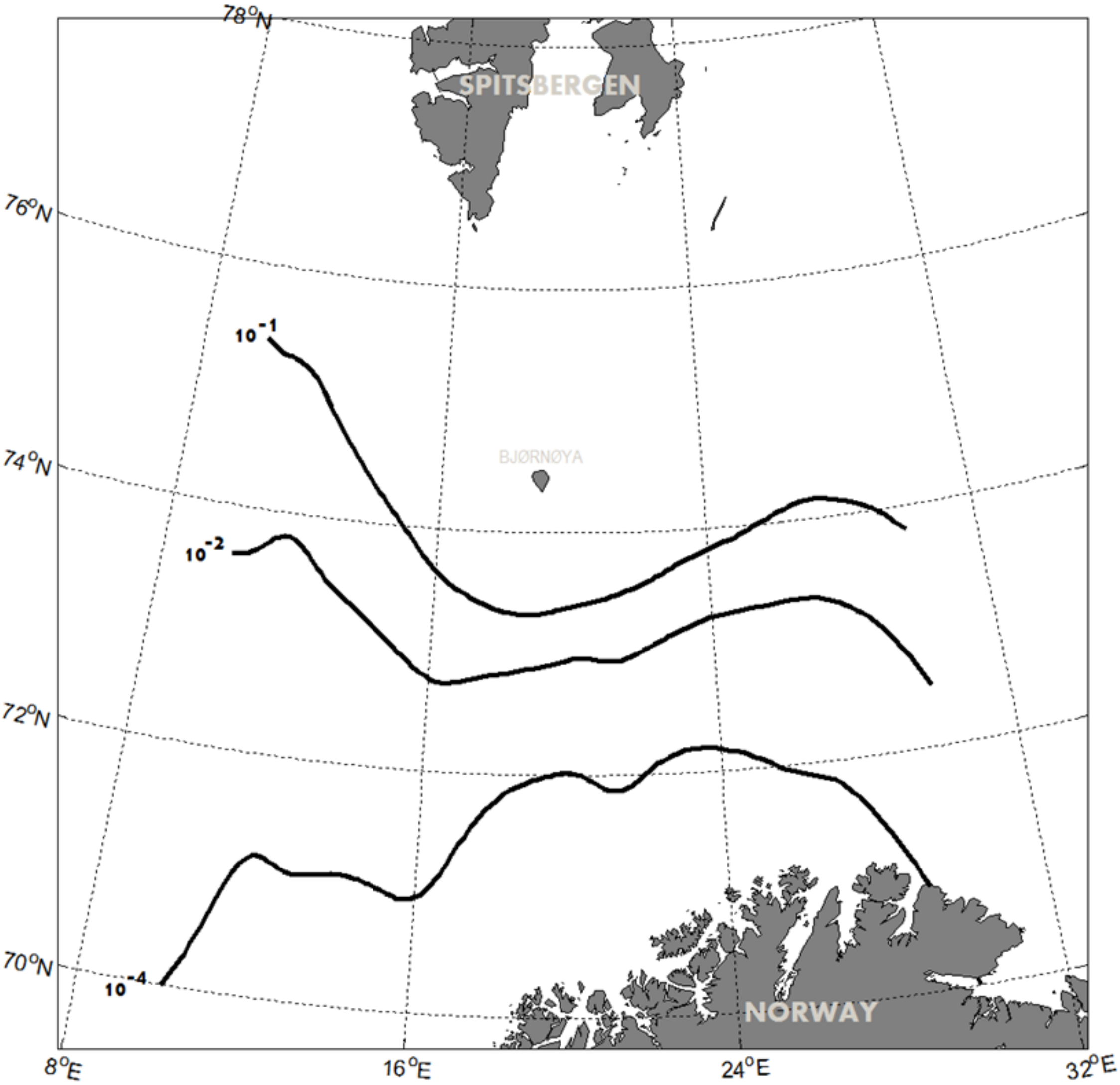

Figure 6 – Limits of sea ice extent in the north-eastern part of the Norwegian Sea and the western Barents Sea with annual probability of exceedance of 10^{-1}, 10^{-2} and 10^{-4} that may be used in lack of more detailed analysis. Note that values given only apply to Norwegian Continental Shelf

NOTE Data source and method for establishing sea ice extent

Sea-ice charts for the period 1967 to 2012 have been analysed. Charts for the period 1967-2002 are gathered from IAPCO (2003). Charts for the period January 2003 – October 2012 have been received from Istjenesten (Ice Service) at met.no, Tromsø.

The files (shapefiles) have been gridded and converted from GIS data format to ascii text files prior to analysis. The resulting gridded files have a 0,14° x 0,14° resolution.

Method

1. All sea-ice with concentration exceeding 10 % is counted as ice. Any sea-ice with less concentration is not detected and hence not classified in the ice charts.
2. The southernmost position of sea-ice in each year, for each longitude between 10°E to 30 °E, with a 0,14° step, is registered as the annual maximum extension. The annual maximum extension may consist of longitudinal maxima taken from different charts (times) within each year. This means in practice that the annual maximum ice extent may not have been observed as an ice border at any time, but is a constructed annual maximum.
3. A matrix consisting of the annual maximum extent given as the latitude (°N) for the period 1967-2012 is then created.

4. The annual maxima are redefined as distance to 90°N, so that higher values indicate larger extent.
5. A Gumbel distribution is then fitted to these values for each longitude between 10°E to 30 °E, with a 0,14° step. For each longitude the values with annual probability of exceedance of AC› 10^{-1}, 10^{-2} and 10^{-4} ‹AC is registered.
6. The extreme values are the defined back to latitude (°N).
7. Prior to plotting, the extreme extents are smoothed using a running. The resulting extents have approximate 30 km resolution.

6.8.3 Mitigation and mitigation effects

All operations planned in regions with probability of sea ice more frequent than 10^{-4} per year shall establish an ice management system. The objective of such a system is in general to reduce the risks related to sea ice. For ice resistant operations and structures, the objective of ice management may be limited to ice surveillance, forecasting and threat evaluation. Use of disconnectable structures may be considered as a part of an ice management system. Definition and implementation of ice management in design, ISO 19906. Further guidance will be available ISO 35104.

6.8.4 Sea ice actions and action effects

To evaluate action effects of sea ice on structures (accounting for ice-structure interaction and dynamic effects), state-of-the-art numerical models, ice tank model tests and relevant full-scale field experience can be applied. Further recommendations found in ISO 19906 shall be used unless more accurate data and methods can be documented.

6.9 Iceberg actions

6.9.1 General

The term “Iceberg” is in this document used for all types of glacial ice including bergy bits and growlers. Reference is made to [94] WMO no. 259 for definitions of iceberg types.

Presence of icebergs cannot be neglected in any part of the Barents Sea and the shelf of Jan Mayen. The probability of impact between structure and icebergs shall be estimated as a part of the design process and prior to operations in the Barents Sea. The impact probability shall take into account:

– iceberg aerial density;
– size of structure;
– average iceberg drift velocity in region;
– average iceberg size in region.

Icebergs may impact both surface piercing structures, subsea equipment and scour the sea bed. Proper considerations of all iceberg threats shall be done. Smaller pieces of ice may be thrown onto the deck of facilities. Such scenarios shall be evaluated when relevant. Repetitive iceberg impacts may take place and low cycle fatigue should be considered.

The average density of icebergs is generally in the range 850 kg/m^3 to 910 kg/m^3.

6.9.2 Actions

If the annual probability of iceberg impact is higher than 10^{-4}, action and action effects from icebergs shall be taken into account in the design of offshore structures. Both local and global actions from icebergs shall be considered. Guidance on how to estimate iceberg actions can be found in ISO 19906.

ISO 19906 provides details on how to estimate annual iceberg impact probability[3] . If site specific iceberg impact probability is not calculated, the 10^{-2} and 10^{-4} zones shown in Figure 7 shall be applied.

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[3] ISO uses «encounter frequency» as a basis for impact probability.

For structures with annual probability of iceberg impacts in the range 10^{-4} to 10^{-5}, operational mitigations such as physical iceberg management and/or disconnection should still be considered in accordance with the ALARP principle and actions from possible iceberg impact scenarios should be evaluated.

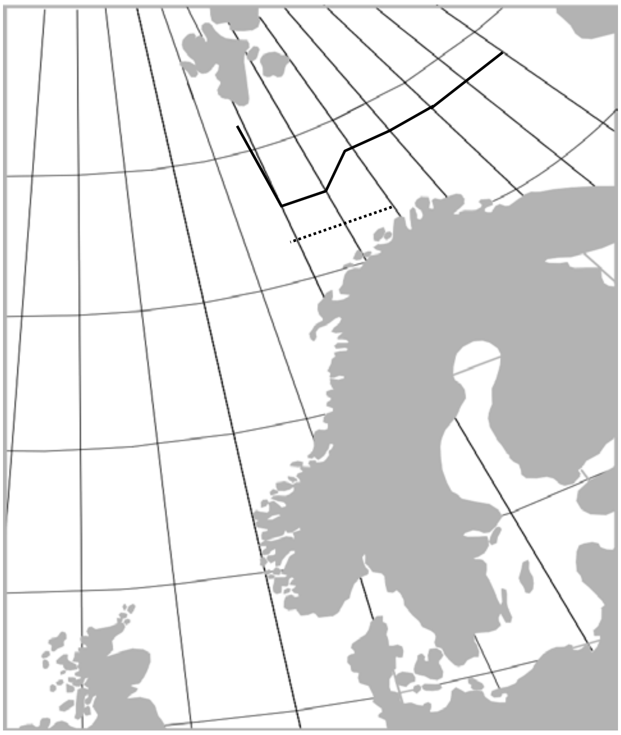

Figure 7 – Limit for collision with icebergs with a probability of exceedance of 10^{-2} (solid line) and 10^{-4} (dotted line). Note that values given only apply to Norwegian Continental Shelf

6.9.3 Mitigations and mitigation effects

All operations planned in regions with probability of iceberg encounters more frequent than 10^{-4} per year shall establish an iceberg management system. The objective of such a system is in general to reduce the risks related to icebergs. For iceberg resistant structures, the objective of iceberg management may be limited to ice surveillance, forecasting and threat evaluation. Use of disconnectable structures may be considered as a part of an ice management system. For definition and implementation of iceberg management in design, reference is made to ISO 19906. Information and further requirements to iceberg management will be available in ISO 35104.

6.9.4 Iceberg actions and action effects

A probabilistic approach should be applied in order to identify actions corresponding to 10^{-2} and 10^{-4} exceedance probability levels. Alternatively, in early phase, a deterministic method, in which extreme or abnormal (e.g. mass or kinetic energy) and nominal values (e.g. compression strength) of iceberg are combined to yield ULS and ALS actions, may be applied. Further requirements and guidance is found in ISO 19906 and in Jordaan et al. (2014).

When estimating actions from icebergs, it shall be taken into account that larger icebergs are more likely to impact a structure than smaller ones and those moving faster are more likely to encounter the structure than the slower. Reason for this is that they will swipe larger areas and thus statistically be more likely to impact. This can be taken into account by updating parent distributions prior to response analysis.

6.10 Miscellaneous

6.10.1 General

A simplified classification of the Barents Sea licence area of the Norwegian continental shelf, with metocean parameters that may be used in lack of more detailed data and evaluations, is given in AC A.15. AC

6.10.2 Wind chill temperature

The wind chill temperature is a measure of the degree of cooling of the human body during exposure to a wind-temperature environment, and shall be used in accordance with NORSOK S-002. Guidance on wind chill temperature for different areas of the Norwegian Continental Shelf is given in Annex B.

6.10.3 Frost burst

The effects of freezing of ballast water, firewater, hydraulic fluids, fuels and other fluids in contained areas shall be taken into account and appropriate precautions shall be provided if necessary.

6.11 Other issues related to metocean actions

6.11.1 Marine growth and biofouling

Marine growth is a common designation for a surface coat on marine structures, caused by plants, animals and colonies/clusters of animals (such as corals, hydroids, bivalves, molluscs, seaweed, anemones, etc.). Marine growth may cause increased hydrodynamic actions, increased weight, increased hydrodynamic additional mass and may influence hydrodynamic instability as a result of vortex shedding and possible corrosion effects.

In the calculation of structural actions, unless more accurate data are available, marine growth as indicated in Table 4 may be assumed. Table 4 shall be used for static and dynamic analysis of structures consisting of slender members to account for increased mass, buoyancy, added mass and drag. If more severe marine growth is experienced during operation, consequences shall be assessed. For more details about marine growth in the northern North Sea and the Norwegian Sea (59º to 72º N), reference is made to Det Norske Veritas (2012).

Hydrodynamic diameter to be used for prediction of added mass and drag actions on tubular members shall be taken as $D + 2t$ where D is the diameter without marine growth. Mass of marine growth to be added to structural mass and added mass in a dynamic analysis is given by the density ρ_{mg} and volume of marine growth. The additional weight of marine growth is given by the submerged weight (dry weight minus buoyancy) per unit volume $W_{mg} = (\rho_{mg} - \rho_w)g$, where ρ_w is sea water density and g is acceleration of gravity.

Table 4 – Thickness and density of marine growth and biofouling. The water depth refers to mean water level

Depth (m)	56º to 59º N		59º to 72º N	
	Thickness t (mm)	Density ρ_{mg} (kg/m³)	Thickness t (mm)	Density ρ_{mg} (kg/m³)
Above +2	0	-	0	-
-15 to +2	100	1300	60	1325
-30 to -15	100	1300	50	1325
-40 to -30	100	1300	40	1325
-60 to -40	50	1300	30	1100
-100 to -60	50	1300	20	1100
Below -100	50	1300	10*	1100*

*) Cold water corals can build up local colonies with no limitation regarding size in water depths between 100 and 800 m. Cold water corals are assumed not to occur for temperatures below 2 deg Celcius, i.e. for the Norwegian continental shelf it may be assumed that these occur in water depths between 100m and 450m. The density of the marine growth should be taken as 1300 kg/m3 in the whole water depth range where cold water corals can be found.

The thickness of marine growth may be assumed to increase linearly to the given values over a period of 2 years after the structure has been placed in the sea.

The thickness and mass of marine growth and biofouling, and its effect on increased mass, drag and added mass, shall be used in action and action effect analyses.

Unless more accurate data are available, the roughness height may be taken as 20 mm below + 2 m. The effect of surface roughness should be taken into consideration when determining the mass and drag coefficients in Morison’s formula.

If marine growth exceeds the values for which the facility is documented, cleaning may be omitted if a new analysis shows that the structure has sufficient strength.

6.11.2 Water level, settlements, subsidence and erosion

When determining water level in the calculation of actions, the tidal water and storm surge shall be included as a maximum value or a minimum value dependent on which is the worst combination for the action and action effect in question. Tidal water and storm surge may be determined in accordance with Table 7. For airgap calculations, see 11.6.3.

Uncertainty of measurements, subsidence in the reservoir or settlement of the structure and possible erosion shall be considered. Calculation methods that take into account the effects that the structure and adjacent structures have on the water level shall be used.

The possibility of, and the consequences of, subsidence of the seabed as a result of changes in the subsoil and in the production reservoir during the service life of the facility, shall be considered. Reservoir settlements and subsequent subsidence of the seabed should be calculated as a conservatively estimated mean value.

The tidal data may be taken from Gjevik et al. (1990).

6.11.3 Appurtenances and equipment

Hydrodynamic actions on appurtenances (e.g. anodes, fenders) shall be taken into account, when relevant. For further details, see ISO 19902.

7 Earthquake actions

7.1 Basis for seismic assessment

Earthquake actions should be determined on the basis of the relevant tectonic conditions, and the historical seismological data. Measured time histories of earthquakes in the relevant area, or other areas with similar tectonic conditions may be adopted.

Earthquake motions at the location may be described by means of response spectra or standardised time histories with the peak ground acceleration to characterise the maximum motion.

The earthquake motion can be described by two orthogonally horizontal oscillatory motions and one vertical motion acting simultaneously. These motion components are assumed to be statistically independent. One of the horizontal excitations should be parallel to a main structural axis, with the major component directed to obtain the maximum value for the action effect considered. Unless more accurate calculations are performed, the orthogonal horizontal component may be set equal to 2/3 of the major component and the vertical component equal to 2/3 of the major component, referred to bedrock. The earthquake may be scaled with the given factors.

In absence of more detailed, site specific assessments, the peak ground acceleration at annual exceedance probabilities of 10^{-2} and 10^{-4} given in seismic zonation maps in NFR/NORSAR (1998) can be applied.

When determining earthquake actions on the structure, interaction between the soil, the structure and the surrounding water should be taken into consideration.

When time histories are used, the action effect should be calculated for at least three sets of time histories. The mean value of the maximum values of the calculated action effects from the time history analyses may be taken as basis for design. The time series shall be selected in such a way that they are representative of earthquakes on the Norwegian continental shelf at the given probability of exceedance.

7.2 Seismic design of structure and foundation

Earthquake design includes ULS (strength) check of components based on earthquakes with an annual probability of occurrence of 10^{-2} and appropriate action and material factors; as well as an ALS check of the overall structure to prevent its collapse during earthquakes with an annual probability of exceedance of 10^{-4} with appropriate action and material factors, given in NORSOK N-001.

Normally the ALS requirement will be governing, implying that earthquakes with an annual probability of exceedance of 10^{-2} can be disregarded.

The assessment of earthquake effects should be carried out with a refinement of analysis methodology that is consistent with the importance of such effects. The assessment of soil-structure integrity could therefore be undertaken in the following steps:

1) Determine appropriate peak ground acceleration given in seismic zonation maps, see i.e. NFR/NORSAR (1998).
2) Determine whether further checks will be necessary, by e.g. the procedure given in 7.3.

For most parts of most structures on the Norwegian Continental Shelf no further check will be necessary. If necessary, for more detailed checks, proceed as follows:

– strength check of soil-structure system under 10^{-2} and 10^{-4} events based on linear elastic action effect using modal superposition, see 11.7.2;
– if strength check is not fulfilled, a ductility check may be carried out, see 11.7.3;
– if the ductility check is not fulfilled, a more accurate analysis of the site specific seismic hazards than those given in 7.3 may be carried out and used to demonstrate compliance with the requirements;
– if the ductility check is still not fulfilled, modification of the design is necessary.

7.3 Response spectra for a single degree of freedom system

Unless more accurate assessments are performed to develop site specific seismic criteria, the response spectrum in Figure 8 may be used as basis for structural action effects for the 10^{-4} as well as the 10^{-2} annual probability earthquake. The figure shows the response spectrum related to bedrock outcrop motions normalised to 10 m/s^2 at 40 Hz. This spectrum may be used together with the peak accelerations given in seismic zonation maps (NFR/NORSAR, 1998). If for instance the peak acceleration is 2,5 m/s^2 for the annual probability of 10^{-4}, the response spectrum for bedrock outcrop motions is obtained by multiplying the normalised spectrum in Figure 8 with 0,25.

For soil deposits, the motions will be different than for bedrock outcrop. Thus scaling functions for the relation between motions in the soil deposits and the bedrock will have to be established. These scaling functions are dependent on the frequency, on the stiffness of the soil and on the magnitude of the motions. The latter is because the stiffness depends on the strain level in the soil and thus on the magnitude of motions. In general, a site response analyses should be performed to derive site-specific response spectra. If not, conservative scaling functions should be used. Such are given in NFR/NORSAR (1998) for the 10^{-4} and 10^{-2} conditions, but only for poorly defined “soft soil” and “stiff soil”. ISO 19901-2 suggests scaling factors for motions at 0,2 s and 1,0 s period for 4 different soil classes defined by soil characteristics and for different values of spectral accelerations of the bedrock spectrum at these periods. For very soft soil for which the referred scaling functions do not apply or for layered soil with large difference in characteristics between the layers, site response analyses should be performed as a rule.

The spectrum is based on the total structural and soil damping of 5 % of critical damping. The spectrum shall be adjusted if the damping differs from this value. The scaling factor for the spectrum due to variation of the (percentage) damping between 2 % and 10 %, may be selected and applied to the entire spectrum using the following formula

$$K = \ln(100/\zeta)/\ln(20) \qquad (18)$$

where ζ is the damping in percentage of critical damping.

Before detailed assessment of earthquake actions is undertaken, a conservative estimate of the global (i.e. base shear) force based on a single dynamic mode of action effect and the response spectrum (see Figure 8) may be used to judge the need for such an analysis.

It should be noted that the amplification factor given for soft soils for the 10^{-4} per year event may be conservative for periods greater than 4 s. On the other hand, a site-specific soil response study is recommended for very soft soils, since they may be outside the range of data used to establish Figure 9, see i.e. NFR/NORSAR (1998).

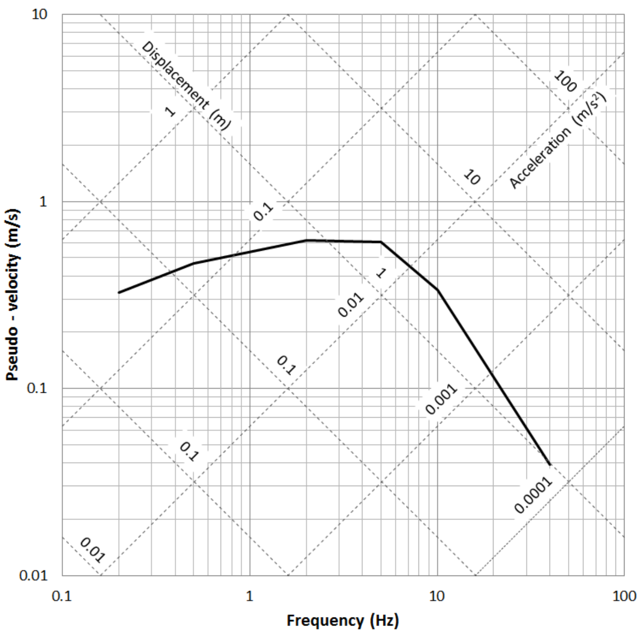

Figure 8 – Response spectrum for bedrock outcrop motions, normalised to 10 m/s² at 40 Hz (NFR/NORSAR, 1998)

NOTE Dot is used as decimal point in the figure.

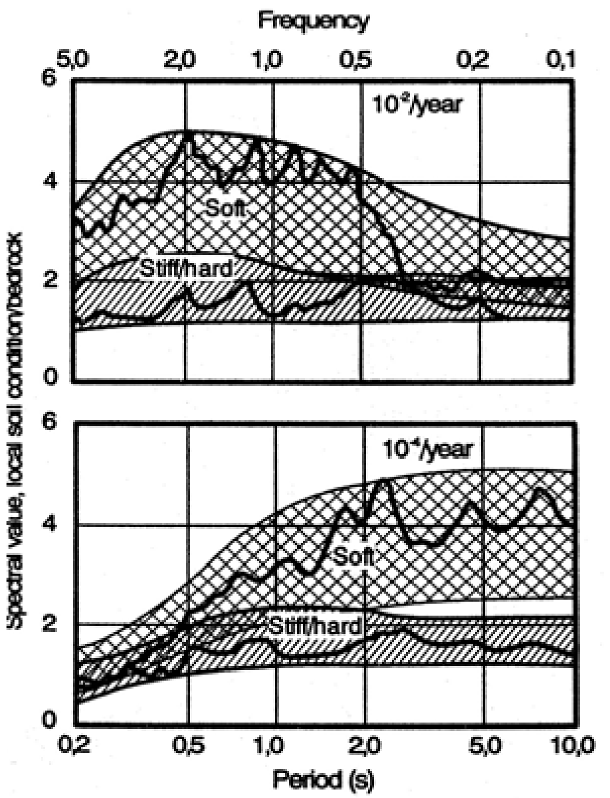

Figure 9 – Ratio between spectral value for soil and bedrock

8 Deformation actions

8.1 General

Deformation actions are actions caused by deformations imposed on the structure. They may be caused by the structure's function or the surrounding metocean conditions, or by the construction processes. Reference is made to NORSOK N-004 and ISO 19903 for relevance and method for application of deformation actions.

8.2 Temperature actions

Structures shall be designed for the most extreme temperature differences they may be exposed to. This for instance applies to

a) storage tanks,
b) structural parts that are exposed to radiation from the top of a flare boom. One hour mean wind with a return period of 1 year may be used to calculate the spatial flame extent and the air cooling in the assessment of heat radiation from the flare boom,
c) structural parts that are in contact with pipelines, risers or process equipment.

The ambient sea or air temperature is calculated as an extreme value with an annual probability of 10^{-2}.

Sea temperature also varies with depth. The local air temperature may be higher as a result of sun radiation. During fabrication of the structure, all dimensions should be related to a reference temperature. Eriksrød and Ådlandsvik (1997) give data for sea temperatures at the sea floor that can be used in an early design phase.

Unless more accurate measurements or calculations are carried out, air and sea temperatures given in AC A.16 AC may be used.

8.3 Actions due to fabrication

During design, reasonable tolerances shall be assumed, and account shall be taken for possible actions that may arise from forces introduced to compensate geometrical mismatch, shrinkage forces of concrete or welding. The effects of errors (e.g. geometric deviations or defects exceeding the tolerance limits) should be considered by the person responsible for the design.

8.4 Actions due to settlement of foundations

Effects of uneven settlements of the foundation shall be considered. Actions on the structure from risers and drill-string as a result of foundation settlements should be considered. Local reaction on the structure during installation due to uneven seabed or boulders shall be considered.

9 Accidental actions

9.1 General

As specified in NORSOK N-001, structures should be designed to be robust by e.g. satisfying ALS requirements and to prevent small damages to escalate into catastrophic failures. The damage should correspond to an annual probability of exceedance of 10^{-4} and can be due to

- accidental actions,
- specified damage conditions,

at the given probability or a lower probability e.g. due to ALARP considerations.

Accidental actions are actions caused by abnormal operation or technical failure. They include for instance fires and explosions, impacts from ships, dropped objects, helicopter crash, change of intended ballast distribution or pressure difference e.g. due to ballasting faults, and frost burst.

Accidental actions should be determined by

- dimensioning accidental actions estimated in risk assessment and relevant accumulated experiences, see NORSOK Z-013 and NORSOK S-001,
- minimum requirements in regulations and standards (PSA regulations and NORSOK N-001),
- necessary robustness with respect to the inherent uncertainty of such actions and future modifications,

with due account of the factors of influence. Such factors may be operational procedures, the arrangement of the facility, equipment, safety systems, control procedures and mitigation actions to reduce the probability of occurrence.

In the design phase particular attention should be given to layout and arrangement of the structure and equipment in order to minimise the adverse effects of accidental events.

The ALS design check should be carried out with a characteristic value for each accidental action, which corresponds to an annual exceedance probability of 10^{-4} for the installation.

If the accidental action with an annual exceedance probability of 10^{-2} per year included safety factors is more onerous than the ALS action, an ULS design check shall be carried out.

For combination of accidental actions, see 10.4.

If the accidental action is described by a single variable, the characteristic accidental action may be determined from an action exceedance diagram or hazard curve (action intensity versus probability of exceedance) for the structure.

The characteristic accidental action on different components of a given facility could be determined as follows:

- establish exceedance diagram for the action on each component;
- allocate a certain portion of the reference exceedance probability (minimum 10^{-4} for ALS case and 10^{-2} for ULS case) to each component;
- determine the characteristic action for each component from the relevant action exceedance diagram and reference probability.

Alternatively, the following, more refined consideration of risk may be used to determine the accidental action:

- component (i) is assumed to be designed for an accidental action with an exceedance probability of pi for that component;
- estimate the probability of total loss due to failure of component (i), implied by the residual risk associated with the accidental action;
- estimate the total probability of failure AC (p_f) AC (associated with the given accidental action on all components;
- compare p_f with the target level;
- re-allocate AC p_i AC's in order to get a more optimal design, while complying with the target level.

If the accidental action is described by several parameters (e.g. heat flux and duration for a fire, pressure peak and duration for an explosion) design values may be obtained from the joint probability distribution by contour curves, see 10.4. However, in view of the uncertainties associated with the probabilistic analysis, more pragmatic approach would normally suffice.

Since a large amount of scenarios associated with each type of accidental action can be envisaged, a focus on those that may have impact on design is necessary.

The characteristic (design) accidental actions should be summarised in the Design Accidental Load (DAL) report.

NOTE In NORSOK Z-013 and the Design Accidental Load specifications characteristic (design) accidental actions are called "dimensioning" accidental loads.

9.2 Fires and explosions

9.2.1 General

The principle fire and explosion events are associated with hydrocarbon leakage from flanges, valves, equipment seals, nozzles, etc.

The following types of fire scenarios shall for instance be considered:

a) burning blowouts in wellhead area;
b) fire related to releases from leaks in risers; manifolds, loading/unloading or process equipment, or storage tanks (including jet fire and fire ball scenarios);
c) burning oil on the sea;
d) fire in equipment or electrical units;
e) fire on helicopter deck;
f) fire in living quarters;
g) pool fires on deck or sea.

The following types of explosion scenarios shall for instance be considered:

a) ignited gas clouds;
b) explosions in enclosed spaces, including machinery spaces and other equipment rooms as well as oil/gas storage tanks.

Structural layout should be selected to limit the effect of fire and explosion, e.g. by using appropriately located and sized fire and blast walls.

For facilities that are winterized, special considerations for fire and explosion actions shall be taken into account. More details about such process accidents are found in NORSOK Z-013 and NORSOK S-001.

Actions due to fire and explosions may be determined by numerical models. The complexity of these phenomena requires the numerical models to be validated by experimental results and used with caution.

9.2.2 Fires

The fire action intensity may be described in terms of thermal flux as a function of time and space or, simply, a standardised temperature-time curve for different locations (substructure, derrick, flare tower, etc.). The fire action is normally given as a uniform load with the associated duration for each fire area. Alternatively, CFD fire simulations can be used to reflect a more realistic exposure of the structure to heat actions.

For objects engulfed by the flame the heat convected away can be neglected. For non-engulfed objects the incident convection is usually negligible.

The temperature in the structure may be calculated by finite difference method or analytical methods. The latter is limited to one-dimensional passage of the heat into the structure.

More detailed information about emissivity constants and thermal parameters may be found in NS-EN 1991-1-2 and associated specific standards for different materials.

Extinguishing fires may lead to use of large quantities of water. The consequences of the corresponding increase of weight and e.g. the effect on stability should be taken into account.

9.2.3 Explosions

The characteristic (design) accidental explosion action should be given as an average pressure (local and global) and duration for each firewall/deck and drag load and duration for each fire area.

Explosions can cause two types of action, namely overpressure on decks and walls and net pressure forces on equipment, which for small sized equipment reduces to drag.

In principle, the explosion actions on various components can be assessed by pressure obtained by CFD methods. However, it might be convenient to determine the actions on slender members by a drag type formula, by using a drag coefficient. Guidance is given e.g. in Chapter 4 of Czujko (2001).

Dynamic amplification factors for explosion response in offshore structures may be found in DNVGL-RP-C204 (to be published, 2016) and Czujko (2001).

Main firewalls in enclosed spaces should at least resist an explosion pressure of 70 kN/m^2 with a duration of 0,2 s and an overpressure rise time of 0,1 s, see guideline to PSA Facility Regulation Section 30.

The damage due to explosion should be determined with due account of the dynamic character of the action effects. Simple, conservative single degree of freedom models may be applied. When necessary, nonlinear time-domain analyses based on numerical methods like the FE method should be applied.

9.2.4 Combined fire and explosion effects

Accidental actions, which are combinations of fire and explosion events, should be considered with an annual probability of 10^{-4} for the joint event.

Fire and explosion events that result from the same scenario of released combustibles and ignition should be assumed to occur at the same time, i.e. to be fully dependent. The fire and blast analyses should be performed by taking into account the effects of one on the other.

The damage done to the fire protection by an explosion preceding the fire should be considered Håverstad (1989).

9.3 Impact actions

9.3.1 General

Impact actions are characterised by kinetic energy, impact geometry and the relationship between action and indentation. Impact actions may for instance be caused by:

a) vessels in service to and from the facility; including supply vessels;
b) tankers loading at the field;
c) ships and fishing vessels passing the facility;
d) floating units, such as flotels;
e) aircraft on service to and from the field;
f) falling or sliding objects;
g) fishing gear;
h) icebergs or ice,
i) anchors.

Structural layout should be selected so as to limit the effect of impacts. Particular attention should be paid to protecting critical component such as risers against impacts.

ALS design checks should be made with impact events corresponding to a minimum exceedance probability of 10^{-4} and possibly ULS check with impact events with an exceedance probability of 10^{-2}.

9.3.2 Vessel collisions

9.3.2.1 General

The collision energy can be determined on the basis of relevant masses, velocities and directions of ships that may collide with the facility. When considering the facility, all traffic in the relevant area should be mapped and possible future changes in vessel operational pattern should be accounted for. Design values for collisions are determined based on an overall evaluation of possible events, as described in Table 5.

Table 5 – ALS Design values for vessel collisions to be used unless further evaluations are performed

Vessel collision scenario	ALS Design values to be used unless further evaluations are performed
Passing ships	See NORSOK Z-013
Visiting supply and intervention vessels	50 MJ
Shuttle tanker collisions	100 MJ

The most probable impact locations and impact geometry should be established based on the dimensions and geometry of the structure and vessel. In particular, the bow with possible bulbs or ice strengthened bow should be considered. In connection with collisions with cantilevered (overhanging) platform decks the superstructure of the ships should be considered. Moreover, account should be made of tidal changes, operational sea-state and motions of the vessel and structure which has free modes of behaviour. Unless more detailed investigations are done for the relevant vessel and platform, the impact zone for supply vessels should be considered to between 10 m below LAT and 13 m above HAT.

Impact scenarios should be established representing bow, stern and side impacts on the structure as appropriate. If a central impact (impact action through the vessel’s centre of gravity) is physically possible, this impact situation should be analysed.

The possibility of a second, less intensive, impact on the damaged structure should be assessed. Possible change of floating position caused by the initial damage should then be assessed.

When the duration of the collision is short compared with the periods governing the motion and the rate of loading is relatively small, the damage caused in the collision in structures with free modes (see 11.1.2) may be determined in two steps, as described below.

1) First the distribution of impact energy between kinetic rotation and translation energy and deformation energy, can be determined by momentum and energy considerations.
2) Then local damage to vessel and facility can be determined so that the energy absorbed by the two structures corresponds to the energy that is to be absorbed as deformation energy. Due account of the energy absorption properties of the ship and facility should be made, by especially considering the effect of possible ice strengthening of the vessel and strengthening of the facility with respect to ship impacts, should be considered.

If the impact duration is long compared with the relevant local or global periods of structural vibration, structural analysis to determine the energy absorption and damage can be done by a quasi-static method of analysis. This analysis can be based on load-indentation curves obtained by laboratory tests and analysis, as outlined in NORSOK N-004 or DNVGL-RP-C204. Otherwise, a dynamic structural analysis should be carried out.

Load-deformation curves for bow, stern, stern corner and broad side impact of standard 6 500 – 10 000 t displacement vessels can be found in DNVGL-RP-C204. Particular considerations are necessary e.g. for impact by ice-strengthened bows.

ULS design is based on strength check with appropriate material and action factors.

9.3.2.2 Passing ships

For passing ship collisions, the impact energy in (head-on impacts) should be based on NORSOK Z-013.

9.3.2.3 Visiting supply and intervention vessels

Experience has shown that the design should take into account collisions by vessels intended for regular service inside the safety zone, see Moan et al. 2016.

The velocity of visiting vessels can be determined based on the assumption of a drifting ship, or on the assumption of erroneous operation of the ship.

If no operational restrictions on allowable visiting vessel size are implemented the displacement of supply ships should not be selected less than 10 000 tons (i.e. AC $1 \cdot 10^7$ AC kilogrammes) in risk assessment. The corresponding speed in head-on collisions shall be set to 0,5 m/s and 3,0 m/s for ULS and ALS design checks, respectively. In sideways and stern impacts, the speed should not be less than 0,5 m/s and 2 m/s for ULS and ALS design checks, respectively. A hydrodynamic (added) mass of 40 % for sideways and 10 % for bow and stern impact can be assumed.

If operational restrictions on vessel size or vessel velocity can be implemented, Figure 10 may be used to determine acceptable combinations of vessel displacement and velocity depending on the impact energy capacity of the structure. Unless a proven velocity limitation system is implemented (that a faulty DP system cannot override) a velocity of 3 m/s shall be assumed in head-on collision. It should be noted that type of vessel may influence the energy absorbtion between vessel and structure, and shall be included in any such operational restriction evaluations.

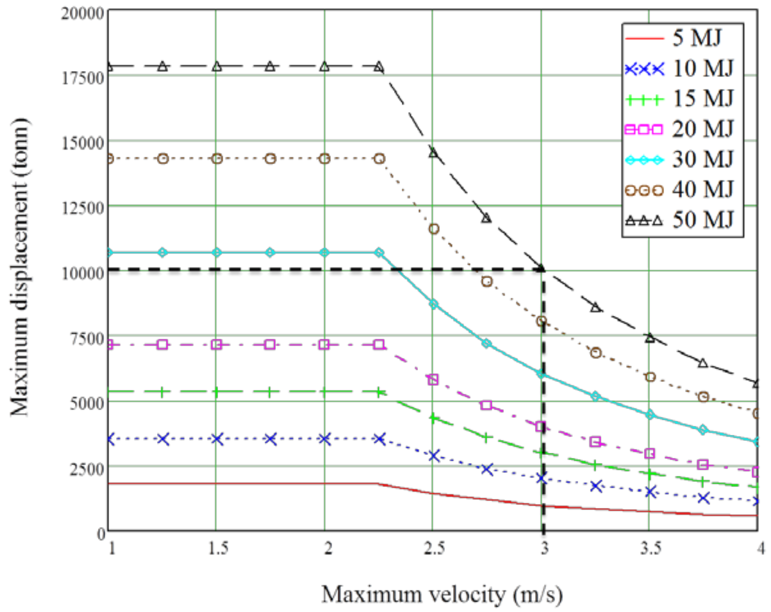

Figure 10 – Operational restrictions in the safety zone - acceptable combination of vessel size and velocity is found beneath the respective curve for the documented impact energy capacity (MJ). Based on head on collision and sideways collision in drift condition (Moan et al. 2016)

NOTE Dot is used as decimal point in the figure

Further information about collision analysis may be found in NORSOK N-004 and DNVGL-RP-C204.

9.3.2.4 Shuttle tanker collisions

For a tandem offloading a collision scenario involving the shuttle hitting the stern of the FPSO with a minimum collision energy of 100 MJ, should be considered. Collision scenarios for other offloading configurations should be determined by risk analysis.

9.3.2.5 Flotel and facility-facility collisions

If a flotel or other facility is positioned close to the facility being analysed, special evaluations shall be carried out as to whether these facilities may inflict collision actions on the other facility.

9.3.3 Dropped objects

Actions due to dropped objects should for instance include following types of incidents:

a) dropped cargo from lifting gear;
b) falling lifting gear;
c) unintentionally swinging objects;
d) loss of valves designed to prevent blow-out or loss of other drilling equipment.
e) falling ice from elevated structures e.g. drilling deck and flare towers

The impact energy from the lifting gear can be determined based on lifting capacity and lifting height, and on the expected weight distribution in the objects being lifted as described in NORSOK Z-013.

The dropped object actions for various deck area shall be based on the classification of the area, in accordance with NORSOK R-002 B.3.3, and as described in the installations Material Handling Philosophy.

The trajectories and velocity of objects dropped in water should be determined based on the initial velocity, impact angle with the water, effect of the impact when the falling object hit the water, possible current velocity and the hydrodynamic resistance. It is non-conservative for impacts in shallow water depths to neglect the effect of water impact. A typical trajectory for slender objects (pipes) is straight motion until it has reached the maximum horizontal excursion. Then it starts to fall, exhibiting horizontal oscillations. The possibility of a straight trajectory for pipes with a rotation about the longitudinal axis should be considered.

The impact effect of long objects such as pipes should be subject to special consideration.

Relevant impact situations can be determined by the operating area of the lifting equipment and the relevant lifting arrangement.

Similarly, it should be considered what damage possible fender systems would cause if they should fall down.

For dropped object actions on pipelines, see DNV-RP-C204. NORSOK U-001 provides values for accidental actions on subsea facilities. These values may be used in evaluations during an early design phase.

9.3.4 Helicopter impacts

A helicopter deck should be designed on the presumption that any point on the deck may be subjected to a single action equal to 75 % of the total weight of the heaviest helicopter used. This action should be regarded as an ordinary ULS action and combined with wind actions from wind with velocity of 30 m/s (1 min, 10 meters above water).

The single action can be assumed evenly distributed over an area of 0,09 m^2. Reference is made to FOR-2007-10-26-1181.

If specific information about the weight distribution of the undercarriage of a helicopter is available, this may be used.

The main structure below the helicopter deck should, as an ALS case, be designed for an action equal to 3 times the total weight of the heaviest helicopter used. The normal weight distribution of the helicopter on the undercarriage may be used. It should be assumed that the helicopter is placed in the most unfavourable position. Alternatively, a suitable impact energy may be used.

9.4 Effect of subsea gas blow-out

Blow-out from subsea wells or gas line rupture may cause a gas – water plume with significant horizontal fluid velocities at the sea surface. The gas –water plume will cause a free surface elevation on top of the plume ("the boiling region").

The actions exerted on the floater by such fluid velocities and the corresponding mooring line tension and vessel heeling shall be considered, if found relevant from design accidental load analysis. Also the horizontal motion and possible contact with neighbouring facilities shall be considered. Measures shall be taken to avoid water filling and loss of stability due to this effect Rew et al. (1995).

9.5 Loss of heading control

For moored units normally operated with heading control, either by weathervaning or thruster assistance, the effect of loss of the heading control shall be evaluated in view of ALS requirements. If ALS criteria cannot be satisfied, other risk reduction actions to keep the likelihood of mooring system failure should be introduced.

