

RULES

FOR THE CLASSIFICATION AND CONSTRUCTION OF FIXED OFFSHORE PLATFORMS

PART II HULL

ND No. 2-020201-027-E



**St. Petersburg
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RULES FOR THE CLASSIFICATION AND CONSTRUCTION OF FIXED OFFSHORE PLATFORMS

Rules for the Classification and Construction of Fixed Offshore Platforms (the FOP Rules) 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 September 2023.

The present Rules are based on the latest version of the Rules for the Classification, Construction and Equipment of Mobile Offshore Drilling Units and Fixed Offshore Platforms, 2022, taking into account the amendments and additions developed immediately before publication.

The Rules set down specific requirements for FOP and supplement the Rules for the Classification and Construction of Sea-Going Ships and the Rules for the Equipment of Sea-Going Ships.

The Rules are published in the following parts:

Part I "Classification";

Part II "Hull";

Part III "Equipment, Arrangements and Outfit";

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 "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".

REVISION HISTORY

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

For this version, there are no amendments to be included in the Revision History.

1 GENERAL

1.1 APPLICATION

1.1.1 The requirements of this Part of the FOP Rules apply to the following:

.1 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 FOP Rules.

1.1.2 This Part contains provisions aimed at ensuring the strength of 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 FOP Rules.

1.2 DEFINITIONS AND EXPLANATIONS

1.2.1 Definitions and explanations pertinent to the general terminology of the 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 Ships¹ and in Part I "Classification" of the FOP Rules.

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

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

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.

FOP hull is an aggregate of structural elements of a FOP which shall take up all the total and local, constant and alternating loads. Where a 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.

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 RS Rules/C".

1.3 SCOPE OF TECHNICAL SUPERVISION

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

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

shell plating and framing of 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 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 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 FOP operation that comply with the requirements of [2.2](#).

Data may be used, as contained in [Appendix 1](#) of the Rules for the Classification and Construction of Fixed Offshore Platforms¹, in the reference data on wind and wave regime available on the RS website, 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 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 FOP operation that may bring about a critical state of the structure. The methods of calculation shall be agreed with the Register;

.4 FOP operating manual including the following:

brief description of the unit;

list of operating modes;

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

loading conditions of a FOP in each operating mode;

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

instructions on the safe operation techniques of a FOP;

drawings with indication of the grades and strength of steels used for 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 Hereinafter referred to as "the FOP Rules".

1.4 STRUCTURAL ELEMENTS

1.4.1 The structural elements of 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 FOP.

1.4.2.1 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.

1.4.2.2 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.

1.4.2.3 Secondary elements:

inner structures not contributing to the overall hull strength;

auxiliary framing of shell plating and plating.

1.4.3 The final classification of 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.

The requirements for materials of steel structures are given in 1.5 of Part XII "Materials" of the FOP Rules.

1.5.2 Reinforced structures.

Requirements for the materials of reinforced structures shall be found under [3.2](#).

1.6 WEAR OF STRUCTURAL ELEMENTS

1.6.1 The scantlings of 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

$$\Delta s = kuT^* \quad (1.6.2)$$

where

u	=	the design wear rate, in mm per year;
T^*	=	$T/2$ for FOP structural elements which can be repaired during service;
T^*	=	T for FOP structural elements which cannot be repaired during the whole period of the platform service;
T	=	the design service period of 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 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 RS Rules/C. In so doing, the congruity of service conditions of the FOP structural elements with those of the components for which data are given in the RS Rules/C shall be borne in mind.

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 shall only be introduced for the elements that are covered by protective measures.

1.6.4.1 For the structures of 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 The requirements for welded structures and joints are given in 1.7 of Part II "Hull" of the MODU Rules.

2 STRUCTURAL DESIGN PRINCIPLES

2.1 GENERAL

2.1.1 The design of 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 FOP is given in [Section 3](#).

2.1.2 Designing 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 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 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 RS Rules/C.

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.

2.2 ENVIRONMENTAL CONDITIONS

2.2.1 General.

General requirements for description of environmental conditions of the area of operation (sea or seas, area or part of sea area) are given in 2.2.1 of Part II "Hull" of the MODU Rules.

2.2.2 Wind.

The requirements for description of wind loads are given in 2.2.2 of Part II "Hull" of the MODU Rules.

2.2.3 Waves.

The requirements for description of wave loads are given in 2.2.3 of Part II "Hull" of the MODU Rules.

2.2.4 Current.

The requirements for description of loads induced by current are given in 2.2.4 of Part II "Hull" of the MODU Rules.

2.2.5 Ice.

The requirements for description of ice loads are given in 2.2.4 of Part II "Hull" of the MODU Rules.

2.2.6 Seabed.

The requirements for description of seabed conditions on the place of operation are given in 2.2.6 of Part II "Hull" of the MODU Rules.

2.2.7 Seismic conditions.

The requirements for description of seismic conditions are given in 2.2.7 of Part II "Hull" of the MODU Rules.

2.2.8 Ambient air temperature.

The requirements for information about ambient air temperature are given in 2.2.8 of Part II "Hull" of the MODU Rules.

2.3 DESIGN CONDITIONS AND LOADS

2.3.1 Classification of loads.

Classification of loads is given in 2.3.1 of Part II "Hull" of the MODU Rules.

2.3.2 Survival conditions or extreme loads.

2.3.2.1 The loads which shall be considered in strength calculations of 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 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 of Part II "Hull" of the MODU Rules and [3.1.2.4](#)).

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 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.

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 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 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.

2.3.5 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 requirements for determination of deck loads are given in 2.3.6 of Part II "Hull" of the MODU Rules.

2.3.7 Watertight bulkhead loads.

The requirements for determination of watertight bulkhead loads are given in 2.3.7 of Part II "Hull" of the MODU Rules.

2.3.8 Wind loads.

The requirements for determination of wind loads are given in 2.3.8 of Part II "Hull" of the MODU Rules.

2.3.9 Hydrodynamic loads.

The requirements for determination of hydrodynamic loads are given in 2.3.9 of Part II "Hull" of the MODU Rules.

2.3.10 Current loads.

Mutual exposure to wave and current shall be considered in accordance with the recommendations of 3.1.5.2 of Part II "Hull" of the MODU Rules.

Current loads applied to FOP are determined in accordance with the recommendations of [3.1.2.2](#) of this Part.

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 FOP structural strength and stability on seabed.

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	–	–	Extreme wind load	Extreme current load	–
2	Extreme wave load	–	–	Extreme 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.

The requirements for mooring impact loads are given in 2.3.12 of Part II "Hull" of the MODU Rules.

2.3.13 Towing operation loads.

The requirements for towing operation loads are given in 2.3.13 of Part II "Hull" of the MODU Rules.

2.4 STRENGTH CRITERIA

2.4.1 The requirements for strength criteria are given in 2.4 of Part II "Hull" of the MODU Rules. FOP fatigue life at wave, seismic or variable ice loads is recommended to determine as for the tension leg platform (TLP) according to Formula (2.4.4.7-3) of Part II "Hull" of the MODU Rules.

2.5 STRENGTH CALCULATION PROVISIONS

2.5.1 General.

General requirements for strength calculations are given in 2.5.1 of Part II "Hull" of the MODU Rules.

2.5.2 Evaluation of common stresses.

The requirements for evaluation of common stresses are given in 2.5.2 of Part II "Hull" of the MODU Rules.

2.5.3 Girder system calculation.

The requirements for girder system calculation are given in 2.5.3 of Part II "Hull" of the MODU Rules.

2.5.4 Calculation of plates.

The requirements for calculation of plates are given in 2.5.4 of Part II "Hull" of the MODU Rules.

2.5.5 Buckling strength of structural elements.

The requirements for calculation of buckling strength of structural elements are given in 2.5.5 of Part II "Hull" of the MODU Rules.

2.5.6 Helideck strength calculation.

The requirements for helideck strength calculation are given in 2.5.6 of Part II "Hull" of the MODU Rules.

