RUSSIAN MARITIME REGISTER OF SHIPPING

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RULES

FOR THE CLASSIFICATION, CONSTRUCTION AND EQUIPMENT OF MOBILE OFFSHORE DRILLING UNITS AND FIXED OFFSHORE PLATFORMS

PART II

HULL

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RULES FOR THE CLASSIFICATION, CONSTRUCTION AND EQUIPMENT OF MOBILE OFFSHORE DRILLING UNITS AND FIXED OFFSHORE PLATFORMS

Rules for the Classification, Construction and Equipment of Mobile Offshore Drilling Units (MODU) and Fixed Offshore Platforms of (FOP) of Russian Maritime Register of Shipping (RS, the Register) have been approved in accordance with the established approval procedure and come into force on 1 July 2022.

The present edition of the Rules is based on the 2018 edition taking into account the amendments and additions developed before publication.

The Rules set down specific requirements for MODU and FOP, consider the recommendations of the Code for the Construction and Equipment of Mobile Offshore Drilling Units (MODU Code), as adopted by the IMO Assembly on 2 December 2009 (IMO resolution A.1023(26)).

The procedural requirements, unified requirements, unified interpretations and recommendations of the International Association of Classification Societies (IACS) and the relevant resolutions of the International Maritime Organization (IMO) have been taken into consideration.

The Rules are published in the following parts:

Part I "Classification";
Part II "Hull";
Part III "Equipment, Arrangements and Outfit of MODU/FOP";
Part IV "Stability";
Part V "Subdivision";
Part VI "Fire Protection";
Part VII "Machinery Installations and Machinery";
Part VIII "Systems and Piping";
Part IX "Boilers, Heat Exchangers and Pressure Vessels";
Part X "Electrical Equipment"
Part XI "Refrigerating Plants";
Part XII "Materials";
Part XIII "Welding";
Part XIV "Automation";
Part XV "MODU and FOP Safety Assessment";
Part XVI "Signal Means";
Part XVII "Life-Saving Appliances";
Part XVIII "Radio Equipment";
Part XIX "Navigational Equipment";
Part XX "Equipment for Prevention of Pollution".

These Rules supplement the Rules for the Classification and Construction of Sea-Going Ships and the Rules for the Equipment of Sea-Going Ships.

REVISION HISTORY[1]

(purely editorial amendments are not included in the Revision History)

Amended paras/chapters/ sections	Information on amendments	Number and date of the Circular Letter	Entry-into-force date
Table 1.5.1.2	Requirements regarding material selection for hull structures have been specified	314-41-1865c of 25.11.2022	15.12.2022

[1] Amendments and additions introduced at re-publication or by new versions based on circular letters or editorial amendments.

1 GENERAL

1.1 APPLICATION

1.1.1 The requirements of this Part of the Rules for the Classification, Construction and Equipment of Mobile Offshore Drilling Units (MODU) and Fixed Offshore Platforms (FOP)[1] apply to the following:

.1 steel self-propelled and non-self-propelled MODUs which types are defined under 1.2 of Part I "Classification" of the MODU/FOP Rules;

.2 tension leg platforms (TLP) which types are specified in 1.2 of Part I "Classification" of the MODU/FOP Rules. The TLP hull is supposed to be made of steel, and be provided with the steel concrete ice belt for ice resistant TLP; tension legs shall be made of steel, foundation may be made of steel, concrete/reinforced concrete or composite;

.3 steel, concrete and composite FOPs including ice-resistant ones which are held to the bottom by gravity, with piles or be a combination of both, and whose types are defined under 1.2 of Part I "Classification" of the MODU/FOP Rules.

1.1.2 This Part contains provisions aimed at ensuring the strength of MODU/FOP hull elements. If alternative approaches to strength analysis are used, they shall be agreed with the Register. The safety level ensured for the structure shall be at least the same as stipulated in the MODU/FOP Rules.

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[1] Hereinafter referred to as "the MODU/FOP Rules".

1.2 DEFINITIONS AND EXPLANATIONS

1.2.1 Definitions and explanations pertinent to the general terminology of the MODU/FOP Rules shall be found in the General Regulations for the Classification and Other Activity, in the Rules for the Classification and Construction of Sea-Going Ships[1] and in Part I "Classification" of the MODU/FOP Rules.

1.2.2 For the purpose of this Part, the following definitions have been adopted.

Topside is the upper section of a MODU/ FOP designed to accommodate equipment and attendants, and not involved in the overall hull strength ensurance.

Ground foundation (anchor) is an underwater TLP element fixed at the seabed.

Structural elements are sections of shell and plating, built-up girders manufactured by welding and rolling, components of shell and plating with adjacent frames, etc.

MODU/FOP hull is an aggregate of structural elements of a MODU/FOP which shall take up all the total and local, constant and alternating loads. Where a MODU/FOP hull is composed of independent (but in all cases interconnected) elements, such as legs and stability columns, underwater stability block, pontoons, braces, upper hull (upper bearing structure), the term "hull structure" can be used with respect to these.

Tension leg is a system of elements connecting hull and subsea foundation aimed at TLP mooring.

Raiser is a system of piping and equipment aimed at connecting a borehole with the platform and supplying the platform with the extracted product.

[1] Hereinafter referred to as "the Rules for the Classification".

1.3 SCOPE OF TECHNICAL SUPERVISION

1.3.1 The hull (hull structures) of MODU/FOP are covered by the requirements of the General Regulations for the Classification and Other Activity and of the Rules for the Classification.

1.3.2 The following structures of MODU/FOP (depending on the type of technical construction) are subject to technical supervision during manufacture:

shell plating and framing of legs and stability columns, pontoons, underwater stability block, braces, upper hull, upper bearing structure, etc.;

watertight bulkheads and tanks;

decks and platforms;

helidecks;

superstructures and deckhouses;

jack houses;

coamings, companions and other guards of openings in MODU/FOP hull;

foundations of main and auxiliary machinery including those of other items subject to technical supervision;

substructure of drilling derrick.

1.3.3 Prior to manufacture of the structures listed in 1.3.2 of this Part, hull documentation shall be submitted to the Register for review in the scope stipulated in 4.1.3 of Part I "Classification" of the MODU/FOP Rules. Besides, the following documents shall be submitted:

.1 basic data, i.e. comprehensive data on ambient conditions (wind, sea, tide, ice, seabed, seismicity, temperature) in the area of MODU/FOP operation that comply with the requirements of 2.2.

Data may be used, as contained in Appendix 1, as well as other data on ambient conditions, provided these are agreed with the Register in advance;

.2 operating mode description, i.e. the volume of data on the operating modes of a MODU/FOP, as stipulated in 2.3. Additional operating modes may be reviewed which agree with the features of the MODU/FOP in question;

.3 strength calculations to the minimal extent necessary for the hull strength confirmation on the basis of criteria adopted for the modes of MODU/ FOP operation that may bring about a critical state of the structure. The methods of calculation shall be agreed with the Register;

.4 MODU/FOP operating manual including the following:

brief description of the unit;

list of operating modes;

permissible values of parameters essential for the MODU/FOP safety in a particular operating mode;

loading conditions of a MODU/FOP in each operating mode;

instructions for the crew on the MODU/FOP maintenance in each operating mode;

instructions on the safe operation techniques of a MODU/FOP;

drawings with indication of the grades and strength of steels used for MODU/FOP structures, list of permissible welding procedures and welding consumables. Where necessary, additional instructions on welding consumables and welding may be given which may include possible restrictions and conditions for repair or conversion.

1.4 STRUCTURAL ELEMENTS

1.4.1 The structural elements of MODU/FOP shall be classified into special, primary and secondary elements proceeding from stress levels and the effect their eventual damage may have upon the strength and serviceability of the technical construction.

1.4.1.1 Special structural elements are those portions of primary structural elements which are in way of critical load transfer points, stress concentrations, etc.

1.4.1.2 Primary structural elements are those which ensure the overall structural strength and integrity (if required proceeding from service conditions), as well as those whose importance is due to their role in the attendants safety ensurance.

1.4.1.3 Secondary structural elements are those which, when damaged, do not substantially impair the safety of the technical construction.

1.4.2 Structural elements of semi-submersible MODU.

1.4.2.1 Special elements:

shell plating in way of stability column connections to decks and lower hulls;

deck plating, stiffened web girders and bulkheads of upper hull or platforms forming box or T-shaped bearing structures in areas subjected to considerable concentrated loads;

main bracings intersections;

semibulkheads, bulkhead and platform sections, as well as framing taking up considerable concentrated loads at intersections of bearing structure elements;

structural elements fitted for load transmission at intersections or connections of main bearing structures.

1.4.2.2 Primary elements:

shell plating of stability columns, upper and lower hulls, and bracings;

deck plating, bulkheads and stiffened web girders of upper hull which form box or T-shaped bearing structures not subjected to considerable concentrated loads.

1.4.2.3 Secondary elements:

internal structures including the bulkheads and recesses of stability columns and lower hulls, leg and bracings framing;

upper platform or upper hull decks except areas where these elements are primary or special ones;

large-diameter stability columns with small length-to-diameter ratios except the connections of a column or intersections.

1.4.3 Structural elements of self-elevating MODU.

1.4.3.1 Special elements:

vertical legs in way of their connections to footings;

intersections of truss-type leg elements with welded components including steel castings.

1.4.3.2 Primary elements:

shell plating of tubular legs;

shell plating of all elements of truss-type legs;

bulkheads, decks, side and bottom plating of the topside which form box or T-shaped bearing structures;

jack house structures of legs and footings, which take up the loads from legs.

1.4.3.3 Secondary elements:

inner framing including bulkheads and web framing members of tubular legs;

inner bulkheads and recesses, as well as framing members of the topside;

inner bulkheads of leg footings except areas where the elements are principal or special ones;

deck plating, side and bottom shell plating of the topside except areas where the elements are primary or special ones.

1.4.4 Structural elements of FOP.

1.4.4.1 Special elements:

structural elements of "skirt" and elements fitted in areas where the skirt is mated to the FOP bottom;

structural elements of ice strake where the platform is an oil reservoir;

structural elements in way of hull structural connections by which the overall strength is ensured, and in areas where the cross section varies abruptly;

structural areas subjected to considerable concentrated loads.

1.4.4.2 Primary elements:

shell plating of hull structures;

watertight bulkhead plating, watertight platform plating by which the overall strength is ensured;

web girders of hull structures;

main framing of shell plating, bulkhead plating, deck plating by which the overall hull strength is ensured.

1.4.4.3 Secondary elements:

inner structures not contributing to the overall hull strength;

auxiliary framing of shell plating and plating.

1.4.5 Structural elements of hull, ground foundation and TLP tension legs.

1.4.5.1 Special elements:

hull structures of a multi-column TLP as specified in 1.4.2.1 for the semi-submersible MODU;

hull structures of a tower-shaped TLP, such as:

structural elements of ice belt where the platform is an oil reservoir;

structural elements in way of hull structural joints by which the overall strength is ensured, and in the areas where the cross section varies abruptly;

structural areas subjected to considerable concentrated loads;

hull structural elements of TLP and ground foundation, interacting with tension legs;

interaction areas of tension legs, the hull and ground foundation and high voltage elements of the devices for tension maintenance in tension legs;

local areas of tension legs subject to possible high tension impact (coupling, welded transverse joints etc.);

areas of ground foundation exposed to substantial loads.

1.4.5.2 Primary elements:

hull structures of a multi-column TLP as specified in 1.4.2.2 for the semi-submersible MODU;

hull structures of a tower-shaped TLP as specified in 1.4.4.2 for the FOP;

tension legs and their elements, except for the areas, in which the elements are special;

structural elements of the ground foundation, except for the areas, in which the elements are special.

1.4.5.3 Secondary elements: the hull structures of the multi-column and tower-shaped TLP, as specified in 1.4.2.3 and 1.4.4.3 for the semi-submersible MODU and FOP, respectively.

No structural elements of tension legs or anchors as well as the areas of hull and tension leg joints shall be classified as the secondary structural elements.

1.4.6 The final classification of MODU/FOP structural elements shall be carried out by the designer and shall be agreed with the Register.

1.5 MATERIALS

1.5.1 Steel structures.

1.5.1.1 For the manufacture of MODU/FOP structures, the steel approved by the Register and complying with the requirements of Part XIII "Materials" of the Rules for the Classification shall be used.

1.5.1.2 The steel grade for particular structural element of MODU/FOP shall be determined from Table 1.5.1.2 reasoning from the design temperature of the structural material and the function of the element according to the requirements of 1.4.

1.5.1.3 The design temperature for structural material is determined either experimentally or by calculation proceeding from the minimal daily average temperature T_a adopted (refer to 1.2.3 of Part II "Hull" of the Rules for the Classification). For the design ambient air temperature, refer to 2.2.8 of this Part. In the absence of the above substantiations, the design temperature of material of exterior abovewater structural elements shall be adopted equal to the ambient air temperature.

Design temperature may be increased, if reliable evidence (obtained either by calculation or experiment) is provided to the Register that under service conditions the temperature of particular structural elements of MODU/FOP would not reach the minimal ambient air temperature stated in the specifications.

1.5.1.4 Special and primary structural elements taking up considerable loads directed through the thickness dimension of rolling shall be manufactured of Z-steel in compliance with the requirements of 3.14 of Part XIII "Materials" of the Rules for the Classification.

Table 1.5.1.2

Structural elements	Steel grade for MODU/FOP	Design temperature of structural material, in °C						
		0	–10	–20	–30	–40	–50	–60
		Max. thickness of structural element, in mm						
Secondary	A	30	20	10	—	—	—	—
	B	40	30	20	10	—	—	—
	D	50	50	45	35	25	15	—
	E	50	50	50	50	45	35	25
	F	50	50	50	50	50	50	45
	A32, A36, A40	40	30	20	10	—	—	—
	D32, D36, D40	50	50	45	35	25	15	
	E32, F36, E40	50	50	50	50	45	35	25
	F32, F36, F40	50	50	50	50	50	50	45
	AH420, AH460, AH500	40	25	10	—	—	—	—
	DH420, DH460, DH500	50	45	35	25	15	—	—
	EH420, EH460, EH500	50	50	50	45	35	25	15
	FH420, FH460, FH500	50	50	50	50	50	45	35

Structural elements	Steel grade for MODU/FOP	Design temperature of structural material, in °C						
		0	–10	–20	–30	–40	–50	–60
		Max. thickness of structural element, in mm						
Primary	A	20	10	—	—	—	—	—
	B	25	20	10	—	—	—	—
	D	45	40	30	20	10	—	—
	E	50	50	50	40	30	20	—
	F	50	50	50	50	40	30	25
	A32, A36, A40	25	20	10	—	—	—	—
	D32, D36, D40	45	40	30	20	10	—	—
	E32, F36, E40	50	50	50	40	30	20	15
	F32, F36, F40	50	50	50	50	50	40	30
	AH420, AH460, AH500	20	—	—	—	—	—	—
	DH420, DH460, DH500	45	35	25	15	—	—	—
	EH420, EH460, EH500	50	50	45	35	25	15	—
	FH420, FH460, FH500	50	50	50	50	45	35	25
Special	A	15	—	—	—	—	—	—
	B	15	—	—	—	—	—	—
	D	30	20	10	—	—	—	—
	E	50	45	35	25	15	—	—
	F	50	50	50	45	35	25	15
	A32, A36, A40	15	—	—	—	—	—	—
	D32, D36, D40	30	20	10	—	—	—	—
	E32, F36, E40	50	45	35	25	15	—	—
	F32, F36, F40	50	50	50	50	40	30	20
	AH420, AH460, AH500	—	—	—	—	—	—	—
	DH420, DH460, DH500	25	15	—	—	—	—	—
	EH420, EH460, EH500	50	40	30	20	10	—	—
	FH420, FH460, FH500	50	50	50	40	30	20	10

N o t e s : 1. For intermediate temperatures, linear interpolation is permissible.

2. Other steel marks may be used if their properties are considered sufficient to ensure the specified level of safety.

Structural elements	Steel grade for MODU/FOP	Design temperature of structural material, in °C						
		0	–10	–20	–30	–40	–50	–60
		Max. thickness of structural element, in mm						
3. Steel grade selection for topsides structures is not regulated.								

1.5.1.5 The design yield stress R_d of material shall be determined from Table 1.5.1.5 proceeding from the standard yield stress R_{eH}.

Table 1.5.1.5

Steel grade for MODU/FOP	Standard yield stress, R_{eH}, in MPa	Design yield stress R_d, in MPa, for thickness, in mm		
		<30	30 — 50	50 — 70
A, B, D, E, F	235	235	215	200
A32, D32, E32, F32	315	315	295	280
A36, D36, E36, F36	355	355	335	320
A40, D40, E40, F40	390	390	370	355
AH420, DH420, EH420, FH420	420	420	390	365
AH460, DH460, EH460, FH460	460	460	430	390
AH500, DH500, EH500, FH500	500	500	480	440

1.5.1.6 The application of normal, higher and high strength steels with a thickness above 75 mm, as well as of steels for which R_{eH} > 500, in MPa, may be permitted by the Register, in case the steels meet the requirements of Part XIII "Materials" of the Rules for the Classification. For such steels, the design yield stress shall be agreed with the Register.

1.5.2 Reinforced structures.

Requirements for the materials of reinforced structures shall be found under 3.4.

1.6 WEAR OF STRUCTURAL ELEMENTS

1.6.1 The scantlings of MODU/FOP structural elements shall be assigned with due regard for corrosion allowance, and for shell plating in way of ice strake of ice-resistant FOP, allowance shall be made for surface abrasion with ice.

1.6.2 Wear allowance Δs, in mm, shall be made for the thickness of structural elements, as obtained by strength calculations, which is determined by the formula

$$\Delta s = kuT^{*} \tag{1.6.2}$$

where u = the design wear rate, in mm per year;
T^{*} = $T/2$ for MODU/FOP structural elements which can be repaired during service;
T^{*} = T for MODU/FOP structural elements which cannot be repaired during the whole period of the platform service;
T = the design service period of MODU/FOP, in years;
k = the factor accounting for the positive effect of protective measures to reduce wear ($k \leq 1$).

1.6.3 The design wear rate u shall be adopted on the basis of data on the wear of selected steels under conditions corresponding to the service conditions of MODU/FOP, the positive effects of wear reduction measures disregarded. In the absence of such data, the design wear rate may be adopted with due regard for the relevant requirements of the Rules for the Classification. In so doing, the congruity of service conditions of the MODU/FOP structural elements with those of the components for which data are given in the Rules for the Construction shall be borne in mind.

When adopting design corrosion rates for structural elements of semi-submersible MODU, one may be guided by the recommendations of Table 1.6.3.

Table 1.6.3

Recommended design corrosion rates for structural elements of semi-submersible MODU

Nos.	Structural element	Design corrosion rate, in mm per year
1	**Bracings:**	
1.1	Horizontal transverse bracings	
	in way of connections to columns and other bracings	0,18
	outside the areas of connection	0,16
1.2	Horizontal diagonal bracings	
	in way of connections to columns and other bracings	0,18
	outside the areas of connection	0,14
1.3	Inclined transverse bracings	
	in way of connections to columns, pontoons and upper hull	0,18
	outside the areas of connection	0,16
1.4	Inclined longitudinal bracings	
	in way of connections to columns and upper hull	0,15
	outside the areas of connection	0,14
2	**Columns**:	
	in way of connections to pontoons	0,14
	on the level of alternating waterline	0,16
	above-water structure	0,10
	underwater structure	0,12
3	**Pontoons**:	
	bottom, deck, sides of ballast and fuel compartments	0,16
	bulkheads	0,14
	bottom, deck, sides of dry compartments	0,13

Nos.	Structural element	Design corrosion rate, in mm per year
4	**Upper hull:**	
	sides and transoms	0,11
	bulkheads	0,10
	supporting beams	0,13
	main deck	0,10
	open sections of upper deck exposed to weather	0,13

1.6.4 The factor k accounting for the positive effects of protective measures to reduce wear may be adopted less than one, provided effective corrosion protection is used for structural elements, or special coats and materials are applied to prevent surface abrasion with ice. The factor is only to be introduced for the elements that are covered by protective measures.

1.6.4.1 For the structures of semi-submersible MODU and FOP, which are equipped with efficient corrosion protection systems, $k = 0{,}5$ shall be adopted, provided both the sides of the structural element are protected, and $k = 0{,}75$ shall be adopted where one of the surfaces of the structural element is protected.

1.6.4.2 The value of $k < 1$ shall be substantiated and agreed with the Register where the exterior structures of the ice strake of FOP are concerned on condition protective measures are taken to reduce wear.

1.6.5 $\Delta s = 1{,}0$ mm, shall be the minimal corrosion allowance.

1.7 WELDED STRUCTURES AND JOINTS

1.7.1 Welded joints of MODU and FOP structures shall meet the requirements of Parts II "Hull" and XIV "Welding" of the Rules for the Classification and of Part XIII "Welding" of the MODU/FOP Rules with regard to welded joints and structures, welding consumables, welding methods and quality control of welded joints.

1.7.2 Welded joints of special structures potentially subjected to excessive stresses across the rolled stock thickness shall be carried out so as to prevent or minimize the possibility of a layered rupture.

1.7.3 Weld dimensions are set according to approved national standards or technical documentation.

2 STRUCTURAL DESIGN PRINCIPLES

2.1 GENERAL

2.1.1 The design of MODU/FOP shall be such that its strength within the service life (as determined for environmental conditions of the anticipated area of operation) meets the accepted criteria in the following design conditions:

transit;

positioning at the site;

operational;

survival or extreme loading;

removal from site.

Besides, if necessary according to conditions of construction strength of structures or separate elements shall be verified during manufacture.

Adjustment of design conditions to the type of MODU/FOP is given in Section 3.

2.1.2 Designing MODU/FOP shall be carried out keeping due note of safe operation requirements including requirements for environmental safety during the whole service life of the structure as well as ensuring convenience of works related to survey/examination and current repair of structures.

2.1.3 It is recommended to equip MODU/FOP with instrumentation for observation of condition of hull structures in order to assess their reliability, timely detection of defects and increment of safety level.

2.1.4 Strength calculations shall be performed in respect of all structural elements of MODU/FOP: special, primary and secondary.

Dimensions of structural elements exposed to local loads only and which don’t contribute to overall strength of the unit (platform) may be determined in accordance with applicable requirements of Part II "Hull" of the Rules for the Classification.

2.1.5 Structural scheme and general arrangement of topside shall consider safety requirements reducing risk of possible environmental exposures. In particular, the accommodation spaces shall be located from the side of dominating winds; derrick and flare — on the opposite side, etc.

2.1.6 MODU hull structures in the place of installation of positioning system machinery shall ensure taking up loads equal to breaking strength of ropes and chains. Permissible stresses shall not be more than $0,95R_{eH}$.

2.2 ENVIRONMENTAL CONDITIONS

2.2.1 General.

2.2.1.1 The description of environmental conditions of the area of operation (sea or seas, area or part of sea area) shall include data on the ambient conditions which may influence the reliability of MODU/FOP, namely: wind, waves, current, ice, seabed, seismic exposure, air temperature, etc).

2.2.1.2 The description of environmental, conditions shall reflect the realistic character of wind and wave formation, currents and ice formation and shall be based upon probabilistic methods and statistical information.

2.2.1.3 The basic parameters of environmental conditions, as determined for the prescribed area of operation shall be agreed with the Register. Data on the wind and waves in different seas are given in Appendix 1.

2.2.1.4 If conditions of MODU/FOP operation are limited by the list of seas, area or part of the sea areas, seasons or permissible limits of environmental conditions then the list of seas, area borders or parts of these areas, seasons and permissible values of characteristics of environmental conditions for relevant areas of operation are specified in the MODU/FOP Operating Manual.

2.2.2 Wind.

2.2.2.1 There are following characteristics of the wind: average wind velocity at anemometer height (z = 10 m), law of change of wind average velocity by height, wind gustiness parametres, spectral characteristics of wind pulsations.

The basic data on the wind conditions are wind velocities $\overline{w}_{10}$ with the averaging period of 10 min at anemometer height (z =10 m) which are called standard velocities and the period of recurrence in the region under consideration over the long period of not less than 20 years.

2.2.2.2 Extreme values of average wind velocities are determined from many years observations as most probable largest values with recurrence period of 100 years, but not less than 25,8 m/s.

2.2.2.3 The interrelation between W_{max} and average $\overline{w}$ velocities is determined by the gustiness coefficient G:

$$W_{max} = G\overline{w};\ G = 1 + \gamma\vartheta_w \tag{2.2.2.3}$$

where γ = numeric coefficient (refer to Table 2.2.2.3);
σ_w = standard deviation of wind velocities;
ϑ_w = coefficient of the wind velocity volatility (refer to 2.2.2.4)

$$\vartheta_w = \sigma_w / \overline{w}.$$

Table 2.2.2.3

Speed averaging period of 10 min	Duration of the maximum gust *n*						
	1	3	6	12	18	36	90
γ	2,94	2,58	2,52	2,10	1,90	1,55	1,00

Maximum velocity is calculated at averaging of n seconds. Recommended n = 3.

2.2.2.4 It is recommended to use the Davenport longitudinal wind pulsation spectrum

$$S(f) = \frac{4K_{hr}\overline{w}_{10}^2 n^2}{f(1+n^2)^{4/3}} \tag{2.2.2.4-1}$$

where $n = 1200/\overline{w}_{10}$
f = frequency, in Hz;

$\overline{w}_{10}$ = average wind velocity at hour averaging (m/s); transitional coefficients between different averaging periods are determined on the basis of Fig. 2.2.2.4;
K_{hr} = head resistance coefficient of underlying surface (refer to Table 2.2.2.4-1).

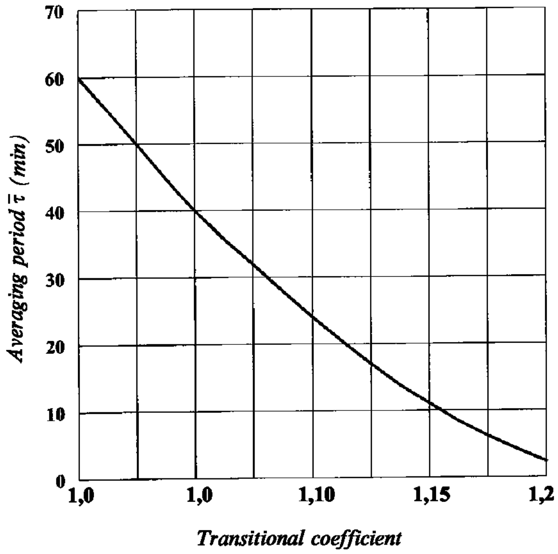

Fig. 2.2.2.4
Diagram of transitional coefficients v_τ/v_{60}

Table 2.2.2.4-1

$\overline{w}_{10}$, m/c	15		20	25	30
K_{hr} 10^3	2,0		2,5	3,0	3,5

Wind velocity by height profile considering time of averaging is determined by the following formula:

$$\overline{w} = \overline{w}_{10}\left[1 + \ln(z/10)^{1/7}(10/t)^{1/20}\right] \qquad (2.2.2.4\text{-}2)$$

where t = the time of averaging, in min,

and according to Table 2.2.2.4-2.

Table 2.2.2.4-2

Time / z, in m	3 s	5 s	15 s	1 min	10 min	60 min
1,0	0,936	0,910	0,856	0,786	0,671	0,581
5,0	1,166	1,140	1,085	1,016	0,901	0,811
10,0	1,265	1,239	1,184	1,115	1,000	0,910
20,0	1,364	1,338	1,283	1,214	1,099	1,009
30,0	1,422	1,396	1,341	1,272	1,157	1,067
40,0	1,463	1,437	1,382	1,313	1,198	1,108
50,0	1,495	1,469	1,414	1,345	1,230	1,140
100,0	1,594	1,568	1,513	1,444	1,329	1,239

2.2.2.5 The variation of average wind velocities by height is shown by the following expressions:
open seas

$$\bar{w}_z = \bar{w}_{10}[1 + \ln(z/10)^{1/7}]; \qquad (2.2.2.5\text{-}1)$$

coastal zones

$$\bar{w}_z = \bar{w}_{10}[1 + \ln(z/10)^{1/5}] \qquad (2.2.2.5\text{-}2)$$

where z = height above sea level, in m; $10 \le z \le 100$ m.

2.2.3 Waves.
2.2.3.1 Waves are described by the following parameters: wave height with 3 % probability of exceeding level, wave height recurring once in 100 years, average wave period, average wave frequency, wave spectral density, average wave length, joint periodicity of wave heights and periods.
2.2.3.2 Joint recurrence of wave heights and periods is determined on the basis of information from specialized organizations for the given area of operation.
2.2.3.3 The ratio between the average period, average wave length and average frequency shall be employed in case of shallow water:

$$\bar{\omega}^2 = \bar{K} g th \bar{K} H; \qquad (2.2.3.3\text{-}1)$$

$$\tau = 2\pi/\bar{\omega} \qquad (2.2.3.3\text{-}2)$$

where $\bar{K}$ = wave number, $\bar{K} = 2\pi/\bar{\lambda}$;
$\bar{\lambda}$ = average wave length, in m;
H = depth of water space, in m.

2.2.3.4 The extreme values of wave heights are determined as the most probable largest wave heights with recurrence period of 100 years ($\tilde{h}_{100}$). In case of absence of information of their values $\tilde{h}_{100}$ may be determined by the formula

$$\tilde{h}_{100} = 0{,}38\sigma_{h_3} \ln n \qquad (2.2.3.4)$$

where $\sigma_{h_3} = \sqrt{\sum_i p_i h_{3i}^2}$;
p_i = recurrence of wave conditions which is featured by h_{3i}, (value of column $\sum_n$ of joint recurrence of wave heights and periods table);

$n = T/\bar{\tau}$;

T = period of time under consideration (T = 100 years);
$\bar{\tau}$ = average wave period over this time period,

$\bar{\tau} = \sum_j p_j \tau_j$,

p_j = value of column $\sum_\tau$ of joint recurrence of wave heights and periods table.

2.2.3.5 Two concepts may be employed to assess extreme values:
the first concept is based on the long term distributions;
the second concept is based on the severest conditions;

The closed system of assessments implies the following interrelation of wave characteristics for these concepts:

$$h_{100} = 1{,}94(h_3)_{max}; \tag{2.2.3.5-1}$$

$$(h_3)_{100} = 2{,}94(h_3)_{max} - 18{,}8 \tag{2.2.3.5-2}$$

where $(h_3)_{max}$ = wave height, in m, of 3 % probability of exceeding level for stationary conditions at which the extreme value of the given exceedance is the most probable for realization;
$(h_3)_{100}$ = wave height, in m, of 3 % probability of exceeding level recurring once in 100 years.

2.2.3.6 JONSWAP spectrum is the recommended design wave spectrum:

$$S_j(\omega) = S_{PM}\gamma^{exp[-(\omega-\omega_m)^2/2\sigma^2\omega_m^2]} \tag{2.2.3.6-1}$$

where S_{PM} = Pierson-Moskowits spectrum determined by the formula

$$S_{PM} = 10^{-2}h_3^2\bar{\tau}(\omega/\bar{\omega})^{-5}\exp[-0{,}44(\omega/\bar{\omega})^{-4}] \tag{2.2.3.6-2}$$

$\bar{\omega} = 2\pi/\bar{\tau}$ = average wave frequency;
ω_m = frequency of spectrum maximum;
γ = ratio of S_j and S_{PM} maxima; average value γ =3,3;
$\sigma = \sigma_a = 0{,}07$ = σ„=0,07 for $\omega < \omega_m$;
$\sigma = \sigma_b = 0{,}07$ = σ„=0,07 for $\omega > \omega_m$.

2.2.4 Current.

2.2.4.1 When influence of current is studied in the given area of operation it is necessary to consider such factors like its nature (tide or wind), distribution by depth, stability over time.

In absence of information about the current profile in the area of operation under consideration it is recommended to use average statistical data from the following expression:

$$v_c = v_{c1}[(H_0 - z)/H_0]^{1/7} + v_{c2}[(H_0 - z)/H_0] \tag{2.2.4.1}$$

where v_c = general current speed at z depth from the water surface;
v_{c1} = high tide speed at a calm water level H_0;
v_{c2} = wind current speed at H_0 level.

2.2.4.2 When reviewing influence of current on parameters of external loads affecting MODU/FOP, it is necessary to consider the effect of interaction of the current and waves. In the random wave field this leads to the transformation of the wave spectrum:

$$S_{v_c}(\omega) = \frac{4S_0(\omega)}{[1+(1+4v_c\omega/g)^{1/2}][(1+4v_c\omega/g)^{1/2}+(1+4v_c\omega/g)]} \tag{2.2.4.2}$$

where $S_0(\omega)$ = the spectrum of the surface waves;
$v_c > 0$ = conjunction of the wave and current directions;
$v_c < 0$ = head directions of waves and current.

2.2.5 Ice.

2.2.5.1 The following physical and mechanical properties characterize level ice, rafted ice and consolidated layer of ridge: density, salt content, compression strength, bending strength, tensile strength, modulus of elasticity, fracture toughness, friction behavior of ice and structure.

2.2.5.2 The following physical and mechanical properties characterize ridge keel: ridge keel adhesion value, angle of internal friction and keel hollowness factor.
2.2.5.3 The following physical and mechanical properties characterize icebergs: iceberg density and compression strength.
2.2.5.4 The following geometrical properties characterize ice: thickness of level ice, thickness of rafted ice, thickness of consolidated layer of ridge, ridge height, ridge keel depth, ridge keel width (normal to the front), ice field area and ice consolidation.
2.2.5.5 The following geometrical properties characterize icebergs: shape and linear dimensions of an iceberg with regard to their variation depending on the vertical level.
2.2.5.6 The ice field thickness and dimensions are specified for the icebergs frozen into surrounding ice.
2.2.5.7 Speed of ice drift is also the initial information for assessment of ice forces when an impact is applied by ice formations to the platform.
2.2.5.8 Physical, mechanical properties and geometrical parameters of ice as well as speed of ice drift are random values. In order to use random values in subsequent calculations, a probabilistic approach (for example, Monte Carlo method) considering laws of random value distribution in respect of specific sea area shall be applied where reasonable. Statistically valid combinations, which determine the most hazardous combinations of ice parameters, may be also used in subsequent calculations. Combination of random values shall have the recurrence period of 100 years which is determined as agreed with the Register.
2.2.6 Seabed.
2.2.6.1 For the area of self-elevating MODU/ FOP installation it is necessary to obtain engineering section of the foundation with indication of depth of stratum and information on normative and calculated value of the physical and mechanical properties of the foundation.
2.2.6.2 There are following attributes of the seabed: type of the seabed (sand, clay, silt, etc.), wet soil weight, deformation modulus (statical and dynamical), Poisson's ratio, adhesion value, angle of internal friction, C_I — non-drained shift resistance, consolidation coefficient, porosity factor, humidity factor, seabed permeability, flow index.
2.2.7 Seismic conditions.
2.2.7.1 The main information on earthquakes in the seismically dangerous region is the intensity of seismic exposure which has a recurrence period over the long period of time — at least 100 years (design earthquake).
Extreme values of seismic exposure are determined on the basis of many years' experience and they shall be extrapolated as the most probable over 500 years (maximum design earthquake).
2.2.7.2 It is recommended to use a Russian scale based on maximum accelerations (refer to Table 2.2.7.2) for evaluation of earthquake force.
Seismic exposure shall be considered if force of calculated seismic activity in the area of self-elevating MODU/FOP operation is 6,5 and more.

Table 2.2.7.2

Earthquake force $J^{initial}$	Seabed acceleration intervals, in cm/s², at a period of 0,1 s and greater	Intervals between earth tremors, in cm/s	Intervals between movements of the centre of gravity of the seismometer pendulum, in mm
6	30 — 60	3,0 — 6,0	1,5 — 3,0
7	61 — 120	6,1 — 12,0	3,1 — 6,0
8	121 — 240	12,1 — 24,0	6,1 — 12,0
9	241 — 480	24,1 — 48,0	12,1 — 24,0

2.2.7.3 The interrelation between the calculated seismical activity $J_{100}^{designed}$ $(J_{500}^{designed})$ and attributes of the local seabed are determined in accordance with Table 2.2.7.3.

Table 2.2.7.3

Seismic categories of seabed	Seabed	Calculated seismic activity force $J_{100}^{designed}$ $(J_{500}^{designed})$ based on the initial seismic activity of the area of operation $J_{100}^{designed}$ $(J_{500}^{designed})$			
		6	7	8	9
I	Non-eroded and poorly eroded rocky seabed of all types (including many years frozen seabed in the frozen and melted condition); seabed of big fragmentary magma pieces, containing up to 20 % of sand and clay filler; speed of the transverse waves propagation $V_s \geq 700$ m/s; interrelation between speeds of longitudinal and transverse waves $V_p/V_s = 1{,}7 - 2{,}2$	–	6	7	8
II	Rocky seabed (except those referred to the I category); seabed of big fragmentary pieces (except those referred to the I category); dust-and-clay seabed with a flow index of $J_L \leq 0{,}5$, porosity factor $e < 0{,}9$ for clays and adobes and $e < 0{,}7$ for clay sand; many years non-rocky ductile and frozen or loose and frozen seabed; $V_s = 250 \div 700$ m/s, $V_p/V_s = 2{,}2 - 3{,}5$	6	7	8	9
III	Loose sands without regard of fineness; semi-gravel sands of large and medium fineness, high or medium density, dust-and-clay seabed with the flow index of $J_L > 0{,}5$ at a porosity factor of $e \geq 0{,}9$ for clays and adobes and $e \geq 0{,}7$ for clay sand; many year frozen and rocky seabed with possible defrosting; silt seabed; $V_s \leq 250$ m/s, $V_p/V_s \geq 3{,}5$ — for saturated seabed	7	8	9	>9

2.2.7.4 Evaluation of the seismical activity shall be matched with the existing Russian seismic charts.

2.2.8 **Ambient air temperature.**

The main source of information about ambient air temperature is information about the lowest average daily temperature in the possible area of the platform operation which is taken from the meteorological historical data over at least 10 years if anything else is not provided in this Part.

2.2.8.2 The minimal design temperature for the platform elements operating underwater at all times is taken for the water temperature –2 °C.

2.3 DESIGN CONDITIONS AND LOADS

2.3.1 Classification of loads.
2.3.1.1 By their nature, all loads affecting the MODU/FOP structure are grouped in two categories:
gravity loads with relevant environmental loads due to waves, wind, current, ice, seismic activity, seabed, temperature etc.;
the loads caused by functioning of machinery, equipment, systems and associated with the operation of the MODU/FOP.
Each category may comprise fixed and variable loads; the latter are determined as static or dynamic depending on structural response to external effects. Depending on the relative dimensions of the exposed area each of the above-mentioned loading categories is subdivided into common and local.
2.3.1.2 Fixed static loads are those which do not change in value, location or direction if environmental conditions have changed. For a structure in calm water condition the gravity forces of this structure and all permanently secured equipment, as well as the buoyancy forces, the platform footing counterpressure (weighing), soil loads and soil weight within the scope depending on the scheme of the interaction between the platform and foundation are treated as the fixed static loading.
Variable static loads are those which change in value and direction during a certain time period. However, the velocity of loading variation is so insignificant that it has practically no effect on the structure.
2.3.1.3 Dynamic loads are those which change in value and direction enough quickly to produce a dynamic effect on the behaviour of the structure. The dynamic effect on the structure may be caused by wind blows, waves, ice, seismic factors.
2.3.2 Survival conditions or extreme loads.
2.3.2.1 The loads which shall be considered in strength calculations of MODU/FOP under extreme loads include:
common and local variable and fixed extreme loads caused by environmental exposure;
common and local functional loads corresponding to extreme state of MODU/FOP in terms of safety.
2.3.2.2 For FOP extreme wind, wave, ice, current and temperature loads are those of the maximum external loads which may affect the platform over the 100 year period. The recurrence of seismic loads is determined in relation to the agreed criterion (refer to 3.1.6 and 3.3.2.4).
For MODU the extreme variable loads are those of the possible maximum external loads which affect MODU over the whole operation period. The variable loads which possibility of excess in the long term distribution is equal to 10^{-8} are taken for the design loads.
Extreme impact loads on the transverse horizontal bracing of the semi-submersible MODU are the loads caused by impact interaction with water during sailing the opposite course relative to the main wave system stationary wave conditions with maximum $h_{3\%}$ and T_{av} in the long-term mutual distribution of heights $h_{3\%}$ and T_{av} periods of waves in the area of operation, their probability is 10^{-4} for this wave mode.
2.3.2.3 The worst practical combinations of external loads which may cause the largest tension of structures shall be considered.
2.3.3 Operating conditions.
2.3.3.1 The loads which shall be considered in strength calculations of MODU/FOP in the operating conditions include:
common and local variable and fixed environmental loads which intensity allows a unit to perform main functions;
common and local functional loads corresponding to the operation mode.

2.3.3.2 The worst possible combinations of practical functional loads which may cause the greatest stresses of structure shall be considered.

The loads recurring once a year are taken for the design values of variable environmental loads. The loads which probability in the long term distribution is equal to 10^{-6} are allowed for MODU.

2.3.4 Transit conditions.

2.3.4.1 Permissible transit conditions are determined for the transportation conditions and specified in the Classification Certificate and the Operation Manual; the design of transit is developed for each transit which contains actions to ensure limitations imposed by environmental conditions and MODU/FOP safety during transportation. The design of transit shall be agreed with the Register.

2.3.4.2 Loads which shall be dealt with in the MODU/FOP strength calculations in transit include common and local fixed and variable loads incurred by the environment and such functional loads which cause the highest expansion in structures in conditions under consideration.

2.3.4.3 Loads with $5 \cdot 10^{-4}$ probability in the longterm distribution in permissible environmental conditions, but not more than $h_{3\%} = 7{,}0$ m are taken for design values of variable loads.

2.3.4.4 Loads with 10^{-6} probability during stationary wave conditions with $h_{3\%}$ and T_{av} permissible for the transit conditions, at the specified $h_{3\%}$, in the long-term distribution and sailing at head seas, but not more than $h_{3\%} = 7{,}0$ m are taken for design values of impact loads applied to semi-submersible MODU bracings during transportation.

2.3.5 Conditions of positioning at and removal from site.

Permissible environmental conditions are determined by the designer and they are subject to agreement with the Register. Structural strength calculations shall be made for the loads corresponding to these conditions of loading.

2.3.6 Deck loads.

The design loads applied to decks shall take into account the most unfavourable combination of functional loads indicated in 2.3.1. In any case, the design loads shall not be less than in Table 2.3.6.

Table 2.3.6

Designation of room/deck	Pressure q, к Pa
Accommodation decks, walkways	4,5
Work areas	9,0
Storage areas:	
general purpose	$7{,}85\rho h$, but not less than 13,0
for cement	$9{,}81\rho h$, but not less than 13,5
N o t e . h=cargo stowage height, m; ρ =mass cargo density, t/m³.	