The risk associated with loss of heading control due to thruster failure may be reduced by redundant thruster system or passive weathervaning. Alternatively, additional actions on the mooring system due to loss of heading control shall be included.

9.6 Abnormal variable actions

Changes in intended pressure differences or buoyancy caused for instance by defects in or wrong use of separation walls, valves, pumps or pipes connecting separate departments as well as safety equipment to control or monitor tank pressure, shall be considered.

Unintended distribution of ballast due to operational or technical faults should also be considered.

9.7 Failure of DP and stationkeeping

If the annual failure rate of stationkeeping components (e.g. mooring lines, thrusters) exceeds 10^{-4} such events shall be considered in the ALS check. If ALS criteria cannot be satisfied, other risk reduction actions shall be introduced.

10 Action combinations

10.1 General

Table 6 shows the actions to be combined in different limit states.

Table 6 – Characteristic actions and action combinations

	TEMPORARY CONDITIONS *1)					NORMAL OPERATIONS				
	SLS	FLS	ULS	ALS Abnormal effect	ALS Damaged conditions	SLS	FLS	ULS	ALS Abnormal effect	ALS Damaged conditions
Permanent actions	EXPECTED VALUE (weight and buoyancy)					EXPECTED VALUE *5)				
Variable functional actions	SPECIFIED VALUE	SPECIFIED ACTION HISTORY	SPECIFIED VALUE			SPECIFIED VALUE*5)				
Metocean and earthquake actions	Based on statistical data	Expected action history	Based on statistical data	NOT APPLICABLE	Based on statistical data	Dependent on operational requirements	Expected action history	Annual probability of exceedance $=10^{-2}$	Annual probability of exceedance $=10^{-4}$	Annual probability of exceedance $=10^{-2}$ *2)
Weather restricted metocean actions	SPECIFIED VALUE	SPECIFIED ACTION HISTORY	SPECIFIED VALUE	NOT APPLICABLE	SPECIFIED VALUE	NOT APPLICABLE				
Deformation actions	SPECIFIED VALUE	Expected action history	Expected extreme value	SPECIFIED VALUE		EXPECTED EXTREME VALUE				
Accidental actions	NOT APPLICABLE			SPECIFIED VALUE (depends on measures taken)	NOT APPLICABLE			Annual probability of exceedance $= 10^{-2}$ *3)	Annual probability of exceedance $= 10^{-4}$ *4)	Not applicable

*1) For time limited operations, it is sufficient to consider the relevant months when determining actions, see i.e. DNVGL-ST-N001

*2) Use 10^{-2} for correlated events, 10^{-1} for uncorrelated events, given that normalization can be achieved within one month.

*3) Accidental actions are applicable for ULS. E.g. ship collision for concrete gravity based structures. The most unfavourable combination of actions with a joint annual probability of exceedance of 10^{-2} should be applied. However, individual accidental, metocean or earthquake actions at a probability level of 10^{-2} will commonly be most unfavourable. Accidental actions should be used in ULS with a partial safety factor, ref NORSOK N-001, equal to the partial safety factor for environmental actions. If a combination of environmental and accidental actions are most unfavourable, a partial factor in the range between 0,7 and 1,3 should be chosen depending on the correlation between these actions (1,3 for fully correlated).

*4) The most unfavourable combination of actions with a joint annual probability of exceedance of 10^{-4} should be applied. However, individual metocean, earthquake or accidental actions at a probability level of 10^{-4} will commonly be most unfavourable. See 10.4.1.

*5) Additional information, see 10.4.2.

Definition and recommendations related to temporary conditions are given in 10.2.

Combinations of metocean and earthquake actions for normal operations are given in 10.3 and combination of accidental actions for normal operations is given in 10.4.

Note that for FLS the total fatigue action is the sum of actions in normal operation and in temporary conditions.

If a high static stress from deformation actions is beneficial in the action combination, e.g. for fatigue calculation, a representative mean stress should be used rather than expected extreme value.

10.2 Temporary conditions

Temporary conditions may refer to conditions during fabrication, installation or short-term situation in operation. Reference is made to DNV-OS-H102.

A key parameter for marine operations is the total duration, called the Operation reference period, T_R. It is defined as the sum of the Planned operation period, T_{POP}, and the Estimated maximum contingency time, T_C, thus $T_R = T_{POP} + T_C$.

If $T_R \leq 96$ hours and $T_{POP} \leq 72$ hours, the operation may be defined as *Weather restricted*. If not, the operation is normally to be defined as *Weather unrestricted*.

If the operation can be halted and the handled object be brought into a safe condition within the same period, i.e. 72/96 hours, the operation may be divided into sub-operations, each defined as weather restricted. The total duration of the operation is thus more than 72/96 hours.

Weather restricted operations may be designed based on environmental criteria defined by the owner or his representative. The chosen criteria should reflect the conditions that could be expected based on area and season, and also appreciate that too strict criteria could lead to excessive periods with waiting on weather.

The uncertainty in the weather forecasts shall be accounted for. One method is given in the DNVGL-ST-N001. A factor, α, depending on T_{POP} and the precautions taken is given. The environmental design criteria (wave height and/or wind speed) are multiplied by this factor in order to define the operational limits. The α-factor is given for $T_{POP} \leq 72$ hours. However, the weather forecasts will normally contain useful information also for longer periods. If it can be documented for a particular weather restricted operation that the weather forecasts can predict any extreme weather conditions for a longer period, AC T_R AC may be increased, and the uncertainty in the weather forecast be duly accounted for. On the other hand, if reliable weather forecasts for such a period are not considered realistic, the Operation reference period should be reduced accordingly.

In polar areas, there may be locally increased wind speeds due to the formation of low pressure systems, polar lows. The wave heights could also be increased depending on the duration of the increased wind speed etc. This phenomenon may not at present be captured by weather forecasts, as polar lows can only be forecasted for rather short lead times (say, less than 24 hours). For longer periods, only indications of polar lows within larger regions can be given. The α-factor given by DNVGL-ST-N001 may not be representative for such areas. Hence, for marine operations in areas where polar lows may occur, precautions should be taken.

Mitigating actions could be to:

– If possible, perform marine operation outside the polar low season. (The formation of polar lows happens more frequently during winter season.)
– Choose a robust design in order to reduce the sensitivity to (unpredicted) strong wind and increased wave heights.
– If feasible, operations should be designed in such a way that the handled object can be brought into a safe condition within a short period of time.
– In due time prior to execution of the operation, the capability of the forecaster to predict the polar lows should be verified, and how to present the results should be agreed upon with the meteorologist (the forecasting company).

Weather unrestricted marine operations cannot be planned based on weather forecasts. The environmental conditions used for planning have to be based on long-term environmental statistics. The environmental actions will then be based on a condition with a given (low) probability to be exceeded, accounting for the

– geographical area,
– season of the year,
– duration of the operation.

The environmental criteria for Weather unrestricted operations may be defined using return periods. Guidance on selecting return periods for a given T_R can be found in DNVGL-ST-N001 and in ISO 19901-6.

The extreme values for wave heights may be found based on scatter diagrams.

The environmental criteria can be derived from hindcast models, possibly calibrated against satellite data or in-situ wave buoy data. Such data may be purchased from commercial vendors.

Accidental actions in temporary phases depend upon the measures taken. Accidental actions and possible abnormal operational conditions in temporary phases are to be determined by risk analysis. In temporary phases of short duration, controls may normally be implemented to ensure that accidental actions are negligible.

Precautions may be taken to ensure that maximum action effects do not occur simultaneously, e.g. that tank pressures associated with pressure testing do not occur during maximum environmental conditions.

10.3 Combinations of metocean and earthquake conditions

Characteristic values of individual metocean and earthquake conditions are defined by annual exceedance probabilities of 10^{-2} (for ULS) and 10^{-4} (for ALS). The long-term variability of multiple conditions is described by a scatter diagram or joint density function including information about direction. Contour curves or surfaces for more than two parameters, can then be derived giving combinations of parameters approximately describing the various actions corresponding to the given exceedance probability.

Table 7 shall be used as a conservative approach to combination of actions in lack of more detailed and verified joint models of actions. Alternatively, calculation of action and action affects can be based on datasets with multiple parameters and all relevant parameters are included in the action calculations, e.g. long simultaneous recordings of winds, waves and currents if these data sets are of sufficient quality and time period, see 6.2.3. Hindcast data or synthetic time series may also be applied conditional the data quality and correlation between parameters is well documented.

Alternatively, the exceedance probabilities can be referred to the action effects. This is particularly relevant when the direction of the action is an important parameter.

For fixed facilities colinear metocean actions are normally most critical, and the action intensities for various types of actions can be selected to correspond to the exceedance probabilities given in Table 7.

For other facilities action combinations which involve a large difference in action direction shall be addressed.

In a short-term period with a combination of waves and fluctuating wind, the individual variations of the two action processes can be taken to be uncorrelated.

For sea ice, wave actions can be disregarded if actions caused by a continuous ice sheet are considered. If impacts from individual ice floes in low ice concentrations are considered, associated waves are crucial in order to estimate both energy in ice impacts and possible locations for ice impacts.

Table 7 – Combination of water levels, metocean and earthquake conditions with expected mean values and annual probability of exceedance 10^{-2} and 10^{-4}

Limit states		Wind	Waves (e	Current (f	Sea spray icing	Sea ice	Ice-bergs	Snow	Earth-quake	Sea level (a
Ultimate limit states	1	10^{-2}	10^{-2}	10^{-1}	-	-	-	-	-	$HAT + S_{10^{-2}}$
	2	10^{-1}	10^{-1}	10^{-2}	-	-	-	-	-	$HAT + S_{10^{-2}}$
	3	10^{-1}	10^{-1}	10^{-1}	10^{-2}	-	-	10^{-1}	-	MWL
	4	10^{-1}	0,63 (c	10^{-1}	-	10^{-2}	-	-	-	MWL
	5	10^{-1}	10^{-1}	10^{-1}	-	-	10^{-2}	-	-	MWL
	6	10^{-1}	10^{-1}	10^{-1}	10^{-1}	-	-	10^{-2}	-	MWL
	7	-	-	-	-	-	-	-	10^{-2}	MWL (b
Accidental limit states	1	10^{-4}	10^{-2}	10^{-1}	-	-	-	-	-	$MWL + S_{10^{-4}}$
	2	10^{-2}	10^{-4}	10^{-1}	-	-	-	-	-	$MWL + S_{10^{-4}}$
	3	10^{-1}	10^{-1}	10^{-4}	-	-	-	-	-	$MWL + S_{10^{-4}}$
	4	10^{-2}	10^{-1}	-	10^{-4}	-	-	-	-	MWL
	5	-	-	-	-	10^{-4} (d	-	-	-	MWL
	6	0,63	0,63	0,63	-	-	10^{-4} (d	-	-	MWL
	7	0,63 (g	0,63 (g	-	-	-	-	10^{-4}	-	MWL
	8		-		-	-	-	-	10^{-4}	MWL (b

a) HAT: Highest Astronomical Tide; MWL: Mean water level; MWL+S: Mean water level, including the effect of storm surge with given q-probability

b) Seismic response analysis should be carried out for the most critical water level.

c) In determination of the combination of sea ice and other action contributions, the exposure period should be evaluated in order to establish associated values of wind, waves and current.

d) With respect to ULS and ALS related sea ice and iceberg actions, it shall be acknowledged that there are several realisations of the ice/iceberg conditions that can cause actions corresponding to these exceedance levels. Further, it shall also be acknowledged that the ice conditions also will depend on the failure mode under consideration

e) If using the contour line approach, the worst combination of Hs/Tp for the given q-probability shall be used.

f) For current a 1-hour mean shall be used for 10^{-1} values and 10 min mean for 10^{-2} and AC⟩ 10^{-4} ⟨AC values, as described in 6.2.2.2

g) If effects of wind drift not are included in ALS estimate for snow actions, action contributions from wind and waves can be excluded

The temperature to be used with the combinations in Table 7 should be set to the mean temperature over 24 hours with an annual probability of exceedance of 10^{-2}.

For stability check recommendations are given in NORSOK N-001.

10.4 Combination of accidental actions

10.4.1 General

When accidental actions occur simultaneously, the probability level (10^{-4}) applies to the combination of these actions. Unless the accidental actions are caused by the same phenomenon (like hydrocarbon gas fires and explosions), the occurrence of different accidental actions can be assumed to be statistical independent.

While in principle, the combination of two different accidental actions with exceedance probability of 10^{-2} or one at 10^{-3} and the other at a 10^{-1} level, correspond to a 10^{-4} event, individual accidental actions at a probability level of 10^{-4}, commonly will be most critical.

10.4.2 Variable and metocean actions in combination with accidental actions

Account shall be taken of the permanent and variable actions that might reasonably be present at the time of the accidental event.

When metocean and accidental actions occur simultaneously, the given probability level applies to the combination of these actions. Unless metocean actions contribute to the occurrence of the accidental actions, these two actions can be assumed to be statistically independent. The expected metocean action occurring together with the AC 10^{-4} AC accidental action can be neglected unless the accidental action is initiated by the metocean action.

10.4.3 Post damaged condition

The damaged structure (resulting from an accidental action event) shall be able to resist relevant permanent and variable actions in a metocean condition corresponding to annual exceedance probability of 10^{-2}. If the accidental event leading to the damaged condition is uncorrelated with metocean actions and normalization can be achieved within one month, an annual exceedance probability of 10^{-1} may be used in checking the damaged structure resistance.

Operational measures in the immediate post damage condition to remediate the situation can worsen the damage if not performed properly. Hence, such operational measures should not be assumed.

10.5 Static and dynamic pressures in tanks

When combining static and hydrodynamic tank pressures, appropriate account of relevant tank filling and acceleration levels for different tanks should be accounted for.

The pressure height, and pressure, associated with pumping operations (see 5.4.1) could be neglected if such operations are documented not to occur at the same time as extreme sea actions.

11 Action effect analyses

11.1 General

11.1.1 Action effects

Action effects, in terms of motions, displacements, or internal forces and stresses in the structure, should be determined with due regard to their variation in time and space. Determination of the characteristic values of these action effects is dependent on:

- type of structure,
- the relevant limit states for design check,
- dynamic character of the action and action effect,
- possible nonlinearities of the action and action effect,
- the desired accuracy.

11.1.2 Classification of structures

The global behaviour of offshore structures may be characterised by the following modes of behaviour:

- Restrained modes of behaviour indicate a stiffness controlled motion (typically a natural frequency above the wave frequency).
- Free modes allow for rigid body motions (typically a natural frequency in or below the wave frequency range).

Table 8 characterises some concepts.

Table 8 – Characterisation of global behaviour

Type of structure	Translation			Rotation		
	Vertical (Heave)	Horizontal (Surge)	Horizontal (Sway)	Vertical plane (Pitch)	Vertical plane (Roll)	Horizontal plane (Yaw)
Fixed structures (jacket, jack-up, GBS, subsea modules)	R	R	R	R	R	R
Installation phase (jacket, subsea modules)	R/F	F	F	F	F	F
Tension-leg, with three legs or more	R	F	F	R	R	F
Floaters (e.g. semi-submersible, shipshape and spar)	F	F	F	F	F	F
Articulated tower	R	-	-	F	F	R
F: free mode (restoring actions primarily due to hydrostatics and mooring actions) R: restrained mode (restoring actions primarily due to structural stiffness)						

The determination of action effects for structures with:

– only restrained modes are described in 11.2,
– free modes are described in 11.3 and 11.4.

Considerations applicable for all types of structures can be found in 11.6.

11.1.3 Classification of analysis types

In cases where inertia and damping actions are important, dynamic analysis should be used. Alternatively, static analysis including equivalent inertia actions can be performed, see 11.2.2.2. Dynamic analysis including intertia actions should also be used for transient action effects relating to impacts and explosions.

A classification of analysis types for structural action effects are found in Table 9. Stability analyses are not included.

Table 9 – Overview of recommended analysis types for structural action effects

Action type	Only restrained mode		Free mode	
	ULS & ALS	FLS	ULS & ALS	FLS
Permanent	Static	N.A.	Static	N.A.
Variable	Static	Vibration from machinery	Static	Vibration from machinery, engines, thrusters etc.
Deformation	Static	N.A.	Static	N.A.
Wave and current	Static or dynamic wave-structure-foundation analysis 11.2 and 11.6	Static or dynamic wave-structure-foundation analysis	Inertia and damping forces shall be included by a global wave motion analysis 11.3 and 11.6	Inertia and damping forces shall be included by a global wave motion analysis 11.3
Wind	Static or dynamic wind-structure-foundation analysis 6.4.7	Static or dynamic wind-structure-foundation analysis 6.4.7	Low frequency and coupled analysis 11.3.3 -11.3.5	N.A.
Sea ice	Static including vibration effects	N.A.	Static NOTE 1	N.A.
Iceberg	Dynamic NOTE 2 and 3	N.A.	Dynamic NOTE 3 and 4	N.A.
Icing	Static	N.A.	Static	N.A.
Snow	Static	N.A.	Static	N.A.
Earthquake	Seismic action effect analysis 11.7	N.A.	N.A.	N.A.
Accidental impact, explosion, etc.	Transient dynamic nonlinear analysis including inertia effects	N.A.	Dynamic nonlinear analysis including inertia effects	N.A.
Fire		N.A.		
NOTE 1 actions need to be determined on the basis of a dynamic system as the actions will depend on the global action effect of the structure NOTE 2 for fixed structures, the usual practice is to omit the dynamic properties of the structure and only assess the dynamic response of the iceberg NOTE 3 For local iceberg actions (which are usually determined on the basis of pressure-area curves), a static action effect analysis is acceptable. NOTE 4 For a floating structure interacting with icebergs, the global actions and action effects are interrelated				

Normal uncertainties in the analyses models are taken into account by the partial safety factors given in NORSOK [AC) N-001[4]. (AC] If uncertainties are particularly high, conservative models shall be selected and sensitivity of the models and the parameters utilised in the models shall be examined. If geometric deviations or imperfections have a significant effect on safety, conservative geometric parameters shall be used in the calculation.

Simplified methods may be applied if they are properly documented and they provide conservative results.

For any novel systems, important action effect predictions should be:

– verified by appropriate model tests, see 11.1.8.
– validated by full scale measurements in the early stages of operation, see 11.1.9.

[4] The partial action factor for environmental actions as specified in NORSOK N-001 has not been calibrated to take account for the uncertainty in sea ice and iceberg actions. For floating structures neither the partial action factor specified in ISO 19906 is valid. A site-specific calibration of action factors is recommended.

11.1.4 Determination of characteristic action effects

Characteristic values of the action effects (displacements, forces or stresses in the structure, mooring and foundation) shall be determined in accordance with the methods given in 6.1.3 for relevant combinations of actions. These methods shall take adequate account of the variation of actions in time and space, the motions of the structure and the limit states to be verified.

The dynamic equilibrium equation for a structure may formally be expressed as

$$(\boldsymbol{M} + \boldsymbol{M_A})\ddot{\boldsymbol{X}} + \boldsymbol{C}\dot{\boldsymbol{X}} + \boldsymbol{K}\boldsymbol{X} = \boldsymbol{F} \tag{19}$$

where $\boldsymbol{M}$, $\boldsymbol{M_A}$, $\boldsymbol{C}$ and $\boldsymbol{K}$ are structural mass, added mass, structural damping and structural stiffness, respectively, $\boldsymbol{X}$, $\dot{\boldsymbol{X}}$ and $\ddot{\boldsymbol{X}}$ are the displacements, velocity and acceleration of the structure, and $\boldsymbol{F}$ is excitation force.

It is noted that Equation (19) needs to be modified if stochastic time-domain analysis is carried out for systems with frequency dependent dynamic properties. Terms should be added to adjust for the frequency dependent added mass and damping coefficients.

Caution should be exercised when using this approach to determine wave excitation, added mass and damping actions at intersections between large diameter members.

Nonlinear and dynamic effects associated with actions and structural response shall be accounted for, whenever relevant.

11.1.4.1 Action effects from static actions

The characteristic action effect of permanents actions, functional actions, deformation actions, snow, icing, and other actions where the structure behaves statically shall be determined directly from the characteristic action.

11.1.4.2 Action effects from waves and current

The characteristic wave- and current action effects may be determined by

– long-term analysis based on a set of stochastic sea states,
– a set of stochastic short-term sea states,
– a design wave approach, specified by a regular long-crested wave with given height and length, or a set of regular waves,

with due consideration of current.

For structural design purpose, a combination of these approaches may be necessary to determine characteristic action effects for design checks.

Actions effects in irregular waves should be estimated with due account of the inherent statistical variation. Stochastic analysis can be done in time or frequency domain. Stochastic analysis is especially necessary for dynamic systems. The natural (inherent) variability of extreme actions in various short-term stationary conditions shall be properly accounted for. Doing a long-term analysis based on the most probable maximum action or action effect in the various short-term se states will generally underestimate the underlying q-probability action, if not conservative assumptions are known to compensate for this underprediction. Different realizations of the most critical sea states should, therefore be used in numerical simulations as well as experimental prediction of the extreme action effects under such conditions. If characteristic actions and action effects are the target quantities, the accumulated duration of the critical weather condition should be sufficiently long to ensure that the natural variability of the extremes are captured. E.g. estimating the 90 % percentile extreme action value will typically require 20 realisations or more of the given 3 hour sea state.

Stress ranges for fatigue design should be determined for a representative set of sea-states using the long-term stochastic approach. A linear frequency domain approach is normally applicable to determine the response in each sea-state. Particular attention to nonlinear wave effects (e.g. a wave attenuation variation, and slamming) in the splash zone as well as possible sloshing in tanks, is, however, necessary. Simplified approaches based on regular waves to determine fatigue action may be used for jacket structure members in splash zone, and for other types of structures if properly validated.

11.1.4.3 Action effects from wind

The fluctuating component of wind velocity is specified by a wind spectrum.

Stochastic wind analysis is described in 6.4.7.

11.1.4.4 Action effects from earthquake

Seismic actions may be described by ground response spectrum or time-domain motion histories, see 11.7.

11.1.4.5 Accidental actions effects

Accidental actions and action effects are described in Clause 9.

11.1.4.6 Action effects from sea ice and iceberg

Action effects from sea ice and iceberg are described in 6.8 and 6.9.

11.1.5 Extreme action effects for ultimate limit states (ULS)

Ultimate limit states shall include the following undesirable states (see i.e. ISO 2394 and ISO 19900):

- loss of equilibrium of the structure or part of it considered as a rigid body;
- instantaneous attainment of the maximum capacity of sections, members or connections by yielding, rupture or excessive deformations;
- failure of members or connections caused by fracture, fatigue or other time-dependent accumulation effects;
- instability of the structure or part of it;
- sudden change of the assumed structural system to a new system, (e.g. snap through, large crack formation);
- foundation failure;
- stability and floatability;
- stationkeeping.

The ultimate limit states may be defined with respect to a single extreme action effect event or from a deterioration process over time followed by a (less) extreme action effect event.

Commonly used design methods are based upon the assumption that design values for the action effect and resistance can be defined separately by introducing partial safety factors on the characteristic action effect and resistance. Hence, normally the ULS check is performed by carrying out a linear elastic response analysis of the structure as a whole to determine forces or stresses in the individual components. However, nonlinearities in the structure, mooring and foundation may change the natural period of the structure, and thus increase the predicted action effects. Hence, check of sensitivity to nonlinearities or a nonlinear analysis should be performed.

Nonlinear action effects e.g. due to ship rolling, ringing, and effects of hydroelastic slamming, VIV should be checked. In addition, transient dynamic effects may influence the type of failure mode that need to be evaluated in the ULS and ALS check. E.g. slamming properly modelled may cause shear to be the critical failure mode rather than bending for certain rise time of the slamming action.

The characteristic action effect for ULS checks refers to a maximum, and in special cases, a minimum action effect corresponding to an annual exceedance probability of 10^{-2}, see Clause 6. For combination of characteristic action effects see Table 6 and Table 7.

In the description of action effects for ULS analysis, the following issues should be considered:

- Ultimate strength control typically considers average stress levels over an area which causes buckling. For the evaluation of buckling strength, mid-plate (membrane) element stress data should be used. For panels with large stress gradients, the variation of stress shall be considered in the buckling evaluation.
- When multiple stress/force components are used to model the component strength, the strength may be expressed by interaction equations.
- For design checks involving multiple stress/force components, various sets of combined response variables (at an annual exceedance probability of AC 10^{-2} AC), should be checked to ensure that the most critical set of characteristic response values is applied in the evaluation of the structure.

11.1.6 Repetitive action effects for fatigue limit states (FLS)

The repetitive action for fatigue limit states is described by the distribution of stress ranges for welded metal structures. For basic (i.e. rolled and cast) metal material the joint distribution of mean stress and stress range is required.

The main contribution to fatigue actions normally comes from the local and global effect of frequently occurring moderate wave and wind conditions. Fatigue design requires a description of the long-term variation of local stresses due to wave as well as possible sum-frequency wave actions, variable buoyancy, splash zone and ship slamming, springing, wave- or current-induced vortex shedding, or, mechanical vibration. The effect of local (e.g. pressure) and global actions shall be properly accounted for.

Account should be made of repetitive actions during fabrication, tow-out, installation as well as temporary and permanent, in-place conditions. For structures with oil storage, the repetitive effects of loading and unloading should be considered.

A linear elastic model of the structure is generally adequate when determining the action effects for FLS. Dynamic analysis to determine action effects should be performed if dynamic effects are significant, e.g. sum frequency wave actions, and other structural vibration. Simplified approaches to include dynamic effects may be used for e.g. jackets with low degree of dynamic sensitivity (Te<2 sec).

Miscellaneous hull appurtenances associated with risers, riser guides, anodes, mooring equipment should be evaluated for fatigue resistance, using local analyses. The calculation of actions on such components should be generated using water particle velocities and accelerations from diffraction analyses of the system that may possibly affect the mentioned components.

Stress ranges due to wide-band Gaussian or non-Gaussian response processes should be determined by an appropriate method of cycle counting, e.g. the rain-flow method. Simple conservative methods for combining (e.g. wave and high or low frequency action effects) may be applied.

Detailed fatigue analyses should be performed using conservative deterministic analyses, spectral techniques, and in particular situations, by stochastic time-domain analysis. Accurate analyses of local action effects in the splash zone would require time-domain analyses.

Linearized kinematics can normally be used for FLS analysis, but should be documented by checking nonlinear contributions.

11.1.7 Accidental limit states (ALS) analyses

The ALS design check requires evaluation of

- the structural damage caused by accidental actions,
- the ultimate capacity of the structure as a whole with damage.

ALS design checks will normally be performed by simplified nonlinear analyses methods both to calculate the damage and the global ultimate strength of the damaged structure. In particular calculation of damage due to accidental actions such as ship impacts and explosions may be based on plastic mechanisms (yield hinge or line methods) with due recognition of possible premature rupture.

Nonlinear FE analyses may also be applied on similar conditions as mentioned in 11.1.10.2.

Further details about such analyses may be found in the design standards for each type of material, e.g. steel, concrete, aluminium.

11.1.8 Model testing

11.1.8.1 Purpose

In the conceptual design phase, the primary objective of model tests in e.g. wave basin, ice tank or wind tunnel, is to confirm that no important feature (e.g. ringing, slamming or cork-screw motion of spar buoys with strakes) has been overlooked for temporary and in-place conditions. Details regarding planning and the execution of model tests can be found in AC A.17. AC

Hydrodynamic model tests should be carried out to:

a) confirm that no important hydrodynamic action has been overlooked (for new types of facilities, metocean conditions, adjacent structure),
b) support theoretical calculations when available analytical methods are susceptible to large uncertainties,
c) verify theoretical methods on a general basis.
d) determination of action/action effects for complex problems where numerical methods are insufficient.

Wind tunnel tests should be carried out when

a) wind actions are significant for overall stability, motions or structural response,
b) when available theoretical methods are susceptible to large uncertainties (e.g. due to new type of facilities or adjacent facility affects the relevant facility in order to support or replace theoretical calculations completely,
c) there is a danger of dynamic instability.

When the wind conditions are determined for helicopter decks and structures with large motion (structures with free modes, see 11.1.2), wind model tests or validated CFD simulations shall be carried out. For helicopter decks see also NORSOK C-004.

Model tests may also be relevant to investigate combined wave-, current- and wind effects, e.g. for assessment of mooring actions and offset of facility.

Theoretical models for calculation of actions from icebergs or drift ice should be checked against validated model tests or full-scale measurements.

For model tests, it is important that the model has sufficient similarity to the actual facility and that the test set-up and registration system provide a basis for reliable, repeatable interpretation, see 11.1.8.2.

In the final design phase, the purpose is to validate the computed action effects (e.g. motions, mooring line tension, run-up, slamming, ice actions) of a particular design. In this case it is important to predict the full-scale response of the actual system.

An important feature of model tests is that the results are obtained without requiring many assumptions about the nature of the action effects. This is generally not true of numerical models. However, model testing has its limitations. In particular, one should be aware that viscous damping effects may be over-estimated in model tests. Numerical predictions and model experiment results should be considered as being complimentary to one another.

Through careful interpretation, each of these results may be used to partially circumvent the limitations of the other.

Implementation of model test results

When implementing experimental test results into design, all relevant deviations between the model test and reality shall be considered. Such deviations may include

- scaling effects,
- model simplifications (e.g. related to damping),
- limitations in testing facilities (e.g. finite dimensions; quality of waves, current and wind; wave absorption),
- simplifications and uncertainties regarding the data acquisition and processing,
- uncertainties with regard to long-term effects,
- the failure mode.

Statistical uncertainties with respect to limited sample maxima of test results shall be included in the determination of model responses.

Extreme values of action effects due to stochastic actions should be determined with due consideration of all information contained in the sample, with particular emphasis on the largest values of the sample. Extrapolation based on fitted distribution should be used.

The model test shall be planned, executed and documented in such a manner that they are repeatable. Some guidance on basin and wind tunnel tests are given in AC A.17. AC Further details can also be found in DNV-RP-C205.

Interpretation of experimental results should be carried out based on support by numerical analyses both in model and in full scale.

11.1.9 Full-scale measurements

Full-scale measurements may be used to update the response prediction of the relevant structure and to validate the methods for analysis of action effects for future design or redesign.

The updated analysis of the actual structure may have implications on operational requirements.

Such measurements tests should especially be devoted to actions and action effects which are difficult to simulate in model scale, i.e. associated with soil conditions and structures subjected to hydrodynamic actions involving both potential and viscous effects, as well as nonlinear effects such as ringing and air gap.

It is crucial to ensure adequate instrumentation to monitor metocean conditions during full-scale measurements.

Interpretation of field observations should be carried out with due account of the inherent uncertainties, especially associated with the relevant environmental conditions.

11.1.10 Modelling of structures by finite element methods

11.1.10.1 General

The structural response may be divided into two broad categories as follows:

- global structural response, which requires global structural models that simulate, with sufficient accuracy, the effects of global actions on the structure;
- local structural response, which normally requires local structural models that simulate with sufficient accuracy the local action effects. In addition, local actions should be included e.g. hydrostatic pressures, tank pressures, point forces, slamming.

The action effects may be influenced by the finite element model by e.g.:

- modelling of boundary conditions;
- choice of element type and mesh relative to action areas and relevant action effect;
- the element models ability to capture the critical action effects (stresses) and especially for nonlinear analysis the realistic failure modes;
- modelling of dynamic effects.

Sensitivity evaluations in order to determine a conservative approach shall be performed.

Boundary conditions will also vary in different phases (temporary and permanent), and sensitivity to realistic boundary conditions will be needed in both cases.

The actions and action effect of secondary structural components not included in the finite element model and local actions shall be included in the analyses.

Considerations of global action effects in local models may be undertaken using one of the methods listed as follows:

- mapping of actions (or action effects) from global model to local model (sub-modelling), by e.g. using displacement or force boundary conditions obtained from the global analysis;
- integration of local model into global model;
- superimposing action effects from the global model on the local model action effects.

Supplementary manual calculations for members subjected to local actions may be adequate in some cases, based on empirical formulas or basic engineering principles. The actions used for these calculations should come from the global FE analysis and from local actions acting on the structure.

For further guidance on FE modelling on steel and concrete structures, see NORSOK N-004 and Holand et al. (2000), respectively.

11.1.10.2 Nonlinear finite element analysis

Nonlinear analyses may be based on engineering theories such as yield hinge or other theories of plastic mechanism for nonlinear material problems and sufficient ductility. Non-linear finite element analysis should be performed in accordance with DNV-RP-C208.

Structural components under compression need to be modelled properly to account for instability (i.e. buckling). Hence, geometrical imperfections and residual stresses should also be modelled, when they have a significant effect on the response (action effect).

When using nonlinear calculation models, adequate consideration shall be given to the fact that results depend on the action history. It shall be demonstrated that the least favourable action history is utilised. Generally, it will be necessary to undertake parametric studies to evaluate different action histories in order to cover all modes of failure and structural elements.

The FE methods / computer codes applied to carry out nonlinear analyses should be verified against test results or observed behaviour of full-scale structures, as well as known analytical solutions or other well-documented FE solutions.

In case nonlinear material properties are used in the nonlinear analysis, local failure criteria should be considered (e.g. rupture).

11.2 Metocean action effects for structures with only restrained modes

11.2.1 Global analysis

11.2.1.1 General

The purpose of the action effect analysis described in this section is to determine metocean action effects relevant for foundation, main structure, topside and other relevant components, e.g. risers, etc.

Excitation by waves, current and wind should be considered. The response analysis shall be carried out with due regard to combined metocean conditions, e.g. wave, current and wind intensity and direction. If joint statistical data for the site or area considered are not available, conservative assumptions shall be made, including consideration of unidirectional versus multidirectional environment.

Special considerations are required to assess possible abnormal environmental conditions or unforeseen action effects at an annual exceedance probability of 10^{-4}, e.g. by model testing for new and novel concepts.

The model, or models, of the structure and soil/foundation shall be selected to adequately represent the simultaneous global and local action and provide the action effects needed for different limit states

The effects of secondary and/or non-structural components shall be considered in an appropriate manner. As a minimum, such components need to be modelled to account for the hydrodynamic forces and inertia forces due to motions.

Adequate soil-structure interaction shall be included in global analyses. Some guidance on modelling of soil properties can be found in DNVGL-RP-C212.

For space frame structures consisting of slender members, a three-dimensional frame model may be used to calculate internal member forces and moments. The effect of joint eccentricity and flexibility, where significant, should be accounted for. Also, possible shear-lag and shear deformations should be accounted for. For space frames integrated with plated structures (e.g. deck) care should be exercised in modelling their interaction with beam elements.

Solid FEs may be required to be applied to represent stresses where three-dimensional stress conditions occur.

11.2.1.2 Static action effects

Static action effects should include actions from:

- permanent pay loads, e.g. gravity, buoyancy, etc.;
- deformation actions due to settlements etc.;
- variable actions from action distribution on deck etc.

Actions from snow and icing, see 6.5 should also be accounted for.

11.2.1.3 Dynamic action effects

Dynamic effects shall be considered in evaluation of the structural design. Dynamic response shall be considered when the period of steady-state action is close to a natural period of the structure or when the structure (or part thereof) is exposed to transient type of action. Dynamic effects may be important for example in connection with:

- wave frequency actions if the natural period of the structure exceeds 3 s and 2 s for ULS and FLS, respectively;
- sum frequency effect of wave actions (e.g. springing, ringing);
- wave slamming and other transient wave actions;
- wind action if the natural period of the structural part exceeds 0,2 s;
- vortex induced vibrations;
- earthquake action.

Dynamic response is obtained in the frequency or time-domain. The linear response to a steady state action may conveniently be determined in the frequency domain. Transient response is most easily determined in the time-domain.

Dynamic effects of impulse-type action may similarly be accounted for by conservatively using recognised charts or formulas for the dynamic amplification for problems that can be represented by a single mode.

Structural damping

The dynamic model involves mass, damping and stiffness. Modelling of damping should be based on in-service experiences with similar types of dynamic structural systems. Caution needs to be exercised to avoid overestimation of damping when combining measured damping with damping implicit in theoretical models. Indicative values can be found in A.18, Table A.6

If the action effects are very dependent on the choice of damping, sensitivity analysis shall be performed and conservative estimates applied.

11.2.1.4 Structural nonlinear effects

Relevant considerations shall be given to nonlinear action and response in the evaluation of the structural design.

11.2.2 Stochastic wave and current action analysis

11.2.2.1 Quasi-static analyses

For structures where inertia and damping forces due to wave and current are small (natural period less than 1 s), response will be in phase with the loading. For such structures, a design wave approach can be used to obtain structural actions and action effects, see 6.2.1.3.

Linear waves may be used in connection with a simplified fatigue analyses for cases not being sensitive to kinematics close to exact surface. If kinematics close to surface is important, nonlinear wave theories should be used as the design wave profile.

The design wave height H_q or design crest height C_q corresponding to an annual exceedance probability of q shall be determined by a long-term analysis, see 6.1.3. Adequate short-term distributions of wave height and crest height are given in 6.2.1.5. The mean period of the design wave, with height H_q, can be taken as $T = 0{,}9T_p$ where T_p is the spectral peak period corresponding to $H_{s,q}$ in the sea-state with annual exceedance probability of q. A design wave is then fitted to $(H_q\,, T)$ or $(C_q\,, T)$, whichever gives the highest action.

The design wave is in most cases adequately modelled by Stokes 5th order theory if the water depth is larger than 15 % of the design wavelength. Stream function theory is applicable for more shallow water. Special considerations should be made for applications that are sensitive to particle velocity at top of crest (e.g. wave in deck impact).

As an alternative, the design wave height may be taken to be 1,9 times the significant wave height H_s corresponding to an annual exeedance probability of q determined by long-term statistics using a duration of the sea-state equal 3 hours.

If the design wave is characterized by the wave height, H_q, the design wave period should be taken as the most unfavourable value in the following range:

$$2{,}55\sqrt{H_q} \le T \le 3{,}32\sqrt{H_q} \tag{20}$$

where the unit of H_q is meters and the unit of T is seconds. Alternatively, a proper period band can be established by a thorough analysis of site specific data.

If the design wave is characterized by the wave crest C_q the design wave period could be taken as the conditional mean period.

As a general recommendation the wave characteristics associated with an annual exceedance probability of q=10^{-4} should be estimated by a full long-term analysis. In absence of more detailed documentation, the wave height, AC⟩ H_q, ⟨AC with annual exceedance probability q=10^{-4} can be taken to be 1,30 times $H_{0,01}$, while the period is increased by 10 %, as compared to the period of $H_{0,01}$. The crest height, C_q with annual exceedance probability q=10^{-4} can be taken to be 1,35 times $C_{0,01}$. For shallow water, special considerations should be made.

An assessment of characteristic actions and action effects is conservatively done, utilizing the omni-directional q-annual probability wave height, H_q, for all directions. If directional q-probability wave heights are adopted, these wave heights are to be corrected according to 6.1.4 prior to estimating characteristic actions and action effects.

For quasi-statically behaving fixed platforms, Stokes 5th order wave profiles can be used for simplified fatigue analysis. If the simplified approach suggests a fatigue close to minimum required, results should be verified by more sophisticated methods.

11.2.2.2 Dynamic analysis by the use of simplified SDOF

For structures where dynamic effects are relatively small (natural period less than 2 seconds), simplified method for estimating a dynamic amplification factor (DAF) based on single degree of freedom system (SDOF) may be used in combination with design wave approach.

11.2.2.3 Dynamic analysis by the use of equivalent dynamic acceleration fields (EDAF)

For structures where inertia forces are important (significant dynamic response due to excitation of natural frequency by sum frequency effects and transient actions) the design wave approach can be used to determine the action effects in ULS and ALS, provided that a representative equivalent dynamic acceleration field (EDAF) can be tuned to properly represent the inertia forces in the structure. In this case the dynamic effect can be accounted for by adding the response from a properly tuned equivalent dynamic acceleration field (EDAF) to the quasi-static response from the design wave.

This approach is typically used in combination with the contour method (see 6.1.3). In the tuning of the acceleration field, three steps should be carried out:

Step 1: Screening to identify the most severe sea state along the environmental contour.

Step 2: From this sea state; estimate quasi-static and dynamic design values for relevant global responses (typically OTM, Base Shear and deck displacement), see 11.2.2.5.

Step 3: Tune the equivalent dynamic acceleration field (EDAF) such that it gives a contribution from inertia forces corresponding to the difference between dynamic and quasi-static response.

The estimation of the design values in Step 2 should be based on a large number of 3-hour realizations (at least 30), see 11.2.2.5. The response should be determined at 0,9 percentile for ULS and 0,95 percentile for ALS using the Gumbel extreme value distribution of the maximum value in each realization. Identical wave realizations for quasi-static and dynamic responses should be used in order to minimize the uncertainty with respect to the ratio between design values for dynamic and quasi-static responses.

The premise for using an equivalent dynamic acceleration field (EDAF) design approach is that the equivalent acceleration field representing the inertia forces in the structure can be established. It should be verified that the approach is suitable for the type of loading and dynamic response modes in the structure analysed.

More information regarding the above approach can be found e.g. in Baarholm et al. (2013).

11.2.2.4 Frequency domain analysis for a short-term period

Frequency domain analysis can be applied if the linear WF actions dominate. The analysis may be carried out to determine wave frequency action effects as a function of wave frequency at a suitable mean condition. The response spectrum obtained together with the assumption of Gaussian process may be used to estimate the response statistics valid for the actual condition.

11.2.2.5 Time-domain analysis for a short-term period

Time-domain solution methods should generally be used in case of significant nonlinear effects. Time-domain analysis is also normally required to determine the transient response after slamming, ringing events, wave in deck, etc.

It is recommended that the surface elevation process is modelled as a second order process with its corresponding second order kinematics. No kinematic reduction factor shall be used.

Time-domain methods should be used for extreme condition analysis, but are normally not required for fatigue analysis or analysis of more moderate conditions where linearized frequency domain analysis may provide sufficient accuracy. However, time-domain analysis of fatigue action effects may be required in connection with local splash zone actions. Deterministic fatigue analyses may be appropriate for jacket type of structures.