3 STRENGTH ISSUES SPECIFIC TO PLATFORMS

3.1 FIXED OFFSHORE PLATFORMS

3.1.1 General.

3.1.1.1 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 [3.1.3](#) are additionally to be met for ice strike structures under extreme loading. In this case, the criterion [2.4.2.3.2](#) for the plates of outer shell of an ice strike 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](#).

3.1.1.2 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 —

$$h_w = \Delta_{100} + 1,2(D/\lambda_{100})^{1/4}h_{100} + 1,5 \quad (3.1.1.2-1)$$

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 length 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 (for rafted or level ice, whatever is applicable/greater) —

$$h_w = 4h_{raf100} + \Delta_{100} + 0,5 \quad (3.1.1.2-2)$$

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

or

$$h_w = 8h_{levi100} + \Delta_{100} + 0,5 \quad (3.1.1.2-3)$$

where $h_{levi100}$ = thickness of level 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.

3.1.1.3 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.

3.1.1.4 Calculating FOP hull strength, the provisions of [2.5](#) shall be followed as well as the provisions of [3.1.4](#).

3.1.2 Loads.

3.1.2.1 Wave loads.

3.1.2.1.1 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](#)).

3.1.2.1.2 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 [3.1.2.1.3 — 3.1.2.1.7](#).

3.1.2.1.3 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

$$\sigma_Q^{hor} = 3 \cdot 10^{-3} \gamma (h_3)_{max} D^2 K_v t h \bar{K} H; \quad (3.1.2.1.3-1)$$

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

$$\sigma_Q^{hor} = 3 \cdot 10^{-3} \gamma (h_3)_{max} D^2 K_v t h \bar{K} H \times \left\{ 1 - \frac{4}{\bar{K} D t g \alpha} (\bar{K} H - 1/\bar{K} H + 1/sh \bar{K} H) + \frac{4}{(\bar{K} D)^2 (t g \alpha)^2} \times \right. \\ \left. \times [2 + (\bar{K} H)^2 - 2 \bar{K} H / t h \bar{K} H] \right\} \quad (3.1.2.1.3-2)$$

where	γ	=	water density, in t/m ³ ;
	$(h_3)_{max}$	=	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;
	$\bar{K} = 2\pi/\bar{\lambda}$	=	wave number;
	$\bar{\lambda}$	=	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);
	α	=	angle of a cone inclination to horizon (the leg is vertical when $\alpha = 90^\circ$);
	H	=	depth in a water area, in m.

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

$$Z_0/H = \frac{1}{\bar{K} H} \cdot (1 - ch \bar{K} H + \bar{K} H \cdot sh \bar{K} H) / sh \bar{K} H. \quad (3.1.2.1.4)$$

3.1.2.1.5 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 [3.1.2.1.7](#)) is determined as well.

3.1.2.1.6 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

$$Q = \sigma_Q \sqrt{2 \ln p N} \quad (3.1.2.1.6)$$

where	σ_Q	=	standard deviation according to 3.1.2.1.3 ;
	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).

3.1.2.1.7 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

$$M_{cap} = \sqrt{(QZ_Q)^2 + M_V^2} \quad (3.1.2.1.7-1)$$

where Q = refer to [3.1.2.1.6](#);
 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

$$M_V = \sigma_{M_V} \sqrt{2 \ln(pN)}; \quad (3.1.2.1.7-2)$$

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

$$\sigma_{M_V} = \frac{\gamma h_3 D^3}{ch(2\pi H/\lambda)} \psi_V \quad (3.1.2.1.7-3)$$

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. 3.1.2.1.7](#).

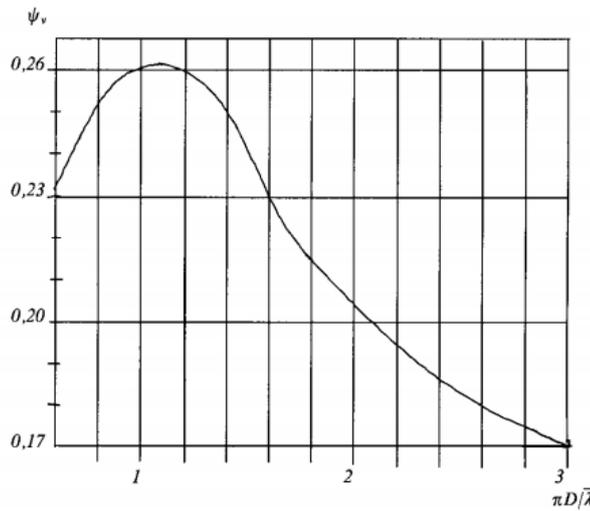


Fig. 3.1.2.1.7

Value of an additional capsizing moment parameter ψ_V

3.1.2.1.8 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

$$Q = n \sigma_Q \gamma_n \sqrt{2 \ln(pN)} \quad (3.1.2.1.8.1)$$

where n = number of legs;
 σ_Q = standard deviation according to [3.1.2.1.3](#);
 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

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

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

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

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

$$Q = (\gamma\pi h/2)D^2(shKd/chKH)\beta \tag{3.1.2.1.8.2}$$

where γ = water density, in t/m³;
 h = design wave height (with 1 % probability), 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) taken into consideration, in m;
 β = coefficient depending on the ratio $\pi D/\bar{\lambda}$ (refer to [Fig. 3.1.2.1.8.2](#)).

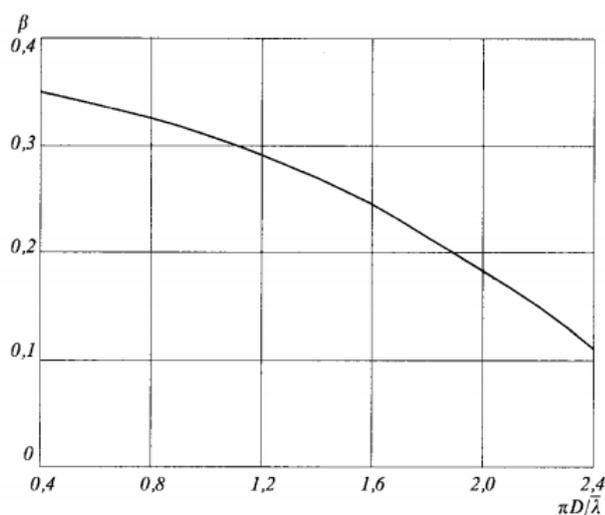


Fig. 3.1.2.1.8.2
Coefficient $\beta - \pi D/\bar{\lambda}$

3.1.2.1.9 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.

3.1.2.1.10 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 3.1.2.1.10](#); 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 3.1.2.1.10

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.

3.1.2.2 Wind and current loads.

3.1.2.2.1 Wind loads are determined by [Formulae \(2.3.8-1\) — \(2.3.8-3\)](#).

3.1.2.2.2 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

$$q = (\rho C_{sr} d/2)(v + v_c)|v + v_c|. \quad (3.1.2.2.2-1)$$

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

$$Q = (\rho C_{sr} d/2)v_c^2 H_0 \quad (3.1.2.2.2-2)$$

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.

3.1.2.2.3 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

$$Q = (\rho C_{sr} d/2) \left\{ v_c^2 H_0 + \sqrt{8/\pi} (1 + v_c^2 / \sigma_{v_0}^2) \left(\frac{g v_0 \sigma_{v_0}}{\bar{\omega}^2 + \omega^2} \right) \right\} \quad (3.1.2.2.3)$$

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.

3.1.2.3 Ice loads.

3.1.2.3.1 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.

3.1.2.3.2 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.

3.1.2.3.3 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.

