RUSSIAN MARITIME REGISTER OF SHIPPING

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RULES

FOR THE CLASSIFICATION AND CONSTRUCTION OF MOBILE OFFSHORE DRILLING UNITS

PART II

HULL

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RULES FOR THE CLASSIFICATION AND CONSTRUCTION OF MOBILE OFFSHORE DRILLING UNITS

Rules for the Classification and Construction of Mobile Offshore Drilling Units (the MODU 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 procedural requirements, unified requirements, unified interpretations and recommendations of the International Association of Classification Societies (IACS) and the relevant resolutions of the International Maritime Organization (IMO) have been taken into consideration.

The Rules set down specific requirements for MODU, consider the recommendations of the Code for the Construction and Equipment of Mobile Offshore Drilling Units, 2009 (2009 MODU Code) (IMO resolution A.1023(26), as amended) 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 MODU Rules apply to the following:

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

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

.3 self-elevating MODU which movable legs capable of raising its hull above the surface of the sea and lowering it back into the sea.

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

1.2 DEFINITIONS AND EXPLANATIONS

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

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

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

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

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

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

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

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

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1 Hereinafter referred to as "the RS Rules/C".

1.3 SCOPE OF TECHNICAL SUPERVISION

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

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

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

watertight bulkheads and tanks;

decks and platforms;

helidecks;

superstructures and deckhouses;

jack houses;

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

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

substructure of drilling derrick.

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

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

Data may be used, as contained in Appendix 1 of this Part, 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, as stipulated in 2.3. Additional operating modes may be reviewed which agree with the features of the MODU in question;

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

.4 MODU operating manual including the following:

brief description of the unit;

list of operating modes;

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

loading conditions of a MODU in each operating mode;

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

instructions on the safe operation techniques of a MODU;

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

1.4 STRUCTURAL ELEMENTS

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

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

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

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

1.4.2 Structural elements of semi-submersible MODU.

1.4.2.1 Special elements:

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

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

main bracings intersections;

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

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

1.4.2.2 Primary elements:

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

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

1.4.2.3 Secondary elements:

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

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

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

1.4.3 Structural elements of self-elevating MODU.

1.4.3.1 Special elements:

vertical legs in way of their connections to footings;

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

1.4.3.2 Primary elements:

shell plating of tubular legs;

shell plating of all elements of truss-type legs;

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

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

1.4.3.3 Secondary elements:

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

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

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

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

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

1.4.4.1 Special elements:

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

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

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

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

structural areas subjected to considerable concentrated loads;

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

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

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

areas of ground foundation exposed to substantial loads.

1.4.4.2 Primary elements:

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

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

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

1.4.4.3 Secondary elements:

hull structures of the multi-column and tower-shaped TLP, as specified in 1.4.2.3.

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

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

1.5 MATERIALS

1.5.1 Steel structures.

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

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

Table 1.5.1.2

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

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

N o t e s : 1. For intermediate temperatures, linear interpolation is permissible.
2. Other steel marks may be used if their properties are considered sufficient to ensure the specified level of safety.
3. Steel grade selection for topsides is not regulated.

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

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

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

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

Table 1.5.1.5

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

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

1.5.2 Reinforced structures.

Requirements for the materials of reinforced structures shall be found under 3.2 of Part II "Hull" of the Rules for the Classification and Construction of Fixed Offshore Platforms[1].

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

1.6 WEAR OF STRUCTURAL ELEMENTS

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

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

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

where u = the design wear rate, in mm per year;
T^* = $T/2$ for MODU structural elements which can be repaired during service;
T = the design service period of MODU, in years;
k = the factor accounting for the positive effect of protective measures to reduce wear ($k \le 1$).

1.6.3 The design wear rate u shall be adopted on the basis of data on the wear of selected steels under conditions corresponding to the service conditions of MODU, 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 MODU structural elements with those of the components for which data are given in the RS Rules/C shall be borne in mind.

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

Table 1.6.3

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

Nos.	Structural element	Design corrosion rate, in mm per year
1	Bracings:	
1.1	Horizontal transverse bracings	
	in way of connections to columns and other bracings	0,18
	outside the areas of connection	0,16
1.2	Horizontal diagonal bracings	
	in way of connections to columns and other bracings	0,18
	outside the areas of connection	0,14
1.3	Inclined transverse bracings	
	in way of connections to columns, pontoons and upper hull	0,18
	outside the areas of connection	0,16
1.4	Inclined longitudinal bracings	
	in way of connections to columns and upper hull	0,15
	outside the areas of connection	0,14
2	Columns:	
	in way of connections to pontoons	0,14
	on the level of alternating waterline	0,16
	above-water structure	0,10
	underwater structure	0,12
3	Pontoons:	
	bottom, deck, sides of ballast and fuel compartments	0,16
	bulkheads	0,14
	bottom, deck, sides of dry compartments	0,13
4	Upper hull:	
	sides and transoms	0,11
	bulkheads	0,10
	supporting beams	0,13

Nos.	Structural element	Design corrosion rate, in mm per year
	main deck	0,10
	open sections of upper deck exposed to weather	0,13

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

1.6.4.1 For the structures of semi-submersible MODU, 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.

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

1.7 WELDED STRUCTURES AND JOINTS

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

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

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

2 STRUCTURAL DESIGN PRINCIPLES

2.1 GENERAL

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

transit;

positioning at the site;

operational;

survival or extreme loading;

removal from site.

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

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

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

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

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

Dimensions of structural elements exposed to local loads only and which do not 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.

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

2.2 ENVIRONMENTAL CONDITIONS

2.2.1 General.

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

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

2.2.1.3 The basic parameters of environmental conditions, as determined for the prescribed area of operation shall be agreed with the Register.

Data on the wind and waves in different seas are given in Appendix 1 of this Part, in the reference data on wind and wave regime available on the RS website.

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

2.2.2 Wind.

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

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

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

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

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

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

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

Table 2.2.2.3

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

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

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

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

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

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

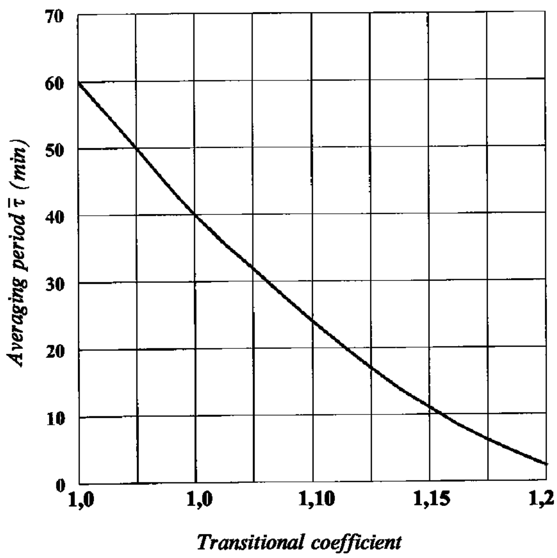

Fig. 2.2.2.4
Diagram of transitional coefficients v_τ/v_{60}

Table 2.2.2.4-1

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

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

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

where t = the time of averaging, in min,

and according to Table 2.2.2.4-2.

Table 2.2.2.4-2

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

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

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

coastal zones

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

2.2.3.6 JONSWAP spectrum is the recommended design wave spectrum:

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

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

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

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

2.2.4 Current.

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

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

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

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

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

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

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

2.2.5 Ice.

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

2.2.5.2 The following physical and mechanical properties characterize ridge keel: ridge keel adhesion value, angle of internal friction and keel hollowness factor.

2.2.5.3 The following physical and mechanical properties characterize icebergs: iceberg density and compression strength.

2.2.5.4 The following geometrical properties characterize ice: thickness of level ice, thickness of rafted ice, thickness of consolidated layer of ridge, ridge height, ridge keel depth, ridge keel width (normal to the front), ice field area and ice consolidation.

2.2.5.5 The following geometrical properties characterize icebergs: shape and linear dimensions of an iceberg with regard to their variation depending on the vertical level.

2.2.5.6 The ice field thickness and dimensions are specified for the icebergs frozen into surrounding ice.

2.2.5.7 Speed of ice drift is also the initial information for assessment of ice forces when an impact is applied by ice formations to the platform.

2.2.5.8 Physical, mechanical properties and geometrical parameters of ice as well as speed of ice drift are random values. In order to use random values in subsequent calculations, a probabilistic approach (for example, Monte Carlo method) considering laws of random value distribution in respect of specific sea area shall be applied where reasonable. Statistically valid combinations, which determine the most hazardous combinations of ice parameters, may be also used in subsequent calculations. Combination of random values shall have the recurrence period of 100 years which is determined as agreed with the Register.

2.2.6 Seabed.

2.2.6.1 For the area of installation of self-elevating MODU it is necessary to obtain engineering section of the foundation with indication of depth of stratum and information on normative and calculated values of the physical and mechanical properties of the foundation.

2.2.6.2 There are following attributes of the seabed: type of the seabed (sand, clay, silt, etc.), wet soil weight, deformation modulus (statical and dynamical), Poisson's ratio, adhesion value, angle of internal friction, C_I — non-drained shift resistance, consolidation coefficient, porosity factor, humidity factor, seabed permeability, flow index.

2.2.7 Seismic conditions.

2.2.7.1 The main information on earthquakes in the seismically dangerous region is the intensity of seismic exposure which has a recurrence period over the long period of time — at least 100 years (design earthquake).

Extreme values of seismic exposure are determined on the basis of many years' experience and they shall be extrapolated as the most probable over 500 years (maximum design earthquake).

2.2.7.2 It is recommended to use a Russian scale based on maximum accelerations (refer to Table 2.2.7.2) for evaluation of earthquake force.

Seismic exposure shall be considered if force of calculated seismic activity in the area of self-elevating MODU installation operation is 6,5 and more.

Table 2.2.7.2

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

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

Table 2.2.7.3

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

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

2.2.8 Ambient air temperature.

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

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

2.3 DESIGN CONDITIONS AND LOADS

2.3.1 Classification of loads.

2.3.1.1 By their nature, all loads affecting the offshore installation structure are grouped in two categories:

environmental loads due to waves, wind, current, ice, seismic activity, seabed, temperature etc.;

loads caused by the weight of an offshore installation and functioning of machinery, equipment, systems, and other loads associated with the operation of the offshore installation.

Each category may comprise fixed and variable loads; the latter are determined as static or dynamic depending on structural response to external effects. Depending on the relative dimensions of the exposed area each of the above-mentioned loading categories is subdivided into common (global) and local.

2.3.1.2 Fixed static loads are those which do not change in value, location or direction if environmental conditions have changed. For a structure in calm water condition the gravity forces of this structure and all permanently secured equipment, as well as the buoyancy forces, the platform footing counterpressure (weighing), soil loads and soil weight within the scope depending on the scheme of the interaction between the platform and foundation are treated as the fixed static loading.

Variable static loads are those which change in value and direction during a certain time period. However, the velocity of loading variation is so insignificant that it has practically no effect on the structure.

2.3.1.3 Dynamic loads are those which change in value and direction enough quickly to produce a dynamic effect on the behaviour of the structure. The dynamic effect on the structure may be caused by wind blows, waves, ice, seismic factors.

2.3.2 Survival conditions or extreme loads.

2.3.2.1 The loads which shall be considered in strength calculations of MODU under extreme loads include:

common and local variable and fixed extreme loads caused by environmental exposure;

common and local functional loads corresponding to extreme state of MODU in terms of safety.

2.3.2.2 For MODU the extreme variable loads are those of the possible maximum external loads which affect MODU over the whole operation period. The variable loads which possibility of excess in the long term distribution is equal to 10^{-8} are taken for the design loads.

Extreme impact loads on the transverse horizontal bracing of the semi-submersible MODU are the loads caused by impact interaction with water during sailing the opposite course relative to the main wave system stationary wave conditions with maximum $h_{3\%}$ and T_{av} in the long-term mutual distribution of heights $h_{3\%}$ and T_{av} periods of waves in the area of operation, their probability is 10^{-4} for this wave mode.

2.3.2.3 The worst practical combinations of external loads which may cause the largest tension of structures shall be considered.

2.3.3 Operating conditions.

2.3.3.1 The loads which shall be considered in strength calculations of MODU in the operating conditions include:

common and local variable and fixed environmental loads which intensity allows a MODU to perform main functions;

common and local functional loads corresponding to the operation mode.

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

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

2.3.4 Transit conditions.

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

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

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

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

2.3.5 Conditions of positioning at and removal from site.

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

2.3.6 Deck loads.

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

Table 2.3.6

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

2.3.7 Watertight bulkhead loads.

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

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

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

2.3.8 Wind loads.

Wind loads are determined by the formula

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

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

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

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

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

where ϕ = permeability factor;

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

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

where η_s = screen factor.

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

2.3.9 Hydrodynamic loads.

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

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

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

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

Table 2.3.9.2

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

2.3.10 Current loads.

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

Current loads applied to MODU are determined in accordance with the recommendations of 3.1.5.1.

2.3.11 Combination of environmental loads.

2.3.11.1 The most dangerous combinations of loads in accordance with 2.3.1 — 2.3.5 shall be considered while calculating the MODU 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.

Attention shall also be given to impact loads from supply vessels alongside the offshore installation as well as the requirements of 3.8 of Part II "Hull" of the RS Rules/C.

2.3.13 Towing operation loads.

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

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

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

When towing large-sized elements of the offshore installation, the intrinsic moment of inertia of the element shall be considered.

2.4 STRENGTH CRITERIA

2.4.1 General.

2.4.1.1 MODU shall be so designed as to meet the following general safety requirement during the service life:

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

where Φ = design value of the generalized force action (for instance, design internal forces, normal, shear or equivalent stresses, design deformations, shifts, design pressure upon plate etc.), which is used to assess marginal state;

R = design value of generalized bearing capacity (design strength of structure) determined by normative documents; this is usually the design yield strength of material or limiting pressure on elements, width of the cracks in concrete etc.;

η = safety factor which depends on the various structural elements responsibility for strength and safety of structure.

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

excessive deformations of material;

buckling;

fatigue cracks;

brittle fracture.

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

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

2.4.2 Ultimate strength criterion.

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

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

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

where σ_d = design structural stress caused by the most unfavourable combination of loads, in MPa;

η_1 = safety factor (refer to 2.4.2.5);

R_d = design yield stress of material in accordance with 1.5.1.5, in MPa.

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

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

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

where $\sigma_e = \sqrt{\sigma_x^2 + \sigma_y^2 - \sigma_x\sigma_y + 3\tau^2}$;

σ_x, σ_y and τ = components of structural stresses in the point under consideration, each of them takes into account mutual action of global and local loads;

.2 while assessing the stresses on the plate supporting contour

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

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

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

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

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

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

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

Table 2.4.2.5

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

[1] The same as for FOP.

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

2.4.3 Buckling strength criterion.

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

2.4.3.2 Buckling strength criterion is determined by the expression

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

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

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

2.4.3.4 Flexibility of isolated compressed elements shall not be more than

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

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

Table 2.4.3.4

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

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

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

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

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

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

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

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

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

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

The safety factor η_2 shall be assumed equal to:
η_2=0,8 for survival conditions or extreme loads;
η_2 = 0,6 for all other modes.

2.4.3.8 Buckling strength calculation of unstiffened tubular elements, the interrelation of common and local buckling may be omitted for:

elements which are subject to bending and compression at

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

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

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

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

If the above inequalities are not executed, then it is necessary to take into account interaction of local and common buckling in calculation of the tubular element buckling strength. Applied methods of calculation shall be agreed with the Register.

2.4.3.9 Register may consider the possibility of buckling of the horizontal plates of primary and secondary structural elements. In such case methods of calculation and permissible stresses shall be justified and agreed with the Register.

2.4.4 Fatigue strength criterion.

2.4.4.1 The fatigue strength criterion stipulates requirements aimed at preventing the origination of dangerous, by possible consequences, fatigue damage during service life of the structure which is caused by unsteady change of operating loads of different magnitude.

