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Research paper

Reliability Estimation of Oil and Gas Trunk Pipelines on a Stochastic Heterogeneous Base

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Abstract

The article discusses buried oil and gas pipeline linear part reliability. Reliability parameters are longitudinal and hoop stresses in the pipeline wall, which are caused by the impact of the soil base uneven deformations, operating pressure and temperature difference. Estimation of the pipeline reliability in soils without special properties was performed. Improving the probabilistic solution of the differential equations curved pipe axis was carried out. Numerical FEM modelling was proposed for modelling system “pipeline – collapsible basis”. It gives the most relevant and simplest results. The most dangerous cases of the system deformations were defined by increasing the soaking area of the base. Justification of the distribution laws was performed for the input parameters of the pipeline reliability function. Field studies allowed us to obtain 200 random variables of the mechanical characteristics of the pipeline collapsible bases in a water-saturated state. The pipeline reliability in collapsible strata with different thickness was estimated. There were established dependencies for pipeline strength and reliability laid in soil conditions without special properties, estimated that small deformations do not require an increase in the thickness of a pipeline wall, but the growth of deformations leads to reduction in the pipeline reliability. It is necessary to ensure normative reliability, almost linear increasing in the pipeline wall.

Keywords: *Buried main pipeline linear part; collapsible strata; failure probability; numerical modelling; random variables; reliability; stochastic functions; wall thickness.*

1. Introduction

Most of the main pipelines, which are laid on the territory of Ukraine, have been exploited for 30-35 years, while their normative term of exploitation is often exceeded. The latter indicates a significant amount of work associated with the full replacement of the linear part, or at least its re-isolation. According to the modern standards of reliability, pipelines are classified into the most relevant objects (the class of consequences - CC3), liability category - A (structures and elements whose failure may lead to the complete unfitness of a building or structure) [1 – 4].

The probability of pipeline failure for the established combinations of loads at the first limit state is 1·10-6, for emergencies it equals to 1·10-5. For such objects, it is imperative to carry out a reliability calculation, but there is no specific methodology that would allow us to assess the reliability of the pipelines [5, 6].

The main factors effecting the linear part of a trunk pipeline (LPTP) are internal operational pressure, temperature variations, uneven deformations of a base, associated with a significant change in its physical and mechanical properties [7, 8]. Fluctuations in operation-al pressure have been studied well and have a statistically substantiated reliability index for loading [9], but the stresses are not yet regulated due to deformation and collapsing of the base.

Despite significant number of studies on the strength of pipelines in difficult engineering and geological conditions, regulations do not provide methods for determining the longitudinal stresses arising in the pipeline from the deformation of the base. The analysis of sources [10 – 14] allows us to conclude that in Ukrainian and foreign norms, the loads and influences caused by the internal operational pressure and temperature fluctuations have been analyzed sufficiently, but there is no method of taking into account the effects associated with uneven deformations of a base, as a result, an un-grounded increase in the thickness of the wall of the pipeline for soils without special properties and its understatement for complex geotechnical conditions may occur.

From the perspective of Ukraine's geotechnical conditions, it should be noted that the issues of the strength of pipelines with the stresses arising from the base collapsing is not considered by regulations [15]. A settlement occurs unevenly, and the deformability parameters of the soil are of a stochastic nature [16]. The shortcomings of the work include the absence of specific distribution laws and statistics of loads and influences associated with the deformation of the pipeline base. The latter does not allow determining the probability of pipeline failure by the parameter of longitudinal stresses [17].

Thus, the derivation of the correct deformation laws for modeling, obtaining numerical values of the heterogeneity of its characteristics, is an actual task in the context of a probabilistic description of influences on LPTP and determining its reliability [18]. Due to the probabilistic simulation by the finite element method (FEM) it is possible to calculate the values of total longitudinal stresses in the walls of the pipeline [19 – 21].