3.1.2.3.4 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 (3.1.2.3.4-1), (3.1.2.3.4-2), (3.1.2.3.4-3), respectively:

$$F_{x1} = mK_L K_V \sigma_c D^{0,85} h^{0,9}; \quad (3.1.2.3.4-1)$$

$$F_{x2} = 1,33h(\rho_i D)^{1/3}(\sigma_c D_1 V)^{2/3}; \quad (3.1.2.3.4-2)$$

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

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

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/m^3 ;
 K_L = factor determined by Formula (3.1.2.3.4-4) 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 (3.1.2.3.4-5) which considers ice speed V and ice thickness h ;

$$K_L = \begin{cases} 1, & \text{at } D_1/D \geq 10; \\ 1 - 0,0667 \left(10 - \frac{D_1}{D}\right), & \text{at } 10 > D_1/D > 3; \\ 0,6 & \text{at } D_1/D \leq 3 \end{cases} \quad (3.1.2.3.4-4)$$

$$K_V = (1,6 - 20V/h), \text{ at } V/h < 3 \times 10^{-2} \quad (3.1.2.3.4-5)$$

$$K_V = 1, \text{ at } V/h \geq 3 \times 10^{-2}.$$

If the load determined by Formula (3.1.2.3.4-1) is less than that determined by Formula (3.1.2.3.4-2), the ice breaking occurs. The global load determined by Formula (3.1.2.3.4-1) 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 (3.1.2.3.4-3) exceeds the load due to ice breaking as per Formula (3.1.2.3.4-1) at speed of ice drift $V = 0,01$ m/s.

When the load determined by Formula (3.1.2.3.4-2) is less than that determined by Formula (3.1.2.3.4-1), floe stop occurs and one of three conditions is met: small floe area ($D_1 \leq 100$ m); small ice consolidation ($C_p \leq 0,7$); load due to ridging determined by Formula (3.1.2.3.4-3) is less than that due to the floe stop determined by Formula (3.1.2.3.4-2). In case of the floe stop the global load determined by Formula (3.1.2.3.4-2) is taken as characteristic load.

The ice ridging occurs when the load determined by Formula (3.1.2.3.4-2) is less than that determined by Formula (3.1.2.3.4-1), 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 (3.1.2.3.4-3) is less than that due to ice-breaking determined by Formula (3.1.2.3.4-1) at $V = 0,01$ m/s, but more than the load due to floe stop determined by Formula (3.1.2.3.4-2). When ridging occurs, the global load determined by Formula (3.1.2.3.4-3) is taken as characteristic load.

3.1.2.3.5 For the structures frozen into ice, with the water level insignificant fluctuations within the day, global loads shall be determined by Formula (3.1.2.3.4-1) 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:

$$F_{1f} = 1,6mK_L\sigma_c D^{0,85}h^{0,9}. \quad (3.1.2.3.5)$$

3.1.2.3.6 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.

3.1.2.3.7 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 .

3.1.2.3.8 The global load on the single-support platform with vertical sides due to consolidated layer of ridge shall be determined by Formulae (3.1.2.3.4-1) — (3.1.2.3.4-3) by substituting thickness of consolidated layer of ridge h_c for h and strength of consolidated layer of ridge σ_{cr} for σ_c .

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

$$F_{k1} = \mu(h_k - h_c)D \left[\frac{(h_k - h_c)\mu\gamma_e}{2} + 2C_k \right] \left(1 + \frac{(h_k - h_c)}{6D} \right); \quad (3.1.2.3.9-1)$$

$$F_{k2} = \gamma_e tg\varphi_k (h_k - h_c) [DW_k + (h_k - h_c)W_k] + C_k W_k D + 2C_k W_k (h_k - h_c); \quad (3.1.2.3.9-2)$$

$$F_k = F_{k1}, \text{ when } F_{k2} > 2F_{k1}; \quad (3.1.2.3.9-3)$$

$$F_k = 2 \frac{F_{k1}F_{k2}}{2F_{k1} + F_{k2}}, \text{ when } F_{k1} > 2F_{k2} \quad (3.1.2.3.9-4)$$

where

- $\mu = tg(45^\circ + \varphi_k/2)$;
- $\gamma_e = g(1 - \pi_k)(\rho_{wat} - \rho_i)$;
- 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/m^3 ;
- W_k = ridge keel width normal to its front, in m.

3.1.2.3.10 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.

3.1.2.3.11 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:

$$F_n = 0,85F_1K_1K_2[1 + (n - 1)(\cos \alpha_i + 0,3 \sin \alpha_i)]; \quad (3.1.2.3.11-1)$$

$$k_1 = 0,83 + 0,17n^{-1/2}; \quad (3.1.2.3.11-2)$$

$$K_2 = \begin{cases} 0,7 + 0,06(L/D - 1) & \text{at } L/D < 6 \\ 1 & \text{at } L/D > 6 \end{cases} \quad (3.1.2.3.11-3)$$

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 (3.1.2.3.4-1) — (3.1.2.3.4-5);
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. 3.1.2.3.11);
D	=	leg diameter, in m.

Formulae (3.1.2.3.11-1) — (3.1.2.3.11-3) are valid at $L/D > 2$. Otherwise, the load on the structure shall be determined as the load on extended structure with length $L(n-1)$.

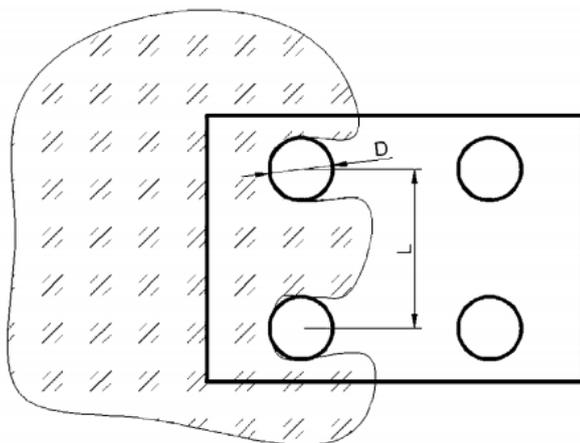


Fig. 3.1.2.3.11
Multileg platform

3.1.2.3.12 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 (3.1.2.3.11-1) and (3.1.2.3.4-2). Thus, in Formula (3.1.2.3.4-2) D shall be replaced with nD .

3.1.2.3.13 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:

$$F_n = 0,85nF_1K_1K_3; \quad (3.1.2.3.13-1)$$

$$K_3 = \begin{cases} 0,7 & \text{at } L/D > 5 \\ 0,45 + 0,05L/D & \text{at } 5 \leq L/D < 3 \end{cases} \quad (3.3.2.3.13-2)$$

where K_3 = factor considering mutual influence of legs;
for F_1 , K_1 refer to [3.1.2.3.11](#).

At $L/D < 3$ the global load shall be determined as a load on the multileg platform for ice clogging between legs (refer to [3.1.2.3.16](#)).

3.1.2.3.14 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 \(3.1.2.3.11-1\)](#) and [\(3.1.2.3.4-2\)](#) 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.

3.1.2.3.15 When the multi-row assembly of legs is frozen into ice, the global horizontal load is determined by the Formulae:

$$F_{nf} = F_1 K_{3f} \quad (3.1.2.3.15-1)$$

$$K_{3f} = n, \text{ at } L/D > 4; \quad (3.1.2.3.15-2)$$

$$K_{3f} = n(0,5 + L/8D), \text{ at } 2 < L/D > 4 \quad (3.1.2.3.15-3)$$

where F_{1f} = load on one leg frozen into ice determined by Formula [\(3.1.2.3.5\)](#).

3.1.2.3.16 The global load on the multileg platform at ice clogging between the legs is determined by the formula

$$F_{nz} = 0,8F_{nf}. \quad (3.1.2.3.16)$$

3.1.2.3.17 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 [\(3.1.2.3.13-1\)](#). Here, F_1 is calculated according to [3.1.2.3.4](#) 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 [\(3.1.2.3.9-1\)](#) 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 .