2.4.4.2 Calculation of the fatigue strength is made for critical points which list is agreed by the designer with the Register.

2.4.4.3 Designing of the platform structures shall be made on the basis of the "safe damage" criterion which implies that fatigue criterion is oriented at the stage of initiation of fatigue macrocrack rather than the stage of crack development. Crack initiation criterion is based on the hypothesis of linear damage summation shown by the expression

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

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

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

2.4.4.5 Parameter η represents a product of

$$\eta = \beta_1 \beta_2$$

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

Table 2.4.4.5-1

β_1 coefficient

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

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

Table 2.4.4.5-2

β_2 coefficient

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

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

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

self-elevating MODU

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

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

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

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

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

semi-submersible MODU

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

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

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

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

where T_{e_i} = effective period of *i*-th process;

a_i and K_i = parameters of scale and form of the *i*-th process (refer to 3.1.2.1.4, 3.1.2.4.4, 3.1.2.3.3 and 3.1.2.3.5 of Part II "Hull" of the FOP Rules);

N_y, σ_y, m = parameters of fatigue curve; σ_y = fatigue limit on the basis of N_y cycles;

m = slope of fatigue curve in coordinates $\lg\sigma - \lg N$.

2.4.4.8 To assess preliminarily the risk of origination of fatigue damage and to determine the main scantlings of hull structures it is recommended to use the Register modified fatigue curves as fatigue curves (refer to Figs. 2.4.4.8-1 and 2.4.4.8-2) in accordance with the international classification of structural types of nodes and joints (classes B, C, D, E, F, F_2, G, W and T).

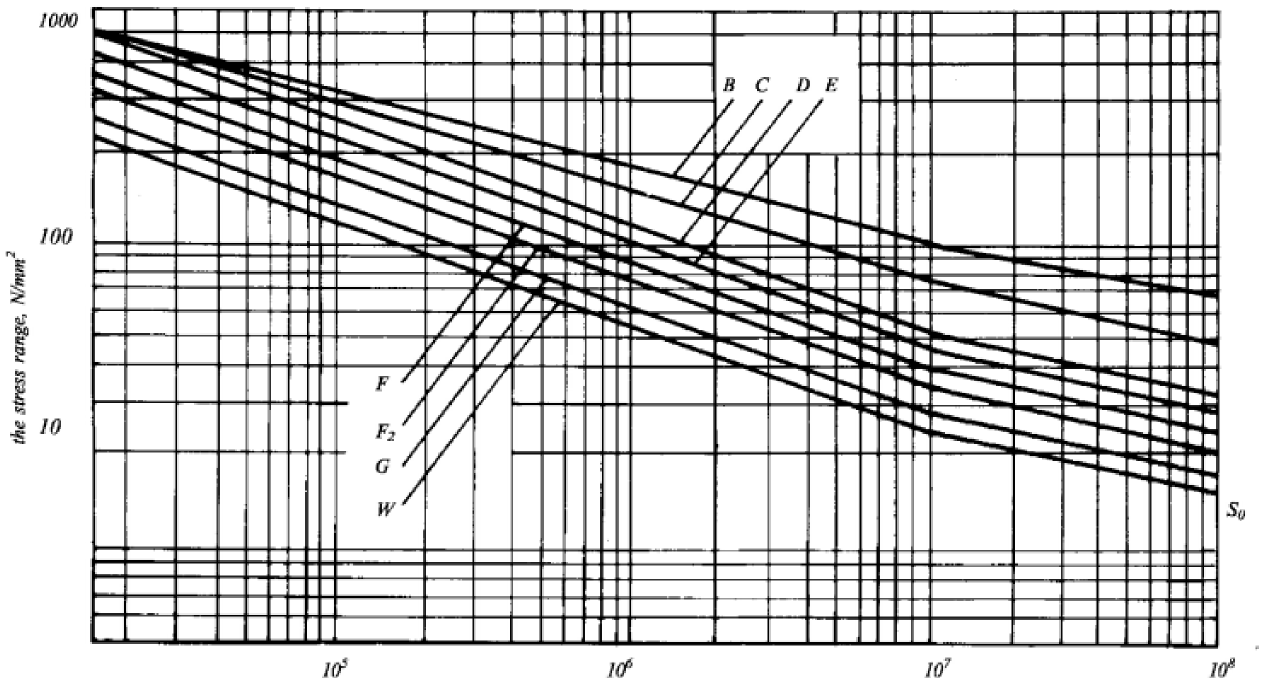

Fig. 2.4.4.8-1
Fatigue curves

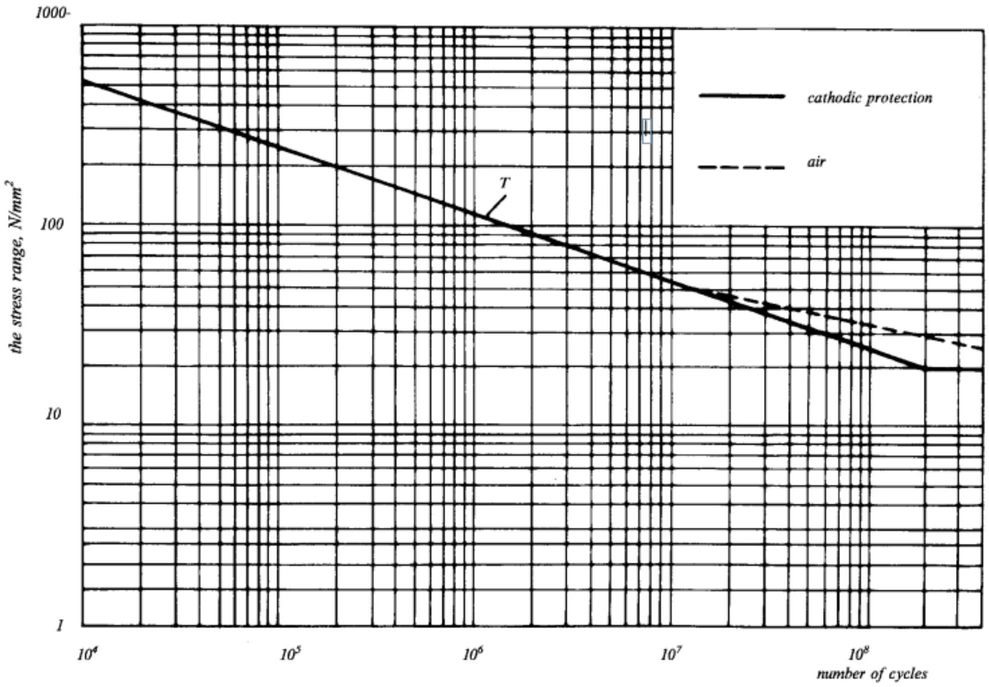

Fig. 2.4.4.8-2
S-N curves for tubular joints

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

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

where t_B = basic thickness;
t = real thickness.

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

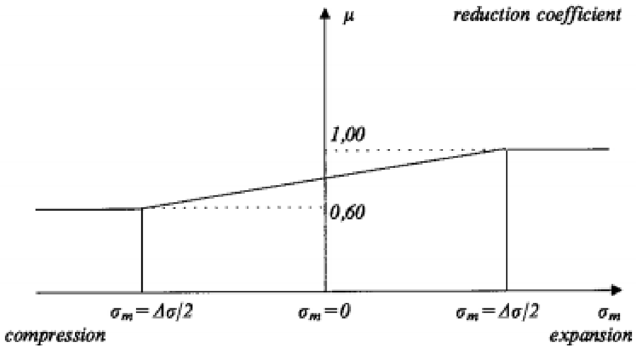

Fig. 2.4.4.10
The stress range for $S-N$ curves. The base metal

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

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

2.5 STRENGTH CALCULATION PROVISIONS

2.5.1 General.

2.5.1.1 Strength calculations are divided into the following stages:

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

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

evaluation of design stresses or evaluation of ultimate loads;

comparison of design values versus requirements of the MODU Rules.

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

2.5.1.2 Calculations shall be made following the recognized methods. Provisions of the RS Rules/C shall be used if applicable.

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

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

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

2.5.1.5 Additional provisions on strength calculations referred to offshore installation of specific type are given in the relevant paras of Section 3.

2.5.2 Evaluation of common stresses.

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

Use of simplified calculation schemes (for parts of structures) is allowed if their use is justified.

In any case the model shall be detailed as far as it is necessary for evaluation of common stresses.

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

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

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

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

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

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

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

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

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

2.5.3 Girder system calculation.

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

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

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

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

$$b_{fl} = l/6; \qquad (2.5.3.2\text{-}1)$$

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

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

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

$$b_{fl} = kb \qquad (2.5.3.2\text{-}3)$$

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

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

Table 2.5.3.2

n	l_{sp}/b						
	1	2	3	4	5	6	7 and more
≥6	0,38	0,62	0,79	0,88	0,94	0,98	1,0
≤3	0,21	0,40	0,53	0,64	0,72	0,78	0,80

Note. For intermediate values of l_{sp}/b and n, coefficient k is determined by means of the linear interpolation.

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

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

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

2.5.4 Calculation of plates.

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

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

2.5.5 Buckling strength of structural elements.

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

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

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

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

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

where σ_e = the Euler normal stress, in MPa.

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

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

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

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

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

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

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

$\lambda = l_e/\rho$;

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

$\sigma_{cr,i}$ = inelastic column critical buckling stress, in MPa;

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

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

2.5.6 Helideck strength calculation.

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

3 STRENGTH ISSUES SPECIFIC TO PLATFORMS

3.1 SELF-ELEVATING MODU

3.1.1 General.

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

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

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

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

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

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

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

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

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

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

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

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

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

3.1.2 Design structural diagram of self-elevating MODU.

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

structural frame ("superelement");

leg section under consideration.

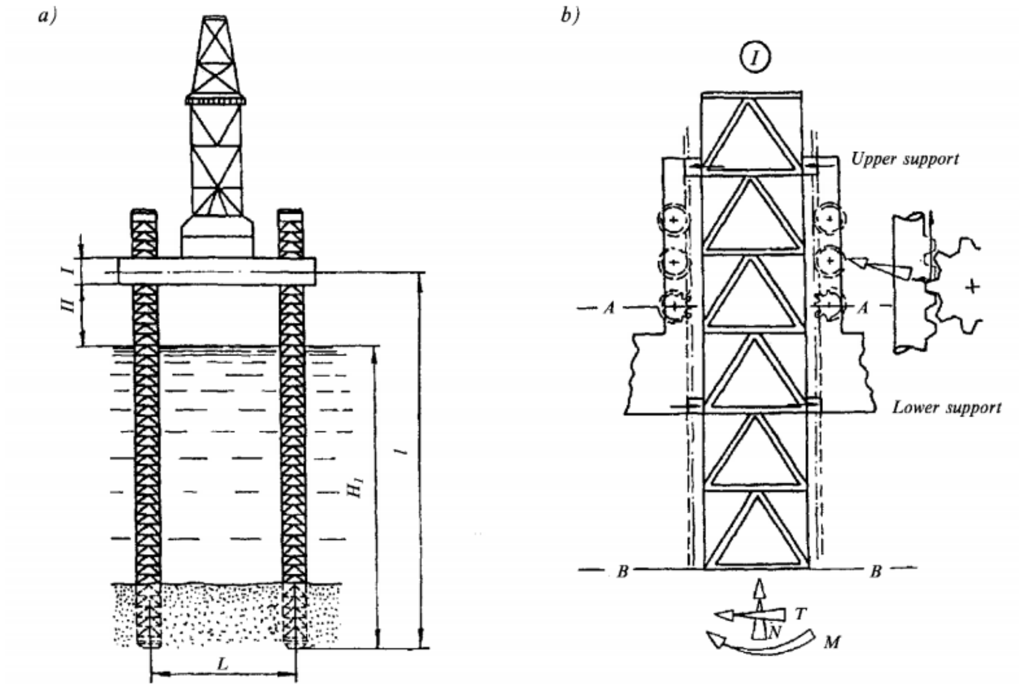

Fig. 3.1.2.1
Idealization of self-elevating MODU structures of different levels:
a) structural frame ("superelement"); *b)* leg section (I) under consideration

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

$$æ = 1/(1 + AEJ_k/l) \tag{3.1.2.2}$$

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

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

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

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

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

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

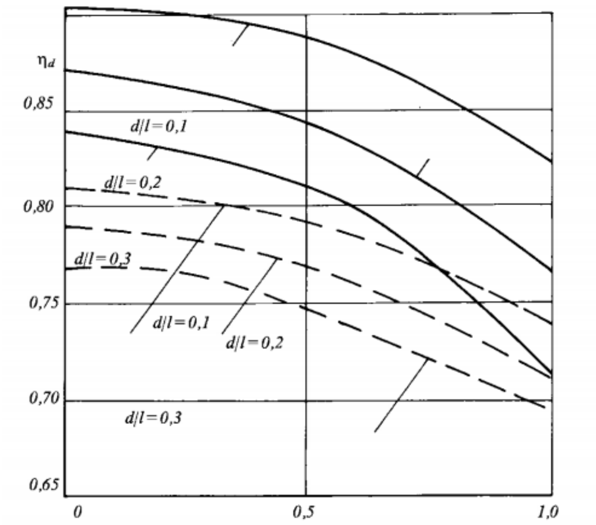

Fig. 3.1.2.4
Correlation between the correction factor η_d and the parameters æ, d/l, B/l
—— $B/l = 0{,}1$ – – $B/l = 0{,}2$

3.1.2.5 The flexibility coefficients A shall be determined by the formulae:
for vertical vibrations —

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

for horizontal vibrations —

$$A = (2 - v)/8Gr_0; \qquad (3.1.2.5\text{-}2)$$

for rotational vibrations —

$$A = 3(1 - v)/8Gr_0^3; \qquad (3.1.2.5\text{-}3)$$

for torsional vibrations —

$$A = 3/16Gr_0^3 \qquad (3.1.2.5\text{-}4)$$

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

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

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

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

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

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

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

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

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

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

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

$$\beta = \frac{1}{1+\Delta_{z_g} GF_c/K_m} \tag{3.1.2.6-2}$$

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

$$K_m = \frac{1}{2}Kb^2 \tag{3.1.2.6-3}$$

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

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

3.1.3 Wind loads.

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

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

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

$$v_w = \omega_{max}/p \tag{3.1.3.2}$$

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

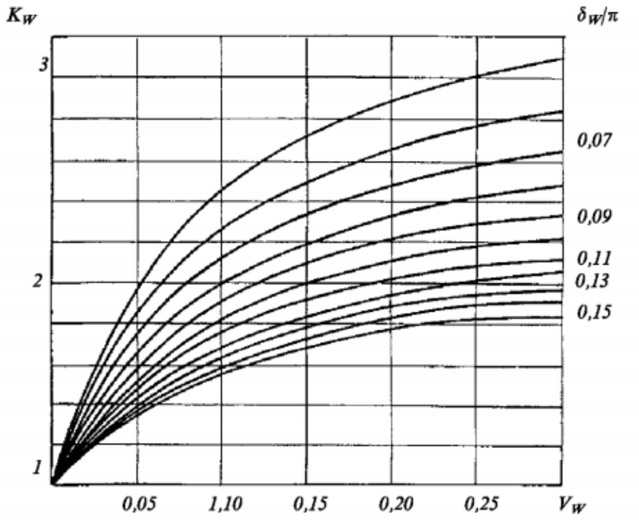

Fig. 3.1.3.2
Amplification factor of wind loads

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

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

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

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

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

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

for a four-legged self-elevating MODU —

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

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

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

for a three-legged self-elevating MODU —

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

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

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

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

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

for four-legged self-elevating MODU —

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

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

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

for three-legged self-elevating MODU —

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

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

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

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

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

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

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

3.1.4 Wave loads.

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

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

where D = bore diameter,

a = tooth root height;

b = rack tooth height.