Hence the purpose of the work is to assess the reliability of the linear part of the underground trunk pipeline by the parameter of annular and longitudinal stresses, which is influenced by deformations of a stochastic heterogeneous base, internal operational pressure, and temperature variations.

2. Main Body

2.1. Reliability estimation of faultless operation of a pipeline by the parameter of annular stresses

In this calculation, the external influences associated with the weight of the soil and the uneven subsidence of the base are absent. The statistical parameters of internal operational pressure are given in Table. 1.

Table 1: The input parameters for determining the reliability of the LPTP interval

Parameter	Value	Interval
External diameter	D_{ex}, m	1,020
The pipe wall thickness used during construction	t , m	0,0096
The expected value of the pipeline steel strength	$\overline{\sigma}_y$, kN/cm^2	58,6
A standard for the pipeline steel strength	$\hat{\sigma}_y$, kN/cm^2	5,86
Interval length	L , m	6036
The expected value of the fill-up soil weight	$\overline{q}_1$, kN/m	32,1
The expected value of the weight of pipeline, isolation, and products	$\sum\overline{q}_{2-4}$, kN/m	2,72
The expected value of linear index of an elastic base	$\overline{c}_{yo}$, kN/m^2	1347
A standard for linear index of an elastic base	$\hat{c}_{yo}$, kN/m^2	482
The expected value of a temperature drop (closure in the warm season)	$\overline{\Delta t}$, ^{0}C	30,6
A standard for a temperature drop (closure in the warm season)	$\Delta\hat{t}$, ^{0}C	10,2
The expected value of an internal pressure at the pipeline entrance	$\overline{p}$, kN/cm^2	0,456
A standard for an internal pressure at the pipeline entrance	$\hat{p}$, kN/cm^2	0,0314

Since in a function of the strength reserve Y by the parameter of annular stresses σ_{AS} there's only an operational stress p and the pipeline steel strength σ_Y, then the mathematical expectation and the standard of the pipeline strength reserve according to the annular stress parameter are determined according to the formula (1)

$$\overline{Y} = \overline{R} - \overline{S} = \overline{\sigma}_y - \overline{\sigma}_{AS} = \overline{\sigma}_y - \frac{\overline{p}D}{2\mathrm{t}_{\mathrm{fact}}} \quad (1.a)$$

$$\hat{Y} = \sqrt{\hat{R}^2 + \hat{S}^2} = \sqrt{\hat{\sigma}^2{}_y + \hat{\sigma}^2{}_{AS}} = \sqrt{\left(\hat{\sigma}_y\right)^2 + \left(\frac{\hat{p}D}{2\mathrm{t}_{\mathrm{fact}}}\right)^2} \quad (1.b)$$

Safety characteristic is the following:

$$\beta = \frac{\overline{Y}}{\hat{Y}} = \frac{\overline{\sigma}_y - \overline{\sigma}_{кц}}{\sqrt{\hat{\sigma}^2{}_y + \hat{\sigma}^2{}_{AS}}} = \frac{\overline{\sigma}_y - \dfrac{\overline{p}D}{2\mathrm{t}_{\mathrm{fact}}}}{\sqrt{\left(\hat{\sigma}_y\right)^2 + \left(\dfrac{\hat{p}D}{2\mathrm{t}_{\mathrm{fact}}}\right)^2}} \quad (2)$$

By the defined safety characteristic, the probability of the LPTP failure is easily calculated (table 2).
Hence, the failure probability of LPTP by the annular stress parameter for a wall thickness calculated according to standards [1], fluctuates within $6\cdot10^{-6} - 2\cdot10^{-10}$. At the same time for the actual construction, there were used pipes with $t_n = 9{,}6$ mm, for which the probability of failure was $1\cdot10^{-8}$. The obtained values of the failure probability may be used as a benchmark for the following assessment of the reliability of the pipeline by the parameter of longitudinal stresses.