Time-domain analysis may be carried out for a limited number of sea-states and generalised to other conditions, e.g. by using an equivalent sea state dependent transfer function. However, such simplifications should be introduced with caution, and in a way that important action effects are conservatively estimated.

Analysis in irregular sea states should normally have a duration 3 hours. A sufficient long start-up period should be included to remove transient effects

For large volume structures, the frequency dependent added mass and wave radiation damping should be included by a convolution formulation.

A wave spectrum is used to generate random time series when simulating irregular wave kinematics. If actions close to the free surface are of importance, second order kinematics, as outlined in 6.2.1.2 should be applied.

The time series resulting from the time-domain analysis of nonlinear systems will generally be non-Gaussian.

A strategy to improve the computational efficiency in time-domain simulation of a complex model exposed to irregular excitation is to simulate critical events (i.e. slam, ringing events) with the refined model for a time duration identified by a simplified approach. A fundamental requirement is that the relevant events shall be captured by the identification procedure. This technique should therefore be applied with care.

The short-term time-domain analysis may be used in conjunction with a smooth joint probability distribution for H_S and T_P as well as other possible parameters.

In order to obtain design values for global action effects in ULS and ALS from long-term analysis, topsides and sub-components of the structure is usually represented in a simplified way in the analysis. Action effects of sub-components should then be determined separately, by refined local models exposed to appropriate design action cases. Design values of global action effects (e.g. shear forces in top of shafts and acceleration of topside) are used as boundary conditions in the local analysis, see also 11.1.10

11.3 Rigid body motion analysis for structures with free modes

11.3.1 Purpose

The purpose of a motion analysis of structures with at least one free mode according to Table 8, is to determine displacements, accelerations, velocities and hydrodynamic pressure relevant for the action on the hull, superstructure, riser and mooring system, as well as relative motions (in free modes) needed to assess requirements related to airgap and green water. Excitation by waves, current and wind should be considered.

11.3.2 Action processes and operational conditions

The response analysis shall be carried out with due regard to combined metocean conditions, e.g. wave, current and wind intensity and direction. Attention should be paid to the fact that current actions may dominate the hydrodynamic actions for some concepts. If joint statistical data for the site or area considered are not available, conservative assumptions shall be made, including consideration of unidirectional versus multidirectional environment.

Design actions and action effects may depend on operational practice. The assumptions made in the analysis regarding operations shall be consistent with relevant instructions and limitations for safe operation. This involves the magnitude and distribution of deck action. For buoyant structures the ballast condition, the floating position (e.g. draught, tilt) and mooring tension, are of concern. For ships, heading direction is also an important issue. The margin to cover the variability of ship heading with respect to the direction of wind, current, wind–waves and swell, should depend on the directional control (active and passive) used.

Possible lack of directional stability of ships or barges that may cause excessive yaw motions ("fish-tail" behaviour) and, hence, roll and heave motions should be noted in connection operational control.

When calculating the action effects due to waves, floating platforms may be assumed to be ballasted to an even keel in a 100 years mean wind condition. This is conditional upon availability of necessary equipment, adequate procedures and sufficient time for correcting the floating condition during change of the metocean condition, i.e. direction and speed of wind.

Active mooring line tension adjustments shall not be considered in evaluation of mooring line tension in ULS design events, see ISO 19901-7.

11.3.3 Static and mean response analysis

Static analyses are carried out to determine

- the static equilibrium position of the platform under gravity and buoyancy actions in still water. A static analysis should be performed for each action condition to be analysed, considering the total platform weight, ballast, buoyancy (displacement), riser and mooring tensions, and hook actions,
- the position of the platform under mean wind force, current force and steady wave drift force, based on the static equilibrium condition,
- overturning moment acting on (floating) platforms with six free modes subject to sustained wind actions in connection with stability check.

The effect of snow and ice shall be taken into account in the prediction of static equilibrium position.

The determination of a mean or equilibrium position due to steady metocean actions is the basis for proceeding with a dynamic analysis. The mean position should be determined with appropriate values of the sustained actions, in view of their correlation with the fluctuating actions in the dynamic analysis.

11.3.4 Dynamic analysis

11.3.4.1 Dynamic model

The dynamic equation of equilibrium is formulated in terms of

- excitation actions,
- inertia actions,
- damping actions,
- restoring actions

relating to the hull, positioning system and risers.

Possible excitation, inertia, damping and restoring actions by the riser and mooring system should normally be included by a coupled dynamic analysis. In cases where these effects can be demonstrated to be of minor importance to the global behaviour of the floater, the effects of these components may be included in a simplified manner and performing an uncoupled analysis considering six degrees of freedom only for the system see 11.3.5.

In dynamic response analyses, excitation actions may conveniently be categorised as:

- wave frequency (WF) actions (affecting all 6 degrees of freedom);
- high frequency (HF) wave actions (affecting restrained modes of motion);
- low frequency (LF) (or slow drift) wave actions (affecting free modes of motion, primarily horizontal motions but for some structures also vertical modes);
- wind gust actions (affecting fixed and restrained modes, e.g. roll, pitch as well as horizontal modes);
- current actions due to vortex shedding;
- transient wave slamming, run-up and ringing actions.

HF or LF wave actions are normally an order of magnitude smaller than wave frequency actions. Their effect may be significant if wave frequency actions are minimised by design, or when they are close to resonance frequencies of the system.

Example of resonance of (lightly damped) modes of motion may for instance include:

- roll of a barge/ship or a spar with a low metacentre height, (free mode);
- heave of a spar buoy (free mode);
- vortex induced motions (VIM) for floating platforms comprising long vertical circular columns. (free modes);
- surge, sway and yaw motion of a catenary moored floating production system (free mode);
- internal U-tube resonances (local dynamic mode);
- ballast or cargo tank sloshing modes (local dynamic mode);
- heave/pitch/roll resonance of TLPs (restrained mode);
- moon pool motion.

The transient response caused by slamming actions would generally be associated with structural dynamic effects, see 11.6.2.

Inertia actions are related to the hull, mooring and riser mass, variable mass (e.g. ballast), and added mass due to of the surrounding water.

Damping has linear contributions from wave radiation and wave drift damping. Viscous damping is normally nonlinear actions associated with the hull, risers, and mooring systems. Assessment of damping is uncertain and shall be conservatively estimated.

The effect of possible thrusters in terms of restoring actions and possible damping should be included. Also, the possible damping effect of internal tanks with free surface shall be included.

Viscous drag actions on slender bodies can be estimated by Morison's formula. The action is generally dependent upon the total, wave frequency and low frequency, relative velocity. The drag coefficient to be used is dependent on if the total combined velocity is used, or if a separate velocity approach is used. Details may be found in DNV-RP-C205.

The current gives rise to damping as well as excitation actions. For resonant motion behaviour it is recommended that analyses are carried out with different values of current velocity, spanning from zero up to design values and that a conservative approach is used in design.

Restoring forces have contributions from the buoyancy effect on the vessel, mooring system and the riser systems. Nonlinear effects in the restoring characteristics of catenary mooring systems should be accounted for. Long-term change of restoring properties of synthetic ropes should be accounted for. Elastic deformations of steel tethers can also normally be neglected in determining the behaviour of free modes of TLPs.

Risers may be represented by dynamic actions as determined by motion analysis for free platform modes or by static forces for constrained platform modes.

11.3.4.2 Time-domain analysis for a short-term period

Time-domain analysis means numerical integration of the equations of motion allowing the inclusion of system nonlinearities. Short-term statistics of action effects induced by a non-Gaussian sea surface model should be determined by time-domain simulations or model tests.

Time-domain solution methods should generally be used in case of significant nonlinear effects. Time-domain analysis is required to determine the transient response after slamming, ringing events, and mooring component failure. The analysis should include the nonlinear drag forces, the effects of actual wave amplitudes, actual motions, and nonlinear restoring effects e.g. related to the station keeping system.

Time-domain methods should be used for extreme condition analysis, but are normally not required for fatigue analysis or analysis of more moderate conditions where linearized analysis may provide sufficient accuracy. However, time-domain analysis of fatigue action effects may be required in connection with local splash zone actions.

Analysis in irregular sea states should normally have a three hours duration. A sufficient long start-up period should be included to remove transient effects. In cases where low frequency response is significant, longer sea state durations or several independent simulations shall be performed to establish a reliable extreme value estimate, see 6.1.3.

For large volume structures the frequency dependent added mass and wave radiation damping should be included by a convolution formulation.

A wave spectrum is used to generate random time series when simulating irregular wave kinematics. If actions close to the free surface are of importance, second order kinematics as outlined in 6.2.1.2 should be applied.

A method to improve the computational efficiency in time-domain simulations of a complex model is to simulate critical events (i.e. slam, ringing events) with a refined model for a time interval identified by a simplified approach. A fundamental requirement of this method is that the relevant events are captured by the identification procedure. Hence, this method shall be applied with due consideration to these concerns.

If fully coupled analyses are not feasible, or for analyses in early design phases, separate wave frequency and low frequency analyses may be performed as outlined in 11.3.5.

11.3.4.3 Frequency domain analysis for a short-term period

Frequency domain analysis can be applied if wave induced actions dominates, and linear wave frequency assumption can be applied. The analysis may be carried out to determine wave frequency motions and mooring actions as a function of wave frequency at a suitable mean condition. The response spectrum obtained together with the assumption of a Gaussian process may be used to estimate the response statistics valid for the actual condition.

11.3.4.4 Low-frequency analysis

A low-frequency dynamic analysis should to be carried out to determine the actions and motions due to combined actions such as:

- second-order wave actions;
- current viscous actions;
- vortex induced motions (VIM) for floating platforms comprising long vertical circular columns. (free modes);
- fluctuating and sustained wind actions;
- nonlinear mooring restoring actions.

All relevant sustained horizontal actions including current, as well as the effect of wave frequency motions are to be included in the analysis.

Frequency range is to include all low-frequency cyclic actions that may excite platform resonant motions.

When estimating damping to be used in low frequency motion analysis due consideration to uncertainty in the damping values shall be included, e.g. in a sensitivity analysis and conservative choices of damping values.

If separate analyses of the low frequency actions are performed, the extreme low frequency wave- and wind-response should be determined with due consideration of wave frequency response as well as the mean metocean actions.

11.3.4.5 Combined extreme wave- and low frequency action effects

If the wave frequency and low frequency action effects are computed independently, the characteristic value, Xc, of the combined wave frequency, X_{WFc}, and total low frequency, X_{LFc}, motions in the same direction depends upon the relative magnitude of low frequency wind- and wave-induced actions, and system characteristics. The wave frequency response, X_{WF}, is determined by using restoring characteristics at a platform location determined by the mean and maximum LF metocean actions. While the WF motions can be well described by a linear model, the dynamic line tension is nonlinear. However, it can be conservatively linearized.

For a linear response with Rayleigh distribution: $X_{WFc} = \sigma_{X_{WF}}\sqrt{\ln N_{WF}}$. The low frequency response can be assumed to follow the exponential distribution: $X_{LFc} = \sigma_{X_{LF}} \ln N_{LF}$, AC where N_{WF} and N_{LF} are the number of wave frequency and low frequency oscillations respectively. AC If the mean action effect X_{mean} is included, the total action effect may be expressed as:

$$X_C = \sqrt{X_{WFc}^2 + X_{LFc}^2 + 2\rho X_{WFc}X_{LFc}} + X_{\text{mean}} \tag{21}$$

where ρ is a correlation coefficient. The correlation coefficient should be determined through time-domain analysis or model test. For horizontal motions of semi-submersibles ρ will typically be between 0,2 and 0,4.

An alternative way of combining WF and LF action effects to obtain the total effect is:

$$X_C = max\left[\left(X_{WF(\text{max})} + X_{LF(\text{sig})}\right); \left(X_{LF(\text{max})} + X_{WF(\text{sig})}\right)\right] + X_{\text{mean}} \tag{22}$$

where the indices "max" and "sig" refer to expected maximum and significant amplitudes, respectively. Equation (22) may be used for motion and mooring tension effects in catenary moored floating platforms. Significant amplitudes are then taken as $X_{LF(\text{sig})} = 2\sigma_{X-LF}$.

11.3.5 Dynamic system analysis for a short-term period

11.3.5.1 General

Hull, risers and station-keeping system is an integrated dynamic system responding to metocean actions from wind, waves and current. This system may be analysed as a decoupled or coupled system. If coupling effects are important, a coupled analysis should be performed.

11.3.5.2 Decoupled analysis

A decoupled analysis is carried out in two steps.

1) In the first step, rigid body floater motions are computed considering static-, low frequency and wave frequency metocean actions. Risers and mooring system are in this analysis represented by the static restoring force characteristics and viscous damping. For the wave frequency analysis, the damping may be linearized. Assessment of the low frequency damping is crucial for the floater motion analysis. Contribution from current action on mooring lines and risers may be represented by a constant external action.
2) In the second step, the action effects in the mooring and risers are computed. The floater motions (and forces in case of TLP tension) computed in first step are applied to the mooring and risers as forced boundary displacements or forces. Wave and current actions on the mooring and risers may also be included. The forced WF motions are to be taken at an offset position determined by the mean offset plus a conservative estimate on the maximum LF motion. A typical example is catenary mooring lines where WF line dynamics can be significantly influenced by the quasi-static LF tension variation. Dynamic effects associated with LF motions should be considered for deep water systems.

The second step is the time consuming part of the decoupled analysis and is normally carried out for critical mooring lines and risers one by one. The computational flexibility contributes to efficient analyses and is the major advantage of the decoupled analysis. After having completed step two, the assumptions used w.r.t. restoring forces and damping in step one should be checked.

11.3.5.3 Coupled analysis

Coupling effects between floater motions and slender structure action effects can be accounted for by including the floater force model in the slender structure model of the complete system including all mooring lines and risers.

The coupled approach requires significant computational efforts. As a compromise, it can be proposed to apply a rather crude slender structure model in the coupled analysis still catching the main coupling effects, e.g. restoring, damping, mass. Detailed slender structure analysis can then be performed as in step two in the decoupled analysis based on the floater motions predicted in the coupled analysis. It can also be suggested to use a rather short coupled simulation for estimation of LF damping form mooring lines and risers. Damping estimate can then be used in a decoupled analysis.

Further guidance on coupled analysis may be found e.g. in DNV-OS-F201.

11.4 Metocean action effects on subsystem structures with free modes

11.4.1 Floater

Action effect analyses shall be carried out for subsystem structures attached to floating structures, e.g. semi-submersibles, ship-shaped floaters or Spar type floaters. Subsystem structures include risers, mooring, dynamic positioning systems, topsides and fluid filled tanks. For floaters with catenary mooring a global analysis is carried out to determine hydrodynamic actions and motions (accelerations) due to actions from waves, wind and current, see 11.3. Slowly varying actions from waves, wind and current shall be combined with wave frequency actions affecting subsystem structures.

Characteristic action effects should be determined based on a stochastic analysis combining short-term and long-term statistics. A conservative design wave approach may be applied for simplified assessment.

Design waves for structures with no restrained modes

Design waves can be used for a simplified assessment of actions effects on floating structures. Maximum action effects for floating structures are normally not caused by waves with extreme heights, but by waves of a defined wavelength and extreme steepness. The wavelength is given by some characteristic dimensions of the floater. The typical action effect(s) where design waves for floating structures should be determined:

- column-stabilised units:
  - maximum split forces between columns;
  - torsional moment about a transverse horizontal axis;
  - longitudinal shear force between columns.
- ship shaped units:
  - sagging moment;
  - hogging moment;
  - torsional moment.

For column-stabilised units, evaluation of responses based on design wave analysis may be performed according to DNV-RP-C103. For ship shaped units, evaluation of responses based on design wave analysis may be performed according to DNV-RP-C102. Guidance for design wave analysis of TLPs is given in DNV-OS-C105.

Design wave approaches used in detailed design need to be calibrated based on stochastic analysis.

A simplified, conservative approach to determine the action effect with annual exceedance probability $q = 0{,}01$ in such situations may be based on a wave with critical period (length) causing for instance maximum splitting forces in floating framework platforms. The corresponding design wave height H_d should be taken as the maximum wave height limited by maximum steepness of a non-breaking wave:
AC

$$H_d = \begin{cases} 0{,}22T^2 & \text{for } T \leq 6 \\ \dfrac{T^2}{4{,}5+0{,}6(T^2-36)/H_{0.01}} & \text{for } T > 6 \end{cases} \qquad (23)$$

AC

where AC $H_{0.01}$ AC is the 0,01 annual probability wave height.

H_d does not need to be taken greater than 1,9 times the significant wave height corresponding to an annual exceedance probability of 0,01, or the wave height with an annual exceedance probability of 0,01 according to the long-term distribution.

Design waves for TLPs

Generally, design wave approach is not recommended for compliant structures. However, for the hull of TLPs the same action effects as given above for columns-stabilized units can be computed using a design wave approach.

11.4.2 Mooring and risers

11.4.2.1 General

Mooring lines and risers are slender structures with similar static and dynamic behaviour. The main difference related to the global behaviour is that risers are influenced by bending stiffness while mooring lines are not. The effect of bending close to the support shall be considered for risers.

In analysis of action effects of mooring and risers due to waves and current, the following nonlinear effects shall all be considered:

– nonlinearities due to hydrodynamic (drag) action and varying wave elevation effects;
– large rotations in 3D space;
– geometric stiffness;
– possible material nonlinearities;
– possible contact between slender structures and hull;
– possible contact between slender structures and seafloor.

The relative importance of these nonlinearities are strongly system and excitation dependent, the first two nonlinearities will, to some extent, always be present whilst the latter ones are more system specific nonlinear effects. Material nonlinearities will for example normally not be relevant for metallic tensioned risers while it is most important for non-bonded flexible pipes and synthetic mooring lines.

A finite element (FE) approach is normally applied using beam and bar elements for efficient modelling and analysis. Beam elements should be used for riser modelling. The response should be computed by a time-domain approach. In some cases a frequency domain approach may be justified in a simplified assessment, i.e. when the hydrodynamic action is the major nonlinear contribution. Inertia, damping and stiffness may then be linearized at the static mean position while including the nonlinear hydrodynamic action according to the Morison's formula. Such a simplified approach shall always be checked with a fully nonlinear time-domain analysis.

11.4.2.2 Tension-leg mooring system

Analysis of TLP tendon tension should be carried out in a time-domain integrated analysis, including modelling of hull, tendons and risers. For simplified assessment a linear frequency domain approach may also be applied. Static tension is determined by pretension, tendon weight, load and ballast conditions, tidal effects, storm surge, overturning moment from wind and current forces, and set-down due to slowly varying offset (from wind, wave and current). Dynamic tension arises from hull motions, direct hydrodynamic action on the tendons and possible ground motion. The extreme dynamic tension may have contributions from wave frequency (WF) actions, low-frequency (LF) wave actions and high-frequency (HF) wave actions as well as from hull VIM and tendon VIV. Effects due to individual tendon load sharing differentials should also be considered when assessing maximum and minimum tendon tension. One of a group of tendons may carry a greater share of the tendon load due to foundation mispositioning and anchor template rotational tolerances.

Requirements to maximum and minimum tendon tension are given in DNVGL-OS-C105.

Usually the major contribution to fatigue of the tendons is from linear wave actions (or WF wave actions). However, fatigue due to springing shall also be taken into account. Combined action effect due to wave frequency, high frequency and low frequency actions shall be considered in fatigue analysis. VIV shall be considered and taken into account in fatigue assessment. This applies to operation as well as non-operational (e.g. tendon free standing) phases.

Tendon failure may have substantial consequences and therefore the tendons shall be designed with sufficient safety margin.

11.4.2.3 Catenary mooring system

The characteristic values of actions and the safety factors applied shall be in accordance with NMA Regulations no. 998 §14 concerning anchoring and positioning on mobile offshore units.

Analysis of catenary mooring systems can be carried out according to the technical requirements in the ISO 19901-7 or DNV-OS-E301.

11.4.2.4 Thrusters and dynamic positioning

Thrusters may be used to assist the mooring system by reducing the mean environmental forces, heading control and damping of low frequency motions or a combination of these functions.

The characteristic values of actions and the safety factors applied shall be in accordance with NMA Regulations no. 998.

More details on thruster assisted mooring is given in DNV-OS-E301.

11.4.2.5 Risers and riser systems

Action effect analyses for riser systems shall consider the following actions:

- metocean actions on risers from waves and current;
- actions due to floater motions induced by wind, waves and current;
- possible actions from internal waves or other actions due to differences in water density (may be relevant for risers with buoyancy elements in deep water);
- possible actions from ice and seismic actions;
- external and internal pressure actions;
- functional actions (e.g. weight, applied tension, thermal).

Analysis of metallic risers should be in accordance with DNV-OS-F201.

Analysis of flexible risers should be in accordance with API 17J and API RP 17B.

The riser analysis shall include relevant ULS and ALS actions as defined for the facility the risers are connected to. This also includes relevant actions from the mooring system.

Effects due to accidental actions on riser systems shall include the following: Fires and explosions, impact actions (e.g. riser interference, impact from dropped objects or from floating objects), failure of support system, overpressure, unlikely environmental actions (earthquake, tsunamis, icebergs). Guidelines on riser interference are given in DNV-RP-F203.

When assessing riser fatigue due consideration shall be taken to the effect of vortex induced vibrations (VIV) of riser sections, including the effect of buoyancy elements. For marine risers with kill and choke lines (piggy back) the possibility of galloping motion shall be considered. Detailed guidelines on riser fatigue are given in DNV-RP-F204.

11.4.3 Effect of fluid motion in tanks

Wave induced floater motions will generate motions of fluid in tanks. Depending upon the size of the tanks, the amount of fluid in the tanks and the motions of the floater at the resonant oscillation period of the fluid in the tank, a dynamic amplification of static pressure occurs, possibly combined with local impact actions. These sloshing actions may be significant from an ultimate strength and a fatigue standpoint.

A sloshing analysis shall be carried out in accordance with recognised calculation procedures, possibly combined with model tests. For ships, the simplified methods given in the rules of recognised classification societies may be applied.

11.5 Topsides

Topsides are normally of two types: integrated decks and modular decks. Generally, a global integrated analysis including the topside should be performed in order to determine realistic action effects both on topside and sub-structure. The action effects on a topside will be caused by the direct actions on the topside, inertia effects due to motions and actions imposed by deformations of the support structure. Integrated decks will experience larger actions imposed by deformation of the support structure compared to modular decks.

Action effects on topside shall at least include:

- actions effects directly on the topside structures, e.g variable functional actions, permanent actions, wind actions (including vortex induced vibrations), run-up, icing, snow etc. Insufficient air gap may also lead to actions from wave impact;
- action effects from imposed deformations;
- inertia action effects from wave and current actions on substructure;
- accidental action effects from waves reaching the topside when relevant, earthquakes, and other accidental actions in accordance with Clause 9.

Hence, all wave and current actions on both the supporting structure and the topsides shall be included in the analysis of the metocean action effects on the topsides.

Actions due to imposed deformations shall be considered.

- For topsides structure supported by a multi-column gravity base structure, the motions of the column tops can result in significant indirect actions applied to the topsides structure (wave actions can act in differing directions on different legs). Hence, the support structure and the topsides structure should normally be analysed together for both ULS/ALS and FLS.
- Sagging and hogging motion of a monohull will introduce deformations at the topsides structure level.

Inertia action effects from wave and current on actions on substructure for floating platforms shall include the distortions of the supporting structures (from wave induced accelerations) and the consequent effects on the support points of the topsides structure.

Significant actions can result from sea water reaching the topside. This may occur due to insufficient air gap, wave run-up and slamming against large diameter legs and columns, or when water inundates the deck for floating structures (green water effects). All actions resulting from the water flow including buoyancy, inertia, drag and slam shall be taken into account.

The topsides structure shall be considered for actions from earthquake (see ISO 19901-2).

In areas where low temperatures can occur, special attention should be given to the effect of icing due to sea spray on topsides structures on floaters.

More details on action effect analyses on topsides are given in ISO 19901-3.

11.5.1 Helideck

Design of helicopter decks shall be in accordance with NORSOK C-004, DNVGL-OS-E401 and DNV-RP-C205. However, the design wind action shall be based on the 3 sec gust wind speeds based on 1-hour average wind speeds given in 6.4.2.

In addition, vortex induced vibration, inertia effects, snow and icing, etc. should be checked helideck support structure.

11.6 Metocean action effects applicable for all types of structures

11.6.1 High frequency action effect analysis

11.6.1.1 General

By modelling the wave action as a superposition of actions from linear wave components, the most important wave action effects may be obtained. However, nonlinearities in the waves and nonlinear interactions between waves and structure may introduce important higher order action effects that may excite the structure at frequencies both below and above the frequency range of the linear wave components. The low-frequency action effects are covered in 11.3. High frequency wave action effects are relevant for restrained modes and local dynamic action effects.

The significant uncertainties in current theoretical methods for predicting high frequency wave actions make it necessary to combine theoretical and experimental methods in order to determine characteristic action effects for design.

11.6.1.2 Ringing

Ringing is a nonlinear transient action that causes a transient action effect. Ringing may occur when steep, high waves encounter vertical components of structures with natural periods in the range from about 2 s up to AC 8 s AC. Ringing is relevant for structures with vertical elongated cylindrical structures crossing the splash zone and is primarily an effect caused by pressure gradients due to potential flow wave diffraction. Ringing is relevant for TLPs and GBSs in ULS and ALS conditions. Modelling of ringing actions is covered in 6.3.5.2.

Ringing effects are of significance only in combination with extreme first order wave frequency effects and ringing should be evaluated in the time-domain with due consideration of first order wave action effects. The magnitude of the first response cycles is governed by the magnitude of the wave force and its duration relative to the resonance period. The ringing response builds up over a few resonant cycles before it reaches its maximum. The decay may be slower than experienced in a free decay. This is due to a continued excitation after the maximum response is reached. The first ringing response cycles are not sensitive to the damping. Perturbation methods for prediction of ringing response should include wave action effects at least up to third order in wave steepness.

Ringing response is primarily of importance for ultimate and accidental limit states. Particular care is needed to establish the high and steep sea states that cause ringing.

For slender vertical columns where drag induced wave action is dominant, resonant response may be excited by second and higher order wave actions from integrating the horizontal drag forces up to the actual instantaneous free surface level. This may cause an action effect that resembles ringing response.

Ringing action effects shall be combined with low-frequency action effects, by methods used for combining wave- and low-frequency action effects, see 11.3. This is in particular relevant for TLPs with large horizontal offset.

11.6.1.3 Springing and whipping

Springing is caused by a stationary nonlinear wave action. The springing action may excite response at the natural frequency of restrained modes. It is particularly relevant for TLPs (vertical/axial tether resonance), ships (bending of hull beam) and GBS (bending of shaft). The springing action effects may be established by considering second order potential theory wave actions. Details on assessing the springing action effects may be found in DNV-RP-C205.

Whipping is induced by actions due to bottom slamming or bow flare actions on ships and should be combined with the continuous wave actions by properly accounting for the phase difference of the two actions.

11.6.2 Slamming

11.6.2.1 General

Slamming may occur if steep waves hit vertical or inclined structural members, or if waves hit underneath the deck. If slamming is likely to occur, calculation or model tests shall be performed to determine the corresponding actions, see 6.3.5.3.

The hydrodynamic action effect is obtained by considering the slamming pressure integrated over a relevant area, i.e. the area between structural members, stiffeners, or beams.

11.6.2.2 Action effects due to slamming in ULS and ALS

Both local and global action effects due to slamming in ULS and ALS shall be considered. Slamming effects should be calculated by applying an appropriate transient dynamic structural model.

Local action effects

In cases where a flat part of the structure is hit by the wave surface with a small relative angle, very high pressures may occur for a small area. Local actions from slamming are characterized by very short rise-time, high pressure acting on small area and short duration. Due to the very short rise-time of the action, inertia forces in the structure surrounding the load area may be important to carry the impact action. Typical failure modes to be checked are e.g. rupture of plating between stiffeners for a steel structure or shear failure in a concrete wall. Hydro-elastic effects can be important for local impact actions.

Action effects in local modes of the structure.

The spatial distribution of pressure during a slam is known to change rapidly as a function of time. The integrated pressure distribution may excite local modes in the structure. Depending on duration and spatial distribution of the slamming pressure relative to the mode shape and it's natural period, the action effect may be reduced (short impact or cancellation due to nodal response during impact) or amplified (dynamic amplification) relative to the static response. Hence, a dynamic analysis should be carried out, and sensitivity with respect to distribution of pressure as a function of time and space should be carried out. Added mass and contribution to the modal response and hydro-elastic effects in the loading may be important. Typical failure modes to be checked are yielding of frames or stiffeners for a steel structure and cracking/crushing due to bending moments in a pre-tensioned concrete wall.

Global action effects caused by slamming.

Relative to the global modes of a structure, slamming actions will typically have high rise time and short duration.

Examples of global action effects caused by slamming:

- Slamming on single shaft GBS can cause global response in first bending mode.
- Bow or bottom slamming on ships can cause whipping.
- Bow slamming on FPSOs in group of extreme waves can enhance low frequency motions.

When assessing global action effects, the following should be considered:

- global load balanced by stiffness or inertia of surrounding structure (e.g. shear towards deck in top of vertical column exposed to impact);
- transient response in global modes (e.g. OTM, BS, inertia loads in topside for GBS, whipping response in hull, etc.);
- dynamic amplification due to repeated impact events (e.g. low frequency motion for moored systems, repeated impacts exciting global structural modes of a structure, etc.).

When assessing global slamming effects due consideration should be made to the relative phasing between the slamming action and the global wave actions.

Slamming is of highly nonlinear and stochastic nature, hence the slamming action effect with annual probability q may occur in sea states with higher probability of occurrence. In special cases model tests of slamming actions for a large number of sea states combined with a long-term analysis may be needed to estimate the characteristic ULS and ALS slamming action effects.

11.6.2.3 Action effects due to slamming in FLS

The slamming action effect on jacket braces in the splash-zone shall be taken into account together with the repetitive action effect due to change of buoyancy when braces cross the free surface. The method of Ridley (1982) can be used to estimate fatigue damage of inclined slender structures in the splash-zone, see 6.3.5.3.

11.6.3 Air gap and wave in deck analysis

11.6.3.1 General

A minimum requirement is a positive airgap above the wave crest with an annual probability of exceedance of 10^{-2}. Due to the complexity and uncertainty associated with determining actions associated with waves hitting the platform decks; designing for a positive airgap above the wave crest with an annual probability of exceedance of 10^{-4} (approximately an increase of 30 % on the wave crest with an annual probability of exceedance of 10^{-2}) is recommended.

Air gap considerations are relevant for deck and topside structures on e.g. fixed platforms (jacket, jack-ups and GBS) and on small water-plane area floating platforms (semi-submersibles and TLPs). It is also relevant for topside structures located on ship-shaped platforms and on twin hull and catamaran ships. For monohull ships freeboard requirements relating to the hull should be adhered to, see also 11.6.4.

When assessing air gap the following effects shall, when relevant, be considered:

- water-level including storm surges, astronomical tides, settlement and subsidence of seabed;
- maximum and minimum operating draughts;
- static mean offset and heel angles;
- wave crest height including wave/structure interaction effects, i.e. wave enhancement;
- wave frequency and low-frequency motions (in all six degrees of freedom);
- combination of different wave action effect (e.g. low frequency vertical motions for semisubmersibles, and low-frequency set-down for TLPs);
- effects on floater motion of interacting systems (e.g. mooring and riser systems);
- spatial statistics to account for finite deck area (different from point statistics);
- effect of climate change on water-level and wave crest height.

The deck structure adjacent to platform columns needs to be designed to resist the possible pressure actions due to run-up along columns.

Local wave impact due to effects not accounted for in the above requirement to positive airgap, may be permitted to occur on any part of a deck structure provided that it can be demonstrated that such actions are properly accounted for in the design. It should, however, be ensured that the water does not threaten personnel's life, or damage pipes and other equipment which may lead to environmental damage. This means that ULS and ALS strength requirements should be fulfilled for events with annual probabilities of 10^{-2} and 10^{-4}, respectively. The characteristic actions for such design checks then need to be determined.

Unless it is demonstrated that the air gap is sufficient to avoid wave in deck impact, relevant deck impact analysis shall be performed. Wave in deck action effects should be evaluated in the time-domain with due consideration of extreme wave action effects. Since air gap and wave in deck analysis are subjected to significant uncertainties, analysis methods should be validated by high quality model tests with due consideration to the large inherent stochastic variability and uncertainty in predicting deck impact actions.

11.6.3.2 Airgap and wave in deck analysis for fixed structures

For bottom fixed structures, where diffraction effects may be ignored, it is important to recognize that airgap based on the Forristall 2nd order wave crest height distribution and combined with storm surge and tide, may be non-conservative with respect to wave in deck impacts. From recent industry research work there are clear indications that higher order wave effects (above 2nd order) may occur, Buchner et al. (2011). Furthermore, especially for local design, it may be relevant to apply spatial statistics when estimating extreme ($q = 10^{-2}$ or 10^{-4}) crest height C_q, accounting for dimensions of the platform like the width of the deck transverse to the wave direction, or the area under the deck, so called area statistics, Forristall AC (2006, 2007, 2011, 2015). AC Because of this, it is strongly recommended to design bottom fixed structures with a still water airgap at least 1.1 times the 2nd order crest height plus the combined tide and storm surge as given in Table 7. Alternatively, the higher order wave effects and spatial statistics will have to be evaluated in detail, or a simplified wave-in-deck procedure as that given in AC A.19 AC will have to be implemented.

A detailed wave-in-deck evaluation should, (in addition to the mentioned higher order and area effects), consider different wave steepness (wave periods), skewed/near breaking and breaking waves. Possible diffraction effects from transparent substructures (e.g. jacket) causing locally enhanced wave crests should be considered.

High quality model tests are recommended to validate the airgap analysis.

The enhanced action effect on the substructures below deck due to disturbed kinematics from wave impact on deck shall be considered. Depending on the underneath deck geometry the fluid particle velocities in the incoming wave may be considerably increased due to wave in deck impact.

Possible diffraction effects from transparent substructures (e.g. jacket) causing locally enhanced wave crests shall be considered.

For large volume fixed structures wave diffraction effects shall be included in the airgap analysis. For such structures higher order diffraction e.g. due to caisson effects may be important.

11.6.3.3 Local wave impact

Local wave impact above maximum crest due to effects not accounted for in the above requirement to positive airgap, may be permitted to occur on any part of a deck structure provided that it can be demonstrated that such actions are properly accounted for in the design. It should, however, be ensured that the water does not threaten personnel's life, or damage pipes and other equipment which may lead to environmental damage. This means that ULS and ALS strength requirements should be fulfilled for events with annual probabilities of AC 10^{-2} and 10^{-4} AC, respectively. The characteristic actions for such design checks then need to be determined.

The deck structure adjacent to platform columns shall be designed to resist the possible pressure actions due to run-up along columns.

For an estimate of the structural capacity of equipment located above the global wave crest, guidance is given in AC A.20. AC

11.6.3.4 Airgap and wave in deck analysis for floating structures

For floating structures, the wave in deck action effects with annual probability $q=10^{-2}$ and $q=10^{-4}$ should be assessed. In wave in deck analysis of large volume floating structures, asymmetry in incoming waves and higher order diffraction effects may be accounted for by using a factor on the first order disturbed wave process (i.e. incoming, diffracted and radiated wave), excluding run-up areas closer than 0.2 times diameter from vertical columns. For further guidance on calculation of airgap on column-stabilized units, see DNVGL-OTG-13. Analysis of airgap shall include wave frequency contributions, low frequency contributions (from waves and wind) and mean contributions to account for a possible mean inclination of the floater during operation and uncertainty in ballasting. Wave frequency and low frequency contributions can be assumed uncorrelated. The analysis results should be validated by model testing.

Wave run-up factors derived from model tests should be used to account for local run-up close to columns. Analysis accounting for nonlinear wave- structure interaction can be applied if it is documented to yield reliable predictions. Dynamic analysis of large volume floating structures is described in 11.3.4.

If the airgap assessments show negative air gap, both vertical and horizontal wave actions (i.e. due to negative airgap on the outside of the deckbox) shall be assessed. Guidance on estimating horizontal actions on deck box of column-stabilized units is given in DNVGL-OTG-14.

11.6.4 Green water

Green water is defined as solid water entering the deck of e.g. ship shaped structures, and occur when the wave elevation exceeds the ship freeboard, see 6.3.3.4. Areas occupied by personnel, or where safety-related equipment is located, shall not be exposed to waves with an annual probability greater than 10^{-2}. The freeboard exceedance shall be assessed by calculations or model tests. Actions from green water shall be assessed by model tests or validated calculations. Green water typically induces pressure actions and local slamming actions on the exposed structures. The influence of green water on the global motions of a floating facility shall be evaluated.

Water entry in hull and closed appurtenances may influence stability, and shall be evaluated.

For ships, relative wave elevation exceeding the freeboard can be determined using potential theory by computing the relative motion along the shipside. It is recommended to design ships with sufficient freeboard to avoid significant amounts of green water.

Determining the amount of green water entering onto deck, and computing flow velocities and resulting action effects on deck structures is more complicated. Analyses using advanced nonlinear methods and/or model tests are recommended for design purposes or for detailed assessment of existing structures.

For early design phases or for a first assessment of an existing structure, green water actions in the bow can be determined using the semi-empirical method described in Buchner (2003). The method uses linear computations of relative motion of the ship as input. The height and velocity of the water flowing on deck is determined by a modified dam-breaking model. An empirical coefficient dependent on the bow flare type and angle is used.

In lack of more accurate information in early design phases, deck structural members could be designed based on the actions given in NORSOK N-004.

The effect of wave slamming, run-up or green water actions should be appropriately combined with the other wave action effects.

11.6.5 Analysis of action effects due to vortex shedding

11.6.5.1 General

Action effects due to vortex shedding on slender elements shall be taken into account, e.g. in terms of:

- vortex-induced vibration (VIV);
- vortex-induced motions (VIM);
- instability (galloping) caused by rotation of the structure relative to wind or current direction;
- wake induced instabilities of a structure in the vicinity of another structure.

Vortex shedding may be of significant importance in the design of slender structures that may respond in resonance modes to this cyclic action, especially when the damping is small.

Frequencies of flow induced vibrations of structural members are usually high and may lead to a critical reduction in fatigue life due to the large number of stress cycles. The cumulative effects of resonant actions during construction, transportation and operation shall be included in the calculations. Both in-line and cross-flow vibrations should be considered.

Flow induced vibrations will lead to increased mean drag. Unless more accurate data are available, the drag coefficient valid for a stationary cylinder should be multiplied with a factor $1+2 \cdot A/D$ where A is the transverse motion amplitude and D is the diameter of the member.

Flow induced vibrations may lead to increased impact velocity of slender structures in close proximity (e.g. riser bundles).

VIV and VIM may be reduced by introducing devices to prevent or reduce the vortex intensity, or by changing the vibration properties, e.g. natural period and damping of the structure.

11.6.5.2 Vortex induced vibrations due to current and waves

Assessment of effects from possible vortex induced vibrations (VIV) shall be made for submerged long slender elements exposed to actions from current and waves. This includes mooring, tethers, risers, conductors, umbilicals and flowlines, pipelines, and subsea modules.

Critical flow velocities for onset of VIV for current and wave induced vortex shedding are given in DNV-RP-C205. Both current velocity and particle velocities in waves shall be considered.

The excitation may be characterised by the motion amplitude of the slender element and/or the fluctuating actions on the member. Vortex shedding of larger parts of the structure due to current or wave excitation should also be considered.

Possible unstable motions (galloping) due to vortex shedding on slender elements with non-circular cross-sections or tandem slender elements (piggyback) should be taken into account.

11.6.5.3 Vortex induced motions due to current

Vortex shedding may introduce cross-flow and in-line motions (VIM) of floaters with large vertical circular cylinders, such as Spar platforms and other deep draught floaters. Effects of floater VIM shall be considered for mooring system design as well as the riser design; both for extreme loading and fatigue, for more details see DNV-OS-E301. Assessment of VIM shall be done by model testing.

11.6.5.4 Vortex-induced vibrations due to wind

Avoidance criteria for onset of vibrations due to wind induced vortex shedding on individual elements may be based on recommendation in DNV-RP-C205. Supporting documentation on vortex-induced vibrations of circular cylinders due to wind is found in Oppen and Kvitrud (1995) and Sjursen (1999). Vortex shedding may occur on slender members with arbitrary cross-sections (NS-EN 1991-1-4). The method proposed in NS-EN 1991-1-4 (Approach E.1.5.2) may be applied to estimate the amplitude of vortex induced vibrations.

Vortex induced vibrations of frame structures or several members vibrating in phase should also be considered. The material- and structural damping of individual elements in welded steel structures should not be set higher than 0,15 % of critical damping when vortex induced vibrations are considered.

11.7 Seismic action effects

11.7.1 General

Seismic action effect analysis is primarily relevant for structures with only restrained modes, such as bottom fixed structures.

11.7.2 Action effects by response spectrum approach

The effect of earthquake actions for strength check of a soil-structure system may be obtained by a linear elastic model and use of modal superposition. An appropriate number of relevant modes to represent the action effect in question shall be included. Particular care is required to model local effects, i.e.

subassemblies. The maximum of a given action effect in each mode can be obtained using the modal amplitude of the action effect and response spectrum in Figure 8, corresponding to the relevant modal period, T, and damping value, ζ. The damping in modes which do not involve soil deformation, or hysteric losses due to cracking or plasticity in the structure, should be carefully estimated because it may be less than the reference value of 5 %.

Deck, appurtenances, derricks, flare booms, critical piping and equipment need special consideration of global platform dynamics as well as possible local dynamic amplification.

An appropriate method should be used for combining modal action effects for different directions. The CQC method may be used for combining modal action effects in a given direction and the SRSS may be used for combining the directional action effects. See i.e. Der Kiureghian (1980).