2.3.7 Watertight bulkhead loads.

For the plating and framing of watertight bulkheads in ballast tanks, cargo or fuel oil tanks, the design pressure head, in kPa, is determined by the dependence

$$p = 9{,}81\rho\left(h_0 + h_p\right) \qquad (2.3.7)$$

where ρ = mass density of ballast, cargo or fuel, in t/m³;
h_0 = vertical distance from the design point to the uppermost point of the compartment under consideration, in m;
h_p = height of air pipe above the uppermost point of the compartment, in m.

2.3.8 Wind loads.
Wind loads are determined by the formula

$$Q_w = 10^{-3}\rho_w(w_{10}^2/2)\sum_i S_i K_{1i} K_{2i} \qquad (2.3.8\text{-}1)$$

where Q_w = resultant of wind forces, in kH;
ρ_w = mass density of air, in kg/m³;
w_{10} = design wind speed at the height of 10 meters above the calm water surface at 10 minutes averaging, in m/s;
S_i = *i*-element windage area, in m²;
K_{1i} = coefficient allowing for the change of wind speed by height (refer to 2.2.2.5);
K_{2i} = *i*-element form strength coefficient (corresponds to Table 2.4.2.3 of Part IV "Stability" of the MODU/FOP Rules).

Whereas some elements of the structures in question may be located within some spacing of each other ("permeable" structures), the following shall be considered:

if several elements are fitted in the plane normal to the wind direction, as for flat framing and columns, the permeability factor ϕ shall be considered. In this case the wind load is determined by the formula

$$Q_\phi = Q_w\phi \qquad (2.3.8\text{-}2)$$

where ϕ – permeability factor;

if two or more parallel structures forming frames are fitted one after the other along the wind direction, the screen factor shall be considered. In this case the wind load is determined by the formula

$$Q_{w_s} = Q_w\eta_s \qquad (2.3.8\text{-}3)$$

where η_s – screen factor.

The screen factor depends on permeability factor ϕ, type of the element and distance between the structures.

2.3.9 Hydrodynamic loads.

2.3.9.1 Wave loads applied to the platform and its elements are determined on the basis of the Moritz equation. For the single obstruction element, the vector of specific wave loads $\{Q\}$, in t/m, is shown by the following expression:

$$\{Q\} = \frac{\rho_v d C_{sr}}{2}\{|v-\dot{y}|(v-\dot{y})\} + \rho_v S\{\dot{v}\} + \rho_v(C_{in}-1)(v-\ddot{y}) \qquad (2.3.9.1)$$

where C_{sr} and C_{in} = speed and inertia resistance factors;
ρ_v = mass density of water, in ts²/m⁴;
S and d = cross section area, in m², and diameter of obstruction at z level from water surface, in m;
v and $\dot{v}$ = orbital speed, in m/s, and water particles acceleration in m/s²;
$\dot{y}$ and $\ddot{y}$ = speed and acceleration of the structural elements.

2.3.9.2 For the large obstruction diameters, d it is necessary to consider the diffraction effects. The proposed values of the diffraction coefficient K_v are set forth in Table 2.3.9.2; the inertial component is directly proportional to K_v, the speed component is proportional to K_v^2.

Table 2.3.9.2

Relative obstruction dimension d/λ	0,05	0,10	0,15	0,20	0,25	0,30	0,40
K_v	1,00	0,97	0,93	0,86	0,79	0,70	0,52

2.3.10 Current loads.

Mutual exposure to wave and current shall be considered in accordance with the recommendations of 3.1.5.2.

Current loads applied to MODU/FOP are determined in accordance with the recommendations of 3.1.5.1 — 3.3.2.2.

2.3.11 Combination of environmental loads.

2.3.11.1 The most dangerous combinations of loads in accordance with 2.3.1 — 2.3.5 shall be considered while calculating the MODU/FOP structural strength and buckling strength.

2.3.11.2 While reviewing the environmental loads it is necessary to consider that there may be several environmental loads acting at a time.

Combination of loads is subject to their statistical peculiarities.

During extreme loading of the structure it is allowed to use combination of common loads in accordance with Table 2.3.11.2 in absence of the probability analysis.

Table 2.3.11.2

Combination	Common environmental loads					
	Main load	Attendant loads				
		ice load	wave load	wind load	current load	seismic load
1	Extreme ice load	–	–	Average statistical wind load	Extreme current load	–
2	Extreme wave load	–	–	Average statistical wind load	Extreme current load	–
3	Extreme seismic load	Average statistical ice load	–	–	–	–
4	Extreme seismic load	–	Average statistical wave load	–	–	–

2.3.12 Mooring impact loads.

Attention shall be also given to impact loads from supply vessels alongside the drilling unit as well as provisions of 3.8 of Part II "Hull" of the Rules for the Classification.

2.3.13 Towing operation loads.

Towing operation loads are the loads applied to separate members of the unit arising during voyage in tow and consisting of the following:

constant component which depends on the unit speed in relation to water and wind;

variable component which depends on seaway and relative motion of unit and tow, conditioned by wave rocking.

When of towing MODU/FOP large-sized elements the intrinsic moment of inertia of the element shall be considered.

2.4 STRENGTH CRITERIA

2.4.1 General.

2.4.1.1 MODU/FOP shall be so designed as to meet the following general safety requirement:

$$\Phi \leq R\eta \tag{2.4.1.1}$$

where Φ = design value of the generalized force action (for instance, design internal forces, normal, shear or equivalent stresses, design deformations, shifts, design pressure upon plate etc.), which is used to assess marginal state;
R = design value of generalized bearing capacity (design strength of structure) determined by normative documents; this is usually the design yield strength of material or limiting pressure on elements, width of the cracks in concrete etc.;
η = safety factor which depends on the various structural elements responsibility for strength and safety of structure.

2.4.1.2 If requirement of 2.4.1.1 is met the following dangerous states can be practically avoided:

excessive deformations of material;
buckling;
fatigue cracks;
brittle fracture.

Accordingly, the following criteria shall be met: ultimate strength; buckling stress; fatigue strength.

2.4.1.3 To prevent brittle fracture of structural elements, the choice of materials, the design of structural details and welding shall comply with the requirements of 1.4 and 1.5.

2.4.2 Ultimate strength criterion.

2.4.2.1 The ultimate strength criterion stipulates requirements aimed at precluding the possibility of reaching a limit state due to plastic deformations and a collapse of MODU/FOP structural element or the entire structure due to single action of the most unfavourable combination of loads possible during service life of the unit.

2.4.2.2 The ultimate strength criterion for survival conditions (extreme impact) is determined by the expression

$$\sigma_d \leq \eta_1 R_d \tag{2.4.2.2}$$

where σ_d = design structural stress caused by the most unfavourable combination of loads, in MPa;
η_1 = safety factor (refer to 2.4.2.5);
R_d = design yield stress of material in accordance with 1.5.1.5, in MPa.

2.4.2.3 Design stresses σ_d in structural elements in the survival conditions or under extreme loads are determined from the following expression:

.1 while assessing stresses in the framing sections and in the plate centre

$$\sigma_d = \sigma_e \tag{2.4.2.3.1}$$

where $\sigma_e = \sqrt{\sigma_x^2 + \sigma_y^2 - \sigma_x\sigma_y + 3\tau^2}$;
σ_x, σ_y and τ = components of structural stresses in the point under consideration, each of them takes into account mutual action of global and local loads;

.2 while assessing the stresses on the plate supporting contour

$$\sigma_d = \sigma_{pl} \tag{2.4.2.3.2}$$

where σ_{pl} = maximum bending stresses determined on the supporting contour under exposure of local loads.

2.4.2.4 The ultimate strength criterion for the operating and transit conditions are determined by the following expressions:

$$\left.\begin{aligned} \sigma_x &\le \eta_1 R_{\dot{d}}, \\ \sigma_y &\le \eta_1 R_{\dot{d}}, \\ \tau &\le 0{,}57\eta_1 R_{\dot{d}} \end{aligned}\right\} \tag{2.4.2.4-1}$$

$$\sigma_{pl} \le \eta_1 R_d \tag{2.4.2.4-2}$$

where σ_x, σ_y and τ = components of structural stresses in the point under consideration, each of them takes into account mutual action of common and local loads, in MPa;
σ_{pl} = maximum bending stresses in a plate determined on the supporting contour under exposure of local loads, in MPa;
η_1 = safety factor (refer to 2.4.2.5);
R_d = design yield stress of material in accordance with 1.5.1.5, in MPa.

2.4.2.5 The safety factors η_1 in connection with the ultimate strength criterion shall not exceed the values shown in Table 2.4.2.5.

Table 2.4.2.5

Design conditions	Type of unit (platform)	Strength criteria	Structural elements		
			Special	Primary	Secondary
Survival or extreme loading	MODU	(2.4.2.3.1)	0,8	0,84	0,86
	TLP/FOP	(2.4.2.3.1)	0,75	0,8	0,83
	MODU	(2.4.2.3.2)	1,3	1,4	1,5
	TLP/FOP	(2.4.2.3.2)	1,25	1,35	1,45
Operation and transit	MODU/FOP	(2.4.2.4-1)	0,63	0,68	0,80
	MODU/FOP	(2.4.2.4-2)	1,02	1,14	1,25

2.4.2.6 Additional criteria of ultimate strength referring to specific type of MODU/FOP as well as explanations required for criteria in Formulae (2.4.2.3.1), (2.4.2.3.2), (2.4.2.4-1) and (2.4.2.4-2) are contained in relevant paragraphs of Section 3.

2.4.3 Buckling strength criterion.

2.4.3.1 The buckling strength criterion stipulates requirements for those parameters of MODU/FOP structural elements which provide stability of the given configuration. Critical buckling strengths are those which cause a structure to pass from one form of equilibrium to another.

2.4.3.2 Buckling strength criterion is determined by the expression

$$\sigma_x \le \eta_2 \sigma_{cr} \tag{2.4.3.2}$$

where σ_x = design stresses for the specified condition of the structural element, in MPa;
σ_{cr} = critical buckling strength, in MPa;
η_2 = safety factor.

2.4.3.3 In buckling strength calculations of compressed and bent cylindrical shells, account shall be taken of geometric imperfections of shape.

2.4.3.4 Flexibility of isolated compressed elements shall not be more than

$$\lambda = l_e/\rho \leq \lambda_{max} \qquad (2.4.3.4)$$

where l_e = effective unsupported length of the beam, in mm;
ρ = minimum radius of inertia of the sectional area, in mm;
λ_{max} = maximum permissible flexibility as per Table 2.4.3.4.

Table 2.4.3.4

Normative yield strength of material, R_{eH}, in MPa	Maximum permissible flexibility λ_{max}
240	165
315	155
355	150
390	150
420	150
460	140
500	130

2.4.3.5 While checking the buckling strength of isolated compressed elements the safety factor η_{20} shall not be more than

$$\eta_{20} = 0{,}67, \text{if } \lambda \geq \lambda_0;$$
$$\eta_{20} = 0{,}84(1 - 0{,}2\lambda/\lambda_0), \text{if } \lambda < \lambda_0; \qquad (2.4.3.5)$$

where $\lambda_0 = \sqrt{2\pi^2 E/R_{eH}}$;
E = Young's modulus, in MPa;
R_{eH} = yield stress of material, in MPa.

2.4.3.6 The safety factor η_2 of bars subjected to combined axial compression and bending shall meet the condition

$$\eta_2/\eta_{20} + \sigma_{xbend}/[\sigma] \leq 1 \qquad (2.4.3.6)$$

where η_{20} = safety factor according to 2.4.3.5;
σ_{xbend} = acting stress caused by bending, in MPa;
$[\sigma]$ = permissible stresses, in MPa, (in accordance with 2.4.2, i.e. $[\sigma] = \eta_1 R_{\mathrm{d}}$.

2.4.3.7 The safety factor of plate elements exposed to system of forces on the edges, which may cause budding, shall be determined by the formula

$$\eta_2 = \sqrt{\sum_{i=1}^{n} \left(\sigma_i/\sigma_{i_{cr}}\right)^2} \qquad (2.4.3.7)$$

where n = number of simple forms of stresses which may be used to represent the actual loaded condition. Examples of such stresses are: compression in x and y directions; average shear stresses;
σ_i = actual stresses of the *i*-th form, in MPa;
$\sigma_{i_{cr}}$ = critical stresses corresponding to the *i*-th form of stresses, in MPa.

The safety factor η_2 shall be assumed equal to:
η_2=0,8 for survival conditions or extreme loads;
η_2 = 0,6 — for all other modes.
2.4.3.8 Buckling strength calculation of unstiffened tubular elements, the interrelation of common and local loss of budding may be omitted for:
elements which are subject to bending and compression at

$$D/t \leq 0{,}1E/R_{eH}; \quad (2.4.3.8\text{-}1)$$

elements which are subject to bending, compression and excessive external pressure at

$$D/t \leq \sqrt{0{,}45E/R_{eH}} \quad (2.4.3.8\text{-}2)$$

where D and t = average diameter and thickness, respectively, mm, of tubular element wall;
E = refer to 2.4.3.5;
R_{eH} = refer to 1.5.1.5.

If the above inequalities are not executed, then it is necessary to take into account interaction of local and common buckling in calculation of the tubular element buckling strength. Applied methods of calculation shall be agreed with the Register.
2.4.3.9 Register may consider the possibility of loosing stability of the horizontal plates of primary and secondary structural elements. In such case methods of calculation and permissible stresses shall be justified and agreed with the Register.
2.4.4 Fatigue strength criterion.
2.4.4.1 The fatigue strength criterion stipulates requirements aimed at preventing the origination of dangerous, by possible consequences, fatigue damage during service life of the structure which is caused by unsteady change of operating loads of different magnitude.
2.4.4.2 Calculation of the fatigue strength is made for critical points which list is agreed by the designer with the Register.
2.4.4.3 Designing of the platform structures shall be made on the basis of the "safe damage" criterion which implies that fatigue criterion is oriented at the stage of initiation of fatigue macrocrack rather than the stage of crack development. Crack initiation criterion is based on the hypothesis of linear damage summation shown by the expression

$$\sum_{i=1}^{i=K} n_i/N_i \leq \eta \quad (2.4.4.3)$$

where n_i = the number of stress cycles at the *i*-th level of loading;
N_i = number of stress cycles prior to appearance of the crack at the *i*-th level of loading;
K = number of loading levels considered;
η = permissible limit level of relative vulnerability.

2.4.4.4 Permissible limit level of relative vulnerability η depends on the class of the structural element (refer to 1.4), category of joint within the class of responsibility, degree of access for examination and repair. Category of the joint is assigned by the designer.
2.4.4.5 Parameter η represents a product of

$$\eta = \beta_1 \beta_2$$

where β_1 and β_2 values are given in Tables 2.4.4.5-1 and 2.4.4.5-2.

Table 2.4.4.5-1

β_1 coefficient

Class of the structural element	Category of joint under consideration	
	I	II
Special	0,8	0,6
Primary	0,9	0,8
Secondary	1,0	1,0

Table 2.4.4.5-2

β_2 coefficient

Access during examination and repair		
No access	Hard-to-reach	Good access
0,5	0,75	1,0

2.4.4.6 The source of cyclical loads is waves, wind, current, ice, seismicity reason, vibration of machinery. The initial data for each type of cyclical loads is recurrence of environmental conditions (refer to 2.2.1 — 2.2.6).

2.4.4.7 Service life is recommended to be determined by the following formula:

self-elevating MODU

$$T_d = N_y \sigma_y^m / \sum_i \sum_j \sum_k \frac{p_{ijk} \Gamma(1+m/K_{ijk})(1+\beta_{ijk}) a_{v_{ijk}}^m}{T_{e_{ijk}}} \qquad (2.4.4.7\text{-}1)$$

where N_y, σ_y, m = parameters of fatigue curve;
σ_y = fatigue limit based on N_f cycles;
m = slope of fatigue curve in coordinates $\lg\sigma - \lg N$.
$T_{e_{ijk}}$ = effective period of the process of wave loads at *ijk*-th stationary conditions, featuring *i*-th height of the 3 % probability of exceeding level, *j*-th average period of waves, *k*-th angle of encounter;
p_{ijk} = recurrence of *ijk*-th stationary conditions;
K_{ijk} and $a_{v_{ijk}}$ = parameters of form and scale of the stress distribution, respectively (refer to 3.1.4.9);
$\beta_{ijk} = a_{w_{ijk}}/a_{v_{ijk}}$ (refer to 3.1.3.6, 3.1.4.9);
$\Gamma(\cdot)$ = gamma function.

In addition to Formula (2.4.4.7-1), the total fatigue damage D_Σ due to waves and wind for self-elevating MODU structures may be determined by the following formula:

$$D_\Sigma = D_{wave} + D_{wind} + D_{vortex}$$

where $D_{wave}, D_{wind}, D_{vortex}$ = fatigue damage due to waves, pulse component of wind loading and vortex component of wind load, respectively;

semi-submersible MODU

$$T_d = N_y \sigma_y^m / \sum_i \sum_j \sum_k \frac{p_{ijk} 2^{m/2} \Gamma(1+m/2) \sigma_{v_{ijk}}^m}{T_{e_{ijk}}} \qquad (2.4.4.7\text{-}2)$$

where $\sigma_{v_{ijk}}$ = standard deviation of the stress process at the *ijk*-th stationary wave conditions.

TLP/FOP fatigue life at wave, seismic or variable ice loads is recommended to determine on the basis of the analytical dependency

$$T_d = N_y \sigma_y^m / \sum_i \frac{p_i \Gamma(1+m/K_i) a_i^m}{T_{e_i} \sigma_y^m} \quad (2.4.4.7\text{-}3)$$

where T_{e_i} = effective period of *i*-th process;
a_i and K_i = parameters of scale and form of the *i*-th process (refer to 3.3.2.1.4, 3.3.2.4.4, 3.3.2.3.3, 3.3.2.3.5);
N_y, σ_y, m = parameters of fatigue curve; σ_y = fatigue limit on the basis of N_y cycles;
m = slope of fatigue curve in coordinates $\lg\sigma - \lg N$.

2.4.4.8 To assess preliminarily the risk of origination of fatigue damage and to determine the main scantlings of hull structures it is recommended to use the Register modified fatigue curves as fatigue curves (refer to Figs. 2.4.4.8-1 and 2.4.4.8-2) in accordance with the international classification of structural types of nodes and joints (classes B, C, D, E, F, F2, G, W and T).

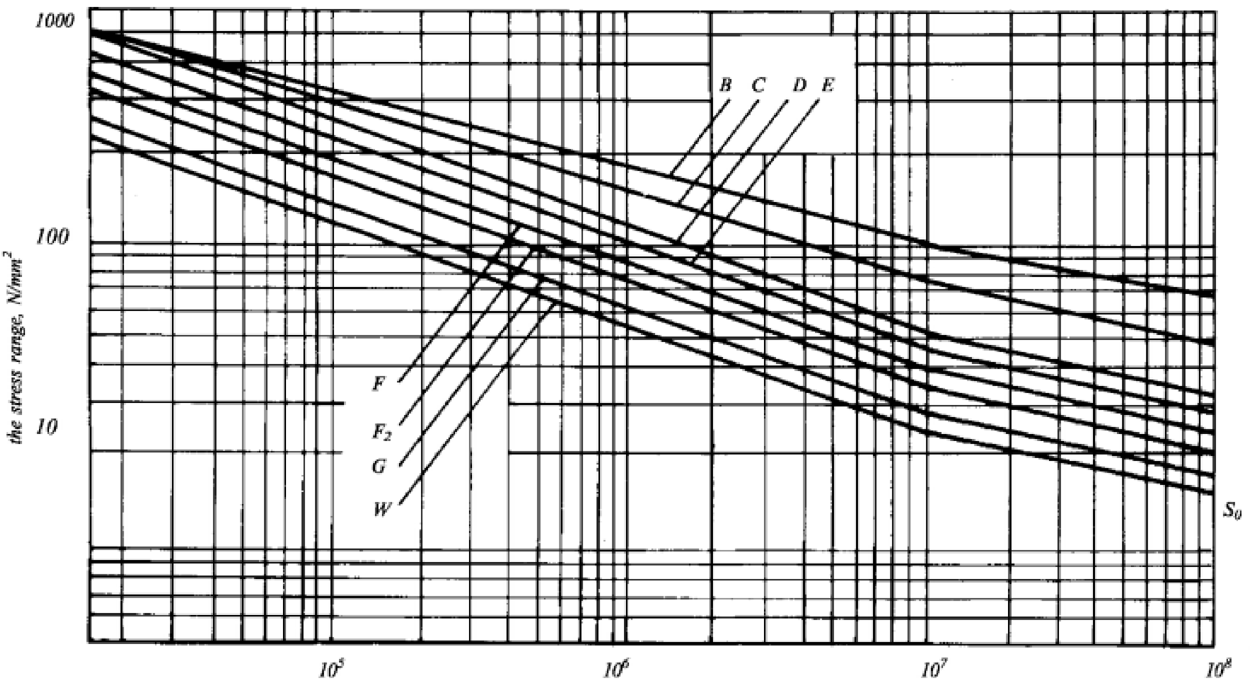

Fig. 2.4.4.8-1
Fatigue curves

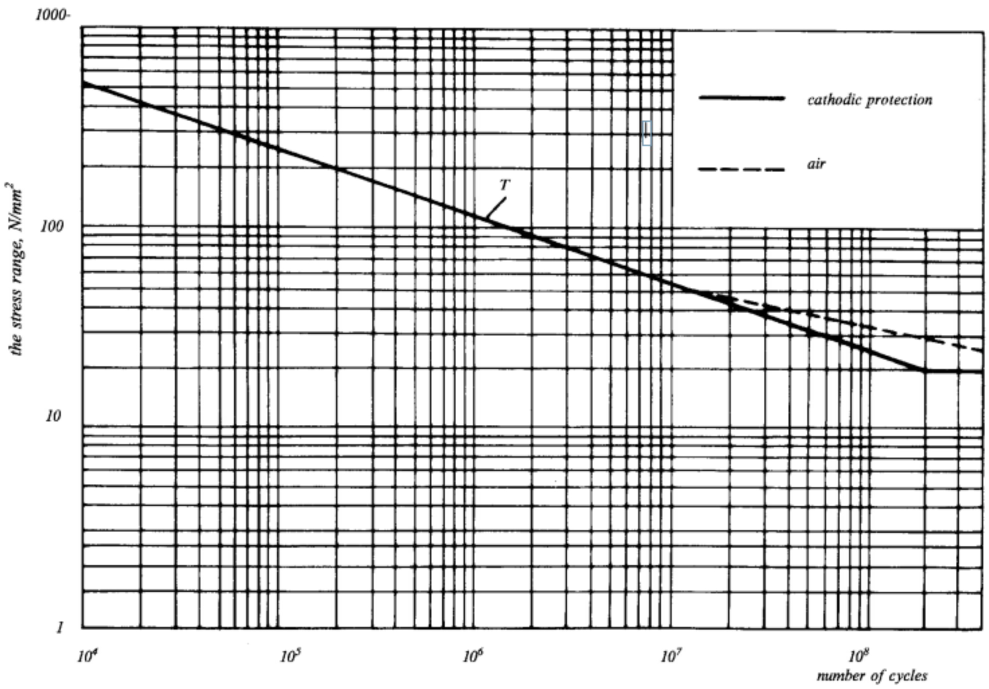

Fig. 2.4.4.8-2
S-N curves for tubular joints

2.4.4.9 Fatigue curves are applicable to 22 mm materials for flat structures and 32 mm for tubular structures. Fatigue limit for the given thickness of elements differing from the basic ones is determined by the formula

$$\sigma_y^+ = \sigma_y (t_B/t)^{1/4} \quad (2.4.4.9)$$

where t_B = basic thickness;
t = real thickness.

2.4.4.10 The design stress range for base metal in calculations of fatigue curves as in Figs. 2.4.4.8-1 and 2.4.4.8-2 may be reduced depending on the mark of average stresses. Reduction coefficient μ which reduces the stress range is shown in Fig. 2.4.4.10.

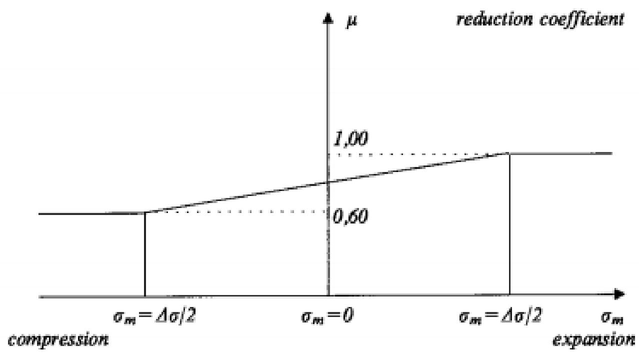

Fig. 2.4.4.10
The stress range for $S - N$ curves. The base metal

2.4.4.11 The service life of the structure T_{ser} is defined by the customer. Design service life T_d of structures shall be $T_d \geq 1{,}1T_{ser}$.

2.4.4.12 Where the obtained results of the service life assessments made in accordance with 2.4.4.8 — 2.4.4.10 suggest that the origination of premature fatigue fractures in welded joints of the structure types used is possible, it is necessary to made more detailed calculations of fatigue strength of the welded joints using methods approved by the Register, which take into account the main design and technological factors and define fatigue serviceability of welded joints. It is also necessary to choose an appropriate implementation of the welded joints and hull assemblies which ensure the required design service life accounting for the use of special technological methods for enhancing the fatigue strength of the joints in accordance with 2.8.7 of Part XIII "Welding" of the MODU/FOP Rules.

2.5 STRENGTH CALCULATION PROVISIONS

2.5.1 General.

2.5.1.1 Strength calculations are divided into the following stages:

evaluation of values, characteristics and distribution of design common and local loads, their combination for the given operating conditions;

evaluation of stresses caused by the common and local loads, adding of stresses;

evaluation of design stresses or evaluation of ultimate loads;

comparison of design values versus requirements of the MODU/FOP Rules.

All major parts of the calculation are equally important, the same requirements are put to their accuracy and justification as to the whole calculation.

2.5.1.2 Calculations shall be made following the recognized methods. Provisions of the Rules for the Classification shall be used if applicable.

2.5.1.3 Idealized structural model shall reflect peculiarities of structure: mutual location and geometry of primary support members, section geometrical characteristics. Meanwhile, the idealized structural models shall be subdivided into subsystems of various levels.

Requirements for the design models are set forth in 2.5.2 — 2.5.5 and Appendix 2.

2.5.1.4 It is allowed not to consider stress components which value is less than 10 % of design yield strength of material while making calculation of the structural elements which are exposed to multicomponent stress and deformation.

2.5.1.5 Additional provisions on strength calculations referred to MODU/FOP of specific type are contained in relevant paragraphs of Section 3.

2.5.2 Evaluation of common stresses.

2.5.2.1 MODU/FOP structural models keeping due note of their macropeculiarities shall be developed in order to evaluate common stresses (or stresses caused by common loads) which relate to common structural deformations. Usually, calculation of the structural stress and deformation is performed on the basis of single calculation scheme, i.e. it is recommended to consider the structure as a whole.

Use of simplified calculation schemes (for parts of structures) is allowed if their use is justified. In any case the model shall be detailed as far as it is necessary for evaluation of common stresses.

2.5.2.2 Calculation of MODU/FOP stresses and deformations is generally recommended to perform following the finite element method on the basis of the beam, plate and beam-and-plate idealization.

2.5.2.3 MODU/FOP strength calculation shall take into account the interaction of the structure with the seabed. While modelling of the "structure- seabed" system, the latter may be represented by reactive forces or elastic springs in the finite element nodes which generally resist to vertical and horizontal shifts.

2.5.2.4 If buckling is allowed under compression forces (refer to 2.4.3.9), it is necessary to reduce flexible members (plates) according to the following scheme:

0,25 of the shorter side of the plate supporting contour adjoining the longitudinal and transverse beams on both sides are not subject to reduction;

residual (reduced) part of plate is used in calculations with the reduction coefficients:

$$\varphi_1 = \sigma_{x,cr}/\sigma_x,\ \varphi_2 = \sigma_{y,cr}/\sigma_y \qquad (2.5.2.4)$$

where σ_x, σ_y = general compressing stresses which act in rigid members (absolute values) in the longitudinal and transverse directions respectively;

$\sigma_{x,cr}$, $\sigma_{y,cr}$ = critical stresses in flexible members which cause buckling if they act simultaneously.

After reduction of flexible members, the design compression stresses acting in rigid members shall be determined in the second approximation. If the second approximation of stresses differs from the stresses of the first approximation by less than 5 %, no more refinement is needed. Otherwise, the third and further approximations are needed.

2.5.3 Girder system calculation.

2.5.3.1 In general case, calculation of the girder system (grillage, frame) or its separate elements shall be based on the calculation scheme which takes into account interrelation of neighboring structural elements.

The beam, plate and beam-and-plate models may be used for the grillage calculation. Simplified calculation schemes may be used if it is justified.

2.5.3.2 Section moduli and moments of inertia of frames during calculation shall be determined taking account the effective flange, which thickness is taken equal to its average thickness in the beam cross section under consideration.

The width of the effective flange b_{fl} of stiffeners is taken equal to the least of the following values determined by the formula

$$b_{fl} = l/6; \tag{2.5.3.2-1}$$

$$b_{fl} = 0{,}5(b_1 + b_2) \tag{2.5.3.2-2}$$

where l = considered frame span between supports, in m;
b_1, b_2 = distance of the considered frame from the nearest frames of the same direction which are located on both sides of the considered frame, in m.

The width of the effective flange of ends is determined by the formula

$$b_{fl} = kb \tag{2.5.3.2-3}$$

where k = coefficient taken from Table 2.5.3.2 in relation to b, the given length of the frame span l_{sp} and number of frames n supported by the deep member in question.

For simply supported ends the given span $l_{sp} = l$, and for clamped ends $l_{sp} = 0{,}6l$. The way in which the framing members shall be supported (fixing or simply supported) is determined on the basis of general engineering principles proceeding from the actual structure (brackets, welding of webs, face plates etc) and it is characterized by presence or absence of bending moment effects in the span point of the member.

Table 2.5.3.2

n	l_{sp}/b						
	1	2	3	4	5	6	7 and more
≥6	0,38	0,62	0,79	0,88	0,94	0,98	1,0
≤3	0,21	0,40	0,53	0,64	0,72	0,78	0,80
Note. For intermediate values of l_{sp}/b and n, coefficient k is determined by means of the linear interpolation.							

2.5.3.3 Transverse section area of stiffeners or girders taking up axial forces shall be determined taking into account effective flange which width is equal to half-sum of distance of the frame in question from the nearest frames of the same direction which are located on both sides.

2.5.3.4 The area of the web cross section shall be determined keeping due note of cut-outs in the design section (net section).

2.5.3.5 Usually, sections with the maximum normal, shear stresses or their combination are taken for design sections. Summation of stresses caused by common and local loads shall be carried in order to meet strength criteria in Formulae (2.4.2.3.1) and (2.4.2.4-1).

2.5.4 Calculation of plates.

Calculation of plating is based on the assumption that they are clamped ends. Usually, design load is treated as equally distributed over the plate.

On the basis of this assumption the bending normal stresses are determined in the middle part of the plate which are summed with the common structural stresses in order to meet strength criterion in Formula (2.4.2.3.1) and maximal bending stresses on the supporting contour in order to meet strength criteria in Formulae (2.4.2.3.2) and (2.4.2.4-2).

2.5.5 Buckling strength of structural elements.

2.5.5.1 Buckling strength calculations are made to meet 2.4.3 criteria. It is recommended to use calculation scheme taking into account interrelation of adjacent structural elements for calculation of structural elements buckling strength. Otherwise, the structural element (girder, frame element, plate, etc.) shall be treated as simply supported along the contour.

2.5.5.2 It is necessary to consider deviation from the Huge's law to determine critical stresses. In such case the critical normal stresses σ_{cr} are determined by the formulae:

$$\sigma_{cr} = \sigma_e \text{ at } \sigma_e \le 0{,}6R_{eH}; \quad (2.5.5.2\text{-}1)$$

$$\sigma_{cr} = R_{eH}(1{,}113 - 0{,}32R_{eH}/\sigma_e) \text{ at } 0{,}6\,R_{eH} < \sigma_e < 2{,}4\,R_{eH}; \quad (2.5.5.2\text{-}2)$$

$$\sigma_{cr} = R_{eH} \text{ at } \sigma_e \ge 2{,}4\,R_{eH} \quad (2.5.5.2\text{-}3)$$

where σ_e = the Euler normal stress, in MPa.

Steel yield strength for shear stresses is $\tau_T = 0{,}57R_{eH}$ when the value of the tangential stresses is determined.

2.5.5.3 When structural members are subjected to axial compression or combined axial compression and bending, they shall comply with the following requirement:

$$\sigma_a/\sigma_a^* + \sigma_{ab}/\sigma_{ab}^* \le 1{,}0 \quad (2.5.5.3)$$

where σ_a = computed axial compressive stress, in MPa;
σ_{ab} = computed compressive stress due to bending, in MPa;
$\sigma_{ab}^* = \sigma_i^*$ or σ_b^* – for bending stress, in MPa;
$\sigma_i^* = \eta R_{eH}$ – for axial bending stress, in MPa;
$\sigma_b^* = \eta\sigma_{cr}$ – for compression or bending, in MPa;
σ_{cr} – critical compressive buckling stress;

$\sigma_a^* = \eta\sigma_{cr,i}(1 - 0{,}13\lambda/\lambda_0)$, if $\lambda < \lambda_0$;

$\sigma_a^* = \eta\sigma_{cr,e} \cdot 0{,}87$, if $\lambda \ge \lambda_0$;

σ_a^* - shall not exceed σ_{ab}^*;
η =0,6 – for static loads;
η = 0,8 – for combined loads;

$\lambda = l_e/\rho$;

$\lambda_0 = \sqrt{2\pi^2 E/\sigma_y}$;

$\sigma_{cr,i}$	=	inelastic column critical buckling stress, in MPa;
$\sigma_{cr,e}$	=	elastic column critical buckling stress, in MPa;
l_e	=	effective unsupported length of the beam, in mm, according to Formula (2.4.3.4);
ρ	=	minimum radius of inertia of the sectional area, mm;
E	=	modulus of elasticity of the material (Young's modulus), in MPa;
σ_y	=	minimum tensile yield stress of the material, in MPa.

2.5.5.4 Provisioning of the local stability of the framing elements (webs, flange), local strengthening shall be carried out in accordance with the Rules for the Classification.

2.5.6 Helideck strength calculation.

2.5.6.1 Dimensions of helideck members and its supporting structures shall be determined according to 6.2 of Part XVII "Distinguishing Marks and Descriptive Notations in the Class Notation Specifying Structural and Operational Particulars of Ships" of the Rules for the Classification.

3 STRENGTH ISSUES SPECIFIC TO PLATFORMS

3.1 SELF-ELEVATING MODU

3.1.1 General.

3.1.1.1 The structural strength of self-elevating MODU shall be tested, on the basis of criteria mentioned under 2.4, for five design conditions:

survival;
operational;
transit;
positioning at a site;
removal from site.

For positioning at site and removal from site (preloading and pulling out of legs), the safety factors and strength criteria shall be chosen as for the survival condition.

3.1.1.2 A self-elevating MODU shall have a clearance, in m, not less than

$$h_c \geq 0{,}6h_{50} + \Delta h_{50} + 1{,}50 \qquad (3.1.1.2)$$

where h_{50} = extreme wave height, in m, (once in 50 years) for the sea area in question;
Δ_{50} = extreme tide, in m, for the basin (once in 50 years).

3.1.1.3 When a self-elevating MODU is prepared for a transit condition to last a day or more, the helideck height $H_{h.d.}$, in m, above the calm waterline shall be determined by the formula

$$H_{h.d.} = 1{,}80 \cdot 10^{-3} q^{2,5} + 3(x/q) + 2(h_{50}/12 - 1) + 1{,}2(\tau - 1)^{0,7} \qquad (3.1.1.3)$$

where $q = \sqrt[3]{\Delta}$;
Δ = cubic displacement, in m³, of a MODU in transit;
x = distance, in m, from the farthest edge of helideck to the centre-of-gravity position of a MODU, as measured along the hull length;
h_{50} = height, in m, of the wave occurring with the periodicity once in 50 years;
τ = voyage duration, in days, not exceeding four.

3.1.1.4 Wind, wave and seismic loads shall be determined for the most unfavourable angles of wave propagation and wind attack.

3.1.1.5 When making the dynamic strength analysis of a self-elevating MODU, the lowest natural frequency (s^{-1}) of bending vibrations shall be determined by the formula

$$p = \sqrt{\frac{12n_k E J_k(1 - G_p/n_k P_e)g}{l^3(4 - 3æ)(G_p + 0{,}5n_k G_k)}}\eta_d \qquad (3.1.1.5)$$

where n_k = leg number;
E = elastic modulus, in kPa, of leg material;
J_k = equivalent moment of inertia, in m⁴, of a leg cross – sectional area with regard to the principal centroidal axis (refer to 3.1.2.3);
G_p = pontoon mass, in kN;
G_k = one leg mass, in kN;
$P_e = \frac{\pi^2 E J_k}{4l^2}(3æ + 1)$ = Euler load, in kN, upon a leg as a part of a space frame;
g = gravity acceleration, in m/s²;
l = design leg length, in m, equal to the distance between the leg footing and a point located at half the distance between the horizontal supports of a pontoon;
$æ$ = supporting pair factor, refer to 3.1.2.2;
η_d = correction factor accounting for the effect of leg securing in a pontoon, refer to 3.1.2.4.

3.1.2 Design structural diagram of self-elevating MODU.

3.1.2.1 For assessing the stressed condition, idealization of structures on several levels is used (refer to Fig. 3.1.2.1):

structural frame ("superelement");

leg section under consideration.

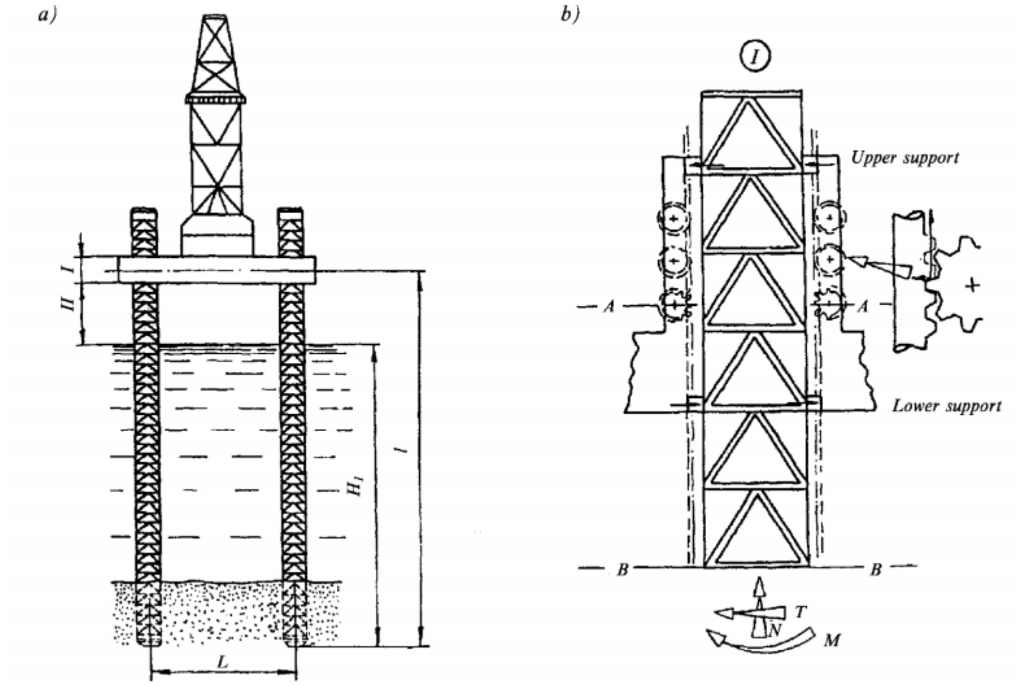

Fig. 3.1.2.1
Idealization of self-elevating MODU structures of different levels:
a) structural frame ("superelement"); *b)* leg section (I) under consideration

3.1.2.2 The leg-seabed interaction is accounted for by the supporting pair factor æ which characterizes the degree of leg embedment with regard to the leg turning in the seabed. In case of leg bending by low-frequency pattern, the value of æ will depend on the leg parameters and seabed, as described by the formula

$$æ = 1/(1 + AEJ_k/l) \tag{3.1.2.2}$$

where A = coefficient of proportionality between the supporting moment and the turning angle of the footing;
for E, l, J_k, refer to 3.1.1.5.

3.1.2.3 The moment of inertia J_k of the surface area of a truss-type leg can be referred to the moment of inertia of an ideal section of all the longitudinal elements forming the leg as

$$J_k = J_u/\mu \tag{3.1.2.3}$$

where J_u = moment of inertia, in m^4, of an ideal section;
μ = reduced rigidity characteristic depending on the type of structural module, geometrical characteristics of its elements and relative leg length.

3.1.2.4 The correction factor depends η_d on the distance $d,$ in m, between the lower and upper horizontal supports, on the correlation between the bend and shift rigidity of the leg (where *B* is leg breadth, in m), on the degree of leg embedment in the seabed (refer to Fig. 3.1.2.4).

In case of a non-typical installation of the lifting mechanism (without dampers, for instance), a special analysis of the area where the leg is fitted in the hole may be submitted to the Register for homologation with a correction of vibration frequences and with load redistribution among mechanisms and supports.

3.1.2.5 The flexibility coefficients A shall be determined by the formulae:

for vertical vibrations —

$$A = (1 - v)/4Gr_0; \qquad (3.1.2.5\text{-}1)$$

for horizontal vibrations —

$$A = (2 - v)/8\mathrm{G}r_0; \qquad (3.1.2.5\text{-}2)$$

for rotational vibrations —

$$A = 3(1 - v)/8\mathrm{G}r_0^3; \qquad (3.1.2.5\text{-}3)$$

for torsional vibrations —

$$A = 3/16\mathrm{G}r_0^3 \qquad (3.1.2.5\text{-}4)$$

where G = seabed shift modulus, in MPa;
v = Poisson's ratio;
r_0 = foundation radius, in m.