3.1.2.3.18 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:

$$F_{xc} = K_V [A_1 \sigma_f h^2 + A_2 \rho_{wat} g h D^2 + A_3 \rho_{wat} g p_1 h_r (D^2 - D_b^2)] A_4; \quad (3.1.2.3.18-1)$$

$$F_{zc} = B_1 F_{xc} + K_V B_2 \rho_{wat} g p_1 h_r (D^2 - D_b^2) \quad (3.1.2.3.18-2)$$

where σ_f = bending strength;
 D = cone diameter at the waterline level;
 D_b = cone diameter at height $h_3 = \min(h_m, h_b)$;
 h_b = upper mark of the conic part of platform (leg) (refer to [Fig. 3.1.2.3.18-1](#));

- $h_r \cong 2h$;
- 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 (3.1.2.3.18-3);
- $A_1, A_2, A_3, A_4, B_1, B_2$ = factors, which values are given in Figs. 3.1.2.3.18-2 — 3.1.2.3.18-6;
- h_m = the maximum possible height of ice crawling onto the platform, which is approximately determined by Formula (3.1.2.3.18-4);

$$K_V = \begin{cases} 1, & \text{at } V < 0,5 \text{ m/s} \\ 1 + 0,7(V - 0,5) & \text{at } V \geq 0,5 \text{ m/s} \end{cases} \quad (3.1.2.3.18-3)$$

$$h_m = \begin{cases} 3 + 4h & \text{at } D/l_c \geq 2,0 \\ 5h \sin \alpha & \text{at } D/l_c \leq 0,5 \\ 5h \sin \alpha + \frac{3+h(4-5 \sin \alpha)}{1,5} \left(\frac{D}{l_c} - 0,5 \right) & \text{at } 0,5 < D/l_c < 2,0 \end{cases} \quad (3.1.2.3.18-4)$$

where α = angle of a side inclination to horizon;

$$l_c = \left(\frac{Eh^3}{12\rho_w g(1-\nu^2)} \right)^{1/4};$$

- E = modulus of elasticity of level or rafted ice, MPa;
- ν = Poisson's ratio, $\nu = 0,3$.

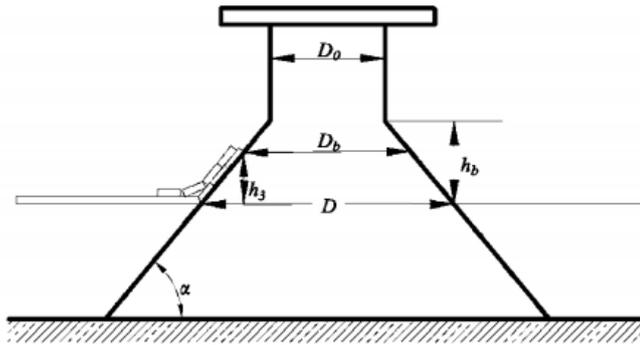


Fig 3.1.2.3.18-1
Platform with flaring conic sides

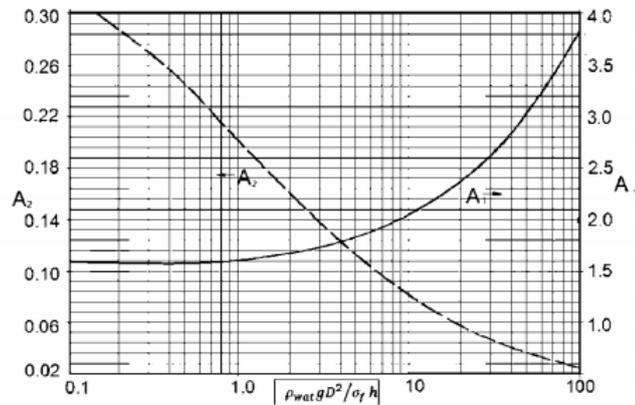


Fig. 3.1.2.3.18-2
Values of factors 1, 2

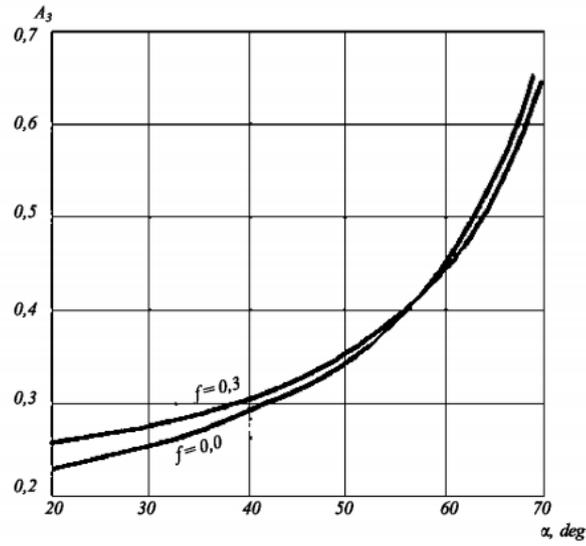


Fig. 3.1.2.3.18-3

Values of factor A_3 against the angle of side inclination for different ice friction coefficients f

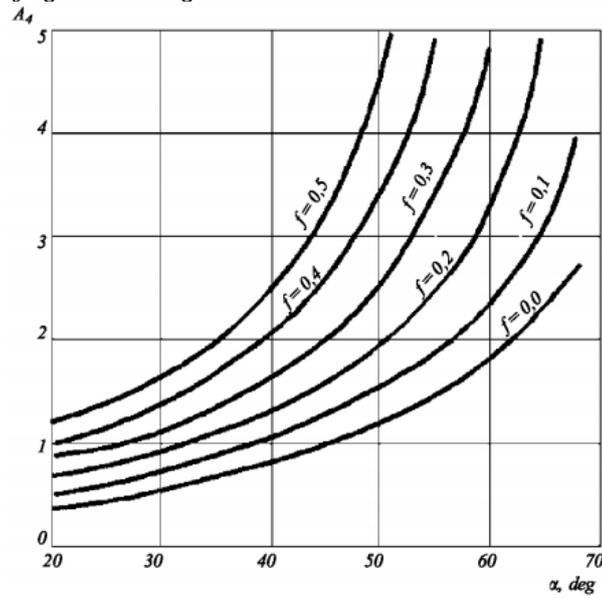


Fig. 3.1.2.3.18-4

Values of factor A_4 against the angle of side inclination for different ice friction coefficients f

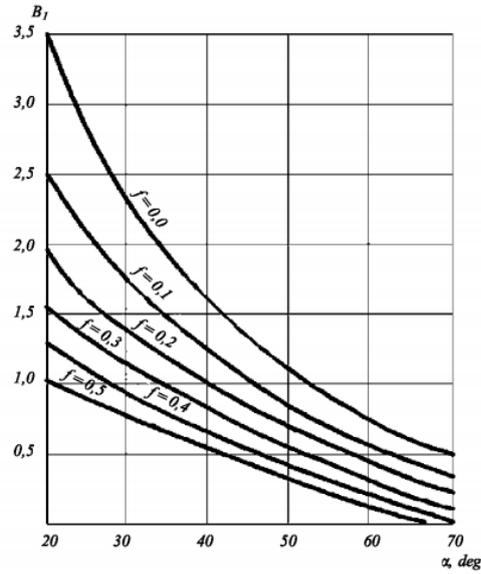


Fig. 3.1.2.3.18-5

Values of factor B_1 against the angle of side inclination for different ice friction coefficients f

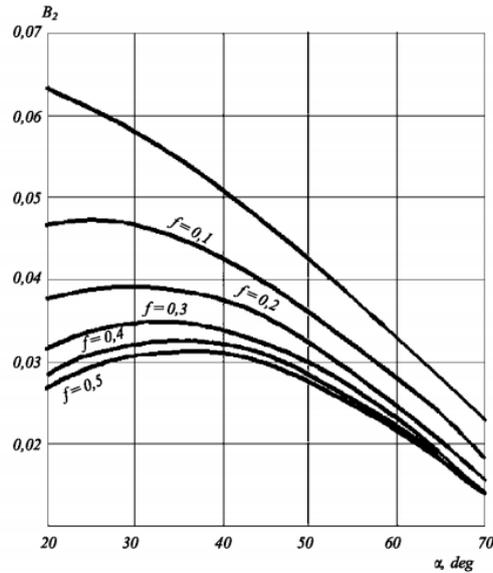


Fig. 3.1.2.3.18-6

Values of factor B_2 against the angle of side inclination for different ice friction coefficients f

3.1.2.3.19 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 (3.1.2.3.18-1), (3.1.2.3.18-2) 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).