11.7.3 Global nonlinear strength analysis

If strength criteria are not fulfilled by action effects determined according to 11.7.2, it is to be demonstrated that the global structure-foundation system remains stable, without excessive deformations during the 10^{-4} earthquake, by taking into account system redundancy and force redistribution by inelastic deformations. A simplified approach based on a static nonlinear, or a dynamic nonlinear analysis may be used for this purpose.

In the simplified approach the action factor (of 1,3) for strength check is reduced depending on ductility and residual strength capability of the structure-foundation system demonstrated by a static nonlinear pushover analysis.

Direct nonlinear dynamic analysis should be carried out in the time-domain, considering three standardised time histories, and using adequate models of the structure, foundation and soil as well as the surrounding water.

Other effects of earthquakes

Consideration should be given to the question whether earthquakes in the relevant area could have other effects, such as:

a) landslide;
b) critical pore pressure build-up in soil;
c) major soil deformations with subsequent deformations of foundation slabs, piles, skirts and pipes;
d) low frequency waves in water;
e) acoustic wave effects on submerged, non-flooded structural parts;
f) tsunamis.

Annex A
(informative)
Commentary

A.1 Overview of document

This Annex provides additional guidance and background to selected clauses of this NORSOK standard. Table A.1 gives an overview of the document, and where to find requirements for different action types.

Table A.1 – Overview of document with references to clauses

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Table A.1 (*continued*)

Accidental action type	Modelling	Actions	Characteristic values of action effects	Action effects
Other	6.9.1 Marine growth 6.9.2 Water levels 6.9.3 Appurtenances			
Earthquake	7	7.3 Response spectrum	11.1.3.4	11.7
Deformation actions	8 8.2 Temperature maps (air and sea)	8.2 Temperature actions 8.3 Fabrication act 8.4 Settlement		
Fire	9.2.2 9.2.4 Fire & expl.			
Explosion	9.2.3 9.2.4 Fire & expl.	9.2.3 Explosion pressure		
Impact	9.3.3 Dropped object	9.2.3 Action values for visiting vessels, Ref to Z-013 for passing, values for shuttle tankers 9.3.4 Ref. FOR for helicopter impact		11.5.1 Helideck
Gas blow out	9.4			
Loss of heading control	9.5			
Abnormal variable actions	9.6			
Floating structures in damaged condition	9.7			
Failure of DP and station keeping	9.8			

A.2 Comm. 6.1.2 Possible consequences of climate changes

Future metocean conditions can be predicted for different climate scenarios by running global climate models in combination with regional downscaling and numerical wave models. However, future wave, wind and sea level conditions are predicted with considerable uncertainty.

The possible change in metocean conditions due to climate change should be taken into account in design of new structures. The uncertainty in the predictions of the future metocean conditions should also be considered. This uncertainty can be modelled and included in a probabilistic analysis to obtain the characteristic values of action effects.

The NOU report “Klima i Norge 2100” (Hanssen-Bauer et al., 2009) gives a description of the expected change in climate in Norway and surrounding waters through the 21st century.

The climate models suggest an increase in the frequency of higher wind speeds, AC⟩ ⟨AC Hanssen-Bauer et al. (2009). However, the accuracy in the predictions are at present not sufficient to quantify consequence on the extreme wind speed prediction. A review of new literature on the topic is recommended to more specifically determine such an increase extreme wind speed.

The climate models predict an increase of about 6–8 % in extreme significant wave heights in the Eastern North Sea and Skagerrak through the 21st century, see Debernard and Røed (2008). A slight increase in extreme wave heights is expected in the Barents Sea, due to possible increased fetch for waves from North. Otherwise little change in extreme wave heights is expected in Norwegian waters.

An increase in water level may be due to climatic effects; e.g. thermal expansion of the oceans and melting of glaciers. The sea level rise through the 21st century is expected to be 0,4-0,7 m in Norwegian waters. The present rate of sea level rise is about 3 mm/year.

The estimate of extreme crest height needs to consider both increase of extreme significant wave height and increase in water level.

In lack of more detailed documentation the following increase in metocean values 50 years ahead may be used:

- extreme significant wave heights: 4 % on q-probability values;
- extreme wind speeds: 4 % on q-probability values;
- sea level: 0,25 m.

A.3 Comm. 6.1.3.2 Definition of characteristic actions – ULS and ALS limit states

Characteristic metocean actions are defined by the annual probability of being exceeded, i.e. 10^{-2} for ULS and 10^{-4} for ALS. The annual exceedance probability is found as the weighted sum of the exceedance probability in each possible metocean condition. The weights are the probabilities of the various metocean conditions.

In practise neither the conditional distribution of the action (or action effect) given the metocean parameters nor the joint distribution of the involved metocean parameters are perfectly known. Uncertainties are both related to the choice of probabilistic models for the various variables (model uncertainties) and to the fitting of these models to available data (statistical uncertainties). These types of uncertainties are often referred to as epistemic uncertainties. In contrast to inherent uncertainties modelled by the fitted distribution functions above, epistemic uncertainties can be reduced by collecting more good quality data and/or by increasing the knowledge of the physics involved in transforming metocean conditions to actions and action effects of the problem under consideration.

The consequences of epistemic uncertainties are normally assumed to be covered by the rule defined safety factors or conservative choices made during the calculation of characteristic actions and action effects. However, when dealing with strongly nonlinear problems, in particular if they are of an on-off nature, one should indicate the magnitude of the consequences of the epistemic uncertainties on the estimated characteristic actions or action effects. Regarding model uncertainties one could select a set of distribution functions to check the sensitivity to choice of model. Bootstrapping can be used for indicating the magnitude of statistical uncertainties. If effects of epistemic uncertainties on estimated characteristic actions are comparable to or larger than the rule defined safety factors proper actions should be taken.

A.4 Comm. 6.1.3.3 Estimation of characteristic actions

All short-term conditions approach

This approach is also called the initial distribution method or all sea state approach when applied to wave conditions. By this approach all possible combinations of the metocean characteristics (or a subset of these combinations) are considered. Target action variable can be the global maximum action (largest action between adjacent zero-up-crossings) or the 3-hour maximum action. This approach is a convenient approach, provided a joint probabilistic model of the selected metocean characteristics is available for all directional sectors, see 6.2.1.6. The only challenge is to establish the conditional distribution function for the target variable given the selected characteristics.

In an ideal case where there are no limitations regarding available data, this approach is likely to give results some few percent on the safe side if correlation among neighbouring metocean characteristics is not accounted for when establishing target exceedance probability corresponding to a given return period.

The advantage with this approach is that it is easy to execute, in particular for linear problems. Models for the long-term variability of slowly varying characteristics are often available in the metocean design reports. The approach is not too sensitive to possible "outliers" in view of the length of time period with data. Outliers in this context is a very severe storm event with an expected return period much larger than the length of the data series.

A disadvantage is that the approach is not convenient if more than 2-3 metocean characteristics in addition to direction are important. It is difficult to include effects of time and weather dependent operational actions taken on board the structure, e.g. varying draft and heading. If model testing is required in order to describe the short-term variability given the metocean characteristic, the method is inconvenient.

The characteristics of an arbitrary short-term stationary metocean condition are of a random nature described by a joint probability density function $f_{Z_i}(z_i)$. The Z_i represent the slowly varying metocean parameters relevant for the problem under consideration, e.g. significant wave height, spectral peak period and mean wind speed. The probability density function should be established by fitting a joint distribution to data from a time span of several decades. The fitted distribution should be valid also outside available observations such that the effect of non-observed short-term conditions is properly accounted for.

Let the largest maximum between zero-up-crossings of the process under consideration be denoted ⟦AC⟩ X ⟨AC⟧. The cumulative distribution of this quantity within a short-term metocean condition is referred to as $F_{X|Z_i}(x|z_i)$. Including all short-term sea states, the long-term distribution of ⟦AC⟩ X ⟨AC⟧ is given by

$$F_X(x) = \int_{z_i} w(z_i) F_{X|Z_i}(x|z_i)\, f_{Z_i}(z_i)\, dz_i \tag{24}$$

$w(z_i) = \frac{\vartheta_0^+(z_i)}{\overline{\vartheta_0^+}}$ is the weighting function where $\vartheta_0^+(z_i)$ is the zero-up-crossing frequency of the process and $\overline{\vartheta_0^+}$ is the long-term average zero-up-crossing frequency. The target action variable x_q with annual probability of exceedance q is estimated by solving the Equation:

$$F_X(x_q) = 1 - \frac{q}{n_x} \tag{25}$$

where n_x is the expected number of zero-up-crossing cycles in one year ⟦AC⟩ ⟨AC⟧.

The distribution given by Equation (24) can also be used for stochastic fatigue analysis by letting X represent the stress width.

For extremes, one should rather use the 3-hour extreme value ⟦AC⟩ X_{3h} ⟨AC⟧ as the basic variable. In the long-term integral in Equation (24) ⟦AC⟩ X ⟨AC⟧is replaced by ⟦AC⟩ X_{3h} ⟨AC⟧ and the weighting factor w is equal to 1,0 since there is only one event per 3-hour period for all short-term conditions.

$$F_{X_{3h}}(x) = \int_{z_i} F_{X_{3h}|Z_i}(x|z_i)\, f_{Z_i}(z_i)\, dz_i \tag{26}$$

The target action variable x_q with annual probability of exceedance q is estimated by solving the Equation:

$$F_{X_{3h}}(x_q) = 1 - \frac{q}{n_{x_{3h}}} \tag{27}$$

where $n_{x_{3h}}$ is the number of 3 hour periods in one year year (approximately 2920).

A disadvantage of this approach is that sampled values of x are closely spaced in time, i.e. adjacent values are correlated, primarily caused by the correlation implicit in adjacent metocean characteristics. The distribution functions estimated using correlated data are fine. The bias is introduced when the expected number of realizations of the action variable per year is used for defining the target exceedance probability. This bias, however, is a conservative bias, i.e. there is a tendency of estimating values slightly on the safe side by this approach.

Establishing a joint probability density function $f_{Z_i}(z_i)$ for more than three metocean variables is a major challenge. If more than three variables are of importance, a long-term analysis using the all short--term conditions approach should rather be done in time domain. Long-term metocean conditions are then represented by time series of simultaneous metocean variables for each 3-hour short-term condition. The conditional probability of non-exceedance should be calculated for all 3-hour conditions from the metocean time series. Using this approach, effects of non-observed events are not accounted for. One should not estimate extreme values corresponding to annual exceedance probabilities less than $q = 1/(2T)$ where T is duration in years of the measurements.

If important aspects of the problem under consideration are time dependent, the all short-term conditions approach as introduced in Equation (24) is not adequate. Examples of such cases can be varying vessel draft or operational change of heading. If such cases are of importance for the action or action effect under consideration, the long-term analysis can be done in time domain, i.e. a short-term analysis is done for all adjacent 3-hour conditions. The aim is to establish the distribution function for the 3-hour extreme value for all 3-hour events of the database, e.g. NORA10 hindcast metocean data together with possible simultaneous histories of operational conditions or crew induced actions. From the 3-hour extreme value distribution, a possible realization of the 3-hour extreme value is generated for each short-term condition. The long-term distribution of the 3-hour maximum is then obtained by fitting a distribution to the simulated database. Since effects of non-observed 3-hour metocean conditions are not accounted for, one should be careful when extrapolating to lower annual target probabilities than indicated in the previous paragraph.

A disadvantage of the all short-term conditions approach is that sampled values of X or X_{3h} are closely spaced in time, i.e. adjacent values are correlated, primarily caused by the correlation implicit in adjacent metocean conditions. The distribution functions estimated using correlated data are fine. Bias is introduced when the expected number of realizations of the action variable per year is used for defining the target exceedance probability. This bias, however, is a conservative bias, i.e. there is a tendency of estimating values slightly on the safe side by this approach.

If the problem under consideration is of a complex nature it will be time consuming to use the approach indicated in the paragraph above. A better approach may be to limit the analysis to include only the severe metocean conditions, i.e. a peak-over-threshold approach.

Storm event approach

This approach is also called the Peak over Threshold (POT) method. In this approach, only “storm” events are considered, see also 6.2.1.6. This may be a reasonable approach if the purpose of the analysis is to estimate q-probability extremes. In an ideal case with an infinite number of good quality data, it is the preferred approach.

The action variable in this method is the maximum action during a storm event. A convenient interpretation is to model the storm maximum action or action effect conditionally on the most probable maximum storm action or action effect. This represents the short-term variability of the problem. The long-term variability is in this case represented by the probability density function of the most probable maximum storm action.

The advantage with this approach is that focus is on the design weather conditions and that several metocean characteristics can easily be included. Operations on board that may affect target actions and action effects can easily be incorporated in the analysis.

A disadvantage is that results may be sensitive to selected storm threshold. The approach requires that simultaneous time series of the metocean characteristics of sufficient length are available. Method may be sensitive to “outliers”.

The target variable in this approach is the storm maximum response X_s instead of the 3-hour maximum X_{3h}. By focusing on severe storms, sampled values of X_s may be regarded as uncorrelated. A storm is here defined as a sequence of short-term conditions where the dominating metocean characteristic, action or action effect exceeds an a priori selected threshold. The threshold should be selected sufficiently high to ensure that sampled values of the dominating characteristic are representative for the upper tail of the distribution. The threshold should also capture 1-3 events from the mildest year of the total data period, but the latter recommendation has lower priority than the first.

Denoting the conditional distribution of AC⟩ X_s ⟨AC given the most probable maximum storm response, $\widetilde{X}_s$, by $F_{X_s|\widetilde{X_s}}(x|\widetilde{x}_s)$, the long-term distribution of storm maximum response is Tromans and Vanderschuren (1995):

$$F_{X_s}(x) = \int_{x_s} F_{X_s|\widetilde{X_s}}(x|\widetilde{x}_s) f_{\widetilde{X_s}}(\widetilde{x}_s) d\widetilde{x}_s \tag{28}$$

The time consuming part of this approach is to determine the parameters of the conditional distribution of the storm maximum. This distribution can in most cases be reasonably well approximated by the Gumbel distribution, i.e.

$$F_{X_s|\widetilde{X_s}}(x|\widetilde{x}_s) = \exp\left\{-\exp\left\{-\frac{x-\widetilde{x}_s}{\beta\widetilde{x}_s}\right\}\right\} \tag{29}$$

The parameter β can be assumed to be independent of the storm severity. For a given problem, β should be estimated as the average of the values obtained for some few of the storms. Depending on the problem, β is likely to be in the range 0,05–0,15, with the lowest value corresponding to a linear action or action effect problem.

$\widetilde{x_s}$ should be calculated for each of the included storms. If a storm includes M 3-hour intervals of stationary conditions and each condition is characterized by metocean parameters z_i, the distribution of X_s for a given storm reads:

$$F_{X_s|STORM}(x) = \prod_{i=1}^{M} F_{X_{3h}|z_i}(x|z_i) \tag{30}$$

The most probable maximum storm action or action effect is approximately found by $F_{X_s|STORM}(\widetilde{x}_s) = 0.37$. The long-term variability is modelled by fitting a distribution to the calculated most probable maximum storm response for all storms.

As $F_{X_s}(x)$ is known, target extremes are estimated by

$$F_{X_s}(x_q) = 1 - \frac{q}{n_s} \tag{31}$$

where n_s is the expected number of storm events in one year.

In order to utilize this method, time histories of simultaneous observations of metocean characteristics are needed for at least all events above the selected threshold. An advantage of this approach is that focus is given to the conditions causing extremes and that it is simple to include simultaneous occurrence of many metocean characteristics. The method is expected to be unbiased, i.e. no conservatism is inherent to the method. The disadvantage is that the distribution function of 3-hour storm step maximum is established for large number of combinations of the metocean characteristics z_i. For complex actions and/or action effects this makes the method not very convenient. The latter is however the case for both long-term approaches reviewed above.

Metocean contour approach

The metocean contour approach is an approximate way of estimating long-term extremes by considering only a few short-term metocean conditions. The advantage of the method is its simplicity, the metocean contours are often given in standard metocean reports. In model test programs, only some few metocean conditions may be tested. The disadvantage is that this is an approximate method. Rather large uncertainties can be associated with the estimated q-probability values. The main difficulty is the choice of α (the percentile of the fitted short-term distribution).

The first step is to establish the contour of the governing metocean characteristics corresponding to an annual exceedance probability q. It is common to base the contour approach on 3-hour metocean conditions, i.e. the contour may be determined such that the exceedance probability per 3-hour condition is $q/2920$. Examples of contours for significant wave height and spectral peak period are shown in Figure A.1.

Given the contour lines or contour surfaces (in the case of more than two metocean parameters), the steps to estimate a q-probability response quantity are the following:

1) Identify the worst metocean condition along the q-probability contour. This can be done by doing a limited number (5–10) of time domain simulations of 3-hour durations or 3-hour model tests for the target response quantity for some few conditions enveloping the worst condition. For the purpose of selecting the worst condition, the 3-hour extreme response can be estimated by $\mu + k\sigma$. μ and σ are the mean and standard deviation of the 3-hour extremes for the considered conditions. k can be taken as 1,3–1,5.
2) When the worst metocean condition is identified, a large number of 3-hour simulations or model tests shall be performed for this condition. The number of simulations should at least be 30, preferably higher. A proper probabilistic model for the 3-hour extreme value should be fitted to the observed extremes. Provided conditions for asymptotic behaviour can be assumed, a Gumbel distribution will generally be a good model.
3) The target extreme value is now estimated by the AC α-percentile AC of the fitted distribution. For most cases α = 0,85–0,90 is adequate for an annual exceedance probability of 10^{-2}, while α = 0,90-0,95 is adequate for an annual exceedance probability of 10^{-4}.

NOTE This is an approximate method. Prior to final design, the adopted target percentile should be verified by long-term analysis of a qualitatively similar problem.

Conditions for this approach to be acceptable are:

1) The target extreme value for the worst metocean condition along the q_1- annual probability contour is to be worse than the target extreme value for the worst metocean condition along the q_2- annual probability contour if $q_2 > q_1$.
2) The target percentiles recommended above assume that the coefficient of variation of the 3-hour extreme value for the target response quantity is of a typical level, i.e. between 0,05–0,2. If the coefficient of variation is 0,5 or larger, target percentiles are to be verified before applying the approach for final design.

It is further required that sufficient metocean data is available to establish robust contours for low annual exceedance probabilities (see also 6.1.1).

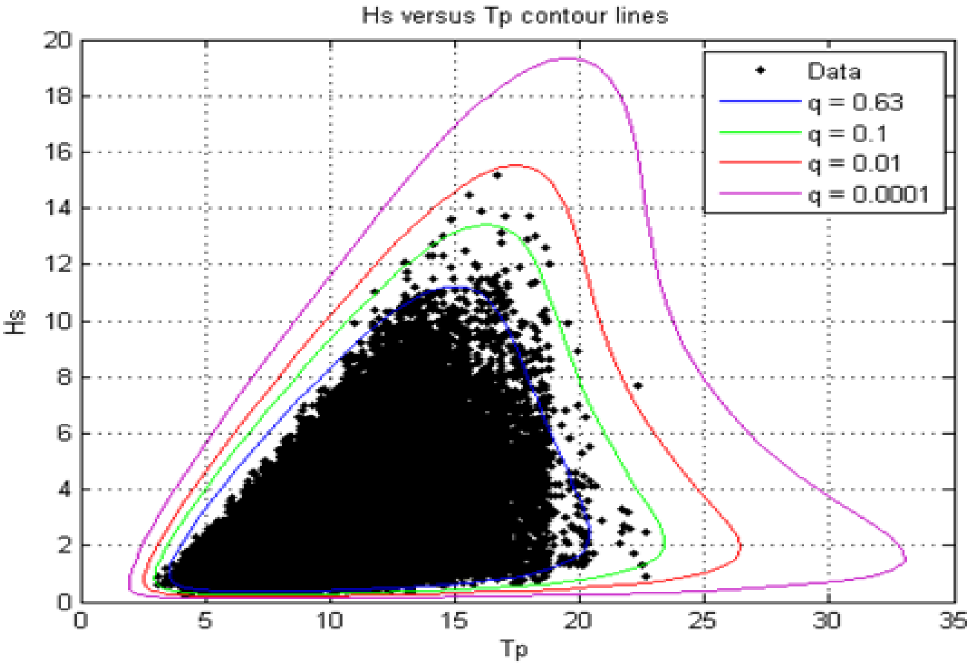

Figure A.1 – Example of contour lines. Hs in [m], Tp in [s]

NOTE Dot is used as decimal point in the figure

Other methods

Other methods may be used for predicting characteristic extremes provided they are sufficiently verified for the intended use.

A.5 Comm 6.2.1 Wave conditions

For preliminary considerations, values for H_s and T_p (based on NORA10 data) as given in Figure A.2 and Figure A.3 may be used.

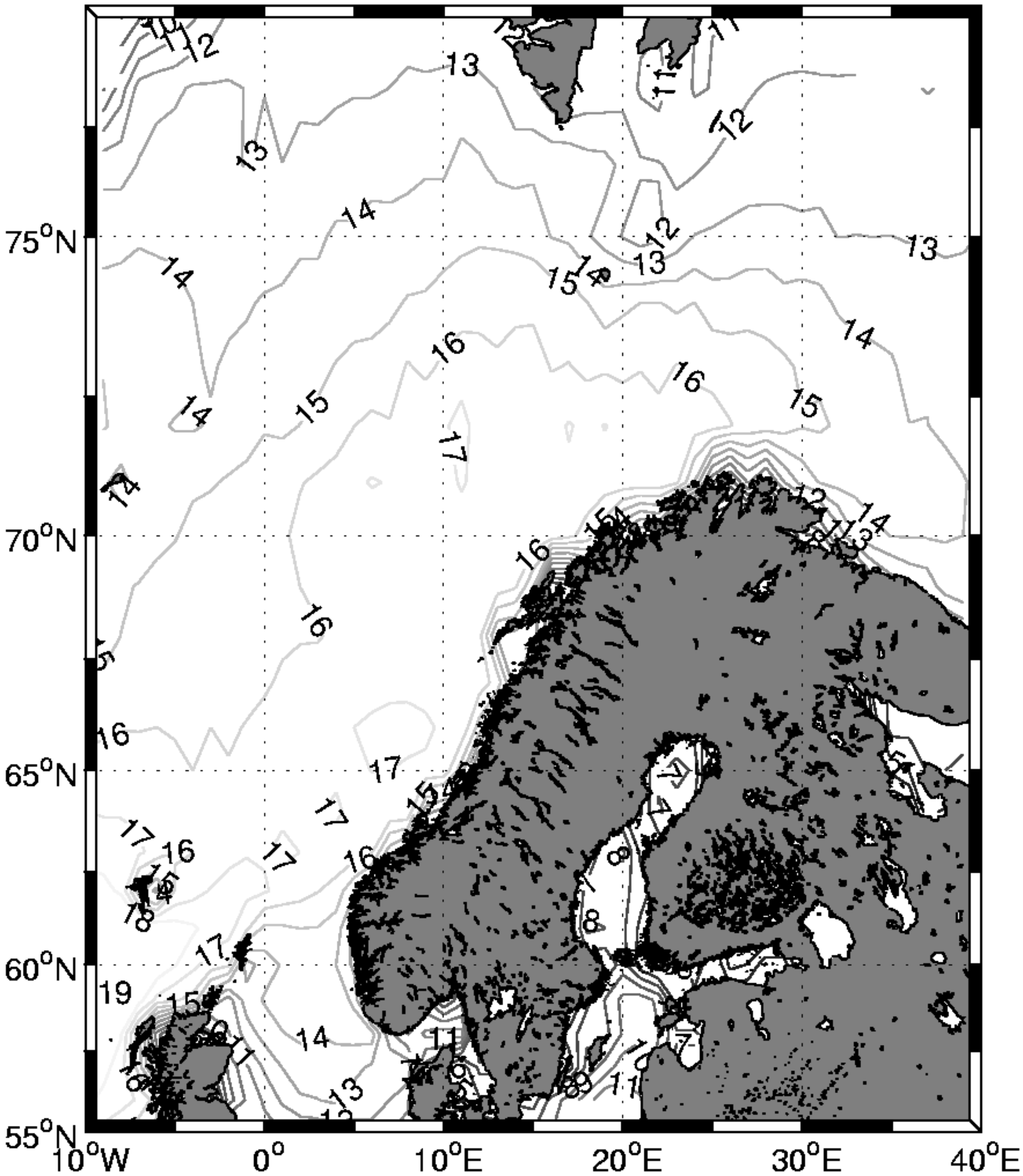

Figure A.2 – Significant wave height contours based on NORA10 1958–2011 corresponding to annual exceedance probability of 10^{-2}

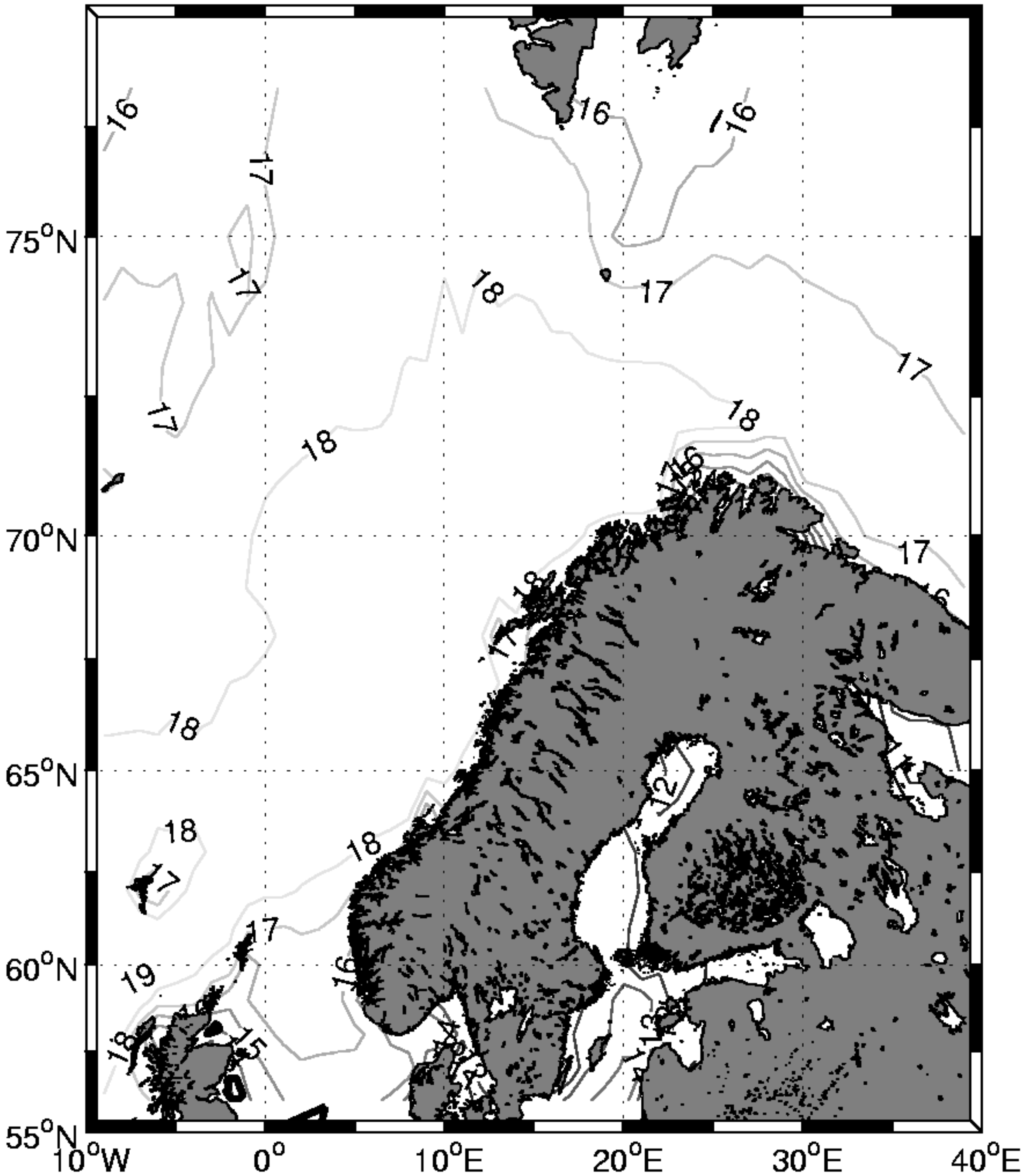

Figure A.3 – Spectral peak period associated with 10^{-2} – annual probability H_s

$$T_p = a \cdot (1{,}0 + H_s)^{(0{,}33+0{,}0029*H_s)}$$

a=5,8 at N55° linear increasing to 6,6 at N78°

A.6 Comm 6.2.1.2 Wave theories

It is well known that calculating kinematics underneath the crest by linear wave theory may result in significant overestimation in particular for high frequencies. Hence, various stretching methods have been proposed. In the Wheeler stretching method (Wheeler, 1970) a measured wave surface is treated as linear and linear theory is used to calculate the kinematics. By stretching of the vertical coordinate, the linear velocity at SWL is transferred to the wave surface. Wheeler stretching in combination with

Gaussian (linear) surface process should not be used for estimating extremes of drag dominated structures. By such an approach, the kinematics in the surface zone as well as at wave crest will be significantly underestimated – in particular for steep waves.

Using a second order surface process, Wheeler stretching can be applied for early phase design evaluations provided a proper implementation of the Wheeler method is done (Birknes et al. 2013). For final design consistent second order kinematics models are recommended. Second order random wave models are presented by Stansberg (1993) and Johannessen (2010). The two second order models are compared in Birknes et.al. (2013). Second order kinematics models compare well with measurements for steep long-crested waves, see Figure A.4.

Second order models for wave kinematics in the vicinity of large volume structures are not recommended due to its computational complexity. For accurate assessment of kinematics close to large volume structures, CFD or model tests are recommended.

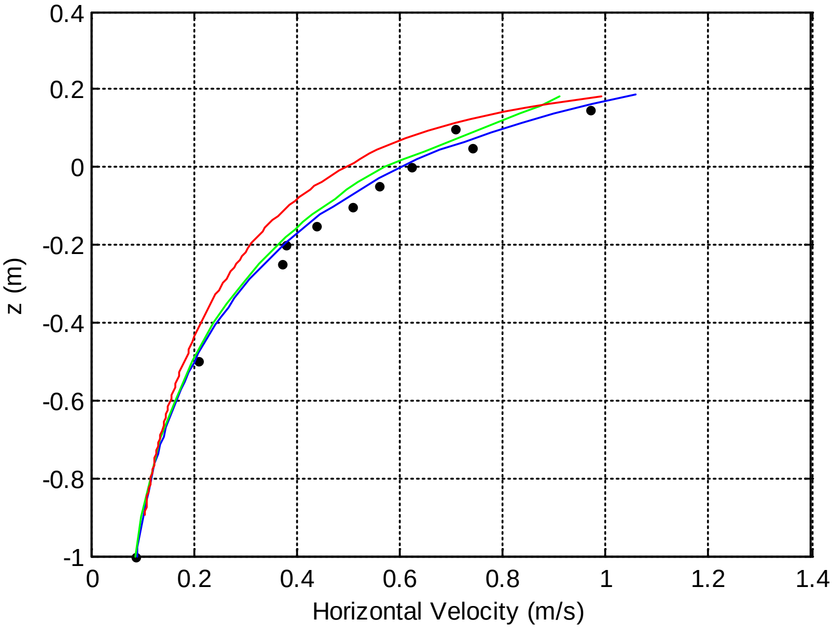

Figure A.4 – Comparison of horizontal velocity profile beneath a steep wave crest. Black dots are the measured velocities, blue line is the second order method of Johannessen, green line is the second order method of Stansberg and the red line is the profile calculated using Wheeler stretching based on the measured wave surface. From DNV (2013)

NOTE Dot is used as decimal point in the figure

A.7 Comm. 6.2.1.5 Design irregular wave conditions

A design sea state is usually assumed to be stationary for 3 hours. In most cases, a sea state is assumed to be characterized by the significant wave height, the spectral peak period and mean direction of propagation.

The mean direction of propagating is usually modelled as a discrete variable, the actual mean direction of propagation for the various sea states is assumed to be within one of a finite number of sectors of equal width. The width should reflect the sensitivity of problem under consideration to mean direction of propagation. A common width is 30°. If the action effect is very sensitive to direction, one should consider using a width of 10° or 15°.

Both significant wave height and spectral peak period are modelled as continuous variables.

The extreme actions and action effects in a given sea state arise from individual waves of the sea state. The maximum action in a short-term condition is caused by a single unfavourable wave or, in case of a problem sensitive to dynamics, to an unfavourable wave group consisting of some few (3–5) unfavourable individual waves. The action and action effect analysis accounts for this by describing the individual waves conditionally with respect to significant wave height, spectral peak period and mean direction of propagation. Depending on problem under consideration, this is done in time domain by simulating time histories of the surface waves or in the probability domain by conditional distribution functions of the various characteristics of the individual waves. The resulting variability seen for the action maxima or the 3-hour maximum action is the short-term variability of the design irregular wave condition.

Ocean waves are short-crested. A short-crested sea state is most often modelled by multiplying the frequency spectrum with a spreading function, see Equations (1) and (2) in 6.2.1.5. In general, the spreading function depends on the wave frequency. It is rather narrow around the spectral peak frequency, but it gets broader for lower and higher frequencies. For action and action effect analyses, the frequency dependence is assumed to be less important than an average spread and is implemented in Equations (1) and (2).

The spreading functions are estimated from measurements, either by a grid of sensors measuring wave elevation over an area of limited size or by floating wave buoys which in addition to heave motion also measure roll and pitch. The estimate involves averaging over the duration of the measurements, which may be in the order of half an hour. This averaging may distort the spreading of the highest and steepest waves in the oceans. Before the degree of spread is assessed conditionally with respect to crest height and wave steepness and no systematic dependency on these quantities are found, it is recommended that for extreme value estimations the sea is modelled to be close to long-crested. The spreading should not be taken to be broader than what is obtained using $n = 10$ in Equation (1).

A.8 Comm. 6.2.2 Current conditions

For non-critical marine operations in areas where no accurate measured data or documented numerical studies are available, the tidal current at still water level given in Figure A.5 may be used. The wind induced current velocities at still water level may be selected equal to 3 % of the 1-hour mean wind velocity at a 10 m elevation, see 6.4.

For early phase design assessments, a current profile specified by 1,25 m/s at the surface and linearly reduced to 0,7 m/s at sea bed can be used if no measurements are available, see Figure A.6. For locations close to the coast where coastal eddies may represent a design condition, one can use a large surface speed and a reduced and constant speed at a depth 15 % below mean surface, see Figure A.6. In the latter case, the current profile can be combined with reduced significant wave height and mean wind speed. For final design, more robust estimates shall be established.

In early phases of a development and for exploration drilling facilities where no accurate measured data or documented model results exist, the current variation with depth may alternatively be chosen in accordance with DNV-RP-C205.

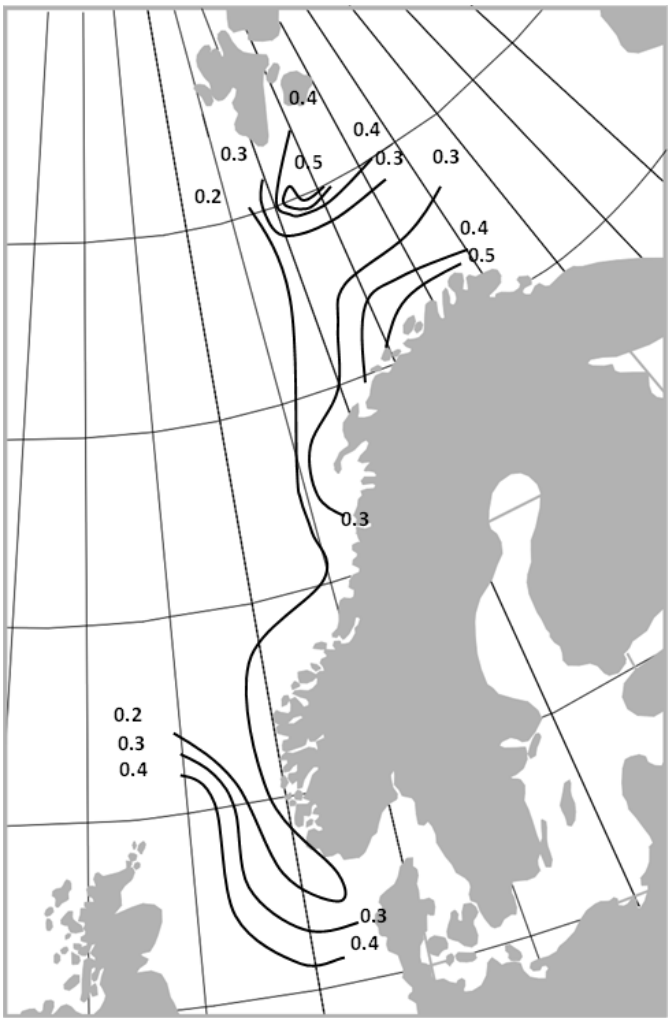

Figure A.5 – Maximum tidal speed (m/s)

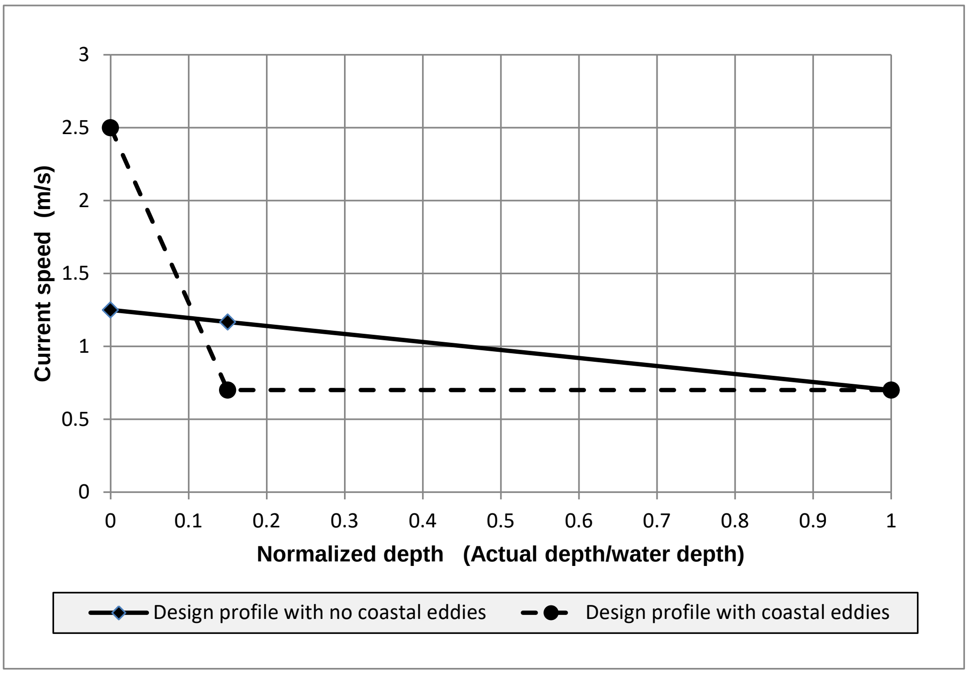

Figure A.6 – Preliminary total current profiles for cases when no current data are available

Stokes drift and wind induced surface current

The wind induced current may be divided into current due to momentum transfer from wind to sea via shear stresses and turbulent mixing, and Stokes drift due to the mean mass flux in surface waves. Current actions on fixed structures and large volume floating structures are not affected by Stokes drift. However, Stokes drift determines the drift of small floating objects (e.g. drift of oil spill slicks and small pieces of ice).

The Stokes drift velocity at the wave surface in a long-crested sea state is given by Webb & Fox-Kemper (2011).

$$U_s = \frac{k_s \pi^3 H_S^2}{g T_Z^3} \tag{32}$$

where H_S is the significant wave height and T_Z is the mean zero up-crossing wave period. The factor k_s can be taken as approximately 1,5 both for wave spectra of the Pierson Moskowitz and Jonswap type (with $= 3{,}3$). The Stokes drift velocity is in the direction of wave propagation. A reduction factor in short-crested seas can be taken as 0,95.

In addition to the Stokes drift a wind shear effect contributes to the surface current. This current is in the order of 1 % of the mean wind speed 10 meters above sea level and its direction is almost perpendicular to the wind direction, to the right of the wind vector at the Northern hemisphere (Ardhuin et al., 2009).

Both the Stokes drift and the current induced by wind shear decays rapidly with depth.

A.9 Comm 6.2.3 Modelling simultaneous occurrence of metocean condition

If more than 3 parameters are to be included in the joint modelling for a response based analysis, the most convenient metocean information would be a time series of simultaneous values for all parameters covering several decades. For such cases, availability of current data series of sufficient length is a limitation. If the current field is independent of wind and waves, one can repeat the available current series (of at least 5 years). If there is a correlation between the current and the other metocean parameters, a current data set may be estimated from these parameters. When such analyses are performed, sensitivity studies should be carried out for sources associated with large uncertainties.

A.10 Comm 6.4.2 Wind conditions

Extreme wind speed contours and annual average wind speed contours at the Norwegian Continental Shelf are given in Figure A.7 and Figure A.8 in respectively.