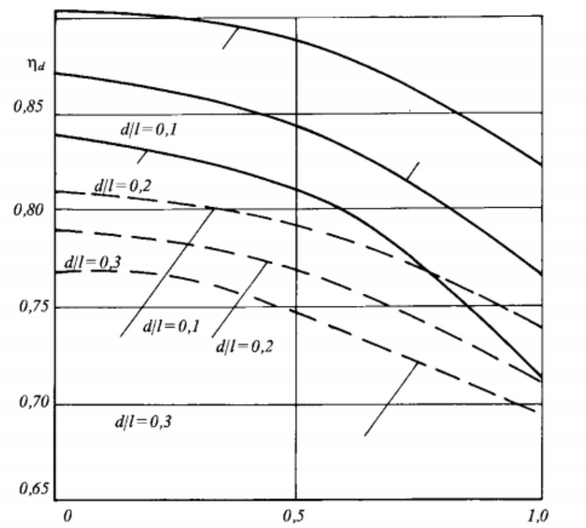

Fig. 3.1.2.4
Correlation between the correction factor η_d and the parameters æ, $d/l,\ B/l$
—— $B/l = 0{,}1$ — — $B/l = 0{,}2$

Where the foundation is rectangular with the sides $B \times L$, an equivalent radius shall be introduced, as follows:

$r_0 = \sqrt{BL/\pi}$ for vertical and horizontal vibrations;

$r_0 = \sqrt[4]{BL^3/3\pi}$ for rotational vibrations around the horizontal axis;

$r_0 = \sqrt[4]{BL^3(B^2 + L^2)/\pi}$ for torsional vibrations.

For embedded legs whose friction layer lies at some depth, the flexibility coefficient for rotational vibrations shall be determined by the formula

$$A = 3(1 - v)/16Gr_0^3. \quad (3.1.2.5\text{-}5)$$

3.1.2.6 The legs' most loaded area is located within the upper and lower rails, where the loads from columns are transmitted to the hull.

The bending moment in this area is partially formed by horizontal forces from guides, partially by vertical forces of the lifting mechanism. Relative contribution of vertical M_v and horizontal M_h forces is determined by the parameter β:

$$\beta = M_v/(M_v + M_h). \quad (3.1.2.6\text{-}1)$$

During total strength analysis the hull stiffness may be generally accepted infinitely large in comparison with the column stiffness. The leg mechanism is presented by rotation spring with rotational stiffness K_m. In this case the parameter β shall be determined by the formula

$$\beta = \frac{1}{1+\Delta_{z_g} GF_c/K_m} \quad (3.1.2.6\text{-}2)$$

where G = shear modulus of the column material;
F_c = shearing area of the column, in m²;
Δ_{z_g} = distance between the upper and lower guides, in m;

$$K_m = \frac{1}{2}Kb^2 \quad (3.1.2.6\text{-}3)$$

where $K = \frac{1}{1/K_{bend}+1/K_{shear}}$;

K_{bend} = ending stiffness of the column;
K_{shear} = shear stiffness of the column.

3.1.3 Wind loads.

3.1.3.1 Wind loads shall be determined by Formulae (2.3.8-1) — (2.3.8-3).

3.1.3.2 It is recommended that the dynamics of wind load application be considered beginning from the period of natural bending vibrations of the first mode $\tau = 130/\overline{w}_{10}$, in s.

In this case, the amplification factor K_w shall be used proceeding from Fig. 3.1.3.2 in which

$$v_w = \omega_{max}/p \quad (3.1.3.2)$$

where $\omega_{max} = 4 \cdot 10^{-3}\overline{w}_{10}$ is the modal frequency of spectral density of wind pulsation;
p = natural bending vibration frequency of a self- elevating MODU;
δ_w/π = relative vibration decrement of self-elevating MODU.

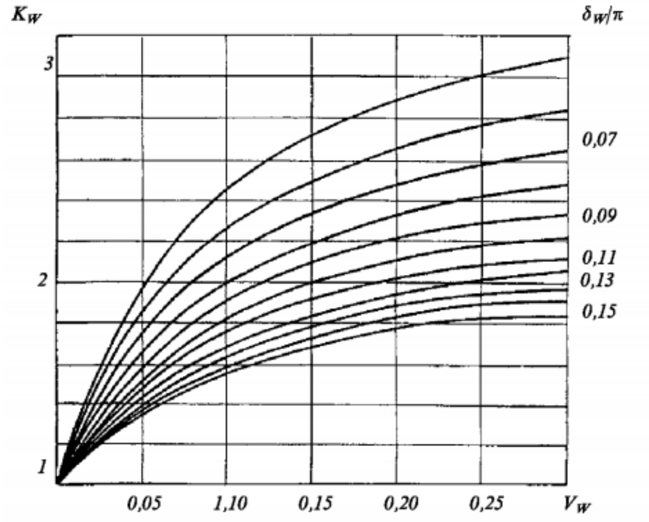

Fig. 3.1.3.2
Amplification factor of wind loads

3.1.3.3 When considering the pulse component of wind loads, the nonsynchronous character of their effect shall be accounted for by using the factor η.

The nonsynchronous factor of wind loads η is determined by the following formula:

$$\eta = \frac{\sum_i K_{li} K_{2i} S_i r_i}{\sum_i K_{li} K_{2i} S_i} \qquad (3.1.3.3)$$

where r_i = coefficients of correlation between wind pressures on the structures:
a) extended the full width and height of the unit;
b) fitted along the wind direction.

In the first approximation, generalized nonsynchronous factor of wind loads shall be assumed as $\eta = 0{,}8$.

3.1.3.4 For each stationary mode, the values of internal forces of proportional wind actions shall be determined:

for a four-legged self-elevating MODU —

$\overline{M_w} = 0{,}09\overline{Q_w}l(2 - æ)$, is the bending moment;

$\overline{T_w} = 0{,}18\overline{Q_w}$, is the shearing force; (3.1.3.4-1)

$\overline{N_w} = 0{,}18\overline{Q_w}\frac{l}{L}(2 - æ)$, is the axial force;

for a three-legged self-elevating MODU —

$\overline{M_w} = 0{,}165\overline{Q_w}l(2 - æ)$, is the bending moment;

$\overline{T_w} = 0{,}33\overline{Q_w}$, is the shearing force; (3.1.3.4-2)

$\overline{N_w} = 0{,}58\overline{Q_w}\frac{l}{L}(2 - æ)$ is the axial force

where $\overline{Q_w}$ = the value for $w_{10} = \overline{w}_{10}$, refer to Formulae (2.3.8-1) — (2.3.8-3);
L = the clear spacing between legs, refer to Fig. 3.1.2.1.

3.1.3.5 For each stationary mode, standard deviations of the components of internal wind pulsation forces shall be determined:

for four-legged self-elevating MODU —

$$\sigma_M^w = 0{,}18\overline{Q_w}\eta l(2-æ)\vartheta_w K_w;$$

$$\sigma_T^w = 0{,}36\overline{Q_w}\eta\vartheta_w K_w; \qquad (3.1.3.5\text{-}1)$$

$$\sigma_M^w = 0{,}36\overline{Q_w}\eta\frac{l}{L}(2-æ)\vartheta_w K_w;$$

for three-legged self-elevating MODU —

$$\sigma_M^w = 0{,}33\overline{Q_w}\eta l(2-æ)\vartheta_w K_w;$$

$$\sigma_T^w = 0{,}66\overline{Q_w}\eta\vartheta_w K_w; \qquad (3.1.3.5\text{-}2)$$

$$\sigma_M^w = 1{,}15\overline{Q_w}\eta\frac{l}{L}(2-æ)\vartheta_w K_w$$

where ϑ_w = wind pulsation variability coefficient, equal to $\vartheta_w = 2{,}45\sqrt{K_{fr}}$;

K_{fr} = front resistance coefficient of underlying surface, refer to Table 2.2.2.4-1.

3.1.3.6 The scale parameter a_w of internal forces due to wind is assessed as

$$a_w = 0{,}85\sigma_w \qquad (3.1.3.6)$$

3.1.4 Wave loads.

3.1.4.1 Wave loads on leg elements of self- elevating MODU shall be determined in accordance with 2.3.9.1. For round and rectangular sections, the values of the inertia coefficient C_{in} and speed resistance coefficient C_{sr} shall not be less than those to be found in Fig. 3.1.4.1. When rack is available, the resistance coefficient C_{drag}^r shall be determined by the formula

$$C_{drag}^r = C_{sr} + 4\frac{a+b/2}{D} \qquad (3.1.4.1)$$

where D = bore diameter,

a = tooth root height;

b = rack tooth height.

For more complex shapes, the estimated values of the C_{in}, and C_{sr} coefficients shall be agreed with the Register.

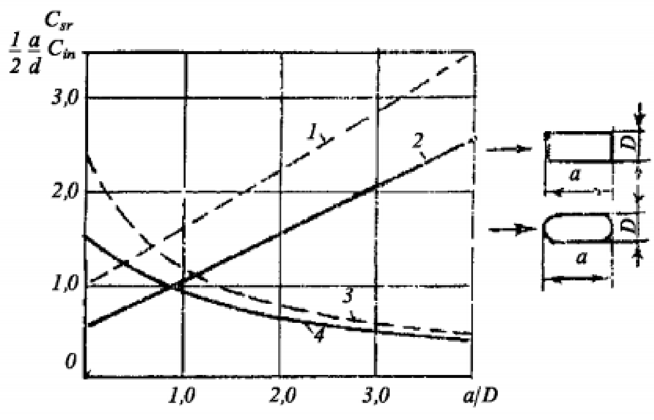

Fig. 3.1.4.1
Speed resistance coefficient C_{sr} (3,4) and inertia resistance coefficient C_{in} (1,2) for sections:
——— elliptical sections (round sections where $a/D = 1$);
- - - - - rectangular sections

3.1.4.2 When determining the wave loads, the effect of marine growth on the structure shall be considered which manifests itself in the increase of scantlings and the values of C_{in}, and C_{sr} coefficients as compared to those given.

3.1.4.3 The dynamic character of wave load application shall be assessed by means of curves given in Fig. 3.1.4.3 where $\overline{\omega}$ is the average period of surface waves, p is the natural bending vibration frequency, δ/π is the relative decrement of vibrations.

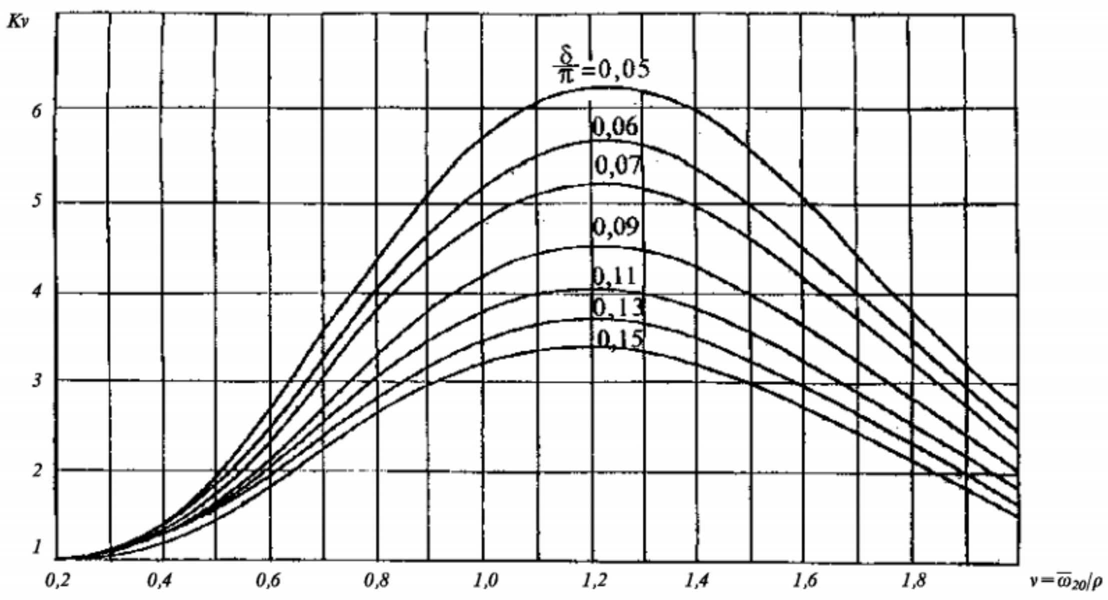

Fig. 3.1.4.3
Amplification factors of speed and inertia components of wave load

3.1.4.4 The hydrodynamic loads on the leg modulus consisting of a combination of vertical, horizontal and inclined members shall be determined by memberwise summation of hydrodynamic loads with due regard for the spacing of members, which is equivalent to introducing factors to account for wave load influence upon the horizontal and inclined members μ_{sr} and μ_{in}:

$$\mu_{sr} = 1 + \frac{\sum_{i=1}^{n\Sigma} n_i d_i l_i C_{sr}^{i}(\theta_i)\cos^2\theta_i}{n_B d_B \Delta z C_{sr}^{B}}; \tag{3.1.4.4-1}$$

$$\mu_{in} = 1 + \frac{\sum_{i=1}^{n\Sigma} n_i d_i^2 l_i C_{in}^{i}(\theta_i)\cos\theta_i}{n_B d_B \Delta z C_{in}^{B}} \tag{3.1.4.4-2}$$

where d_B = transverse dimension, in m, of vertical batten;
n_Σ = total number of horizontal and inclined members;
$d_i,\ l_i$ = diameter and length, in m, of inclined and horizontal members, respectively;
Δz = module height, in m;
θ_i = angle, in deg, formed by an inclined member and a plane perpendicular to the direction of wave propagation;
$C_{sr}^B,\ C_{in}^B$ = speed and inertia resistance coefficients of vertical members (bearing battens);
$C_{sr}^i,\ C_{in}^i$ = speed and inertia resistance coefficients of inclined and horizontal members.

The values of $C_{sr}^B,\ C_{in}^B, \mu_{sr}, \mu_{in}$ shall be determined for the design course angle φ_d in accordance with 3.1.4.6.

3.1.4.5 For the purpose of wave load calculations, there may be omitted:
inertia component where

$$h_3 \geq 8{,}5 d_B C_{in''}^{B} \cdot \bar{u}_{in'} / C_{sr}^{B} \mu_{sr} \bar{u}_{sr}; \tag{3.1.4.5-1}$$

speed component where

$$h_3 \leq 2{,}1 d_B C_{in''}^{B} \cdot \bar{u}_{in'} / C_{sr}^{B} \mu_{sr} \bar{u}_{sr} \tag{3.1.4.5-2}$$

where $\bar{u}_{in}$ and $\bar{u}_{sr}$ = form ordinate values of leg vibrations of a self- elevating MODU on the level of the applicate of wave pressure resultants corresponding to the inertia and speed components:

$\bar{u}_{in} = \bar{u}$ if $z = z_{in} = H_1(1 - \Phi)$;

$\bar{u}_{sr} = \bar{u}$ if $z = z_{sr} = H_1(1 - \Phi/2)$;

$$\bar{u} = \frac{6(1-æ)}{4-3æ} \cdot \frac{z}{l} + \frac{3æ}{4-3æ}(z/l)^2 - \frac{2}{4-3æ}(z/l)^3;$$

z = to be measured from the footing upwards, in m;
$\overline{\omega_0}$ = average frequency of surface waves;

$\Phi = g/H_1\omega_0^2$

H_1 = distance, in m, from leg footing to calm water level.

3.1.4.6 The stressed condition of structures of a self-elevating MODU shall be assessed for the most unfavourable course angles denoted later as design angles. The design course angles φ_d shall be assessed on the basis of the following formulae:
for four-legged self-elevating MODU —

$$\varphi_d = \frac{\pi}{4}(2i - 1), i = 1, 2, 3, 4; \tag{3.1.4.6-1}$$

for three-legged self-elevating MODU —

$$\varphi_d = \frac{\pi}{3}(2i - 1), i = 1, 2, 3 \qquad (3.1.4.6\text{-}2)$$

where i = direction number.

3.1.4.7 The standard values of speed components of wave loads for design course angles shall be determined on the basis of the following dependences:

for four-legged MODU —

$$\sigma_M^{sr} = 0{,}35\overline{u_{sr}}\sigma_Q^{sr}K_v l(2 - æ)\gamma_4;$$

$$\sigma_T^{sr} = 0{,}70\overline{u_{sr}}\sigma_Q^{sr}K_v\gamma_4; \qquad (3.1.4.7\text{-}1)$$

$$\sigma_N^{sr} = 0{,}70\overline{u_{sr}}\sigma_Q^{sr}K_v l/L(2 - æ)\gamma_4;$$

for three-legged MODU —

$$\sigma_M^{sr} = 0{,}5\overline{u_{sr}}\sigma_Q^{sr}K_v l(2 - æ)\gamma_3;$$

$$\sigma_T^{sr} = \overline{u_{sr}}\sigma_Q^{sr}K_v\gamma_3; \qquad (3.1.4.7\text{-}2)$$

$$\sigma_N^{sr} = 1{,}7\overline{u_{sr}}\sigma_Q^{sr}K_v l/L(2 - æ)\gamma_3;$$

where $\sigma_Q^{sr} = 1{,}34 \cdot 10^{-2} m_k C_{sr}^B \mu_{sr}\gamma d_B h_3^2$

m_k = number of vertical members;

K_v = amplification factor of wave loads, to be determined from Fig. 3.1.4.3;

γ_3 and γ_4 = factors accounting for the effects of leg spacing upon wave loads;

$$\gamma_4 = \frac{1}{\sqrt{2}}\sqrt{1 + \cos(\varpi^2 L_4/g)};$$

$$\gamma_3 = \frac{1}{\sqrt{2}}\sqrt{1 + \cos(\varpi^2 L_3/g)};$$

$$L_4 = \sqrt{2}L;$$

$$L_3 = \left(\sqrt{3}/3\right)L.$$

3.1.4.8 The static characteristics of internal forces generated in leg structures, which are in accordance with the inertia component of wave loads for the course angles mentioned under 3.1.4.6, shall be determined by Formulae (3.1.4.7-1) and (3.1.4.7-2), with substituting $\overline{u_{sr}}$ for $\overline{u_{in}}$, σ_Q^{sr} for

$$\sigma_Q^{in} = 18{,}7 \cdot 10^{-2} m_k C_{in}^B \mu_{in} S h_3 \qquad (3.1.4.8)$$

where S = sectional area, in m², of vertical member contour.

3.1.4.9 The distribution parameters of the static internal forces a_{v_0} and k_0 due to wave effects in each stationary mode shall be determined on the basis of curves to be found in Fig. 3.1.4.9, *a* and *b* proceeding from the value of the relationship

$$\frac{\sigma_{in}}{\sigma_{sr}} = \frac{6{,}2\sqrt{S}\mu_{in}C^B_{in}\bar{u}_{in}}{h_3\mu_{sr}C^B_{sr}\bar{u}_{sr}} \qquad (3.1.4.9\text{-}1)$$

The parameter of $\sigma_{Q_{sr}}$ in Fig. 3.1.4.9 shall be determined on the basis of Fig. 3.1.4.13-1.

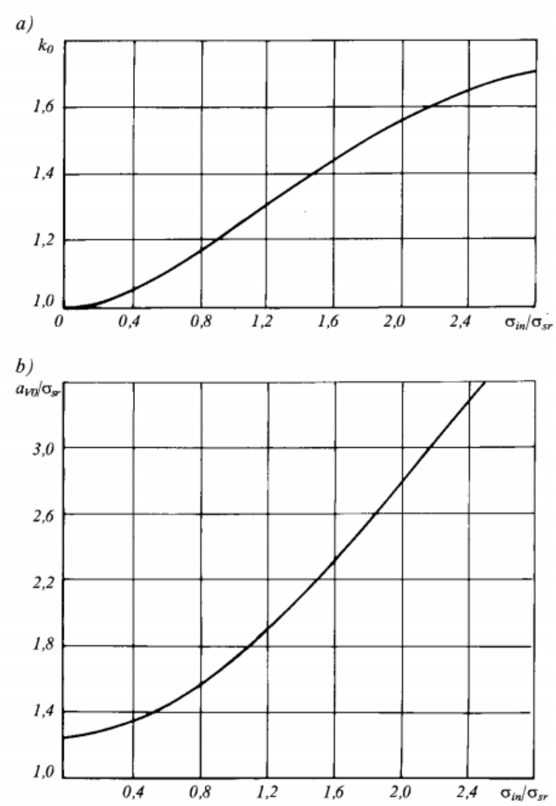

Fig. 3.1.4.9
Dependence of Weibull distribution parameters k_0 and a_{v_0} on the relationship σ_{in}/σ_{sr}

Static values of a_{v_0} and k_0 shall be specified on the basis of dynamic effects by the formulae:

$$k = k_0 + \frac{(2-k_0)v^4}{1+(1-v)^4}; \qquad (3.1.4.9\text{-}2)$$

$$a_v = a_{v_0} 5^{1/k_0 - 1/k} \qquad (3.1.4.9\text{-}3)$$

where $v = \bar{\omega}/p$

3.1.4.10 For each stationary mode, the extreme values of alternating internal forces shall be determined in the design leg cross section under the combined effects of waves and wind:

$$M_{e_{ij}} = a_{v_{M_{ij}}} \left[\ln n_{ij}(1+\beta_{ij})\right]^{1/K_{ij}}; \qquad (3.1.4.10\text{-}1)$$

$$T_{e_{ij}} = a_{v_{T_{ij}}} \left[\ln n_{ij} \left(1 + \beta_{ij}\right)\right]^{1/K_{ij}}; \qquad (3.1.4.10\text{-}2)$$

$$N_{e_{ij}} = a_{v_{N_{ij}}} \left[\ln n_{ij} \left(1 + \beta_{ij}\right)\right]^{1/K_{ij}} \qquad (3.1.4.10\text{-}3)$$

where $a_{v_{M_{ij}}}, a_{v_{T_{ij}}}, a_{v_{N_{ij}}}$ = distribution parameters of wave bending moments, shearing and axial forces, respectively;

$\beta_{ij} = a_{w_{ij}}/a_{v_{ij}}$;

$n_{ij} = 10^8 p h_3 \bar{\tau}$ = volume of sample corresponding to a stationary mode occurring at regular intervals;

$n_{ij} = 10^6 p h_3 \bar{\tau}$ = volume of sample for the operating mode.

3.1.4.11 For each stationary mode, the values of internal forces shall be determined with due regard for the static effects of wind, pontoon weight and tide forces which shall be added to the values obtained by 3.1.4.10, namely:

$$M_{\Sigma} = M_e + M_p + \bar{M}_w + M_c; \qquad (3.1.4.11\text{-}1)$$

$$T_{\Sigma} = T_e + \bar{T}_w + T_c; \qquad (3.1.4.11\text{-}2)$$

$$N_{\Sigma} = N_e + N_p + \bar{N}_w + N_c \qquad (3.1.4.11\text{-}3)$$

where M_p and N_p = bending moments and axial forces due to pontoon weight, respectively;
for M_e, T_e, N_e refer to 3.1.4.10;
for $\bar{M}_w$, $\bar{T}_w$, $\bar{N}_w$ refer to 3.1.3.4;
for M_c, T_c, N_c, refer to 3.1.5.

3.1.4.12 The greatest value obtained by 3.1.4.11 will be considered the design value.

3.1.4.13 In shallow water, the standard deviation of the inertia component of wave load σ_Q^{in} per leg will be determined by the formula

$$\sigma_Q^{in} = 18{,}7 \cdot 10^{-2} m_k C_{in}^B \mu_{in} S h_3 \cdot th \bar{k} H \qquad (3.1.4.13)$$

where $\bar{k} = 2\pi/\bar{\lambda}$;

the standard deviation of the velocity component of wave load σ_Q^{sr} per leg will be determined from the curve in Fig. 3.1.4.13-1.

The applicates of the resultants Q_{sr} and Q_{in} (counting from water level) will be determined from Figs. 3.1.4.13-2 and 3.1.4.13-3, respectively.

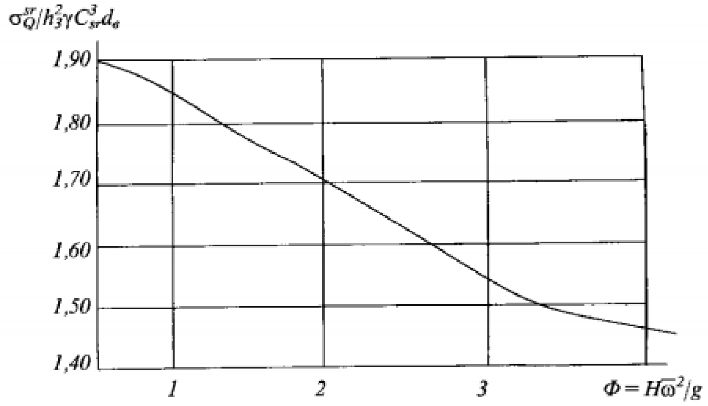

Fig. 3.1.4.13-1
Relationship between $\sigma_Q^{sr}/h_3^2\gamma C_{sr}^3 d_B$ and parameter $\Phi = H\overline{\omega}^2/g$

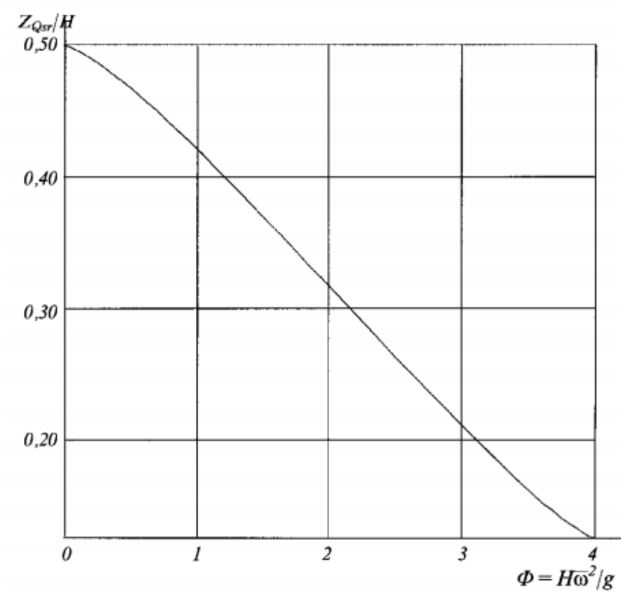

Fig. 3.1.4.13-2
Relationship between $Z_{Q_{sr}}/H$ and parameter $\Phi = H\overline{\omega}^2/g$

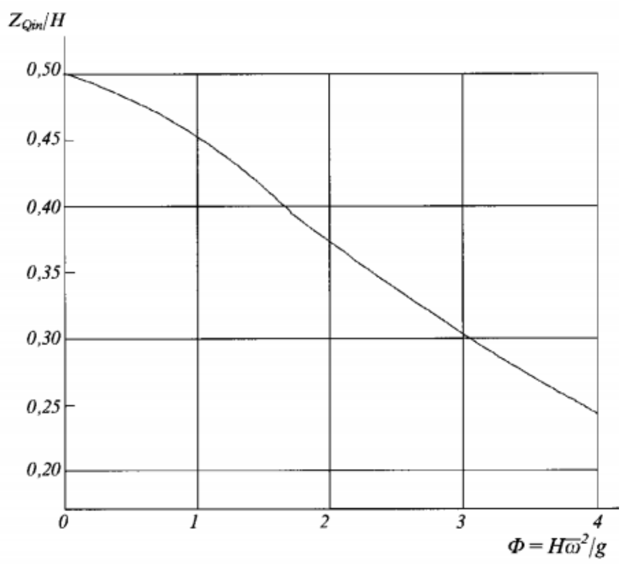

Fig.3.1.4.13-3
Relationship between $Z_{Q_{in}}/H$ and parameter $\Phi = H\bar{\omega}^2/g$

3.1.5 Current loads.

3.1.5.1 At an optional leg cross section, the internal forces generated by the continuous component of current will be equal to the following:

for a three-legged self-elevating MODU ($\varphi = 60°$) —

$$M_c = (Q_c\bar{u}_c l/2)\left(2\frac{z}{l} - æ\right);$$

$$N_c = \left(2\sqrt{3}Q_c\bar{u}_c l/L\right)(2 - æ); \qquad (3.1.5.1\text{-}1)$$

$$T_c = Q_c\bar{u}_c;$$

for a four-legged self-elevating MODU ($\varphi = 0°$) —

$$M_c = \left(\sqrt{2}Q_c\bar{u}_c l/4\right)\left(2\frac{z}{l} - æ\right);$$

$$N_c = (2Q_c\bar{u}_c l/L)(2 - æ); \qquad (3.1.5.1\text{-}2)$$

$$T_c = Q_c\bar{u}_c$$

where $Q_c = \rho C_{sr} d H_0 v_c^2/2$;
H_0 = water depth, in m;
v_c = current velocity, in m/s;
$\bar{u}_c$ = value of the и parameter (refer to 3.1.4.5) in section $H_1/2$.

3.1.5.2 Under the combined effect of wind and tide, an approximation is possible, as follows:

$$Q_{\Sigma} = Q_{sr} + 2\sqrt{Q_{sr}Q_c} + Q_c \quad (3.1.5.2)$$

where Q_{sr} = speed component of wave loads, determined in the following way

$Q_{sr} = a_v(\ln n)^{1/K}$

where a_v and K = scale parameters and Weibull forms of distribution determined in accordance with diagrams to be found in Fig. 3.1.4.9.

3.1.6 Seismic loads.

3.1.6.1 In some areas, the seismic loads on a self- elevating MODU can be comparable to wave loads.

The integral seismic effect on a self-elevating MODU shall be determined by the formula

$$Q = M_{po}\beta_{\Sigma}a_{max} \quad (3.1.6.1)$$

where M_{po} = reduced mass of a pontoon;
a_{max} = maximum value of acceleration amplitude;
β_{Σ} = generalized dynamic coefficient as adopted from Fig. 3.1.6.1.

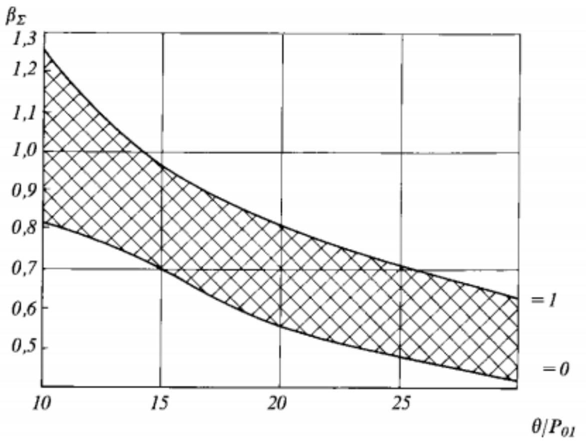

Fig. 3.1.6.1
Generalized dynamic coefficient:
θ — bearing frequency of axilogram;
P_{01} — lowest frequency of horizontal vibrations

3.1.6.2 The ultimate load Q_{ult}^{Σ}, which a self-elevating MODU can withstand is determined by the formula

$$Q_{ult}^{\Sigma} = nQ_{ult}^{modulus}\left(1 + \frac{l}{d} \cdot \frac{2-æ}{2}\right) \quad (3.1.6.2)$$

where n = leg number;
$Q_{ult}^{modulus}$ = ultimate load upon leg modulus, to be determined on the basis of considering the kinematic condition of the modulus. For a typical truss of a self-elevating MODU with K-type connections;

$Q_{ult}^{modulus} = 2{,}32 R_d \pi D_p t_p$

where R_d = design yield strength, in MPa, of diagonal bracings;
D_p and t_p = diameter and thickness, in m, respectively, of diagonal bracings;

l = leg length, in m;
d = distance, in m, between the upper and lower shaller;
æ = supporting pair coefficient (refer to 3.1.2.2).

3.1.6.3 Safety factor for seismic loads:
in case of earthquakes occurring once in 100 years —

$$K_a = Q_{ult}^{\Sigma}/Q \geq 1{,}25; \quad (3.1.6.3\text{-}1)$$

in case of earthquakes occurring once in 500 years —

$$K_a = Q_{ult}^{\Sigma}/Q \geq 1. \quad (3.1.6.3\text{-}2)$$

3.1.7 Pre-loading and pulling out of the legs.

3.1.7.1 Considerable forces may be generated in the structure of a self-elevating MODU during pre-loading and pulling out of the legs. Due to this, a strength analysis of the legs and pontoon shall be carried out.

The analysis shall be aimed at the following:

determining the permissible levels of controlled parameters (heel and trim angles) proceeding from the conditions of leg strength;

testing the strength of pontoon structures when resting on diagonal supports in the case of a four-legged self-elevating MODU, or on the basis of the deadweight with due regard for the ballast taken in the pontoon compartment in the case of a three- legged self-elevating MODU.

3.1.7.2 The permissible values of heel and trim angles shall be determined on the assumption that, when pulling out the legs, the actual stresses in the most stressed points of the structure will not exceed the permissible values.

When drawing up the strength provisions, the most dangerous locations of the points of the pontoon reactions application on the leg modulus height, and the most stressed areas of the structure shall be considered. Among these are the supporting sections of horizontal struts and bracings, as well as the midspans of vertical struts of legs.

3.1.7.3 The forces by which the interaction of leg and pontoon is manifested will be determined proceeding from the condition of ensuring concurrent movement of the pontoon and legs, and shall be characterized by heel and trim angles, as well as by the vertical axial force.

When determining unknown forces, gravity forces and the coordinates of the application point of the gravity force resultant shall be considered, as well as the floatability forces and the under-water hull shape, forces and moments generated in the supporting section of the leg as a result of interacting with the seabed.

3.1.7.4 The range of permissible values of heel and trim angles, and of the axial force will be determined proceeding from the strength conditions. These characteristics shall be considered the basic data for drawing up the service manual.

3.1.7.5 Where the axial force value is specified, the permissible values of heel and trim angles shall be determined by the procedure described in 3.1.8.3 assuming buoyancy forces to be zero and the force vector to be opposite to that of the forces generated during the pulling out.

3.1.8 Stressed condition of legs.

3.1.8.1 The leg strength analysis is aimed at determining the stressed condition of the structure where the legs are attached to the hull of the self-elevating MODU and where they are embedded in the seabed.

The leg structure shall be idealized as a space frame system.

The stressed condition of leg structures shall be analyzed by methods which would make it possible to consider the peculiarities of the stressed condition of leg components, for instance, by the finite element analysis. In this case, the structure of pontoons and holes for legs can be considered to be absolutely rigid.

3.1.8.2 As the leg area by which the former is attached to the hull, a leg section shall be considered which is limited from below by a cross section lying below the lower shaller at midlength between the upper and lower shaller, and from above it is limited by a cross section removed by $0{,}25d$ upwards from the uppermost point of contact with the upper shaller, or by the end cross section of the leg (refer to Fig. 3.1.2.1).

3.1.8.3 As the dynamic utmost conditions for the lowest cross section of the leg area considered, the integral forces and moments shall be adopted which are determined on the simplified model of a self- elevating MODU (refer to 3.1.2.1).

When setting up restrictions for the vertical movements within the lower section of the leg area, to be included in the dynamic utmost conditions are vertical forces transmitted to the leg by the jacking system. The value and application pattern of these forces will depend on the design of the jacking system, type of its attachment to the hull of the self-elevating MODU and the possibility of non- uniform distribution of these forces being prescribed to ensure the tooth strength conditions.

3.1.8.4 As a result of analysing an idealized leg construction attached to the pontoon hull, the values characterizing the movements and internal forces in the assemblies of the bar system will be determined, as well as those of response and stress distribution in the structural components.

The stress analysis for the components of the most stressed moduli shall be effected through the height of three cross sections at midlength and at supporting sections.

3.1.8.5 As the embedded leg area, the leg section shall be considered whose upper side is limited by a cross section lying within $0{,}5d$ from the connection line of leg and footing.

For modelling the leg footing or another similar structure forming part of the lower end of the leg, an idealization shall be used to generate a plane stress in the components of the structure. Dynamic utmost conditions shall be prescribed for the upper end of the considered leg section in the same way as under 3.1.2.1. As a result of the analysis, the values of internal forces and movements shall be obtained and the stress distribution established.

3.1.8.6 Where the provisions of 3.1.8.2 — 3.1.8.5 cannot be complied with in view of principal differences from the above calculation procedure, a calculation procedure taking into account the structural peculiarities of the self-elevating MODU may be submitted to the Register for homologation.

3.1.9 Loads on self-elevating MODU legs in transit

Where in transit the legs are mostly completely upturned. Combination of inertia loads during rolling or pitching together with wind load provokes extensive bending moments and axial forces in the legs as well as considerable jet forces in the portal and hull structures.

Rolling parameters may be obtained either based upon the results of model tests or calculation. One shall regard the results of rolling parameters calculation very carefully due to the non-conventional peculiarities of pontoons of self-elevating MODU that cause a number of non-linearities.

For the calculation in transit conditions the following shall be taken into consideration:

inertia forces corresponding to the specified amplitude of rolling or pitching with natural period of platform;

static forces corresponding to the maximum inclination of legs during rolling or pitching;

specified wind forces.

Effect of rolling, drifting or yawing shall be considered by means of introduction of the correction coefficient $\gamma = 1{,}2$.

Rolling or pitching is assumed to be calculated with the aid of the relation

$$\theta = \theta_0 \sin\frac{2\pi t}{T_0} \qquad (3.1.9\text{-}1)$$

where t = time, in s;
T_0 = natural period of rolling or pitching;
θ_0 = amplitude of rolling or pitching, in deg.

It is considered that the oscillation center is located within the water line level.

Acceleration of lumped masses at the distance r, in m, from the oscillation center, in m/s², is determined as follows:

$$a = -(2\pi/T_0)^2\theta_0 r \sin\frac{2\pi t}{T_0}. \qquad (3.1.9\text{-}2)$$

Amplitude values of the forces per the leg unit length are determined by z coordinate:

transverse forces —

$F_{TS} = m(z)g\sin\theta_0$ — static force;

$F_{TD} = m(z)\varepsilon_0 d$ — inertia force; (3.1.9-3)

$F_W = \frac{1}{2}\rho_W C_D[W(z)\cos\theta_0]^2$ — wind force;

longitudinal forces —

$F_{LS} = m(z)g\cos\theta_0$— static force;

$F_{LD} = m(z)\varepsilon_0 d$ — inertial force (3.1.9-4)

where $m(z)$ = unit mass;
$W(z)$ = wind velocity at z level;
g = acceleration of gravity force;

$\varepsilon_0 = 2\pi/T_0$

When reducing the forces to resultant values the intrinsic moment of inertia of the structure in question shall be taken into consideration, e.g., when reducing the leg section with length l and mass per unit length m, the intrinsic moment of inertia M_j is equal to the following:

$$M_j = \frac{ml^3}{12}.$$

Natural period of rolling or pitching may be determined by the formula

$$T_0 = 2p\sqrt{(r_0^2 - a_0^2)/gGM} \tag{3.1.9-5}$$

where r_0 = radius of inertia for rolling or pitching in relation to the axis located in the water line level, in m;
a_0 = vertical distance between the water surface and true rotation axis during rolling or pitching, in m,
GM = transverse or longitudinal metacentric height, in m.

The distance a_0 for preliminary analysis may be accepted between the water surface and centre of gravity.

The radius of inertia r_0 may be determined as follows:

$$r_0 = \sqrt{I_m/M_m} \tag{3.1.9-6}$$

where $I_m = I_L + I_H + I_A$ – moment of inertia of the masses with regard to relation of rolling and pitching;

$M_m = nM_L + M_H$ – mass;

n = the number of legs;
I_L = moment of inertia of the leg masses;
I_H = moment of inertia of the hull mass;
I_A = added mass of the moment of inertia;
M_L = mass of the leg;
M_H = mass of the hull.

3.1.10 Leg pounding against seabed during self- elevating MODU positioning at a site.

During preloading and pulling out the leg may be subjected to pounding against seabed, caused by the unit rolling.

Pounding force caused by rolling may be determined by the simplified method based on the following:

only one leg touches seabed;

the lower end of the leg comes to a stop immediately upon touching seabed;

seabed is extremely hard.

The unit rotation energy is absorbed by the leg structure that gives the pounding force P:

$$P = \frac{2\pi\theta_0}{T_0}\sqrt{KI_m} \tag{3.1.10-1}$$

where I_m = moment of inertia of the unit mass in relation to rolling or pitching;
θ_0 = amplitude of rolling;
K = total transverse stiffness of the leg.

The result will depend on wave condition intensity and water area depth.

The maximum permissible pounding force may be determined on the basis of strength criterion. The maximum permissible amplitude of rolling and pitching during preloading and pulling out shall be as follows:

$$[\theta_0] = TP_{max}/2\pi\sqrt{KI_m}. \tag{3.1.10-2}$$

3.1.11 Ice strength of self-elevating MODU legs.

Generally, self-elevating MODU are not designed for ice operation. Nevertheless, prolongation of the period of self-elevating MODU operation in ice inclusive leads to necessity of safety assuring in view of prolongation of drilling time.

For solving this task it is necessary to identify the intensity of interaction of the ice field with the mass m_i moving with the speed v_i, and self-elevating MODU structures. At this, one shall consider flexibility of the self-elevating MODU leg.

As the load from moving ice fields the least of the two following shall be taken: that of the field stop or breaking the ice.

3.1.12 Vortical loads.

Vortical loads are determined for "permeable" structures in relation to critical wind velocities determined from the following formula:

in the direction normal to the wind flow

$$(w_{cr})_y = \frac{d}{(T)_y Sh} \qquad (3.1.12\text{-}1)$$

where $(T)_y$ = natural period of vibrations of the structure in question;
Sh = Strouhal number;
d = cross-sectional dimensions of a structure;

in the direction along the wind flow

$$(w_{cr})_x = \frac{d}{(T)_x Sh}. \qquad (3.1.12\text{-}2)$$

The value of Strouhal number is determined by the sectional shape of a structure, angle of inflow and dimensionless Reynolds number:

$$Re = \frac{d\bar{w}_{10}}{v}$$

where v = air kinematic viscosity coefficient.

The relationship $Sh = Sh(\alpha)$ for a particular nonstandard structure shall be determined by model experiments. Where the experimental data are not available, expert appraisals are recommended. In the first approximation the following formula may be used:

$$C_x Sh = 0{,}26(1 - e^{-2{,}38C_x}) \qquad (3.1.12\text{-}3)$$

where C_x = head drag coefficient of the structure in question.

3.2 SEMI-SUBMERSIBLE MODU

3.2.1 General.

3.2.1.1 In accordance with 2.4, the structural integrity of a semi-submersible MODU shall be tested for three different modes:

survival;
operation;
transit;
according to 2.4.

General recommendations concerning loads to be assigned during each of the above modes shall be found under 2.3.

3.2.1.2 A unit in the survival mode shall have a clearance h_c, in m, determined as

$$h_c < 0{,}6h_{50} + 0{,}5 \qquad (3.2.1.2)$$

where h_{50} = maximum wave height, in m, for the particular area of operation in question (once in 50 years).

3.2.1.3 For assessing by the fatigue strength criterion, the whole of the long-term distribution spectrum $P(h_3, T_c)$ for the area in question or an area with the severest wave conditions shall be used, as well as the service life of the unit as a whole.

3.2.1.4 The wave load represents a system of mutually balanced hydrodynamic loads on the surface of the unit and of three-dimensional inertia loads due to the proper weight of the unit, which are generated by the unit's rolling in waves.

For determining the loads, the linear theory of rolling in waves may be applied.

3.2.1.5 When making hull strength calculations for a semi-submersible MODU, one shall be guided by the provisions of 2.5 and by the instructions below.

3.2.1.6 Any damage to a primary hull member or bracing shall not involve a collapse of the hull of the unit. The Register may require for calculations to be submitted to confirm that the hull strength will be ensured with a primary hull member or bracing damaged under external loads corresponding to the maximum loads during a year for the area of operation in question.