3.1.2.3.20 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

$$F_{1f} = 1,6k_f\sigma_c D^{0,85} h^{0,9} \quad (3.1.2.3.20)$$

where k_f = factor depending on the angle of side inclination which values are given in Table 3.1.2.3.20.

Table 3.1.2.3.20

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

3.1.2.3.21 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 (3.1.2.3.18-1) — (3.1.2.3.18-4) 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

$$F_{xc} = F_k \sin \alpha, F_{zk} = F_k \cos \alpha \quad (3.1.2.3.21)$$

where F_k shall be determined by Formula (3.1.2.3.9-1).

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 (3.1.2.3.9-1).

The point of application of global forces due to consolidated layer of ridge and ridge keel shall be taken according to 3.1.2.3.10.

3.1.2.3.22 The extent of an ice strake of the platform l , in m, shall be at least

$$l = \Delta_{100} + 2\alpha_1 h_{c,100} \quad (3.1.2.3.22)$$

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.

3.1.2.3.23 Local ice pressures on the structure within an ice strake of a conic obstacle are determined by the formula

$$p = \bar{\sigma}_c \left(1 + 2 \sqrt{\frac{2}{A_1}} \right) \frac{\sqrt{\alpha}}{8,5}, \text{ MPa}, \quad (3.1.2.3.23)$$

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.; $18^\circ < \alpha < 72^\circ$.

The pressure p at $\alpha > 18^\circ$ is determined as at 18° and at $\alpha > 72^\circ$ 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.

3.1.2.3.24 Local ice pressures on the structure in areas above and below an ice strake (refer to 3.1.2.3.23) 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 3.1.2.3.23. 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 3.1.2.3.23.

3.1.2.3.25 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

$$\Delta_i = (0,0054 + 0,00146|T_{aver}|)N \quad (3.1.2.3.25-1)$$

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

$$G_1 = 0,8h_n\Delta_i g \rho_n \quad (3.1.2.3.25-2)$$

where ρ_n = density of accumulated ice, in kt/m^3 .

3.1.2.3.26 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:

$$T = \frac{600h^{0,2}D^{0,8}}{VE} \sigma_c, \text{ in s.} \quad (3.1.2.3.26-1)$$

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:

$$T = 7h/V. \quad (3.1.2.3.26-2)$$

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.

3.1.2.3.27 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.

3.1.2.4 Seismic effects.

3.1.2.4.1 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.

3.1.2.4.2 Designing structures it shall be taken into account that seismic forces may have any space orientation, horizontal and vertical inclusive.

3.1.2.4.3 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.

3.1.2.4.4 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.

3.1.2.4.5 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.

3.1.2.4.6 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.

3.1.2.4.7 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.

3.1.2.4.8 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:

$$a_p^{de} = k_\tau^{de} g A_{100}; \quad (3.1.2.4.8-1)$$

$$a_p^{mde} = k_\tau^{mde} g A_{500} \quad (3.1.2.4.8-2)$$

- 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 3.1.2.4.8-1](#);
 - 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 3.1.2.4.8-1](#);
 - 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 3.1.2.4.8-2](#);
 - A_{500} = design amplitude of a base acceleration expressed as the g fraction; the value of A_{500} is adopted according to [Table 3.1.2.4.8-2](#).

Table 3.1.2.4.8-1

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 3.1.2.4.8-2

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

3.1.2.4.9 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 $\vec{P}_i(\vec{x})$, 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 \vec{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 $P_{ijk,j} = 1, 2, 3$ are determined by the formulae:

$$P_{ijk}^{de} = 0,5k_H k_\psi m_k a_p^{de} \beta_i \eta_{ikj}, \quad (3.1.2.4.9-1)$$

$$P_{ijk}^{mde} = 0,5k_H k_\psi m_k a_p^{mde} \beta_i \eta_{ikj} \quad (3.1.2.4.9-2)$$

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 k (with due regard to the added mass of water);

a_p^{de} and a_p^{mde} = refer to [3.1.2.4.8](#);

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.

3.1.2.4.10 The value of the form coefficient η_{ikj} is determined by the formula

$$\eta_{ikj} = U_{ikj} \sum_k m_k \sum_{j=1}^3 U_{ikj} \cos(\theta_{ikj}) / \sum_k m_k \sum_{j=1}^3 U_{ikj}^2 \quad (3.1.2.4.10)$$

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 node k (with due regard for the added mass of water).

3.1.2.4.11 The values of dynamic factor β_i , are determined by the following formulae (or by the graphs in [Fig. 3.1.2.4.11](#)):

$$\left. \begin{aligned} \beta(T_i) &= 1 + \frac{T_i}{T_1}(\beta_0 - 1), \quad 0 < T_i \leq T_1; \\ \beta(T_i) &= \beta_0, \quad T_1 < T_i \leq T_2; \\ \beta(T_i) &= \beta_0 T_2^{0,5}, \quad T_2 < T_i \end{aligned} \right\} \quad (3.1.2.4.11)$$

where β_0, T_1, T_2 = parameters whose values are given in [Table 3.1.2.4.11](#);
 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 [3.1.2.4.9](#).

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.

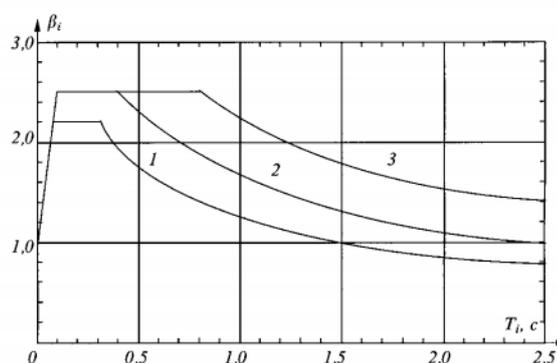


Fig. 3.1.2.4.11

 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 3.1.2.4.11

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 $20 \text{ m} < H_s < 40 \text{ m}$, are obtained by a linear interpolation between the values of these parameters at $H_s \leq 20 \text{ m}$ and $H_s \geq 40 \text{ m}$.

3.1.2.4.12 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

$$W_j = \sqrt{\sum_{i=1}^q W_{ij}^2} \quad (3.1.2.4.12)$$

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.

3.1.2.4.13 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.

3.1.2.4.14 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.

3.1.2.4.15 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.

3.1.2.5 Seabed loads applied to a gravity FOP bottom.

3.1.2.5.1 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.

3.1.2.5.2 Shear stresses across the contact surface of a FOP footing with a foundation soil caused by vertical forces are usually ignored in strength calculations.

3.1.2.5.3 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 3.1.2.5.3](#) for the main types of soils.

Table 3.1.2.5.3

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.	

3.1.3 Additional strength criteria for ice-resistant FOP structures.

3.1.3.1 The ultimate strength criterion for calculation of outer side grillages is defined by the expressions:

$$\begin{aligned} \sigma &\leq R_{eH}; \\ \tau &\leq 0,57R_e \end{aligned} \tag{3.1.3.1}$$

where σ and τ = maximum normal and shear stresses in the elements of girder cross-sections under local ice pressures.

3.1.3.2 The ultimate strength criterion for calculation of separate structural elements of an outer side (plates, stiffeners) is defined by the expression

$$P_p \leq P_{ult}/\gamma \tag{3.1.3.2}$$

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.

3.1.4 Peculiarities of strength calculation for ice strake structures.

3.1.4.1 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.

3.1.4.2 Where a structure is loaded as a whole, the global ice loads calculated according to [3.1.2.3](#) 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.