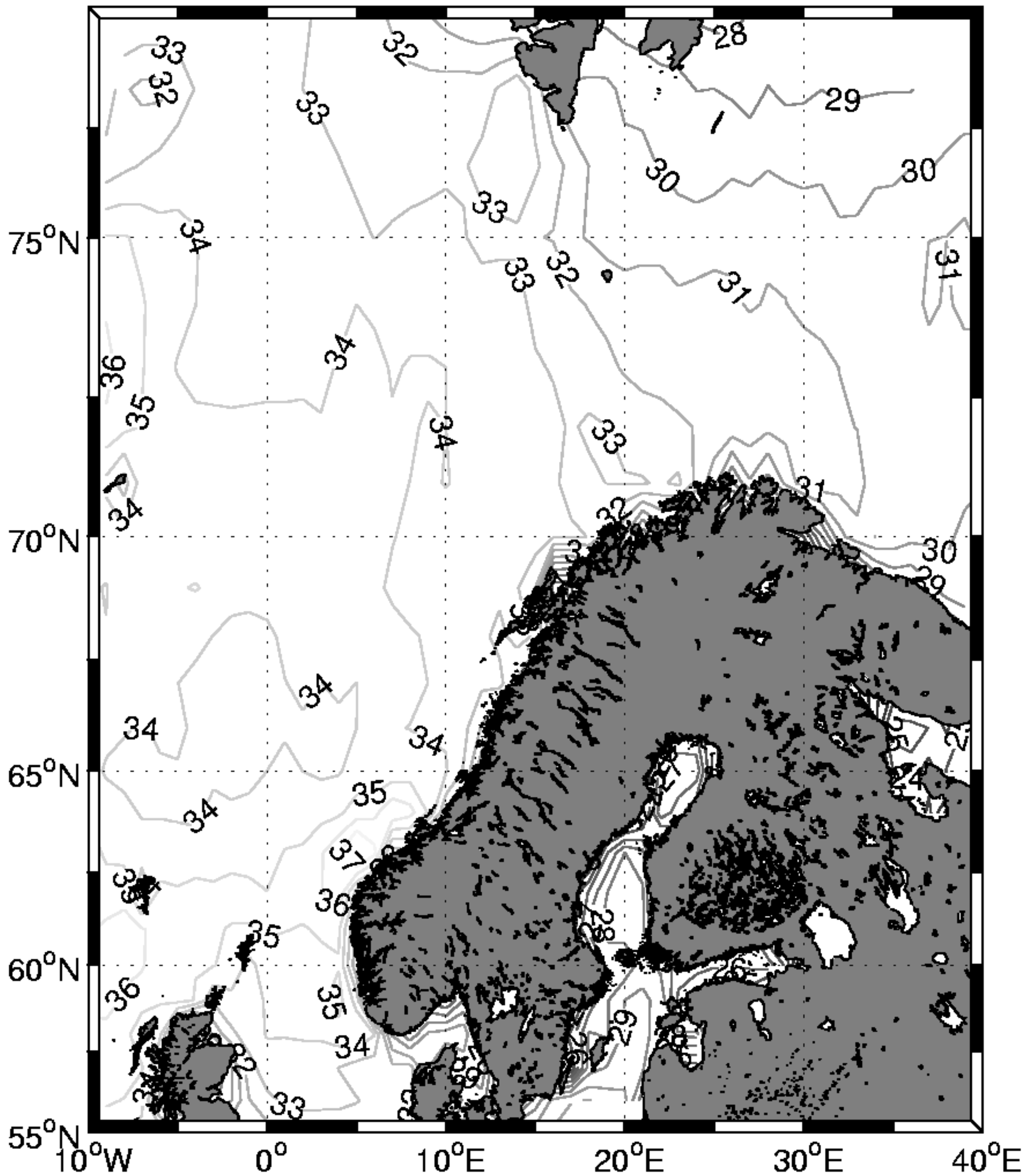

Figure A.7 – 1-hour extreme wind speed contours at 10 m above MSL based on NORA10 1958–2011 corresponding to annual exceedance probability of 10^{-2}. Wind speed above 15 m/s is slightly adjusted as described in 6.4.2

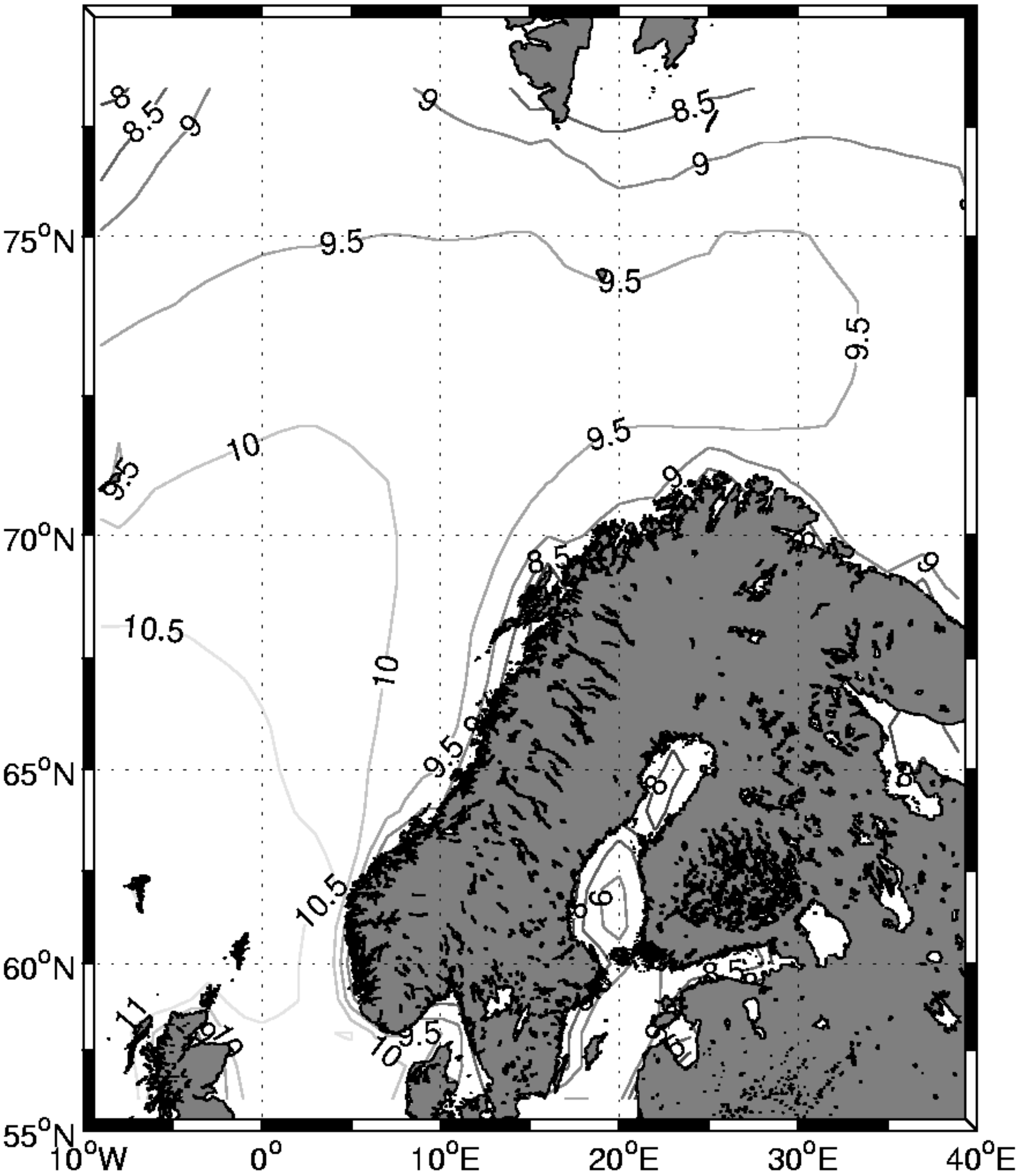

Figure A.8 – Annual mean wind speed for the Norwegian Continental Shelf, NORA10 1958–2011
NB! 100 m above mean sea level

A.11 Comm 6.5 Actions from snow

Precipitation under air temperatures colder than 1 to 2 degrees Celcius (ºC) usually forms as snow crystals. Unlike rain droplets, the snow may accumulate on horizontal, sloped and vertical structures.

The accretion of snow may have different natures and effects, such as causing additional weight, uneven weight distribution and potential loss of vessel stability, changed structural shapes, obstruction and changed friction.

Snow density is a strongly varying property of high importance for estimating actions on structures. Snow density depends on parameters such as temperature during formation and during, and after deposition, grain shape and size, overburden pressure, wind speeds and liquid water content. The large variability of density is important when converting snow depths to actions. Table A.2 presents snow densities used in standards for calculating actions.

Table A.2 – Density of snow according to NS-EN 1991-1-3

Type of snow	Density (kg/m^3)
Dry fresh	100
Settled (hours or days old)	200
Old (weeks or months old)	250-350
Wet	400

A.12 Comm 6.6 Actions from sea spray icing

Icing caused by freezing sea spray is widely reported and usually the most serious form of icing at sea. Critical stability problems in high winds and heavy seas can result from asymmetric loading and consequently a rise in the vertical centre of gravity of buoyant structures by the accumulated ice.

Sea spray icing includes interaction spray and white cap spray from waves. Due consideration to all relevant physical environmental parameters and structure characteristic should be taken into account when assessing sea spray icing actions.

Accretion of sea spray icing depends on the following parameters:

- wind speed;
- air temperature;
- sea water temperature;
- wave height and period;
- geometry of the structure and response to waves.

The main processes in sea spray icing are:

- spray generation;
- spray transport;
- heat transport by airflow;
- spray accumulation by freezing.

In particular, designers should be aware of significant uncertainties related to spray generation and spray transport and proper attention should be given to uncertainties in estimates on total accreted ice masses.

For time limited operations and at early stages in project development tabulated values can be applied. 10^{-2} and 10^{-4} exceedance probability values for thickness and associated density of accumulated ice may be selected as indicated in Table A.3 and Table A.4, respectively.

The recognised Overland formulation (Overland, 1990) for icing severity is considered useful and relevant in order to evaluate the potential for sea spray icing at a location and in order to compare icing severity at different geographical sites. The Overland formulation is however not applicable for calculations of ULS/ALS icing conditions on offshore structures.

Table A.3 – Ice actions from sea spray icing with annual probability of exceedance 10^{-2} (ULS)

Height above Sea level m	Sea spray icing		
	56° N to 68° N mm	**> 68° N**	**Density** kg/m³
0 to 5	Linear increase from 0 mm to 80 mm	Linear increase from 0 mm to 650 mm	900
5 to 10	80	Linear decrease from 650 mm to 150 mm	900
10 to 25	Linear reduction from 80 to 3	Linear decrease from 150 to 5 mm	900
Above 25	3	5 mm	900
Extreme Icing will not occur at levels above and below 5 m above sea level simultaneously. Icing below 5 m will not occur in combination with extreme waves (ULS and ALS conditions)			

Table A.4 – Ice actions from sea spray icing with annual probability of exceedance 10^{-4} (ALS)

Height above Sea level m	Sea spray icing		
	56° N to 68° N mm	**> 68° N** mm	**Density** kg/m³
0 to 5	Linear increase from 0 mm to 140 mm	Linear increase from 0 mm to 1100 mm	900
5 to 10	140	Linear decrease from 1100 mm to 260 mm	900
10 to 25	Linear reduction from 140 to 5	Linear decrease from 260 to 10 mm	900
Above 25	5	10 mm	900
Abnormal icing will not occur at levels above and below 5 m above sea level simultaneously. Icing below 5 m will not occur in combination with extreme waves (ULS and ALS conditions)			

There is a strong gradient in icing severity from South to North on the Norwegian Continental Shelf that should be given attention in plans for operation and design. Contours for mean number of days with heavy and moderate sea spray icing, respectively, are given in Figure A.9 and Figure A.10.

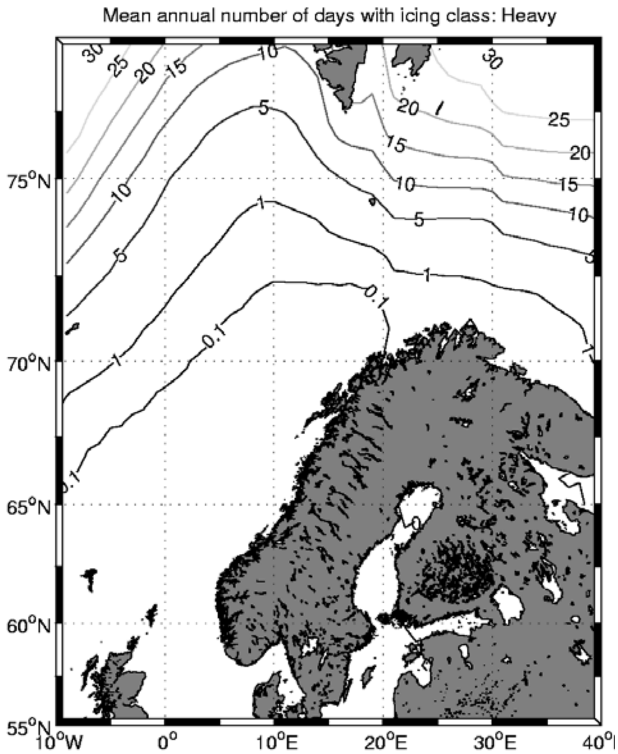

Figure A.9 – Mean number of days per year with heavy icing, Heavy icing: 2–4 cm/hour

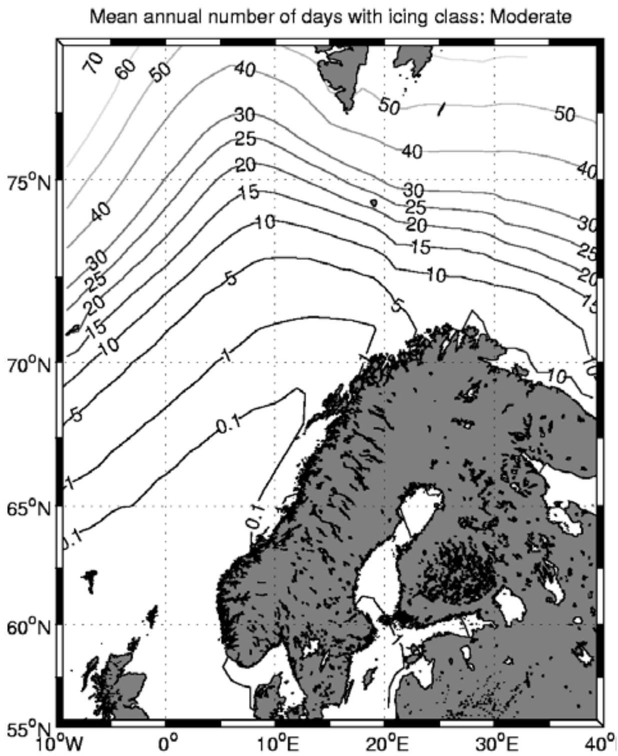

Figure A.10 – Mean no of days per year with moderate icing. Moderate icing: 0,7–2,0 cm/hour

A.13 Comm 6.7 Actions from [AC⟩ atmospheric ⟨AC] icing

There are four classifications of atmospheric icing:

1) Wet snow icing:
- a form of precipitation icing;
- typical for snow falling at temperatures between 0 and 3 °C;
- stick to all surfaces, but may build up to cylindrical ice of 5 kg/m and more at slender structural elements (diameter less than 30 cm), see ISO 12494;
- increase somewhat with the wind speed, but may also stick to cables at calm weather;
- typical density 500 kg/m^3, but may vary;
- may remain at the structure when the temperature drops below 0 °C after the icing episode;
- should be considered offshore.

2) Freezing rain
- A form of precipitation icing. May also be referred to as glaze.
- Typical for water and drizzle falling at surfaces with temperature below 0°C.
- For cold weather, this ice sticks to all surfaces hit by the precipitation. Close to 0°C the ice amounts will depend on surface properties.
- Density 900 kg/m^3.
- Typical at cold, lowland areas onshore, but should also be considered offshore.

3) In-cloud icing
- Also called rime or hard rime.
- This ice is due to supercooled cloud droplets hitting structures (masts and towers).

- Stick to surfaces exposed to wind, especially slender objects, corners and irregular surfaces.
- Typical density 500 kg/m^3, but may vary.
- Typical at coastal mountains where humid air masses are lifted and form clouds around the mountain tops. May then grow to several hundred kilo per meter structure at the most exposed sites.
- Not to be considered offshore for structures below 200 m. One exception may be sea smoke near the ice edge.

4) Hoarfrost

- This ice forms when water vapour transforms directly to ice.
- Forms at low temperatures.
- Hoarfrost is of low density and strength.
- May occur near open sea, near cold land and sea ice surfaces.
- No significant actions.

A.14 Comm. 6.8 Sea ice actions

Sea ice is frozen seawater. Sea ice is formed when low air temperatures cools down the ocean and the surface temperatures drops below the freezing point (approximately -1,9 °C in salt water). The sea ice drift is dynamic under the influence of winds, waves and currents or can be considered static (landfast) in coastal regions. The characteristics of sea ice can vary significantly based on a number of parameters (e.g. location, season, freezing degree-days, salinity, geography, etc.). In relation with assessments of ice-structure interactions, it is important to distinguish between different possible realizations of ice-structure interactions, e.g. a structure will behave differently when exposed to a continuous ice cover compared to a field with broken ice and open water or ice slush in between ice floes.

A.15 Comm. 6.10 [AC⟩ Micellaneous ⟨AC]

A simplified classification of the Barents Sea licence area of the Norwegian continental shelf is given in Figure A.11 and Table A.5 for area for early phase assessments.

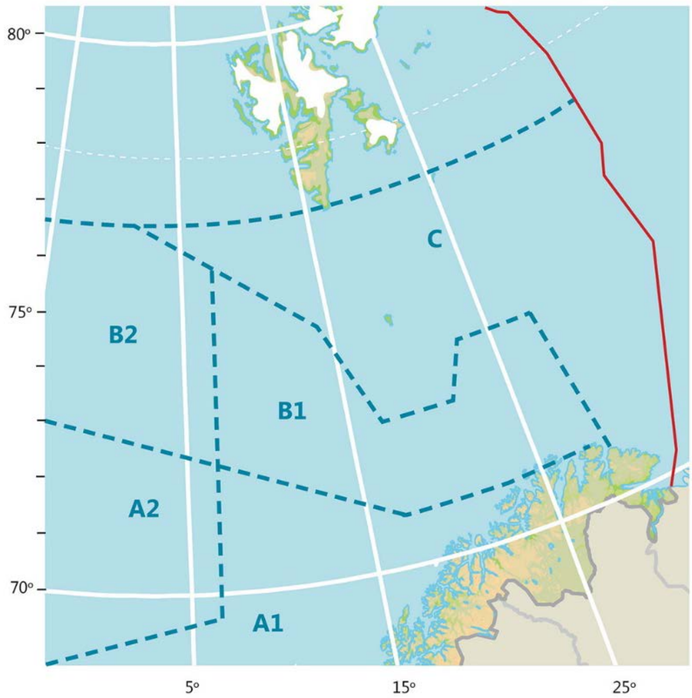

Figure A.11 – Classification of oil and gas licence area Barents Sea

Table A.5 – Simplified classification of the Barents Sea licence area metocean data

Zone	Metocean conditions
A1	Wind chill temperature (T_{wc}) should be expected to exceed -30ºC 50 hours per year. Reference is made to Annex B for further details.
	Temperatures of down to -20 °C will occur with an annual probability of 10^{-2}.
	Moderate icing may occur minimum 1 day annually. Heavy icing may occur 1 day every 10th year.
	Snow of 0,8 kPa with an annual probability of 10^{-2} may occur.
	Polar lows may occur
A2	As for A1, and in addition: Wind chill temperature (T_{wc}) should be expected to exceed -30ºC up to 500 hours per year. Reference is made to Annex B for further details.
	Sea ice may occur with an annual probability of 10^{-2}.
B1	Winter season may be defined from November to March.
	Wind chill temperature (T_{wc}) should be expected to exceed -30ºC up to 200 hours per year. Reference is made to Annex B for further details.
	Heavy icing should be expected to occur 5 days annually, moderate icing should be expected to occur up to 30 days annually.
	Snow of 0,8 kPa with an annual probability of 10^{-2} may occur.
	Temperatures of down to -30 °C will occur with an annual probability of 10^{-2}.
	Sea ice will occur with an annual probability of 10^{-4}.
	Iceberg collisions may occur with an annual probability of 10^{-4}.
	Polar lows may occur
B2	Winter season may be defined from November to March.
	As for B1, and in addition: Wind chill temperature (T_{wc}) should be expected to exceed -30 °C up to 800 hours per year. Reference is made to Annex B for further details.
	Sea ice will occur with an annual probability of 10^{-1}.
C	Winter season may be defined as October to April.
	Wind chill temperature (T_{wc}) should be expected to exceed -30ºC up to 2000 hours per year. Reference is made to Annex B for further details.
	Heavy icing should be expected to occur 25 days annually, moderate icing should be expected to occur up to 50 days annually.
	Snow of 1,0 kPa with an annual probability of 10^{-2} may occur.
	Temperatures of down to -40 °C will occur with an annual probability of 10^{-2}.
	Sea ice will occur annually.
	Iceberg collisions may occur with an annual probability of 10^{-2}.
	Polar lows may occur
	Visibility < 1000m 8,58 % of the time Visibility < 10000m 31,76 % of the time

A.16 Comm. 8.2 Temperature actions

Unless more accurate measurements or calculations are carried out, air and sea temperatures may be taken from Figures A.12–A.14. Sea temperature also varies with depth. The local air temperature may be higher as a result of sun radiation. During fabrication of the structure, all dimensions should be related to a reference temperature.

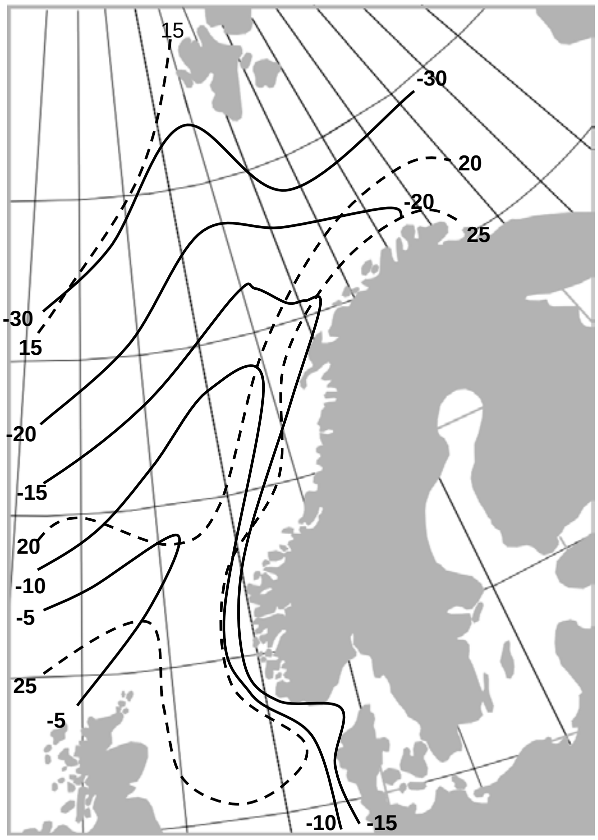

Figure A.12 – Highest and lowest air temperature with an annual probability of exceedance of 10^{-2} (the temperatures are given in °C)

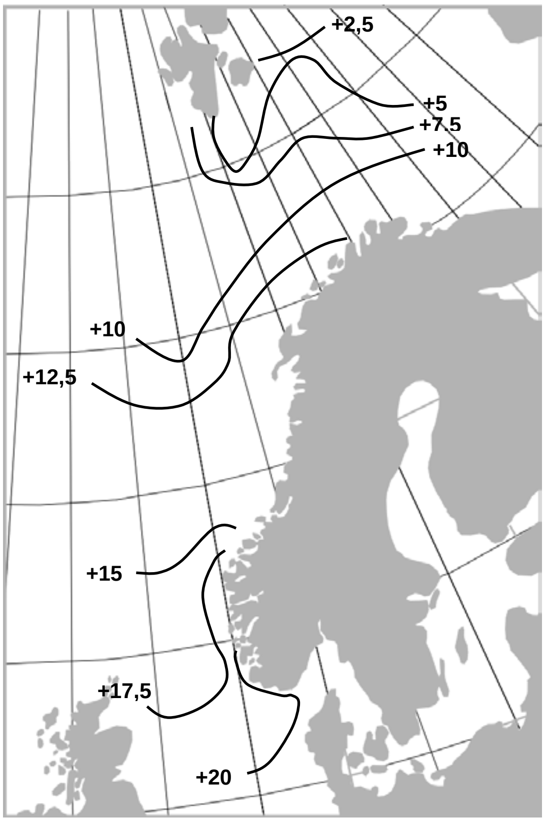

Figure A.13 – Highest surface temperature in the sea with an annual probability of exceedance of 10^{-2} (the temperatures are given in °C)

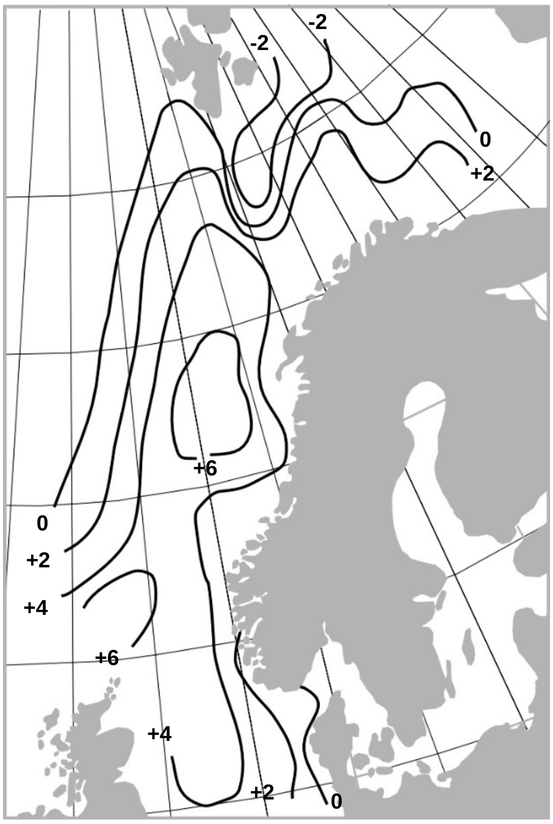

Figure A.14 – Lowest surface temperature in the sea with an annual probability of exceedance of 10^{-2} (the temperatures are given in °C)

A.17 Comm. 11.1.8 Model testing

Planning of basin model tests

When planning a model test, the following should be considered:

- The action effects to be considered by the model test should be clearly identified a priori.
- Prior to detail planning, it should be decided for which response parameters the model test is primarily done for calibrating computer tools and for which parameters characteristic design values are to be estimated directly from the model test.
- For each response problem it should be decided if it is sufficient to run the test with waves only or if also simultaneous wind or both wind and current should be included. One should also consider if regular or irregular waves should be utilized. For most cases, irregular waves will be preferable.
- For each response parameter the worst range along the q-probability contour line should be identified. This range will not necessarily be the same for all included response problems.
- The number of repetitions of the various sea states should be addressed. For the purpose of identifying the worst sea state along the target contour line or if purpose mainly is calibration of computer tools, a rather limited number of repetitions is necessary, 4 is recommended. For action effects where characteristic design values are to be estimated from the model test data, the required number of repetitions for the worst sea state is much higher, 20 is recommended. If the upper tail of the distribution function involves strong nonlinear mechanisms, even more repetitions may be necessary.

For further information on basin model tests, see DNV-RP-C205.

<u>*Guideline for wind tunnel tests for stability checks for floaters*</u>

Wind model tests for floaters are recommended performed as follows:

- Laboratory: The wind tunnel test should be carried out by a recognized institution.
- Model: The model should include all structural details and equipment with may effect the result of the wind tunnel tests. Separate tests should be performed for the above water construction and the underwater body.
- Wind profile: The above water part of the model should be exposed to a simulated natural ocean wind. Reference height for velocities should be 10 m above the water level. Wind profiles should be according to 6.4.3. Curves showing both the measured velocity profile and the target velocity profile should be documented in the model test report. The turbulence intensity profile in the wind direction of the simulated wind should also be considered. The turbulence intensity should accordingly be measured in the wind tunnel at several vertical positions and reported.
- Current profile: The underwater body should be exposed to the desired current profile.
- Scale effects: Provision should be made to ensure the same character of the flow in the model scale as in the full-scale conditions. This should be documented by measurements at various Reynolds numbers for both the above water construction and the underwater body.
- Measurements: Three forces and three moments (corresponding to the main axes) should be measured for each test run. The results should be presented as non-dimensional coefficients. The instruments should be calibrated prior to each trial. The calibration before and after at test series should give the same results. If not, re-testing is necessary.
- Critical axis: The wind direction at which the highest wind overturning moments will be encountered, shall be determined on the basis of a test of each draught applicable. The combined results of the above water construction and underwater constructions should also be considered. The wind direction should be changed in steps of 10 degrees from 0 to 360 degrees. Moment coefficients, etc. should be shown in a diagram.
- Tests: Tests of both above water construction and underwater body should be carried out for a sufficient number of draughts to establish wind moment curves as a function of draught. For each draught, tests should be performed for at least three, but preferably four angles of inclinations (e.g. 0, 5, 10,15 and 20 degrees). The wind direction should be perpendicular to the critical axis, but also test runs at directions 10, 20 and 30 degrees to each side of the perpendicular shall be performed and the maximum moment component to the critical axis should be determined. The reason for testing at wind directions 10, 20 and 30 degrees to each side of the critical wind direction, is to make sure to use the maximum overturning moment for the inclined conditions. Curves showing heeling moments as a function of the inclination angles for each test draught should be drawn. Further, curves showing heeling moments as a function of the draught should be drawn for each inclination angle.
- Documentation: The test results including wind forces and moments should be submitted together with sufficient explanatory information and the concluding remarks of the test results should be included in the test report. Photos of the model prior to testing as well as each test condition should also be included.

A.18 Comm. 11.2.1.3 Dynamic action effects

Indicative values can be found in Table A.6 based on measurements of total damping (including structural damping, hydrodynamic damping, aerodynamic damping and damping from soil and foundation). These indicative values should only be used in early design stages or as indications of damping ratios.

If the action effects are very dependent on the choice of damping, sensitivity analysis shall be performed and conservative estimates applied.

Table A.6 – Indicative values of total damping

Environment	Structural type:	% of critical damping
In water	Jacket structure (FLS)	1,5
	Jacket structure (ULS & ALS)	2,0
	Jack-ups (FLS)	2,0
	Jack-ups (ULS & ALS)	3,0
	GBS	2,0
In air	Flare towers	0,2
	Heli deck support frame	0,2
	Derricks	0,2
	Individual members	0,15

A.19 Comm. 11.6.3.2 Airgap and wave in deck analysis for fixed structures

The following wave in deck calculation procedure is recommended for the design of bottom fixed transparent structures where diffraction effects may be ignored. The wave in deck actions can be calculated based on an extreme crest event in an irregular design sea state or a regular design event. A regular design wave approach must be shown to be conservative. The wave event approach should include the following:

- Determine q annual probability crest height C_q based on the Forristall distribution and accounting for higher-order effects and spatial statistics.
- Adjust height of top of crest to include storm surge and tidal level according to Table 7.
- Fit wave event to (C_q, T_{Cq}) with due consideration to top of crest kinematics. The wave period corresponding to C_q can be taken as $0{,}9 \cdot T_p$.
- For the regular design wave approach velocity in the crest is not to be reduced due to wave spreading
- Since different failure modes may be triggered depending on the duration of the action, the steepness (wave period) of wave event should be varied.
- Analyse the interaction between wave event and deck structure by means of CFD with due consideration to spatial and temporal convergence. Possible effect of compressed air should be included in the analysis. Simplified methods may be used if these methods are verified by CFD analysis or validated by model tests (DNV-RP-C205).
- Due consideration should be given to details in the geometric modelling of the underside of the deck, including girders that will affect the wave in deck actions.
- The effect of near breaking and breaking waves needs to be assessed.

A.20 Comm. 11.6.3.3 Local wave impact

For an estimate of the structural capacity of equipment located above the global wave crest, but within the reach of a local wave, a pressure of 80 kN/m^2 should be applied. For a fixed platform, it may be assumed that the local wave can reach a maximum height of 1,4 times the ULS wave crest height for the relevant sector. For impact on walls, this pressure should be applied over a vertical area at right angles to the wave direction. The area can be limited to a rectangle 10 m wide by 3 m high. If the actual projected area of the wall is less than the above values, then the actual area can be used. For impact on pipework and deck equipment, this pressure should be applied to the projected equipment vertical area relating to/facing the wave direction. For circular smooth pipes, the local wave force can be reduced by a factor of 0,7. For vertical force on horizontal areas a pressure of 40 kN/m^2 should be applied. The vertical force should be assumed to act in combination with the horizontal local wave.

Annex B
(informative)
Wind chill temperature

B.1 Wind chill temperature

The wind chill temperature is a measure of the degree of cooling of the human body during exposure to a wind-temperature environment. Scientists and medical experts in U.S. and Canada developed a Wind Chill index by iterating a model of skin temperature under various wind speeds and temperatures. The model use standard engineering correlations of wind speed and heat transfer rate. Heat transfer was calculated for a bare face in wind, facing the wind, while walking into it at 1,4 m/s. The model corrects initially the wind at 10 m elevation to wind speed at the face height, assuming the person is in an open field. The wind chill index, T_{wc} is defined by [64] Osczevski, R. and Bluestein, M., (2005):

$$T_{wc} = 13{,}12 + 0{,}6215 \cdot T_a - 11{,}37 \cdot V^{0.16} + 0{,}3965 \cdot T_a \cdot V^{0{,}16} \quad [°\mathrm{C}] \tag{33}$$

where T_a (ºC) is air temperature and V (km/h) is wind speed at 10 m.

Equation (31) is applicable if human body is on ground (1,5 m elevation) while winds are taken from 10 m height elevation. Reason for this is that both forecasted and recorded winds onshore often represent 10 m elevation winds. Offshore, winds are usually recorded at higher elevations and converted to the deck elevations by empirical formulations. If winds are from the same elevation as wind chill temperature is calculated, the following transformed version of Equation (32) is recommended:

$$T_{wc} = 13{,}12 + 0{,}6215 \cdot T_a - 14{,}651 \cdot U_a^{0{,}16} + 0{,}5109 \cdot T_a U_a^{0.16} \tag{34}$$

where U_a (m/s) is wind speed at the elevation wind chill is to be calculated for.

Based on data from the Nora10 hindcast archive, contour lines showing mean number of hours per year with wind chill temperatures in the range -5 °C to -35 °C, have been established (see Figures B.1–B7).