3.2.1.7 Watertight submersible or semi-submersible hull structural elements (compartments) shall be equipped with the watertightness break detection facilities.

3.2.2 Global loads.

3.2.2.1 The global loads upon the hull of a unit in waves may be determined by means of a calculation procedure approved by the Register and taking the rolling of the unit and the random character of waves into consideration, or by an experimental procedure based on special model tests which ensure the dependability of results and their adequacy as compared to those of full-scale testing.

3.2.2.2 Global wave loads may be represented as distributed loads or design values of integral characteristics of load components with indication of calculation methods for the relevant distributed loads and design load combinations to determine the global stresses for each design mode of operation.

The distributed loads which are generally determined in respect of an idealization of the hull of the self-elevating MODU used for rolling calculations and to determine the deflected mode of hull structures shall be transformed bearing in mind the adopted idealization and the realization of the finite element method applied.

3.2.2.3 As the integral characteristics, the four components of the wave load are generally considered: Q_1 the symmetrical component; Q_2 the oblique-symmetrical component; Q_3 the torque acting in the centre plane of the unit; and Q_4 the shearing force applied to one fourth of the unit length in the long-term distribution of wave modes or their dispersion in a stationary mode.[1]

[1] These components are present in a semi-submersible MODU of classical type whose structure includes two pontoons, 4 — 8 stability columns, upper hull and, generally, bracings.

3.2.2.3.1 The integral characteristic of the symmetrical horizontal component of load:

$$Q_1^l = \frac{1}{2}\int_L \left(q_y^l - q_y^r\right) dx = -Q_1^r. \qquad (3.2.2.3.1\text{-}1)$$

The relevant horizontal distributed load, in t/m,

$$q_1^l = Q_1^l/L, \text{ and } q_1^r = Q_1^r/L = -Q_1^l/L \qquad (3.2.2.3.1\text{-}2)$$

is applied in the waterline plane when in the transit mode and in the pontoon deck plane when in the operating and survival modes.

3.2.2.3.2 The integral characteristic of the asymmetrical component

$$Q_2^l = \frac{1}{2}\int_L \left(q_z^l - q_z^r\right) dx = -Q_2^r. \qquad (3.2.2.3.2\text{-}1)$$

The relevant distributed load is represented in each pontoon section by the distributed vertical force q, in t/m, and moment M, in t:

$$q_2^l = Q_2^l/L, \text{ and } M_2^l = \left(Q_2^l/L\right)(b_0 + B_1) \qquad (3.2.2.3.2\text{-}2)$$

and

$$q_2^r = -Q_2^l/L, \text{ and } M_2^r = M_2^l \qquad (3.2.2.3.2\text{-}3)$$

with q_2 applied in the centre plane of the pontoons and with M_2 acting with regard to the crossing line of the pontoon centre plane and the waterline when in the transit mode, and of the centre plane and the pontoon deck plane when in the operating and survival modes.

3.2.2.3.3 The integral characteristic of the torsional component (in the centre plane of the unit):

$$Q_3^l = \frac{1}{2}\int_L x\left(q_z^l - q_z^r\right) dx = -Q_3^r. \qquad (3.2.2.3.3\text{-}1)$$

The relevant vertical distributed forces, in t/m:

$$q_3^l = \frac{12Q_3^l}{L^3}x \text{ and } q_3^r = \frac{12Q_3^r}{L^3}x \qquad (3.2.2.3.3\text{-}2)$$

are applied in the centre plane of the pontoons.

3.2.2.3.4 The integral characteristic of the symmetrical vertical component

$$Q_4^l = \frac{1}{2}\int_{L/4}\left(q_z^l - q_z^r\right) dx = Q_4^r. \qquad (3.2.2.3.4\text{-}1)$$

The relevant vertical distributed forces, in t/m:

$$q_4^l = q_4^r = \left(2\pi Q_4^l/L\right)\cos(2\pi x/L) \qquad (3.2.2.3.4\text{-}2)$$

are applied in the centre plane of the pontoons.

The relationships under 3.2.2.3.1 — 3.2.2.3.4 include the following parameters:

q_z^l, q_y^l, M^l and q_z^r, q_y^r, M^r = distributed vertical and horizontal components of force and the moment for the left and right pontoon, respectively, each of which represents a sum of disturbing, restoring, hydrodynamic and inertia masses, as well as proper weights, of forces and moments whose major vector and moment are zero;
L, B_1 = length and breadth of pontoon hull, respectively;
b_0 = distance between inner sides of pontoons.

3.2.2.4 Stresses determined on the basis of design values of the integral characteristics of load components have the same probability of exceedance as those characteristics. To determine global design stresses, load component compositions shall be used, as given in Table 3.2.2.4.

Table 3.2.2.4

Operation mode of semi-submersible MODU	Design wave height	Position of unit with regard to the waves	Design load composition
Transit conditions	$h_{3\%} = h_{per}$[1] $T_c = \overline{T_c}$[2]	Straight course ($\varphi = 0$ or to 180°)	Q_4; $0{,}3Q_1$; $0{,}3Q_2$ and $0{,}3Q_3$
		Oblique course	Q_3; $0{,}3Q_1$; $0{,}3Q_2$ and $0{,}3Q_4$
		Course beam to the sea	Q_1; Q_2; $0{,}3Q_2$ and $0{,}3Q_4$
Operating conditions and survival	$h_{\min}^{3} \le h_{3\%} \le h_{per}$ Where the range of $h_{3\%}$ is in accordance with the values prescribed for the mode of operation, T_c shall comply with the long-term distribution	Straight course ($\varphi = 0$ or to 180°)	Q_4; $0{,}3Q_1$; $0{,}3Q_2$ and $0{,}3Q_3$
		Oblique course	Q_3; $0{,}3Q_1$; $0{,}3Q_2$ and $0{,}3Q_4$
		Course beam to the sea	Q_1; Q_2; $0{,}3Q_2$ and $0{,}3Q_4$

[1] h_{per} = permissible wave height with 3 % probability of exceeding level for the particular mode of operation.
[2] T_c = mean value of average period of stationary sea conditions with $h_{3\%} = h_{per}$.
[3] $h_{\min}$ = minimal wave height with 3 % probability of exceeding level for survival (for operating conditions h_{min} = 0).

For stresses determined on the basis of distributed loads, long-term distribution or stress dispersions on stationary seas shall be determined to be able to determine the magnitude of stresses with the same probability of exceedance as prescribed under 2.3 for wave conditions during the design modes of a semi-submersible MODU operation.

3.2.2.6 Hydrodynamic loads upon bracings shall be determined in accordance with 2.3.9.

The design values of the added mass coefficient C_m shall be determined by the procedure approved by the Register. The design values of the resistance coefficient C_{sr} shall not be less than indicated in Fig. 3.1.4.1.

3.2.2.7 The velocity component of the load may be ignored for calculation purposes, provided:

$$h_{3\%} \le \pi D C_{in} / C_{sr} \tag{3.2.2.7}$$

where $h_{3\%}$ = wave height with 3 % probability of exceeding level;

$C_{in} = 1 + C_m$.

3.2.2.8 When using semi-submersible MODU in ice-covered water areas, their interaction with different ice formations typical for specific time period in this sea area shall be considered. The components of global ice loads shall be calculated by the methods specified in 3.3.2 with regard to the fixed platforms.

3.2.3 Local loads.

3.2.3.1 Local loads account for the intensity of the transverse load upon the shell plating, stiffeners, deck of pontoons, stability columns and upper hull.

3.2.3.2 The total local load includes a permanent and a variable component.

3.2.3.3 The permanent load is determined as the difference between the external (with regard to the compartment) and the internal pressure.

As the design value, the most unfavourable value of this difference shall be considered. Where the internal pressure is generated by a consumable cargo or ballast, it shall be adopted zero when determining the design local load.

3.2.3.4 Variable local pressures, in kPa, shall be determined by the following formulae:

for submerged section of structure —

$$p = gp\frac{h_{3\%}}{2}ce^{-kz}; \qquad (3.2.3.4\text{-}1)$$

for above-water section of structure —

$$p = gp\left(\frac{h_{3\%}}{2}c - z_1\right) \text{ but not less than 5 kPa} \qquad (3.2.3.4\text{-}2)$$

where $h_{3\%}$ = wave height with 3 % probability of exceeding level, in m, for the sea condition whose probability $P\ (h_{3\%}, T_c) \approx 10^{-2}$, for the long-term distribution in the area of operation in question;

c = factor accounting for wave diffraction and pressure field non-uniformity on the contour of a submerged element, c = 1,5;

$k = 4\pi^2/gT_c^2$;

T_c = average period, in s, in the sea condition of 10^{-2} probability;

z = immersion, in m, of a point of submerged section of structure under free surface of water;

z_1 = height, in m, of a point of the above-water section of structure above free surface of water;

g = gravity acceleration, in m/s²;

p = sea water density, in t/m³.

3.2.3.5 Impact loads upon bracings.

3.2.3.5.1 The design rate v_0 of relative displacement of the forward transverse horizontal bracing, provided it is immersed (the amplitude R_0 of the relative displacement $R > 2b_1$ where b_1 is the distance from the lower bracing edge to the water surface), may be described by the expression

$$v_0^2 = 2D_v(4\ln 10 - 2b_1^2/D_R) \qquad (3.2.3.5.1)$$

where D_v and D_R = rate and displacement dispersions of the relative movement of the bracing.

The dispersions D_v and D_R shall be determined with due regard for the wave motion of water, as well as the heaving and pitching of the unit, on the encounter angle with regard to the waves in the transit and survival modes (refer to 2.3.2.2 and 2.3.4.4) and by procedures agreed by the Register.

3.2.3.5.2 The design distributed impact load, in t/m, is determined as

$$q_{ym} = 1{,}47 v_0^2 \rho D \quad (3.2.3.5.2)$$

where D = bracing diameter, in m.

3.2.3.5.3 The maximum design amplitude of displacement z_0 of the middle section of the bracing in the process of elastic vibrations and the relevant stresses σ_y is determined by the formulae:

$$z_0 = \frac{F_{red}}{K_{red}} \frac{2{,}72 a_1}{(1+a_1^2)^2} \left\{ [2a_1 + (1 + a_1^2)\omega_1 t] \left(\frac{1}{2{,}72}\right) - 2a_1 \cos \omega_1 t - (1 - a_1^2) \sin \omega_1 t \right\}; \quad (3.2.3.5.3\text{-}1)$$

$$\sigma_y = (ED/2) z_0 f_1''(y) \quad (3.2.3.5.3\text{-}2)$$

where $a_1 = v_0 / 0{,}145 D \omega_1$;
$\omega_1 = K_{red}/M_{red}$ = basic frequency of elastic vibrations of the bracing;
$F_{red} = \int_0^l q_{ym} f_1 dy + F_{l/2} f_1$ = reduced force;
$K_{red} = \int_0^l EJ \left(f_1''\right)^2 dy + K_{l/2} f_1$ = reduced rigidity;
$M_{red} = \int_0^l m f_1^2 dy + m_{l/2} f_1^2$ = reduced mass;
f_1 = basic vibration mode to be determined by the formula:

$$f_1 = 0{,}5\left(\cos\frac{2\pi y}{l} - 1\right) + \left(1 - \sin\frac{\pi y}{l}\right), \quad (3.2.3.5.3\text{-}3)$$

or the shape of girder bending shall be determined with due regard for the rigidity of embedding, for stiffener fitted in the span and having a rigidity $K_{l/2}$ and for other peculiarities of the girder revealed under the effect of a uniformly distributed load involving a single deflection at the point of its reduction.

3.2.3.5.4 Stresses σ_y shall be considered when determining the total stresses in the bracing due to local and total loads in the transit and survival condition.

3.2.3.6 When using semi-submersible MODU in ice-covered water areas, local ice loads determined similarly as for the fixed structures (refer to 3.3.2.3.23 — 3.3.2.3.24) shall be considered.

3.2.3.7 Dynamic aspects of ridged ice loads on MODU shall be additionally studied when non-dimensional parameter which determines the relative proximity of ice load frequency to natural frequency of the flexible structure is given by the following formula:

$$\chi = \frac{0{,}4TV}{w_{ridge} + X_{st}} \quad (3.2.3.7)$$

where T = natural period of horizontal vibrations for MODU in static equilibrium;
w_{ridge} = ridge width;
X_{st} = structure shift due to the maximum load equal to ridge load on the fixed structure;
V = speed of ice drift.

3.2.3.8 As dynamic effects are significantly dependent on the ice formation parameters, MODU geometric features, performance of its position-keeping system and ice speeds, dynamic aspects of ice loads on MODU shall be accessed, where reasonable, on the basis of mathematical modeling of ice formations and moored structure interaction using the software approved by the Register.

3.2.4 Determination of deflected mode.

3.2.4.1 A platform construction is considered a linear system. Therefore, stresses in the structure may be generated as a result of superposition of the effects of particular load components.

3.2.4.2 For analysing the deflected mode of the construction of a semi-submersible MODU as a whole, the method of finite elements is recommended. To this end, the following three-dimensional models may be applied: beam model, plate model and plate-beam model.

The application of the beam model is advisable at initial design stages. It is also convenient where there are many bracings fitted at random.

The application of the plate and the plate-beam models is advisable at the final stages of design. The latter model implies an idealization in the form of finite beam elements or bracings only, or bracings, stability columns and pontoons.

3.2.4.3 When analysing the deflected mode of the construction of a semi-submersible MODU as a whole on the basis of the beam model, the following shall be done:

.1 finite beam elements with six degrees of freedom in the connection shall be used which would account for bending and shifting deformations in two planes, tension-compression and twisting deformations;

.2 geometrical characteristics of cross sections of the elements by which pontoons, stability columns and bracings are approximated shall be determined proceeding from the condition that the longitudinal members including shell plating, longitudinal stiffeners and other longitudinal elements will contribute with their full area to the construction behaviour;

.3 geometrical characteristics of cross sections of the elements by which the structures of the upper hull are approximated shall be determined in accordance with the provisions of 3.2.4.4;

.4 where elements of large cross sections are connected (stability column to a pontoon or bracing, for instance, (refer to Fig. 3.2.4.3.4), etc. either "absolutely rigid" finite elements shall be introduced or finite beam elements with rigid ends shall be used.

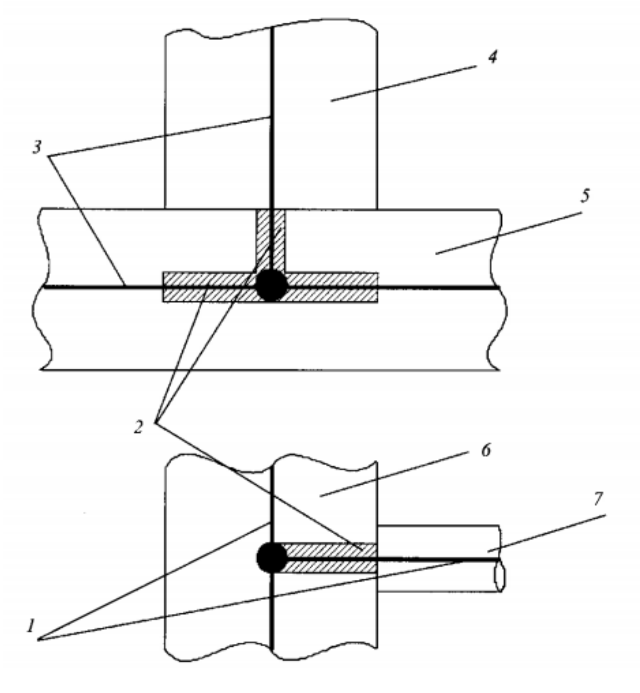

Fig. 3.2.4.3.4
"Absolutely rigid" finite elements:
1,3 — bar finite elements; *2* — "absolutely rigid" finite elements; *4, 6* — stability column;
5 — pontoon; 7 — bracing

3.2.4.4 Provisions for determining the geometrical characteristics of cross sections of elements by which the upper hull structure is approximated stipulate the following.

3.2.4.4.1 The upper hull structure (refer to Fig. 3.2.4.4.1-1) may be represented as a system of beam elements (refer to Fig. 3.2.4.4.1-2) possessing the properties of a real structure. The geometrical characteristics of cross sections of the beam elements are as follows:

J_x = inertia moment of the cross-sectional area of an element with regard to the horizontal axis;
J_z = inertia moment of the cross-sectional area of an element with regard to the vertical axis;
J_T = torsional inertia moment of the cross-sectional area of an element;
F_x = cross-sectional area of an element taking up a shift in the horizontal direction;
F_z = cross-sectional area of an element taking up a shift in the vertical direction;
F_{t-com} = cross-sectional area of an element in tension and compression.

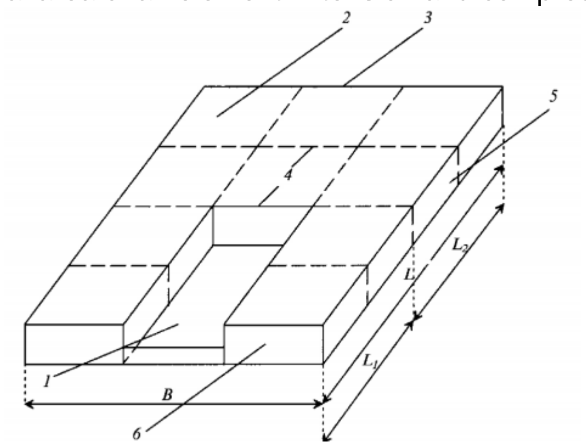

Fig. 3.2.4.4.1-1
Plan of upper hull structure (example);
B — upper hull breadth; L — upper hull length; L_1, L_2 lengthwise span between stability column axes;
1,2 — deck; *3,6* — transom; *4* — bulkheads; 5 — side

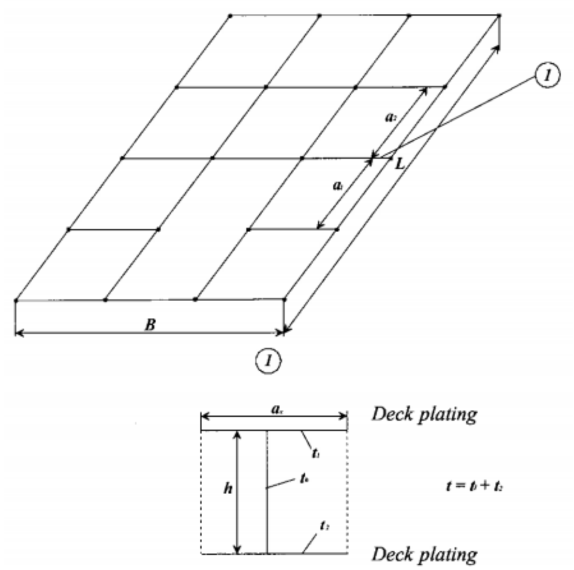

Fig. 3.2.4.4.1-2
Beam model of upper hull (example):
1 — bar element considered and its cross section; a_x — effective flange breadth; a_1 and a_2 — distance from the element in question to the nearest element arranged in the same direction;
B — upper hull breadth; L — upper hull length

3.2.4.4.2 The inertia moment J_x is determined by the web height and the effective flange breadth a_x which depends on the span length of the element b and the type of structural deformation. All the longitudinal members adjoining the element shall be considered.

For transverse elements, the span length b shall be adopted equal to the upper hull breadth where intermediate supports in the form of bracings are missing, or to half the upper hull breadth where intermediate supports in the form of bracings are fitted. For longitudinal elements, the span length b shall be adopted equal to the lengthwise distance between the stability columns axes.

The following types of structural deformation are distinguished: symmetrical bending and oblique-symmetrical bending. In case of symmetrical bending, the width of the effective flange of elements is adopted equal to the lesser of the values determined by the formulae:

$$a_x = \frac{1}{3}b; \tag{3.2.4.4.2-1}$$

$$a_x = 0{,}5(a_1 + a_2). \tag{3.2.4.4.2-2}$$

In case of oblique-symmetrical bending, the width of the effective flange of elements shall be adopted equal to the lesser of the values determined by the formulae:

$$a_x = \frac{1}{6}b; \tag{3.2.4.4.2-3}$$

$$a_x = 0{,}5(a_1 + a_2) \tag{3.2.4.4.2-4}$$

where a_1, a_2 = distances from the element considered to the nearest elements arranged in the same direction and fitted on both sides of the former elements, m.

3.2.4.4.3 The inertia moment J_z, m⁴, is determined by the formula

$$J_z = (ta_z^3/12)(1/n_z) \tag{3.2.4.4.3-1}$$

where t = total design thickness, in m, of deck plating;
a_z = effective flange thickness, in m, in the case of bending with regard to the vertical axis;
n_z = number of transverse elements on the upper hull length for the purpose of J_z determination for transverse elements, or the number of longitudinal elements on the upper hull breadth for the purpose of J_z determination for longitudinal elements.

The effective flange width a_z shall be determined by the formula

$$a_z = B\sqrt[3]{(L/2B)\frac{1}{1+v}} \tag{3.2.4.4.3-2}$$

where B = upper hull breadth, in m, for transverse elements, or upper hull length, in m, for longitudinal elements;
L = upper hull length, in m, for transverse elements, or upper hull breadth, in m, for longitudinal elements;
v = Poisson's ratio.

3.2.4.4.4 For the purpose of determining the inertia moment J_T, the upper hull shall be considered, in each of its longitudinal or transverse sections, as a closed system(s) bounded on its contour by the plating of deck and sides (transoms, bulkheads).

The inertia moment J_T, m⁴, is determined by the formula

$$J_T = (4S^2/\int dl/t_c)(1/n_T) \quad (3.2.4.4.4)$$

where S = area, in m, of a closed contour formed by the plating of decks and sides (transoms, bulkheads) of the closed system considered;
dl and t_c = element of the contour perimeter length and its web thickness, in m, at the perimeter point under consideration;
n_T = number of transverse elements forming the closed system, for the purpose of determining J_T for transverse elements, or the number of longitudinal elements forming the closed system, for the purpose of determining J_T for longitudinal elements.

The bending moment discontinuities at the points of longitudinal and transverse member intersection of the upper hull, which are due to the method of J_T specification, shall be smoothed by averaging the bending moment values.

3.2.4.4.5 Unless it is proved that the shift may be ignored ($F_x \to \infty$), the cross-sectional area F_x is determined by the formula

$$F_x = 0{,}5(a_1 + a_2)t \quad (3.2.4.4.5)$$

where for a_1, a_2 refer to 3.2.4.4.2;
for t, refer to 3.2.4.4.3.

3.2.4.4.6 Unless it is proved that the shift can be ignored ($F_z \to \infty$), the cross-sectional area F_z, m, is determined by the formula

$$F_z = ht_h \quad (3.2.4.4.6)$$

where h = height, in m, of element cross section;
t_h = design web thickness of element, in m.

3.2.4.4.7 The cross-sectional area F_{t-com} depends on the web height and the effective flange width a_{t-com}. All the longitudinal members adjoining the element shall be considered.

The effective flange width is determined by the formula

$$a_{t-com} = 0{,}5(a_1 + a_2) \quad (3.2.4.4.7)$$

where for a_1, a_2, refer to 3.2.4.4.2.

3.2.4.5 In accordance with 2.4.3.9, the loss of stability of the plates of primary and secondary structural elements may only be permitted for the deck plating of the upper hull. In this case, the reduction of elastic members (plates) under the effect of design compressive stresses shall be considered for calculation purposes.

3.3 FIXED OFFSHORE PLATFORMS

3.3.1 General.

3.3.1.1 The strength of a FOP structure shall be checked in accordance with the strength criteria specified in 2.4 for design modes specified in 2.1.1.

The criteria of 3.3.3 are additionally to be met for ice strake structures under extreme loading. In this case, the criterion 2.4.2.3.2 for the plates of outer shell of an ice strake shall be met only for local hydrostatic and wave loads (and also attendant thereto) and may be determinative merely in the event when local ice pressures are comparable in value with the other local loads.

Safety factors and strength criteria for the modes of positioning at and removal from site shall be assumed as for a transit mode. Based on these requirements, the permissible environmental conditions for positioning at and removal from site mode shall be refined.

The mode of removal from site in terms of strength assurance shall be considered for FOP, which may repeatedly change their location area during life cycle.

For FOP, which operation is expected to be at one location only during the entire life cycle, the mode of removal from site shall meet the requirements in 2.3.5.

3.3.1.2 The FOP topside clearance h_w, in m, shall not be less than the largest of values determined for extreme effects of waves and ice:

for waves —

$$h_w = \Delta_{100} + 1{,}2(D/\lambda_{100})^{1/4}h_{100} + 1{,}5 \qquad (3.3.1.2\text{-}1)$$

where Δ_{100} = peak amplitude of a sea level change which is probable once in 100 years, in m;
h_{100} = wave height with 0,1 % probability which is probable once in 100 years, in m;
λ_{100} = associated average wave height corresponding to the wave height with 0,1 % probability which is probable once in 100 years, in m;
D = diameter of a cylindrical leg or the cross dimension of a conic leg at the waterline level, in m;

for ice —

$$h_{‰} = 4h_{raf100} + \Delta_{100} + 0{,}5 \qquad (3.3.1.2\text{-}2)$$

where h_{raf100} = thickness of rafted ice which is probable once in 100 years, in m.

Where structural details like lugs, inserts, etc. are available, the value of clearance for ice conditions is determined experimentally.

3.3.1.3 In shallow waters a whipping (splashing over) phenomenon may be observed. At present, its severity is effectively determined by an experimental approach only and its determination is necessary while evaluating the clearance value.

3.3.1.4 Calculating FOP hull strength, the provisions of 2.5 shall be followed as well as the provisions of 3.3.4.

3.3.2 Loads.

3.3.2.1 Wave loads.

3.3.2.1.1 Wave load applied to a platform and its elements are determined on the basis of the Morison equation (refer to 2.3.9.1 and 2.3.9.2).

3.3.2.1.2 Only one inertia component for a FOP at $D > h_{100}/\pi$ shall be considered. Then, in order to determine velocities and accelerations of water particles, the linear theory of waves of small amplitude may be used.

FOP may be represented by combination of different architectural forms. As the basic elements the cylindrical and conic legs are generally used. With reference to these elements, the wave load parameters are given in 3.3.2.1.3 — 3.3.2.1.7.

3.3.2.1.3 For structures of an exactly cylindrical configuration, the standard deviation of the horizontal component of a wave load, MN, may be determined by the formula

$$\sigma_Q^{hor} = 3 \cdot 10^{-3}\gamma(h_3)_{max}D^2K_v th\bar{K}H; \qquad (3.3.2.1.3\text{-}1)$$

and the standard deviation of the horizontal component of a wave load applied to a conic leg, by the formula

$$\sigma_Q^{hor} = 3 \cdot 10^{-3}\gamma(h_3)_{max}D^2K_v th\bar{K}H \times \left\{1 - \frac{4}{\bar{K}Dtg\propto}(\bar{K}H - 1/\bar{K}H + 1/sh\bar{K}H) + \frac{4}{(\bar{K}D)^2(tg\propto)^2} \times \right.$$
$$\left. \times\ [2 + (\bar{K}H)^2 - 2\bar{K}H/th\bar{K}H]\right\} \qquad (3.3.2.1.3\text{-}2)$$

where γ = water density, in t/m³;
$(h_3)_{max}$ = wave height with 3 % probability of exceeding level, in m (refer to 2.2.2.5);
D = diameter of a cylindrical leg or the cross dimension of a conic leg at the bottom level, in m;
$\bar{K} = 2\pi/\bar{\lambda}$ = wavenumber;
$\bar{\lambda}$ = average wave length, in m;
K_v = diffraction correction (refer to 2.3.9.2; in this case the diameter d refers to that at the waterline level);
$\propto$ = angle of a cone inclination to horizon (the leg is vertical when $\propto = 90°$);
H = depth in a water area, in m.

3.3.2.1.4 The coordinate of a horizontal wave load component applied to a cylindrical leg and measured from the seabed level is determined by the formula

$$Z_0/H = \frac{1}{\bar{K}H} \cdot (1 - ch\bar{K}H + \bar{K}H \cdot sh\bar{K}H)/sh\bar{K}H. \qquad (3.3.2.1.4)$$

3.3.2.1.5 The vertical force due to waves effect depends on permeability of a base associated with the soil type. In the absence of permeability, the vertical force may be neglected. For a rocky or large-shingle bed, the vertical force shall be taken into account and when an additional capsizing moment (refer to 3.3.2.1.7) is determined as well.

3.3.2.1.6 The design value of a horizontal component of a wave load applied to a platform in the most severe mode is allowed to determine using the formula

$$Q = \sigma_Q\sqrt{2lnpN} \qquad (3.3.2.1.6)$$

where σ_Q = standard deviation according to 3.3.2.1.3;
p = recurrence of an extreme mode determined by statistical data for the given operational area of operation defined by the $(h_3)_{max}$, value (refer to 2.2.3.5);
N = sampling extent appropriate to the entire life cycle (with a view to an ice period).

3.3.2.1.7 The design value of an overall capsizing moment due to waves effect on a vertical cylindrical platform standing on permeable seabed is determined by the formula

$$M_{cap} = \sqrt{(QZ_Q)^2 + M_V^2} \qquad (3.3.2.1.7\text{-}1)$$

where Q = refer to 3.3.2.1.6;
Z_Q = coordinate of load application to a cylindrical leg, in m;

M_V = additional capsizing moment due to vertical wave pressures determined by the formula

$$M_V = \sigma_{M_V}\sqrt{2ln(pN)}; \quad (3.3.2.1.7\text{-}2)$$

σ_{M_V} = standard deviation of an additional capsizing moment determined as

$$\sigma_{M_V} = \frac{\gamma h_3 D^3}{ch(2\pi H/\bar{\lambda})}\psi_V \quad (3.3.2.1.7\text{-}3)$$

ψ_V = coefficient of the additional capsizing moment due to waves effect on the foundation of an obstacle with due regard for permeability of a base determined according to Fig. 3.3.2.1.7.

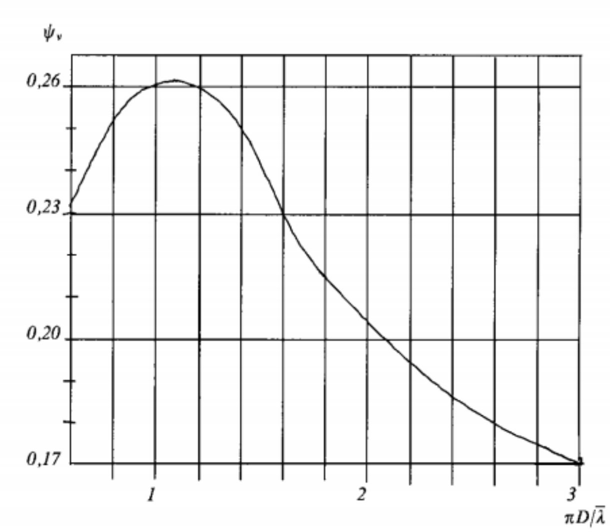

Fig. 3.3.2.1.7
Value of an additional capsizing moment parameter ψ_V

3.3.2.1.8 The design wave load applied to a multileg structure is determined as the sum of wave loads applied to the legs and the load applied to an underwater pontoon:

.1 the wave load applied to legs is determined by the formula

$$Q = n\sigma_Q\gamma_n\sqrt{2ln(pN)} \quad (3.3.2.1.8.1)$$

where n = number of legs;

σ_Q = standard deviation according to 3.3.2.1.3;

γ_n = coefficient of influence of the distance L between n legs on a wave load which corresponds to the heading angle φ_d determined as

$\varphi_d = \frac{\pi}{n}(2;-1), i = 1, 2, \dots n,$

$$\gamma_n = \frac{1}{\sqrt{2}}\sqrt{1-\cos(\bar{\omega}^2 L_n/g)};$$

$$L_n = \begin{cases} L_4 = \sqrt{2}L \\ L_4 = \dfrac{\sqrt{3}}{3}L; \end{cases}$$

p = recurrence of an extreme mode;

N = sampling extent, $N = 10^8$;

$$\varphi_d = \frac{\pi}{n}(2i-1), i = 1, 2, \dots n;$$

.2 the load applied to an underwater pontoon is determined by the formula

$$Q = (\gamma \pi h/2) D^2 (shKd/chKH)\beta \quad (3.3.2.1.8.2)$$

where γ = water density, in t/m³;
h = design wave height (with 1 % probability of exceeding level, in m);
D = reduced diameter of pontoon, in m;

$$D = \sqrt{4S/\pi}$$

where S = pontoon area in plan, in m²;

d = pontoon height, in m;
H = water area depth, in m;
λ = wave length (with 1 % probability) of exceeding level taken into consideration, in m;
β = coefficient depending on the ratio $\pi D/\bar{\lambda}$ (refer to Fig. 3.3.2.1.8.2).

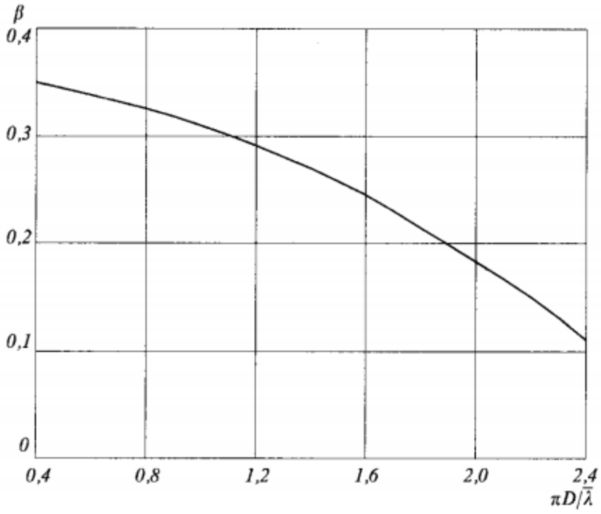

Fig. 3.3.2.1.8.2
Coefficient $\beta - \pi D/\bar{\lambda}$

3.3.2.1.9 Where a FOP differs architecturally from the forms considered, the adequate calculation methods shall be used; the experimental analysis shall be used, when deemed necessary.

3.3.2.1.10 The peak value of a local wave pressure applied to a hull structure at large and its separate components in an open water area for the region of alternating waterlines (by 8 m up and down from a design waterline) shall be assumed in accordance with Table 3.3.2.1.10; for the structures located above the region of alternating waterlines and below the clearance height in terms of waves, the design pressure is 0,05 MPa, and for the above located structures, extending the outboard side, at least 0,02 MPa.

Table 3.3.2.1.10

Surface orientation	Pressure, MPa	
	FOP in service	Transportation of a FOP at large or of its separate blocks[1]
Vertical	0,15	0,10
Deviation from a vertical for more than 30°	0,10	0,05

[1] Data are given without regard for slamming; to be increased if slamming is allowed for.

3.3.2.2 Wind and current loads.

3.3.2.2.1 Wind loads are determined by Formulae (2.3.8-1) — (2.3.8-3).

3.3.2.2.2 Current loads are obtained with due regard for summation of current velocities v_c, m/s, and orbital velocities of water particles in wave v, m/s.

In this case, wave pressures on a cylindrical element are determined as

$$q = (\rho C_{sr} d/2)(v + v_c)|v + v_c| \tag{3.3.2.2.2-1}$$

The v component of load only due to a current acting on a cylindrical leg is determined by the formula

$$Q = (\rho C_{sr} d/2) v_c^2 H_0 \tag{3.3.2.2.2-2}$$

where ρ = mass water density, in t/m³;
C_{sr} = speed resistance coefficient of an obstacle;
d = obstacle diameter, in m;
H_0 = water area depth, in m.

3.3.2.2.3 Evaluating wave loads with use of spectral transformations, the transformation of a spectrum with due regard for a current shall be considered (refer to 2.2.4.2). As a basis calculation velocity-generated forces applied to a cylindrical leg, it is recommended to use the expression

$$Q = (\rho C_{sr} d/2)\left\{v_c^2 H_0 + \sqrt{8/\pi}\left(1 + v_c^2/\sigma_{v_0}^2\right)\left(\frac{g v_0 \sigma_{v_0}}{\bar{\omega}^2 + \omega^2}\right)\right\} \tag{3.3.2.2.3}$$

where g = acceleration of gravity, in m/s²;
v_0 = amplitude of the orbital velocity of surface wave water particles, in m/s;
σ_{v_0} = standard deviation of the orbital velocity of surface wave water particles,

$\sigma_{v_0} = 0{,}19 h_3 \bar{\omega}$;

$\bar{\omega}$ = average waves frequency, in s^{-1};
h_3 = wave height with 3 % probability of exceeding level, in m;

ω = waves frequency, in s^{-1}.

3.3.2.3 Ice loads.

3.3.2.3.1 Ice loads are classified as global and local. The global loads are divided into horizontal and vertical. The global loads may comprise permanent and variable parts. These loads may be considered as static and dynamic loads.

3.3.2.3.2 Global loads are determined by loads of level ice, rafted ice and ridges. Global loads are used for strength analysis, assessment of stability and possible fatigue damage of the structure. Local loads are used for strength analysis and calculation of fatigue damage of the structure materials.

3.3.2.3.3 Interaction between level or rafted ice and single-support platform with vertical sides shall be considered with regard to different possible scenarios. When the structure is intruded into the ice at a distance when the maximum possible contact area is reached, ice is broken. If the floe kinetic energy is not enough for its breaking, the floe stops. After the floe stop ridging may occur when the stopped floe will accumulate the loads over the large contact area and transfer them to the structure.

3.3.2.3.4 When the single-support platform with vertical sides interacts with moving ice field, the global loads due to ice breaking, stopping and ridging are compared as per Formulae (3.3.2.3.4-1), (3.3.2.3.4-2), (3.3.2.3.4-3), respectively:

$$F_{x1} = mK_L K_V \sigma_c D^{0,85} h^{0,9}; \quad (3.3.2.3.4\text{-}1)$$

$$F_{x2} = 1{,}33h(\rho_i D)^{1/3}(\sigma_c D_1 V)^{2/3}; \quad (3.3.2.3.4\text{-}2)$$

$$F_{x3} = 2h^{1{,}25} D_1^{0{,}5},\ \text{MN, at } 100\text{ m} < D_1 \le 1500\text{ m} \quad (3.3.2.3.4\text{-}3)$$

$$F_{x3} = 77{,}5,\ \text{MN, at } D_1 > 1500\text{ m},$$

where m = plan leg shape factor in direction of ice motion (m = 0,9 for circular cross-section and polygonal cross-section structures; m = 1 for rectangular cross-section structures);
σ_c = uniaxial ice compression strength, in MPa;
ρ_i = ice density, in kt/m³;
K_L = factor determined by Formula (3.3.2.3.4-4) which considers influence of ratio between the field area (equivalent field diameter $D_1 = 2\sqrt{A_i/\pi}$) and structure diameter D on the load;
K_V = factor determined by Formula (3.3.2.3.4-5) which considers ice speed V and ice thickness h;

$$K_L = \begin{cases} 1, \text{ at } D_1/D \ge 10; \\ 1 - 0{,}0667\left(10 - \frac{D_1}{D}\right), \text{ at } 10 > D_1/D > 3; \\ 0{,}6 \text{ at } D_1/D \le 3 \end{cases} \quad (3.3.2.3.4\text{-}4)$$

$$K_V = (1{,}6 - 20V/h), \text{at } V/h < 3 \times 10^{-2}$$
$$K_V = 1, \text{at } V/h \ge 3 \times 10^{-2}. \quad (3.3.2.3.4\text{-}5)$$

If the load determined by Formula (3.3.2.3.4-1) is less than that determined by Formula (3.3.2.3.4-2), the ice breaking occurs. The global load determined by Formula (3.3.2.3.4-1) is taken as characteristic load. Formulae corresponding to ice breaking are also used for large ice consolidation C_p (ice-covered area) ($C_p > 0{,}7$) when the load due to ridging as per Formula (3.3.2.3.4-3) exceeds the load due to ice breaking as per Formula (3.3.2.3.4-1) at speed of ice drift $V = 0{,}01$ m/s.

When the load determined by Formula (3.3.2.3.4-2) is less than that determined by Formula (3.3.2.3.4-1), floe stop occurs and one of three conditions is met: small floe area ($D_1 \le 100$ m); small ice consolidation ($C_p \le 0{,}7$); load due to ridging determined by Formula (3.3.2.3.4-3) is less than that due to the floe stop determined by Formula (3.3.2.3.4-2). In case of the floe stop the global load determined by Formula (3.3.2.3.4-2) is taken as characteristic load.

The ice ridging occurs when the load determined by Formula (3.3.2.3.4-2) is less than that determined by Formula (3.3.2.3.4-1), and in case of large ice consolidation ($C_p > 0{,}7$) and large floe area ($D_1 > 100$ m), the load due to ridging determined by Formula (3.3.2.3.4-3) is less than that due to ice-breaking determined by Formula (3.3.2.3.4-1) at $V = 0{,}01$ m/s, but more than the load due to floe stop determined by Formula (3.3.2.3.4-2). When ridging occurs, the global load determined by Formula (3.3.2.3.4-3) is taken as characteristic load.

3.3.2.3.5 For the structures frozen into ice, with the water level insignificant fluctuations within the day, global loads shall be determined by Formula (3.3.2.3.4-1) at $V = 0{,}01$ m/s.

In case the structure is frozen into ice for more than three days at ambient temperature below – 5 °C, the global load is determined by the following formula:

$$F_{1f} = 1{,}6mK_L\sigma_c D^{0{,}85}h^{0{,}9}. \qquad (3.3.2.3.5)$$

3.3.2.3.6 The point located at a distance of $0{,}3h$ below design water level is taken as the point of application of the resultant of ice pressure.

3.3.2.3.7 The global load on the single-support platform with vertical sides due to ridge load shall be determined as a sum of loads due to consolidated layer of ridge F_c and ridge keel F_k.

3.3.2.3.8 The global load on the single-support platform with vertical sides due to consolidated layer of ridge shall be determined by Formulae (3.3.2.3.4-1) — (3.3.2.3.4-3) by substituting thickness of consolidated layer of ridge h_c for h and strength of consolidated layer of ridge σ_{cr} for σ_c.

3.3.2.3.9 The global horizontal load on the single-support platform with vertical walls due to ridge keel is determined by the following Formulae:

$$F_{k1} = \mu(h_k - h_c)D\left[\frac{(h_k-h_c)\mu\gamma_e}{2} + 2C_k\right]\left(1 + \frac{(h_k-h_c)}{6D}\right); \qquad (3.3.2.3.9\text{-}1)$$

$$F_{k2} = \gamma_e tg\varphi_k(h_k - h_c)[DW_k + (h_k - h_c)W_k] + \mathrm{C}_k W_k D + 2\mathrm{C}_k W_k(h_k - h_c); \qquad (3.3.2.3.9\text{-}2)$$

$$F_k = F_{k1}, \text{ when } F_{k2} > 2F_{k1}; \qquad (3.3.2.3.9\text{-}3)$$

$$F_k = 2\frac{F_{k1}F_{k2}}{2F_{k1}+F_{k2}}, \text{ when } F_{k1} > 2F_{k2} \qquad (3.3.2.3.9\text{-}4)$$

where $\mu = tg(45° + \varphi_k/2)$;
$\gamma_e = g(1 - \pi_k)(\rho_{wat} - \rho_i)$;
h_k = ridge keel depth;
C_k = adhesion of ridge keel material, in MPa;
φ_k = angle of internal friction of ridge keel;
p_k = porosity/hollowness factor of ridge keel;
ρ_{wat} = sea water density, in kt/m³;
W_k = ridge keel width normal to its front, in m.