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

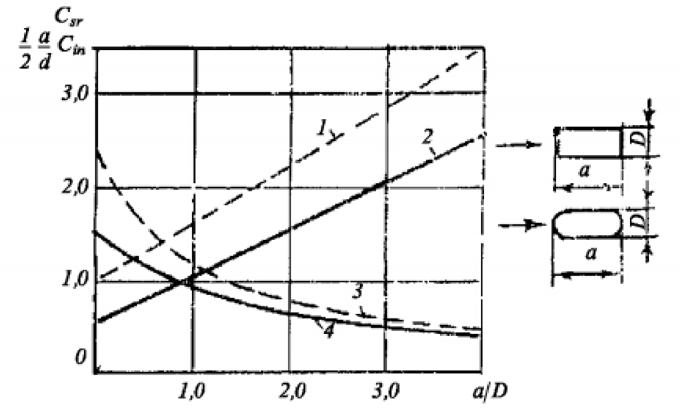

Fig. 3.1.4.1
Speed resistance coefficient C_{sr} (3,4) and inertia resistance coefficient C_{in} (1,2) for sections:
——— elliptical sections (round sections where $a/D = 1$);
- - - - - rectangular sections

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

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

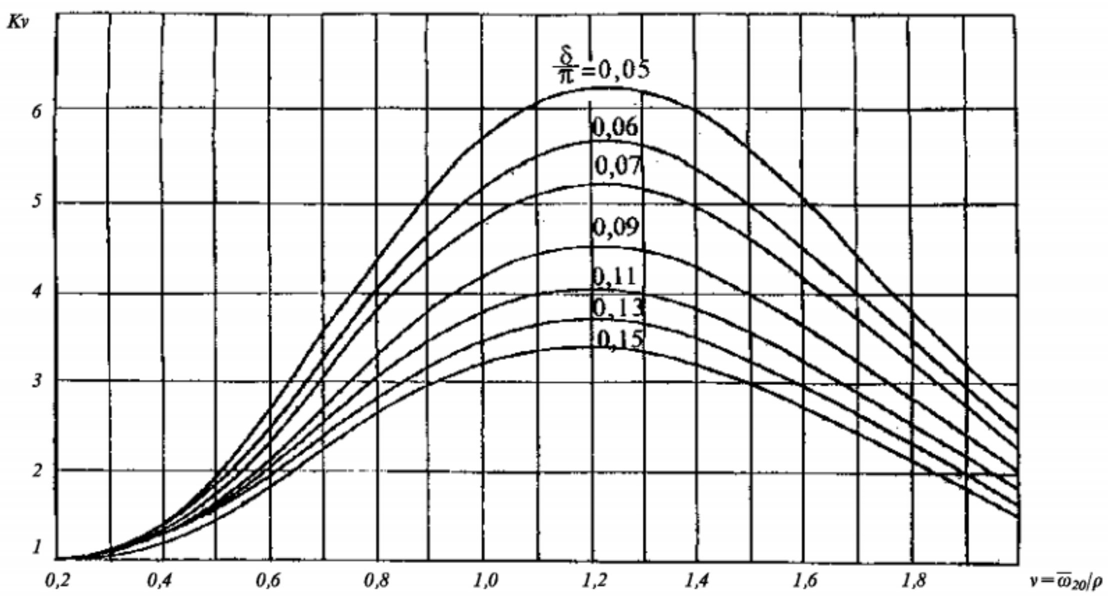

Fig. 3.1.4.3
Amplification factors of speed and inertia components of wave load

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

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

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

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

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

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

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

speed component where

$$h_3 \leq 2{,}1 d_B C_{in}^B \cdot \bar{u}_{in} / C_{sr}^B \mu_{sr} \bar{u}_{sr} \quad (3.1.4.5\text{-}2)$$

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

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

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

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

z = to be measured from the footing upwards, in m;

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

$\overline{\omega_0}$ = average frequency of surface waves;
H_1 = distance, in m, from leg footing to calm water level.

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

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

for three-legged self-elevating MODU —

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

where i = direction number.

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

for four-legged MODU —

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

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

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

for three-legged MODU —

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

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

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

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

m_k = number of vertical members;

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

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

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

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

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

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

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

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

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

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

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

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

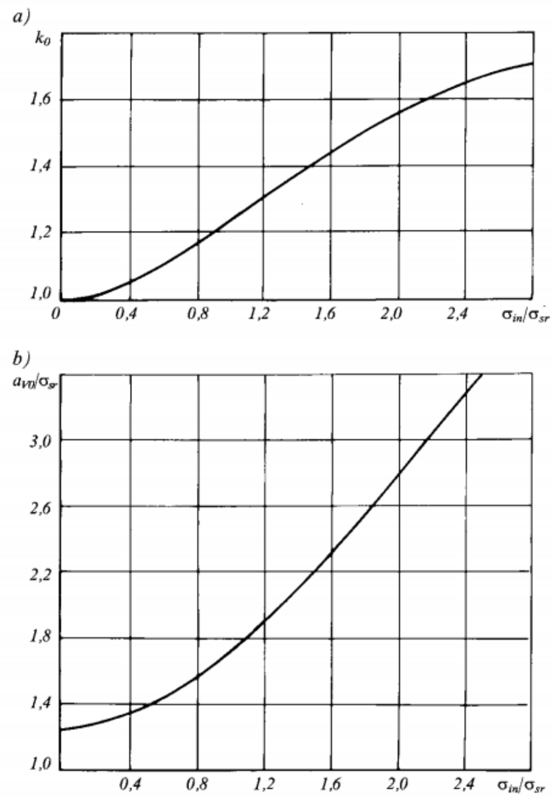

Fig. 3.1.4.9
Dependence of Weibull distribution parameters k_0 and a_{v_0} on the relationship σ_{in}/σ_{sr}

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

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

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

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

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

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

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

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

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

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

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

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

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

$$M_{\Sigma} = M_e + M_p + \bar{M}_w + M_c; \tag{3.1.4.11-1}$$

$$T_{\Sigma} = T_e + \bar{T}_w + T_c; \tag{3.1.4.11-2}$$

$$N_{\Sigma} = N_e + N_p + \bar{N}_w + N_c \tag{3.1.4.11-3}$$

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

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

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

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

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

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

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

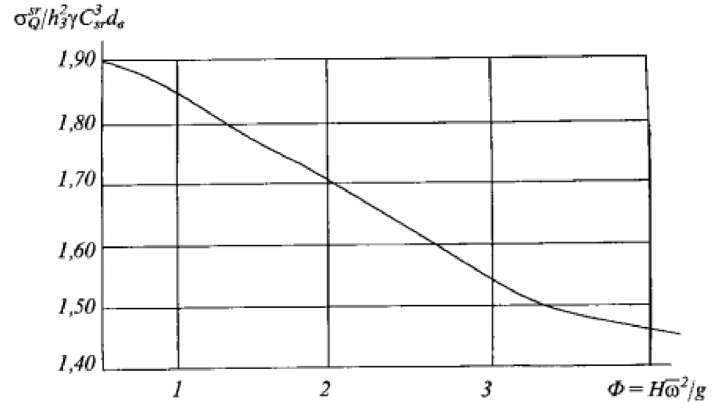

Fig. 3.1.4.13-1
Relationship between $\sigma_Q^{sr}/h_3^2\gamma C_{sr}^3 d_B$ and parameter $\Phi = H\bar{\omega}^2/g$

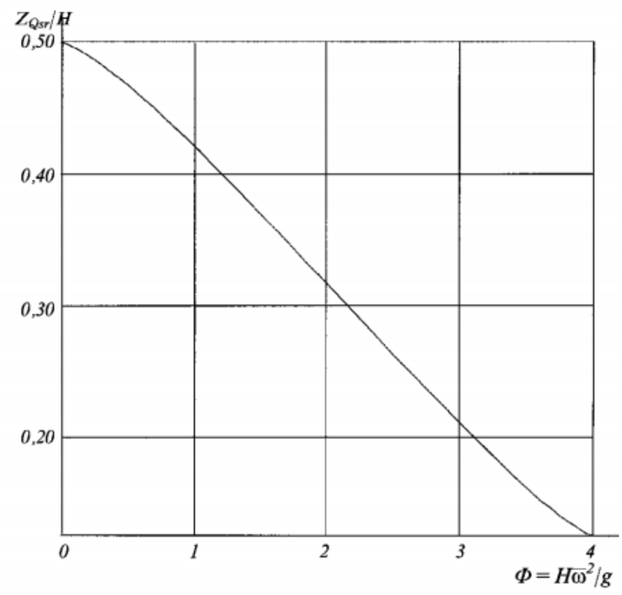

Fig. 3.1.4.13-2
Relationship between $Z_{Q_{sr}}/H$ and parameter $\Phi = H\bar{\omega}^2/g$

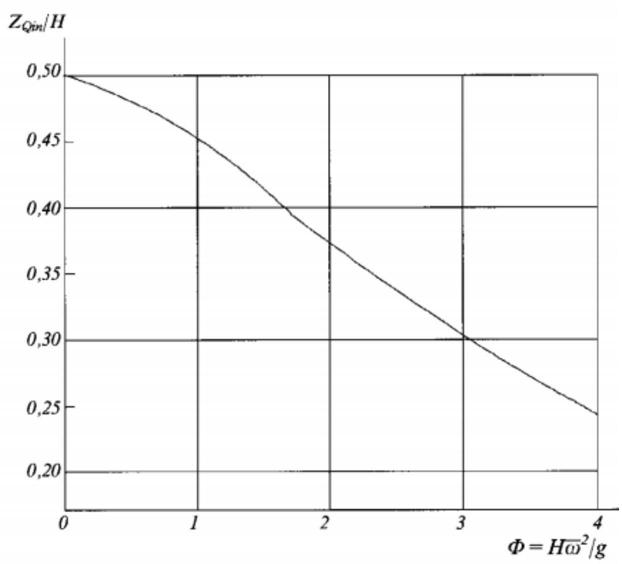

Fig.3.1.4.13-3
Relationship between $Z_{Q_{in}}/H$ and parameter $\Phi = H\bar{\omega}^2/g$

3.1.5 Current loads.

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

3.1.6 Seismic loads.

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

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

$$Q = M_{po}\beta_{\Sigma}a_{max} \tag{3.1.6.1}$$

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

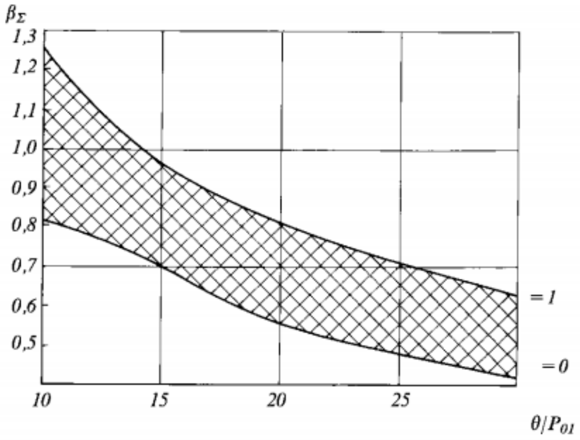

Fig. 3.1.6.1
Generalized dynamic coefficient:
θ — bearing frequency of axilogram;
P_{01} — lowest frequency of horizontal vibrations

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

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

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

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

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

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

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

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

in case of earthquakes occurring once in 500 years —

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

3.1.7 Pre-loading and pulling out of the legs.

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

The analysis shall be aimed at the following:

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

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

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

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

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

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

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

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

3.1.8 Stressed condition of legs.

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

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

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

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

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

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

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

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

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

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

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

3.1.9 Loads on self-elevating MODU legs in transit

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

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

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

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

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

specified wind forces.

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

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

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

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

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

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

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

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

transverse forces —

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

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

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

longitudinal forces —

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

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

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

$\varepsilon_0 = 2\pi/T_0$

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

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

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

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

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

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

The radius of inertia r_0 may be determined as follows:

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

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

$M_m = nM_L + M_H$ – mass;

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

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

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

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

only one leg touches seabed;

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

seabed is extremely hard.

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

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

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

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

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

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

3.1.11 Ice strength of self-elevating MODU legs.

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

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

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

3.1.12 Vortical loads.

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

in the direction normal to the wind flow

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

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

in the direction along the wind flow

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

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

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

where v = air kinematic viscosity coefficient.

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

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

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

3.2 SEMI-SUBMERSIBLE MODU

3.2.1 General.

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

survival;

operation;

transit;

according to 2.4.

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

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

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

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

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

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

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

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

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

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

3.2.2 Global loads.

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

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

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

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

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

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

The relevant horizontal distributed load, in t/m,

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

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

3.2.2.3.2 The integral characteristic of the asymmetrical component

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

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

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

and

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

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

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

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

The relevant vertical distributed forces, in t/m:

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

are applied in the centre plane of the pontoons.

3.2.2.3.4 The integral characteristic of the symmetrical vertical component

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

The relevant vertical distributed forces, in t/m:

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

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

are applied in the centre plane of the pontoons.
The relationships under 3.2.2.3.1 — 3.2.2.3.4 include the following parameters:

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

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

Table 3.2.2.4

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

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

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

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

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

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

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

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

$C_{in} = 1 + C_m$.

3.2.2.8 When using semi-submersible MODU in ice-covered water areas, their interaction with different ice formations typical for specific time period in this sea area shall be considered. The components of global ice loads shall be calculated by the methods specified in 3.1.2 of Part II "Hull" of the FOP Rules with regard to the fixed platforms.

3.2.3 Local loads.

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

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

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

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

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

for submerged section of structure —

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

for above-water section of structure —

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

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

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

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

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

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

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

g = gravity acceleration, in m/s²;

p = sea water density, in t/m³.

3.2.3.5 Impact loads upon bracings.

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

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

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

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

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

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

where D = bracing diameter, in m.

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

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

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

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

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

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

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

3.2.3.6 When using semi-submersible MODU in ice-covered water areas, local ice loads determined similarly as for the fixed structures (refer to 3.1.2.3.23 and 3.1.2.3.24 of Part II "Hull" of the FOP Rules) shall be considered.