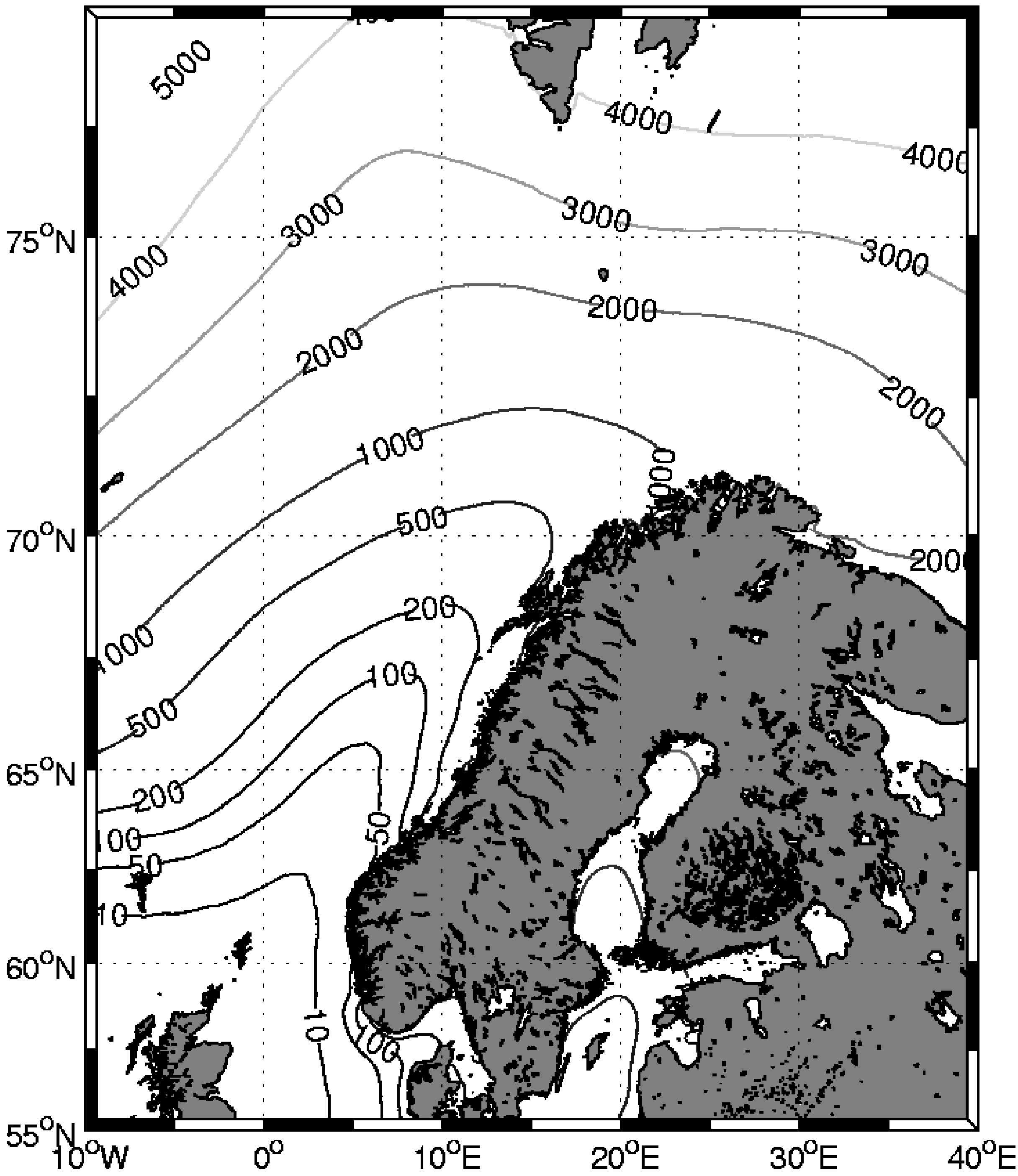

Figure B.1 – Annual mean number of hours with WCT < -5 °C, 30 m above MWL

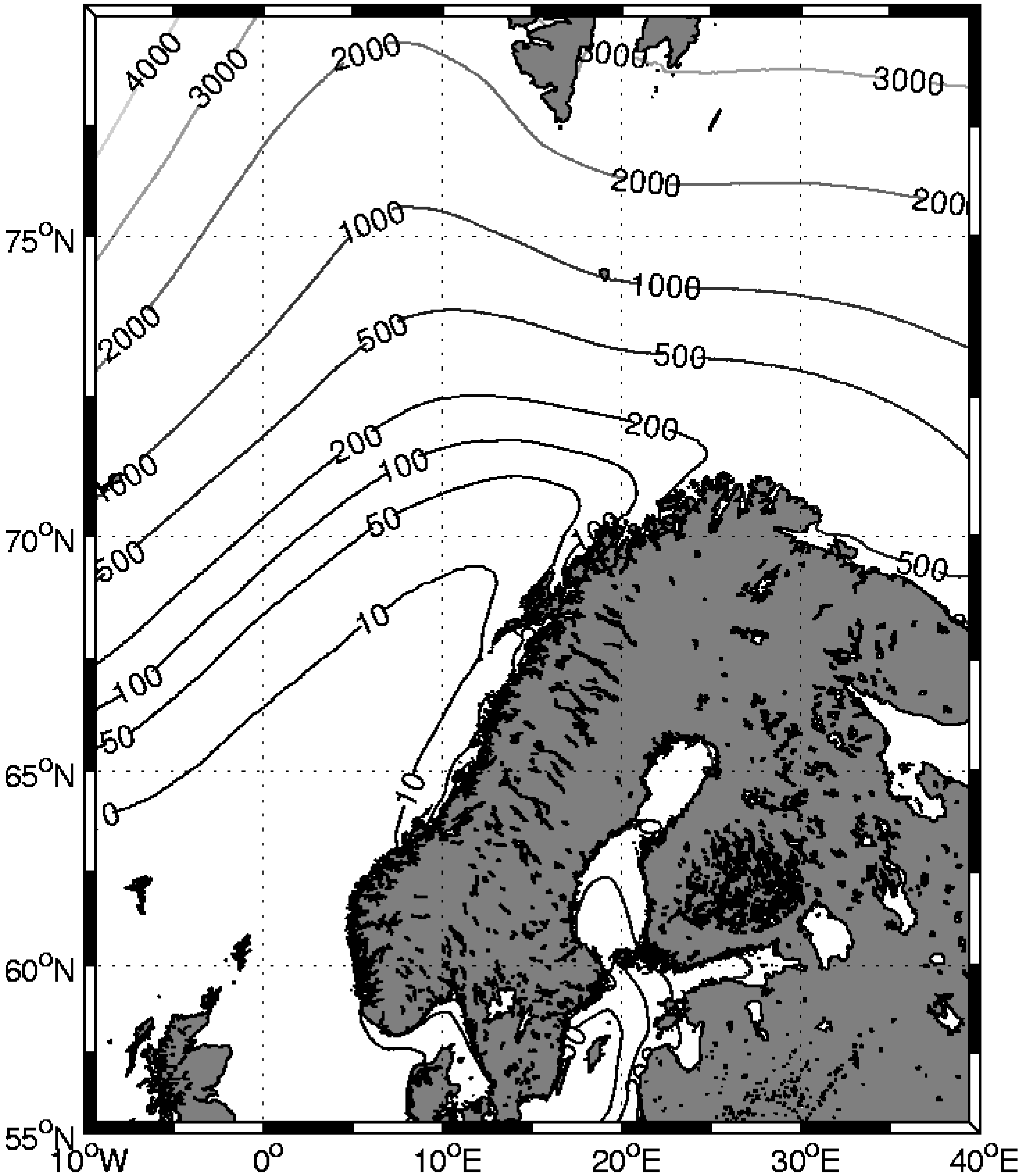

Figure B.2 – Annual mean number of hours with WCT < -10 °C, 30 m above MWL

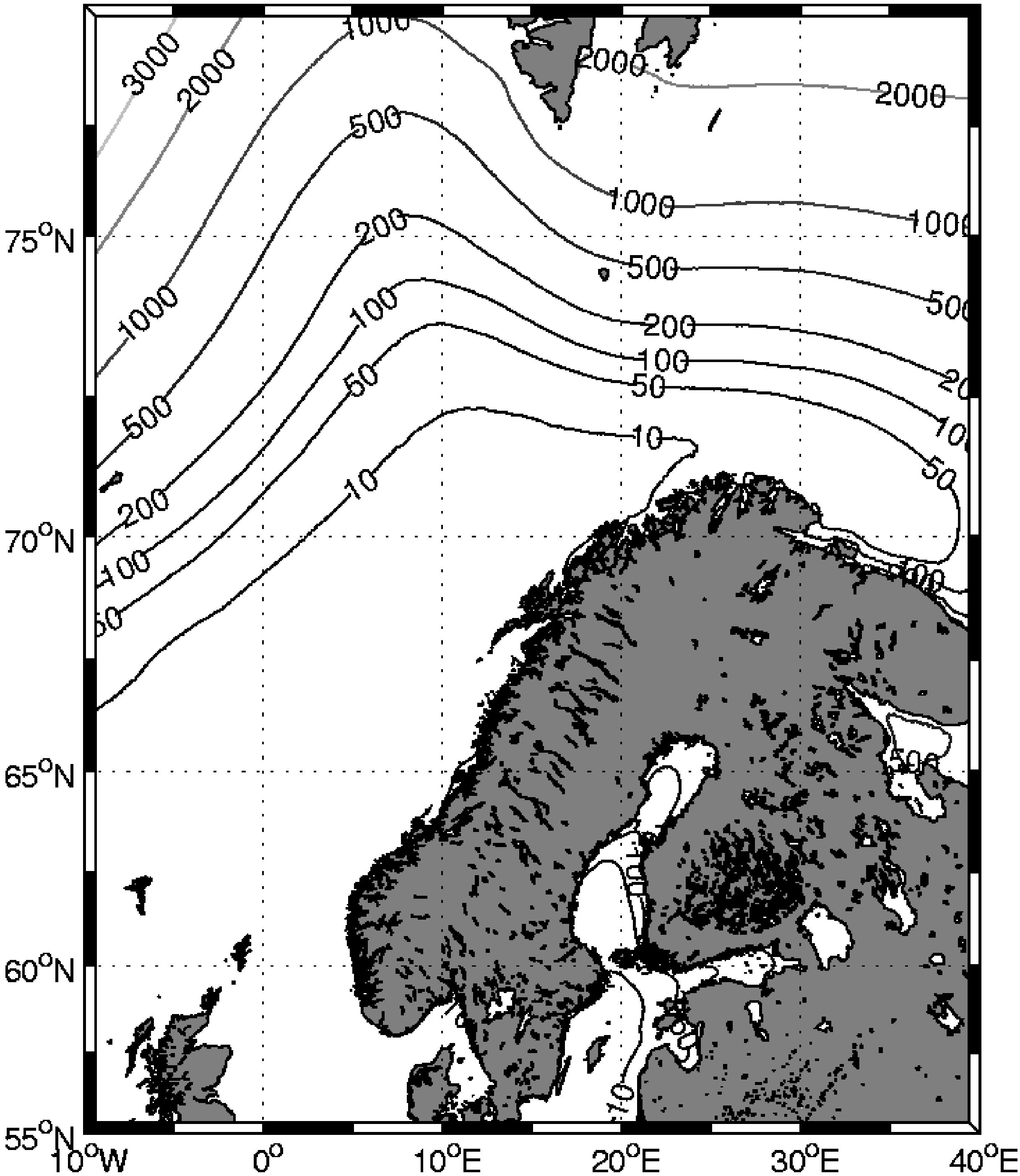

Figure B.3 – Annual mean number of hours with WCT < -15 °C, 30 m above MWL

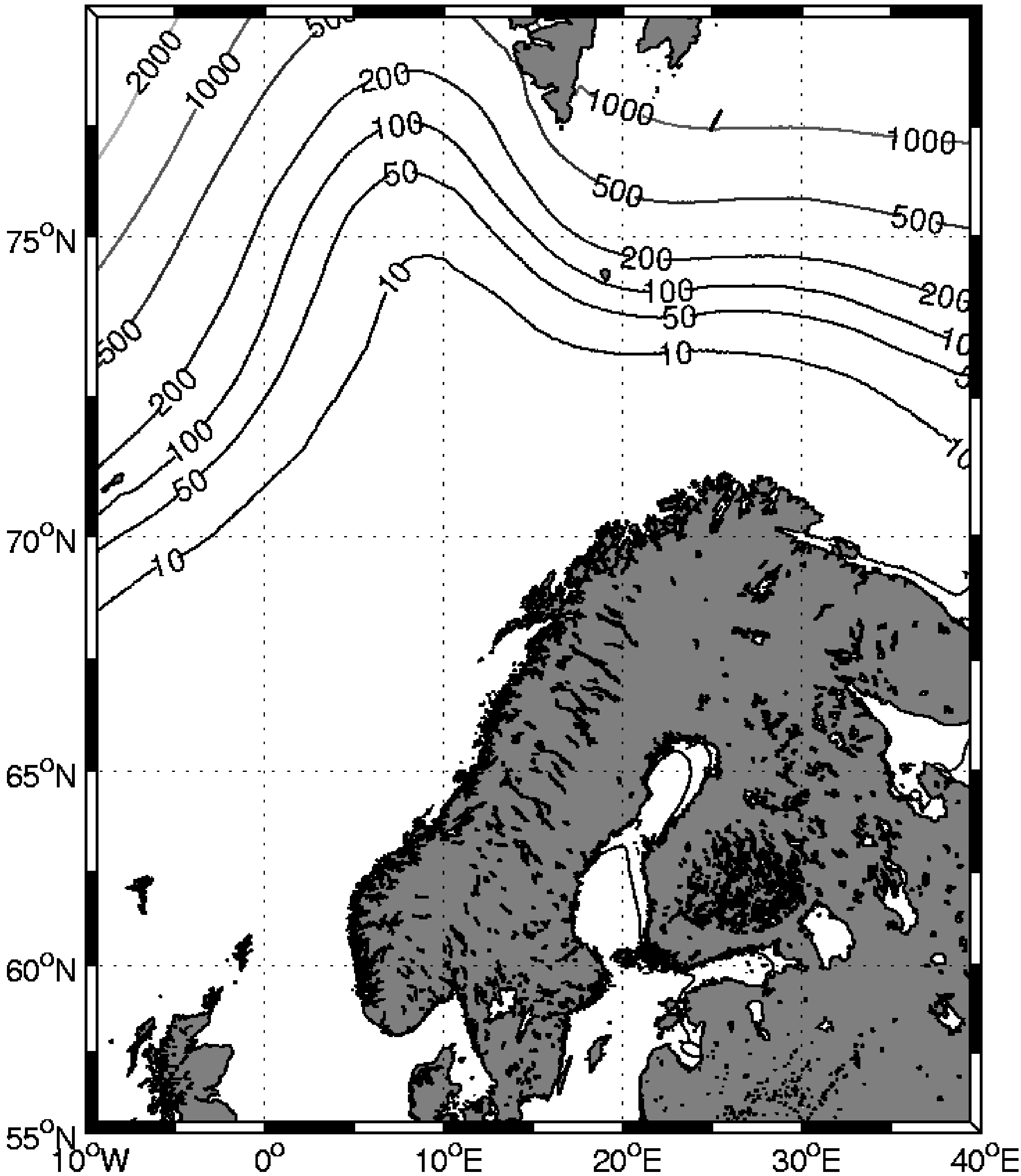

Figure B.4 – Annual mean number of hours with WCT < -20 °C, 30 m above MWL.

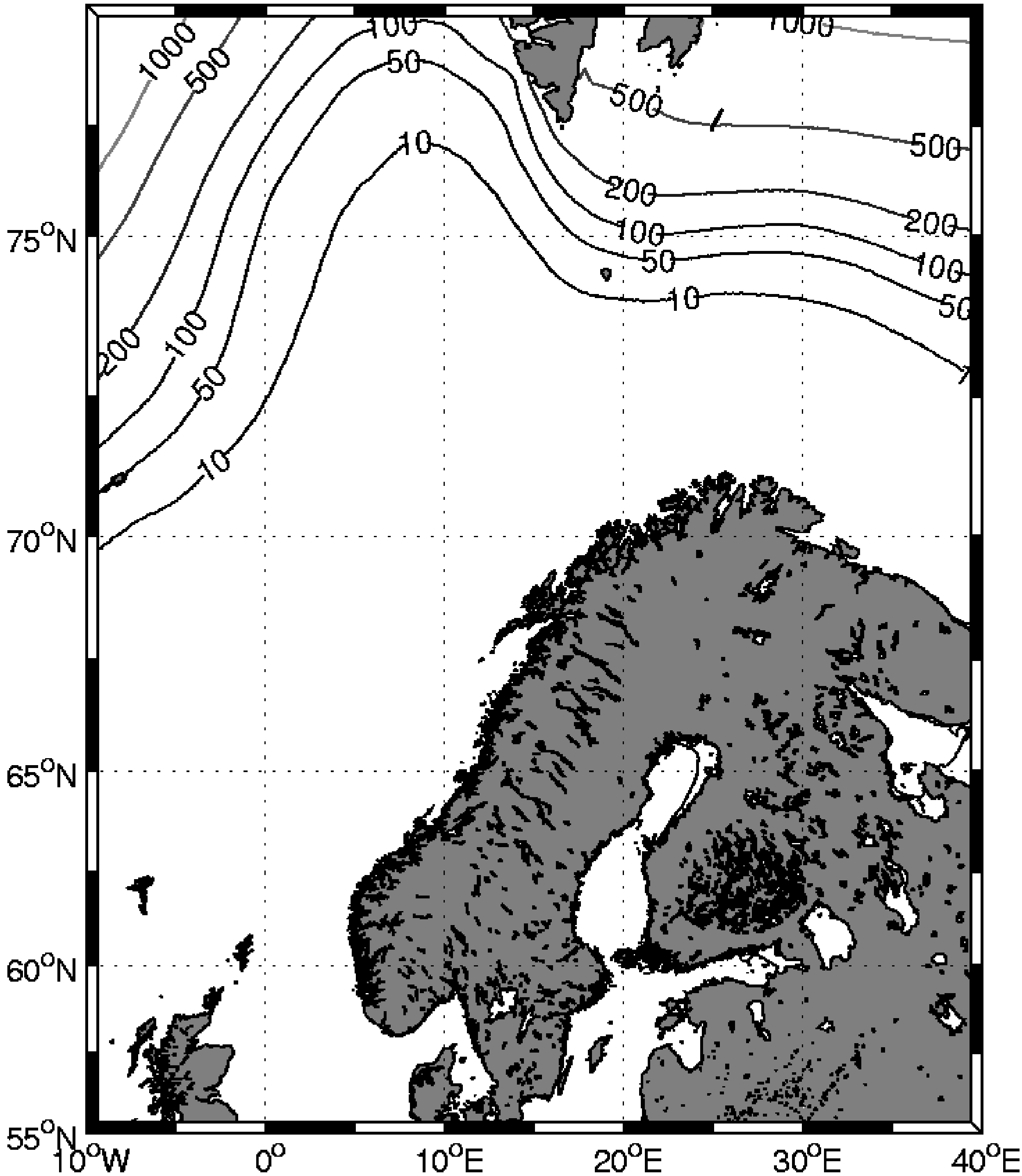

Figure B.5 – Annual mean number of hours with WCT < -25 °C, 30 m above MWL

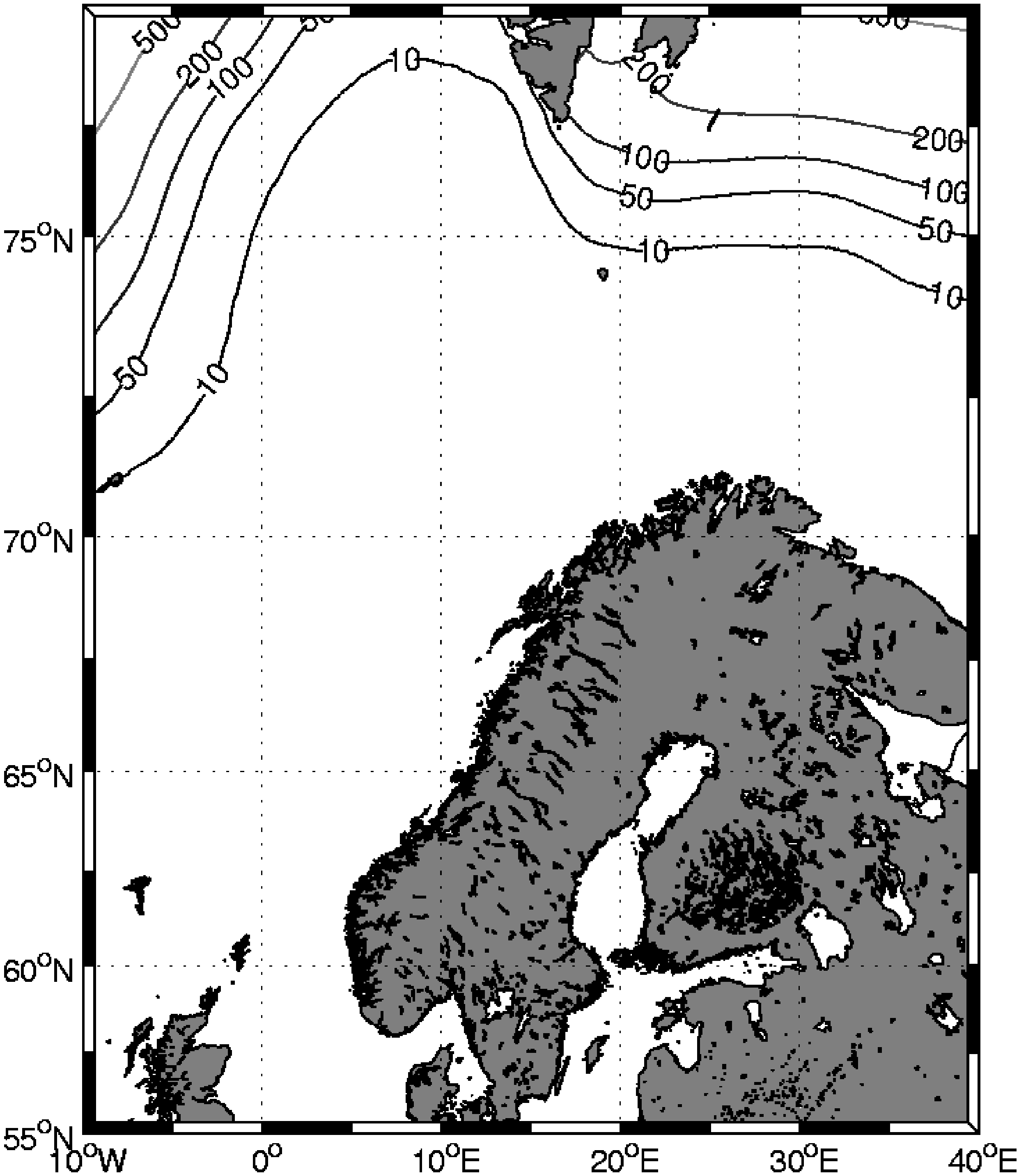

Figure B.6 – Annual mean number of hours with WCT < -30 °C, 30 m above MWL

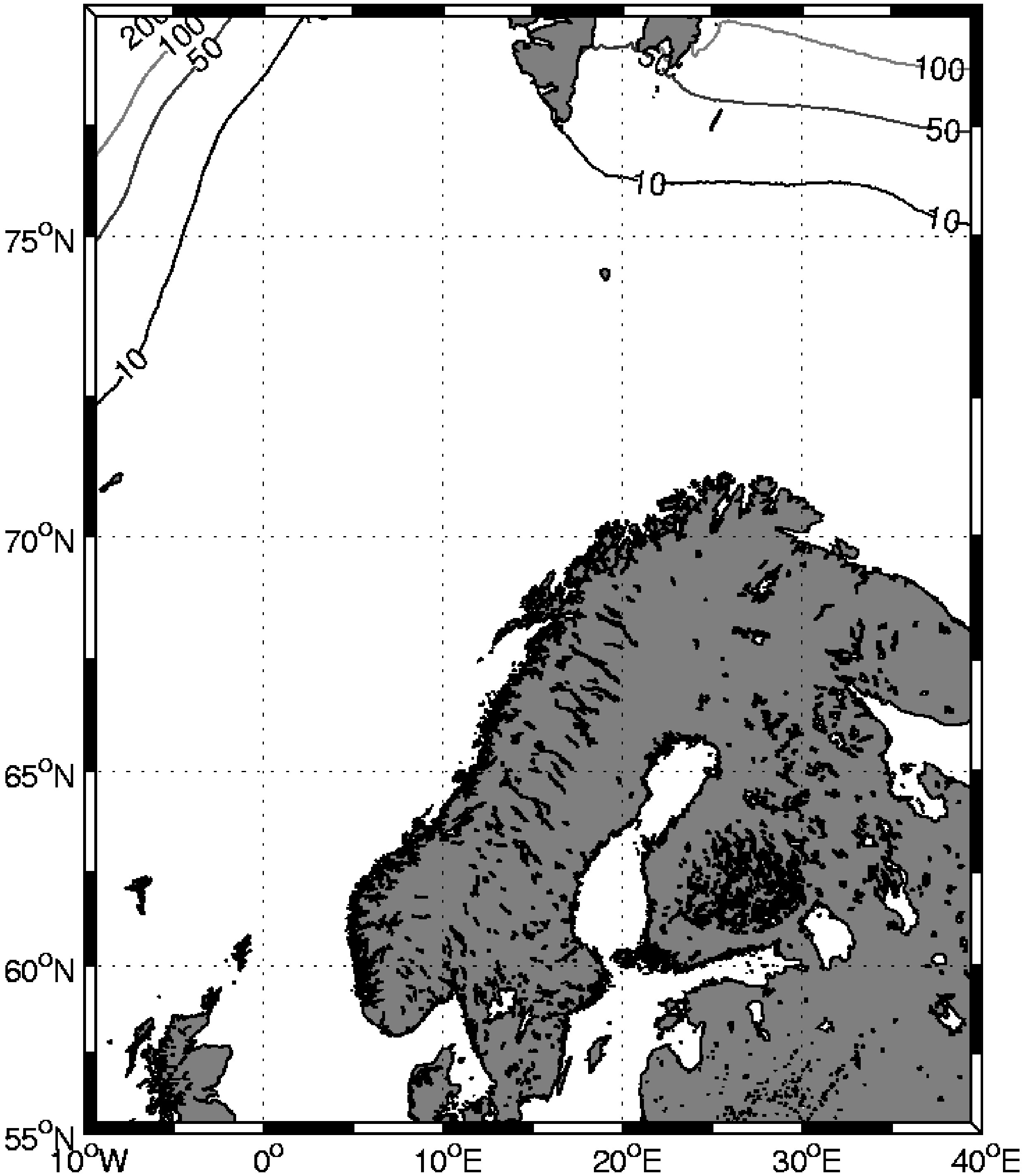

Figure B.7 – Annual mean number of hours with WCT < -35 °C, 30 m above MWL

Annex C
(AC informative~~INFORMATIVT~~ AC)
Maximum and minimum air temperature on the Norwegian Continental shelf

C.1 Maximum and minimum air temperature on the Norwegian Continental shelf

Figures C.1 to C.27 give air temperatures at the conditions indicated in the figure legend.

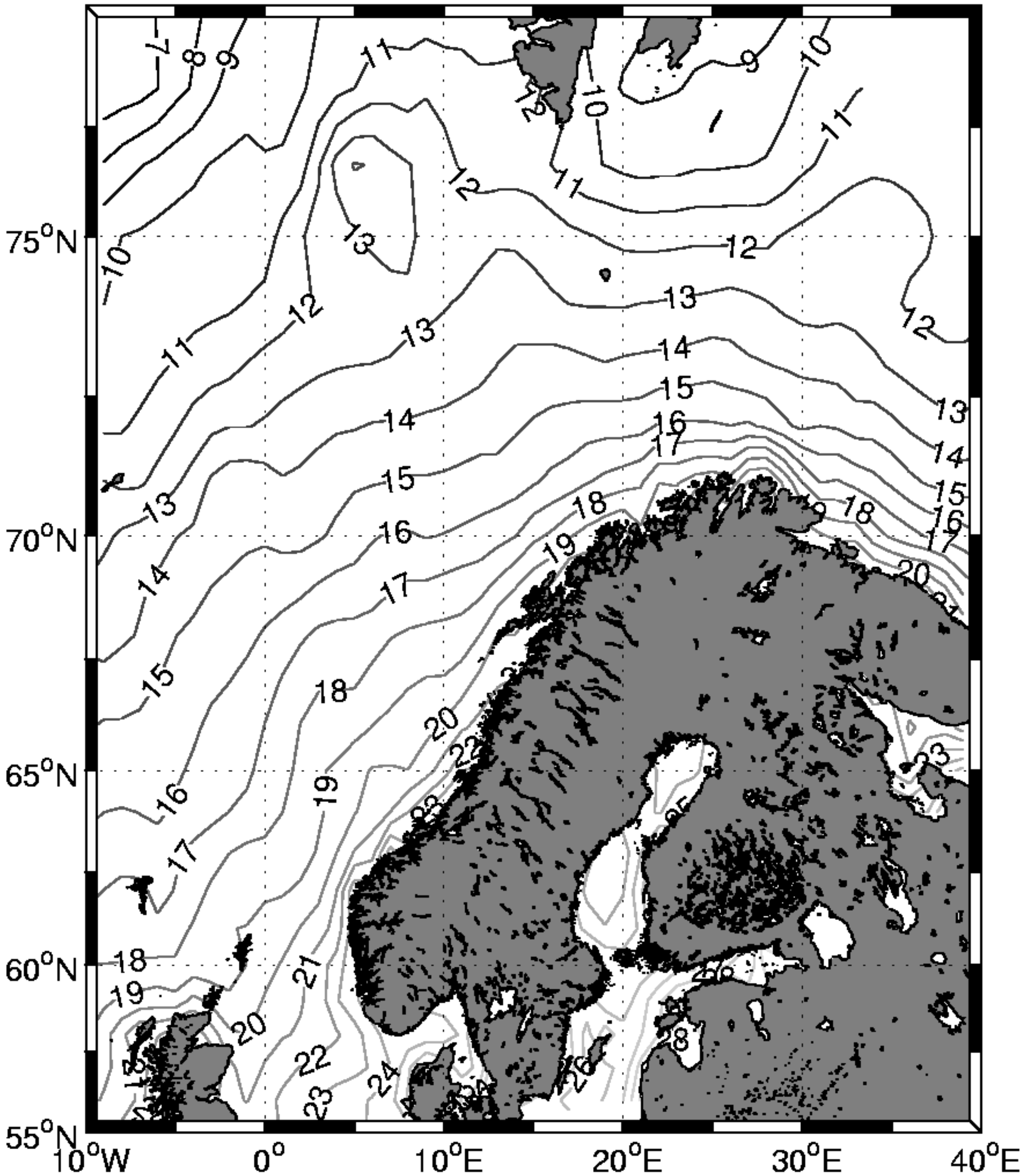

Figure C.1 – Maximum temperature 1-hour duration at 2 meters above SWL with annual probability of exceedance of 10^{-2} on the Norwegian Continental Shelf (= maximum in database +2 °), data period 1958–2011

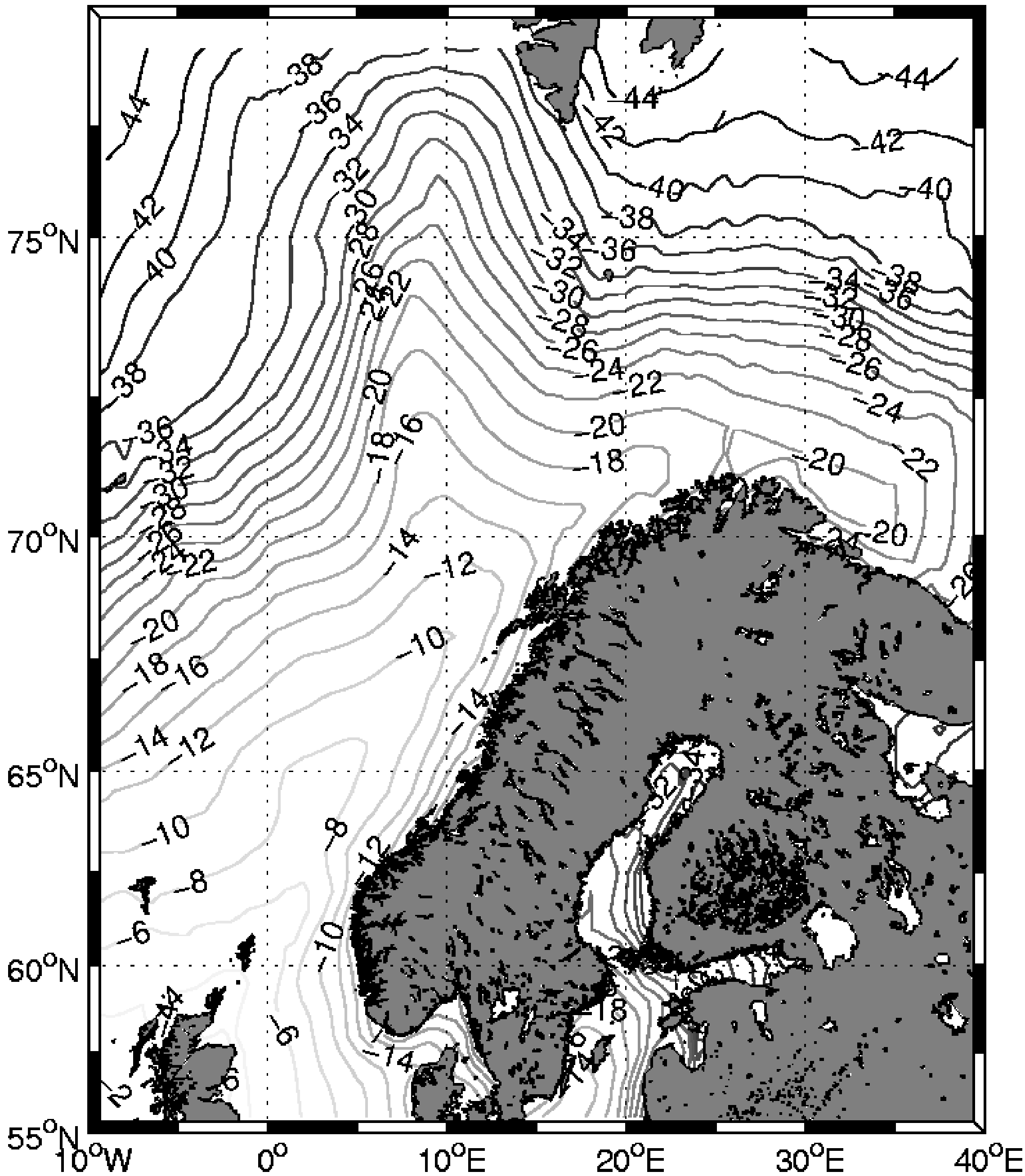

Figure C.2 – Minimum temperature 1-hour duration at 2 meters above SWL with annual probability of exceedance of 10^{-2} on the Norwegian Continental Shelf (= maximum in database +2 °), data period 1958–2011

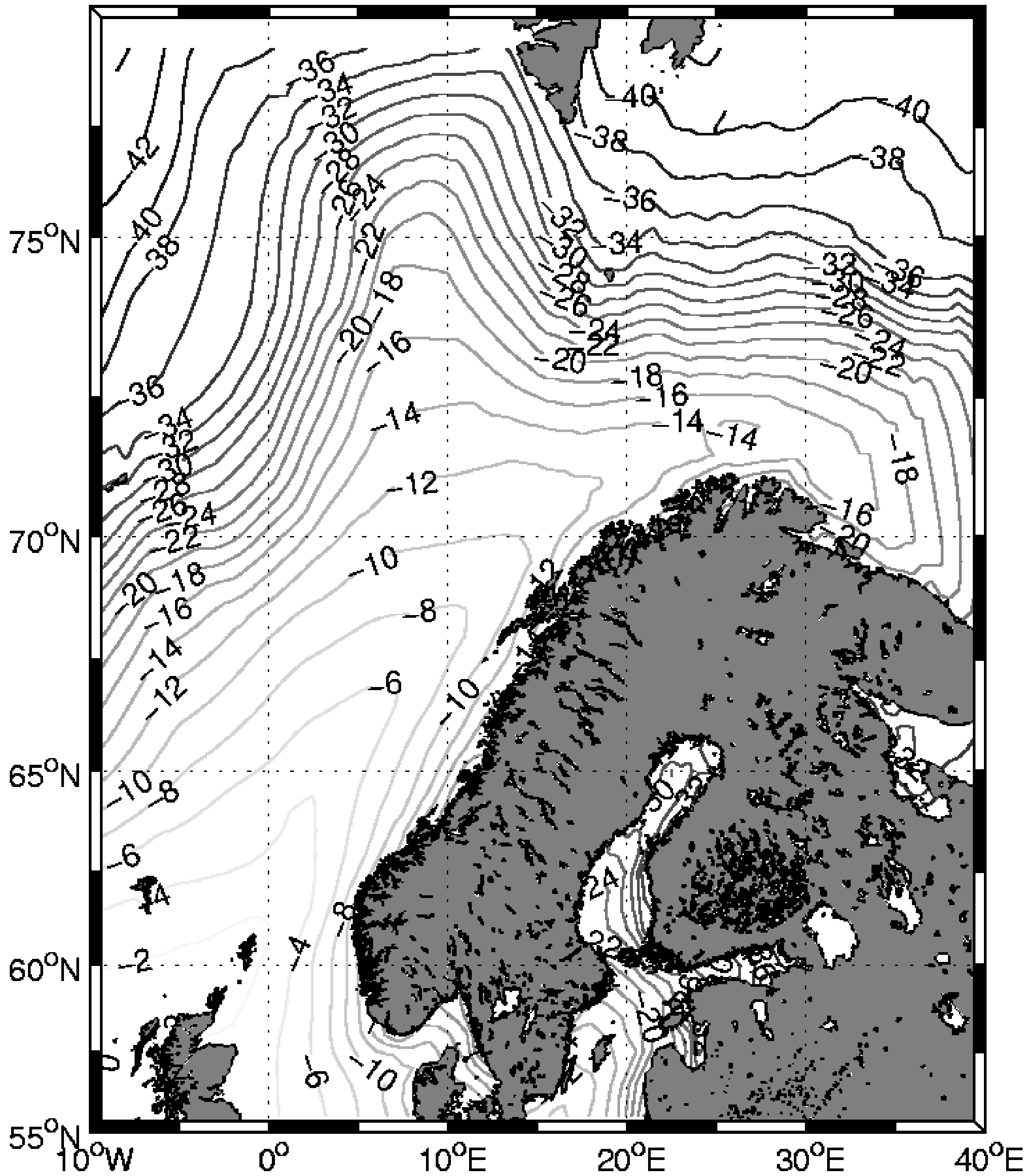

Figure C.3 – Minimum temperature 24 hour duration at 2 meters above SWL with annual probability of exceedance of 10^{-2} on the Norwegian Continental Shelf (= maximum in database +2 °), data period 1958–2011

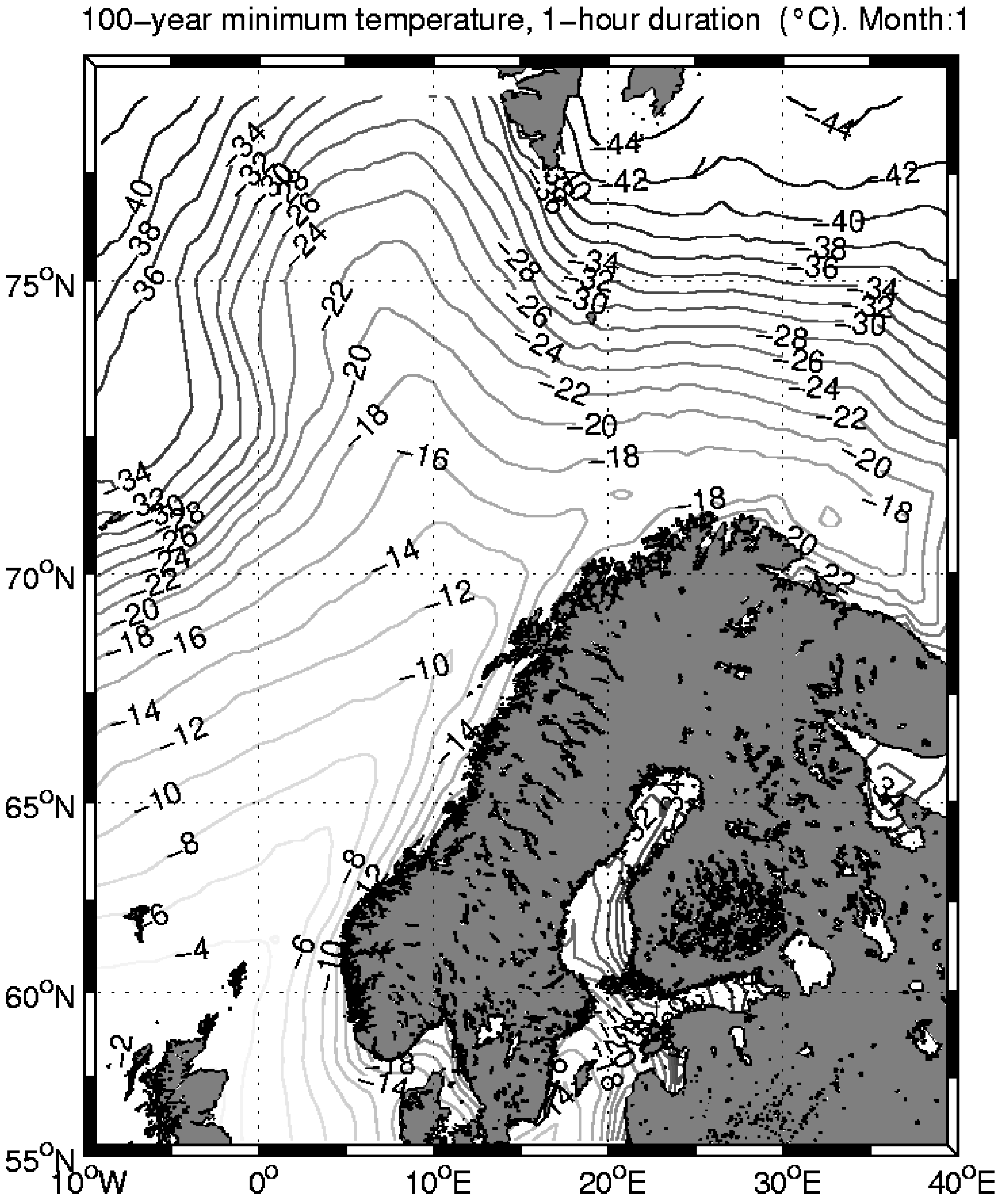

Figure C.4 – January: 1-hour duration minimum temperature at 2 meters above SWL on the Norwegian Continental Shelf (= minimum in database -2 °), data period 1958–2011

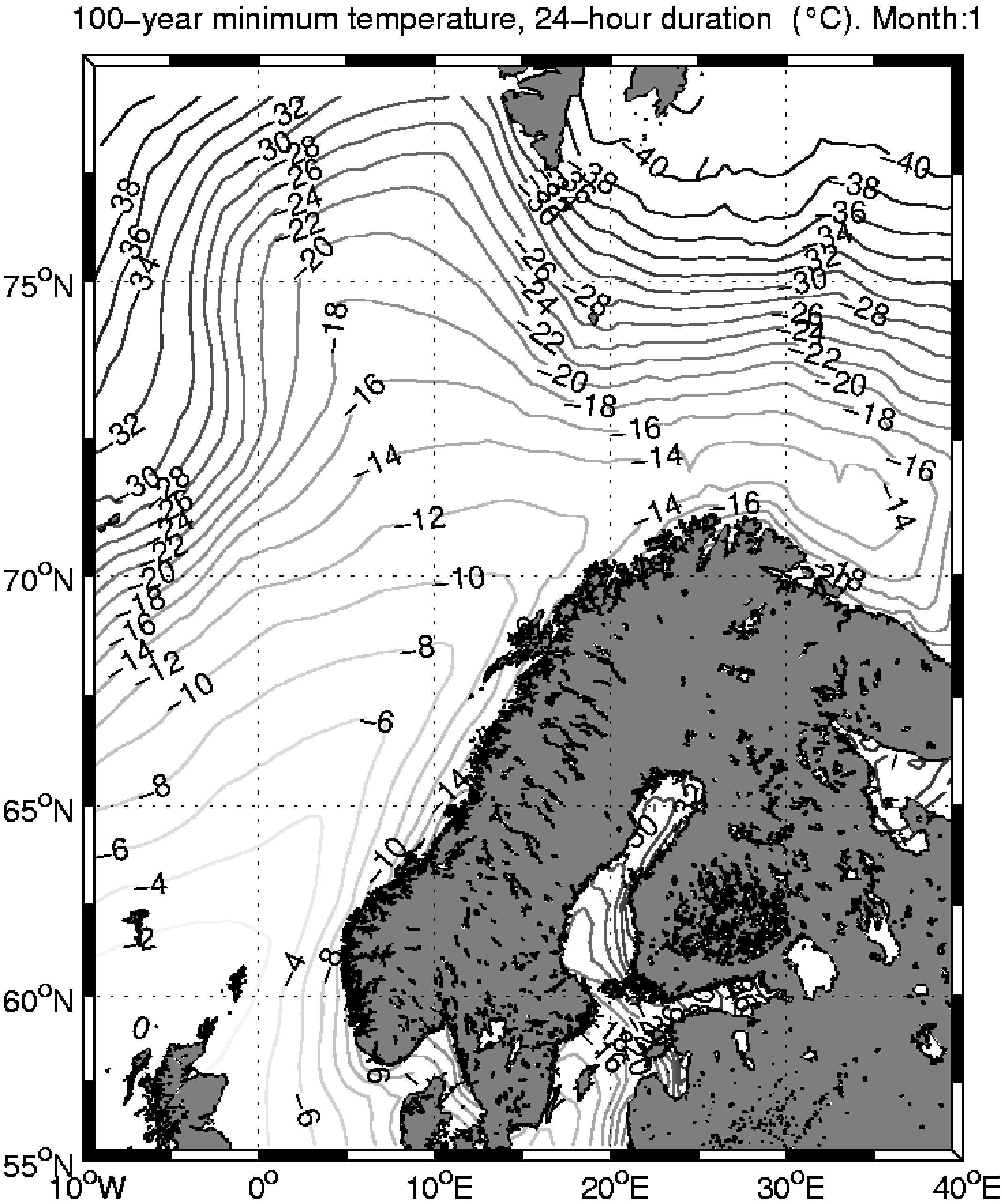

Figure C.5 – January: 24-hour duration minimum temperature at 2 meters above SWL on the Norwegian Continental Shelf (= minimum in database -2 °), data period 1958–2011

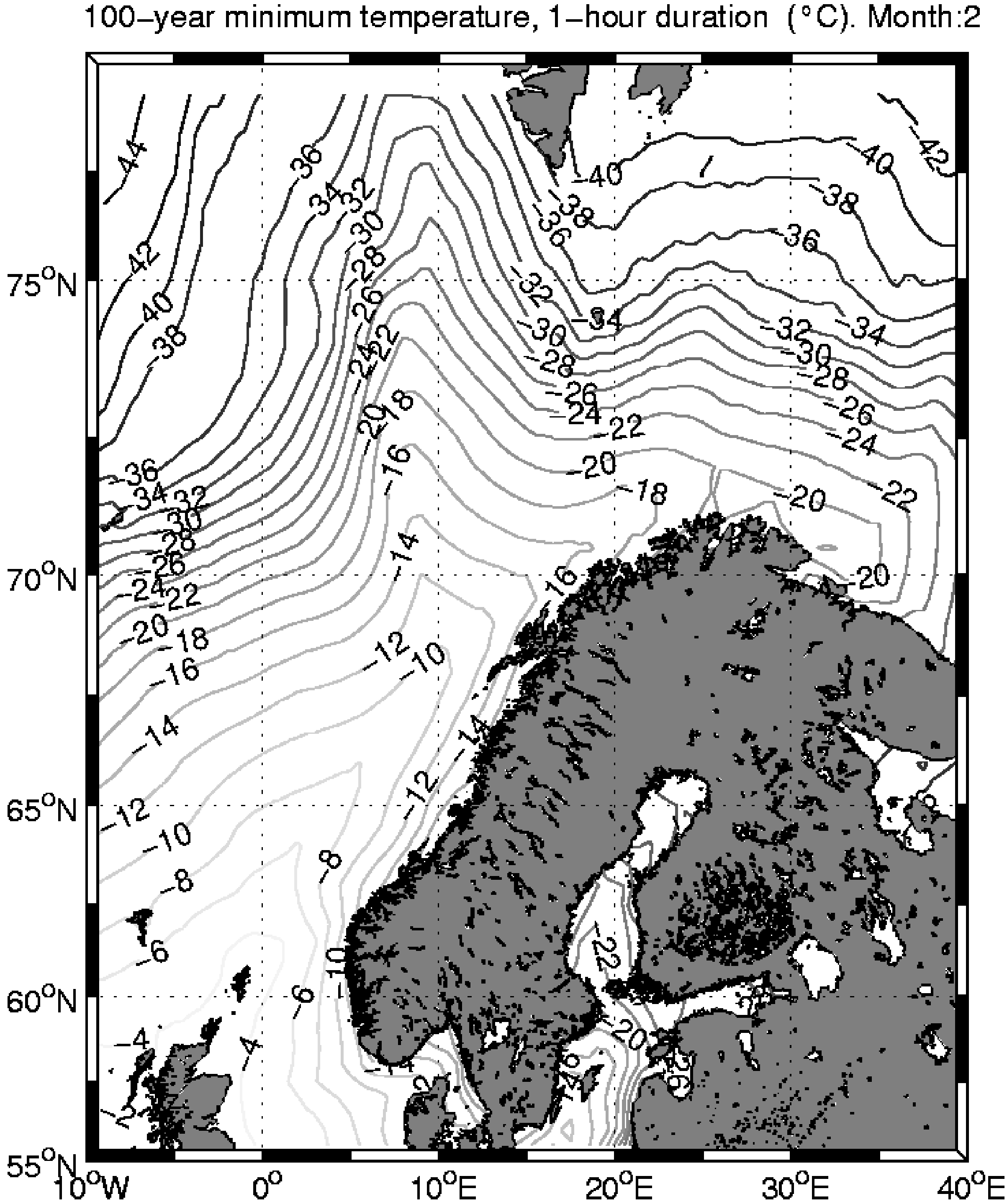

Figure C.6 – February: 1-hour duration minimum temperature at 2 meters above SWL on the Norwegian Continental Shelf (= minimum in database -2 °), data period 1958–2011