3.3.2.3.10 The point located at a depth of $0{,}3h_c$ from the water surface is taken as the point of application of the resultant of horizontal global load due to consolidated layer of ridge. The point of application of the resultant of horizontal component of the global load due to keel is taken $1/3(h_k - h_c)$ below the boundary of consolidated layer.

The maximum height of ice ridge in front of the single-support platform with vertical sides is taken equal to eight thicknesses of surrounding level ice.

3.3.2.3.11 The global load for interaction between level or unlimited rafted ice and multileg platform with single-row assembly of n vertical legs of cylinder type within ice load area is determined by the following Formulae:

$$F_n = 0{,}85F_1K_1K_2[1 + (n - 1)(\cos\alpha_i + 0{,}3\sin\alpha_i)]; \qquad (3.3.2.3.11\text{-}1)$$

$$k_1 = 0{,}83 + 0{,}17n^{-1/2}; \qquad (3.3.2.3.11\text{-}2)$$

$$K_2 = \begin{cases} 0{,}7 + 0{,}06(L/D - 1) \text{ at } L/D < 6 \\ 1 \text{ at } L/D > 6 \end{cases} \qquad (3.3.2.3.11\text{-}3)$$

where n = number of legs;
F_n = total load on the platform;
F_1 = load on the single platform under the same ice conditions calculated by Formulae (3.3.2.3.4-1) — (3.3.2.3.4-5);
α_i = angle between the direction of ice motion and the normal to the structure front;
K_1 = factor considering ice discontinuities;
K_2 = factor considering mutual influence of front legs;
L = distance between centers of adjacent legs along the front, in m (refer to Fig. 3.3.2.3.11);
D = leg diameter, in m.

Formulae (3.3.2.3.11-1) — (3.3.2.3.11-3) are valid at $L/D > 2$. Otherwise, the load on the structure shall be determined as the load on extended structure with length $L(n-1)$.

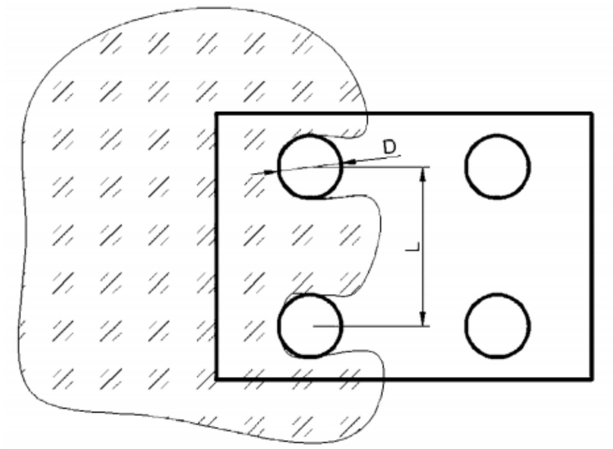

Fig. 3.3.2.3.11
Multileg platform

3.3.2.3.12 The global horizontal load on the single-row leg assembly due to limited ice field shall be selected as the minimum value of the values determined by Formulae (3.3.2.3.11-1) and (3.3.2.3.4-2). Thus, in Formula (3.3.2.3.4-2) D shall be replaced with nD.

3.3.2.3.13 The global horizontal load for interaction between level or unlimited rafted ice and multileg platform with multi-row assembly of n vertical legs of cylinder type within ice load area is determined by the following Formulae:

$$F_n = 0{,}85 n F_1 K_1 K_3; \qquad (3.3.2.3.13\text{-}1)$$

$$K_3 = \begin{cases} 0{,}7 \text{ at } L/D > 5 \\ 0{,}45 + 0{,}05 L/D \text{ at } 5 \le L/D < 3 \end{cases} \qquad (3.3.2.3.13\text{-}2)$$

where K_3 = factor considering mutual influence of legs;
for F_1, K_1 refer to 3.3.2.3.11.

At $L/D < 3$ the global load shall be determined as a load on the multileg platform for ice clogging between legs (refer to 3.3.2.3.16).

3.3.2.3.14 The global horizontal load on the multi-row assembly of legs due to limited ice field shall be determined as follows. When ice is moving in the normal direction to the line connecting the front legs, the loads for front legs shall be compared as per

Formulae (3.3.2.3.11-1) and (3.3.2.3.4-2) by replacing D with $n_1 D$ where n_1 is a number of the front legs. The minimum load shall be taken as the characteristic one.

3.3.2.3.15 When the multi-row assembly of legs is frozen into ice, the global horizontal load is determined by the Formulae:

$$F_{nf} = F_1 K_{3f} \tag{3.3.2.3.15-1}$$

$$K_{3f} = n, \text{ at } L/D > 4; \tag{3.3.2.3.15-2}$$

$$K_{3f} = n(0,5 + L/8D), \text{ at } 2 < L/D > 4 \tag{3.3.2.3.15-3}$$

where F_{1f} = load on one leg frozen into ice determined by Formula (3.3.2.3.5).

3.3.2.3.16 The global load on the multileg platform at ice clogging between the legs is determined by the formula

$$F_{nz} = 0,8 F_{nf}. \tag{3.3.2.3.16}$$

3.3.2.3.17 The global load on the multileg structure due to ridges shall be determined as a sum of loads due to consolidated layer of ridge and ridge keel. The global load due to consolidated layer of ridge shall be determined by Formula (3.3.2.3.13-1). Here, F_1 is calculated according to 3.3.2.3.4 by substituting parameters of consolidated layer of ridge. The global load due to ridge keel shall be determined as a sum of loads on separate legs with regard to the local properties of ridge contacting with each leg at time corresponding to the maximum global load. Several positions of ridge relative the legs shall be considered, and the position, which corresponds to the maximum load, shall be selected. The load due to ridge keel on each leg is determined by Formula (3.3.2.3.9-1) by replacing $(h_k - h_c)$ with $(h_{ki}(t) - h_c)$ where $h_{ki}(t)$ is the ridge keel depth for the *i*-th leg at time t.

3.3.2.3.18 The horizontal F_{xc}. and vertical F_{zc} components of the global load on platforms with flaring conic sides (legs) or on vertical platforms with flaring conic inserts, or on polygonal structures with similar inclined sides due to level or rafted ice are determined to the quasistatic approximation by the Formulae:

$$F_{xc} = K_V \left[A_1 \sigma_f h^2 + A_2 \rho_{wat} g h D^2 + A_3 \rho_{wat} g p_1 h_r \left(D^2 - D_b^2\right)\right] A_4; \tag{3.3.2.3.18-1}$$

$$F_{zc} = B_1 F_{xc} + K_V B_2 \rho_{wat} g p_1 h_r \left(D^2 - D_b^2\right) \tag{3.3.2.3.18-2}$$

where
σ_f = bending strength;
D = cone diameter at the waterline level;
D_b = cone diameter at height;
h_b = upper mark of the conic part of platform (leg) (refer to Fig. 3.3.2.3.18-1);
$h_r \cong 2h$;
p_1 = porosity of floes on the structure surface (if there is no data, $p_1 = 0{,}6$);
K_V = factor depending on speed of ice drift which is determined by Formula (3.3.2.3.18-3);
$A_1, A_2, A_3, A_4, B_1, B_2$ = factors, which values are given in Figs. 3.3.2.3.18-2 — 3.3.2.3.18-6;
h_m = the maximum possible height of ice crawling onto the platform, which is approximately determined by Formula (3.3.2.3.18-4);

$$K_V = \begin{cases} 1, \text{ at } V < 0{,}5 \text{ m/s} \\ 1 + 0{,}7(V - 0{,}5) \text{ at } V \geq 0{,}5 \text{ m/s} \end{cases} \tag{3.3.2.3.18-3}$$

$$h_m = \begin{cases} 3 + 4h \text{ at } D/l_c \geq 2{,}0 \\ 5h\sin\alpha \text{ at } D/l_c \leq 0{,}5 \\ 5h\sin\alpha + \frac{3+h(4-5\sin\alpha)}{1{,}5}\left(\frac{D}{l_c} - 0{,}5\right) \\ \text{at } 0{,}5 < D/l_c < 2{,}0 \end{cases} \quad (3.3.2.3.18\text{-}4)$$

where α = angle of a side inclination to horizon;

$l_c = \left(\frac{Eh^3}{12\rho_w g(1-v^2)}\right)^{1/4}$;

E = modulus of elasticity of level or rafted ice, MPa;
v = Poisson's ratio, $v = 0{,}3$.

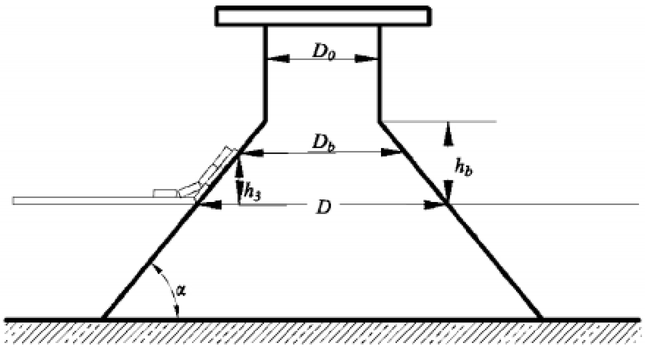

Fig 3.3.2.3.18-1
Platform with flaring conic sides

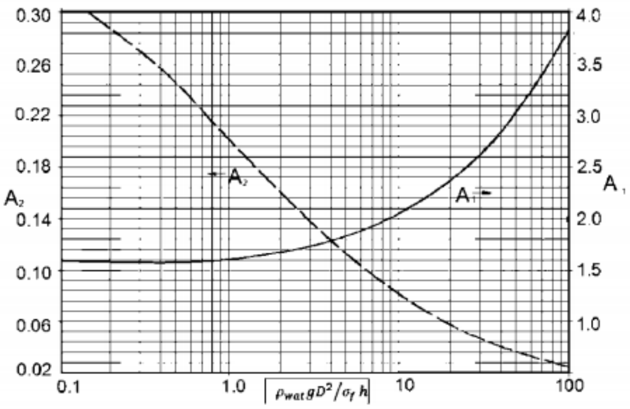

Fig. 3.3.2.3.18-2
Values of factors 1, 2

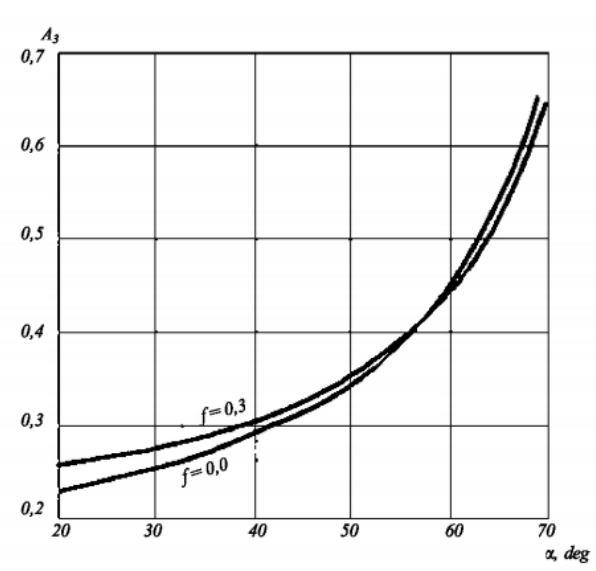

Fig. 3.3.2.3.18-3
Values of factor A_3 against the angle of side inclination for different friction coefficients f

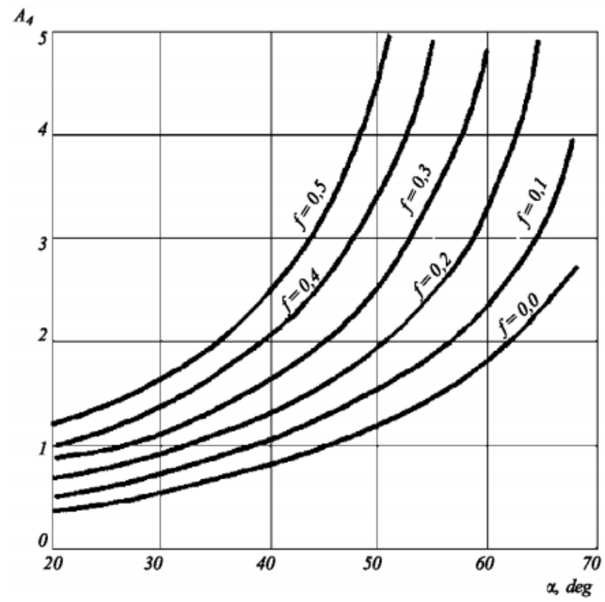

Fig. 3.3.2.3.18-4
Values of factor A_4 against the angle of side inclination for different friction coefficients f

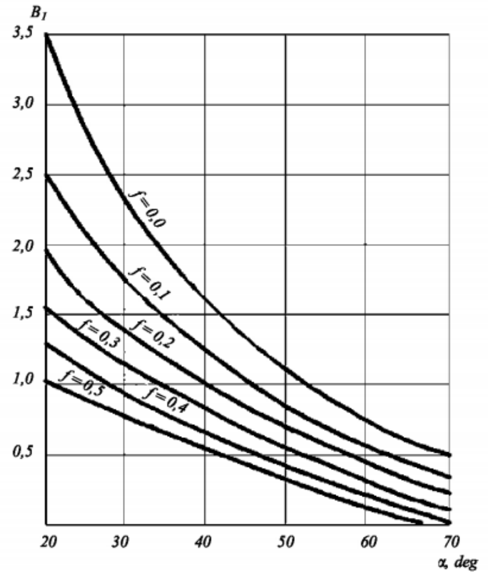

Fig. 3.3.2.3.18-5
Values of factor B_1 against the angle of side inclination for different friction coefficients f

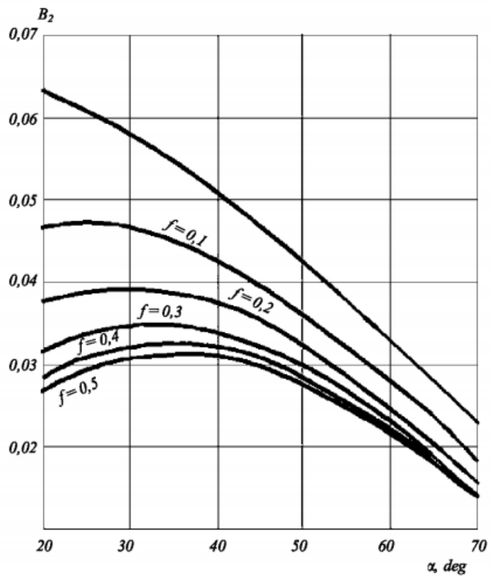

Fig. 3.3.2.3.18-6
Values of factor B_2 against the angle of side inclination for different friction coefficients f

3.3.2.3.19 The horizontal and vertical components of the global load on platforms with flared inclined sides due to level or rafted ice shall be determined by Formulae (3.3.2.3.18-1), (3.3.2.3.18-2) by replacing ρ_{wat} with $(\rho_{wat} - \rho_i)$, assuming $h_r \cong 2h$ and determining D_b on a mark corresponding to the bottom of the inclined part of the platform (leg).

3.3.2.3.20 In case of shift of the ice field frozen to the platform with inclined sides or with conic inserts, the horizontal component of the global load is determined by the formula

$$F_{1f} = 1{,}6 k_f \sigma_c D^{0{,}85} h^{0{,}9} \qquad (3.3.2.3.20)$$

where k_f = factor depending on the angle of side inclination which values are given in Table 3.3.2.3.20.

Table 3.3.2.3.20

Values of factor k_c against the angle of inclination α of generators of the cone or sides to horizon

Angle of inclination of the generator of α cone (sides) to horizon, in deg.	45	60	75
k_f	0,6	0,7	0,9

3.3.2.3.21 The global load on platforms with inclined sides due to ridges shall be determined as a sum of loads due to consolidated layer of ridge and ridge keel. The global load due to consolidated layer of ridge shall be determined by Formulae (3.3.2.3.18-1) — (3.3.2.3.18-4) by replacing the parameters of level (rafted) ice with those of consolidated layer of ridge. The global load on the platform with inclined sides due to ridge keel is determined by the formula

$$F_{xc} = F_k \sin\alpha\,, F_{zk} = F_k \cos\alpha \qquad (3.3.2.3.21)$$

where F_k shall be determined by Formula (3.3.2.3.9-1).

When performing dynamic calculations of the structures, the current value $h_k(t)$, which is determined by its underwater shape shall be substituted to Formula (3.3.2.3.9-1).

The point of application of global forces due to consolidated layer of ridge and ridge keel shall be taken according to 3.3.2.3.10.

3.3.2.3.22 The extent of an ice strake of the platform l, in m, shall be at least

$$l = \Delta_{100} + 2\alpha_1 h_{c,100} \qquad (3.3.2.3.22)$$

where Δ_{100} = maximum swing of the sea level change relative to the average level probable once in 100 years, in m;
α_1 = safety factor; $\alpha_1 = 1{,}1$;
$h_{c,100}$ = thickness of the consolidated layer of ridge probable once in 100 years (in the absence of ridge, the thickness of level or rafted ice), in m.

The l value is symmetrically plotted up and down relative to an average water level.

3.3.2.3.23 Local ice pressures on the structure within an ice strake of a conic obstacle are determined by the formula

$$p = \bar{\sigma}_c \left(1 + 2\sqrt{\frac{2}{A_1}}\right)\frac{\sqrt{\alpha}}{8{,}5}, \text{ МПа}, \qquad (3.3.2.3.23)$$

where $\bar{\sigma}_c$ = average ice compression strength, in MPa. In the absence of average ice strength data, the following relation may be used as the first approximation:

$$\bar{\sigma}_c = 0{,}75\bar{\sigma}_c^{100}$$

where $\bar{\sigma}_c^{100}$ = ice compression strength recurring once in 100 years;

A_1 = contact area, in m²;
α = angle of inclination of the generator of a cone to horizon, in deg.; $18° < \alpha < 72°$.

The pressure p at $\alpha > 18°°$ is determined as at 18° and at $\alpha > 72°$ the pressure p is determined as at 72°. The pressure p is set equal to 10 MPa when $p > 10$ MPa.

Local ice pressures on the structure within an ice strake of a vertical obstacle are determined by the above formula in which factor $\sqrt{\alpha}/8{,}5$ shall be omitted.

3.3.23.24 Local ice pressures on the structure in areas above and below an ice strake (refer to 3.3.2.3.23) are determined as the part of pressures on ice strake structures.

Local ice pressures on the structure below an ice strake with the extent of $0{,}5h_k$ are determined in accordance with the relation $p_b = p/4$, but shall be not less than 2 MPa where p is in line with 3.3.2.3.23. The value of ice pressures below the areas specified (where possible) is determined with due regard to the details of ice situation in the area of operation.

Local ice pressures on the structures above an ice strake within the area of ridge height are determined in accordance with the relation $p_a = p/8$, but shall be not less than 1,5 MPa where p is in line with 3.3.2.3.23.

3.3.2.3.25 The vertical tubular or rectangular platforms shall be calculated with due account of icing. Icing results in additional vertical load due to weight of accrued ice and in additional horizontal load due to increase in transverse dimensions of the platform.

The structures with inclined sides shall be calculated with due account of the change in angle of the platform inclination due to icing.

The thickness of an ice accrued on the leg Δ_i, in m, due to fluctuations of the water level is determined by the formula

$$\Delta_i = (0{,}0054 + 0{,}00146|T_{aver}|)N \tag{3.3.2.3.25-1}$$

where T_{aver} = average negative temperature for the period N, in days, within which the icing occurs.

The stable icing starts in 10 — 15 days after steady negative temperatures. Icing ends at positive daily temperatures in spring, or when average daily temperature exceeds –2 °C.

Height of ice part of the leg h_n is equal to a difference between the maximum tide mark (due to atmospheric pressure and storm surge) and the minimum tide.

Mass of accrued ice formation per 1 m length along the leg contour, in MN/m, is determined by the formula

$$G_1 = 0{,}8h_n\Delta_i g\rho_n \tag{3.3.2.3.25-2}$$

where ρ_n = density of accumulated ice, in kt/m³.

3.3.2.3.26 Dynamic aspects of ice loads on FOP.

Dynamic aspects of ice loads shall be considered when calculating the FOP local strength as well as during analysis of the structure vibrations to define fatigue damage for the structure members and normal operating conditions for the crew.

When accessing dynamic aspects of ice loads on the platforms with vertical sides, the ice load may be presented as a sine with a swing equal to half maximum ice load on the fixed structure. The following formula may be used as a loading period for approximate calculations:

$$T = \frac{600h^{0,2}D^{0,8}}{VE}\sigma_c, \text{ in s.} \tag{3.3.2.3.26-1}$$

When considering dynamic aspects of the structures with inclined sides, the ice load shall be represented as a sine with a swing equal to half maximum ice load on the fixed structure. The following formula may be used as a loading period for approximate calculations:

$$T = 7h/V. \qquad (3.3.2.3.26\text{-}2)$$

If the ice load period turns out to be close to the FOP natural periods, the vibrations of the platform exposed to ice loads shall be additionally analyzed. Dynamic aspects of the structure exposed to ice loads shall be calculated according to specific programs approved by the Register.

3.3.2.3.27 The values of ice loads and dynamic aspects of ice loads on FOP may be refined on the basis of field observations or laboratory research data, and also on the basis of specific procedures and programs approved by the Register.

3.3.2.4 Seismic effects.

3.3.2.4.1 FOP operation in the areas of seismic activity is associated with loads of an essential value applied to a structure, which may result in quite adverse consequences. In some areas, the seismic effect may be adopted as the design case of loading which defines FOP structural decisions.

FOP shall be designed and operated so as:

to prevent the threat to people's safety, of pollution of the environment with oil and gas production products, and to keep up repair ability of the structure and equipment on exposure to a design earthquake;

to avoid FOP capsizing and catastrophic pollution of the marine environment on exposure to a maximum design earthquake; in this case, other damages, which may upset the normal operation of a structure, are allowed.

Seismic stability is ensured by the following:

the selection of a seismically favourable building site, a structural and planning diagram, and materials;

application of special structural arrangements;

the relevant calculation of structures;

quality of construction and installation work execution;

inclusion in FOP designs of a special section on the earthquake monitoring during the structure operation.

3.3.2.4.2 Designing structures it shall be taken into account that seismic forces may have any space orientation, horizontal and vertical inclusive.

3.3.2.4.3 In calculations of FOP seismic stability the following seismic loads shall be considered:

inertia forces generated by seismic structure shakings, and distributed in a structure space and its base;

hydrodynamic pressure on a structure generated by an inertia effect of the liquid part vibrating along with the structure, and distributed across the surface of the structure contact with water;

hydrodynamic pressure due to seismic sea waves in an earthquake.

3.3.2.4.4 Calculating seismic stability of equipment and structures located in the above-water part of a FOP, seismic effects are specified by accelerations transmitted to these structures and equipment by bearing structures of the FOP hull. In this case, the peculiarities of the dynamic interaction of objects and structures in question shall be considered.

3.3.2.4.5 FOP are calculated for the seismic effects of design and maximum design earthquake levels using methods of a dynamic theory of seismic stability.

At preliminary stages of design, a linearly spectral theory of seismic stability may be used.

3.3.2.4.6 The calculations of structures for a design earthquake according to the dynamic theory of seismic stability are conducted with use of a linear time dynamic analysis wherein the structure materials and base soils are assumed linearly elastic, and geometric and structural nonlinearity in behaviour of a structure-base system is absent.

The linear time dynamic analysis is carried out either by the method based on the solution expansion into a series to the forms of natural vibrations of the structure or by the method based on stepwise integration of a set of differential equations.

3.3.2.4.7 The calculations of structures for a maximum design earthquake according to the dynamic theory of seismic stability are conducted with the use of any techniques of time dynamic analysis (as a rule, nonlinear based on the method of stepwise integration).

The nonlinear dynamic analysis is carried out according to the special procedures and programs approved by the Register.

The calculations of structures for a maximum design earthquake according to the dynamic theory of seismic stability with use of a linear dynamic analysis are carried out similarly to the calculations of structures for a design earthquake according to the dynamic theory of seismic stability.

3.3.2.4.8 The calculations of a FOP according to a dynamic theory of seismic stability shall be conducted for design accelerograms selected (of instrumental records, analog or synthesized accelerograms) with such values of a maximum peak acceleration a_p in a base that the values of these accelerations a_p^{de} (in calculation for a design earthquake) and a_p^{mde} (in calculation for a maximum design earthquake) have the values meeting recurrences of 100 and 500 years, respectively. In this case, the following conditions shall be fulfilled:

$$a_p^{de} = k_\tau^{de} g A_{100}; \qquad (3.3.2.4.8\text{-}1)$$

$$a_p^{mde} = k_\tau^{mde} g A_{500} \qquad (3.3.2.4.8\text{-}2)$$

where k_τ^{de} = coefficient allowing for the probability of a seismic event under consideration over the design lifecycle of a FOP for the design earthquake, refer to Table .3.2.4.8-1;

k_τ^{mde} = coefficient allowing for the probability of a seismic event under consideration over the design lifecycle of a FOP for the maximum design earthquake, refer to Table 3.3.2.4.8-1;

g = acceleration of gravity, 9,81 m/s²;

A_{100} = design amplitude of a base acceleration expressed as the g fraction; the value of A_{100} is adopted according to Table 3.3.2.4.8-2;

A_{500} = design amplitude of a base acceleration expressed as the g fraction; the value of A_{500} is adopted according to Table 3.3.2.4.8-2.

Table 3.3.2.4.8-1

Design lifecycle, years	k_τ^{de}	k_τ^{mde}
10	0,5	0,70
20	0,63	0,80
50	0,70	0,90

Table 3.3.2.4.8-2

Design seismicity of a building site $J_{100}^{designed}$ ($J_{500}^{designed}$) magnitude	Reference seismicity of a building site $J_{100}^{initial}$ ($J_{500}^{initial}$) magnitude			
	6	7	8	9
6,5	0,08	0,10	–	–
7,0	0,10	0,13	0,17	–
7,5	–	0,16	0,22	–
8,0	–	0,21	0,28	0,36
8,5	–	–	0,35	0,49
9,0	-	–	0,45	0,61

3.3.2.4.9 The calculations of FOP seismic stability according to a linearly spectral theory are allowed to perform by the solution of static problems of elasticity when solid inertia forces of the $\overrightarrow{P}_i(\vec{x})$, intensity corresponding to the *i*-th form of natural vibrations are applied to structures.

Where the system "structure – base" in a calculation is split into separate discrete volumes, then as inertia loads are used nodal inertia forces $\overrightarrow{P}_{ik}$, acting on the structure element assigned to the node k at the *i*-th form of natural vibrations.

In this case, the values of nodal force components $P_{ikj,j} = 1, 2, 3$ are determined by the formulae:

$$P_{ijk}^{de} = 0{,}5 k_H k_\psi m_k a_p^{de} \beta_i \eta_{ikj}\ , \tag{3.3.2.4.9-1}$$

$$P_{ijk}^{mde} = 0{,}5 k_H k_\psi m_k a_p^{mde} \beta_i \eta_{ikj} \tag{3.3.2.4.9-2}$$

where k_H = coefficient allowing for a structure height; its value are taken equal to:
1,0 — for structures 100 m high and over;
0,8 — for structures 60 m high and less;
for structures having a height from 60 m to 100 m the values are determined by interpolation between 1.0 and 0,8;
k_ψ = coefficient allowing for shock absorbing properties of structures; its values are taken equal to:
1,0 — for metal structures, and for concrete and reinforced concrete ones at a design seismicity not exceeding a magnitude 8;
0,8 — for concrete and reinforced concrete structures at a design seismicity over a magnitude 8;
m_k = mass of the structure element assigned to a node к (with due regard to the added mass of water);
a_p^{de} and a_p^{mde} = refer to 3.3.2.4.8;
β_i = dynamic factor corresponding to the *i*-th tone of natural vibrations of the structure;
η_{ikj} = coefficient of the natural vibrations form of the structure for the *i*-th form of vibrations.

3.3.2.4.10 The value of the form coefficient η_{ikj} is determined by the formula

$$\eta_{ikj} = U_{ikj} \sum_k m_k \sum_{j=1}^{3} U_{ikj} \cos(\theta_{ikj}) \,/\, \sum_k m_k \sum_{j=1}^{3} U_{ikj}^2 \tag{3.3.2.4.10}$$

where U_{ikj} = projections along the *j*-th directions of the *k*-th mode shifts for the *i*-th form of natural vibrations of the structure;
$\cos(\theta_{ikj})$ = cosines of angles between the directions of a seismic effect vector and displacements U_{ikj};
m_k = mass of the structure element assigned to the *k*-th node (with due regard for the added mass of water).

3.3.2.4.11 The values of dynamic factor β_i, are determined by the following formulae (or by the graphs in Fig. 3.3.2.4.11):

$$\left.\begin{aligned} \beta(T_i) &= 1 + \frac{T_i}{T_1}(\beta_0 - 1),\ \ 0 < T_i \le T_1; \\ \beta(T_i) &= \beta_0,\ \ T_1 < T_i \le T_2; \\ \beta(T_i) &= \beta_0 T_2^{0{,}5},\ \ T_2 < T_i \end{aligned}\right\} \tag{3.3.2.4.11}$$

where β_0, T_1, T_2 = parameters whose values are given in Table 3.3.2.4.11;
T_i = natural period of platform's vibrations, in s.

The value of a product $k_\psi \beta$ shall be at least 0,80, the value of the coefficient k_ψ therewith is determined in accordance with 3.3.2.4.9.

In addition to the calculations made with use of standard functions $\beta(T_i)$, it is allowed to perform the calculations in which reaction spectra of single-component design accelerograms are used.

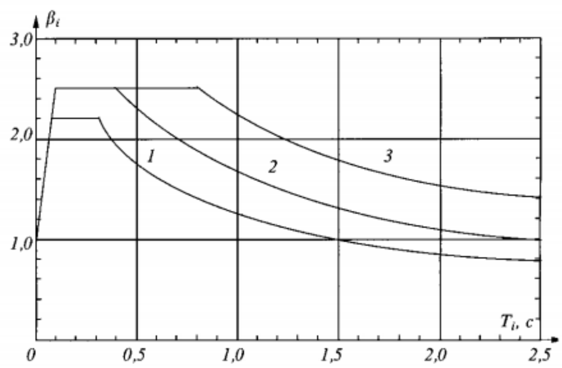

Fig. 3.3.2.4.11
Dynamic factors $\beta(T_i)$:
Curve 1 — for soils of a I category;
Curve 2 — for soils of a I — II, II, II — III, III category for the thickness of an upper soil layer not more than 20 m;
Curve 3 — for soils of a I — II, II, II — III, III category for the thickness of an upper soil layer at least 40 m

Table 3.3.2.4.11

Soil categories by seismic properties	β_0	T_1	T_2
I	2,2	0,08	0,318
I-II, II, II-III, III $H_s \leq 20$ m	2,5	0,10	0,41
I-II, II, II-III, III $H_s \geq 40$ m	2,5	0,10	0,81
Notes: 1. Categories I — II and II — III meet the cases dealing with soils, which, by its composition, are ranked between the soils of I and II or II and III categories respectively. 2. H_s — thickness of soils with the category I soil underlain. 3. The values of parameters T_1 and T_2 at 20 m < H_s < 40 m, are obtained by a linear interpolation between the values of these parameters at $H_s \leq 20$ m and $H_s \geq 40$ m.			

3.3.2.4.12 The design values of shift components (deformations, stresses or forces) with allowance made for all the forms of natural vibrations of a structure to be considered in the calculation are determined by the formula

$$W_j = \sqrt{\sum_{i=1}^{q} W_{ij}^2} \qquad (3.3.2.4.12)$$

where W_j = generalized value of the components of design shifts (deformations, stresses or forces) brought about in the points or sections under consideration by seismic effects;

W_{ij} = generalized value of the components of shifts (deformations, stresses or forces) brought about in the points and sections under consideration by the seismic loads corresponding to the *i*-th form of natural vibrations;

q = number of natural vibration forms considered in calculations.

3.3.2.4.13 The number of natural vibration forms considered in the calculations according to a linearly spectral theory is selected so that the further refinement of calculation results may be neglected with the increase of that number.

3.3.2.4.14 In calculation of FOP strength with due regard for seismic effects in all the cases of contact of lateral surfaces of a structure with the soil, the influence of seismic forces of the soil on the value of a lateral earth pressure shall be allowed for.

Seismic soil forces in the calculation of a structure are determined from a common dynamic calculation of a system, which includes the structure, base and soil used for backfilling.

3.3.2.4.15 With the availability of cohesionless or weakly cohesive soils (e.g. fine — grained sand) in the FOP base, special emphasis shall be placed on the assessment of potential liquefaction of these soils with the reduction of their resistance to shifting under the action of seismic loads.

3.3.2.5 Seabed loads applied to a gravity FOP bottom.

3.3.2.5.1 The pressures on a bearing surface contacting a foundation soil shall be known for use in the calculations of bottom structures strength. These contact pressures are determined with due regard for the shape of a FOP footing and the soil type using the formulae of eccentric compression. Where necessary, these pressures are determined according to the results of a deflected state calculations for a structure — foundation soil system using the methods of mechanics of continua.

3.3.2.5.2 Shear stresses across the contact surface of a FOP footing with a foundation soil caused by vertical forces are usually ignored in strength calculations.

3.3.2.5.3 The maximum design pressure on the bottom from the direction of seabed during a structure operation is determined by multiplication of a mean design pressure by a nonuniformity coefficient dependent on soil properties. The values of the nonuniformity coefficient are given in Table 3.3.2.5.3 for the main types of soils.

Table 3.3.2.5.3

Type of soil surface layer	Value of a nonuniformity coefficient
Silt, clay and loams of sloppy and sloppy-wet consistency at an index of liquidity $l_L > 0{,}75$; sandy loose soils	1,2
Tough and soft clay soils with an index of liquidity $0{,}25 \leq l_L \leq 0{,}75$, sandy firm soils and sandy soils of medium solidity	1,4
Clay soils of stiff and hard consistency ($l_L < 0{,}25$); very firm sandy soils; fluvial soils; large fragmented soils with a sandy aggregate	2,0
Note. The value of a nonuniformity coefficient is where necessary, refined with due regard for specific design conditions.	

3.3.3 Additional strength criteria for ice-resistant FOP structures.

3.3.3.1 The ultimate strength criterion for calculation of outer side grillages is defined by the expressions:

$$\sigma \leq R_{eH};\quad \tau \leq 0{,}57R_e \tag{3.3.3.1}$$

where σ and τ = maximum normal and shear stresses in the elements of girder cross-sections under local ice pressures.

3.3.3.2 The ultimate strength criterion for calculation of separate structural elements of an outer side (plates, stiffeners) is defined by the expression

$$P_p \leq P_{ult}/\gamma \tag{3.3.3.2}$$

where P_p = design load on a structural element due to local ice pressures;
P_{ult} = ultimate load on a structural element;
γ = ultimate load safety factor equal to:
1,2 — for special structural elements;
1,1 — for primary structural elements.

3.3.4 Peculiarities of strength calculation for ice strake structures.

3.3.4.1 When calculating structure strength under ice loads the following loading stages are recommended to be distinguished:

loading of separate structural elements, i.e. plates, stiffeners;

loading of structure grillages;

loading of the structure at large.

In accordance with these stages, the structural elements of an ice strake shall be calculated as specified below.

3.3.4.2 Where a structure is loaded as a whole, the global ice loads calculated according to 3.3.2.3 are assumed as design. All adverse potential loading cases shall be considered.

Evaluating structure strength, the fashion of global ice load distribution may be assumed uniform in the front and height of ice formation (level ice, rafted ice or the consolidated part of a ridge).

The calculation is aimed at the verification of compliance with the strength criteria by Formulae (2.4.2.3.1) and (2.4.2.3.2). When evaluating general and local stresses in structural elements, the provisions of 2.5.2, 2.5.3 and 2.5.4 shall be followed.

3.3.4.3 Where structure grillages are under load, the local ice pressures according to 3.3.2.3.11 are assumed as design loads. In this case, a design contact area A is determined as:

$$A = 10 \text{ m}^2, \text{if } S_{gr} \leq 10 \text{ m}^2;$$

$$A = S_{gr}, \text{if } S_{gr} > 10 \text{ m}^2$$

where S_{gr} = grillage surface area within a rest, in m².

Based on the grillage calculation, the dimension of web girders are selected and the strength criterion for them indicated in 3.3.3.1 shall be met. In determination of stresses, the provisions of 2.5.3 shall be followed.

3.3.4.4 If separate structural members (plates, stiffeners) are loaded, the local ice pressures determined according to 3.3.2.3.11 are assumed as design loads. In this case, a design contact area A is determined as:

$$A = 1 \text{ m}^2, \text{if } S_p \leq 1 \text{ m}^2;$$

$$A = S_p, \text{if } S_p > 1 \text{ m}^2$$

where S_p = plate surface area or the loaded area of a stiffener.

As an ultimate load $\bar{P}_{ult}$ shall be considered:

p_{ult} = ultimate pressure on a plate;

Q_{ult} = ultimate load on a stiffener.

The ultimate pressure p_{ult}, on a plate restrained on a rest and loaded by an equidistributed load across the plate surface is determined by the formula

$$p_{ult} = 4R_d(s/a)^2[1 + 2(a/b)^2] \tag{3.3.4.4-1}$$

where R_d = design yield stress of a material according to 1.5.1.5, in MPa;
s = design plate thickness, in m;
a = length of the lesser side of a plate rest, in m;
b = length of the longer side of a plate rest, in m.

The ultimate load Q_{ult} on a stiffener restrained at its ends and loaded by an equidistributed load is determined by the formula

$$Q_{ult} = \frac{16W_u}{l}R_r\bar{Q} \tag{3.3.4.4-2}$$

where $\bar{Q} \leq 1$ = functional coefficient allowing for the effect of shear forces in sections of support;

$$\bar{Q} = \frac{1}{1+5,77[(W_{ult}/(F_w l^2))(l-0,5a)]^2};$$

F_w = design area of a stiffener web cross-section, in m²;
W_{ult} = ultimate section modulus with due regard for an effective flange, in m³;
a = spacing between stiffeners, in m;
l = span of a stiffener between its supports, in m.

For plates and stiffeners, the strength criteria indicated in 3.3.3 shall be met.

3.4 FOP REINFORCED AND STEEL CONCRETE STRUCTURES

3.4.1 General.

3.4.1.1 This Section of the Rules sets the basic requirements for design and construction of FOP hulls made wholly or in part (of a composite modification) of the following materials based on an ordinary concrete without prestressing:

reinforced concrete consisting of a concrete and metal bar reinforcement dispersedly arranged in it in accordance with a calculation and structural requirements;

steel concrete consisting of a concrete and metal plate reinforcement arranged on exterior surfaces of a structural element and attached to the concrete with adequate strength and rigidity in accordance with design and structural requirements;

composite reinforced concrete, i.e. the material occupying an intermediate position between the above two in which, additionally to a concrete, metal plate reinforcement is attached to one or the both exterior surfaces of a structural element in order to improve tightness and to increase bearing capacity of the last.

Hereinafter, steel concrete structures with exterior plate reinforcement and composite structures are called in the MODU/FOP Rules as steel concrete structures and appropriate refinements are made where necessary.

3.4.1.2 Design of prestressed reinforced and steel concrete structures of FOP hulls may be executed according to specialized regulatory documents approved by the Register.

3.4.1.3 In design of reinforced concrete, steel concrete and composite structures of FOP hulls, the provisions of the Rules for the Hull Construction of Sea-Going Ships and Floating Structures Using Reinforced Concrete may be used where applicable.

3.4.2 Loads.

3.4.2.1 The design values of loads on reinforced and steel concrete structures of FOP hulls due to various types of effects at their potential combinations are determined in accordance with the provisions of 2.3, 3.3.1 and 3.3.2.

3.4.2.2 The elements of massive steel concrete structures whose exterior plate reinforcement acts as forms, and also of precast-cast-in-situ reinforced concrete structures shall be designed for two stages of structure functioning:

prior to reaching the preset strength of a freshly laid concrete under its gravity and other loads relevant for this stage of structure construction;

after reaching the preset strength of concrete relevant for operational loads.

3.4.3 Key design requirements.

3.4.3.1 Reinforced and steel concrete structures shall meet the requirements of the calculation for bearing capacity (limit states of the 1st group) and fitness for normal operation (limit states of the 2nd group).

When the requirements of calculations for limit states are fulfilled, it is practically to be excluded:

for limit states of the 1st group:

brittle and ductile failures, loss of form buckling strength, fatigue failure (calculation for structures endurance under repeated loads), etc.;

for limit states of the 2nd group:

cracking in the concrete of crack-resistant structures, excessive opening of cracks in the concrete of structures for which cracking is allowable under operational conditions, excessive displacements, etc.

3.4.3.2 Reinforced and steel concrete structures shall be designed so that the general safety requirement stated in 2.4.1.1 may be fulfilled during the FOP entire life cycle. In this case, safety factors η shall be taken according to Table 2.4.2.5 as for the strength criterion given in Formula (2.4.2.3.1).

3.4.4 Materials.

3.4.4.1 Concrete and its components.

3.4.4.1.1 The concrete of reinforced and steel concrete structures of FOP hulls shall meet the requirements of national standards, the Rules for the Hull Construction of Sea-Going Ships and Floating Structures Using Reinforced Concrete and the requirements of this Section of the MODU/FOP Rules.

3.4.4.1.2 For reinforced and steel concrete structures, structural concretes shall be used:

normal-weight concrete, air-hardened or heat- treated at an atmospheric pressure, having an average density over 2300 up to 2500 kg/m³ inclusive;

fine-grained concrete (Abram's fineness of sand is over 2,0), air-hardened or heat-treated at an atmospheric pressure.

It is allowed to use:

light-weight dense and fine aggregate concrete having an average density over 1800 kg/m³.

3.4.4.1.3 In design of concrete compositions and procedures of concrete making and placement the following peculiarities shall be taken into account:

complexity of the configuration of the structure volume to be filled;

work performance under conditions of a Northern climatic zone;

concreting afloat;

concrete placement with concrete pumps;

concrete placement without vibration effects;

improved requirements for density, freezing resistance and water tightness in the zones of ice and wave load effects.