Table 2: The pipeline failure probability (D_{ex} = 1020 mm) by the annular stress parameter for the wall thickness calculated by SNiP method [1]

	Nominal safety class and respective wall thickness			
	Norm.	Average	**Design**	High
	t_n =7,4 mm	t_n =8,7 mm	t_n =9,6 mm	t_n =10,8 mm
EV of annular stress, $\overline{\sigma}_{AS}$, MPa	312,2	265,5	**256,7**	213,9
Standard for annular stress, $\hat{\sigma}_{AS}$,MPa	21,6	18,4	**17,7**	14,8
EV of a steel strength $\overline{Y}$, MPa	273,8	320,4	**329,3**	372,1
Standard for a steel strength $\hat{Y}$, MPa	62,5	61,4	**61,2**	60,5
Safety characteristic, β	4,38	5,22	**5,38**	6,16
Failure probability, $Q(\beta)$	$6\cdot10^{-6}$	$5\cdot10^{-8}$	**$1\cdot10^{-8}$**	$2\cdot10^{-10}$

2.2. Reliability assessing of a faultless pipeline operation by the parameter of longitudinal stress

For longitudinal stresses in the pipeline in soils without special properties, the LPTP strength condition goes down to the requirement that the total wall stresses in modulus $\left|\sum\sigma_l\right|$ would be less than yielding strength of the pipe metal σ_y . The reliability function for LPTP is

$$P(L) = 1 - Q(L) = P[\sup_{0\le x\le L}\left|\sum\sigma_l\right| < \sigma_y] \quad (3)$$

where $\sup_{0\le x\le L}\left|\sum\sigma_l\right|$ is a top limit of the function values $\left|\sum\sigma_l\right|$ within the range of $0 \le x \le L$; $Q(L)$ is the failure probability function for LPTP along the pipeline length L.
The law of distribution of the carrying capacity of the pipeline is of Gaussian nature, which is due to the large number of input parameters. By influence on the final function it is possible to rank them as follows: a steel strength $\tilde{\sigma}_y$; internal operational pressure $\tilde{p}_{in}$; temperature fluctuations $\Delta\tilde{t}$; bending moment $\tilde{M}$ in the pipeline, which in turn is determined by the change in the stiffness of the base, $\tilde{c}_{yo}$ – parameter of the soil deformability.

$$\tilde{Y} = f(\tilde{\sigma}_y, \tilde{N}_{\Delta t}, \tilde{N}_p, \tilde{M}) \quad (4)$$

Of the above parameters, only the latter has a distribution law other than normal. Therefore, the distribution of total longitudinal stresses, as well as the reserve of bearing capacity, are also distributed according to normal law. The failure probability will be determined in the form of absolute maxima [22]:

$$Q(L) = \exp[0{,}5(\gamma_0^2 - \beta^2)] \quad (5)$$

where γ_0 is a characteristic maximum of a stochastic function $\sum\tilde{\sigma}_l(x)$; β is a safety characteristic.

According to formula (5), the main parameters for determining the probability of pipeline failure are the safety characteristic β and the spectral density of the curvature of the pipeline $S_{\chi}(\omega)$, which is required to determine the characteristic maximum γ_0. The safety characteristic in formula (5) can be determined based on the function of the pipeline strength reserve (4).

The technological aspect of laying the pipeline should also be accounted for, namely: the closure of the thread of the pipeline, as a rule, it is carried out in the warm season when the temperature difference in the pipeline is negative, causing tensile stresses, longitudinal stresses from the operational pressure are also tensile. Therefore, it is advisable to consider the pipeline as a stretched-bent rod. Longitudinal force in the pipeline N is determined from internal operational pressure and temperature difference.