3.1.4.3 Where structure grillages are under load, the local ice pressures according to 3.1.2.3.11 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 m^2 .

Based on the grillage calculation, the dimensions of web girders are selected and the strength criterion for them indicated in [3.1.3.1](#) shall be met. In determination of stresses, the provisions of [2.5.3](#) shall be followed.

3.1.4.4 If separate structural members (plates, stiffeners) are loaded, the local ice pressures determined according to [3.1.2.3.11](#) 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

$$p_{ult} = 4R_d(s/a)^2[1 + 2(a/b)^2] \tag{3.1.4.4-1}$$

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

$$Q_{ult} = \frac{16W_u}{l} R_r \bar{Q} \quad (3.1.4.4-2)$$

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 [3.1.3](#) shall be met.

3.2 FOP REINFORCED AND STEEL CONCRETE STRUCTURES

3.2.1 General.

3.2.1.1 This Chapter sets the basic requirements for design and construction of FOP hulls made wholly or partially (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 Rules and FOP Rules as steel concrete structures and appropriate refinements are made where necessary.

3.2.1.2 Design of prestressed reinforced and steel concrete structures of FOP hulls may be executed according to specialized regulatory documents approved by the Register.

3.2.1.3 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 Concrete¹ may be used where applicable.

3.2.2 Loads.

3.2.2.1 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](#), [3.1.1](#) and [3.1.2](#).

3.2.2.2 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.

3.2.3 Key design requirements.

3.2.3.1 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.

3.2.3.2 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](#)).

¹ Hereinafter referred to as "the Concrete Rules".

3.2.4 Materials.

3.2.4.1 Concrete and its components.

3.2.4.1.1 The concrete of reinforced and steel concrete structures of FOP hulls shall meet the requirements of national standards, the Concrete Rules and the requirements of this Section.

3.2.4.1.2 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.

3.2.4.1.3 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.

3.2.4.1.4 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.

3.2.4.1.5 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 3.2.4.1.5.2](#);

Table 3.2.4.1.5.2

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 3.2.4.1.5.3](#).

Table 3.2.4.1.5.3

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.

3.2.4.1.6 For ancillary hull structures it is allowed to use light-weight concretes of compressive strength classes: B30, B35 and B40.

3.2.4.1.7 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.

3.2.4.1.8 In design of FOP, strength classes on the basis of tension may be established if specially substantiated.

3.2.4.1.9 It is not allowed to use a fine-grained concrete without an experimental justification for structures subjected to a many times repeated loading.

3.2.4.1.10 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.

3.2.4.1.11 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.

3.2.4.1.12 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 3.2.4.1.12](#).

Table 3.2.4.1.12

Concrete Design class	Characteristic strength of concrete, design strength of concrete for the 2 nd group limit states, MPa		Design strength of concrete for the 1 st 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
B40	29,0	2,10	22,0	1,40	36,0

Concrete Design class	Characteristic strength of concrete, design strength of concrete for the 2 nd group limit states, MPa		Design strength of concrete for the 1 st 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}	
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.

3.2.4.1.13 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 3.2.4.1.13-1](#). 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 3.2.4.1.13-2](#), is taken into account.

Table 3.2.4.1.13-1

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 3.2.4.1.13-2
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 3.2.4.1.5.2](#), 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 3.2.4.1.13-2

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

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
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.

3.2.4.1.14 For concretes subjected to repeated freezing and thawing the values of an initial modulus of elasticity given in [Table 3.2.4.1.12](#) shall be multiplied by the coefficient of operational conditions γ_{b3} assumed according to [Table 3.2.4.1.13-1](#).

3.2.4.1.15 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 3.2.4.1.15](#).

Table 3.2.4.1.15

Compressive strength class of concrete	B20	B30	B40	B50	B60
Reduction coefficient v'	23	18	10	8	5

3.2.4.1.16 The coefficient of linear thermal deformation of concrete d_{bt} shall be assumed in calculations equal to $1/10^{-5} \text{ }^\circ\text{C}^{-1}$.

3.2.4.1.17 The initial coefficient of lateral deformation of concrete (Poisson's ratio) μ is assumed equal to 0,2.

3.2.4.1.18 The shear modulus of concrete is assumed equal to 0,4 of appropriate values for initial moduli of elasticity of concrete specified in [Table 3.2.4.1.12](#).

3.2.4.2 Reinforcement.

3.2.4.2.1 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 cannot 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).

3.2.4.2.2 For FOP hull structures operating under severe and very severe climatic conditions (refer to [3.2.4.1.5](#)) it is not allowed to use bar reinforcement over 16 mm in diameter made of semikilled steel.

3.2.4.2.3 The characteristic strength of bar and wire reinforcement for the classes specified in [3.2.4.2.1](#) and the design strength of reinforcement for limit states of the 1st and 2nd groups (refer to [3.2.3.1](#)) depending on the loading nature, and also moduli of elasticity and relative elongations are given in [Table 3.2.4.2.3](#).

Table 3.2.4.2.3

Reinforcement class	Diameter, mm	Characteristic tensile strength (yield stress), design tensile strength for the 2 nd 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 R_{SC}			
			Longitudinal R_S	Transverse (clamps and web bars) R_{SW}				
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.

3.2.4.2.4 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](#).

3.2.4.2.5 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.

3.2.4.2.6 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 3.2.4.2.6-1](#) and [3.2.4.2.6-2](#).

Table 3.2.4.2.6-1

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 3.2.4.2.6-2

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 3.2.6.5	0,9

3.2.5 Requirements for design of FOP hulls of composite concrete-based materials.

3.2.5.1 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.

3.2.5.2 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.

3.2.5.3 A reinforcement diameter for FOP sides, decks and bottom shall be at least 12 mm, and inside internal wall members, at least 8 mm.

3.2.5.4 The minimum thickness of plate reinforcement shall be 10 mm for composite reinforced concrete structures, and 15 mm for steel concrete structures.

3.2.5.5 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.

3.2.5.6 Bar reinforcement for reinforced and composite reinforced structures shall be designed in the form of reinforced trusses, reinforced packet, welded cages and fabrics. The types of reinforced structures shall provide for the possibility of mechanized concrete supply, its thorough handling or self-consolidation.

3.2.5.7 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.

3.2.5.8 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.

3.2.5.9 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,

3.2.5.10 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 (haunches) 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.

3.2.5.11 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 [3.2.6.5](#); the use of exterior plate reinforcement with a profiled surface is also recommended.

3.2.5.12 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.

Note. 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.

3.2.5.13 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.

3.2.5.14 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.

3.2.5.15 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 given in Part II «Hull» of the RS Rules/C. In this case, the values of loads determined according to [3.2.6.5](#) of this Part are used in the calculation.

3.2.5.16 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.

3.2.6 Calculation for strength and endurance.

3.2.6.1 Basic design provisions.

3.2.6.1.1 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.

3.2.6.1.2 Calculations of the stress state of members in bending basing on the preconditions specified in [3.2.6.1.1](#) 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.

3.2.6.1.3 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:

$$F_{bl} = \sum(F_b + F_s E_s / E_b); \quad (3.2.6.1.3-1)$$

$$F_{sl} = \sum(F_s + F_b E_b / E_s) \quad (3.2.6.1.3-2)$$

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.

3.2.6.1.4 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](#)).

3.2.6.1.5 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.

3.2.6.1.6 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 [3.2.6.1.2 — 3.2.6.1.5](#).

3.2.6.1.7 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.

3.2.6.1.8 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.

3.2.6.1.9 In calculation of reinforced concrete and steel concrete members for combined torsion and bending the following condition shall be observed:

$$M_T \leq 0,1R_b b^2 h \quad (3.2.6.1.9-1)$$

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 B30 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

$$\tau_{max} \leq 1,86R_{btn} \quad (3.2.6.1.9-2)$$

where R_{btn} = characteristic strength of concrete to axial tension.