3.2.3.7 Dynamic aspects of ridged ice loads on MODU shall be additionally studied when non-dimensional parameter $1/3 < \chi < 3$ which determines the relative proximity of ice load frequency to natural frequency of the flexible structure is given by the following formula:

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

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

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

3.2.4 Determination of deflected mode.

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

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

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

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

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

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

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

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

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

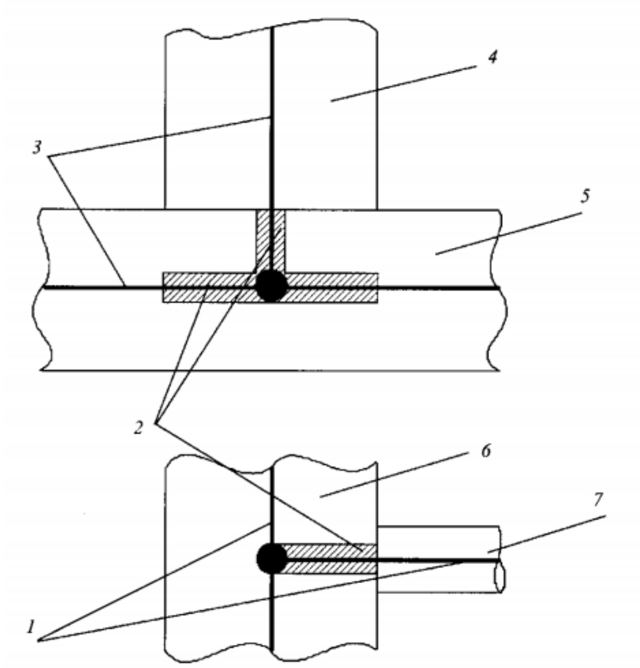

Fig. 3.2.4.3.4
"Absolutely rigid" finite elements:
1,3 — bar finite elements; *2* — "absolutely rigid" finite elements; *4, 6* — stability column;
5 — pontoon; 7 — bracing

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

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

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

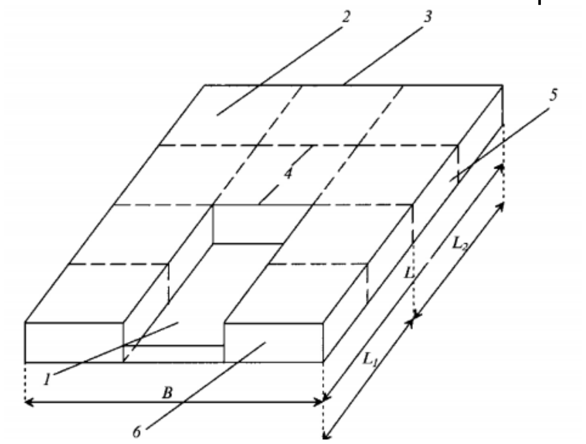

Fig. 3.2.4.4.1-1
Plan of upper hull structure (example);
B — upper hull breadth; L — upper hull length; L_1, L_2 lengthwise span between stability column axes;
1,2 — deck; *3,6* — transom; *4* — bulkheads; 5 — side

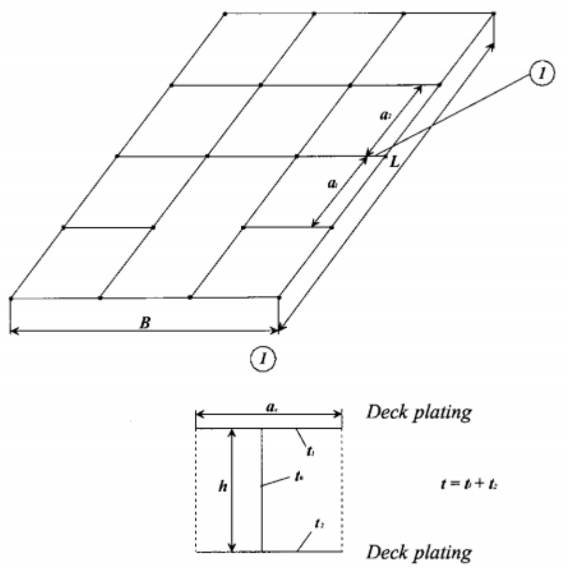

Fig. 3.2.4.4.1-2
Beam model of upper hull (example):
1 — bar element considered and its cross section; a_x — effective flange breadth; a_1 and a_2 — distance from the element in question to the nearest element arranged in the same direction;
B — upper hull breadth; L — upper hull length

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

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

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

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

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

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

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

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

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

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

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

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

The effective flange width a_z shall be determined by the formula

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

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

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

The inertia moment J_T, m^4, is determined by the formula

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

where S = area, in m^2, of a closed contour formed by the plating of decks and sides (transoms, bulkheads) of the closed system considered;
dl and t_c = element of the contour perimeter length and its web thickness, in m, at the perimeter point under consideration;
n_T = number of transverse elements forming the closed system, for the purpose of determining J_T for transverse elements, or the number of longitudinal elements forming the closed system, for the purpose of determining J_T for longitudinal elements.

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

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

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

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

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

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

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

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

The effective flange width is determined by the formula

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

where for $a_1,\ a_2$, refer to 3.2.4.4.2.

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

3.3 OFFSHORE PLATFORMS WITH CONSTANT PLACE OF OPERATION

3.3.1 In designing MODU which operation is expected to be at one location only during the entire life cycle, the requirements of 3.1 of Part II "Hull" of the FOP Rules may be used where applicable.

3.4 REINFORCED AND STEEL CONCRETE STRUCTURES

3.4.1 If MODU hull is made wholly or partially of reinforced concrete and/or steel concrete structures, it shall comply with the requirements of 3.2 of Part II "Hull" of the FOP Rules.

3.5 TENSION LEG PLATFORMS

3.5.1 General.

3.5.1.1 TLP consists of the following three basic components (structural elements):

hull;

tension legs;

ground foundation (anchor).

Designing TLP structural elements shall be carried out keeping due note of the acceptable requirements of Sections 1 and 2, considering additional instructions and regulations contained in this Chapter.

3.5.1.2 The structural strength of TLP shall be tested on the basis of criteria mentioned under 3.5.3 for the following design conditions:

extreme load;

operational;

transit;

positioning;

removal from site;

replacement of tension legs, if provided for during operation period.

Replacement of tension legs means that one (or more) tension leg may be removed for survey, maintenance or replacement. This mode shall be determined considering anticipated frequency of the leg removal and duration of out-of-service period.

3.5.1.3 The TLP clearance h_c, in m, shall be not less than the largest of the values determined by the formulae:

$$h_c = \Delta_{100} + 1{,}2(D/\lambda_{100})^{1/4}h_{100} + 1{,}5; \qquad (3.5.1.3\text{-}1)$$

$$h_c = \Delta_{100} + 4h_{raf_{100}} + 0.5 \qquad (3.5.1.3\text{-}2)$$

where Δ_{100} = peak amplitude of a sea level change, which is probable once in 100 years, including storm surge, in m;

h_{100}, λ_{100} = wave height and length, respectively, which are probable once in 100 years, in m;

$h_{raf_{100}}$ = thickness of rafted ice, which is probable once in 100 years, in m;

D = diameter or the cross dimension of a conic leg at the waterline level, in m.

3.5.1.4 In addition to the requirements of 1.3 the following TLP structures are subject to technical supervision during manufacture and positioning:

tension legs;

ground foundation.

All the requirements set out in the above paragraph are applicable to the mentioned structures.

3.5.1.5 If the design technical requirements specify that the tension legs are subject to replacement during operation period, i.e. their life cycle is shorter than that of a TLP, the wear margin for tension legs shall be determined basing upon the actual life cycle.

3.5.1.6 Requirements for the materials of TLP steel structures are set out in 3.5.5. Requirements for the materials of reinforced concrete or composite structures of ground foundation and TLP hull ice belt shall correspond to 1.5.2.

3.5.1.7 The requirements of this Chapter to the tension legs are drawn up mainly for the legs consisting of steel tubular elements. For design of tension legs consisting of ropes or chain cables one shall consider the requirements of Part III "Equipment, Arrangements and Outfit" as well as the requirements of this Part, as far as applicable.

3.5.2 Loads.

3.5.2.1 When determining wind, current, ice, seismic, deck and mooring loads for a TLP and its elements one shall consider the requirements of 2.2, 2.3.6 — 2.3.10, 2.3.12, 2.3.13 and 3.1.2 of Part II "Hull" of the FOP Rules. When drawing up load combinations it is recommended to take into account the requirements of 2.3.11 of this Part. Additional requirements to be considered in determining loads are set out in 3.5.2.2 — 3.5.2.6 of this Part.

3.5.2.2 Alternating wind loads.

The relation between maximum w_{max} and average $\overline{w}$ velocities is determined by the gustiness coefficient G similarly to 2.2.2.3.

Keeping in mind that the profile of the TLP above water hull consists of extended elements, the pulse component of wind loads Q_w shall be determined considering coefficient of wind pulse correlation. General coefficient of correlation η shall be accepted equal to 0,8.

3.5.2.3 Wave loads.

Wave loads are of great importance for TLP both by their intensity and frequency, since natural periods of the system "tension legs — hull" are often found within the range of power bearing waves.

Taking into account irregular nature of wave conditions, the methodology for determining wave loads shall be based upon statistical approaches.

3.5.2.3.1 For assessing statistical characteristics of wave loads one of the two approaches is recommended. According to the first approach, the life cycle is presented as the set of wave stationary modes. Leg reactions characterized by the wave height of a given probability, average wave period, course angle and recurrence are being found for each mode, whereupon all the reactions are summarized.

Another approach is based on the concept of the severest conditions, at which the extreme value of reaction is the most probable for realization (refer to 2.2.3.5, 3.1.4.10, 3.1.4.11, 3.1.4.12).

3.5.2.3.2 As the basis dependence for determining wave loads one may use the Morison equation considering diffraction corrections (refer to 2.3.9.1 and 2.3.9.2).

In obtaining probabilistic characteristics of wave loads on the basis on the Morison equation one shall use the Weibull distribution, the parameters of which (scale and form) shall be determined from the diagrams and dependences in Fig. 3.1.4.9. On X-axis on the diagrams one shall single out the relation of standard deviations of the wave load inertia and speed components.

3.5.2.3.3 The method based on the Morison equation suggests that the structure does not cause distortion of speed and acceleration field of the liquid particles motion in a wave, thus preventing from fully considering the diffraction effects and hydrodynamic interaction of the structure elements. Though, this method makes it possible to fully use different wave theories, consider viscosity effects, extremities of the wave amplitude and the structure vibrations, shallow water effects.

It is reasonable to use the method in case the dimensions of a structure or its elements are so small that viscosity forces prevail in wave loads.

For large structural diameters $(D/\lambda > 0{,}2)$ and relatively small wave heights it is advisable to use diffraction theory of calculation. The Morison equation is preferably used for smaller diameters $(D/\lambda < 0{,}2)$.

Method of linear diffraction theory is based on the assumption of smallness of a wave height and the platform vibration amplitude, potential character of liquid motion preventing from fully considering the viscous effects. Though, this method helps to consider the diffraction effects originated from flow past large bodies and connected with distortion of speed field in the wave. The method is applicable in calculation of loads for the structures with dimensions equally large longitudinally and horizontally, multi-column platforms, when the diffraction effects are sufficient and viscosity forces are negligible.

3.5.2.3.4 Requirements of 3.5.2.3.1 — 3.5.2.3.3 also consider application of other approaches, approved by the Register upon appropriate review. In particular, the method of calculation of wave loads on TLP with the aid of ANCHORED STRUCTURES software package approved by the Register may be used.

3.5.2.3.5 Disturbance forces of wave loads affecting TLP are permitted to be accepted equal to those of the semi-submersible MODU of a relevant structural and architectural type, while for local loads on TLP the wave loads shall be considered according to 3.2.3.4.

3.5.2.4 High frequency wave loads.

During a TLP operation the low frequency wave load may be subjected to superposition of high frequency loads generally of a pulse or impact nature (i.e. during realization of "springing" and "ringing" phenomena), respectively, vertical high frequency vibration of a TLP caused by pulse loads, and vertical high frequency vibration of a TLP caused by cyclic load from the vertical vibrations, rolling or pitching of a TLP with resonant or near-resonant periods. Because of perceptible presence of a high frequency component, in a number of cases the issue is considered in the context of its impact on fatigue life.

3.5.2.5 Vortical loads.

Vortical forces due to the current effecting the hull structures and tension legs are determined in accordance with the following.

3.5.2.5.1 Vortical vibration of a TLP bluff elements, caused by the current, may lead to undesired consequences at a certain current velocity. At this, frequency of vortex separation, determined by the formula, is of great importance

$$f = Sh\frac{v_t \sin\varphi}{D}, \text{ in Hz,} \qquad (3.5.2.5.1)$$

where Sh = Strouhal number;
v_t = current velocity, in m/s;
D = typical cross sectional dimensions of a structure (diameter), in m;
φ = angle between axis of the structure and direction of the current.

The frequency given in Formula (3.5.2.5.1) corresponds to the alteration of vortical forces across the current; the frequency of forces alteration along the current is half as low as that determined by Formula (3.5.2.5.1).

3.5.2.5.2 Generally, the value of Sh corresponding to the frequency of vortex separation is determined in relation to Reynolds number R_e. On the basis of numerous field researches for determining the disturbing forces effecting the blunt structures, the following dependences may be used:

$Sh = 0{,}20$ for $R_e \le 2{,}5 \times 10^5$;

$Sh = 0{,}27$ for $R_e > 2{,}5 \times 10^5$.

3.5.2.5.3 Coincidence of frequencies of unsteady forces with natural frequencies of the structure causes resonant phenomena with the possible considerable oscillation amplitude. Generally, vortical vibration is of hydroelastic nature and shall be studied by appropriate methods. The main peculiarity of natural vibrations is represented by velocity-expanded zones of resonant vibrations resulted from synchronization of natural vortex separation.

3.5.2.5.4 In the extended structures, such as tension legs, the resonant vibrations may emerge at all the operational velocities of the current. As a rule, transverse vibrations effecting extended tension legs are more intense than those directed along the current.

3.5.2.6 Dynamic aspects of a TLP behavior.

3.5.2.6.1 Dynamic characteristics of TLP are very important when assessing wind, wave, ice, seismic loads as well as load from current.

3.5.2.6.2 Frequency of the platform natural vibrations at *i*-th degree of freedom is determined by the formula

$$p_i = \sqrt{K_i / M_i} \qquad (3.5.2.6.2)$$

where K_i and M_i = respectively, stiffness of a TLP system, including tension legs, and mass of a TLP with added mass (or moment of inertia of masses with respect to the drilling location).

3.5.2.6.3 Stiffness of the vertical leg system during horizontal shift for small rotation angles, in kN/m, is determined by the formula

$$K_x = \frac{n_{t.l.} T_{t.l.}}{L_{Rt.l.}} + \frac{n_r T_r}{L_{Rr}} + n_{t.l.} \left(W_{t.l.} - \gamma \frac{\pi D_{t.l.}^2}{4} \right) \frac{L_{At.l.}}{2 L_{Rt.l.}} + n_r \left(W_r - \gamma \frac{\pi D_r^2}{4} \right) \frac{L_r}{2 L_{Rr}} \qquad (3.5.2.6.3)$$

where $n_{t.l.}$ = number of tension legs;
n_r = number of raisers;
$T_{t.l.}$ = pretension of a tension leg, in kN;
T_r = tension of a raiser;
$W_{t.l.}$ = weight of a tension leg per length unit in the air, in kN/m;
W_r = weight of a raiser per length unit, including the liquid contained therein, in the air, in kN/m;
$D_{t.l.}$, D_r = diameter of a tension leg and raiser, respectively;
$L_{At.l.}$, L_{Ar} = effective axial length of a tension leg and raiser, respectively, in m;
$L_{Rt.l.}$, L_{Rr} = rotation radius of a tension leg;
γ = specific weight of water.

3.5.2.6.4 Non-linearity of the system in the horizontal direction shall be considered in the following ratio:
for the vertical leg system

$$u / L_{Rt.l.} \geq 0{,}02; \qquad (3.5.2.6.4\text{-}1)$$

for the inclined leg system

$$u / L_{Rt.l.\beta} \cos\beta \geq 0{,}1. \qquad (3.5.2.6.4\text{-}2)$$

3.5.2.6.5 Stiffness of the vertical leg system during vertical vibrations, in kN/m, is determined by the formula

$$K_z = n_{t.l.} K_l + \rho g S \qquad (3.5.2.6.5)$$

where S = total area of floatation waterline, in m²;
ρ = mass water density, in kN·s²/m⁴;
G = acceleration of gravity, in m/s².

3.5.2.6.6 Stiffness of the inclined leg system during vertical vibrations, in kN/m, is determined by the formula

$$K_z = n_{t.l.} K_l \sin\beta + \rho g S. \qquad (3.5.2.6.6)$$

3.5.2.6.7 Stiffness of the system during rotational vibrations, in kN/m, is determined by the formula

$$K_{xz} = 4K_l a^2 - \lambda_B \qquad (3.5.2.6.7)$$

where $\lambda_e = G\overline{KG} - F_B\overline{KB}$;
G and F_B = mass of the structures and buoyancy integral;
$\overline{KG}$ and $\overline{KB}$ = respectively, distance between the centre of gravity and centre of buoyancy from the level of hawses, in m;
a = half of the distance between the hawses connecting tension legs, in m.

3.5.2.6.8 Added masses and their moments of inertia for the *i*-th degree of freedom depend on the wave frequency and are determined on the basis of the certain theoretical solutions for the simple-shaped bodies and model tests for the irregular shaped bodies.