There is no correlation dependence between the tensile stresses, in contrast to compression, and the bending moment, therefore, as a result of the partial differentiation of function (4), we have a safety characteristic in the following form:

$$\beta = \frac{\overline{\sigma}_y - \dfrac{\overline{N}_{\Delta t} - \overline{N}_p}{A}}{\sqrt{\hat{\sigma}_y^2 + \dfrac{1}{A^2}\hat{N}_{\Delta t}^2 + \dfrac{1}{A^2}\hat{N}_p^2 + \dfrac{1}{W^2}\cdot \hat{M}^2}} \qquad (6)$$

Where $\hat{M} = EI \cdot \left[\int_0^{\infty} S_{\chi}(\omega) d\omega\right]^{1/2}$ – is a standard for a bending moment, which is determined from the spectral density of the pipeline axis curvature; A is the cross-sectional area of the pipeline; W is the moment of resistance of the cross-section of the pipeline.

Expression (7) shows a correlation between the spectral density of the soil conditions heterogeneity function and the curvature of the pipeline axis:

$$S_{\chi}(\omega) = \frac{\omega^4 \cdot S_r(\omega)}{(\omega^4 EI + c_{y0})^2} \qquad (7)$$

where EI is is a bending rigidity of the cross-section of the pipeline; ω is a wave number. By the ω parameter the integration with the spectral representation of correlation dependencies is carried out for an input stochastic function of soil heterogeneity. Main parameters of the function (7): c_{y0} is a unitary coefficient of a normal soil resistance and spectral density of soil heterogeneity function $S_r(\omega)$ along the pipeline length, which we will consider as stationary and ergodic. As a realization, the statistical data on the quality control of a sand compaction is used (fig. 1), which were selected so as to form a route close to the "direct", the distance between the points of sampling by cutting rings is about 20 m. The bulk soil is a fine-grained sand, homogeneous, Poisson's ratio is $\mu = 0{,}3$. A total statistical sample group had 51 values.

According to the results of measurements, the numerical characteristics of the stationary ergodic stochastic function are determined – an expected value and a standard. To determine the normalized correlation function it's necessary to multiply the value of the centered function, separated by intervals between the sections ξ = 0, 20, 40…, and divide the sum of the products respectively by n – 0 = 51, n – 1 = 50 and the dispersion of a stochastic function.

$$k_x(\xi) = \frac{\left(\dfrac{1}{n-m}\right)}{D}\sum_{i=1}^{n-m}\left[\left(x(l_i) - m_x(l)\right)\cdot\left(x(l_{i+m}) - m_x(l)\right)\right] \qquad (8)$$

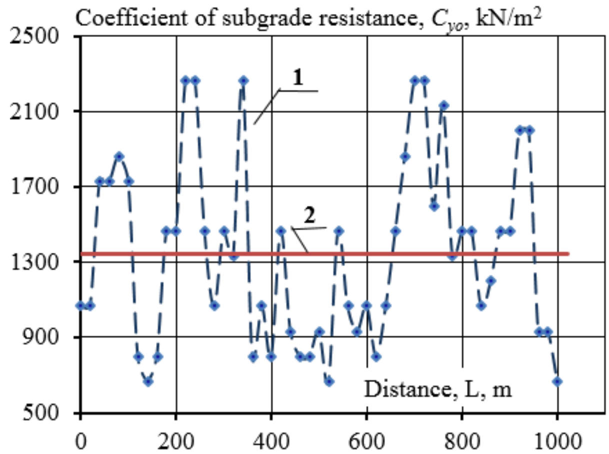

Fig. 1: A stochastic function of the soils' conditions heterogeneity along the pipeline length: 1 is a stochastic function realization; 2 is an expected value of the elastic base index c_{yo}=1347,5 kN/m^2