3.4.4.1.4 For hull structures with high requirements for strength, water tightness and freezing resistance, for instance, for exterior structures located within an alternating waterline, it is necessary to provide the use of surfactant admixtures and micro fillers. The optimum content of admixtures and fillers shall be determined experimentally while selecting concrete compositions.

3.4.4.1.5 In conformity with the type, purpose and operational conditions the concretes of the following classes and brands shall be used for special and main reinforced and steel concrete structures of FOP hulls:

.1 compressive strength classes meeting the value of guaranteed strength, in MPa, with the probability of exceedance 0,95, normal-weight concrete: B30, B35, B40, B45, B50, B55 and B60;

.2 freezing resistance brands: F100, F150, F200, F300, F400, F500 and F600 specified according to the data in Table 3.4.4.1.5.2;

Table 3.4.4.1.5.2

Operational conditions	Freezing resistance brand of a concrete at the number of repeated cycles of freezing and thawing per year				
	up to 50 ind.	over 50 up to 75	over 75 up to 100	over 100 up to 150	over 150 up to 200
Moderate	F50	F100	F150	F200	F300
Severe	F100	F150	F200	F300	F400
Very severe	F200	F300	F400	F500	F600

Notes: 1. Operational conditions feature the monthly average air temperature of the coldest month: moderate — over – 10 °C ; severe — from – 10 °C to – 20 °C; very severe — below – 20 °C.
2. For exterior structures under operational conditions at the number of repeated cycles of freezing and thawing in winter over 200 (the monthly average air temperature of the coldest month is below – 30 °C, salt content is from 20 g to 36 g per 1 l of water) the freezing resistance brand of a concrete is specially to be substantiated and specified in each particular case on the basis of the analysis of specific operational conditions, and agreed with the Register.

.3 watertightness brands for water-contacted reinforced concrete structures: W6, W8, W10, W12 and above specified in conformity with the head gradient determined as the ratio of a maximum head to the structure thickness (in meters), and with the water temperature, in accordance with Table 3.4.4.1.5.3.

Table 3.4.4.1.5.3

Water temperature, °C	Watertightness brand of concrete at the head gradient		
	over 5 up to 10	over 10 up to 20	over 20 up to 30 incl.
Up to 10 inclusive	W4	W6	W8
Over 10 up to 30 incl.	W6	W8	W10
Over 30	W8	W10	W12
Notes: 1. For structure with the head gradient over 30 the watertightness brand of concrete shall be W16 and above. 2. For exterior structures exposed to seawater and its splashes, and also being in contact with ice formations and a seabed soil the watertightness brand of concrete shall be not below W8.			

3.4.4.1.6 For ancillary hull structures it is allowed to use light-weight concretes of compressive strength classes: B30, B35 and B40.

3.4.4.1.7 For FOP massive hull structures, for instance, steel concrete floor structures having thickness over 1 m, provided that the concrete is largely used as a solid ballast and involved in carrying only local loads, it is allowed to use normal-weight concretes of lower compressive strength classes: B20 and B25.

3.4.4.1.8 In design of FOP, strength classes on the basis of tension may be established if specially substantiated.

3.4.4.1.9 It is not allowed to use a fine-grained concrete without an experimental justification for structures subjected to a many times repeated loading.

3.4.4.1.10 In design of reinforced and steel concrete structures the compressive strength class of concrete is established at 28 days. In all cases, external forces and other effects on the concrete are allowed only when they reach at least 70 % of the strength appropriate for the strength class adopted.

3.4.4.1.11 In order to grout joints and element assemblies of precast structures the concretes of strength classes shall be used, and freezing resistance and water tightness brands, which are not interior to those, adopted for abutting elements.

3.4.4.1.12 In design of FOP the values of characteristic strength of concrete in axial compression (prism strength) R_{bn} and in axial tension R_{btn}, the values of design strength of concrete in compression and tension for the 1st group limit states (as to a bearing capacity) R_b and R_{bt}, and for the 2nd group limit states (as to serviceability) R_{b2} and R_{bt2}, determined by the division of the values of characteristic strength by the relevant concrete reliability coefficients in compression and tension, and initial moduli of elasticity for an air-hardened concrete in compression and tension depending on the compressive strength classes shall be adopted according to Table 3.4.4.1.12.

Table 3.4.4.1.12

Concrete Design class	Characteristic strength of concrete, design strength of concrete for the 2nd group limit states, MPa		Design strength of concrete for the 1st group limit states, MPa		Initial modulus of elasticity in concrete compression and tension $E_b \cdot 10^{-3}$, MPa
	Axial compression (prism strength) $R_{bn} = R_{b2}$	Axial tension $R_{btn} = R_{bt2}$	Axial compression R_b	Axial tension R_{bt}	
Normal-weight concrete					
B20	15,0	1,40	11,5	0,90	27,0
B30	22,0	1,80	17,0	1,20	32,5

Concrete Design class	Characteristic strength of concrete, design strength of concrete for the 2nd group limit states, MPa		Design strength of concrete for the 1st group limit states, MPa		Initial modulus of elasticity in concrete compression and tension $E_b \cdot 10^{-3}$, MPa
	Axial compression (prism strength) $R_{bn} = R_{b2}$	Axial tension $R_{btn} = R_{bt2}$	Axial compression R_b	Axial tension R_{bt}	
B40	29,0	2,10	22,0	1,40	36,0
B50	36,0	2,30	27,5	1,55	39,0
B60	43,0	2,50	33,0	1,65	40,0
Light-weight and fine-grained concrete					
B30	22,0	1,80	17,0	1,20	26,0/19,5[1]
B40	29,0	2,10	22,0	1.40	28,5/21,0[1]

[1] In numerator — a fine-grained concrete, and in denominator a light-weight concrete.
Note. When intermediate compressive strength classes of concrete are used, the values of parameters are determined by linear interpolation.

3.4.4.1.13 The values of design strength of concrete for the 1st group limit states R_b and R_{bt} shall be entered into a calculation with due regard for the coefficients of operational conditions γ_{bi}, whose values are given in Table 3.4.4.1.13-1. Specifying the values of design strength of concrete for the 2nd group limit states R_{b2} and R_{bt2} only one coefficient of operational conditions of concrete γ_{b1}, according to Table 3.4.4.1.13-2, is taken into account.

Table 3.4.4.1.13-1

Nos.	Factors leading to introduction of the coefficient of operational conditions for concrete	Coefficient of operational conditions for concrete	
		Symbol	Value
1	Many times repeated load	γ_{b1}	Refer to Table 3.4.4.1.13-2
2	Concreting in vertical position with the height of a concreting layer over 1,5 m	γ_{b2}	0,85
3	Repeated freezing and thawing	γ_{b3}	
	a) in a water saturation state at the design winter temperature of outdoor air:		
	below – 40 °C		0,70
	below – 20 °C down to – 40 °C inclusive		0,85
	below – 5 °C down to – 20 °C inclusive		0,90
	– 5 °C and over		0,95
	b) in conditions of random water saturation:		
	below – 40 °C		0,90
	– 40 °C and over		1,00
4	Concrete in reinforced concrete structures	γ_{b4}	1,1

Notes: 1. Coefficients of operational conditions for items 1,3 and 4 shall be taken into account in calculating the values of design strength R_b, and R_{bt}, and for item 2, in the calculation of R_b only.
2. For structures under a many times repeated loading the coefficient γ_{b1} is considered only in the calculation for endurance and when cracks in concrete are formed.
3. When the freezing resistance brand of the concrete, as compared with that required according to Table 3.4.4.1.5.2, is exceeded, the coefficient γ_{b3} may be increased by 0,05 for each step of excess, but it may not be over 1,0.
4. Coefficients of operational conditions of the concrete are introduced independently of one another, but their product shall be not less than 0,45.

Table 3.4.4.1.13-2

Humidity state of concrete	Coefficient of operational conditions of concrete at many times repeated loads and a coefficient of cycle asymmetry ρ_b equal to:						
	0 – 0,1	0,2	0,3	0,4	0,5	0,6	0,7
Natural humidity	0,75	0,8	0,85	0,9	0,95	1,00	1,00
Water saturation	0,50	0,60	0,70	0,80	0,90	0,95	1,0
Note. A coefficient of cycle asymmetry ρ_b is equal to the ratio of the least stress in concrete to the largest one during the cycle of loading change.							

3.4.4.1.14 For concretes subjected to repeated freezing and thawing the values of an initial modulus of elasticity given in Table 3.4.4.1.12 shall be multiplied by the coefficient of operational conditions γ_{b3} assumed according to Table 3.4.4.1.13-1.

3.4.4.1.15 When calculating reinforced and steel concrete structures for endurance, nonelastic deformations of concrete in the compressed zone shall be taken into account by the reduction of a modulus of elasticity adopting a steel to concrete reduction coefficient according to Table 3.4.4.1.15.

Table 3.4.4.1.15

Compressive strength class of concrete	B20	B30	B40	B50	B60
Reduction coefficient v'	23	18	10	8	5

3.4.4.1.16 The coefficient of linear thermal deformation of concrete d_{bt} shall be assumed in calculations equal to $1/10^{-5}$ °C^{-1}.

3.4.4.1.17 The initial coefficient of lateral deformation of concrete (Poisson's ratio) μ is assumed equal to 0,2.

3.4.4.1.18 The shear modulus of concrete is assumed equal to 0,4 of appropriate values for initial moduli of elasticity of concrete specified in Table 3.4.4.1.12.

3.4.4.2 Reinforcement.

3.4.4.2.1 As untensioned bar and wire reinforcement shall be used:

.1 bar reinforcement of Class A-III — for longitudinal and transverse reinforcement;

.2 bar reinforcement of Class A-II — for transverse reinforcement and for longitudinal one where the other types of reinforcement can not be used due to operational conditions;

.3 bar reinforcement of Class A-I and reinforcing wire of Class B_p-I — for longitudinal and transverse reinforcement.

Reinforcement of Class A-III, 10 — 40 mm in diameter, in exterior structures at the temperature not lower than – 40 °C may be used only for tied cages and fabrics (without welding).

3.4.4.2.2 For FOP hull structures operating under severe and very severe climatic conditions (refer to 3.4.4.1.5) it is not allowed to use bar reinforcement over 16 mm in diameter made of semikilled steel.

3.4.4.2.3 The characteristic strength of bar and wire reinforcement for the classes specified in 3.4.4.2.1 and the design strength of reinforcement for limit states of the 1st and 2nd groups (refer to 3.4.3.1) depending on the loading nature, and also moduli of elasticity and relative elongations are given in Table 3.4.4.2.3.

Table 3.4.4.2.3

Reinforcement class	Diameter, mm	Characteristic tensile strength (yield stress), design tensile strength for the 2nd group limit states, MPa, $R_{Sn} = R_{S2}$	Design strength for the 1st group limit states, MPa			Modulus of elasticity E, MPa	Relative elongation, %	Bending test in cold state (c – mandrel thickness, d– bar diameter)
			Tension		Compression			
			Longitudinal R_S	Transverse (clamps and web bars) R_{SW}	R_{SC}			
A-I	6÷40	235	225	175	225	$2.05/10^5$	≥25	180° $c = 0{,}5d$
A-II	6÷40	295	280	225	280	$2.05/10^5$	≥19	180° $c = 3d$
A-III	6÷40	390	355	285[1]	355	$2.00/10^5$	≥14	90° $c = 3d$
B_p-I	3	410	375	270	375	$1{,}70/10^5$	–	–
	4	405	365	265	365	$1{,}70/10^5$	–	–
	5	395	360	260	360	$1{,}70/10^5$	–	–

[1] In welded cages, transverse bars (clamps) in Class A-III reinforcement whose diameter is less than a third of the diameter of longitudinal bars, the values are taken equal to 255 MPa.

3.4.4.2.4 The characteristic and design strength of steel plate reinforcement for composite reinforced concrete structures and steel concrete structures are determined in accordance with 1.5.1.5.

3.4.4.2.5 The design strength of reinforcement for reinforced concrete structures, steel concrete and composite structures compression as to limit states of the 1st group shall be assumed in all cases not more than $R_{SC} = 400$ MPa.

3.4.4.2.6 The values of design strength of reinforcement R_S, R_{SC} and R_{SW} for limit states of the 1st group are used in a calculation with due regard for coefficients of operational conditions γ_S and γ_{Si} which values are given in Tables 3.4.4.2.6-1 and 3.4.4.2.6-2.

Table 3.4.4.2.6-1

Factor leading to introduction of the coefficient of operational conditions for bar reinforcement	Coefficients of operational conditions for bar reinforcement γ_S
Reinforced concrete members	1,05
Composite reinforced concrete members	1,0
Many times repeated loads at a coefficient of cycle asymmetry ρ_S:	
-1,0 – 0	0,6[1]
0 – 0,4	0,7[1]
0,4 – 0,8	0,9[1]
0,8 – 1,0	1,0[1]

[1] Where welded joints of reinforcement of the following types are available:
contact butt joint without mechanical dressing;
butt joint made in a weld pool on a steel backing; the weld pool length shall be three or more diameters of the least abutting bar;
twin-symmetrical strap butt joint.
Note. The coefficient of cycle asymmetry ρ_S is equal to the ratio of the least stress in reinforcement to the largest one during the cycle of loading change.

Tables 3.4.4.2.6-2

Factor leading to introduction of the coefficient of operational conditions for plate reinforcement	Coefficients of operational conditions for plate reinforcement γ_{Si}
Plate reinforcement without special treatment of concrete-contacted surface and without anchors	0,5
Plate reinforcement with the ribbed surface of contact with concrete, without anchors	0,7
Plate reinforcement anchored in concrete according to the calculation in compliance with the requirements of 3.4.6.5	0,9

3.4.5 Requirements for design of FOP hulls of composite concrete-based materials.

3.4.5.1 Cross-section dimensions of reinforced, composite reinforced and steel concrete members of FOP hull structures shall be determined by calculation reasoning from the conditions of strength, cracking resistance or restrictions on cracks opening.

In all cases therewith the total sectional area of longitudinal principal bar and plate tension reinforcement shall be not less than 0,4 % of a concrete section area.

3.4.5.2 The thickness of a protective layer for the concrete of reinforced concrete structures shall be adopted not less than:

.1 on the surface exposed to water effects:

50 mm — for principal reinforcement;

30 mm — for distribution reinforcement and clamps;

.2 on surfaces not exposed to sea water effects:

30 mm or at least a bar diameter — for principal reinforcement;

20 mm or at least a bar diameter — for distribution reinforcement and clamps.

3.4.5.3 A reinforcement diameter for FOP sides, decks and bottom shall be at least 12 mm, and inside internal wall members, at least 8 mm.

3.4.5.4 The minimum thickness of plate reinforcement shall be 10 mm for composite reinforced concrete structures, and 15 mm for steel concrete structures.

3.4.5.5 The thickness of reinforced concrete grillages of a bottom and sides within the range of an alternating water level, direct ice effects and in an underwater part is recommended to be at least:

0,6— 0,8 m, for hulls in the form of a cylindrical or conic shell, and for composite and cast-in-situ hulls;

0,4 — 0,5 m — for hulls of cellular structure.

3.4.5.6 Bar reinforcement for reinforced and composite reinforced structures shall be designed in the form of reinforced trusses, welded cages and fabrics. The types of reinforced structures shall provide for the possibility of mechanized concrete supply, its thorough handling or self-consolidation.

3.4.5.7 All the bearing parts of a FOP hull shall be free of abrupt changes of cross-sections, and of curvatures. The cross-section of reinforcement is reasonably to change through reduction of the bars diameter with no change of their quantity.

It is allowed to connect in one section of a structure not more than 30 % of bars in a tension zone, and not more than 50 per cent, in a compression zone.

3.4.5.8 In design of reinforcement the measures ensuring the reliable anchorage of its ends shall be provided. The anchorage of reinforcement shall be effected by welding of a curtailed bar to transverse distribution reinforcement or by bar lengthening from a place where it is needed by a calculation for at least 30 diameters for tension deformed reinforcement, and for at least 20 diameters for compression one.

3.4.5.9 Angle joints of flat members of FOP hulls shall be designed to ensure equal or greater strength of joined members. In reinforced and composite reinforced concrete structures, opposing reinforcement in side-deck angle joints shall be welded or extended from one slab into another for at least 15 reinforcement diameters. In side-bottom angle joints provision shall be made for sections thickening by at least 1,5 times or bevelling of comers of inner or inner and outer assembly surfaces with installation of additional reinforcement along bevel surfaces.

On T and cross joints, opposing reinforcement shall be welded or extended from one slab into another in sections of at least 15 reinforcement diameters long over their outline,

3.4.5.10 In steel concrete structures of FOP hull grillages it is recommended to provide for transverse members of sheet steel (diaphragms) between exterior (outer and inner) steel plates, in angle bottom-side and side-deck joints it shall be provided for bevelling of inner assembly surfaces and for gusset plates (brackets), their welding to inner surfaces of joined flat structures. The spacing of gusset plates shall be not less than the thickness of the thinner among the members. The gusset plate metal thickness shall be not more than that of a lining.

3.4.5.11 In design of composite reinforced and steel concrete members of FOP hulls it shall be provided for measures ensuring the joint operation of plate reinforcement in concrete according to the calculation in compliance with the requirements of 3.4.6.5; the use of exterior plate reinforcement with a profiled surface is also recommended.

3.4.5.12 In selection of the rest and anchor design one shall proceed from simplicity and reliability of their securing to plate reinforcement by means of continuous or intermittent welding.

It is allowed to use various structural types of members and anchors, namely: rigid and flexible rests, various anchors including of reinforcement bars, as well as combined members like the loop anchors of reinforcement bars welded to rigid rests, etc.

N o t e . The rests or anchor members of the cuts of rolling sections with their axis parallel to the exterior steel plate plane and perpendicular to a shear force are considered as rigid if stiffeners, brackets, etc. are available, and as flexible if the last are lacking.

3.4.5.13 In sections of contact surfaces remoted from supports it is preferable to use flexible or combined rests and anchors which, to a lesser extent, impact the process of cracking in concrete.

3.4.5.14 Outer plate reinforcement shall be securely anchored in concrete to prevent the buckling (buckling between anchors) under compression stresses in bending of a steel concrete grillage. In order to ensure reinforcement buckling strength up to the yield stress, the anchors spacing shall be determined in accordance with the standards for steel structures design and assumed to be not more than 25δ thicknesses for normal strength steel and not more than 20δ thicknesses for higher strength steel.

3.4.5.15 The attachment of rests, anchors, etc. to a steel part shall be computed in accordance with the instructions on the calculation of welded steel structure joints of the Rules for the Classification. In this case, the values of loads determined according to 3.4.6.5 of this Part are used in the calculation.

3.4.5.16 In design of rests and anchors the following conditions shall be fulfilled:

a clear distance between rigid rests shall be at least 3,5-fold height of the design area of concrete bearing by the rest;

the design of rigid rests shall ensure uniform concrete deformations across a bearing area, i.e. there shall not be any comers or other convex surfaces on crushing surfaces, which may cause concrete cleavage. When the surface transferring pressure of the rest onto concrete is convex, the zone of local concrete compression by the rest shall be reinforced.

3.4.6 Calculation for strength and endurance.

3.4.6.1 Basic design provisions.

3.4.6.1.1 In calculation of FOP hull the internal forces due to general and local loads, as well as due to forced movements (as a consequence of the change of a temperature, concrete moisture, etc.) shall be determined following the instructions of 2.5.1 and 2.5.2 with due regard for inelastic behaviour of loaded structures, caused by concrete cracking and creep and by a non-linear relationship between stresses and material deformations, according to methods approved by the Register.

In cases when the calculation methods with due regard for inelastic behaviour are not developed or the calculation is carried out at the intermediate stage of platform design, forces in cross-sections shall be determined assuming an elastic operation of structures. The height of a compressive zone of the concrete in them therewith is determined basing on a plane-sections hypothesis. In non-crack-resistant structures the operation of tensile concrete is ignored and the form of a concrete stress diagram within the compression zone of sections is assumed as triangular.

3.4.6.1.2 Calculations of the stress state of members in bending basing on the preconditions specified in 3.4.6.1.1 are applicable when the ratios of a working (effective) height of a member to the distance between the points of a zero bending moment are less than 1/2 or the ratio of a working height to a span is less than 1/3. If these ratios exceed the above values, the members shall be calculated as deep beams.

3.4.6.1.3 The geometric characteristics of cross- sections of members are determined for sections reduced to one material. The areas of design cross-sections reduced to concrete or steel are determined by the formulae:

$$F_{bl} = \sum(F_b + F_s E_s/E_b); \tag{3.4.6.1.3-1}$$

$$F_{sl} = \sum(F_s + F_b E_b/E_s) \tag{3.4.6.1.3-2}$$

where F_b and F_s = cross-sectional areas of concrete and longitudinal reinforcement of the member in question, respectively;
E_b and E_s = initial moduli of elasticity for concrete and steel.

3.4.6.1.4 If the cross-section of a structure under consideration includes a compression steel member, that may lose its buckling strength, the section area relevant to it shall be included into a reduced area with a reduction factor (refer to 2.5.2).

3.4.6.1.5 In determination of the main tension, compression and shear stresses in concrete, the structure sections reduced to the concrete assuming the elastic operation of materials with due regard for the concrete in a tensile zone are taken into account.

3.4.6.1.6 In analytical assessments of the deflected state of reinforced and steel concrete grillages, for determination of internal forces in grillage sections it is recommended to use the design diagram of a plate with due regard to the provisions of 3.4.6.1.2 — 3.4.6.1.5.

3.4.6.1.7 Calculations for members strength under the action of a bending moment and an axial force shall be performed for sections normal to a longitudinal axis and also inclined to it along the most critical directions.

3.4.6.1.8 Ultimate resistance forces in bending in the section normal to the longitudinal axis of a member shall be determined following the preconditions:

concrete resistance to tension is assumed equal to zero;

concrete stresses in a compression zone are adopted equal to a design strength R_b and the form of compression stress diagram is taken as rectangular;

tension and compression stresses in reinforcement are accepted as not exceeding the design ones.

3.4.6.1.9 In calculation of reinforced concrete and steel concrete members for combined torsion and bending the following condition shall be observed:

$$M_T \le 0{,}1 R_b b^2 h \qquad (3.4.6.1.9\text{-}1)$$

where M_T = torque moment;
b, h = lesser and larger dimensions of member sides, respectively.

In this case the value of R_b for the concretes whose classes are higher than Class B39 is taken as for the Class B30 concrete.

The calculation of three-dimensional reinforced and steel concrete structures for torsion at intermediate stages of platform's design is allowed to perform assuming an elastic operation of a structure with regard to the tensile concrete. The maximum shear stresses in concrete therewith shall meet the condition

$$\tau_{max} \le 1{,}86 R_{btn} \qquad (3.4.6.1.9\text{-}2)$$

where R_{btn} = characteristic strength of concrete to axial tension.

3.4.6.1.10 When a sizable concentrated loading is applied to the limited area of a member, the check of its local strength for bearing, forcing through, breaking away, etc. shall be performed.

3.4.6.2 Calculation of members strength in sections normal to the longitudinal axis of the member.

3.4.6.2.1 The calculation of members strength in sections normal to the longitudinal axis of the member shall be conducted in accordance with 3.4.3.2 and 3.4.6.1.8 observing the condition

$$\xi = x/h_0 < \xi_R \qquad (3.4.6.2.1\text{-}1)$$

where ξ, x = relative and true height of the compressed zone of concrete;
h_0 = working section height equal to the distance from the resultant of forces in tensile reinforcement the compression face of a concrete section;
ξ_R = boundary height of the compression zone to be accepted according to Table 3.4.6.2.1.

Sections with double bars (in tensile and compression zones in bending) shall meet the condition

$$M < R_b S_0 \qquad (3.4.6.2.1\text{-}2)$$

where M = bending moment acting in a section;
R_b = design strength of concrete in compression for the 1st group limit states;
S_0 = static moment of the entire cross-sectional area of concrete (less a protective layer in a tensile zone) about the centre of gravity of a section of tensile reinforcement.

Table 3.4.6.2.1

Reinforcement class	Boundary values of ξ_R for concrete class	
	B20, B25, B30	B35 and over
A-I	0,65	0,60
A-II,B_p-I, A-III	0,60	0,50

3.4.6.2.2 The calculation of sections of flexural concrete members of any symmetric form shall be performed by the formulae:

$$\Phi = M;\ R = \gamma_b R_b S_b + \gamma_s R_{sc} S_s; \quad (3.4.6.2.2\text{-}1)$$

$$\gamma_s R_s F_s - \gamma_s R_{sc} F_s' = \gamma_b R_b F_b. \quad (3.4.6.2.2\text{-}2)$$

For a rectangular symmetrical section (refer to Fig. 3.4.6.2.2-1):

$$S_b = bx(h_0 - 0{,}5x);\ S_s = F_s'(h_0 - a');\ F_b = bx.$$

If the height of a compression zone determined without regard for compression reinforcement is less than the double thickness of a protective layer, i.e. less than $2a'$ (refer to Fig. 3.4.6.2.2-1), the compression reinforcement may be ignored in calculation.

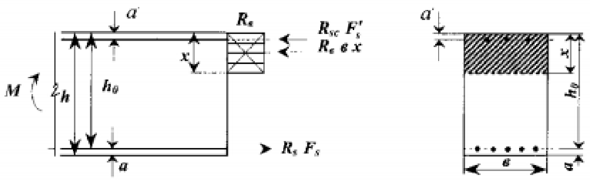

Fig. 3.4.6.2.2-1
Diagram of forces on the section normal to the longitudinal axis of a flexural reinforced concrete member in calculation of its strength

The calculation of flexural composite reinforced concrete members shall be performed without considering the compliance of the connective seam of plate reinforcement with the concrete according to the formulae (refer to Fig. 3.4.6.2.2-2):

$$\Phi = M; R = \gamma_b R_b bx(h_0 - 0{,}5x) + \gamma_s R_{sc} F_s'(h_0 - a') + \gamma_{si} R_{si} F_{si}'(h_0 + 0{,}5d_{si}); \quad (3.4.6.2.2\text{-}3)$$

$$\gamma_s R_s F_s - \gamma_s R_{sc} F_s' + \gamma_{si} R_{si} F_{si} - \gamma_{si} R_{si} F_{si}' = \gamma_b R_b bx. \quad (3.4.6.2.2\text{-}4)$$

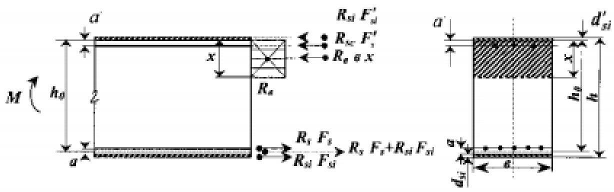

Fig. 3.4.6.2.2-2
Diagram of forces on the section normal to the longitudinal axis of a flexural composite reinforced concrete member in calculation of its strength

Where the cross-sectional area of compression reinforcement in the section of a composite reinforced concrete member is equal or more than the cross-sectional area of tensile reinforcement the design bending strength of the section is determined by the formula

$$R = (\gamma_{si} R_{si} F_{si} + \gamma_s R_s F_s)(h_0 + 0{,}5d_{si}') \quad (3.4.6.2.2\text{-}5)$$

where R = design bending strength of section;

S_b and S_s	=	static moments of the section area of a compression concrete zone, and of the section area of compression reinforcement respectively, about the centre of gravity of tensile reinforcement;
F_s, F'_s, F_{si} and F'_{si}	=	cross-sectional areas of tensile and compression bar and plate reinforcement, respectively;
R_b, R_s, R_{sc} and R_{si}	=	design strength of concrete, tensile and compression bar and plate reinforcement, respectively (refer to 3.4.4);
γ_b, γ_s, γ_{si}	=	coefficients of operational conditions of concrete, bar and plate reinforcement, respectively, accepted according to Tables 3.4.4.1.13-1, 3.4.4.2.6-1 and 3.4.4.2.6-2.

The calculation of steel concrete members shall be performed according to Formulae (3.4.6.2.2-3) — (3.4.6.2.2-5) assuming $F_s = F'_s = 0$.

3.4.6.2.3 The calculation of eccentrically compression and tensile members with a rectangular form cross-section, and also the calculation of members having T and double-T cross-sections in bending, eccentrical compression and tension are recommended to perform according to the formulae of the Rules for the Construction of Hulls of Sea-Going Ships and Floating Facilities Using Reinforced Concrete on the basis of design characteristics of materials and design coefficients accepted in the MODU/FOP Rules with regard to the provisions of 3.4.6.2.1 and 3.4.6.2.2.

3.4.6.3 Calculation of member strength in sections inclined to the longitudinal axis of the member under the action of a transverse force.

3.4.6.3.1 When calculating the member section strength in bending, eccentrical compression and tension for the action of a transverse force the following condition shall be observed:

$$Q \le 0{,}25\eta\gamma_{b4}R_b bh_0 \qquad (3.4.6.3.1)$$

where b = minimum sectional width of a member;
η = safety factor (refer to 2.4.1.1 and 3.4.3.2).

3.4.6.3.2 The calculation of member sections may be ignored if the following condition is fulfilled:

$$Q \le \eta\delta(0{,}5 + 2\xi)R_{bt}bh_0 \qquad (3.4.6.3.2)$$

where $\delta = 2/(1 + M/Qh_0)$, but not more than 1,5 and not less than 0,5;
$M,\ Q$ = forces in a normal section through the end of a sloping section in the compression zone;
ξ = relative height of the compression zone being determined:

for flexural members —

$\xi = \mu R_s/R_b$;

for eccentrically compression and tensile members with a large eccentricity when $S_b \le 0{,}8S_0$, —

$\xi = \mu R_s/R_b \pm N/bh_0R_b$;

μ = coefficient of reinforcing determined as the ratio of the section area of a longitudinal reinforcement in the tensile zone of the section to the cross-sectional area bh_0 of the member.

N o t e . The signs "plus" and "minus" shall be used for eccentrically compression and tensile members, respectively.

3.4.6.3.3 For eccentrically tensioned members with a small eccentricity when $S_b > 0{,}8S_0$ the strength calculation of the sections inclined to the longitudinal axis of the member is compulsory in all the cases when a transverse force acts.

3.4.6.3.4 The calculation of transverse reinforcement in the sloping section of a reinforced, composite reinforced and steel concrete member (refer to Fig. 3.4.6.3.4) shall be performed by the following formulae:

for flexural, eccentrically compressed and tensioned with a large eccentricity members —

$$\Phi = Q,\ R = \sum \gamma_s R_{sw} F_{sw} + \sum \gamma_{si} R_{si} F_{swi} + \gamma_b \delta (0{,}5 + 2\xi) R_{bt} b h_0; \qquad (3.4.6.3.4\text{-}1)$$

for eccentrically tensioned members with a small eccentricity —

$$\Phi = Q,\ R = \sum \gamma_s R_{sw} F_{sw} + \sum \gamma_{si} R_{si} F_{swi}, \qquad (3.4.6.3.4\text{-}2)$$

where $\sum \gamma_s R_{sw} F_{sw}$ and $\sum \gamma_{si} R_{si} F_{swi}$ = sums of forces in all transverse bars (clamps) and transverse plate members in a sloping section;
F_{sw}, F_{swi} = cross-sectional areas of transverse bar and plate reinforcement;
ξ = relative height of a sectional compression zone according to 3.4.6.3.2;
$\gamma_b, \gamma_s, \gamma_{si}$ = coefficients of operational conditions of concrete and reinforcement taken according to Tables 3.4.4.1.13-1, 3.4.4.2.6-1 and 3.4.4.2.6-2.

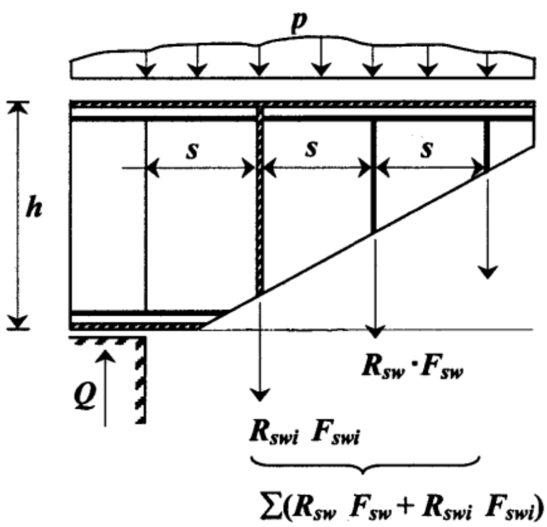

Fig. 3.4.6.3.4
Diagram of forces in the section inclined to the longitudinal axis of a composite reinforced concrete member in calculation of its strength for the action of a transverse force

3.4.6.3.5 In calculation of the plate members of reinforced concrete structures, having the height-design length ratio $h_0/l \le 1/3$, for the action of a transverse force in the plate plane conditions (3.4.6.1.9-1) and (3.4.6.1.9-2) shall be met. In this case the maximum shear stresses are determined in accordance with the provisions of 3.4.6.1.5. The strength of transverse reinforcement (clamps) and longitudinal reinforcement distributed over the section height shall be checked in this case for the action of main tensile stresses in way of the neutral axis of the section.

3.4.6.3.6 The distance between transverse members (refer to Fig. 3.4.6.3.4), in case of inclined bars, between the end of the preceding and the beginning of the following bend which is the nearest to a support, shall be not less than the value of s_{max} determined by the formula

$$s_{max} = \eta(0{,}5 + 2\xi)\gamma_b R_{bt} b h_0^2 / Q_1 \quad (3.4.6.3.6)$$

where $Q_1 = R$ being determined by Formula (3.4.6.3.4-1).

3.4.6.4 Calculation of reinforced and steel concrete structures for endurance.

3.4.6.4.1 The calculations of structure members for endurance shall be performed in all the cases when the number of loading change cycles is equal to $2/10^6$ and more over the entire design period of the FOP operation.

3.4.6.4.2 The calculation of structure members for endurance shall be performed, assuming the elastic operation of materials, by way of comparison of stresses in extreme fibres of concrete and of stresses in tensile bar and plate reinforcement with the design strength of materials adopted with the relevant coefficients of operational conditions (refer to Tables 3.4.4.1.13-1, 3.4.4.2.6-1 and 3.4.4.2.6-2).

3.4.6.4.3 In crack-resistant members, stresses in extreme concrete fibres and in bar and plate reinforcement are determined for reduced sections (refer to 3.4.6.1.3) with regard to the operation of entire section concrete and to the provisions of 3.4.4.1.15.

In crack-nonresistant members the geometric characteristics of sections are determined for reduced sections without regard to the concrete of a tensile zone and with regard to the provisions of 3.4.4.1.15.

The calculation of compression reinforcement for endurance is not conducted.

3.4.6.4.4 In the members of reinforced and steel concrete structures when calculated for endurance of sloping sections, the main tensile stresses are carried by the concrete if their values do not exceed R_{bt}, the resultant of the main tensile stresses shall be fully transmitted to the transverse bar and plate reinforcement. The stresses in the reinforcement therewith shall not exceed the design values of R_{sw} and R_{swi} irrespectively.

3.4.6.5 Calculation of strength of sheet steel-to- concrete joint in steel concrete structures.

3.4.6.5.1 The structure of a sheet steel-to-concrete joint across their contact surfaces shall be calculated for longitudinal shear forces arising in the member during bending in the plane of bending for longitudinal shear forces due to temperature effects concrete shrinkage etc, for the forces in the transverse direction in the plane of bending during local forcing through the concrete under an external loading, etc.

3.4.6.5.2 The strength of connection or linkage of plate reinforcement with concrete is ensured if the following condition is observed:

$$T \le m T_{sup} n \quad (3.4.6.5.2)$$

where
T = total shear force acting in the contact surface within the steel concrete member part under consideration;
m = coefficient of rest nonuniformity operation with $m = 0{,}9$ — for jointly operating rests of a different design. $m = 1{,}0$ — for rests of the same design;
T_{sup} = shear force carried by one rest and determined according to 3.4.6.5.7 and 3.4.6.5.8;
n = number of rests within the member part under consideration.

3.4.6.5.3 The longitudinal shear force in bending of the steel concrete member acting on the rests and anchors in the contact of plate reinforcement with concrete at the layout spacing 'u' is determined by the formulae:

$$T = QS_{red}u/J_{red} \qquad (3.4.6.5.3\text{-}1)$$

or

$$T = N_{p(i-1)} - N_{p(i)} \qquad (3.4.6.5.3\text{-}2)$$

where Q = shearing force acting on the member part under consideration between rests;
S_{red} = reduced static moment of the steel plate cross-section about the neutral axis of the design member section;
J_{red} = reduced moment of inertia of the steel plate cross-section;
$N_{p(i-1)}$ or $N_{p(i)}$ = longitudinal tensile forces in plate reinforcement in cross-sections at the boundaries of the part under consideration.

3.4.6.5.4 In design of longitudinal anchorage with respect to a transverse force it shall be ensured the rigidity and strength of a steel-to-concrete joint at support and end parts of a flexural member for which purpose the structures of a rigid rest type (refer to 3.4.5.12) shall be fitted in support sections. In continuous structures with transverse diaphragms in the planes of intermediate supports provision made for their strengthening with brackets is sufficient. At end parts of members the rigid rests are recommended to arrange outside support sides where practicable. The structural design of support structures of end parts shall provide for not only strength and rigidity of transverse diaphragms, but also to involve exterior steel plates of reinforcing in the operation in the support section.

3.4.6.5.5 The calculation of steel-to-concrete joint structures at the support parts of a member shall be performed for a total design longitudinal force in a plate steel in the plane of bending transmitted to the concrete, which is determined by the formula

$$N_p = R_{si}F_{si} \qquad (3.4.6.5.5)$$

where R_{si} = design strength of a plate steel material;
F_{si} = design area of a plate steel cross-section.

In addition, the structure of a steel-to-concrete integration on the support is checked for shear forces determined by Formula (3.4.6.5.3-1).

3.4.6.5.6 The calculation of steel-to-concrete joint structures shall be performed:

.1 with rigid rests — assuming the rectangular diagram of compression stresses transmitted to the concrete by the compression design surface of the rest;

.2 with flexible rests — reasoning from the conditions of concrete bearing under the rest with the rest operation in bending, according to 3.4.6.5.8;

.3 with sloping anchors — reasoning from the conditions of an anchor operation in the combination of tension and bending with concrete bearing.

3.4.6.5.7 The calculation of the structure of a joint on rigid rests shall be performed by the formulae:

for strength —

$$T_{sup} \le 1{,}6R_bF_{b,sm}; \qquad (3.4.6.5.7\text{-}1)$$

for endurance —

$$T_e \leq 1{,}5\gamma_{b1}R_bF_{b,sm} \quad (3.4.6.5.7\text{-}2)$$

where T_{sup}, T_e = shear forces carried by one rest in calculation for strength and endurance, respectively;
$F_{b,sm}$ = design area of a rest or anchor placed normally to a shear force.

3.4.6.5.8 The calculations of strength for the structures of plate steel-to-concrete joints on flexible rests and bar anchors shall be performed by the formulae:

for flexible rests in the form of rolled channels, I beams, angles (without stiffening ribs like brackets) —

$$T_{sup} \leq 0{,}55(t_{fr} + 0{,}5t_w)b_{dz}\sqrt{10R_b}, \text{ in kN}; \quad (3.4.6.5.8\text{-}1)$$

for flexible rests like round bars welded to plate reinforcement by its face end, at $2{,}5 \leq l/d \leq 4{,}2$ —

$$T_{sup} \leq 0{,}24ld\sqrt{10R_b}, \text{ in kN}; \quad (3.4.6.5.8\text{-}2)$$

for similar flexible rests in the form of round bars at $l/d > 4{,}2$ —

$$T_{sup} \leq d^2\sqrt{10R_b}, \text{ in kN}; \quad (3.4.6.5.8\text{-}3)$$

for flexible rests in the form of round bars the following condition is additionally to be fulfilled:

$$T_{sup} \leq 0{,}063d^2mR_\gamma, \text{ in kN}, \quad (3.4.6.5.8\text{-}4)$$

where t_{fr} = sum of the radius of curvature and of the largest thickness of a rolling section flange, in cm;
t_w = thickness of a rolling section web, in cm;
l = length of the round bar of a flexible rest, in cm;
d = diameter of the flexible rest or anchor bar, in cm;
b_{dz} = width of the area of concrete bearing by a rest, cm;
R_b = design compressive strength of concrete for the 1st group limit states;
R_γ = design strength of a steel structure material;
m = coefficient of operational conditions of a steel structure.

3.4.7 Calculation of reinforced and steel concrete structure members for cracking, crack opening and deformations.

3.4.7.1 The calculations of structure members for cracking and crack opening in concrete shall be performed:

in design of crack-resistant structures;

in design of structures with limited crack opening;

in identification of cracking zones to account for the reduction of rigidity characteristics of members in the calculations of redundant bar and massive structures.

The condition of cracking corresponds to an equality sign, and the condition of cracking resistance (prevention of cracks), to an inequality sign in the design formulae that correspond to the structure of the condition preventing the limit state 2.4.1.1 and 3.4.3.2:

.1 for centrally tensioned members, by the formula

$$\Phi = N, R = 1{,}5R_{bt2}F_{red} \qquad (3.4.7.1.1)$$

where F_{red} = reduced area of a member cross-section;

.2 for flexural members, by the formula

$$\Phi = M, R = 1{,}75R_{bt2}W_{red} \qquad (3.4.7.1.2)$$

where W_{red} = reduced modulus of section in bending for a tension side;

.3 for eccentrically compressed members:

$$\Phi = M/W_{red} - N/F_{red}, R = 1{,}75R_{bt2}, \qquad (3.4.7.1.3)$$

.4 for eccentrically tensioned members:

$$\Phi = M/W_{red} - N/F_{red}, R = 1{,}2R_{bt2}. \qquad (3.4.7.1.4)$$

3.4.7.2 The calculation for the formation of cracks inclined to the longitudinal axis of a member shall be performed by the formula

$$\Phi = \sigma_{mt}, R = 1{,}5R_{bt2} \qquad (3.4.7.2)$$

where σ_{mt} = main tensile stresses in concrete determined according to the provisions of 3.4.6.1.5.

3.4.7.3 The calculations for cracking under repeated loads shall be performed reasoning from the condition

$$\Phi = \sigma_{bt}, R = \gamma_{b1}R_{bt2} \qquad (3.4.7.3)$$

where σ_{bt} = maximum normal and main tensile stresses in concrete;
γ_{b1} = coefficient of operational conditions of concrete under repeated loads.

3.4.7.4 In calculations for concrete cracking the presence of reinforcement in the compressive zone of a section may be ignored.

3.4.7.5 In crack-nonresistant members of reinforced and steel concrete structures the calculation for opening of cracks which are normal to the longitudinal axis of a member shall be performed subject to the condition

$$\Phi = a_c,\ R = [a_c] \qquad (3.4.7.5)$$

where a_c = design width of crack opening, in mm;
$[a_c]$ = permissible width of crack opening, in mm, (refer to 3.4.7.8).