3.2.6.1.10 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.

3.2.6.2 Calculation of members strength in sections normal to the longitudinal axis of the member.

3.2.6.2.1 The calculation of members strength in sections normal to the longitudinal axis of the member shall be conducted in accordance with [3.2.3.2](#) and [3.2.6.1.8](#) observing the condition

$$\xi = x/h_0 < \xi_R \quad (3.2.6.2.1-1)$$

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

$$M < R_b S_0 \quad (3.2.6.2.1-2)$$

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 3.2.6.2.1

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

3.2.6.2.2 The calculation of sections of flexural concrete members of any symmetric form shall be performed by the formulae:

$$\Phi = M; R = \gamma_b R_b S_b + \gamma_s R_{sc} S_s; \quad (3.2.6.2.2-1)$$

$$\gamma_s R_s F_s - \gamma_s R_{sc} F'_s = \gamma_b R_b F_b. \quad (3.2.6.2.2-2)$$

For a rectangular symmetrical section (refer to [Fig. 3.2.6.2.2-1](#)):

$$S_b = bx(h_0 - 0,5x); S_s = F'_s(h_0 - a'); F_b = bx.$$

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. 3.2.6.2.2-1](#)), the compression reinforcement may be ignored in calculation.

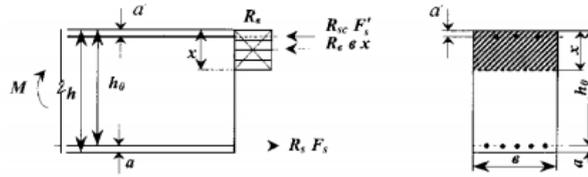


Fig. 3.2.6.2.2-1

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. 3.2.6.2.2-2](#)):

$$\Phi = M; R = \gamma_b R_b bx(h_0 - 0,5x) + \gamma_s R_{sc} F'_s(h_0 - a') + \gamma_{si} R_{si} F'_{si}(h_0 + 0,5d_{si}); \quad (3.2.6.2.2-3)$$

$$\gamma_s R_s F_s - \gamma_s R_{sc} F'_s + \gamma_{si} R_{si} F_{si} - \gamma_{si} R_{si} F'_{si} = \gamma_b R_b bx. \quad (3.2.6.2.2-4)$$

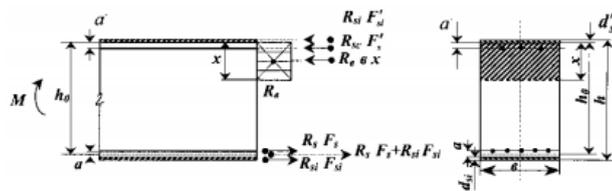


Fig. 3.2.6.2.2-2

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

$$R = (\gamma_{si} R_{si} F_{si} + \gamma_s R_s F_s)(h_0 + 0,5d'_{si}) \quad (3.2.6.2.2-5)$$

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 3.2.4);
$\gamma_b, \gamma_s, \gamma_{si}$	=	coefficients of operational conditions of concrete, bar and plate reinforcement, respectively, accepted according to Table 3.2.4.1.13-1 , 3.2.4.2.6-1 and 3.2.4.2.6-2 .

The calculation of steel concrete members shall be performed according to Formulae [\(3.2.6.2.2-3\)](#) — [\(3.2.6.2.2-5\)](#) assuming $F_s = F'_s = 0$.

3.2.6.2.3 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 [3.2.6.2.1](#) and [3.2.6.2.2](#).

3.2.6.3 Calculation of member strength in sections inclined to the longitudinal axis of the member under the action of a transverse force.

3.2.6.3.1 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:

$$Q \leq 0,25\eta\gamma_{b4}R_bbh_0 \quad (3.2.6.3.1)$$

where b = minimum sectional width of a member;
 η = safety factor (refer to [2.4.1.1](#) and [3.2.3.2](#)).

3.2.6.3.2 The calculation of member sections may be ignored if the following condition is fulfilled:

$$Q \leq \eta\delta(0,5 + 2\xi)R_{bt}bh_0 \quad (3.2.6.3.2)$$

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 $S_b \leq 0,8S_0$, —

$$\xi = \mu R_s/R_b \pm N/bh_0R_b;$$

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.

Note. The signs "plus" and "minus" shall be used for eccentrically compression and tensile members, respectively.

3.2.6.3.3 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.

3.2.6.3.4 The calculation of transverse reinforcement in the sloping section of a reinforced, composite reinforced and steel concrete member (refer to [Fig. 3.2.6.3.4](#)) shall be performed by the following formulae:

for flexural, eccentrically compressed and tensioned with a large eccentricity members —

$$\Phi = Q, R = \sum \gamma_s R_{sw} F_{sw} + \sum \gamma_{si} R_{si} F_{swi} + \gamma_b \delta (0,5 + 2\xi) R_{bt} b h_0; \quad (3.2.6.3.4-1)$$

for eccentrically tensioned members with a small eccentricity —

$$\Phi = Q, R = \sum \gamma_s R_{sw} F_{sw} + \sum \gamma_{si} R_{si} F_{swi}, \quad (3.2.6.3.4-2)$$

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 [3.2.6.3.2](#);
 $\gamma_b, \gamma_s, \gamma_{si}$ = coefficients of operational conditions of concrete and reinforcement taken according to [Tables 3.2.4.1.13-1, 3.2.4.2.6-1 and 3.2.4.2.6-2](#).

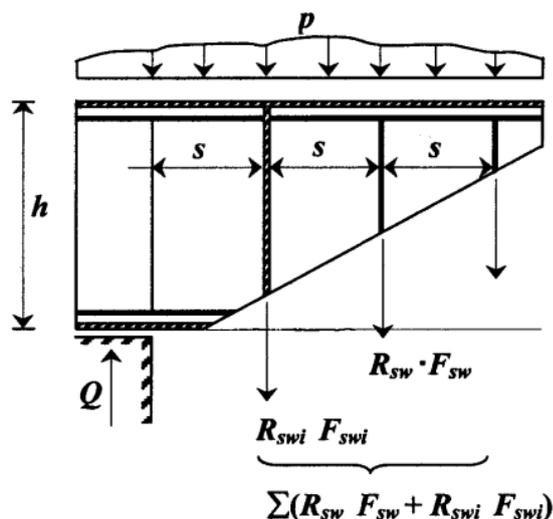


Fig. 3.2.6.3.4

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

3.2.6.3.5 In calculation of the plate members of reinforced concrete structures, having the height-design length ratio $h_0/l \leq 1/3$, for the action of a transverse force in the plate plane conditions ([3.2.6.1.9-1](#)) and ([3.2.6.1.9-2](#)) shall be met. In this case the maximum shear stresses are determined in accordance with the provisions of [3.2.6.1.5](#). 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.

3.2.6.3.6 The distance between transverse members (refer to [Fig. 3.2.6.3.4](#)), 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

$$s_{max} = \eta(0,5 + 2\xi)\gamma_b R_{bt} b h_0^2 / Q_1 \quad (3.2.6.3.6)$$

where $Q_1 = R$ being determined by Formula ([3.2.6.3.4-1](#)).

3.2.6.4 Calculation of reinforced and steel concrete structures for endurance.

3.2.6.4.1 The calculations of structure members for endurance shall be performed in all the cases when the number of loading change cycles is equal to $2 \cdot 10^6$ and more over the entire design period of the FOP operation.

3.2.6.4.2 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 3.2.4.1.13-1](#), [3.2.4.2.6-1](#) and [3.2.4.2.6-2](#)).

3.2.6.4.3 In crack-resistant members, stresses in extreme concrete fibres and in bar and plate reinforcement are determined for reduced sections (refer to [3.2.6.1.3](#)) with regard to the operation of entire section concrete and to the provisions of [3.2.4.1.15](#).

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 [3.2.4.1.15](#).

The calculation of compression reinforcement for endurance is not conducted.