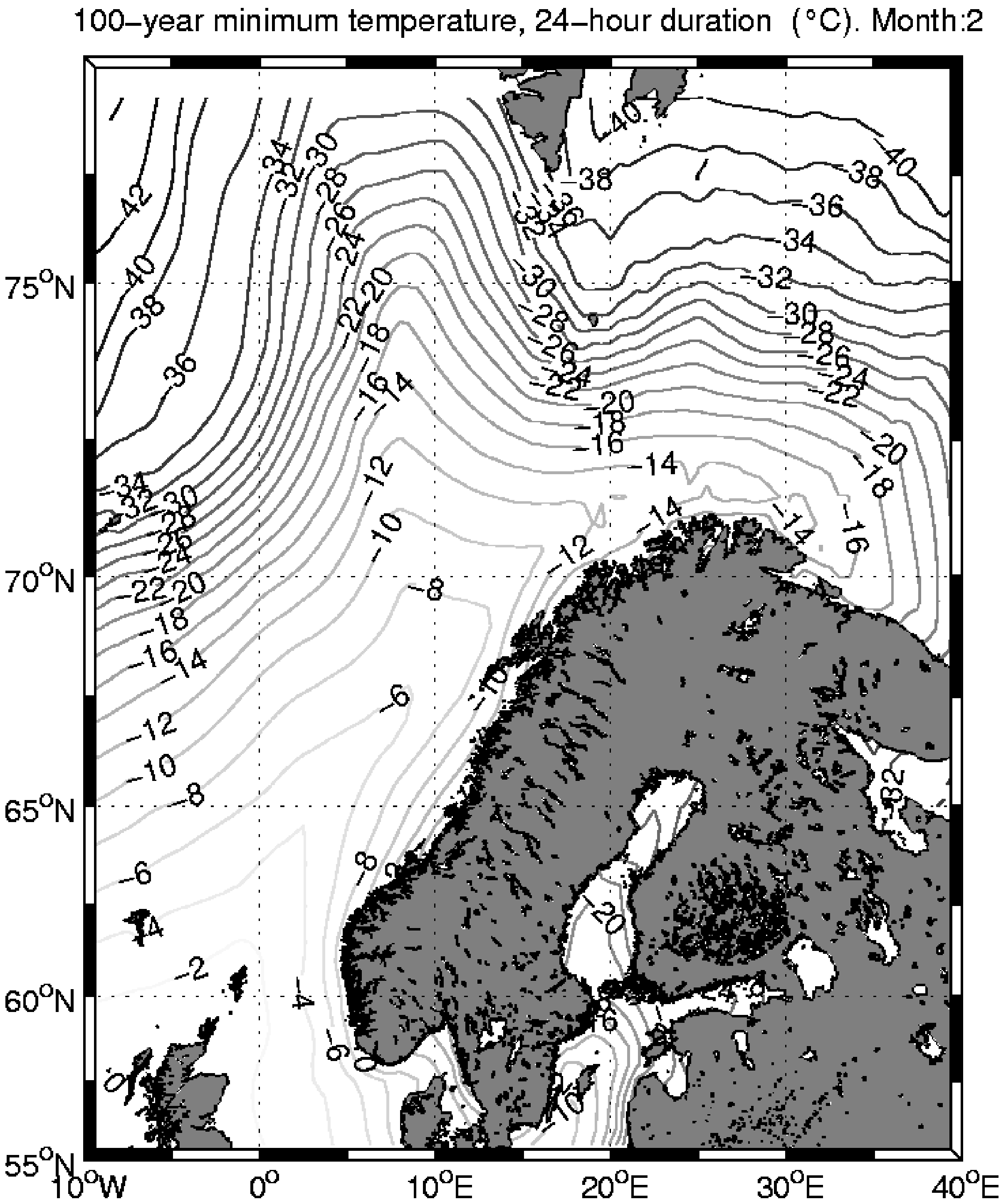

Figure C.7 – February: 24-hour duration minimum temperature at 2 meters above SWL on the Norwegian Continental Shelf (= minimum in database -2 °), data period 1958–2011

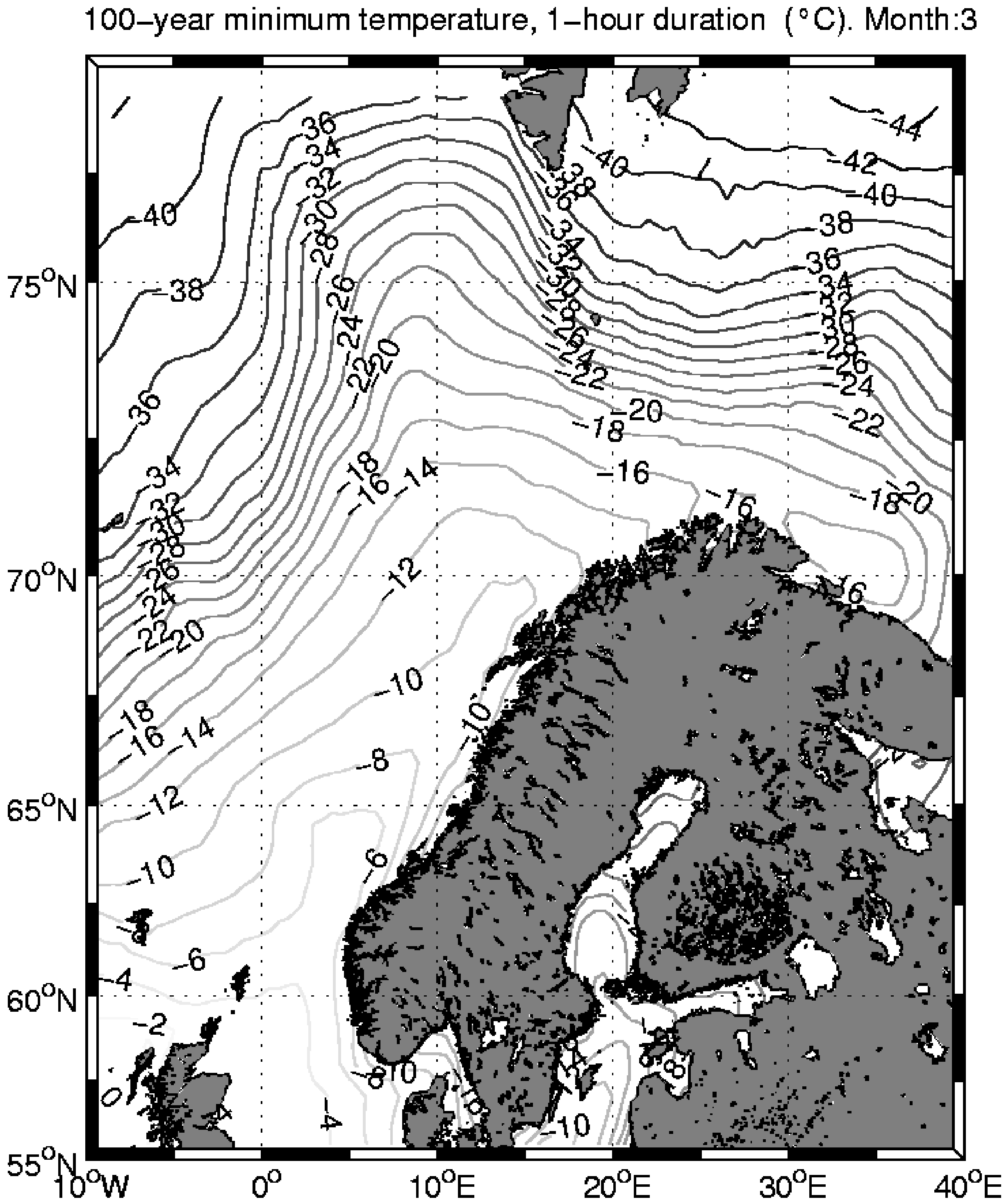

Figure C.8 – March: 1-hour duration minimum temperature at 2 meters above SWL on the Norwegian Continental Shelf (= minimum in database -2 °), data period 1958 – 2011

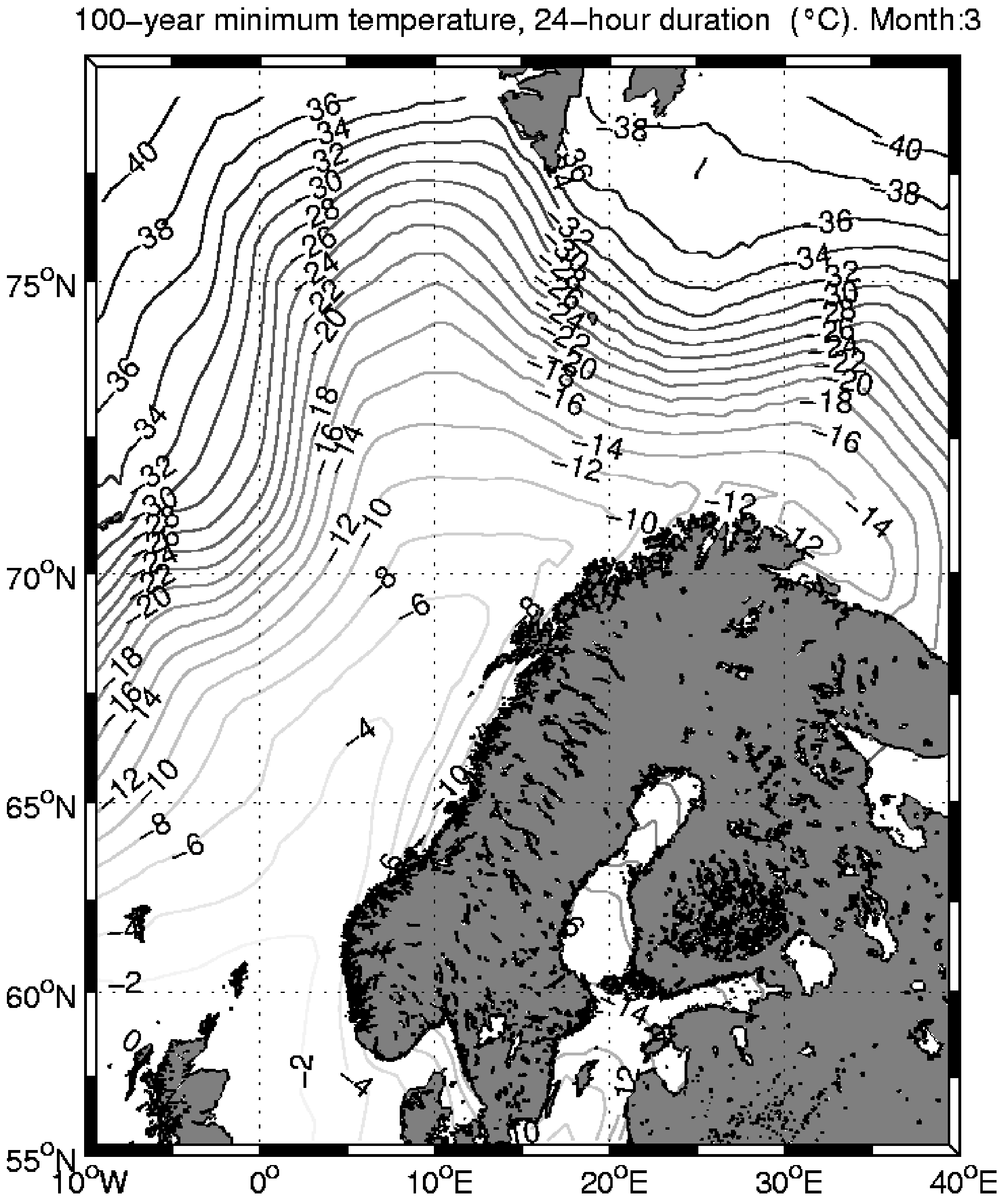

Figure C.9 – March: 24-hour duration minimum temperature at 2 meters above SWL on the Norwegian Continental Shelf (= minimum in database -2 °), data period 1958–2011

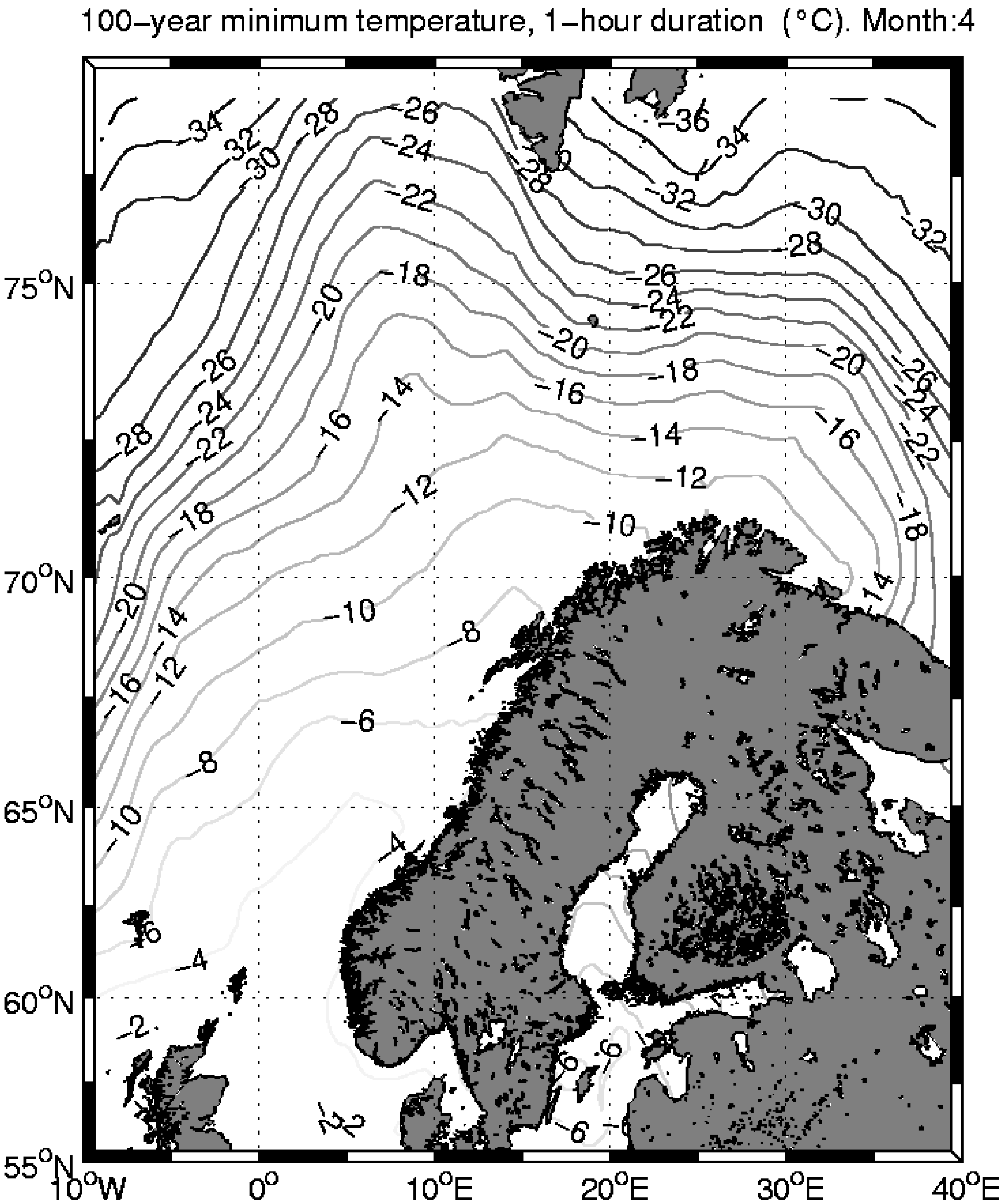

Figure C.10 – April: 1-hour duration minimum temperature at 2 meters above SWL on the Norwegian Continental Shelf (= minimum in database -2 °), data period 1958–2011

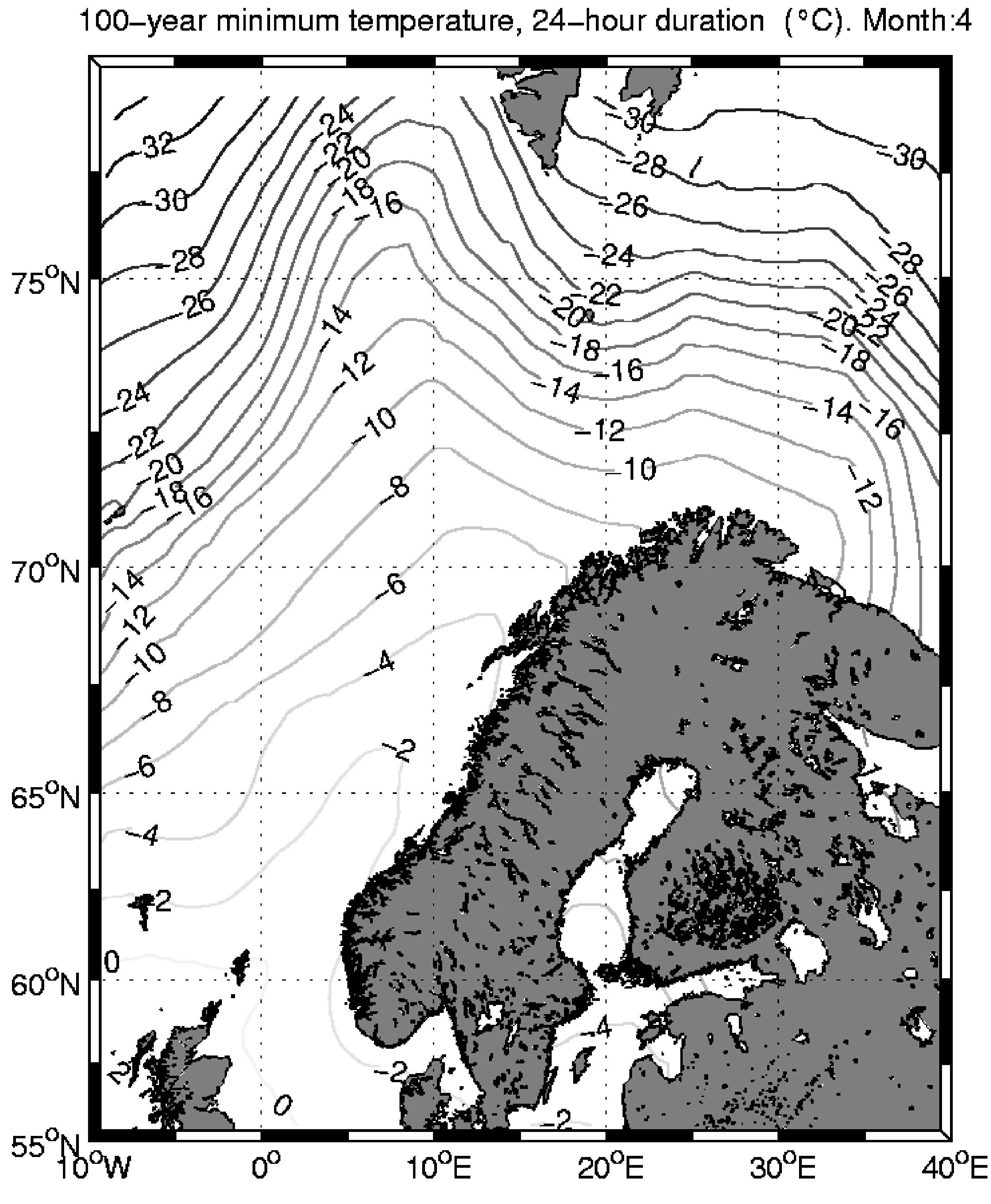

Figure C.11 – April: 24-hour duration minimum temperature at 2 meters above SWL on the Norwegian Continental Shelf (= minimum in database -2 °, data period 1958–2011

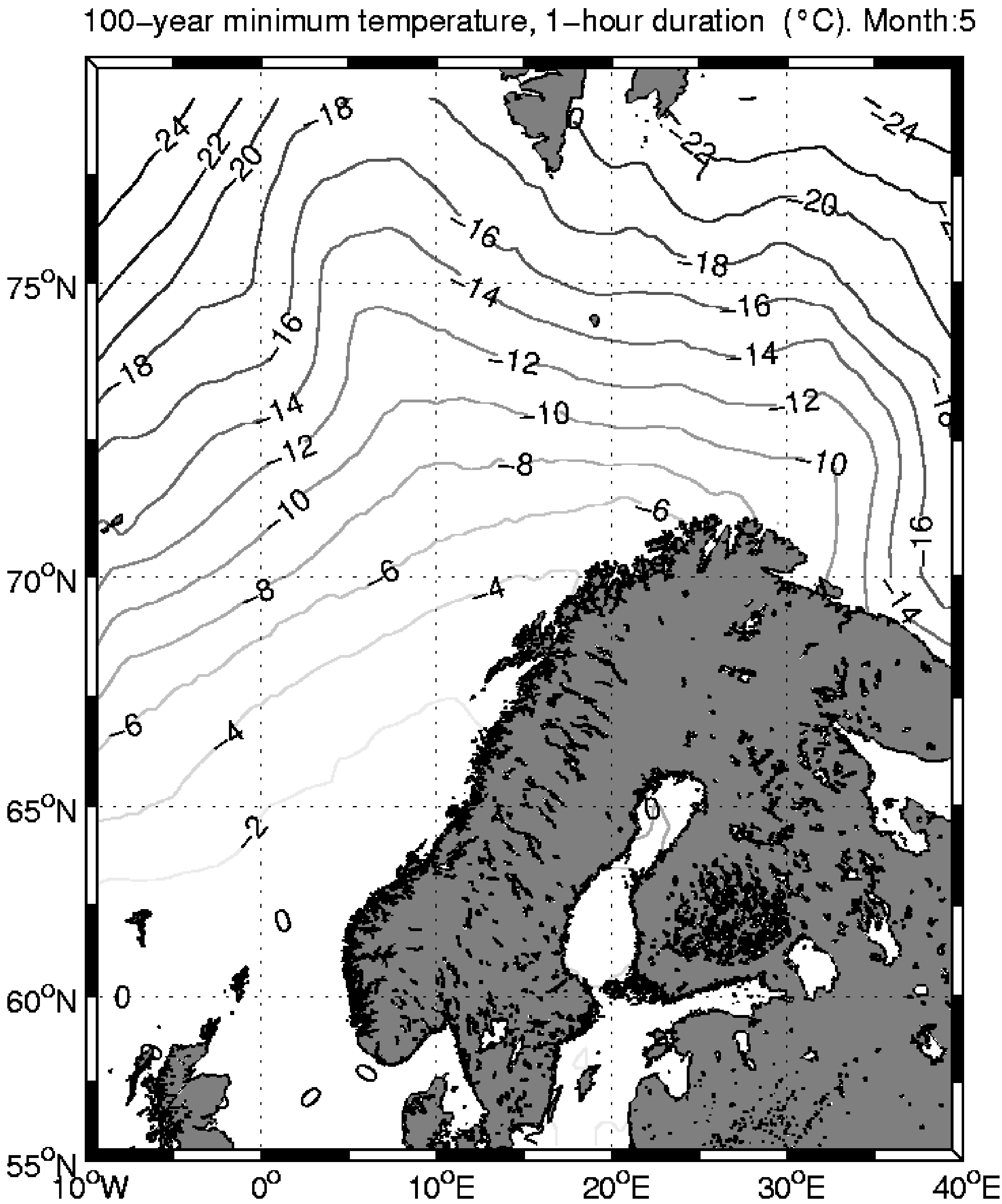

Figure C.12 – May: 1-hour duration minimum temperature at 2 meters above SWL on the Norwegian Continental Shelf (= minimum in database -2 °), data period 1958–2011

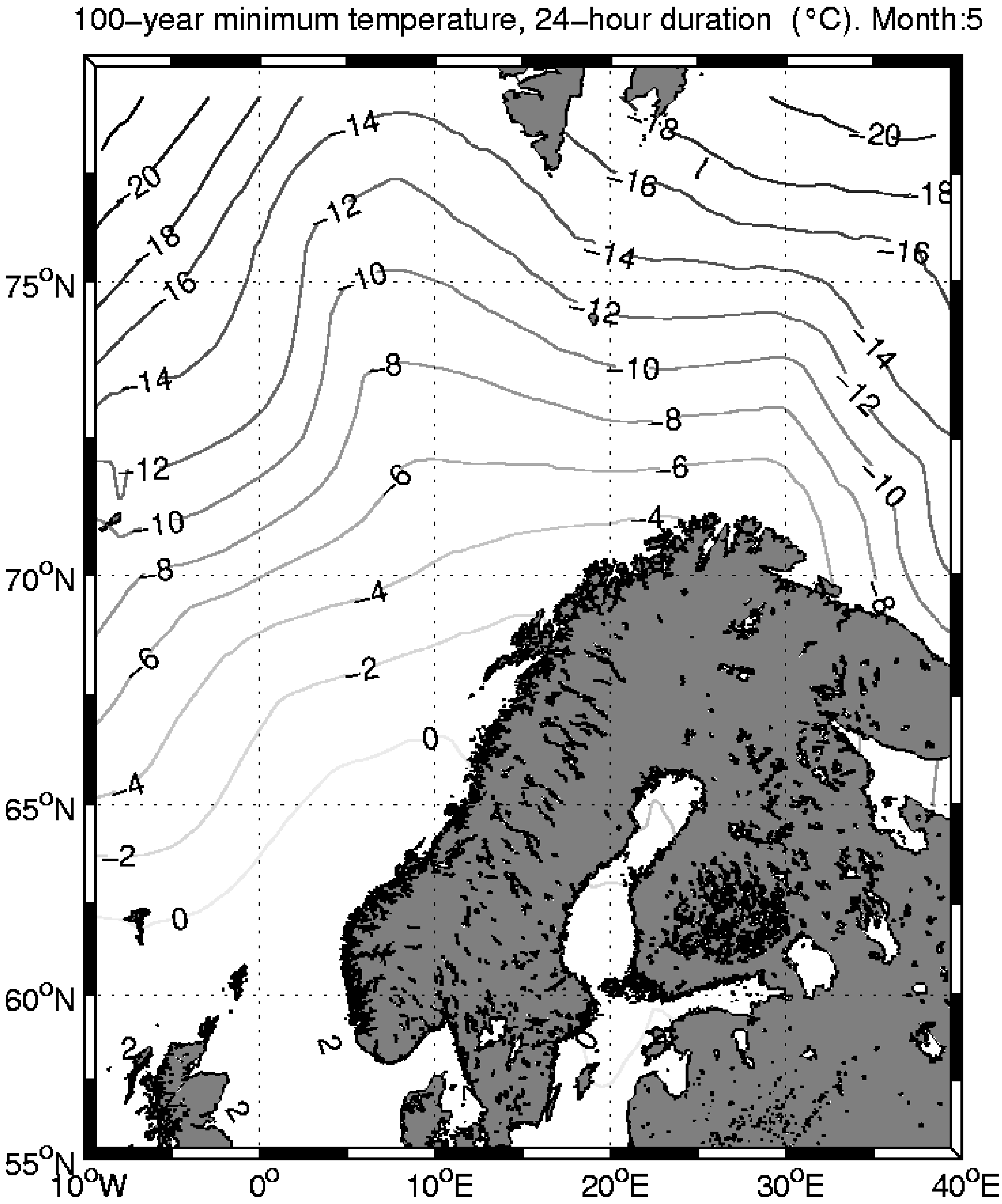

Figure C.13 – May: 24-hour duration minimum temperature at 2 meters above SWL on the Norwegian Continental Shelf (= minimum in database -2 °), data period 1958–2011

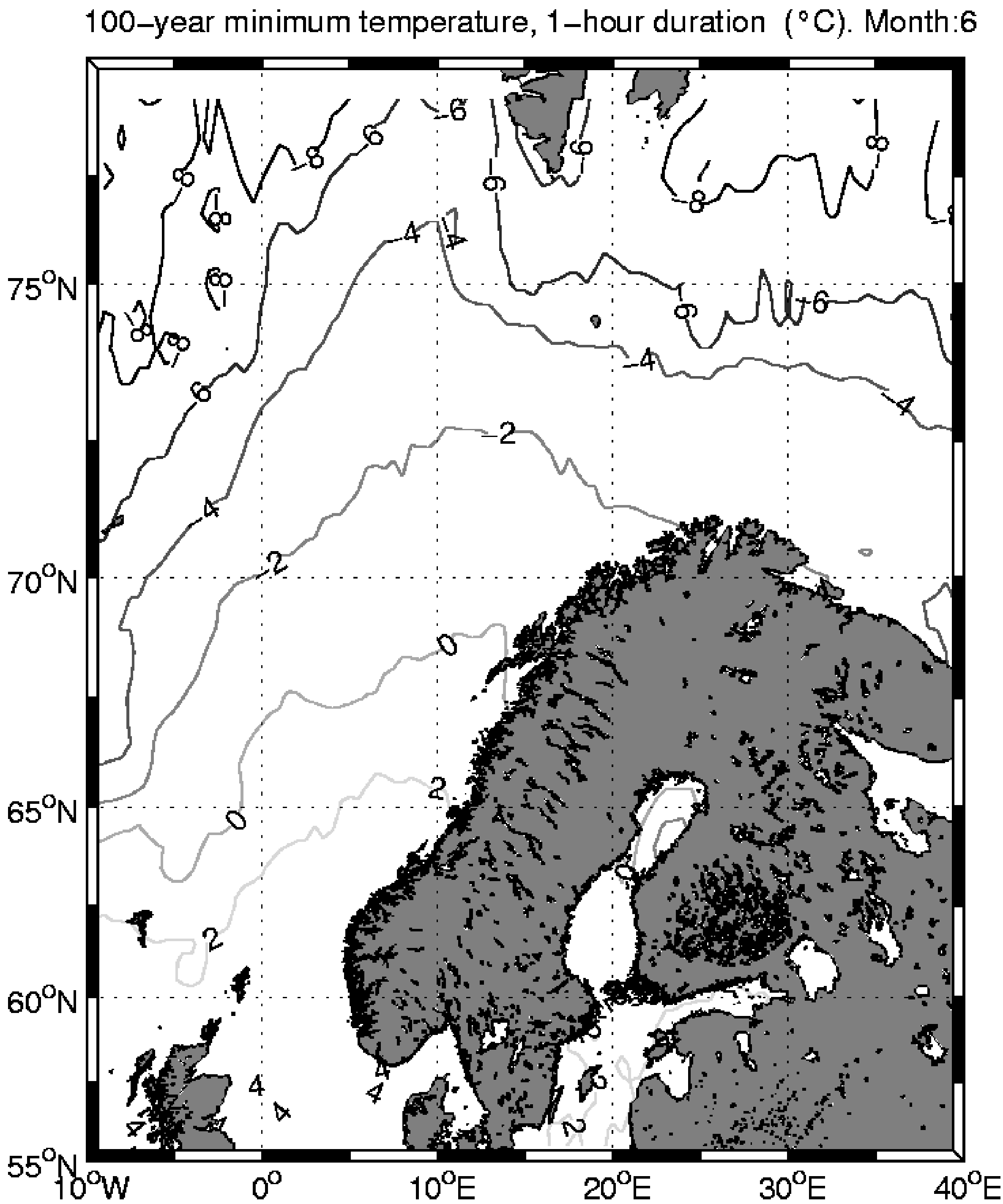

Figure C.14 – June: 1-hour duration minimum temperature at 2 meters above SWL on the Norwegian Continental Shelf (= minimum in database -2 °), data period 1958–2011

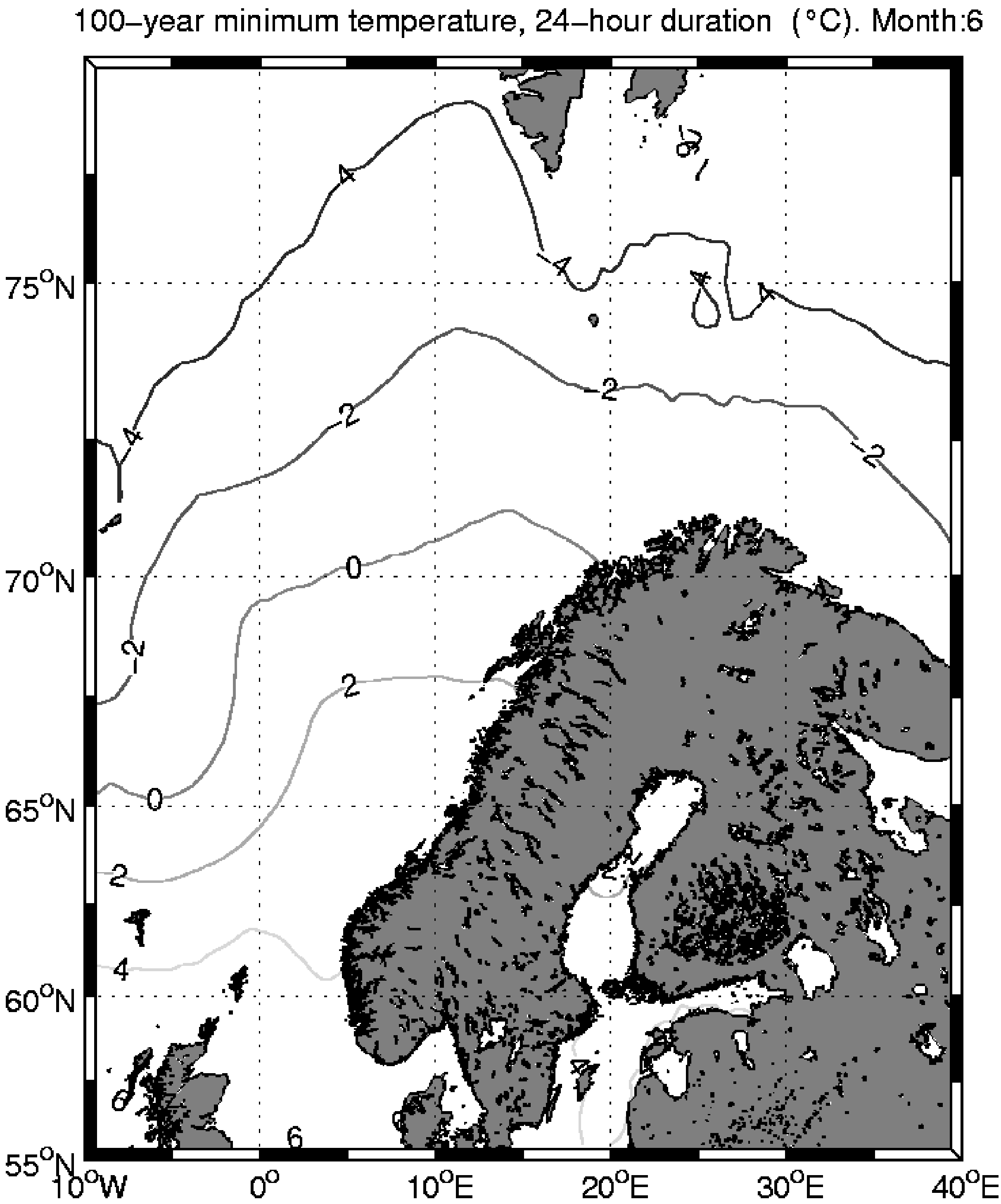

Figure C.15 – June: 24-hour duration minimum temperature at 2 meters above SWL on the Norwegian Continental Shelf (= minimum in database -2 °), data period 1958–2011

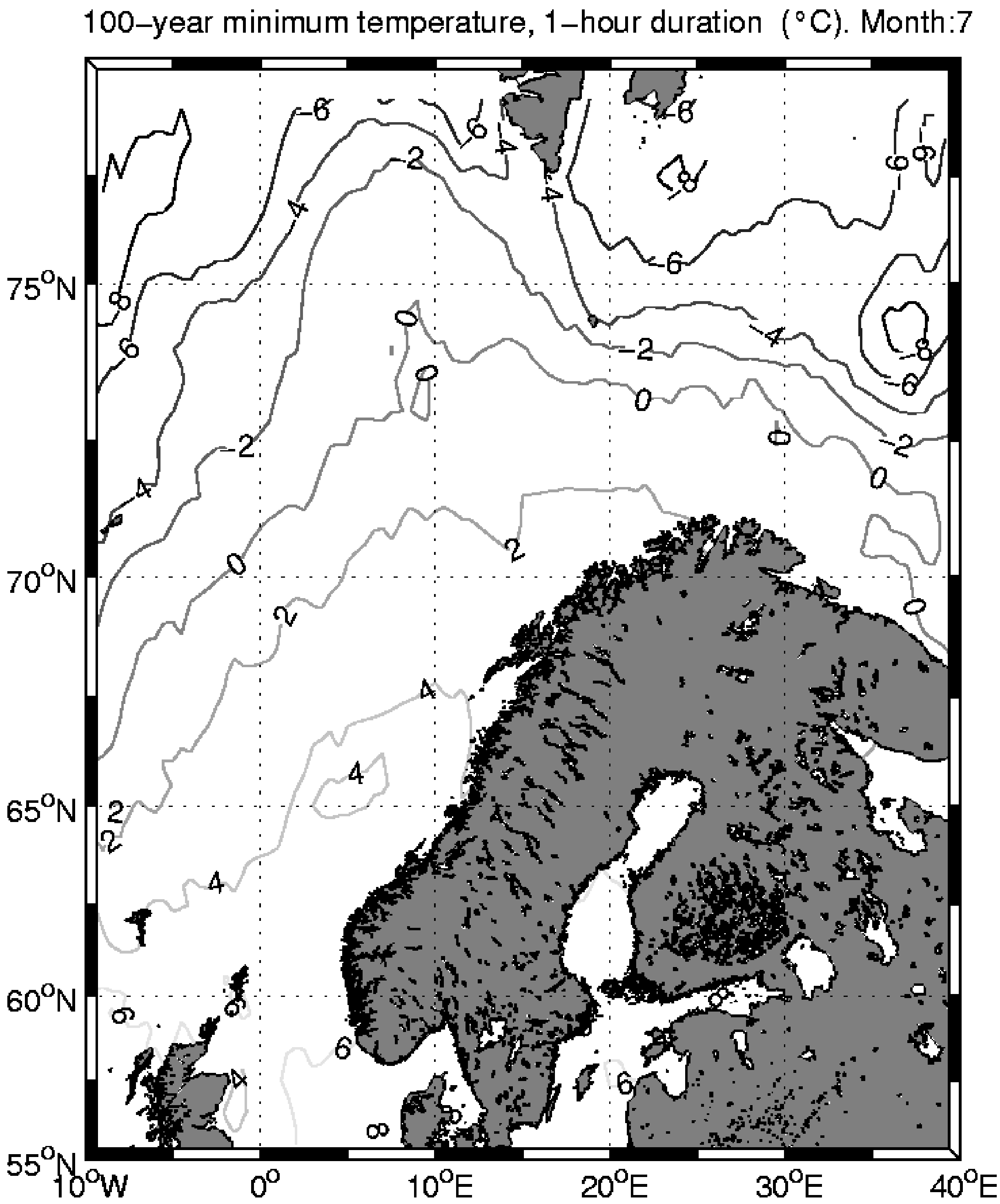

Figure C.16 – July: 1-hour duration minimum temperature at 2 meters above SWL on the Norwegian Continental Shelf (= minimum in database -2 °), data period 1958–2011