Approximation of the correlation connection between the sections of the stochastic function of soil heterogeneity was carried out by the following functions (fig. 2):

$$k_x(\xi) = e^{-\alpha|\xi|} \qquad (9)$$

$$k_x(\xi) = e^{-\alpha|\xi|}\cos\beta\xi\,. \qquad (10)$$

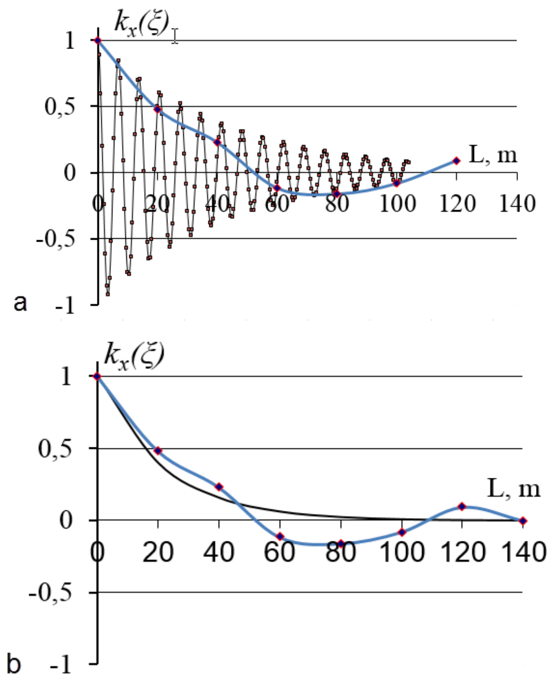

Fig. 2: Approximation of the correlation stochastic function of soil heterogeneity along the pipeline length: a is a function with a periodic component $k_x(\xi) = e^{-0{,}0238|\xi|}\cos 0{,}91\xi$; b is a function without a periodic component $k_x(\xi) = e^{-0{,}046|\xi|}$

From Fig. 2, b, it's possible to conclude that there is a drawback in the function of a periodic component since for its exact construction it is necessary to select a sample of the base with a periodicity of 1 m, which is unrealistic for the construction of a several kilometers length.

In addition, the value of the numerical parameters of functions (9) and (10) is obtained for approximating the experimental correlation (Fig. 2).

Due to the method of spectral representations, it is quite easy to construct a spectral density, which corresponds to the normalized correlation function (NCF) of a stochastic process of soil heterogeneity.

$$S_r(\omega) = \frac{\theta^2 q_0^2}{\pi} \frac{0{,}046}{\omega^2 + 0{,}046^2} \tag{11}$$

$$S_r(\omega) = \frac{0{,}024 \cdot \theta^2 q_0^2}{\pi} = \left[\frac{1}{(\omega - 0{,}911)^2 + 0{,}024^2} + \frac{1}{(\omega + 0{,}911)^2 + 0{,}024^2} \right] \tag{12}$$

where q_0 is total load on the pipeline (the sum of pressures from the upper layers of the soil, pipe weight and insulation); θ – the coefficient characterizing the load distribution (i.e. the sum of the pressure of the upper layers of the soil, the reaction of the base, the weight of the pipe, the hydrostatic pressure of the liquid) and θ = 1 is the initial inequality of the surface upon which the pipeline is laid.

To calculate the reliability level by the parameter of longitudinal stresses of LPTP the experimental data on the gas pipeline repair report was used. By skipping mathematical calculations, the results of the calculation are given in Table. 3.

Table 3: Failure probability of LPTP by the parameter of longitudinal stresses

Parameter	Values	NCF (9)	NCF (10)
Standard for bending moment	$\hat{M}$, kNm	58,6	44,6
Resistance moment of a tensile force	$\bar{N}$, kN	3970	3970
Standard for a tensile force	$\hat{N}$, kN	929	929
Safety characteristic	β	7,07	7,13
Characteristic maximum of stochastic function	γ_0	3,305	2,49
LPTP failure probability	$Q(t)$	$3{,}3 \cdot 10^{-11}$	$1{,}0 \cdot 10^{-10}$

Despite the considerable spread of the values of the strength of the tubular steels and a significant fluctuation in the values of the working pressure and temperature differences, a high level of reliability was obtained by the parameter of longitudinal stresses $1{,}0 \cdot 10^{-10}$ – $1{,}2 \cdot 10^{-12}$, which can be explained by the fact that the stresses caused by uneven deformations of the base without special properties do not exceed 10 – 15 MPa.