3.4.7.6 The width of crack opening a_c, (in mm) in members of reinforced and steel concrete structures shall be determined by the formula

$$a_c = 7C_d\varphi\varepsilon_s(4 - 100\mu)d^{0,5} \quad (3.4.7.6)$$

where C_d = coefficient assumed equal to:
1,0 — with regard to a temporary loading;
1,0 — at $F_l/F_c < 2/3$,
1,3 — at $F_l/F_c \geq 2/3$;

F_l and F_c = maximum generalized forces (bending moment, longitudinal force, etc.) due to a full loading (permanent, operational effects and environmental loads) and only to permanent and long duration loads, respectively; with regard to a repeated load $C_d = 2 - \rho_s$
ρ_s = coefficient of cycle asymmetry;

φ = coefficient dependent on the reinforcement type and assumed equal to:
1,0 — for deformed bar reinforcement,
1,2 — for deformed wire reinforcement,
1,4 — for plain bar and plate reinforcement;
ε_s = deformations in tensile reinforcement computed as σ_s/E_s without regard to the operation of the concrete tensioned in a section.
μ = coefficient of sectile reinforcing, $\mu = F_s/bh_0$, but not more than 0,02;
d = reinforcement diameter, in mm.

Notes: 1. In calculation of members with exterior plate reinforcement instead of d in Formula (3.4.7.6) $d_r = 1{,}5(F_{si}/\pi)^{0,5}$ shall be used, where F_{si} = area of plate tensile reinforcement in mm², at the section part 0,1 m wide.
2. In calculation of eccentrically and centrally tensioned members the calculation result by Formula (3.4.7.6) shall be increased by 20 %.

3.4.7.7 The width of opening for the cracks inclined to the longitudinal axis of a member in way of the action of the maximum shearing forces shall be regulated by the restriction of the level of the maximum shear stresses in concrete, namely, the following condition shall be met:

$$\tau_{max} \leq 1{,}86R_{bt2} \quad (3.4.7.7)$$

at that τ_{max} shall be determined in accordance with 3.4.6.1.5.

3.4.7.8 The permissible width of crack opening $[a_c]$ shall be taken reasoning from the operational conditions of a structure, data on environmental corrosive effects, intactness of bar and plate reinforcement, the impact of freezing and thawing processes, and not more than the values specified in Table 3.4.7.8.

Table 3.4.7.8

FOP hull area	Permissible width of crack opening $[a_c]$, mm		
	Reinforced concrete structures		Steel Concrete structures
	seaside	inner side	
Bottom and sides in underwater part	0,10	0,15	0,25
Ice strake zone	0	0,10	0,25
Sides above water level	0,15	0,20	0,25
Decks and internal walls	0,20	0,20	0,25

3.5 TENSION LEG PLATFORMS

3.5.1 General.

3.5.1.1 TLP consists of the following three basic components (structural elements):
hull;
tension legs;
ground foundation (anchor).

Designing TLP structural elements shall be carried out keeping due note of the acceptable requirements of Sections 1 and 2, considering additional instructions and regulations contained in this Chapter.

3.5.1.2 The structural strength of TLP shall be tested on the basis of criteria mentioned under 3.5.3 for the following design conditions:
extreme load;
operational;
transit;
positioning;
removal from site;
replacement of tension legs, if provided for during operation period.

Replacement of tension legs means that one (or more) tension leg may be removed for survey, maintenance or replacement. This mode shall be determined considering anticipated frequency of the leg removal and duration of out-of-service period.

3.5.1.3 The TLP clearance h_c, in m, shall be not less than the largest of the values determined by the formulae:

$$h_c = \Delta_{100} + 1{,}2(D/\lambda_{100})^{1/4}h_{100} + 1{,}5; \tag{3.5.1.3-1}$$

$$h_c = \Delta_{100} + 4h_{raf_{100}} + 0.5 \tag{3.5.1.3-2}$$

where Δ_{100} = peak amplitude of a sea level change, which is probable once in 100 years, including storm surge, in m;
h_{100}, λ_{100} = wave height and length, respectively, which are probable once in 100 years, in m;
$h_{raf_{100}}$ = thickness of rafted ice, which is probable once in 100 years, in m;
D = diameter or the cross dimension of a conic leg at the waterline level, in m.

3.5.1.4 In addition to the requirements of 1.3 the following TLP structures are subject to technical supervision during manufacture and positioning:
tension legs;
ground foundation.

All the requirements set out in the above paragraph are applicable to the mentioned structures.

3.5.1.5 If the design technical requirements specify that the tension legs are subject to replacement during operation period, i.e. their life cycle is shorter than that of a TLP, the wear margin for tension legs shall be determined basing upon the actual life cycle.

3.5.1.6 Requirements for the materials of TLP steel structures are set out in 3.5.5. Requirements for the materials of reinforced concrete or composite structures of ground foundation and TLP hull ice belt shall correspond to 1.5.2.

3.5.1.7 The requirements of this Chapter to the tension legs are drawn up mainly for the legs consisting of steel tubular elements. For design of tension legs consisting of ropes or chain cables one shall consider the requirements of Part III "Equipment, Arrangements and Outfit of MODU/FOP" as well as the requirements of this Part, as far as applicable.

3.5.2 Loads.

3.5.2.1 When determining wind, current, ice, seismic, deck and mooring loads for a TLP and its elements one shall consider the requirements of 2.2, 2.3.6 — 2.3.10, 2.3.12, 2.3.13 and 3.3.2. When drawing up load combinations it is recommended to take into account the requirements of 2.3.11. Additional requirements to be considered in determining loads are set out in 3.5.2.2 — 3.5.2.6.

3.5.2.2 Alternating wind loads.

The relation between maximum w_{max} and average $\bar{w}$ velocities is determined by the gustiness coefficient G similarly to 2.2.2.3.

Keeping in mind that the profile of the TLP above water hull consists of extended elements, the pulse component of wind loads Q_w shall be determined considering coefficient of wind pulse correlation. General coefficient of correlation η shall be accepted equal to 0,8.

3.5.2.3 Wave loads.

Wave loads are of great importance for TLP both by their intensity and frequency, since natural periods of the system "tension legs — hull" are often found within the range of power bearing waves.

Taking into account irregular nature of wave conditions, the methodology for determining wave loads shall be based upon statistical approaches.

3.5.2.3.1 For assessing statistical characteristics of wave loads one of the two approaches is recommended. According to the first approach, the life cycle is presented as the set of wave stationary modes. Leg reactions characterized by the wave height of a given probability, average wave period, course angle and recurrence are being found for each mode, whereupon all the reactions are summarized.

Another approach is based on the concept of the severest conditions, at which the extreme value of reaction is the most probable for realization (refer to 2.2.3.5, 3.1.4.10, 3.1.4.11, 3.1.4.12).

3.5.2.3.2 As the basis dependence for determining wave loads one may use the Moritz equation considering diffraction corrections (refer to 2.3.9.1 and 2.3.9.2).

In obtaining probabilistic characteristics of wave loads on the basis on the Moritz equation one is to use the Weibull distribution, the parameters of which (scale and form) shall be determined from the diagrams and dependences in Fig. 3.1.4.9. On X-axis on the diagrams one shall single out the relation of standard deviations of the wave load inertia and speed components.

3.5.2.3.3 The method based on the Moritz equation suggests that the structure does not cause distortion of speed and acceleration field of the liquid particles motion in a wave, thus preventing from fully considering the diffraction effects and hydrodynamic interaction of the structure elements. Though, this method makes it possible to fully use different wave theories, consider viscosity effects, extremities of the wave amplitude and the structure vibrations, shallow water effects.

It is reasonable to use the method in case the dimensions of a structure or its elements are so small that viscosity forces prevail in wave loads.

For large structural diameters ($D/\lambda > 0{,}2$) and relatively small wave heights it is advisable to use diffraction theory of calculation. The Moritz equation is preferably used for smaller diameters ($D/\lambda < 0{,}2$).

Method of linear diffraction theory is based on the assumption of smallness of a wave height and the platform vibration amplitude, potential character of liquid motion preventing from fully considering the viscous effects. Though, this method helps to consider the diffraction effects originated from flow past large bodies and connected with distortion of speed field in the wave.

The method is applicable in calculation of loads for the structures with dimensions equally large longitudinally and horizontally, multi-column platforms, when the diffraction effects are sufficient and viscosity forces are negligible.

3.5.2.3.4 Requirements of 3.5.2.3.1 — 3.5.2.3.3 also consider application of other approaches, approved by the Register upon appropriate review. In particular, the method of calculation of wave loads on TLP with the aid of ANCHORED STRUCTURES software package approved by the Register may be used.

3.5.2.3.5 Disturbance forces of wave loads affecting TLP are permitted to be accepted equal to those of the semi-submersible MODU of a relevant structural and architectural type, while for local loads on TLP the wave loads shall be considered according to 3.2.3.4.

3.5.2.4 High frequency wave loads.

During a TLP operation the low frequency wave load may be subjected to superposition of high frequency loads generally of a pulse or impact nature (i.e. during realization of "springing" and "ringing" phenomena), respectively, vertical high frequency vibration of a TLP caused by pulse loads, and vertical high frequency vibration of a TLP caused by cyclic load from the vertical vibrations, rolling or pitching of a TLP with resonant or near-resonant periods. Because of perceptible presence of a high frequency component, in a number of cases the issue is considered in the context of its impact on fatigue life.

3.5.2.5 Vortical loads.

Vortical forces due to the current effecting the hull structures and tension legs are determined in accordance with the following.

3.5.2.5.1 Vortical vibration of a TLP bluff elements, caused by the current, may lead to undesired consequences at a certain current velocity. At this, frequency of vortex separation, determined by the formula, is of great importance

$$f = Sh\frac{v_t \sin\varphi}{D}, \text{ in Hz,} \qquad (3.5.2.5.1)$$

where Sh = Strouhal number;
v_t = current velocity, in m/s;
D = typical cross sectional dimensions of a structure (diameter), in m;
φ = angle between axis of the structure and direction of the current.

The frequency given in Formula (3.5.2.5.1) corresponds to the alteration of vortical forces across the current; the frequency of forces alteration along the current is half as low as that determined by Formula (3.5.2.5.1).

3.5.2.5.2 Generally, the value of Sh corresponding to the frequency of vortex separation is determined in relation to Reynolds number R_e. On the basis of numerous field researches for determining the disturbing forces effecting the blunt structures, the following dependences may be used:

$Sh = 0{,}20$ for $R_e \leq 2{,}5 \times 10^5$;

$Sh = 0{,}27$ for $R_e > 2{,}5 \times 10^5$.

3.5.2.5.3 Coincidence of frequencies of unsteady forces with natural frequencies of the structure causes resonant phenomena with the possible considerable oscillation amplitude. Generally, vortical vibration is of hydroelastic nature and shall be studied by appropriate methods. The main peculiarity of natural vibrations is represented by velocity-expanded zones of resonant vibrations resulted from synchronization of natural vortex separation.

3.5.2.5.4 In the extended structures, such as tension legs, the resonant vibrations may emerge at all the operational velocities of the current. As a rule, transverse vibrations effecting extended tension legs are more intense than those directed along the current.

3.5.2.6 Dynamic aspects of a TLP behavior.

3.5.2.6.1 Dynamic characteristics of TLP are very important when assessing wind, wave, ice, seismic loads as well as load from current.

3.5.2.6.2 Frequency of the platform natural vibrations at *i*-th degree of freedom is determined by the formula

$$p_i = \sqrt{K_i/M_i} \qquad (3.5.2.6.2)$$

where K_i and M_i = respectively, stiffness of a TLP system, including tension legs, and mass of a TLP with added mass (or moment of inertia of masses with respect to the drilling location).

3.5.2.6.3 Stiffness of the vertical leg system during horizontal shift for small rotation angles, in kN/m, is determined by the formula

$$K_x = \frac{n_{t.l.}T_{t.l.}}{L_{Rt.l.}} + \frac{n_r T_r}{L_{Rr}} + n_{t.l.}\left(W_{t.l.} - \gamma\frac{\pi D_{t.l.}^2}{4}\right)\frac{L_{At.l.}}{2L_{Rt.l.}} + n_r\left(W_r - \gamma\frac{\pi D_r^2}{4}\right)\frac{L_r}{2L_{Rr}} \qquad (3.5.2.6.3)$$

where $n_{t.l.}$ = number of tension legs;
n_r = number of raisers;
$T_{t.l.}$ = pretension of a tension leg, in kN;
T_r = tension of a raiser;
$W_{t.l.}$ = weight of a tension leg per length unit in the air, in kN/m;
W_r = weight of a raiser per length unit, including the liquid contained therein, in the air, in kN/m;
$D_{t.l.}$, D_r = diameter of a tension leg and raiser, respectively;
$L_{At.l.}$, L_{Ar} = effective axial length of a tension leg and raiser, respectively, in m;
$L_{Rt.l.}$, L_{Rr} = rotation radius of a tension leg;
γ = specific weight of water.

3.5.2.6.4 Non-linearity of the system in the horizontal direction shall be considered in the following ratio:

for the vertical leg system

$$u/L_{Rt.l.} \geq 0{,}02; \qquad (3.5.2.6.4\text{-}1)$$

for the inclined leg system

$$u/L_{Rt.l.\beta}\cos\beta \geq 0{,}1. \qquad (3.5.2.6.4\text{-}2)$$

3.5.2.6.5 Stiffness of the vertical leg system during vertical vibrations, in kN/m, is determined by the formula

$$K_z = n_{t.l.}K_l + \rho g S \qquad (3.5.2.6.5)$$

where S = total area of floatation waterline, in m²;
ρ = mass water density, in kN·c²/m⁴;
G = acceleration of gravity, in m/c².

3.5.2.6.6 Stiffness of the inclined leg system during vertical vibrations, in kN/m, is determined by the formula

$$K_z = n_{t.l.} K_l \sin\beta + \rho g S. \tag{3.5.2.6.6}$$

3.5.2.6.7 Stiffness of the system during rotational vibrations, in kN/m, is determined by the formula

$$K_{xz} = 4K_l a^2 - \lambda_B \tag{3.5.2.6.7}$$

where $\lambda_e = G\overline{KG} - F_B\overline{KB}$;
G and F_B = mass of the structures and buoyancy integral;
$\overline{KG}$ and $\overline{KB}$ = respectively, distance between the centre of gravity and centre of buoyancy from the level of hawses, m;
a = half of the distance between the hawses connecting tension legs, in m.

3.5.2.6.8 Added masses and their moments of inertia for the *i*-th degree of freedom depend on the wave frequency and are determined on the basis of the certain theoretical solutions for the simple-shaped bodies and model tests for the irregular shaped bodies.

3.5.2.6.9 Dynamic aspects of iceberg loads on TLP may be significant when calculating the common and local strength of the structures as well as safety factors of the anchor lines. Dynamic effects are related to vibrations of TLP and iceberg due to iceberg contact with the TLP hull or anchor lines. Further contact interaction depends largely on the complex law of the iceberg displacement in relation to TLP.

3.5.2.6.10 Dynamic aspects of the TLP-iceberg interaction shall be analyzed with due regard to the loss of stability by the iceberg which can result in its capsizing in close proximity of TLP due to the contact interaction with the hull or anchor lines.

3.5.2.6.11 Since dynamic effects significantly depend on the iceberg parameters, geometric properties of icebergs and TLP, performance of position-keeping system and ice speed, dynamic aspects of iceberg loads on TLP shall be accessed, where reasonable, on the basis of mathematical modeling of the iceberg and moored structure interaction using the software approved by the Register.

3.5.2.7 Global iceberg loads on TLP.

3.5.2.7.1 Global iceberg loads on TLP shall be determined based on the contact area of iceberg and structure. The time-dependent contact area shall be calculated with due regard to the interacting objects, their mutual orientation, direction of the iceberg drift and platform flexibility according to the procedures or by means of the software approved by the Register.

3.5.2.7.2 Global iceberg loads on TLP are determined by the following formula:

$F_{horiz} = 7{,}4A$ MPa at $A \le 1\ m^2$;

$F_{horiz} = 7{,}4A^{0{,}3}$ MPa at $1\ m^2 < A \le 1\ m^2$; (3.5.2.7.2)

$F_{horiz} = 1{,}48A$ MPa at $A \le 10\ m^2$

where A = projection of the current contact area on a plane normal to direction of the structure intrusion into iceberg.

3.5.3 Strength criteria.

3.5.3.1 General

3.5.3.1.1 Generally, the main requirements set out in 2.4.1 are applicable for the TLP. Additionally to the dangerous states listed in 2.4.1.2 the following shall be included: slackening of a tension leg. Respectively, the criterion of leg tension maintenance shall be observed.

3.5.3.1.2 The tensile leg angle in the upper and lower coupling shall be chosen such that the leg remains undamaged in the area of its interaction with the hull and anchor structural elements considering characteristics of the flexible elements.

3.5.3.1.3 Buckling strength criterion for the TLP hull and foundation structural elements shall be in compliance with 2.4.3.

3.5.3.2 Ultimate strength criterion.

3.5.3.2.1 The ultimate strength criterion for extreme loading of the hull and anchor is determined by expression (2.4.2.2) considering expressions (2.4.2.3.1) and (2.4.2.3.2). At that, safety factor η_1 shall be determined according to 3.5.3.2.4.

3.5.3.2.2 The ultimate strength criterion during extreme loading of tension legs is determined by the expression

$$T_\Sigma < \eta_1 A \sigma_t, T_\Sigma \leq \eta_1 T_b \tag{3.5.3.2.2}$$

where T_Σ = total design tension of a leg, caused by all the possible static and alternating loads (tension components are characterized in 3.5.4), in kN;
A = design cross-section area of the leg, in m²;
σ_t = temporary resistance of the leg material, in MPa;
T_b = breaking stress of the leg, in kN.

3.5.3.2.3 The TLP ultimate strength criterion for the operating and transit conditions, the conditions of positioning at and removal from site and safety factors, respectively, shall be in compliance with 2.4.2.4, 2.4.2.5 and 3.3.1.1.

3.5.3.2.4 Safety factors η_1 for the criteria set out in 3.5.3.2.1 and 3.5.3.2.2 shall not exceed the values given in Table 3.5.3.2.4.

Table 3.5.3.2.4

Name of a structure	Strength criterion	Structural elements		
		Special	Primary	Secondary
Hull and foundation beyond the area of interaction with tension legs	p. 3.5.3.2.1, criterion 2.4.2.3.1	0,75	0,80	0,83
	p. 3.5.3.2.1, criterion 2.4.2.3.2	1,25	1,35	1,45
Hull and foundation in the area of interaction with tension legs	p. 3.5.3.2.1, criterion 2.4.2.3.1	0,65	0,70	-
	p. 3.5.3.2.1, criterion 2.4.2.3.2	1,20	1,30	-
Tension legs	3.5.3.2.2	0,55	0,60	–

3.5.3.2.5 The safety factor for a tension leg replacement practice is applied in compliance with Table 3.5.3.2.4. When making calculations, the changing of TLP trim due to ballasting shall be considered.

3.5.3.2.6 Additional ultimate strength criteria for the ice belt structures of the ice-resistant TLP shall be in compliance with 3.3.3.

3.5.3.3 Fatigue strength criterion.

3.5.3.3.1 Fatigue strength criterion shall be applied to the structural elements, for which the strength may represent the ultimate form of destruction, for example, tension legs —

foundation — hull structure joints, as well as tension leg elements. The list of joints shall be agreed by the designer with the Register.

3.5.3.3.2 Designing of the platform structures shall be performed on the basis of the "safe damage" criterion, according to which the fatigue criterion realization is oriented at the stage of fatigue macrocrack initiation rather than the crack development. Characteristics of ultimate relative vulnerability are determined in 2.4.4.

3.5.3.3.3 The sources of cyclical loads are waves, wind, current, ice and seismicity reason. The initial data for each type of cyclical loads is recurrence of environmental conditions.

3.5.3.3.4 In the absence of sufficient statistics on the structure loading the fatigue life at wave, seismic or alternating ice loads is recommended to be determined on the basis of the analytical dependencies.

3.5.3.3.5 Influence of high frequency components of a wave load from "springing" and "ringing" (refer to 3.5.2.4) on the tension leg fatigue life is determined by a reduction coefficient γ dependent on relation of standard deviations of high frequency and low frequency components, as well as on m parameter.

3.5.3.4 Criterion of tension maintenance in the leg.

3.5.3.4.1 The tension maintenance criterion stipulates the requirements aimed at preventing the origination of a tension leg slackening, as the result of which the tension leg is considered to be out of service.

3.5.3.4.2 This criterion may be defined in the following way:

$$T_{\sum\vartheta} \leq \eta_1 * T_0 \tag{3.5.3.4.2}$$

where $T_{\sum\vartheta}$ = design leg tension dependent on design loads leading to tension minimizing by elimination of the original (initial) tension on still water;

T_0 = original (initial) tension on still water;

$\eta_1 *$ = safety factor; $\eta_1 *= 0{,}70$.

3.5.4 Peculiarities of strength calculation and TLP design.

3.5.4.1 General.

3.5.4.1.1 Generally, the main requirements set out in 2.5.1 and 2.5.2 are applicable for the TLP design. In addition it shall be noted that the TLP vital reactions are represented by linear and angular motion of the hull, as well as internal axial forces in the TLP.

3.5.4.1.2 Damage of one leg shall not lead to progressive breaking of other legs or excessive damaging of the huh or foundation in the areas of interaction with tension legs. The Register may require the calculation confirming that being exposed to the environmental loads maximum for the given area of operation for a one-year period the structural strength of the platform with a damaged tension leg will be maintained.

3.5.4.1.3 In designing of TLP one shall consider that the hull shift with regard to the ground foundation would not cause damage of the structure and, finally, accident situations.

3.5.4.2 Hull.

3.5.4.2.1 A particular method of the TLP hull design shall be determined according to the peculiarities of the structure. When calculating hull strength for the multi-column TLP the regulations of 3.2 related to the semi-submersible MODU may be applied. When calculating hull strength for the tower-shaped TLP the regulations of 3.3 related to monopods (monocones) may be applied.

3.5.4.2.2 Calculation of girder system, separate girders, plates, calculation of structural elements' buckling strength shall be carried out in accordance with 2.5.3 — 2.5.5.

3.5.4.2.3 Ice belt structures of ice-resistant TLP shall be calculated in accordance with 3.3.4. At that one shall consider that ice formation shall not touch with the areas of the hull and tension legs joint.

3.5.4.2.4 Calculation of steel concrete ice belt shall be carried out in accordance with 3.4.
3.5.4.3 Tension legs.
3.5.4.3.1 A tension leg consists of the three basic elements:
area of interaction with the hull;
area of interaction with the foundation;
basic part of the leg — joints between all the above elements.
The area of interaction with the hull is designed for the following functions: control and regulation of the required tension, joining tension legs with the hull, perception of transverse forces and bending moments. The area of interaction with the foundation is designed for the following functions: maintenance of structural joints between the foundation and a leg, perception of transverse forces and bending moments.
The operational peculiarities of each area determine the deflected mode character and appropriate approaches to the structural strength calculation.
3.5.4.3.2 Tension in any tension leg is the sum of a range of components possessing different physical values, i.e.:

$$T_\Sigma = \sum_{i=1}^{n} T_i \qquad (3.5.4.3.2)$$

where n = number of considered components.

These components are subdivided into two radically different groups: deterministic (including static) and occasional.
3.5.4.3.2.1 The full formulation of tension components is as follows:
deterministic (or quasideterministic) components:
T_0 = original tension at the mean water depth;
T_t = tension from storm surge;
T_λ = leg tension dependent on alteration of ballast, cargo etc. weight;
T_m = tension caused by capsizing moment from wind load and current;
T_s (wave or ice) = tension caused by sagging due to static loads and slowly changing shift (wave drift or constant component of ice load, wind, current);
T_f (wave or ice) = tension caused by foundation shift under the influence of water or ice;
occasional components:
T_w (or T_{ice}) = alternating tension component from wave or ice forces with regard to average shift (includes tension from horizontal forces, vertical forces, vibrations (rolling and pitching), generally form rotation forces);
T_i = tension caused by vertical oscillation, rolling or pitching at the natural oscillation frequency of the platform (ringing and springing, including possible underdeck slamming loads).
3.5.4.3.2.2 Standard deviation of total tension is determined by the formula

$$\sigma_{T_\Sigma}^2 = \sum_i \sigma_{T_i}^2 + 2\sum_i \sum_j \rho_{ij}\sigma_{T_i}\sigma_{T_j}. \qquad (3.5.4.3.2.2)$$

where σ_{T_i}, σ_{T_j} = standard deviations of separate components determined basing upon the idea of statistical dynamics;
ρ_{ij} = coefficient of correlation between separate tension components.

3.5.4.3.2.3 As the wave tension distribution law the Weibull distribution with the parameters of scale a and K form (refer to 3.1.4.8 and 3.1.4.9) is recommended.
3.5.4.3.2.4 As the ice tension distribution law the Weibul distribution with the parameters of scale $\bar{b}$ and K form, determined depending on relation a_h/D, where a_h is the parameter of

ice thickness distribution scale, D is the obstacle diameter at the waterline level, (refer to 3.3.2.3.3 and 3.3.2.3.5) is recommended.

3.5.4.3.3 Basic stages of tension leg design procedure including consideration of ultimate and fatigue strength as well as the hull and foundation impact on legs, are as follows:

platform dimensions - determining the TLP general configuration;

leg predesign - assessment of pretension and other input data necessary to determine the TLP dimensions;

analysis of reactions - determining the structure's shift and minimum /maximum leg tension;

horizontal leg reactions - determining bending moments in legs and horizontal vibrations;

minimum tension — determining the minimum leg tension;

pretension analysis — check of the preliminary maximum stresses and fatigue life;

check of service limitations - check of admissible shifts of the structure as well as check of vibrations and leg shifts;

fatigue life — determining fatigue strength under effect of axial and bending forces combination;

final check — check of maximum stresses, minimum tension, fatigue life, etc.;

mutual analysis - determining necessity of carrying out the mutual reactions analysis;

model tests (not obligatory) - confirmation of vibrations and loads on the leg.

3.5.4.4 Ground foundation.

3.5.4.4.1 Primarily ground foundation is aimed at tension leg mooring, TLP loads perception and transmission fully or partially to the seabed foundation soil.

The main requirement to the foundation systems shall together with tension legs to reliably buoy the floating structure at the certain area of the open sea, restrict its shift within the specified area and, thus, provide normal operation conditions. Safety of the whole unit depends on operational reliability of the positioning system; breaking from the positioning point is inadmissible.

3.5.4.4.2 For the central tension legs buoyancy the foundation structures with ram piles as well as those of gravitational or mixed type may be used. The units may be made whether as a separate, supported by piles or masses or their combination, one-piece structure, to which all the tension legs and raisers are fixed, or as a system of separate, independent foundation structures for groups or strands of tension legs and borehole pipes.

Besides, the anchors consisting of one or several suction piles as well as the anchors of Stevmanta or SEPLA type may be considered as peripheral foundation systems.

3.5.4.4.3 The load may be transmitted to the ground in several ways, i.e. through the tension legs joined to the piles directly, through the surface ground-based foundation plates (templates) that transmit tension leg forces through the piles to the ground through surface gravity foundation.

3.5.4.4.4 Calculation of ground foundation shall include calculation of the foundation structure deformation and strength and calculation of the foundation buckling strength and shifts with regard to the ground.

In designing the TLP foundation structure the following issues regarding peculiarities of the structure's operating conditions shall be considered:

load eccentricities being the result of alteration of the tension leg forces within the group;

consequences of a tension leg/raiser installation — possible raise (pulling out) and re-location of the tension legs/raisers during service life of the platform;

position (installation) and operation (regulatory) design tolerances;

issues on survey and check of compliance of the foundations with the required operational capabilities.

3.5.4.4.5 Strength calculation for steel, reinforced concrete and steel concrete structures of ground foundations shall be carried out in accordance with acceptable requirements set out in 2.5 and 3.4.

3.5.4.4.6 Buckling strength calculation for seabed foundations shall meet the requirements stated in Section 4.

3.5.4.5 Joints.

3.5.4.5.1 The hull — tension legs joint.

3.5.4.5.1.1 The structures joining the hull and tension legs take up leg reaction by means of the two supporting areas:

upper area, taking up mainly tension force of legs;

lower area, taking up transverse reactions originated from the platform horizontal shift.

3.5.4.5.1.2 Supporting structures of the upper supporting area shall be designed for impact of a tensioner or maximum possible vertical leg reaction. At that, sufficient resistance of the structure considering statics and dynamics of the platform shift shall be checked:

to loss of buckling strength and necessary stiffness;

to collapse, shift or bending stresses.

Possibility of unequal distribution of the tension leg reaction distribution shall be considered.

3.5.4.5.1.3 Supporting structures of the lower supporting area shall be designed for the impact of a flexible element and maximum possible horizontal reaction considering statics and dynamics of the platform shift.

Sufficient resistance of the above structures shall be checked:

resistance to loss of buckling strength;

resistance to collapse, shift or bending stresses, local peak contact stresses.

3.5.4.5.1.4 Sufficient stiffness of work contact surfaces shall be specified in order to maintain their operating capability during whole service life of the platform.

3.5.4.5.2 Anchor — tension legs joint.

3.5.4.5.2.1 Structures of anchor — tension legs joints are represented by the two supporting areas:

upper area with a flexible element taking up horizontal reactions originated from the platform shift;

lower area taking up vertical reactions originated from the platform shift.

3.5.4.5.2.2 Supporting structures of the anchor upper supporting area shall be designed for the impact of flexible element or maximum possible horizontal reaction considering static and dynamic impact on the platform and the leg. The following structural resistance shall be checked:

resistance to loss of buckling strength;

resistance to collapse, shift or bending stresses;

resistance to local peak contact stresses;

resistance to ambient pressure for dry sea chests and closed spaces.

3.5.4.5.2.3 Supporting structures of the lower supporting area shall be designed primarily for the impact of maximum possible vertical leg tension considering statics and dynamics. The anchor structural resistance shall be checked by calculation of the following:

loss of buckling strength and necessary stiffness;

collapse, shift or bending stresses;

local peak contact stresses.

3.5.4.5.2.4 Structures of anchor — tension leg joints shall be provided with thickness margin considering high erosion and abrasion wear and probable chemical corrosion of steel.

3.5.4.5.2.5 When ropes are used as tension legs, the structures of the upper and lower areas may join together.

3.5.4.5.3 Joints of tension leg elements.

3.5.4.5.3.1 The main constructive way in designing joints equal to the tension leg in strength is the reduction of effective stresses through the extension of sectional area of the joint.

3.5.4.5.3.2 In design of the tension leg joints the strength calculation shall include:

total leg tension;

total leg bending in the area in question;

local bending induced by sectional eccentricity;

local concentration of stresses caused by peculiarities of the joint and/or weld.

3.5.4.5.3.3 In case the internal volume of the leg is isolated, the strength calculation shall additionally consider the impact of internal and environmental stresses on the joint stressed state.

3.5.4.5.3.4 For the structures of joints and tension leg elements check calculation of local impact strength against the reactions transmitted from the upper and lower supporting areas of the anchor and hull structures considering deformation of their flexible elements shall be carried out.

3.5.4.5.3.5 If strength check for the tension leg joints shows that their strength and service life shall not provide the platform real service life, the above calculations shall be re-performed considering replacement of tension legs during the platform operation.

3.5.4.5.4 Hydrodamping devices structures.

3.5.4.5.4.1 General.

3.5.4.5.4.1.1 Application.

The present provisions refer to hydrodamping devices for floating facilities subject to substantial motion in operating conditions which requires its damping, e.g. for a $Spar$ platform.

3.5.4.5.4.1.2 Definitions and explanations.

3.5.4.5.4.1.2.1 The following definitions have been adopted in the present recommendations:

Flap height is a full height of cylindrical surface normal to the disk (or double distance from the disk plane to the extreme flap point).

Damping devices are horizontal disks attached to the platform hull along its height.

Damper flaps are cylindrical surfaces, erected on the edges of a damping disk.

Perforation coefficient is the ratio between nonperforated area and the total disk surface area; the perforation coefficient equal to 1 corresponds to a nonperforated disk.

Relative factor of disk resistance is a resistance coefficient of a disk or a system of disks in question related to the resistance coefficient of a solid nonperforated disk.

Perforation are ring- or round-shape holes in a damping disk.

$Spar$ platform is a platform on tension and anchor legs intended mostly for deepwater operations, and which hull is elongated in vertical direction.

3.5.4.5.4.1.3 Structural elements.

The platform structural elements shall be classified into special, primary and secondary elements proceeding from stress levels and the effect their eventual damage may have upon the strength and serviceability of the floating facilities.

Due to their serviceability, the hydrodamping devices (disks) shall be related to the primary elements which ensure the overall strength of the hull (in case of hydrodamping disks failure the motions will increase with all ensuing consequences of the overall strength reduction).

3.5.4.5.4.2 Hydrodamping devices design principles.

3.5.4.5.4.2.1 General.

The design of hydrodamping devices shall be so that:

their operating parameters will satisfy the requirements for motions damping;

their strength within the service life (as determined for environmental conditions of the anticipated area of operation) meets the accepted criteria in the following design modes of operation: transit, operating, survival and extreme loading.

3.5.4.5.4.2.2 Loads.

In calculations of hydrodamper the loads due to wind, waves and current effects shall be considered.

The loads to be estimated for the structural strength analysis of hydrodamping devices shall include:

common and local hydrodynamic loads in the extreme wave conditions considering wind effects;

hydrodynamic cyclical loads;

current loads;

high frequency loads of pulse or impact nature (vertical high frequency vibration of a structure caused by pulse loads, and vertical high frequency vibration of a structure caused by cyclic loads from the vertical vibrations, pitching or rolling with resonant or near-resonant periods). Because of perceptible presence of a high frequency component, in a number of cases the issue is considered in the context of its impact on fatigue life;

vortical loads, forces due to the current effecting the damping devices are determined in accordance with the condition

$$f = Sh\frac{v_t}{D}, \text{ in Hz}, \qquad (3.5.4.5.4.2.2)$$

where Sh = Strouhal number,
v_t = current velocity, in m/s,
D = typical cross sectional dimensions of a damping device structure (diameter),
f = frequency of vortex separation.

The frequency determined by Formula (3.5.4.5.4.2.2) corresponds to the alteration of vortical forces across the current; the frequency of forces alteration along the current is half as low as that determined by Formula (3.5.4.5.4.2.2).

Generally, the value of Sh corresponding to the frequency of vortex separation is determined in relation to Reynolds number R_e.

Coincidence of frequencies of unsteady forces with natural frequencies of the structure (or its separate parts) causes resonant phenomena with the possible considerable oscillation amplitude. Generally, the vortical vibration is of hydroelastic nature and shall be studied by appropriate methods.

3.5.4.5.4.2.3 Strength criteria.

The damping disks structure shall be so designed as to meet the general safety requirement

$$\Phi \le R\eta \qquad (3.5.4.5.4.2.3\text{-}1)$$

where Φ = design value of the generalized force action which is used to assess marginal state;
R = design value of generalized bearing capacity,
η = safety factor which depends on the damping disk responsibility for strength and safety of the structure.

If requirement (3.5.4.5.4.2.3-1) is met the following dangerous states may be practically avoided:

excessive deformations of material;

buckling;

fatigue cracks;

Accordingly, the criteria of ultimate and fatigue strength, as well as buckling strength criterion shall be met.

For damping disks structural elements, in terms of their strength and stability, the MODU/FOP Rules provisions apply.

The ultimate strength criterion for survival condition (extreme impact) is determined by the expression

$$\sigma_d \leq \eta_1 R_d \qquad (3.5.4.5.4.2.3\text{-}2)$$

where σ_d = design structural stress caused by the most unfavourable combination of loads, in MPa;
η_1 = safety factor: for damping devices structures the safety factor is taken as $\eta_1 = 0{,}8$;
R_d = design yield stress of material.

The buckling strength criterion stipulates the requirements for those parameters of damper structural elements which provide stability of the given configuration. Critical buckling strengths are those which cause a structure to pass from one form of equilibrium to another.

Buckling strength criterion is determined by the expression

$$\sigma_x \leq \eta_2 \sigma_{cr} \qquad (3.5.4.5.4.2.3\text{-}3)$$

where σ_x = design stress for the specified condition of the structural element, in MPa;
σ_{cr} = critical stress, in MPa;
η_2 = safety factor; the safety factor shall not exceed the value

$$\eta_2 = 0{,}84(1 - 0{,}2R_d/\sigma_e) + 0{,}06$$

where σ_e = Euler's stress corresponding to the minimum value of all considered Euler's stresses and forms of stress state, in MPa.

Influence of high frequency components of a wave load from phenomena described in 3.5.4.5.4.2.2 on the tension leg fatigue life is determined by a reduction coefficient dependent on relation of standard deviations of high frequency and low frequency components.

3.5.4.5.4.3 Structural design issues specific to hydrodamping devices.

3.5.4.5.4.3.1 The hydrodynamic forces acting on the system of hydrodamping devices may be determined in the first approximation without considering horizontal oscillations, while vertical oscillations in a viscous fluid shall be considered. At a sufficient distance from a streamlined body the water may be considered as motionless relative to the seabed. For the damper itself the adhesion condition shall be adopted (i.e. water velocity coincides with the damper velocity). The damper can move progressively (stationary flow) and oscillate (non-stationary flow). At the inflow, outflow and at the lateral boundary, the velocity equal to inflow velocity shall be set up, which means negligible damper effect on the boundary of contact.

3.5.4.5.4.3.2 In case of slott perforation, the task is symmetrical relative to disk central axis, which stipulates current study in 2D domain.

For the case of modelling flow around a disk damper with round or square perforated holes, three-dimensional task shall be considered.

3.5.4.5.4.3.3 The hydrodynamic load acting on hydrodamping devices includes two components: the velocity (viscous) and inertia component.

3.5.4.5.4.3.4 The velocity (viscous) component of the hydrodynamic load is proportional to the square of velocity and velocity resistance C_v:

$$C_v = \frac{F_v}{(\rho V^2 S)/2} \qquad (3.5.4.5.4.3.4\text{-}1)$$

where F_v = disk resistance force component;
V = inflow velocity;
S = disk area;
ρ = mass density of water.

The inertia component of hydrodynamic load is proportional to the added mass.
The added mass is determined by the formula

$$m_a = F_a/a_y \qquad (3.5.4.5.4.3.4\text{-}2)$$

where a_y = disk acceleration.

3.5.4.5.4.3.5 For optimization of the hydrodynamic loads acting on hydrodamping devices it is recommended to study the following factors: disks perforation degree, perforation shape, "flaps" effect, dampers system and seabed effect.

3.5.4.5.4.3.5.1 Perforation degree. The perforation degree effect may be ambiguous: with reduction of perforation coefficient, the resistance coefficient can first considerably increase, and then it can decrease smoothly, while the added mass tends to rapid reduction. Generally, the best effect can be achieved with the perforation coefficient equal to 0.9.

In case of perforation with small diameter holes, the dynamic resistance coefficient and added mass grow insignificantly.

3.5.4.5.4.3.5.2 Perforation shape. The damper square and round perforation provides the qualitatively and quantitatively similar effects as in case of concentric slot perforation. At that, the dynamic coefficient increases 1,5 — 1,8 times, and the added mass is reduced by 20 — 30 % as compared to a non-perforated damper.

The hole shape (round or square) of the damper perforation does not practically affect its dynamic characteristics.

3.5.4.5.4.3.5.3 Flap effect. The presence of flaps results in reduction of resistance coefficient and in greater value of the added mass. When the flaps height grows, the resistance coefficient grows together with increase of the added mass. The application of flaps result in increase of damper strength characteristics. When cone-shaped flaps are used, the dynamic resistance coefficient grows with increase of α angle from 0° (direct flaps) to 90° (absence of flaps) almost in a linear manner, while the added mass first grows slightly, then decreases.

The wave length (period) effect is, as a rule, insignificant.

3.5.4.5.4.3.5.4 Stiffeners. The application of concentric stiffeners to improve the damper strength characteristics does not practically affect its hydrodynamic properties.

3.5.4.5.4.3.5.5 Damper system. With increase of the distance between the adjacent disks in a three-disk system, the relative resistance coefficient and dimensionless added mass increase.

3.5.4.5.4.3.5.6 Perforated dampers system. The resistance coefficients and added mass of a three perforated damper system substantially improve the resistance of a solid non-perforated damper.

3.5.4.5.4.3.5.7 Bottom effect. Where the distance between the disks of a three disk system is sufficiently great, the proximity to the seabed increases the relative resistance coefficient, while the dimensionless added mass remains practically unchanged.

3.5.4.5.4.3.6 The application of hydrodamping devices can substantially change the platform added mass consequently increasing the period of the platform natural vertical vibrations.

3.5.4.5.4.3.7 The application of hydrodamping devices can reduce the vertical wave load acting on the platform.

3.5.4.5.4.3.8 The application of the hydrodampng device system can reduce the platform vertical vibrations amplitude several times.

3.5.4.5.4.3.9 In calculating hydrodamping devices it is recommended to use the porous medium model which allows to obtain the results qualitatively similar to those obtained from the real holes simulation. At that, the these calculation data may be considered as asymptotic estimates of the resistance and added mass coefficients for the case of "infinite number of holes having infinitesimal diameter" with fixed perforation coefficient.

3.5.4.6 Peculiarities of structural design in the seismically dangerous regions.

3.5.4.6.1 When designing TLP in the seismically dangerous regions anchor strength and bearing capacity, tension legs and hull strength considering large-scale seabed deformations, possible ground dilution, as well as "seaquake", i.e. underwater acoustic impact on TLP structures shall be ensured.

3.5.4.6.2 It is necessary to avoid anchor arrangement at the seabed areas where earthquakes may cause large scale ground deformation.

If, nevertheless, anchors were arranged in the areas of considerable seismic shifts, the anchor bearing capacity shall be checked taking into account the specified ground shifts (i.e. slide of subsea slopes).

3.5.4.6.3 When assessing anchor stiffness considering seismic loads both maintenance of structural strength and bearing capacity of the seabed subjected to dynamic loads shall be provided.

One shall take into account possible temporary decrease of anchor bearing capacity caused by dynamic dilution of the ground. At that, one shall define the extent of possible degradation of bearing capacity, as well as the period of bearing capacity recovery (based upon the time required for ground consolidation).

The time specified shall be considered when choosing design foundation ground characteristics with regard to various combinations of loads and stresses.

3.5.4.6.4 In calculation of the anchor stressed state and buckling strength one shall take into account the anchor mass as well as the additional masses of water and anchor legs.

3.5.4.6.5 Consequences of seismic load transmission from the waterarea bottom to the TLP hull through the tension legs shall be considered when seismic horizontal and vertical shifts of the waterarea bottom in the anchor legs location areas exceed relative permissible TLP shifts caused by wave loads at drilling. In such cases the tension leg forces shall be determined basing upon the values of anticipated seismic shifts of anchor legs with regard to the water area bottom.

The values of the above forces shall be used for checking tension leg strength and calculating TLP strength including the elements joining tension legs to the TLP hull and bearing.