3.5.2.6.9 Dynamic aspects of iceberg loads on TLP may be significant when calculating the common and local strength of the structures as well as safety factors of the anchor lines. Dynamic effects are related to vibrations of TLP and iceberg due to iceberg contact with the TLP hull or anchor lines. Further contact interaction depends largely on the complex law of the iceberg displacement in relation to TLP.

3.5.2.6.10 Dynamic aspects of the TLP-iceberg interaction shall be analyzed with due regard to the loss of stability by the iceberg which can result in its capsizing in close proximity of TLP due to the contact interaction with the hull or anchor lines.

3.5.2.6.11 Since dynamic effects significantly depend on the iceberg parameters, geometric properties of icebergs and TLP, performance of position-keeping system and ice speed, dynamic aspects of iceberg loads on TLP shall be accessed, where reasonable, on the basis of mathematical modeling of the iceberg and moored structure interaction using the software approved by the Register.

3.5.2.7 Global iceberg loads on TLP.

3.5.2.7.1 Global iceberg loads on TLP shall be determined based on the contact area of iceberg and structure. The time-dependent contact area shall be calculated with due regard to the interacting objects, their mutual orientation, direction of the iceberg drift and platform flexibility according to the procedures or by means of the software approved by the Register.

3.5.2.7.2 Global iceberg loads on TLP are determined by the following formula:

$$F_{horiz} = 7{,}4A \text{ MPa at } A \le 1\,m^2;$$

$$F_{horiz} = 7{,}4A^{0,3} \text{ MPa at } 1\,m^2 < A \le 1\,m^2; \qquad (3.5.2.7.2)$$

$$F_{horiz} = 1{,}48A \text{ MPa at } A \le 10\,m^2$$

where A = projection of the current contact area on a plane normal to direction of the structure intrusion into iceberg.

3.5.3 Strength criteria.

3.5.3.1 General.

3.5.3.1.1 Generally, the main requirements set out in 2.4.1 are applicable for the TLP. Additionally to the dangerous states listed in 2.4.1.2 the following shall be included: slackening of a tension leg. Respectively, the criterion of leg tension maintenance shall be observed.

3.5.3.1.2 The tensile leg angle in the upper and lower coupling shall be chosen such that the leg remains undamaged in the area of its interaction with the hull and anchor structural elements considering characteristics of the flexible elements.

3.5.3.1.3 Buckling strength criterion for the TLP hull and foundation structural elements shall be in compliance with 2.4.3.

3.5.3.2 Ultimate strength criterion.

3.5.3.2.1 The ultimate strength criterion for extreme loading of the hull and anchor is determined by expression (2.4.2.2) considering expressions (2.4.2.3.1) and (2.4.2.3.2). At that, safety factor η_1 shall be determined according to 3.5.3.2.4.

3.5.3.2.2 The ultimate strength criterion during extreme loading of tension legs is determined by the expression

$$T_{\Sigma} < \eta_1 A \sigma_t, T_{\Sigma} \leq \eta_1 T_b \qquad (3.5.3.2.2)$$

where T_{Σ} = total design tension of a leg, caused by all the possible static and alternating loads (tension components are characterized in 3.5.4), in kN;
A = design cross-section area of the leg, in m²;
σ_t = temporary resistance of the leg material, in MPa;
T_b = breaking stress of the leg, in kN.

3.5.3.2.3 The TLP ultimate strength criterion for the operating and transit conditions, the conditions of positioning at and removal from site and safety factors, respectively, shall be in compliance with 2.4.2.4 and 2.4.2.5 of this Part and 3.1.1.1 of Part II "Hull" of the FOP Rules.

3.5.3.2.4 Safety factors η_1 for the criteria set out in 3.5.3.2.1 and 3.5.3.2.2 shall not exceed the values given in Table 3.5.3.2.4.

Table 3.5.3.2.4

Name of a structure	Strength criterion	Structural elements		
		Special	Primary	Secondary
Hull and foundation beyond the area of interaction with tension legs	p. 3.5.3.2.1, criterion 2.4.2.3.1	0,75	0,80	0,83
	p. 3.5.3.2.1, criterion 2.4.2.3.2	1,25	1,35	1,45
Hull and foundation in the area of interaction with tension legs	p. 3.5.3.2.1, criterion 2.4.2.3.1	0,65	0,70	-
	p. 3.5.3.2.1, criterion 2.4.2.3.2	1,20	1,30	-
Tension legs	3.5.3.2.2	0,55	0,60	–

3.5.3.2.5 The safety factor for a tension leg replacement practice is applied in compliance with Table 3.5.3.2.4. When making calculations, the changing of TLP trim due to ballasting shall be considered.

3.5.3.2.6 Additional ultimate strength criteria for the ice belt structures of the ice-resistant TLP shall be in compliance with the requirements of 3.1.3 of Part II "Hull" of the FOP Rules.

3.5.3.3 Fatigue strength criterion.

3.5.3.3.1 Fatigue strength criterion shall be applied to the structural elements, for which the strength may represent the ultimate form of destruction, for example, tension legs — foundation — hull structure joints, as well as tension leg elements. The list of joints shall be agreed by the designer with the Register.

3.5.3.3.2 Designing of the platform structures shall be performed on the basis of the "safe damage" criterion, according to which the fatigue criterion realization is oriented at the stage of fatigue macrocrack initiation rather than the crack development. Characteristics of ultimate relative vulnerability are determined in 2.4.4.

3.5.3.3.3 The sources of cyclical loads are waves, wind, current, ice and seismicity reason. The initial data for each type of cyclical loads is recurrence of environmental conditions.

3.5.3.3.4 In the absence of sufficient statistics on the structure loading the fatigue life at wave, seismic or alternating ice loads is recommended to be determined on the basis of the analytical dependencies.

3.5.3.3.5 Influence of high frequency components of a wave load from "springing" and "ringing" (refer to 3.5.2.4) on the tension leg fatigue life is determined by a reduction coefficient γ dependent on relation of standard deviations of high frequency and low frequency components, as well as on m parameter.

3.5.3.4 Criterion of tension maintenance in the leg.

3.5.3.4.1 The tension maintenance criterion stipulates the requirements aimed at preventing the origination of a tension leg slackening, as the result of which the tension leg is considered to be out of service.

3.5.3.4.2 This criterion may be defined in the following way:

$$T_{\sum\vartheta} \leq \eta_1 * T_0 \qquad (3.5.3.4.2)$$

where $T_{\sum\vartheta}$ = design leg tension dependent on design loads leading to tension minimizing by elimination of the original (initial) tension on still water;

T_0 = original (initial) tension on still water;

$\eta_1 *$ = safety factor; $\eta_1 *= 0{,}70$.

3.5.4 Peculiarities of strength calculation and TLP design.

3.5.4.1 General.

3.5.4.1.1 Generally, the main requirements set out in 2.5.1 and 2.5.2 are applicable for the TLP design. In addition it shall be noted that the TLP vital reactions are represented by linear and angular motion of the hull, as well as internal axial forces in the TLP.

3.5.4.1.2 Damage of one leg shall not lead to progressive breaking of other legs or excessive damaging of the hull or foundation in the areas of interaction with tension legs. The Register may require the calculation confirming that being exposed to the environmental loads maximum for the given area of operation for a one-year period the structural strength of the platform with a damaged tension leg will be maintained.

3.5.4.1.3 In designing of TLP one shall consider that the hull shift with regard to the ground foundation would not cause damage of the structure and, finally, accident situations.

3.5.4.2 Hull.

3.5.4.2.1 A particular method of the TLP hull design shall be determined according to the peculiarities of the structure. When calculating hull strength for the multi-column TLP the regulations of 3.2 of this Part related to the semi-submersible MODU may be applied. When calculating hull strength for the tower-shaped TLP the regulations of 3.1 of Part II "Hull" of the FOP Rules related to monopods (monocones) may be applied.

3.5.4.2.2 Calculation of girder system, separate girders, plates, calculation of structural elements' buckling strength shall be carried out in accordance with 2.5.3 — 2.5.5.

3.5.4.2.3 Ice belt structures of ice-resistant TLP shall be calculated in accordance with 3.1.4 of Part II "Hull" of the FOP Rules. At that one shall consider that ice formation shall not touch with the areas of the hull and tension legs joint.

3.5.4.2.4 Calculation of steel concrete ice belt shall be carried out in accordance with 3.2 of Part II "Hull" of the FOP Rules.

3.5.4.3 Tension legs.

3.5.4.3.1 A tension leg consists of the three basic elements:

area of interaction with the hull;

area of interaction with the foundation;

basic part of the leg — joints between all the above elements.

The area of interaction with the hull is designed for the following functions: control and regulation of the required tension, joining tension legs with the hull, perception of transverse forces and bending moments. The area of interaction with the foundation is designed for the following functions: maintenance of structural joints between the foundation and a leg, perception of transverse forces and bending moments.

The operational peculiarities of each area determine the deflected mode character and appropriate approaches to the structural strength calculation.

3.5.4.3.2 Tension in any tension leg is the sum of a range of components possessing different physical values, i.e.:

$$T_{\Sigma} = \sum_{i=1}^{n} T_i \qquad (3.5.4.3.2)$$

where n = number of considered components.

These components are subdivided into two radically different groups: deterministic (including static) and occasional.

3.5.4.3.2.1 The full formulation of tension components is as follows:

deterministic (or quasideterministic) components:

T_0 = original tension at the mean water depth;

T_t = tension from storm surge;

T_λ = leg tension dependent on alteration of ballast, cargo etc. weight;

T_m = tension caused by capsizing moment from wind load and current;

T_s (wave or ice) = tension caused by sagging due to static loads and slowly changing shift (wave drift or constant component of ice load, wind, current);

T_f (wave or ice) = tension caused by foundation shift under the influence of water or ice;

occasional components:

T_w (or T_{ice}) = alternating tension component from wave or ice forces with regard to average shift (includes tension from horizontal forces, vertical forces, vibrations (rolling and pitching), generally form rotation forces);

T_i = tension caused by vertical oscillation, rolling or pitching at the natural oscillation frequency of the platform (ringing and springing, including possible underdeck slamming loads).

3.5.4.3.2.2 Standard deviation of total tension is determined by the formula

$$\sigma_{T_{\Sigma}}^2 = \sum_i \sigma_{T_i}^2 + 2\sum_i \sum_j \rho_{ij}\sigma_{T_i}\sigma_{T_j} \qquad (3.5.4.3.2.2)$$

where σ_{T_i}, σ_{T_j} = standard deviations of separate components determined basing upon the idea of statistical dynamics;

ρ_{ij} = coefficient of correlation between separate tension components.

3.5.4.3.2.3 As the wave tension distribution law the Weibull distribution with the parameters of scale a and K form (refer to 3.1.4.8 and 3.1.4.9) is recommended.

3.5.4.3.2.4 As the ice tension distribution law the Weibull distribution with the parameters of scale $\bar{b}$ and K form, determined depending on relation a_h/D, where a_h is the parameter of ice thickness distribution scale, D is the obstacle diameter at the waterline level (refer to 3.1.2.3.3 and 3.1.2.3.5 of Part II "Hull" of the FOP Rules) is recommended.

3.5.4.3.3 Basic stages of tension leg design procedure including consideration of ultimate and fatigue strength as well as the hull and foundation impact on legs, are as follows:

platform dimensions — determining the TLP general configuration;

leg predesign — assessment of pretension and other input data necessary to determine the TLP dimensions;

analysis of reactions — determining the structure's shift and minimum /maximum leg tension;

horizontal leg reactions — determining bending moments in legs and horizontal vibrations;

minimum tension — determining the minimum leg tension;

pretension analysis — check of the preliminary maximum stresses and fatigue life;

check of service limitations — check of admissible shifts of the structure as well as check of vibrations and leg shifts;

fatigue life — determining fatigue strength under effect of axial and bending forces combination;

final check — check of maximum stresses, minimum tension, fatigue life, etc.;

mutual analysis — determining necessity of carrying out the mutual reactions analysis;

model tests (not obligatory) — confirmation of vibrations and loads on the leg.

3.5.4.4 Ground foundation.

3.5.4.4.1 Primarily ground foundation is aimed at tension leg mooring, TLP loads perception and transmission fully or partially to the seabed foundation soil.

The main requirement to the foundation systems shall together with tension legs to reliably buoy the floating structure at the certain area of the open sea, restrict its shift within the specified area and, thus, provide normal operation conditions. Safety of the whole unit depends on operational reliability of the positioning system; breaking from the positioning point is inadmissible.

3.5.4.4.2 For the central tension legs buoyancy the foundation structures with ram piles as well as those of gravitational or mixed type may be used. The units may be made whether as a separate, supported by piles or masses or their combination, one-piece structure, to which all the tension legs and raisers are fixed, or as a system of separate, independent foundation structures for groups or strands of tension legs and borehole pipes.

Besides, the anchors consisting of one or several suction piles as well as the anchors of Stevmanta or SEPLA type may be considered as peripheral foundation systems.

3.5.4.4.3 The load may be transmitted to the ground in several ways, i.e. through the tension legs joined to the piles directly, through the surface ground-based foundation plates (templates) that transmit tension leg forces through the piles to the ground through surface gravity foundation.

3.5.4.4.4 Calculation of ground foundation shall include calculation of the foundation structure deformation and strength and calculation of the foundation buckling strength and shifts with regard to the ground.

In designing the TLP foundation structure the following issues regarding peculiarities of the structure's operating conditions shall be considered:

load eccentricities being the result of alteration of the tension leg forces within the group;

consequences of a tension leg/raiser installation — possible raise (pulling out) and re-location of the tension legs/raisers during service life of the platform;

position (installation) and operation (regulatory) design tolerances;

issues on survey and check of compliance of the foundations with the required operational capabilities.

3.5.4.4.5 Strength calculation for steel, reinforced concrete and steel concrete structures of ground foundations shall be carried out in accordance with acceptable requirements set out in 2.5 of this Part and 3.2 of Part II "Hull" of the FOP Rules.

3.5.4.4.6 Buckling strength calculation for seabed foundations shall meet the requirements stated in Section 4.

3.5.4.5 Joints.

3.5.4.5.1 The hull — tension legs joint.

3.5.4.5.1.1 The structures joining the hull and tension legs take up leg reaction by means of the two supporting areas:

upper area, taking up mainly tension force of legs;

lower area, taking up transverse reactions originated from the platform horizontal shift.

3.5.4.5.1.2 Supporting structures of the upper supporting area shall be designed for impact of a tensioner or maximum possible vertical leg reaction. At that, sufficient resistance of the structure considering statics and dynamics of the platform shift shall be checked:

to loss of buckling strength and necessary stiffness;

to collapse, shift or bending stresses.

Possibility of unequal distribution of the tension leg reaction distribution shall be considered.

3.5.4.5.1.3 Supporting structures of the lower supporting area shall be designed for the impact of a flexible element and maximum possible horizontal reaction considering statics and dynamics of the platform shift.

Sufficient resistance of the above structures shall be checked:

resistance to loss of buckling strength;

resistance to collapse, shift or bending stresses, local peak contact stresses.

3.5.4.5.1.4 Sufficient stiffness of work contact surfaces shall be specified in order to maintain their operating capability during whole service life of the platform.

3.5.4.5.2 Anchor — tension legs joint.

3.5.4.5.2.1 Structures of anchor — tension legs joints are represented by the two supporting areas:

upper area with a flexible element taking up horizontal reactions originated from the platform shift;

lower area taking up vertical reactions originated from the platform shift.

3.5.4.5.2.2 Supporting structures of the anchor upper supporting area shall be designed for the impact of flexible element or maximum possible horizontal reaction considering static and dynamic impact on the platform and the leg. The following structural resistance shall be checked:

resistance to loss of buckling strength;

resistance to collapse, shift or bending stresses;

resistance to local peak contact stresses;

resistance to ambient pressure for dry sea chests and closed spaces.

3.5.4.5.2.3 Supporting structures of the lower supporting area shall be designed primarily for the impact of maximum possible vertical leg tension considering statics and dynamics. The anchor structural resistance shall be checked by calculation of the following:

loss of buckling strength and necessary stiffness;

collapse, shift or bending stresses;

local peak contact stresses.