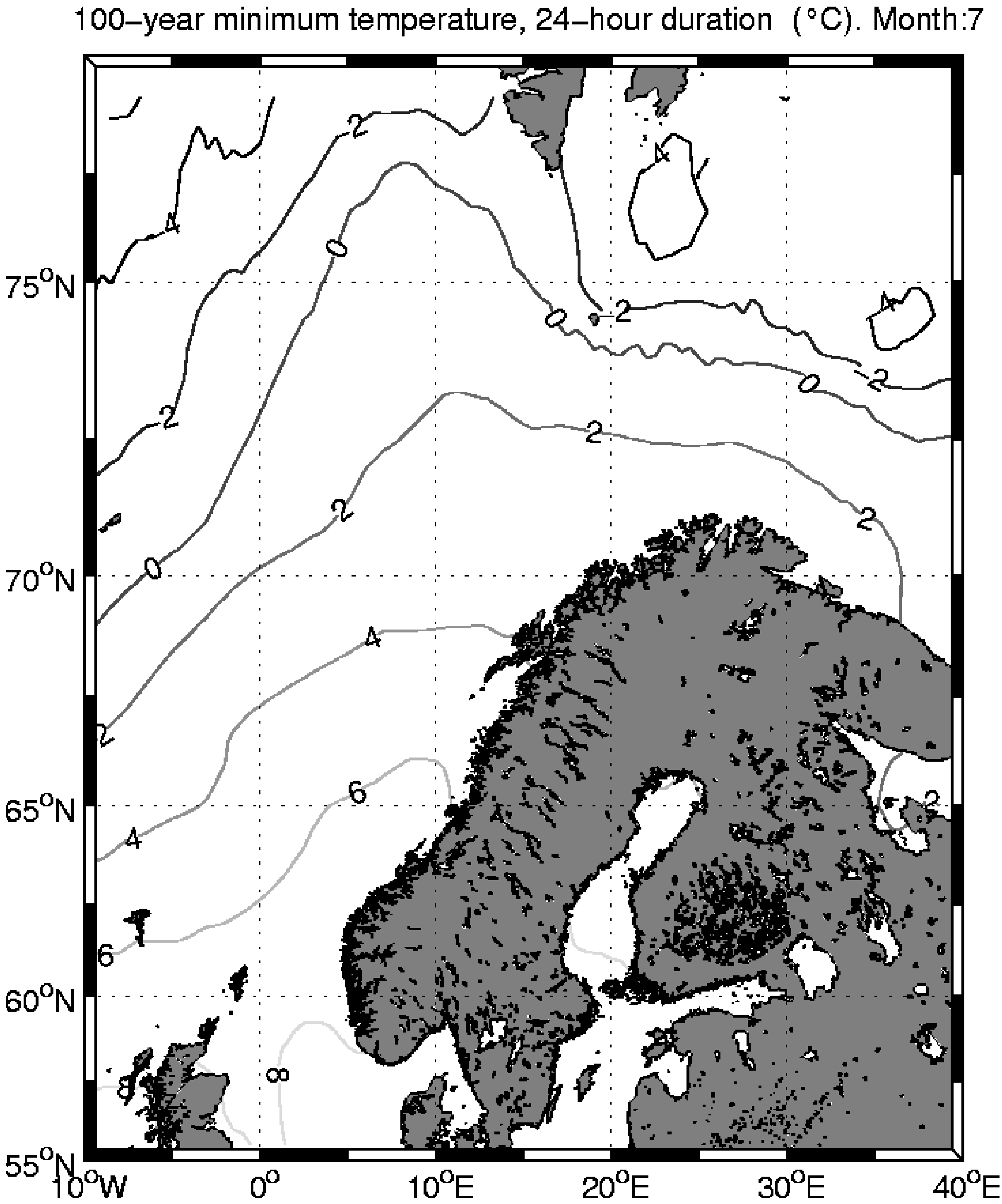

Figure C.17 – July: 24-hour duration minimum temperature at 2 meters above SWL on the Norwegian Continental Shelf (= minimum in database -2 °), data period 1958–2011

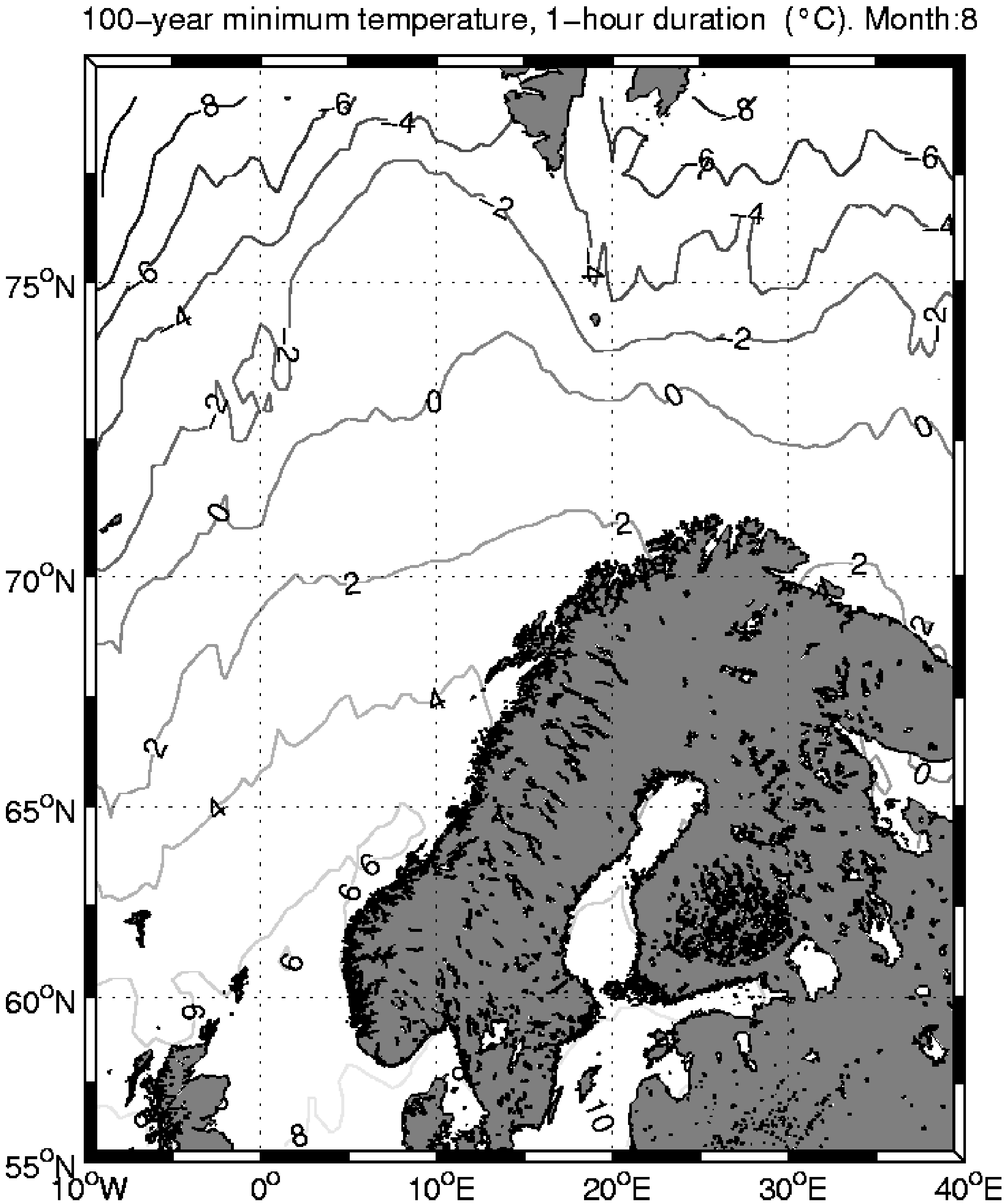

Figure C.18 – August: 1-hour duration minimum temperature at 2 meters above SWL on the Norwegian Continental Shelf (= minimum in database -2 °), data period 1958–2011

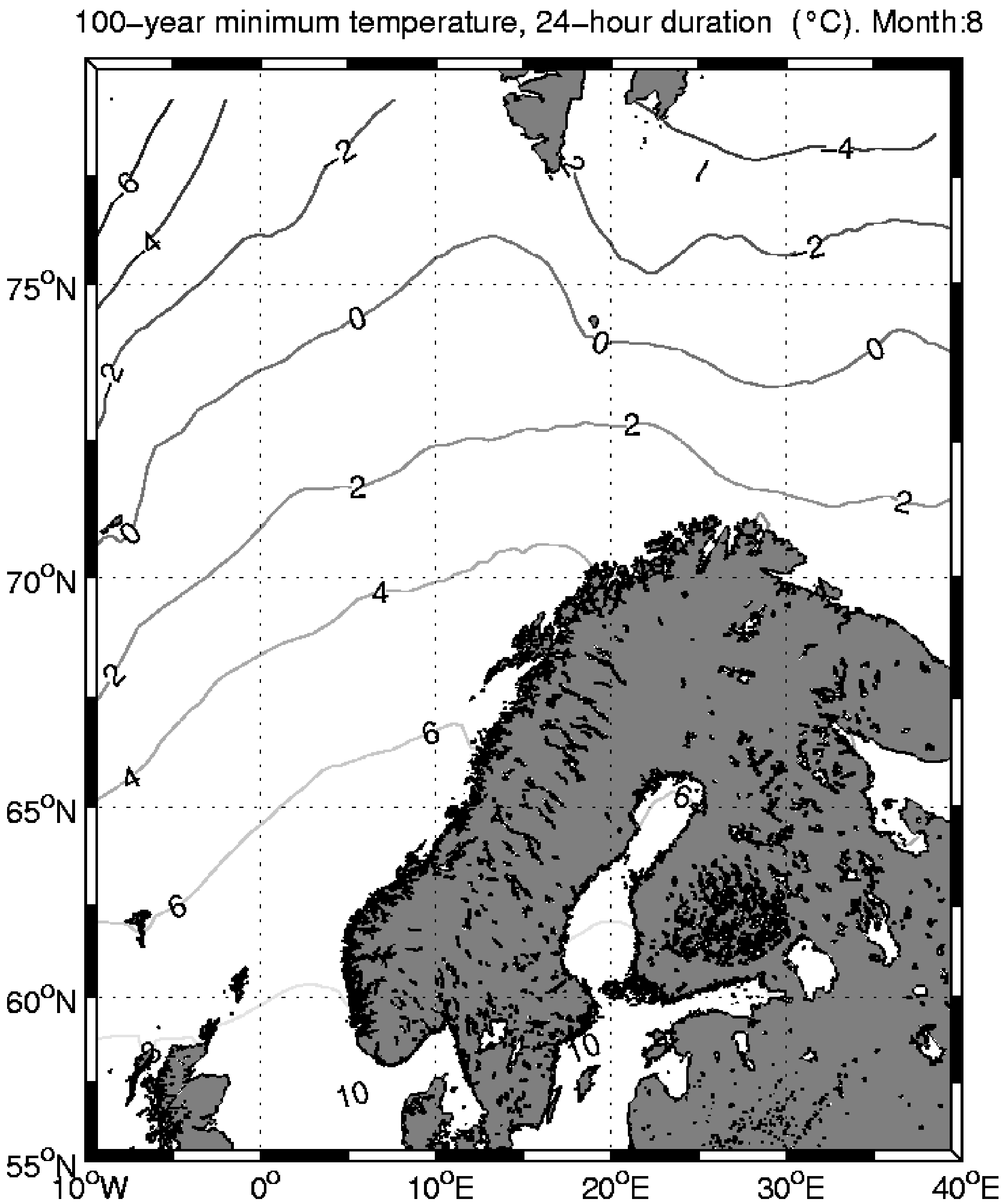

Figure C.19 – August: 24-hour duration minimum temperature at 2 meters above SWL on the Norwegian Continental Shelf (= minimum in database -2 °), data period 1958–2011

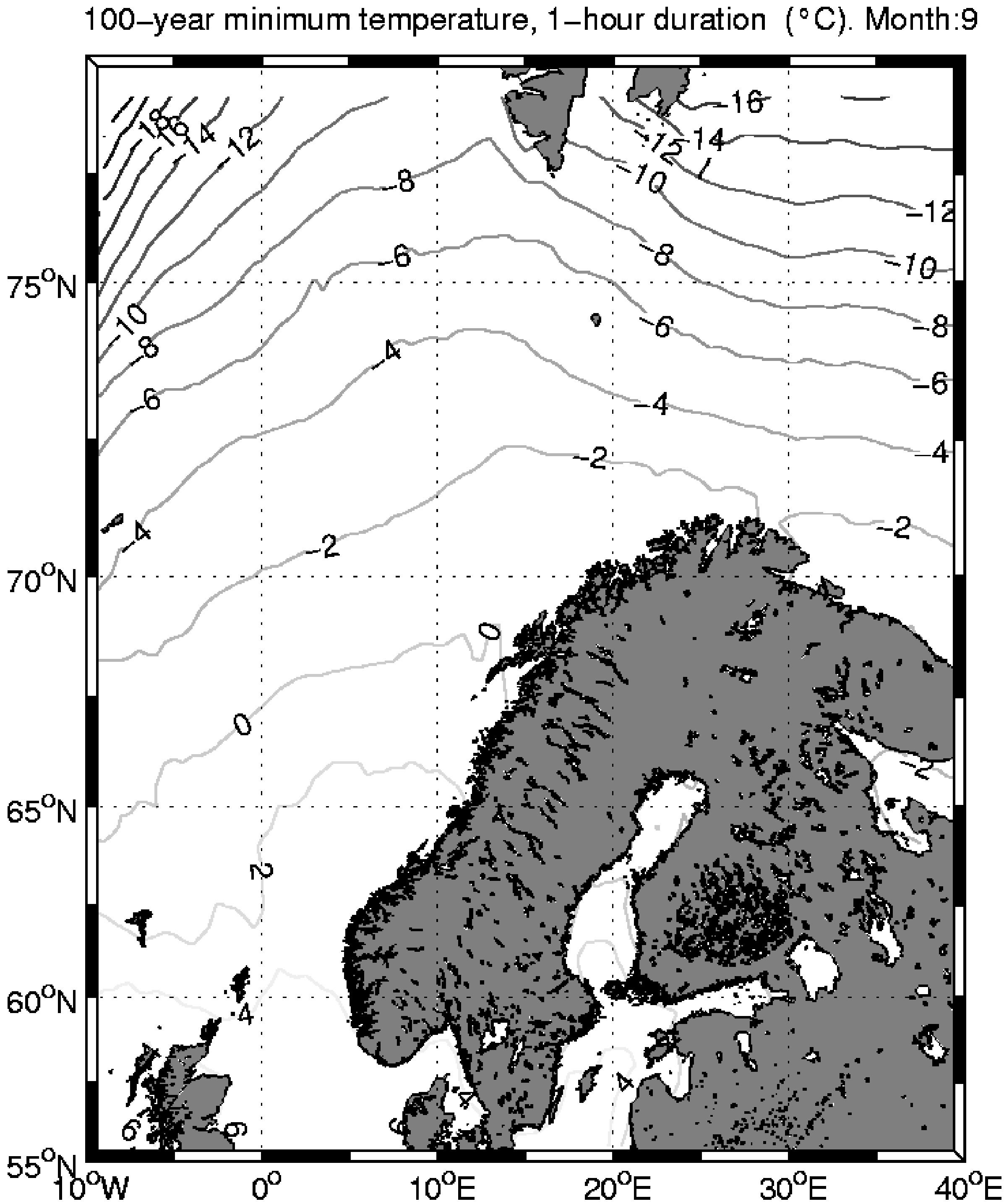

Figure C.20 – September: 1-hour duration minimum temperature at 2 meters above SWL on the Norwegian Continental Shelf (= minimum in database -2 °), data period 1958–2011

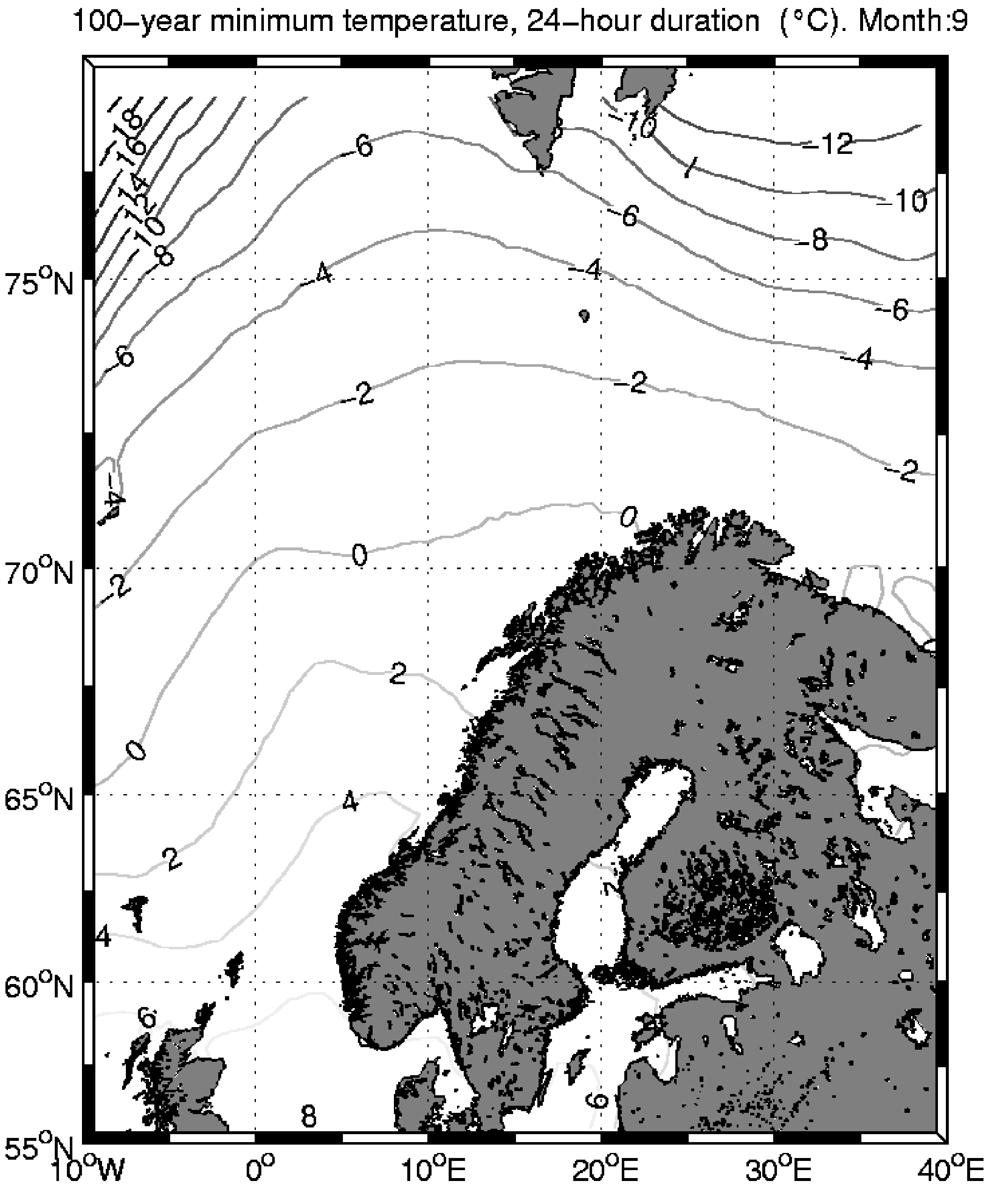

Figure C.21 – September: 24-hour duration minimum temperature at 2 meters above SWL on the Norwegian Continental Shelf (= minimum in database -2 °), data period 1958–2011

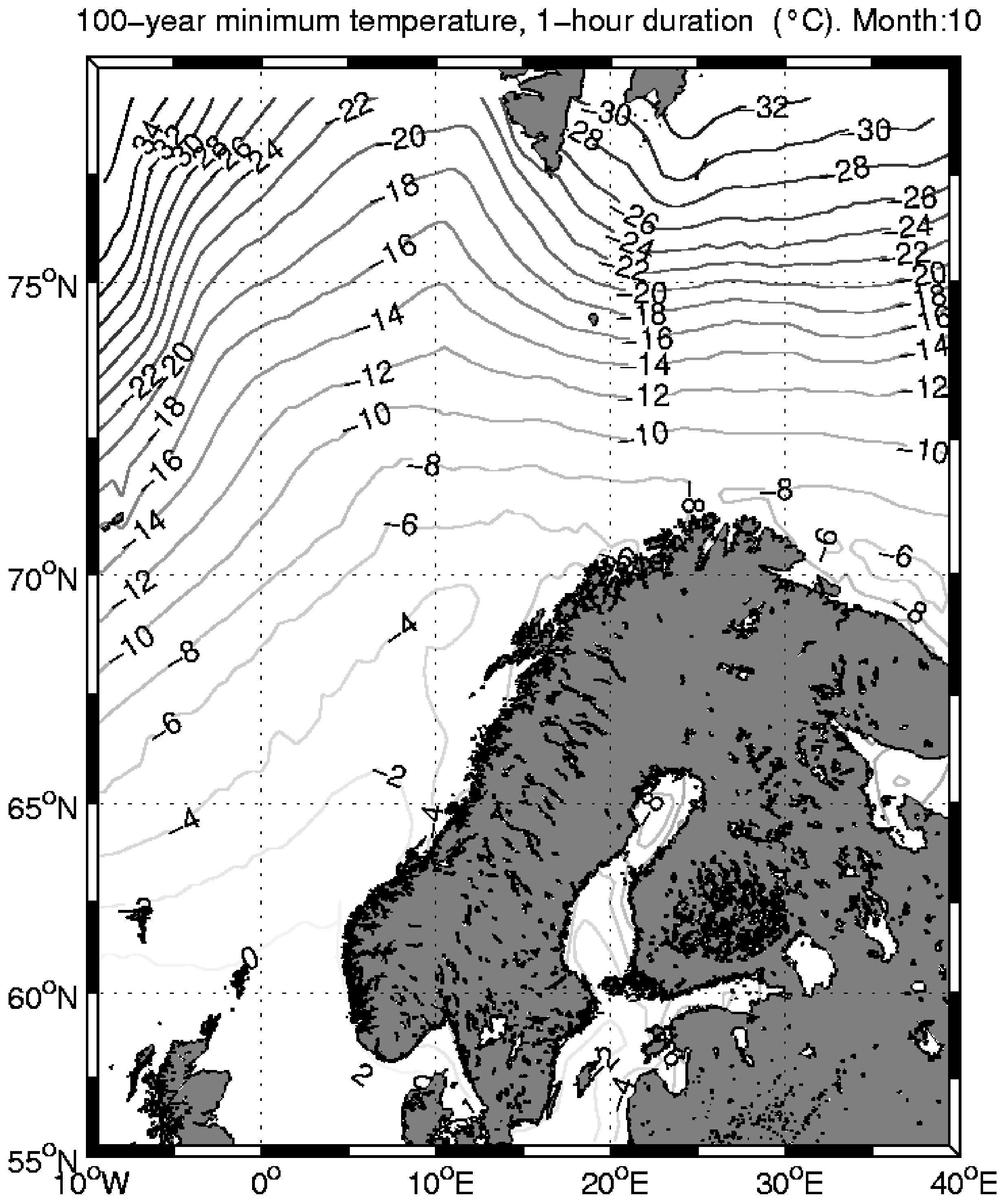

Figure C.22 – October: 1-hour duration minimum temperature at 2 meters above SWL on the Norwegian Continental Shelf (= minimum in database -2 °), data period 1958–2011

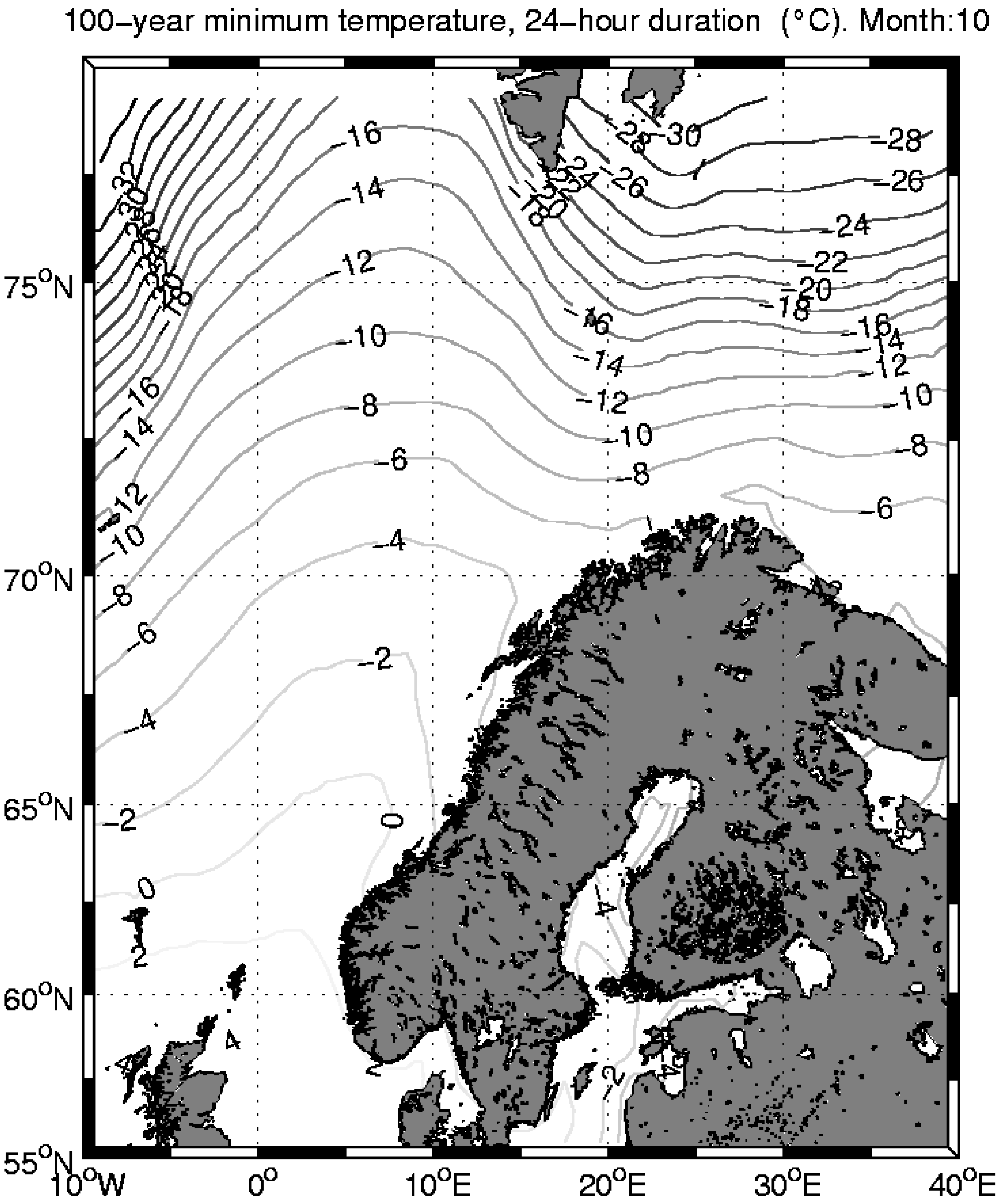

Figure C.23 – October: 24-hour duration minimum temperature at 2 meters above SWL on the Norwegian Continental Shelf (= minimum in database -2 °), data period 1958–2011

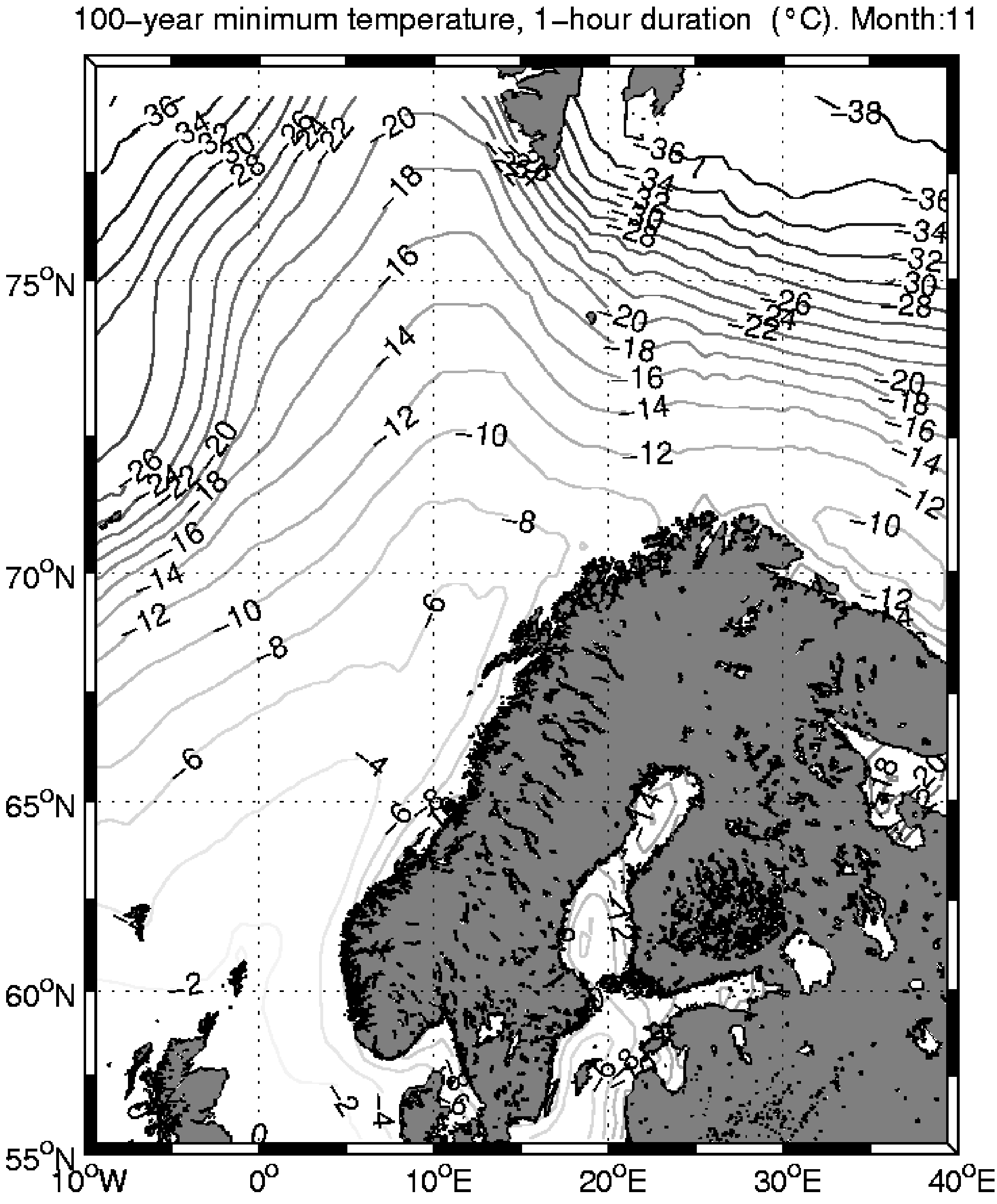

Figure C.24 – November: 1-hour duration minimum temperature at 2 meters above SWL on the Norwegian Continental Shelf (= minimum in database -2 °), data period 1958–2011

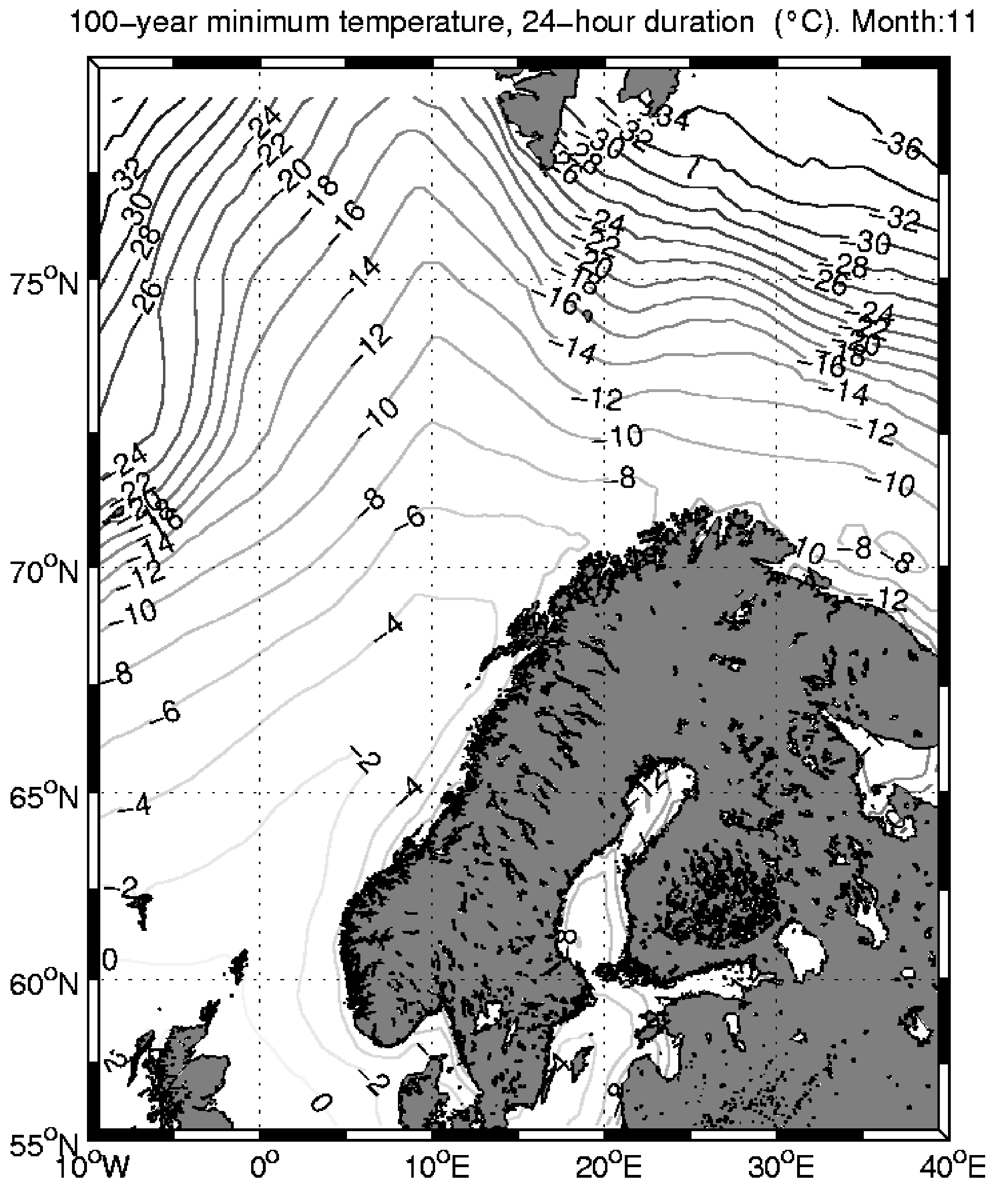

Figure C.25 – November: 24-hour duration minimum temperature at 2 meters above SWL on the Norwegian Continental Shelf (= minimum in database -2 °), data period 1958–2011

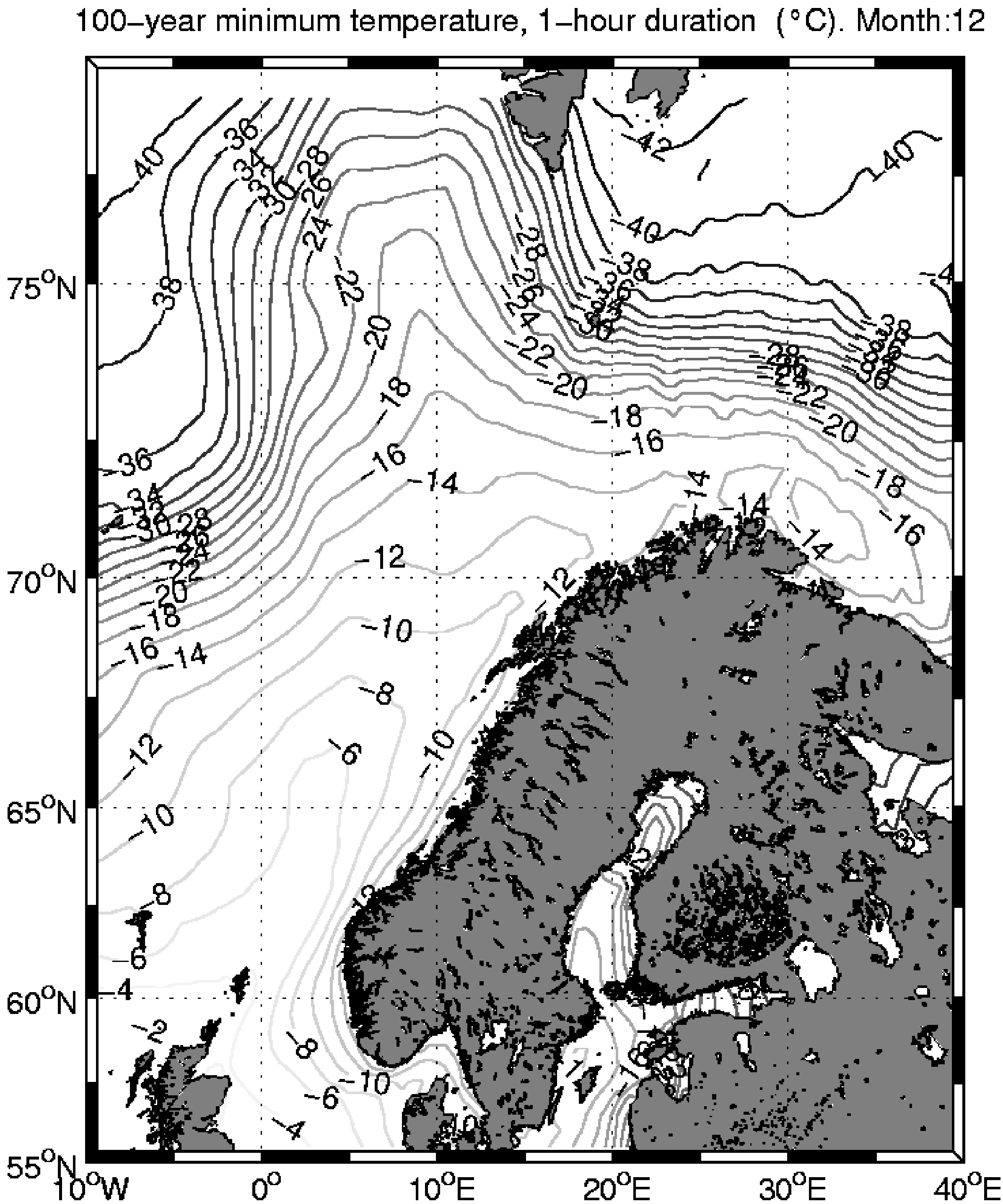

Figure C.26 – December: 1-hour duration minimum temperature at 2 meters above SWL on the Norwegian Continental Shelf (= minimum in database -2 °), data period 1958–2011

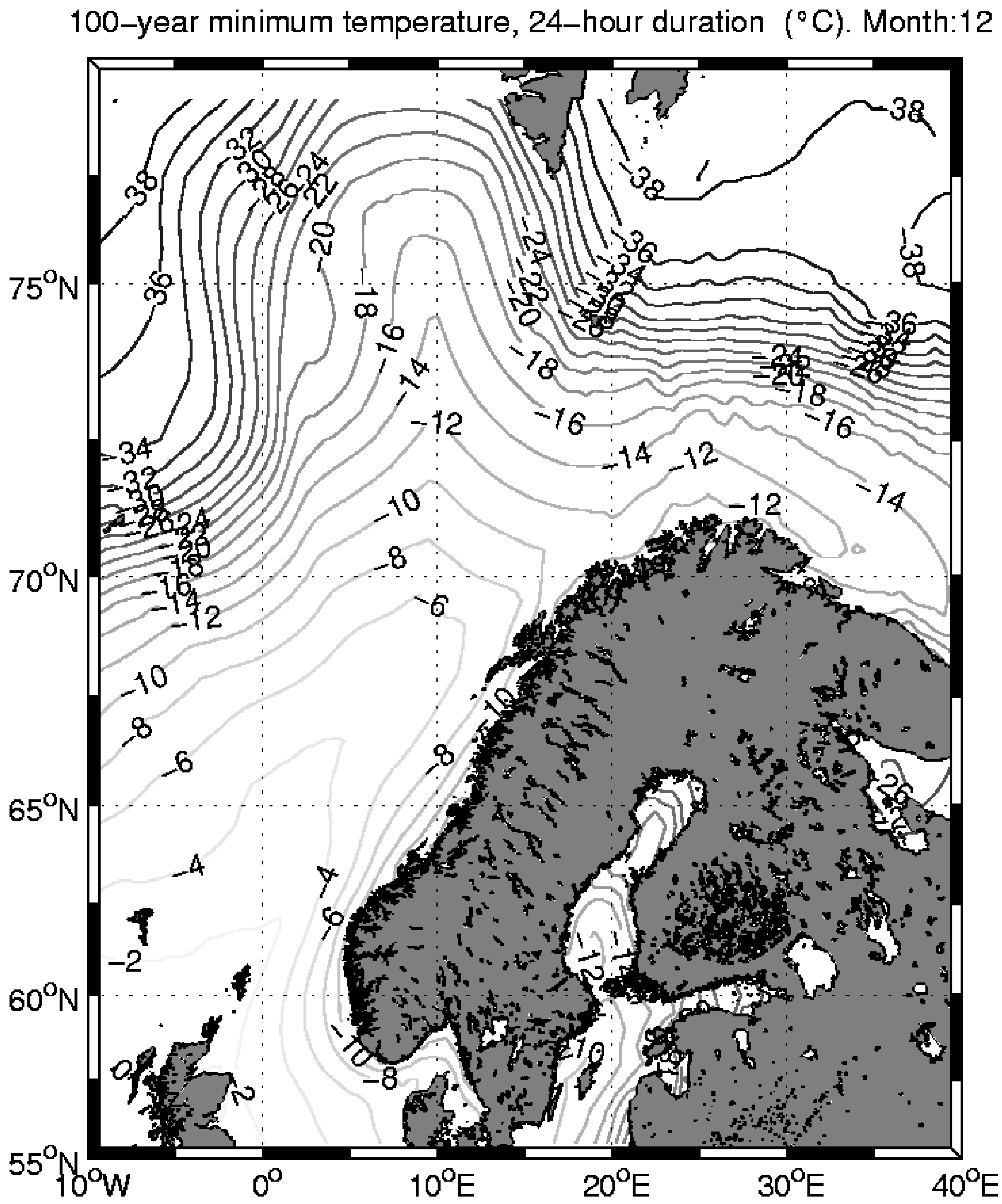

Figure C.27 – December: 24-hour duration minimum temperature at 2 meters above SWL on the Norwegian Continental Shelf (= minimum in database -2 °), data period 1958–2011

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