3.5.4.6.6 As the design underwater acoustic load on TLP hull and tension legs one shall regard the hydrodynamic pressure applied to the TLP bottom that is time-varying according to the harmonic law with period T^{de} amplitude value p_{amp}^{de} determined by the formula

$$p_{amp}^{de} = kT^{de} \exp\left(0{,}72J^{de}\right) \qquad (3.5.4.6.6)$$

where k = 0,003, MPa·s^{-1}.

In the absence of sufficient seismic information it is permitted to accept T^{de} value equal to 0,50 s.

3.5.5 Requirements to materials.

3.5.5.1 General.

3.5.5.1.1 Materials used for manufacturing the TLP hull and anchor structures beyond the areas of tension leg joints shall be in conformity with 1.5.1 of this Part and Part XII "Materials" of the MODU/FOP Rules.

3.5.5.1.2 This Section includes the specified requirements for the hull and anchor material in the areas of tension leg joints (mainly, special structural elements), as well as the requirements for the tension leg material that shall be regarded as addition to 1.5.1 of this Part and Part XII "Materials" of the MODU/FOP Rules.

3.5.5.1.3 For all the structures listed in 3.5.5.1.2 the resistance of hydrogen brittleness materials shall be assessed.

3.5.5.2 Hull.

3.5.5.2.1 Set of mechanical properties of materials of the hull structures interacting with a tension leg, tensioner or flexible element shall be assessed additionally with regard to the following damage types caused by local contact stresses on the work surfaces of the structural elements:

plastic straining, collapse or pressing out;

erosional and abrasive wear of contact surfaces;

fatigue fracture of wear surface;

brittle fracture of wear surface and crumbling of material.

3.5.5.2.2 For the hull structures exposed to tension leg reactions it is required to use steel materials at least 70 mm thick with yield stress $\sigma_{0,2} \leq 550\sigma_t$, in MPa.

3.5.5.2.3 Temporary resistance of the hull structural materials σ, aimed to adequately provide strength and plasticity shall correspond to the relation $\sigma_{0,2} \leq 0{,}85\sigma_t$.

3.5.5.2.4 In order to adequately provide plasticity of special structures of the TLP hull the material shall have a residual relative contraction Z_z during elongation of the material in the direction perpendicular to the plate surface: $Z_z \geq 25\,\%$. Relative elongation A_5 during testing of samples shall be not less than $A_5 \geq 18\,\%$.

3.5.5.2.5 Contact work surfaces of the hull special structures shall be designed for collapse and be stiff enough to prevent abrasive wear.

3.5.5.2.6 Taking into account considerable dynamic components of loading normal strength steels shall not be used for manufacturing the hull special structures.

3.5.5.3 Anchor.

3.5.5.3.1 Mechanical properties of the anchor structures interacting with tension legs shall be assessed with regard to the damage types listed in 3.5.5.2.1, as well as regarding:

additional abrasion wear induced by water-risen seabed soils;

high chemical corrosion;

stress corrosion cracking resistance.

3.5.5.3.2 For the special anchor structures it is recommended to use steel materials less than 120 mm in thickness and having a yield stress less than $\sigma_{0,2} \leq 550$ MPa with the continuity check at thicknesses above 70 mm.

3.5.5.3.3 Temporary resistance of material shall be in compliance with 3.5.5.2.3.

3.5.5.3.4 The material of the anchor special structures shall have a residual relative contraction during elongation of the material in the direction perpendicular to the plate thickness: $Z_z \geq 20\,\%$. Relative elongation A_5 checked during cutting out the through-plate-thickness samples from the center of the plate shall be not less than $A_5 \geq 18\,\%$.

3.5.5.3.5 Contact work surfaces of the anchor special structures shall be in compliance with 3.5.5.2.5.

3.5.5.3.6 Crack-resistance characteristics of the anchor special structures shall meet the requirements of 3.5.5.2.6. The testing samples shall be cut from the subsurface layer of material.

3.5.5.4 Tension leg.

3.5.5.4.1 Mechanical properties of the tension leg material shall be tested with regard to the types of possible fracture resistance corresponding to the leg functionality.

3.5.5.4.2 Requirements to the material of the area of interaction with the anchor shall be in accordance with 3.5.5.1 and 3.5.5.3.

3.5.5.4.3 Requirements to the material of the area of interaction with the hull shall be in accordance with 3.5.5.1 and 3.5.5.2.

3.5.5.4.4 Requirements to the material of the tension leg's middle section shall be in accordance with 3.5.5.1 and 3.5.5.2.

3.5.5.4.5 Circumferential yield stress $\sigma_{0,2}^{0}$ and ultimate strength σ_{t}^{0} of the leg shall be in accordance with the requirements:

$$\sigma_{0,2}^{0} \geq 0{,}9\sigma_{0,2}; \qquad (3.5.5.4.5\text{-}1)$$

$$\sigma_{0,2}^{0} \geq 0{,}9\sigma_{\mathrm{t}}. \qquad (3.5.5.4.5\text{-}2)$$

3.5.5.4.6 For leg coupling impact energy shall be equal to:

for the specimens cut in the rolling direction KVL≥68 J;

for the specimens cut across the rolling direction KVT≥46 J;

CTOD value at the temperatures equal to the temperatures of impact toughness testing shall be equal to:

for basic metal CTOD≥0,25 mm;

for heat-affected zone CTOD≥0,18 mm (with welding);

The temperature value of zero fracture toughness shall be less than NDT≤ - 40 °C with the wall thickness being less than 40 mm.

4 SELF-ELEVATING MODU/FOP STABILITY ON THE SEABED

4.1 GENERAL

4.1.1 The interaction of self-elevating MODU/ FOP supporting structures with the seabed has a significant impact on characteristics in terms of general stability of structures.

4.1.2 The way to keep a FOP on the seabed depends on the overall dimensions of the structure, the actual load level, soil properties, the external effects dynamics, the extent of environmental importance of the structure. For the way to keep on the seabed, FOP are divided into:

gravity;

pile-supported;

combined (combination of gravity and pile- supported) FOP.

4.1.3 The way to keep a self-elevating MODU on the seabed is gravitational with pre-loading of legs into the seabed.

4.2 STABILITY OF SELF-ELEVATING MODU ON THE SEABED

4.2.1 Stability against capsizing on the seabed.

The safety factor against self-elevating MODU capsizing on the seabed shall be not less than

$$K_{cap} = M_r/M_{cap} \geq 1{,}50 \quad (4.2.1\text{-}1)$$

where M_r = righting moment due to self-elevating MODU weight forces, in kN m;
M_{cap} = total capsizing moment due to extreme effect of external forces about the plane of self-elevating MODU support on the seabed, in kN m;

The worst combination of a righting and capsizing moments depending on the loading condition of a self-elevating MODU, the values and directions of external effects shall be considered.

With reasonably developed in area supporting surfaces of footings, the presence of a support moment shall be considered, i.e. the following condition shall be considered as criterion

$$M_r/\left(M_{cap} - M_{sup}\right) \geq 1{,}50 \quad (4.2.1\text{-}2)$$

where M_{sup} = support bending moment from the direction of seabed, in kN m.

4.2.2 Stability in shifting.

The safety factor against self-elevating MODU shifting on the seabed shall be not less than

$$K_{sh} = Pf/T \quad \geq 1{,}50 \quad (4.2.2)$$

where P = gravity load of a self-elevating MODU per leg with regard to the displaced water;
T = design value of a total shear force in way of a foundation;
f = friction coefficient of a supporting surface against the seabed.

The worst combination of a pontoon weight depending on the self-elevating MODU loading condition, and of a total shear force depending on the direction of extreme external effects shall be considered.

4.2.3 Stability in subsidence.

A safety factor for the subsidence of one of self- elevating MODU legs into the seabed shall be not less than

$$N_3/N > K_{sub} \quad (4.2.3)$$

where N_3 = pre-loading force;
N = design value of a total axial force;
K_{sub}= 1,0 — for four-leg units;
K_{sub}= 1,05 — for three-leg units.

The worst situation in terms of leg subsidence shall be considered as the very condition that is commonly critical. The subsidence condition establishes the necessary amount of ballast for three-leg units and impacts the volume and arrangement of spaces in a pontoon.

4.3 STABILITY OF FOP ON THE SEABED

4.3.1 Gravity FOP.

4.3.1.1 General.

The gravity FOP structure shall be designed so as to ensure the proper conditions for FOP positioning at a site and to exclude the following kinds of limit states:

loss of bearing capacity of a FOP-base system;

FOP capsizing;

excessive FOP shifting (subsidence, horizontal shifts, turning angles);

excessive seabed pressure on a skirt and inner ribs resulting in violation of the conditions of skirt-FOP structure assembly strength.

In design of a FOP foundation the following shall be also prevented:

limit state by the conditions of noncohesive soils liquefaction under dynamic effects;

significant seabed scour near legs.

The methods of the calculation of gravity FOP stability on the seabed including design values of loads, resistance and reliability indices shall be agreed with the Register. The main criteria are given in 4.3.1.2 — 4.3.1.6.

4.3.1.2 Criterion by conditions of gravity FOP positioning.

4.3.1.2.1 During FOP positioning the opportunity of skirt and inner ribs pressing-in into base ground for their entire height shall be assured what ensures the proper conditions of the FOP joint operation with the base.

4.3.1.2.2 The criterion of positioning conditions control is determined by the expression

$$N > KN_u \qquad (4.3.1.2.2)$$

where N = vertical force transmitted from a FOP to a base at the instant of its setting, in kN;

N_u = force of soil resistance to skirt and inner ribs pressing-in determined depending on their perimeters, heights, thicknesses and the results of static probing of the upper layer of base soil within which the skirt and ribs are pressed in, in kN;

K = normalized value of the assurance coefficient ensuring full pressing-in of the "skirt" structure into the soil.

The force N_u may be determined experimentally by pressing in the fragments of a ribbed structure into the base soil.

4.3.1.3 Criterion of bearing capacity of FOP-base system.

4.3.1.3.1 The criterion of bearing capacity of the system regulates the requirements for the relation between a force effect F and resistance forces R. This criterion shall be met in all potential schemes of ultimate equilibrium attainment (for a plane and depth shear at the different potential outline of shear surfaces).

4.3.1.3.2 The criterion of bearing capacity of a platform-base system is determined by the expression

$$R/F \geq k_{s,n} \qquad (4.3.1.3.2)$$

where F = design value of a generalized force effect used for estimating a limit state;

R = design value of a generalized resistance force (bearing capacity) counteracting the force F;

$k_{s,n}$ = normalized value of the coefficient of bearing capacity.

4.3.1.3.3 The bearing capacity of the system may also be estimated according to the results of deflected state calculations by the comparison of operational loads acting on the system with the loads bringing the system about a limit state with the formation of significant plastic zones in the base.

General stability of the structure on the seabed at dynamic loads is recommended to be assessed considering changes in the soil strength properties.

4.3.1.4 Criterion of ultimate eccentricity in application of loading resultant.

4.3.1.4.1 This criterion sets the requirements aimed at prevention of the potential hazardous state associated with the capsizing of gravity FOP with large eccentricities in application of loads causing arising of tensile zones at the contact of a bearing block bottom with base earth.

4.3.1.4.2 The criterion of ultimate eccentricity is determined by the expression

$$e \leq e_{ult} k_{s,n} \tag{4.3.1.4.2}$$

where e = eccentricity of application of the resultant of all loads (excepting a lateral earth pressure) acting on the FOP, in m;
e_{ult} = ultimate value of a loads resultant eccentricity set by the design specification; it is allowed to assume $e_l = B/6$ for the bottom of a rectangular foundation;
B = dimension of a bearing block in the direction of shearing load application.

4.3.1.5 Criterion of ultimate shifts.

4.3.1.5.1 The criterion of ultimate shifts sets the requirements aimed at prevention of the potential emergence of a hazardous state associated with the violation of the condition of a platform's normal operation.

4.3.1.5.2 The criterion of ultimate shifts is determined by the expression

$$S \leq S_{ult} \tag{4.3.1.5.2}$$

where S = joint deformation of a base and structure (subsidence, horizontal shifts, heel, etc.);
S_{ult} = ultimate values of the joint deformation of a base and FOP set by the design specification and equipment maintenance rules (in setting the value the potential disturbance of utility systems associated with the structure shall be taken into account).

4.3.1.6 Criterion of value of soil pressure on skirt and inner ribs.

4.3.1.6.1 This criterion sets the requirements aimed at prevention of the potential hazardous states associated with the break of strength of rib-structured members caused by the soil pressure.

The criterion shall be met for all the members of a ribbed structure and loading combinations.

4.3.1.6.2 The criterion of ultimate soil pressure determined by the expression

$$P \leq P_{ult} \tag{4.3.1.6.2}$$

where P = characteristic value of soil pressure diagram;
P_{ult} = ultimate value of P; the P_{ult} value corresponds to maximum allowable stresses in the skirt, inner ribs and adjoining areas of the FOP.

4.3.2 **Pile FOP.**

4.3.2.1 General.

4.3.2.1.1 The structure of a FOP pile foundation shall be designed to prevent the possibility of arising advent of the following kinds of a limit state:

loss of bearing capacity of a FOP-base system;

deformation of the entire base or its separate elements causing the break of the normal operation of a structure.

In design of a FOP pile foundation the following shall be also prevented arising of:

limit states as to strength and cracking (crack opening) for piles and piled mat foundation under a horizontal loading and bending moment;

limit state as to the conditions of noncohesive soils liquefaction under dynamic effects;

significant seabed scour near legs.

The methods of calculation of pile FOP stability on the seabed including design values of loads, resistance and reliability indices shall be approved by the Register. The main criteria are given in 4.3.2.2 — 4.3.2.3.

4.3.2.2 Criterion of bearing capacity of pile base.

4.3.2.2.1 The criterion of bearing capacity of a base soil for a single pile being part of a foundation and out of it takes the form

$$N \le F_d/\gamma_k \qquad (4.3.2.2.1)$$

where F_d = design bearing capacity of a single pile, in kN;
γ_k = reliability index determined depending on the way of bearing capacity determination and on the number of piles in the foundation;
N = design loading transmitted on a pile (longitudinal force arising in it due to the design loads acting on the foundation at their most adverse combination).

4.3.2.2.2 The design load on a pile shall be determined considering the foundation as a framed structure taking up vertical and horizontal loads, and bending moments. The design load on a pile for foundations with vertical piles may be determined by the formula

$$N = N_d/n \pm M_x y/\sum y_i^2 \pm M_y x/\sum x_i^2 \qquad (4.3.2.2.2)$$

where N_d = design compression force, in kN;
M_x, M_y = design bending moments, in kNm, about the principal central axes x and y of the piles plan in the plane of a mat foundation bottom;
n = number of piles in the foundation;
x_i, y_i = distance from principal axes to each pile axis, in m;
x, y = distance from principal axes to the axis of each pile for which the design load is computed, in m.

4.3.2.2.3 The design bearing capacity by pile foundation soil as a whole may be determined as the stun of bearing capacities of independent single piles where the distance between pile axes is over three pile diameters. In other cases, the mutual influence of piles shall be considered or the relevant substantiation of ignoring shall be provided.

4.3.2.3 Criterion of ultimate deformations.

4.3.2.3.1 The criterion of ultimate deformations sets the requirements aimed at prevention of the potential hazardous state associated with the violation of normal operation conditions.

4.3.2.3.2 The criterion of ultimate deformations takes the form

$$s \le s_{ult} \qquad (4.3.2.3.2)$$

where s = joint deformation of a pile, in m, pile foundation and structure (subsidence, displacement, a turning angle, the relative difference of subsidences of piles, pile foundations, etc.);
s_{ult} = ultimate value for joint deformation of the base of a pile, in m, a pile foundation and structure set by the design and equipment maintenance rules.

4.3.2.3.3 In calculation of pile deformations due to horizontal loading and bending moment it is allowed to use appropriate calculation methods for other similar structures, approved by the Register. The methods in use shall reflect the nonlinear nature of a "load — pile head displacement" relation.

4.3.2.3.4 The horizontal load acting on a foundation with vertical piles of the same cross-section may be assumed as uniformly distributed among all piles.

APPENDIX 1

CHARACTERISTICS OF WIND AND WAVE CONDITIONS

Table 1

Extremes of wind velocity and wave height with recurrence period of 50 years

Sea	Average wind velocity (10 min average period) $\bar{W}_{50}$, m/s	Wave height with 3 % probability of exceeding level h_{50}, m
Caspian Sea	45,0	13,0
Black Sea	43,0	12,5
Barents Sea	46,0	19,0
Okhotsk Sea	48,0	19,0

Table 2

Recurrence of wave heights and periods in the Caspian Sea, %

$\bar{\tau}$, c	$h_{3\%}$, m											
	0–1	1–2	2–3	3–4	4–5	5–6	6–7	7–8	8–9	9–10	10–11	11–12
0–1	7,11											
1–2	14,58											
2–3	6,44	20,21										
3–4	0,62	9,11	5,24									
4–5	0,33	6,32	10,02	5,36								
5–6	0,08	3,17	1,12	0,70	0,68							
6–7	0,07	1,54	0,66	0,49	0,44	0,05						
7–8	0,05	1,38	0,40	0,29	0,37	0,06	0,04	0,03				
8–9	0,03	0,97	0,27	0,23	0,21	0,07	0,06	0,03	0,005			
9–10	0,02	0,05	0,05	0,16	0,17	0,12	0,05	0,02	0,015	0,013	0,010	0,005
10–11	0,009	0,009	0,01	0,05	0,05	0,05	0,04	0,01	0,010	0,010	0,005	0,005
11–12	0,005	0,005	0,005	0,03	0,03	0,03	0,03	0,005	0,005	0,003	0,002	0,001
12–13	0,002	0,002	0,001	0,005	0,01	0,01	0,01	0,001	0,003	0,002	0,001	0,001

Table 3

Recurrence of wave heights and wind velocities in the Caspian Sea, %

$\bar{W}$, m/s	$h_{3\%}$ M											
	0–1	1–2	2 – 3	3 – 4	4–5	5 – 6	6–7	7 – 8	8 – 9	9–10	10–11	11–12
2 – 4	7,34	6,82	2,59	0,78	0,22	0,15	0,03	0,001				
4 – 6	7,80	7,76	4,42	1,02	0,43	0,26	0,08	0,003	0,001			
6 – 8	6,22	7,87	2,89	1,51	0,31	0,12	0,07	0,002	0,001			
8–10	4,32	7,95	2,80	0,71	0,17	0,06	0,01	0,007	0,002	0,002		
10–12	2,25	5,88	2,06	0,68	0,16	0,03	0,01	0,005	0,002	0,002		
12–14	1,15	3,35	1,58	0,57	0,15	0,01	0,01	0,009	0,009	0,005	0,004	
14–16	0,88	3,24	0,37	0,34	0,13	0,01	0,009	0,008	0,006	0,005	0,004	0,002
16–18		0,76	0,26	0,24	0,12	0,009	0,008	0,007	0,006	0,005	0,005	0,003
18 – 20		0,01	0,01	0,13	0,11	0,009	0,006	0,006	0,005	0,004	0,002	0,003
20 – 22		0,008	0,008	0,009	0,09	0,009	0,006	0,006	0,005	0,004	0,002	0,002
22 – 24		0,005	0,005	0,008	0,08	0,004	0,002	0,002	0,001	0,001	0,001	0,002
24 – 26		0,005	0,005	0,006	0,008	0,003	0,002	0,001	<0,001	<0,001	<0,001	<0,001

Table 4

Recurrence of wave heights and periods in the Black Sea, %

$\bar{\tau}$, c	$h_{3\%}$ M											
	0–1	1–2	2–3	3–4	4–5	5–6	6–7	7–8	8–9	9–10	10–11	11–12
0–1	2,31											
1–2	12,10											
2–3	16,45	10,14										
3–4	8,42	9,95	3,83									
4–5	6,36	8,90	4,34	1,20								
5–6	1,72	4,12	0,85	0,33	0,26							
6–7	0,94	3,11	0,30	0,17	0,14	0,06						
7–8	0,80	0,50	0,10	0,12	0,14	0,11	0,05	0,01				
8–9	0,49	0,21	0,05	0,09	0,13	0,10	0,04	0,01	0,007			
9–10	0,24	0,06	0,02	0,07	0,11	0,08	0,01	0,009	0,006	0,002	0,002	0,001
10–11	0,11	0,007	0,006	0,02	0,02	0,01	0,008	0,008	0,004	0,002	0,002	0,001
11–12	0,06	0,003	0,002	0,006	0,007	0,008	0,008	0,007	0,002	<0,001	<0,001	<0,001
12–13	0,004	0,001	<0,001	0,001	0,005	0,006	0,007	0,006	0,001	<0,001	<0,001	<0,001

Table 5

Recurrence of wave heights and wind velocities in the Black Sea, %

$\bar{W}$, m/s	$h_{3\%}$. m											
	0–1	1–2	2–3	3–4	4–5	5–6	6–7	7–8	8–9	9–10	10–11	11–12
2 – 4	16,22	2,99	1,45	0,20	0,11	0,02	0,01	0,001				
4 – 6	13,67	6,21	2,94	0,90	0,24	0,03	0,006	0,002				
6 – 8	8,87	6,46	1,72	0,76	0,09	0,07	0,02	0,01	0,001	0,001		
8–10	5,34	5,62	1,45	0,42	0,08	0,05	0,02	0,01	0,008	0,001	0,001	
10–12	2,65	3,01	1,05	0,14	0,07	0,04	0,02	0,01	0,005	0,001	0,001	0,001
12–14	1,60	1,30	0,93	0,08	0,05	0,02	0,01	0,006	0,004	0,001	0,001	<0,001
14–16	0,70	0,72	0,45	0,07	0,03	0,01	0,008	0,005	0,003	0,001	0,001	<0,001
16–18	0,53	0,39	0,34	0,05	0,01	0,008	0,006	0,004	0,002	0,001	< 0,001	<0,001
18 – 20	0,42	0,32	0,08	0,03	0,009	0,007	0,005	0,003	0,002	<0,001	< 0,001	<0,001
20 – 22	0,01	0,06	0,07	0,01	0,007	0,006	0,003	0,001	0,001	<0,001	< 0,001	<0,001
22 – 24	<0,001	0,04	0,05	0,006	0,005	0,005	0,002	0,001	0,001	<0,001	< 0,001	<0,001
24 – 26	<0,001	0,02	0,03	0,002	0,002	0,001	0,001	<0,001	<0,001	<0,001	< 0,001	<0,001
26 – 28	<0,001	0,009	0,01	0,001	0,001	0,001	0,001	<0,001	<0,001	<0,001	< 0,001	<0,001

Table 6

Recurrence of wave heights and periods in the Barents Sea, %

$\bar{\tau}$, c	$h_{3\%}$. m													
	0–1	1–2	2–3	3–4	4–5	5–6	6–7	7–8	8–9	9–10	10–11	11–12	12–13	13–14
0–1	0,51													
1 – 2	1,62													
2 – 3	3,65	4,22												
3 – 4	2,75	16,18	8,03											
4–5	1,88	10,92	6,03	2,21										
5 – 6	0,82	3,33	5,86	5,72	3,64									
6 – 7	0,46	1,18	2,98	2,35	2,05	1,03	0,75							
7 – 8	0,15	0,59	1,73	0,99	0,43	0,35	0,21	0,19	0,08	0,06				
8 – 9	0,08	0,46	1,02	0,72	0,19	0,18	0,12	0,11	0,07	0,05	0,01	0,008		
9 – 10	0,05	0,07	0,78	0,57	0,14	0,13	0,10	0,10	0,06	0,04	0,02	0,01	0,007	0,006
10–11	0,01	0,03	0,44	0,32	0,06	0,06	0,05	0,05	0,02	0,02	0,02	0,01	0,008	0,003
11 – 12	0,01	0,009	0,12	0,10	0,02	0,02	0,02	0,01	0,01	0,01	0,01	0,008	0,006	0,001
12–13	0,006	0,007	0,007	0,009	0,01	0,01	0,01	0,01	0,01	0,009	0,007	0,006	0,005	0,001
13 – 14	0,003	0,003	0,003	0,008	0,01	0,01	0,01	0,009	0,009	0,008	0,007	0,006	0,003	<0,001
14–15	0,001	0,001	0,001	0,004	0,006	0,006	0,007	0,008	0,009	0,007	0,006	0,005	0,002	<0,001

Table 7

Recurrence of wave heights and wind velocities in the Barents Sea, %

$\bar{W}$, m/s	$h_{3\%}$. m													
	0 – 1	1–2	2 – 3	3 – 4	4 – 5	5 – 6	6 – 7	7 – 8	8 – 9	9 – 10	10–11	11 – 12	12–13	13 – 14
2 – 4	3,56	8,02	1,14	0,21	0,05	0,02	0,004							
4 – 6	4,67	9,56	3,51	0,56	0,45	0,10	0,07	0,04	0,03	0,009	0,002			
6 – 8	2,30	7,60	5,65	1,58	0,67	0,11	0,08	0,06	0,01	0,007	0,003			
8 –10	0,47	5,96	5,43	3,00	1,16	0,40	0,16	0,10	0,04	0,005	0,003			
10–12	<0,001	3,65	4,92	2,61	0,34	0,21	0,12	0,10	0,03	0,02	0,007			
12–14	<0,001	1,98	3,61	2,08	0,31	0,18	0,08	0,06	0,03	0,02	0,008			
14–16	<0,001	0,23	2,04	1,97	0,23	0,17	0,05	0,04	0,03	0,02	0,001			
16–18	<0,001	0,006	0,55	0,50	0,19	0,16	0,05	0,04	0,03	0,01	0,01	0,008	0,006	
18 – 20	<0,001	<0,001	0,15	0,32	0,16	0,15	0,04	0,03	0,03	0,01	0,01	0,01	0,005	0,001
20 – 22	<0,001	<0,001	<0,001	0,09	0,09	0,08	0,04	0,03	0,02	0,01	0,01	0,01	0,004	0,002
22 – 24	<0,001	<0,001	<0,001	0,07	0,06	0,06	0,03	0,02	0,02	0,01	0,01	0,008	0,004	0,002
24 – 26	<0,001	<0,001	<0,001	0,01	0,01	0,01	0,01	0,01	0,02	0,01	0,01	0,008	0,003	0,001
26 – 28	<0,001	<0,001	<0,001	0,005	0,006	0,007	0,008	0,009	0,009	0,009	0,009	0,007	0,003	0,001
>28	<0,001	<0,001	<0,001	<0,001	<0,001	0,001	0,005	0,009	0,009	0,009	0,008	0,005	0,002	0,001

Table 8

Recurrence of wave heists and periods in the Okhotsk Sea, %

$\bar{\tau}$, c	$h_{3\%}$. m														
	0–1	1–2	2–3	3–4	4–5	5–6	6–7	7–8	8–9	9–10	10–11	11–12	12–13	13–14	14–15
0 – 1	0,15														
1–2	1,32														
2 – 3	1,46	1,70													
3 – 4	6,26	7,54	4,88												
4 – 5	5,54	7,22	3,99	3,56											
5 – 6	3,88	6,82	3,82	2,52	1,24										
6 – 7	0,85	5,41	2,50	1,28	0,77	0,55									
7 – 8	0,24	3,96	2,38	0,60	0,58	0,51	0,34	0,07							
8 – 9	0,12	2,48	2,32	0,45	0,26	0,22	0,11	0,14	0,05						
9–10	0,09	1,39	1,75	0,21	0,17	0,15	0,10	0,09	0,07	0,06	0,04	0,02	0,006	0,006	
10–11	0,03	1,11	1,10	0,17	0,15	0,12	0,09	0,07	0,04	0,03	0,03	0,02	0,005	0,004	0,003
11–12	0,02	0,47	0,97	0,11	0,08	0,06	0,04	0,03	0,02	0,02	0,02	0,01	0,004	0,003	0,002
12–13	0,01	0,03	0,64	0,03	0,02	0,02	0,01	0,01	0,01	0,006	0,002	0,001	0,001	0,001	0,001
13–14	0,006	0,02	0,08	0,02	0,01	0,01	0,009	0,005	0,005	0,002	0,001	0,001	0,001	0,001	0,001
14–15	0,004	0,01	0,007	0,005	0,003	0,003	0,002	0,002	0,002	0,002	0,001	<0,001	<0,001	<0,001	<0,001
15–16	0,002	0,002	0,003	0,003	0,001	0,001	0,001	0,001	<0,001	<0,001	<0,001	<0,001	<0,001	<0,001	<0,001
16–17	0,001	0,001	0,002	0,002	<0,001	<0,001	<0,001	<0,001	<0,001	<0,001	<0,001	<0,001	<0,001	<0,001	<0,001

Table 9

Recurrence of wave heights and wind velocities in the Okhotsk Sea, %

$\bar{W}$, m/s	$h_{3\%}$. m														
	0–1	1–2	2–3	3–4	4–5	5–6	6–7	7–8	8–9	9–10	10–11	11–12	12–13	13–14	14–15
2 – 4	3,60	13,20	2,12	0,92	0,11	0,03	0,01	0,008	0,002	0,001	0,001	0,001			
4 – 6	8,12	6,27	2,26	1,17	0,13	0,06	0,02	0,01	0,002	0,001	0,001	0,001			
6 – 8	5,00	6,43	3,69	2,41	0,31	0,08	0,05	0,02	0,005	0,003	0,003	0,001	0,001		
8–10	2,96	7,98	3,06	1,71	1,01	0,16	0,05	0,03	0,01	0,006	0,005	0,002	0,001		
10–12	0,16	7,86	2,69	1,20	0,63	0,24	0,10	0,05	0,03	0,02	0,01	0,005	0,001		
12–14	0,14	5,18	2,34	1,03	0,55	0,42	0,14	0,11	0,03	0,02	0,02	0,005	0,001	0,001	
14–16	<0,001	1,27	1,49	0,71	0,54	0,51	0,18	0,17	0,04	0,03	0,03	0,01	0,003	0,001	0,001
16–18	<0,001	0,01	0,69	0,36	0,31	0,25	0,21	0,04	0,04	0,04	0,002	0,01	0,004	0,002	0,001
18 – 20	<0,001	0,01	0,61	0,25	0,17	0,11	0,08	0,02	0,02	0,02	0,01	0,005	0,003	0,002	0,001
20 – 22	<0,001	<0,001	0,56	0,03	0,14	0,08	0,04	0,02	0,01	0,009	0,008	0,005	0,004	0,003	0,002
22 – 24	<0,001	<0,001	0,15	0,02	0,10	0,06	0,02	0,01	0,008	0,006	0,007	0,003	0,003	0,001	0,001
24 – 26	<0,001	<0,001	<0,001	<0,001	<0,001	< 0,001	0,01	0,01	0,007	0,004	0,004	0,003	0,002	0,001	0,001

APPENDIX 2

REQUIREMENTS FOR DESIGN MODELS BASED ON FINITE ELEMENT METHOD

1 GENERAL

1.1 The calculation of a structure by the finite element method generally comprises the following stages:

definition of the type and size of a problem;

drawing up of the finite element model of a structure and boundary conditions;

simulation of loads;

estimation of model correctness and calculation performance;

presentation of obtained results.

1.2 In modelling of a structure, boundary conditions and loading, depending on the calculation objectives and structure type, the certain assumptions and simplifications are possible and necessary. The particular potentials of a calculation are defined by the parameters of software and hard ware as well as by the size of a problem. The size of a problem may change with the accumulation of information on the peculiarities of a structure operation.

2 DEFINITION OF TYPE AND DIMENSIONS OF DESIGN MODEL

2.1 The type of a deflected state, and the size of a problem as it affects the simulation of a structure, boundary conditions and loading shall be defined.

2.2 For MODU/FOP structures, deformations and stresses are divided into the following types depending on external loads and structure operation conditions:

general deformations and stresses in MODU/ FOP structures;

local deformations and stresses in structural members;

concentration stresses and deformations in local zones of a structure and in intercostal members.

2.3 The objective of a calculation and the load simulation technique shall comply with one of the above types of a structure deflected state.

2.4 The problem size and thus the dimensions of a design model are defined by the model boundaries selected and by the necessary dimensions of a finite element mesh.

2.5 The problem nature (linear or nonlinear) depends on structure features and deformation values. If the parameters of a structure deflected state are determined under regulated design loads, the linear calculation is usually sufficient, particularly for thick-slab structures. Nonlinear effects are caused by material properties, large deformations and of essential importance in the following cases:

for relatively flexible structures with large deformations (geometric nonlinearity);

in investigation of partial failure of structure elements, e.g. the loss of flat panels buckling strength;

when plastic deformations in the structure area happen (physical nonlinearity).

3 STRUCTURE SIMULATION

3.1 Selection of design model types.

3.1.1 In calculations of MODU/FOP structures strength it is recommended to apply the following design models:

general model of the MODU/FOP hull;

model of a hull structure or large hull component;

model of a grillage;

framed model;

local models.

3.1.2 For all models excepting the general hull model it shall be ensured the introduction of boundary conditions for correct compliance with the conditions of interaction with adjacent structures. Where the results may adversely be affected by idealized boundary conditions, the distance between model boundaries and the structure area under consideration shall be increased.

3.1.3 The general hull model shall be used for determination of general stresses in the MODU/FOP structure. The three-dimensional simulation of the main members of a hull allows to ensure the application of loads in the form, which is the best for simulation of a real case, and to simulate the behaviour of complex hull structures with a high accuracy.

3.1.4 The model of a hull structure or large hull component (usually a three-dimensional model) shall be used for determination of general stresses in the hull part under consideration.

3.1.5 The model of a grillage shall be used for determination of general and local stresses in flat structures formed by shell plates strengthened at one or both sides with stiffeners and/or walls (grillages like a double bottom, bulkheads, decks), and also for the calculation of a transverse load transmitted to a grillage rest and for the estimation of deformations and stresses associated with it.

3.1.6 The framed model shall be used in calculation of the strength of structures deformed (mainly, bent) in their plane, e.g. of the transverse members of the MODU pontoon, FOP underwater bearing block, etc.

3.1.7 Local models are recommended for use in calculations of the strength of separate structure elements and for determination of concentration stresses in components of structures and intercostal members.

3.2 Selection of finite elements type.

3.2.1 The type of a finite element assumed in performance of the strength calculation on each particular problem is of crucial significance. So, in selection of the finite element the recommendations given below shall be followed.

3.2.2 In calculations of structure strength the following types of elements are recommended for use:

bar elements (one-dimensional elements having axial stiffness, but without flexural stiffness);

beam elements (one-dimensional elements having axial, shear, flexural and torsional stiffness);

elements of a plane stress state (two-dimensional elements having membrane stiffness in the plate plane, but without flexural stiffness about the axes in the plate plane);

plate and shell elements (two-dimensional elements having membrane, flexural and torsional stiffness);

solid elements (three-dimensional elements);

boundary and spring elements.

When different type elements are used, emphasis shall be focused on jointedness of displacements and on the possibility of ultimate loads and stresses transfer, particularly when the elements having flexural stiffness and without it are joined in nodes.

3.2.3 The element types selected shall reflect deformations and stresses for the loading conditions under analysis and, when needed, inherent values or limit states in determination of the ultimate load value.

3.2.4 It shall be defined to which extent in the given specific strength calculation the bending of structure components shall be considered. In cases of pure bending behaviour in accordance with the theory of beam bending or plate bending, particularly for flat panels, stiffeners, grillages and transverse frames, the beam and plate elements are suitable. Where the elements of a plane stress state or solid elements are used, then for a possibility to allow for bending in the plane of the largest stiffness, the finite elements with additional intermediate nodes shall be selected or a more fine mesh shall be used.

3.2.5 If general deformations and stresses are only determined, the elements of a plane stress state may be used for three-dimensional models. In this case only the membrane stiffness of a simulated flat structure is considered.

3.2.6 Structural braces of minor importance, e.g. plate stiffeners, are taken into account with the degree of conditionality which is defined by the contribution of these braces into the deflected state being analyzed.

3.2.7 If brace bending in the case considered is of importance, the flexural stiffness of the brace shall be more precisely simulated (e.g. a web is simulated by flat elements, and a loose flange, by a bar or plate element). In some cases, flexural stiffness shall be taken into account by additional beam elements.

3.2.8 In other cases, stiffeners may be considered arbitrarily in the form of an additional plate thickness. As the generalized stiffness of a strengthened plate is different in mutually orthogonal directions depending on the orientation of stiffeners, it is taken into account in design models by the introduction of orthotropic properties for the plate of an effective thickness

$$E_2 = E_1(F_{pl} + F_{st})/F_{pl}; \qquad (3.2.8)$$

$$E_1 = E$$

where E = initial modulus of elongation for a plate material;
E_1 = modulus of elasticity in the direction orthogonal to the stiffeners orientation;
E_2 = modulus of elasticity in the direction parallel to the stiffeners orientation;
F_{pl} = area of a plate cross-section;
F_{st} = area of a stiffener cross-section.

3.2.9 In local models, all stiffness components including the secondary ones, are of a significant importance; this being so, the plate, shell and solid elements are used. Exception may relate to flat structures loaded in its plane. For instance, in analysis of concentration stresses at cutout edges they are simulated by the plates of a plane stress state.

3.2.10 In order to obtain the information on deformations between two nodes, e.g. at free edges of a plate, the bar elements of a negligible cross-section shall be introduced. Uniaxial stresses of such an element present edge stresses.

3.3 Break-down into finite elements.

3.3.1 The size of a finite element mesh is defined by the characteristics of finite elements and shall be selected subject to sufficient accuracy in simulation of:

the stiffness parameters of a structure;

the type of stresses analyzed;

potential failure forms.

The recommendations given below shall be followed while selecting the dimensions of the finite element mesh.

3.3.2 In selection of a finite element mesh the structure geometry, load disposition and nature, and supports layout shall be properly taken into account.

3.3.3 The three-dimensional models of a structure as a whole or parts of the structure may have a rather gross idealization. As the characteristic size of a finite element may be used the frame spacings of the main structure components. It is allowable in calculations of a general stress state provided that the flexural behaviour of the main structure components is reflected by the selected type of a finite element with an adequate accuracy. The same relates to grillage models and to the models for the calculation of local strength of stiffeners if the width of elements in shell plates is equal to the stiffeners spacing or its half.

3.3.4 The element characteristics and its dimensions shall be so selected that stiffness, resulting deformations and stresses may properly reflect the structure behaviour. For simple finite elements the ratio of element side dimensions shall not be usually more than three.

3.3.5 In computation of local concentration stresses the size of a finite element mesh shall be varied gradually in accordance with the stress gradient expected.

3.4 Introduction of simplifying assumptions.

3.4.1 Due to the complexity of a MODU/FOP structure the assumptions aimed at simplifications shall be introduced in simulation. Simplifications are acceptable if they do not give rise to significant errors in the results.

3.4.2 A typical simplification in overall strength calculation is the integration of several components of a structure into a single one. Integration may concern stiffeners or beams. Integrated components shall have an equivalent stiffness and be placed in the geometrical centre of composing components.

3.4.3 Small components and pieces, which define the stiffness of small parts, may be completely ignored in simulation. The example of such components and pieces for the calculation of overall strength is small cutouts, frame brackets, stiffeners, reinforcements which prevent buckling.

3.4.4 Large cutouts (cutouts for access into internal spaces, windows and doors) shall be always taken into account. With a structurally stable finite element mesh, such cutouts are taken into account by the stiffness reduction at the expense of the element thickness reduction or the reduction of a modulus of rigidity and a modulus of elongation in longitudinal and transverse directions.

3.4.5 Flat elements shall be placed in the middle surface of the relevant components of a structure. For the analysis of general strength of thin-walled structures, the elements, as an approximation, may be arranged along the lines of an external surface.

3.4.6 Flat two-dimensional elements in inclined or curved surfaces shall be usually placed in the geometrical centre of the area simulated in order to reflect with the greater accuracy general stiffness characteristics

3.5 Boundary conditions and fixings.

3.5.1 Assignment of boundary conditions and fixings is intended for:

elimination of displacements and turns of a model as a rigid entity;

allowing in a design model for actually existing supports and fixings;

allowing for the interaction of the model for the part of a structure along its boundaries with adjacent parts.

Kinematic boundary conditions and fixings are introduced by the assignment of prescribed values for displacements and turning angles in nodal points of a design model. In introduction of fixings the appearance in the model of nonexistent, in actual behaviour of a structure, restrictions for displacements and turning angles shall be avoided.

3.5.2 The exception of displacements and turns of a model as a rigid body (FEM programs do not ensure the automatic exception of such displacements) shall be materialized by means of introduction of supports or fixings in various sections of the model. The reactions at these supports and fixings that are lacking in actual structures shall be kept to a minimum by means of loading the model by a self-balanced system of loads. The displacements and turns of a solid body may be eliminated by introduction in the design model of a distributed elastic foundation by means of spring elements what, for instance, may closely agree with the actual conditions of a FOP position on the seabed or the conditions of a FOP hull position afloat.

3.5.3 Actual supports that take forces and moments shall be simulated with the high degree of approximation to actual conditions.

3.5.4 The interaction of hull structure parts with adjacent structures along the model boundaries shall be simulated with the possibly high degree of approximation to a reality. The structure symmetry shall be taken into account and the model shall be developed for its symmetric part only. The conditions of a symmetric and antisymmetric deformation are introduced across symmetry planes and a load is resolved into symmetric and antisymmetric components. The interaction along a boundary shall be taken into account by the relevant assignment of stresses, forces and moments. These values are obtained as the result of a structure calculation according to a general model.

3.5.5 In use of some element types the need, due to nonexistent stiffness, in suppressing degrees of freedom in nodes may arise. In doing so, the restrictions of actual deformations are not allowed. If, with degrees of freedom suppressed, the elements give additional stiffness, their dimensions shall be so selected that they may provide the stiffness reasonably reflecting actual behaviour.

4 LOAD SIMULATION

4.1 Loads shall be simulated with a high degree of approximation to a reality. When needed, structure simulation shall be adapted for load simulation.

4.2 Distributed loads during calculations are converted into equivalent nodal forces and into nodal moments in accordance with the finite element type in use.

4.3 If the deformations along the boundary of a local model are obtained from the calculation according to the general model of a structure with a structurally stable mesh, the relevant interpolation of a deformation for the intermediate nodes of the local model shall be used. In addition, the relevant loads acting within the local area of the structure shall be applied.

5 ASSESSMENT OF RESULTS VALIDITY

5.1 The results shall be verified for validity. Such a verification includes:

special visual display of deformation for the assessment of their distribution compliance with the loads applied, boundary conditions, supports and fixings;

check of compliance of the deformation values obtained with the range expected.

5.2 It shall be checked whether the values of forces and moments at supports comply with the values expected. For self-balanced loads it shall be checked whether reaction forces are small enough to be negligible.

5.3 For local models with preassigned deformations at a boundary obtained from the general models of structures, the mutual compliance of the stress near the boundaries in question for two models shall be checked.

5.4 For nonlinear calculations the exactness of a solution in a nonlinear zone shall be checked.

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