3.5.4.5.2.4 Structures of anchor — tension leg joints shall be provided with thickness margin considering high erosion and abrasion wear and probable chemical corrosion of steel.

3.5.4.5.2.5 When ropes are used as tension legs, the structures of the upper and lower areas may join together.

3.5.4.5.3 Joints of tension leg elements.

3.5.4.5.3.1 The main constructive way in designing joints equal to the tension leg in strength is the reduction of effective stresses through the extension of sectional area of the joint.

3.5.4.5.3.2 In design of the tension leg joints the strength calculation shall include:

total leg tension;

total leg bending in the area in question;

local bending induced by sectional eccentricity;

local concentration of stresses caused by peculiarities of the joint and/or weld.

3.5.4.5.3.3 In case the internal volume of the leg is isolated, the strength calculation shall additionally consider the impact of internal and environmental stresses on the joint stressed state.

3.5.4.5.3.4 For the structures of joints and tension leg elements check calculation of local impact strength against the reactions transmitted from the upper and lower supporting areas of the anchor and hull structures considering deformation of their flexible elements shall be carried out.

3.5.4.5.3.5 If strength check for the tension leg joints shows that their strength and service life shall not provide the platform real service life, the above calculations shall be re-performed considering replacement of tension legs during the platform operation.

3.5.4.5.4 Hydrodamping devices structures.

3.5.4.5.4.1 General.

3.5.4.5.4.1.1 Application.

The present provisions refer to hydrodamping devices for floating facilities subject to substantial motion in operating conditions which requires its damping, e.g. for a $Spar$ platform.

3.5.4.5.4.1.2 Definitions and explanations.

3.5.4.5.4.1.2.1 The following definitions have been adopted in the present recommendations:

Flap height is a full height of cylindrical surface normal to the disk (or double distance from the disk plane to the extreme flap point).

Damping devices are horizontal disks attached to the platform hull along its height.

Damper flaps are cylindrical surfaces, erected on the edges of a damping disk.

Perforation coefficient is the ratio between nonperforated area and the total disk surface area; the perforation coefficient equal to 1 corresponds to a nonperforated disk.

Relative factor of disk resistance is a resistance coefficient of a disk or a system of disks in question related to the resistance coefficient of a solid nonperforated disk.

Perforation are ring- or round-shape holes in a damping disk.

$Spar$ platform is a platform on tension and anchor legs intended mostly for deepwater operations, and which hull is elongated in vertical direction.

3.5.4.5.4.1.3 Structural elements.

The platform structural elements shall be classified into special, primary and secondary elements proceeding from stress levels and the effect their eventual damage may have upon the strength and serviceability of the floating facilities.

Due to their serviceability, the hydrodamping devices (disks) shall be related to the primary elements which ensure the overall strength of the hull (in case of hydrodamping disks failure the motions will increase with all ensuing consequences of the overall strength reduction).

3.5.4.5.4.2 Hydrodamping devices design principles.

3.5.4.5.4.2.1 General.

The design of hydrodamping devices shall be so that:

their operating parameters will satisfy the requirements for motions damping;

their strength within the service life (as determined for environmental conditions of the anticipated area of operation) meets the accepted criteria in the following design modes of operation: transit, operating, survival and extreme loading.

3.5.4.5.4.2.2 Loads.

In calculations of hydrodamper the loads due to wind, waves and current effects shall be considered.

The loads to be estimated for the structural strength analysis of hydrodamping devices shall include:

common and local hydrodynamic loads in the extreme wave conditions considering wind effects;

hydrodynamic cyclical loads;

current loads;

high frequency loads of pulse or impact nature (vertical high frequency vibration of a structure caused by pulse loads, and vertical high frequency vibration of a structure caused by cyclic loads from the vertical vibrations, pitching or rolling with resonant or near-resonant periods). Because of perceptible presence of a high frequency component, in a number of cases the issue is considered in the context of its impact on fatigue life;

vortical loads, forces due to the current effecting the damping devices are determined in accordance with the condition

$$f = Sh\frac{v_t}{D}, \text{ in Hz,} \qquad (3.5.4.5.4.2.2)$$

where Sh = Strouhal number,
v_t = current velocity, in m/s,
D = typical cross sectional dimensions of a damping device structure (diameter),
f = frequency of vortex separation.

The frequency determined by Formula (3.5.4.5.4.2.2) corresponds to the alteration of vortical forces across the current; the frequency of forces alteration along the current is half as low as that determined by Formula (3.5.4.5.4.2.2).

Generally, the value of Sh corresponding to the frequency of vortex separation is determined in relation to Reynolds number R_e.

Coincidence of frequencies of unsteady forces with natural frequencies of the structure (or its separate parts) causes resonant phenomena with the possible considerable oscillation amplitude. Generally, the vortical vibration is of hydroelastic nature and shall be studied by appropriate methods.

3.5.4.5.4.2.3 Strength criteria.

The damping disks structure shall be so designed as to meet the general safety requirement

$$\Phi \le R\eta \qquad (3.5.4.5.4.2.3\text{-}1)$$

where Φ = design value of the generalized force action which is used to assess marginal state;
R = design value of generalized bearing capacity,
η = safety factor which depends on the damping disk responsibility for strength and safety of the structure.

If requirement (3.5.4.5.4.2.3-1) is met the following dangerous states may be practically avoided:

excessive deformations of material;

buckling;

fatigue cracks;

Accordingly, the criteria of ultimate and fatigue strength, as well as buckling strength criterion shall be met.

For damping disks structural elements, in terms of their strength and stability, the MODU Rules provisions apply.

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

$$\sigma_d \le \eta_1 R_d \qquad (3.5.4.5.4.2.3\text{-}2)$$

where σ_d = design structural stress caused by the most unfavourable combination of loads, in MPa;
η_1 = safety factor: for damping devices structures the safety factor is taken as $\eta_1 = 0{,}8$;
R_d = design yield stress of material.

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

Buckling strength criterion is determined by the expression

$$\sigma_x \leq \eta_2 \sigma_{cr} \qquad (3.5.4.5.4.2.3\text{-}3)$$

where σ_x = design stress for the specified condition of the structural element, in MPa;
σ_{cr} = critical stress, in MPa;
η_2 = safety factor; the safety factor shall not exceed the value

$$\eta_2 = 0{,}84(1 - 0{,}2R_d/\sigma_e) + 0{,}06$$

where σ_e = Euler's stress corresponding to the minimum value of all considered Euler's stresses and forms of stress state, in MPa.

Influence of high frequency components of a wave load from phenomena described in 3.5.4.5.4.2.2 on the tension leg fatigue life is determined by a reduction coefficient dependent on relation of standard deviations of high frequency and low frequency components.

3.5.4.5.4.3 Structural design issues specific to hydrodamping devices.

3.5.4.5.4.3.1 The hydrodynamic forces acting on the system of hydrodamping devices may be determined in the first approximation without considering horizontal oscillations, while vertical oscillations in a viscous fluid shall be considered. At a sufficient distance from a streamlined body the water may be considered as motionless relative to the seabed. For the damper itself the adhesion condition shall be adopted (i.e. water velocity coincides with the damper velocity). The damper can move progressively (stationary flow) and oscillate (non-stationary flow). At the inflow, outflow and at the lateral boundary, the velocity equal to inflow velocity shall be set up, which means negligible damper effect on the boundary of contact.

3.5.4.5.4.3.2 In case of slott perforation, the task is symmetrical relative to disk central axis, which stipulates current study in 2D domain.

For the case of modelling flow around a disk damper with round or square perforated holes, three-dimensional task shall be considered.

3.5.4.5.4.3.3 The hydrodynamic load acting on hydrodamping devices includes two components: the velocity (viscous) and inertia component.

3.5.4.5.4.3.4 The velocity (viscous) component of the hydrodynamic load is proportional to the square of velocity and velocity resistance C_v:

$$C_v = \frac{F_v}{(\rho V^2 S)/2} \qquad (3.5.4.5.4.3.4\text{-}1)$$

where F_v = disk resistance force component;
V = inflow velocity;
S = disk area;
ρ = mass density of water.

The inertia component of hydrodynamic load is proportional to the added mass.

The added mass is determined by the formula

$$m_a = F_a/a_y \qquad (3.5.4.5.4.3.4\text{-}2)$$

where a_y = disk acceleration.

3.5.4.5.4.3.5 For optimization of the hydrodynamic loads acting on hydrodamping devices it is recommended to study the following factors: disks perforation degree, perforation shape, "flaps" effect, dampers system and seabed effect.

3.5.4.5.4.3.5.1 Perforation degree. The perforation degree effect may be ambiguous: with reduction of perforation coefficient, the resistance coefficient can first considerably increase, and then it can decrease smoothly, while the added mass tends to rapid reduction. Generally, the best effect can be achieved with the perforation coefficient equal to 0,9.

In case of perforation with small diameter holes, the dynamic resistance coefficient and added mass grow insignificantly.

3.5.4.5.4.3.5.2 Perforation shape. The damper square and round perforation provides the qualitatively and quantitatively similar effects as in case of concentric slot perforation. At that, the dynamic coefficient increases 1,5 — 1,8 times, and the added mass is reduced by 20 — 30 % as compared to a non-perforated damper.

The hole shape (round or square) of the damper perforation does not practically affect its dynamic characteristics.

3.5.4.5.4.3.5.3 Flap effect. The presence of flaps results in reduction of resistance coefficient and in greater value of the added mass. When the flaps height grows, the resistance coefficient grows together with increase of the added mass. The application of flaps results in increase of damper strength characteristics. When cone-shaped flaps are used, the dynamic resistance coefficient grows with increase of α angle from 0° (direct flaps) to 90° (absence of flaps) almost in a linear manner, while the added mass first grows slightly, then decreases.

The wave length (period) effect is, as a rule, insignificant.

3.5.4.5.4.3.5.4 Stiffeners. The application of concentric stiffeners to improve the damper strength characteristics does not practically affect its hydrodynamic properties.

3.5.4.5.4.3.5.5 Damper system. With increase of the distance between the adjacent disks in a three-disk system, the relative resistance coefficient and dimensionless added mass increase.

3.5.4.5.4.3.5.6 Perforated dampers system. The resistance coefficients and added mass of a three perforated damper system substantially improve the resistance of a solid non-perforated damper.

3.5.4.5.4.3.5.7 Bottom effect. Where the distance between the disks of a three disk system is sufficiently great, the proximity to the seabed increases the relative resistance coefficient, while the dimensionless added mass remains practically unchanged.

3.5.4.5.4.3.6 The application of hydrodamping devices can substantially change the platform added mass consequently increasing the period of the platform natural vertical vibrations.

3.5.4.5.4.3.7 The application of hydrodamping devices can reduce the vertical wave load acting on the platform.

3.5.4.5.4.3.8 The application of the hydrodamping device system can reduce the platform vertical vibrations amplitude several times.

3.5.4.5.4.3.9 In calculating hydrodamping devices it is recommended to use the porous medium model which allows to obtain the results qualitatively similar to those obtained from the real holes simulation. At that, these calculation data may be considered as asymptotic estimates of the resistance and added mass coefficients for the case of "infinite number of holes having infinitesimal diameter" with fixed perforation coefficient.

3.5.4.6 Peculiarities of structural design in the seismically dangerous regions.

3.5.4.6.1 When designing TLP in the seismically dangerous regions anchor strength and bearing capacity, tension legs and hull strength considering large-scale seabed deformations, possible ground dilution, as well as "seaquake", i.e. underwater acoustic impact on TLP structures shall be ensured.

3.5.4.6.2 It is necessary to avoid anchor arrangement at the seabed areas where earthquakes may cause large scale ground deformation.

If, nevertheless, anchors were arranged in the areas of considerable seismic shifts, the anchor bearing capacity shall be checked taking into account the specified ground shifts (i.e. slide of subsea slopes).

3.5.4.6.3 When assessing anchor stiffness considering seismic loads both maintenance of structural strength and bearing capacity of the seabed subjected to dynamic loads shall be provided.

One shall take into account possible temporary decrease of anchor bearing capacity caused by dynamic dilution of the ground. At that, one shall define the extent of possible degradation of bearing capacity, as well as the period of bearing capacity recovery (based upon the time required for ground consolidation).

The time specified shall be considered when choosing design foundation ground characteristics with regard to various combinations of loads and stresses.

3.5.4.6.4 In calculation of the anchor stressed state and buckling strength one shall take into account the anchor mass as well as the additional masses of water and anchor legs.

3.5.4.6.5 Consequences of seismic load transmission from the waterarea bottom to the TLP hull through the tension legs shall be considered when seismic horizontal and vertical shifts of the waterarea bottom in the anchor legs location areas exceed relative permissible TLP shifts caused by wave loads at drilling. In such cases the tension leg forces shall be determined basing upon the values of anticipated seismic shifts of anchor legs with regard to the water area bottom.

The values of the above forces shall be used for checking tension leg strength and calculating TLP strength including the elements joining tension legs to the TLP hull and bearing.

3.5.4.6.6 As the design underwater acoustic load on TLP hull and tension legs one shall regard the hydrodynamic pressure applied to the TLP bottom that is time-varying according to the harmonic law with period T^{de} amplitude value p_{amp}^{de} determined by the formula

$$p_{amp}^{de} = kT^{de}\exp\left(0{,}72J^{de}\right) \qquad (3.5.4.6.6)$$

where k = 0,003, MPa·s^{-1}.

In the absence of sufficient seismic information it is permitted to accept T^{de} value equal to 0,50 s.

3.5.5 Requirements to materials.

3.5.5.1 General.

3.5.5.1.1 Materials used for manufacturing the TLP hull and anchor structures beyond the areas of tension leg joints shall be in conformity with 1.5.1 of this Part and Part XII "Materials".

3.5.5.1.2 This Section includes the specified requirements for the hull and anchor material in the areas of tension leg joints (mainly, special structural elements), as well as the requirements for the tension leg material that shall be regarded as addition to 1.5.1 of this Part and Part XII "Materials".

3.5.5.1.3 For all the structures listed in 3.5.5.1.2 the resistance of hydrogen brittleness materials shall be assessed.

3.5.5.2 Hull.

3.5.5.2.1 Set of mechanical properties of materials of the hull structures interacting with a tension leg, tensioner or flexible element shall be assessed additionally with regard to the following damage types caused by local contact stresses on the work surfaces of the structural elements:

plastic straining, collapse or pressing out;

erosional and abrasive wear of contact surfaces;
fatigue fracture of wear surface;
brittle fracture of wear surface and crumbling of material.

3.5.5.2.2 For the hull structures exposed to tension leg reactions it is required to use steel materials at least 70 mm thick with yield stress $\sigma_{0,2} \leq 550\sigma_t$, in MPa.

3.5.5.2.3 Temporary resistance of the hull structural materials σ, aimed to adequately provide strength and plasticity shall correspond to the relation $\sigma_{0,2} \leq 0{,}85\sigma_t$.

3.5.5.2.4 In order to adequately provide plasticity of special structures of the TLP hull the material shall have a residual relative contraction Z_z during elongation of the material in the direction perpendicular to the plate surface: $Z_z \geq 25\,\%$. Relative elongation A_5 during testing of samples shall be not less than $A_5 \geq 18\,\%$.

3.5.5.2.5 Contact work surfaces of the hull special structures shall be designed for collapse and be stiff enough to prevent abrasive wear.

3.5.5.2.6 Taking into account considerable dynamic components of loading normal strength steels shall not be used for manufacturing the hull special structures.

3.5.5.3 Anchor.

3.5.5.3.1 Mechanical properties of the anchor structures interacting with tension legs shall be assessed with regard to the damage types listed in 3.5.5.2.1, as well as regarding:
additional abrasion wear induced by water-risen seabed soils;
high chemical corrosion;
stress corrosion cracking resistance.