3.2.6.4.4 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.

3.2.6.5 Calculation of strength of sheet steel-to-concrete joint in steel concrete structures.

3.2.6.5.1 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.

3.2.6.5.2 The strength of connection or linkage of plate reinforcement with concrete is ensured if the following condition is observed:

$$T \leq m T_{sup} n \quad (3.2.6.5.2)$$

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 [3.2.6.5.7](#) and [3.2.6.5.8](#);
 n = number of rests within the member part under consideration.

3.2.6.5.3 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:

$$T = QS_{red}u/J_{red} \quad (3.2.6.5.3-1)$$

or

$$T = N_{p(i-1)} - N_{p(i)} \quad (3.2.6.5.3-2)$$

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.

3.2.6.5.4 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 [3.2.5.12](#)) 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.

3.2.6.5.5 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

$$N_p = R_{si}F_{si} \quad (3.2.6.5.5)$$

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 ([3.2.6.5.3-1](#)).

3.2.6.5.6 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 [3.2.6.5.8](#);
- .3 with sloping anchors — reasoning from the conditions of an anchor operation in the combination of tension and bending with concrete bearing.

3.2.6.5.7 The calculation of the structure of a joint on rigid rests shall be performed by the formulae:

for strength —

$$T_{sup} \leq 1,6R_bF_{b,sm}; \quad (3.2.6.5.7-1)$$

for endurance —

$$T_e \leq 1,5\gamma_{b1}R_bF_{b,sm} \quad (3.2.6.5.7-2)$$

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.

3.2.6.5.8 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) —

$$T_{sup} \leq 0,55(t_{fr} + 0,5t_w)b_{dz}\sqrt{10R_b}, \text{ in kN}; \quad (3.2.6.5.8-1)$$

for flexible rests like round bars welded to plate reinforcement by its face end, at $2,5 \leq l/d \leq 4,2$ —

$$T_{sup} \leq 0,24ld\sqrt{10R_b}, \text{ in kN}; \quad (3.2.6.5.8-2)$$

for similar flexible rests in the form of round bars at $l/d > 4,2$ —

$$T_{sup} \leq d^2\sqrt{10R_b}, \text{ in kN}; \quad (3.2.6.5.8-3)$$

for flexible rests in the form of round bars the following condition is additionally to be fulfilled:

$$T_{sup} \leq 0,063d^2mR_\gamma, \text{ in kN} \quad (3.2.6.5.8-4)$$

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.

3.2.7 Calculation of reinforced and steel concrete structure members for cracking, crack opening and deformations.

3.2.7.1 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 [3.2.3.2](#):

.1 for centrally tensioned members, by the formula

$$\Phi = N, R = 1,5R_{bt2}F_{red} \quad (3.2.7.1.1)$$

where F_{red} = reduced area of a member cross-section;

.2 for flexural members, by the formula

$$\Phi = M, R = 1,75R_{bt2}W_{red} \quad (3.2.7.1.2)$$

where W_{red} = reduced modulus of section in bending for a tension side;

.3 for eccentrically compressed members:

$$\Phi = M/W_{red} - N/F_{red}, R = 1,75R_{bt2}, \quad (3.2.7.1.3)$$

.4 for eccentrically tensioned members:

$$\Phi = M/W_{red} - N/F_{red}, R = 1,2R_{bt2}. \quad (3.2.7.1.4)$$

3.2.7.2 The calculation for the formation of cracks inclined to the longitudinal axis of a member shall be performed by the formula

$$\Phi = \sigma_{mt}, R = 1,5R_{bt2} \quad (3.2.7.2)$$

where σ_{mt} = main tensile stresses in concrete determined according to the provisions of [3.2.6.1.5](#).

3.2.7.3 The calculations for cracking under repeated loads shall be performed reasoning from the condition

$$\Phi = \sigma_{bt}, R = \gamma_{b1}R_{bt2} \quad (3.2.7.3)$$

where σ_{bt} = maximum normal and main tensile stresses in concrete;
 γ_{b1} = coefficient of operational conditions of concrete under repeated loads.

3.2.7.4 In calculations for concrete cracking the presence of reinforcement in the compressive zone of a section may be ignored.

3.2.7.5 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

$$\Phi = a_c, R = [a_c] \quad (3.2.7.5)$$

where a_c = design width of crack opening, in mm;
 $[a_c]$ = permissible width of crack opening, in mm, (refer to [3.2.7.8](#)).

3.2.7.6 The width of crack opening a_c , (in mm) in members of reinforced and steel concrete structures shall be determined by the formula

$$a_c = 7C_d\varphi\varepsilon_s(4 - 100\mu)d^{0,5} \quad (3.2.7.6)$$

- 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 (3.2.7.6) $d_r = 1,5(F_{si}/\pi)^{0,5}$ 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 (3.2.7.6) shall be increased by 20 %.

3.2.7.7 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:

$$\tau_{max} \leq 1,86R_{bt2} \quad (3.2.7.7)$$

at that τ_{max} shall be determined in accordance with 3.2.6.1.5.

3.2.7.8 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 3.2.7.8.

Table 3.2.7.8

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

4 FOP STABILITY ON THE SEABED

4.1 GENERAL

4.1.1 The interaction of 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.

4.2 STABILITY OF FOP ON THE SEABED

4.2.1 Gravity FOP.

4.2.1.1 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 [4.2.1.2 — 4.2.1.6](#).

4.2.1.2 Criterion by conditions of gravity FOP positioning.

4.2.1.2.1 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.

4.2.1.2.2 The criterion of positioning conditions control is determined by the expression

$$N > KN_u \quad (4.2.1.2.2)$$

- 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.

4.2.1.3 Criterion of bearing capacity of FOP-base system.

4.2.1.3.1 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).

4.2.1.3.2 The criterion of bearing capacity of a platform-base system is determined by the expression

$$R/F \geq k_{s,n} \quad (4.2.1.3.2)$$

- 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.

4.2.1.3.3 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.

4.2.1.4 Criterion of ultimate eccentricity in application of loading resultant.

4.2.1.4.1 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.

4.2.1.4.2 The criterion of ultimate eccentricity is determined by the expression

$$e \leq e_{ult} k_{s,n} \quad (4.2.1.4.2)$$

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, in m.

4.2.1.5 Criterion of ultimate shifts.

4.2.1.5.1 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.

4.2.1.5.2 The criterion of ultimate shifts is determined by the expression

$$S \leq S_{ult} \quad (4.2.1.5.2)$$

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).

4.2.1.6 Criterion of value of soil pressure on skirt and inner ribs.

4.2.1.6.1 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.

4.2.1.6.2 The criterion of ultimate soil pressure determined by the expression

$$P \leq P_{ult} \quad (4.2.1.6.2)$$

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.

4.2.2 **Pile FOP.**

4.2.2.1 General.

4.2.2.1.1 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 [4.2.2.2 — 4.2.2.3](#).

4.2.2.2 Criterion of bearing capacity of pile base.

4.2.2.2.1 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

$$N \leq F_d / \gamma_k \quad (4.2.2.2.1)$$

where F_d = design bearing capacity of a single pile, in kNm;
 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), in kN.

4.2.2.2.2 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

$$N = N_d / n \pm M_x y / \sum y_i^2 \pm M_y x / \sum x_i^2 \quad (4.2.2.2.2)$$

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.

4.2.2.2.3 The design bearing capacity by pile foundation soil as a whole may be determined as the sum 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.

4.2.2.3 Criterion of ultimate deformations.

4.2.2.3.1 The criterion of ultimate deformations sets the requirements aimed at prevention of the potential hazardous state associated with the violation of normal operation conditions.

4.2.2.3.2 The criterion of ultimate deformations takes the form

$$s \leq s_{ult} \quad (4.2.2.3.2)$$

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.

4.2.2.3.3 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.

4.2.2.3.4 The horizontal load acting on a foundation with vertical piles of the same cross-section may be assumed as uniformly distributed among all piles.

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