3.5.5.3.2 For the special anchor structures it is recommended to use steel materials less than 120 mm in thickness and having a yield stress less than $\sigma_{0,2} \leq 550$ MPa with the continuity check at thicknesses above 70 mm.

3.5.5.3.3 Temporary resistance of material shall be in compliance with 3.5.5.2.3.

3.5.5.3.4 The material of the anchor special structures shall have a residual relative contraction during elongation of the material in the direction perpendicular to the plate thickness: $Z_z \geq 20\,\%$. Relative elongation A_5 checked during cutting out the through-plate-thickness samples from the center of the plate shall be not less than $A_5 \geq 18\,\%$.

3.5.5.3.5 Contact work surfaces of the anchor special structures shall be in compliance with 3.5.5.2.5.

3.5.5.3.6 Crack-resistance characteristics of the anchor special structures shall meet the requirements of 3.5.5.2.6. The testing samples shall be cut from the subsurface layer of material.

3.5.5.4 Tension leg.

3.5.5.4.1 Mechanical properties of the tension leg material shall be tested with regard to the types of possible fracture resistance corresponding to the leg functionality.

3.5.5.4.2 Requirements to the material of the area of interaction with the anchor shall be in accordance with 3.5.5.1 and 3.5.5.3.

3.5.5.4.3 Requirements to the material of the area of interaction with the hull shall be in accordance with 3.5.5.1 and 3.5.5.2.

3.5.5.4.4 Requirements to the material of the tension leg's middle section shall be in accordance with 3.5.5.1 and 3.5.5.2.

3.5.5.4.5 Circumferential yield stress $\sigma^0_{0,2}$ and ultimate strength σ^0_t of the leg shall be in accordance with the requirements:

$$\sigma^0_{0,2} \geq 0{,}9\sigma_{0,2}; \qquad (3.5.5.4.5\text{-}1)$$

$$\sigma^0_{0,2} \geq 0{,}9\sigma_{\mathrm{t}}. \qquad (3.5.5.4.5\text{-}2)$$

3.5.5.4.6 For leg coupling impact energy shall be equal to:

for the specimens cut in the rolling direction KVL≥68 J;

for the specimens cut across the rolling direction KVT≥46 J;

CTOD value at the temperatures equal to the temperatures of impact toughness testing shall be equal to:

for basic metal CTOD≥0,25 mm;

for heat-affected zone CTOD≥0,18 mm (with welding);

The temperature value of zero fracture toughness shall be less than NDT≤ - 40 °C with the wall thickness being less than 40 mm.

4 SELF-ELEVATING MODU STABILITY ON THE SEABED

4.1 GENERAL

4.1.1 The interaction of self-elevating MODU 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 self-elevating MODU on the seabed is gravitational with pre-loading of legs into the seabed.

4.2 STABILITY OF SELF-ELEVATING MODU ON THE SEABED

4.2.1 Stability against capsizing on the seabed.

The safety factor against self-elevating MODU capsizing on the seabed shall be not less than

$$K_{cap} = M_r/M_{cap} \geq 1{,}50 \tag{4.2.1-1}$$

where M_r = righting moment due to self-elevating MODU weight forces, in kN m;
M_{cap} = total capsizing moment due to extreme effect of external forces about the plane of self-elevating MODU support on the seabed, in kN m;

The worst combination of a righting and capsizing moments depending on the loading condition of a self-elevating MODU, the values and directions of external effects shall be considered.

With reasonably developed in area supporting surfaces of footings, the presence of a support moment shall be considered, i.e. the following condition shall be considered as criterion

$$M_r/\left(M_{cap} - M_{sup}\right) \geq 1{,}50 \tag{4.2.1-2}$$

where M_{sup} = support bending moment from the direction of seabed, in kN m.

4.2.2 Stability in shifting.

The safety factor against self-elevating MODU shifting on the seabed shall be not less than

$$K_{sh} = Pf/T \geq 1{,}50 \tag{4.2.2}$$

where P = gravity load of a self-elevating MODU per leg with regard to the displaced water;
T = design value of a total shear force in way of a foundation;
f = friction coefficient of a supporting surface against the seabed.

The worst combination of a pontoon weight depending on the self-elevating MODU loading condition, and of a total shear force depending on the direction of extreme external effects shall be considered.

4.2.3 Stability in subsidence.

A safety factor for the subsidence of one of self-elevating MODU legs into the seabed shall be not less than

$$N_3/N > K_{sub} \tag{4.2.3}$$

where N_3 = pre-loading force;
N = design value of a total axial force;
K_{sub}= 1,0 — for four-leg units;
K_{sub}= 1,05 — for three-leg units.

The worst situation in terms of leg subsidence shall be considered as the very condition that is commonly critical. The subsidence condition establishes the necessary amount of ballast for three-leg units and impacts the volume and arrangement of spaces in a pontoon.

APPENDIX 1

CHARACTERISTICS OF WIND AND WAVE CONDITIONS

Table 1

Extremes of wind velocity and wave height with recurrence period of 50 years

Sea	Average wind velocity (10 min average period) $\bar{W}_{50}$, m/s	Wave height with 3 % probability of exceeding level h_{50}, m
Caspian Sea	45,0	13,0
Black Sea	43,0	12,5
Barents Sea	46,0	19,0
Okhotsk Sea	48,0	19,0

Table 2

Recurrence of wave heights and periods in the Caspian Sea, %

$\bar{\tau}$, s	$h_{3\%}$, m											
	0–1	1–2	2–3	3–4	4–5	5–6	6–7	7–8	8–9	9–10	10–11	11–12
0–1	7,11											
1–2	14,58											
2–3	6,44	20,21										
3–4	0,62	9,11	5,24									
4–5	0,33	6,32	10,02	5,36								
5–6	0,08	3,17	1,12	0,70	0,68							
6–7	0,07	1,54	0,66	0,49	0,44	0,05						
7–8	0,05	1,38	0,40	0,29	0,37	0,06	0,04	0,03				
8–9	0,03	0,97	0,27	0,23	0,21	0,07	0,06	0,03	0,005			
9–10	0,02	0,05	0,05	0,16	0,17	0,12	0,05	0,02	0,015	0,013	0,010	0,005
10–11	0,009	0,009	0,01	0,05	0,05	0,05	0,04	0,01	0,010	0,010	0,005	0,005
11–12	0,005	0,005	0,005	0,03	0,03	0,03	0,03	0,005	0,005	0,003	0,002	0,001
12–13	0,002	0,002	0,001	0,005	0,01	0,01	0,01	0,001	0,003	0,002	0,001	0,001

Table 3

Recurrence of wave heights and wind velocities in the Caspian Sea, %

$\bar{W}$, m/s	$h_{3\%}$, m											
	0–1	1–2	2 – 3	3 – 4	4–5	5 – 6	6–7	7 – 8	8 – 9	9–10	10–11	11–12
2 – 4	7,34	6,82	2,59	0,78	0,22	0,15	0,03	0,001				
4 – 6	7,80	7,76	4,42	1,02	0,43	0,26	0,08	0,003	0,001			
6 – 8	6,22	7,87	2,89	1,51	0,31	0,12	0,07	0,002	0,001			
8–10	4,32	7,95	2,80	0,71	0,17	0,06	0,01	0,007	0,002	0,002		
10–12	2,25	5,88	2,06	0,68	0,16	0,03	0,01	0,005	0,002	0,002		
12–14	1,15	3,35	1,58	0,57	0,15	0,01	0,01	0,009	0,009	0,005	0,004	
14–16	0,88	3,24	0,37	0,34	0,13	0,01	0,009	0,008	0,006	0,005	0,004	0,002
16–18		0,76	0,26	0,24	0,12	0,009	0,008	0,007	0,006	0,005	0,005	0,003
18 – 20		0,01	0,01	0,13	0,11	0,009	0,006	0,006	0,005	0,004	0,002	0,003
20 – 22		0,008	0,008	0,009	0,09	0,009	0,006	0,006	0,005	0,004	0,002	0,002
22 – 24		0,005	0,005	0,008	0,08	0,004	0,002	0,002	0,001	0,001	0,001	0,002
24 – 26		0,005	0,005	0,006	0,008	0,003	0,002	0,001	<0,001	<0,001	<0,001	<0,001

Table 4

Recurrence of wave heights and periods in the Black Sea, %

$\bar{\tau}$, s	$h_{3\%}$, m											
	0–1	1–2	2–3	3–4	4–5	5–6	6–7	7–8	8–9	9–10	10–11	11–12
0–1	2,31											
1–2	12,10											
2–3	16,45	10,14										
3–4	8,42	9,95	3,83									
4–5	6,36	8,90	4,34	1,20								
5–6	1,72	4,12	0,85	0,33	0,26							
6–7	0,94	3,11	0,30	0,17	0,14	0,06						
7–8	0,80	0,50	0,10	0,12	0,14	0,11	0,05	0,01				
8–9	0,49	0,21	0,05	0,09	0,13	0,10	0,04	0,01	0,007			
9–10	0,24	0,06	0,02	0,07	0,11	0,08	0,01	0,009	0,006	0,002	0,002	0,001
10–11	0,11	0,007	0,006	0,02	0,02	0,01	0,008	0,008	0,004	0,002	0,002	0,001
11–12	0,06	0,003	0,002	0,006	0,007	0,008	0,008	0,007	0,002	<0,001	<0,001	<0,001
12–13	0,004	0,001	<0,001	0,001	0,005	0,006	0,007	0,006	0,001	<0,001	<0,001	<0,001

Table 5

Recurrence of wave heights and wind velocities in the Black Sea, %

$\bar{W}$, m/s	$h_{3\%}$, m											
	0–1	1–2	2–3	3–4	4–5	5–6	6–7	7–8	8–9	9–10	10–11	11–12
2 – 4	16,22	2,99	1,45	0,20	0,11	0,02	0,01	0,001				
4 – 6	13,67	6,21	2,94	0,90	0,24	0,03	0,006	0,002				
6 – 8	8,87	6,46	1,72	0,76	0,09	0,07	0,02	0,01	0,001	0,001		
8–10	5,34	5,62	1,45	0,42	0,08	0,05	0,02	0,01	0,008	0,001	0,001	
10–12	2,65	3,01	1,05	0,14	0,07	0,04	0,02	0,01	0,005	0,001	0,001	0,001
12–14	1,60	1,30	0,93	0,08	0,05	0,02	0,01	0,006	0,004	0,001	0,001	<0,001
14–16	0,70	0,72	0,45	0,07	0,03	0,01	0,008	0,005	0,003	0,001	0,001	<0,001
16–18	0,53	0,39	0,34	0,05	0,01	0,008	0,006	0,004	0,002	0,001	< 0,001	<0,001
18 – 20	0,42	0,32	0,08	0,03	0,009	0,007	0,005	0,003	0,002	<0,001	< 0,001	<0,001
20 – 22	0,01	0,06	0,07	0,01	0,007	0,006	0,003	0,001	0,001	<0,001	< 0,001	<0,001
22 – 24	<0,001	0,04	0,05	0,006	0,005	0,005	0,002	0,001	0,001	<0,001	< 0,001	<0,001
24 – 26	<0,001	0,02	0,03	0,002	0,002	0,001	0,001	<0,001	<0,001	<0,001	< 0,001	<0,001
26 – 28	<0,001	0,009	0,01	0,001	0,001	0,001	0,001	<0,001	<0,001	<0,001	< 0,001	<0,001

Table 6

Recurrence of wave heights and periods in the Barents Sea, %

$\bar{\tau}$, s	$h_{3\%}$, m													
	0–1	1–2	2–3	3–4	4–5	5–6	6–7	7–8	8–9	9–10	10–11	11–12	12–13	13–14
0–1	0,51													
1 – 2	1,62													
2 – 3	3,65	4,22												
3 – 4	2,75	16,18	8,03											
4–5	1,88	10,92	6,03	2,21										
5 – 6	0,82	3,33	5,86	5,72	3,64									
6 – 7	0,46	1,18	2,98	2,35	2,05	1,03	0,75							
7 – 8	0,15	0,59	1,73	0,99	0,43	0,35	0,21	0,19	0,08	0,06				
8 – 9	0,08	0,46	1,02	0,72	0,19	0,18	0,12	0,11	0,07	0,05	0,01	0,008		
9 – 10	0,05	0,07	0,78	0,57	0,14	0,13	0,10	0,10	0,06	0,04	0,02	0,01	0,007	0,006
10–11	0,01	0,03	0,44	0,32	0,06	0,06	0,05	0,05	0,02	0,02	0,02	0,01	0,008	0,003
11 – 12	0,01	0,009	0,12	0,10	0,02	0,02	0,02	0,01	0,01	0,01	0,01	0,008	0,006	0,001
12–13	0,006	0,007	0,007	0,009	0,01	0,01	0,01	0,01	0,01	0,009	0,007	0,006	0,005	0,001
13 – 14	0,003	0,003	0,003	0,008	0,01	0,01	0,01	0,009	0,009	0,008	0,007	0,006	0,003	<0,001
14–15	0,001	0,001	0,001	0,004	0,006	0,006	0,007	0,008	0,009	0,007	0,006	0,005	0,002	<0,001

Table 7

Recurrence of wave heights and wind velocities in the Barents Sea, %

$\bar{W}$, m/s	$h_{3\%}$, m													
	0 – 1	1–2	2 – 3	3 – 4	4 – 5	5 – 6	6 – 7	7 – 8	8 – 9	9 – 10	10–11	11 – 12	12–13	13 – 14
2 – 4	3,56	8,02	1,14	0,21	0,05	0,02	0,004							
4 – 6	4,67	9,56	3,51	0,56	0,45	0,10	0,07	0,04	0,03	0,009	0,002			
6 – 8	2,30	7,60	5,65	1,58	0,67	0,11	0,08	0,06	0,01	0,007	0,003			
8 –10	0,47	5,96	5,43	3,00	1,16	0,40	0,16	0,10	0,04	0,005	0,003			
10–12	<0,001	3,65	4,92	2,61	0,34	0,21	0,12	0,10	0,03	0,02	0,007			
12–14	<0,001	1,98	3,61	2,08	0,31	0,18	0,08	0,06	0,03	0,02	0,008			
14–16	<0,001	0,23	2,04	1,97	0,23	0,17	0,05	0,04	0,03	0,02	0,001			
16–18	<0,001	0,006	0,55	0,50	0,19	0,16	0,05	0,04	0,03	0,01	0,01	0,008	0,006	
18 – 20	<0,001	<0,001	0,15	0,32	0,16	0,15	0,04	0,03	0,03	0,01	0,01	0,01	0,005	0,001
20 – 22	<0,001	<0,001	<0,001	0,09	0,09	0,08	0,04	0,03	0,02	0,01	0,01	0,01	0,004	0,002
22 – 24	<0,001	<0,001	<0,001	0,07	0,06	0,06	0,03	0,02	0,02	0,01	0,01	0,008	0,004	0,002
24 – 26	<0,001	<0,001	<0,001	0,01	0,01	0,01	0,01	0,01	0,02	0,01	0,01	0,008	0,003	0,001
26 – 28	<0,001	<0,001	<0,001	0,005	0,006	0,007	0,008	0,009	0,009	0,009	0,009	0,007	0,003	0,001
>28	<0,001	<0,001	<0,001	<0,001	<0,001	0,001	0,005	0,009	0,009	0,009	0,008	0,005	0,002	0,001

Table 8

Recurrence of wave heists and periods in the Okhotsk Sea, %

$\bar{\tau}$, s	$h_{3\%}$, m														
	0–1	1–2	2–3	3–4	4–5	5–6	6–7	7–8	8–9	9–10	10–11	11–12	12–13	13–14	14–15
0 – 1	0,15														
1–2	1,32														
2 – 3	1,46	1,70													
3 – 4	6,26	7,54	4,88												
4 – 5	5,54	7,22	3,99	3,56											
5 – 6	3,88	6,82	3,82	2,52	1,24										
6 – 7	0,85	5,41	2,50	1,28	0,77	0,55									
7 – 8	0,24	3,96	2,38	0,60	0,58	0,51	0,34	0,07							
8 – 9	0,12	2,48	2,32	0,45	0,26	0,22	0,11	0,14	0,05						
9–10	0,09	1,39	1,75	0,21	0,17	0,15	0,10	0,09	0,07	0,06	0,04	0,02	0,006	0,006	
10–11	0,03	1,11	1,10	0,17	0,15	0,12	0,09	0,07	0,04	0,03	0,03	0,02	0,005	0,004	0,003
11–12	0,02	0,47	0,97	0,11	0,08	0,06	0,04	0,03	0,02	0,02	0,02	0,01	0,004	0,003	0,002
12–13	0,01	0,03	0,64	0,03	0,02	0,02	0,01	0,01	0,01	0,006	0,002	0,001	0,001	0,001	0,001
13–14	0,006	0,02	0,08	0,02	0,01	0,01	0,009	0,005	0,005	0,002	0,001	0,001	0,001	0,001	0,001
14–15	0,004	0,01	0,007	0,005	0,003	0,003	0,002	0,002	0,002	0,002	0,001	<0,001	<0,001	<0,001	<0,001
15–16	0,002	0,002	0,003	0,003	0,001	0,001	0,001	0,001	<0,001	<0,001	<0,001	<0,001	<0,001	<0,001	<0,001
16–17	0,001	0,001	0,002	0,002	<0,001	<0,001	<0,001	<0,001	<0,001	<0,001	<0,001	<0,001	<0,001	<0,001	<0,001

Table 9

Recurrence of wave heights and wind velocities in the Okhotsk Sea, %

$\bar{W}$, m/s	$h_{3\%}$, m														
	0–1	1–2	2–3	3–4	4–5	5–6	6–7	7–8	8–9	9–10	10–11	11–12	12–13	13–14	14–15
2 – 4	3,60	13,20	2,12	0,92	0,11	0,03	0,01	0,008	0,002	0,001	0,001	0,001			
4 – 6	8,12	6,27	2,26	1,17	0,13	0,06	0,02	0,01	0,002	0,001	0,001	0,001			
6 – 8	5,00	6,43	3,69	2,41	0,31	0,08	0,05	0,02	0,005	0,003	0,003	0,001	0,001		
8–10	2,96	7,98	3,06	1,71	1,01	0,16	0,05	0,03	0,01	0,006	0,005	0,002	0,001		
10–12	0,16	7,86	2,69	1,20	0,63	0,24	0,10	0,05	0,03	0,02	0,01	0,005	0,001		
12–14	0,14	5,18	2,34	1,03	0,55	0,42	0,14	0,11	0,03	0,02	0,02	0,005	0,001	0,001	
14–16	<0,001	1,27	1,49	0,71	0,54	0,51	0,18	0,17	0,04	0,03	0,03	0,01	0,003	0,001	0,001
16–18	<0,001	0,01	0,69	0,36	0,31	0,25	0,21	0,04	0,04	0,04	0,002	0,01	0,004	0,002	0,001
18 – 20	<0,001	0,01	0,61	0,25	0,17	0,11	0,08	0,02	0,02	0,02	0,01	0,005	0,003	0,002	0,001
20 – 22	<0,001	<0,001	0,56	0,03	0,14	0,08	0,04	0,02	0,01	0,009	0,008	0,005	0,004	0,003	0,002
22 – 24	<0,001	<0,001	0,15	0,02	0,10	0,06	0,02	0,01	0,008	0,006	0,007	0,003	0,003	0,001	0,001
24 – 26	<0,001	<0,001	<0,001	<0,001	<0,001	< 0,001	0,01	0,01	0,007	0,004	0,004	0,003	0,002	0,001	0,001

APPENDIX 2

REQUIREMENTS FOR DESIGN MODELS BASED ON FINITE ELEMENT METHOD

1 GENERAL

1.1 The calculation of a structure by the finite element method generally comprises the following stages:

definition of the type and size of a problem;

drawing up of the finite element model of a structure and boundary conditions;

simulation of loads;

estimation of model correctness and calculation performance;

presentation of obtained results.

1.2 In modelling of a structure, boundary conditions and loading, depending on the calculation objectives and structure type, the certain assumptions and simplifications are possible and necessary. The particular potentials of a calculation are defined by the parameters of software and hard ware as well as by the size of a problem. The size of a problem may change with the accumulation of information on the peculiarities of a structure operation.

2 DEFINITION OF TYPE AND DIMENSIONS OF DESIGN MODEL

2.1 The type of a deflected state, and the size of a problem as it affects the simulation of a structure, boundary conditions and loading shall be defined.

2.2 For MODU structures, deformations and stresses are divided into the following types depending on external loads and structure operation conditions:

general deformations and stresses in MODU structures;

local deformations and stresses in structural members;

concentration stresses and deformations in local zones of a structure and in intercostal members.

2.3 The objective of a calculation and the load simulation technique shall comply with one of the above types of a structure deflected state.

2.4 The problem size and thus the dimensions of a design model are defined by the model boundaries selected and by the necessary dimensions of a finite element mesh.

2.5 The problem nature (linear or nonlinear) depends on structure features and deformation values. If the parameters of a structure deflected state are determined under regulated design loads, the linear calculation is usually sufficient, particularly for thick-slab structures. Nonlinear effects are caused by material properties, large deformations and of essential importance in the following cases:

for relatively flexible structures with large deformations (geometric nonlinearity);

in investigation of partial failure of structure elements, e.g. the loss of flat panels buckling strength;

when plastic deformations in the structure area happen (physical nonlinearity).

3 STRUCTURE SIMULATION

3.1 Selection of design model types.

3.1.1 In calculations of MODU structures strength it is recommended to apply the following design models:

general model of the MODU hull;

model of a hull structure or large hull component;

model of a grillage;

framed model;

local models.

3.1.2 For all models excepting the general hull model it shall be ensured the introduction of boundary conditions for correct compliance with the conditions of interaction with adjacent structures. Where the results may adversely be affected by idealized boundary conditions, the distance between model boundaries and the structure area under consideration shall be increased.

3.1.3 The general hull model shall be used for determination of general stresses in the MODU structure. The three-dimensional simulation of the main members of a hull allows to ensure the application of loads in the form, which is the best for simulation of a real case, and to simulate the behaviour of complex hull structures with a high accuracy.

3.1.4 The model of a hull structure or large hull component (usually a three-dimensional model) shall be used for determination of general stresses in the hull part under consideration.

3.1.5 The model of a grillage shall be used for determination of general and local stresses in flat structures formed by shell plates strengthened at one or both sides with stiffeners and/or walls (grillages like a double bottom, bulkheads, decks), and also for the calculation of a transverse load transmitted to a grillage rest and for the estimation of deformations and stresses associated with it.

3.1.6 The framed model shall be used in calculation of the strength of structures deformed (mainly, bent) in their plane, e.g. of the transverse members of the MODU pontoon.

3.1.7 Local models are recommended for use in calculations of the strength of separate structure elements and for determination of concentration stresses in components of structures and intercostal members.

3.2 Selection of finite elements type.

3.2.1 The type of a finite element assumed in performance of the strength calculation on each particular problem is of crucial significance. So, in selection of the finite element the recommendations given below shall be followed.

3.2.2 In calculations of structure strength the following types of elements are recommended for use:

bar elements (one-dimensional elements having axial stiffness, but without flexural stiffness);

beam elements (one-dimensional elements having axial, shear, flexural and torsional stiffness);

elements of a plane stress state (two-dimensional elements having membrane stiffness in the plate plane, but without flexural stiffness about the axes in the plate plane);

plate and shell elements (two-dimensional elements having membrane, flexural and torsional stiffness);

solid elements (three-dimensional elements);

boundary and spring elements.

When different type elements are used, emphasis shall be focused on jointedness of displacements and on the possibility of ultimate loads and stresses transfer, particularly when the elements having flexural stiffness and without it are joined in nodes.

3.2.3 The element types selected shall reflect deformations and stresses for the loading conditions under analysis and, when needed, inherent values or limit states in determination of the ultimate load value.

3.2.4 It shall be defined to which extent in the given specific strength calculation the bending of structure components shall be considered. In cases of pure bending behaviour in accordance with the theory of beam bending or plate bending, particularly for flat panels, stiffeners, grillages and transverse frames, the beam and plate elements are suitable. Where the elements of a plane stress state or solid elements are used, then for a possibility to allow for bending in the plane of the largest stiffness, the finite elements with additional intermediate nodes shall be selected or a more fine mesh shall be used.

3.2.5 If general deformations and stresses are only determined, the elements of a plane stress state may be used for three-dimensional models. In this case only the membrane stiffness of a simulated flat structure is considered.

3.2.6 Structural braces of minor importance, e.g. plate stiffeners, are taken into account with the degree of conditionality which is defined by the contribution of these braces into the deflected state being analyzed.

3.2.7 If brace bending in the case considered is of importance, the flexural stiffness of the brace shall be more precisely simulated (e.g. a web is simulated by flat elements, and a loose flange, by a bar or plate element). In some cases, flexural stiffness shall be taken into account by additional beam elements.

3.2.8 In other cases, stiffeners may be considered arbitrarily in the form of an additional plate thickness. As the generalized stiffness of a strengthened plate is different in mutually orthogonal directions depending on the orientation of stiffeners, it is taken into account in design models by the introduction of orthotropic properties for the plate of an effective thickness

$$E_2 = E_1(F_{pl} + F_{st})/F_{pl}; \qquad (3.2.8)$$

$$E_1 = E$$

where E = initial modulus of elongation for a plate material;
E_1 = modulus of elasticity in the direction orthogonal to the stiffeners orientation;
E_2 = modulus of elasticity in the direction parallel to the stiffeners orientation;
F_{pl} = area of a plate cross-section;
F_{st} = area of a stiffener cross-section.

3.2.9 In local models, all stiffness components including the secondary ones, are of a significant importance; this being so, the plate, shell and solid elements are used. Exception may relate to flat structures loaded in its plane. For instance, in analysis of concentration stresses at cutout edges they are simulated by the plates of a plane stress state.

3.2.10 In order to obtain the information on deformations between two nodes, e.g. at free edges of a plate, the bar elements of a negligible cross-section shall be introduced. Uniaxial stresses of such an element present edge stresses.

3.3 Break-down into finite elements.

3.3.1 The size of a finite element mesh is defined by the characteristics of finite elements and shall be selected subject to sufficient accuracy in simulation of:

the stiffness parameters of a structure;

the type of stresses analyzed;

potential failure forms.

The recommendations given below shall be followed while selecting the dimensions of the finite element mesh.

3.3.2 In selection of a finite element mesh the structure geometry, load disposition and nature, and supports layout shall be properly taken into account.

3.3.3 The three-dimensional models of a structure as a whole or parts of the structure may have a rather gross idealization. As the characteristic size of a finite element may be used the frame spacings of the main structure components. It is allowable in calculations of a general stress state provided that the flexural behaviour of the main structure components is reflected by the selected type of a finite element with an adequate accuracy. The same relates to grillage models and to the models for the calculation of local strength of stiffeners if the width of elements in shell plates is equal to the stiffeners spacing or its half.

3.3.4 The element characteristics and its dimensions shall be so selected that stiffness, resulting deformations and stresses may properly reflect the structure behaviour. For simple finite elements the ratio of element side dimensions shall not be usually more than three.

3.3.5 In computation of local concentration stresses the size of a finite element mesh shall be varied gradually in accordance with the stress gradient expected.

3.4 Introduction of simplifying assumptions.

3.4.1 Due to the complexity of a MODU structure the assumptions aimed at simplifications shall be introduced in simulation. Simplifications are acceptable if they do not give rise to significant errors in the results.

3.4.2 A typical simplification in overall strength calculation is the integration of several components of a structure into a single one. Integration may concern stiffeners or beams. Integrated components shall have an equivalent stiffness and be placed in the geometrical centre of composing components.

3.4.3 Small components and pieces, which define the stiffness of small parts, may be completely ignored in simulation. The example of such components and pieces for the calculation of overall strength is small cutouts, frame brackets, stiffeners, reinforcements which prevent buckling.

3.4.4 Large cutouts (cutouts for access into internal spaces, windows and doors) shall be always taken into account. With a structurally stable finite element mesh, such cutouts are taken into account by the stiffness reduction at the expense of the element thickness reduction or the reduction of a modulus of rigidity and a modulus of elongation in longitudinal and transverse directions.

3.4.5 Flat elements shall be placed in the middle surface of the relevant components of a structure. For the analysis of general strength of thin-walled structures, the elements, as an approximation, may be arranged along the lines of an external surface.

3.4.6 Flat two-dimensional elements in inclined or curved surfaces shall be usually placed in the geometrical centre of the area simulated in order to reflect with the greater accuracy general stiffness characteristics.

3.5 Boundary conditions and fixings.

3.5.1 Assignment of boundary conditions and fixings is intended for:

elimination of displacements and turns of a model as a rigid entity;

allowing in a design model for actually existing supports and fixings;

allowing for the interaction of the model for the part of a structure along its boundaries with adjacent parts.

Kinematic boundary conditions and fixings are introduced by the assignment of prescribed values for displacements and turning angles in nodal points of a design model. In introduction of fixings the appearance in the model of nonexistent, in actual behaviour of a structure, restrictions for displacements and turning angles shall be avoided.

3.5.2 The exception of displacements and turns of a model as a rigid body (FEM programs do not ensure the automatic exception of such displacements) shall be materialized by means of introduction of supports or fixings in various sections of the model. The reactions at these supports and fixings that are lacking in actual structures shall be kept to a minimum by means of loading the model by a self-balanced system of loads. The displacements and turns of a solid body may be eliminated by introduction in the design model of a distributed elastic foundation by means of spring elements what, for instance, may closely agree with the actual conditions of a MODU hull position afloat.

3.5.3 Actual supports that take forces and moments shall be simulated with the high degree of approximation to actual conditions.

3.5.4 The interaction of hull structure parts with adjacent structures along the model boundaries shall be simulated with the possibly high degree of approximation to a reality. The structure symmetry shall be taken into account and the model shall be developed for its symmetric part only. The conditions of a symmetric and antisymmetric deformation are introduced across symmetry planes and a load is resolved into symmetric and antisymmetric components. The interaction along a boundary shall be taken into account by the relevant assignment of stresses, forces and moments. These values are obtained as the result of a structure calculation according to a general model.

3.5.5 In use of some element types the need, due to nonexistent stiffness, in suppressing degrees of freedom in nodes may arise. In doing so, the restrictions of actual deformations are not allowed. If, with degrees of freedom suppressed, the elements give additional stiffness, their dimensions shall be so selected that they may provide the stiffness reasonably reflecting actual behaviour.

4 LOAD SIMULATION

4.1 Loads shall be simulated with a high degree of approximation to a reality. When needed, structure simulation shall be adapted for load simulation.

4.2 Distributed loads during calculations are converted into equivalent nodal forces and into nodal moments in accordance with the finite element type in use.

4.3 If the deformations along the boundary of a local model are obtained from the calculation according to the general model of a structure with a structurally stable mesh, the relevant interpolation of a deformation for the intermediate nodes of the local model shall be used. In addition, the relevant loads acting within the local area of the structure shall be applied.

5 ASSESSMENT OF RESULTS VALIDITY

5.1 The results shall be verified for validity. Such a verification includes:

special visual display of deformation for the assessment of their distribution compliance with the loads applied, boundary conditions, supports and fixings;

check of compliance of the deformation values obtained with the range expected.

5.2 It shall be checked whether the values of forces and moments at supports comply with the values expected. For self-balanced loads it shall be checked whether reaction forces are small enough to be negligible.

5.3 For local models with preassigned deformations at a boundary obtained from the general models of structures, the mutual compliance of the stress near the boundaries in question for two models shall be checked.

5.4 For nonlinear calculations the exactness of a solution in a nonlinear zone shall be checked.

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