2.3. Verification of "LPTP – collapsible base" system

According to the engineering experience and previous research, the hypothesis of the equivalence of the base and pipeline deformations was tested in the work.

Therefore, the following steps are important: 1) selection of the model of the collapsible layer deformation in the natural and water-saturated state from the action of its own soil weight; 2) simulation of the interactive nature of the pipeline with the surrounding soil massif in order to get the correct stress in the pipeline.

For verification of the first stage, the simulation of collapsible soil massif with the depth of 9 m has been carried out. For these conditions, the value of subsidence from its own weight by engineering calculation was $S_{s\ell g} = 266$ mm.

According to FEM simulation (Fig. 3), uneven deformations values of a nonlinear model with Drucker-Prager strength criterion and a simpler one – linear, within the pressures from the soil own weight for such conditions for both models were $\Delta S = 330 - 46 = 284$mm.

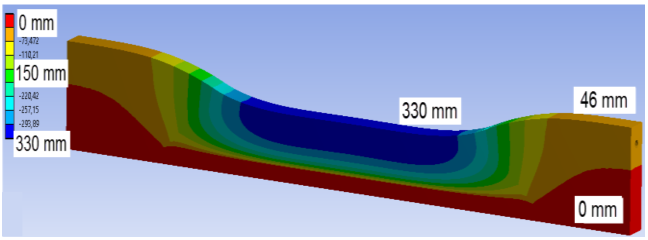

Fig. 3: FEM simulation results of "LPTP – collapsible base" system

From the comparison of the results of modeling and engineering calculations, it's clear that even for the collapsible depth of 9 m the relative difference of the obtained subsidence was only 4.8% in the larger side which goes into the calculation reserves. This proves the correctness of using the linear deformation model of the bases for modeling the soil's subsidence from its own gravity.

To derive a universal design scheme of the LPTP interaction with the surrounding soil massif by FEM modeling 120 examples of watering the base from point and site sources of different lengths are counted (fig. 4, a).

For comparison, 72 examples have been calculated according to the numerical integration of the differential equation of a bent axis of the LPTP by Runge-Kutta method (fig. 4, b). In the study, the pipeline is taken in diameter D_{ex} = 1000 mm. The four parameters varied: the length of the watering area from above B_w = 1, 5, 10, 20, 50 m; the inclination angle of the propagation of moisture in the massif β = 20° , 30°, 40°; the wall thickness of the pipeline t = 10, 15, 20 mm; type of contact between the pipe and the watering spot: Bonded (rigid contact), Frictional (friction between the elements), Contact absent (no contact). At this stage only the deformation of the base considered without internal pressure and temperature deformations affecting the system.

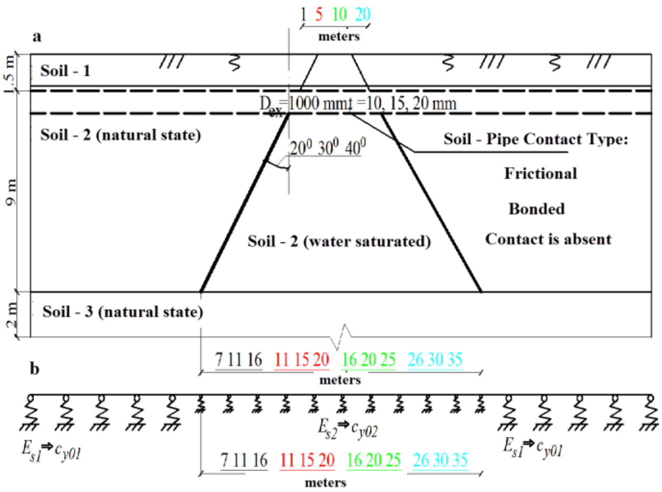

Fig. 4: Calculation schemes for modeling the LPTP on the interval of local soil watering: a – FEM modeling; b – numerical integration by Runge-Kutta method

The deformations modeling of "LPTP – collapsible base" system as a beam on an elastic base has certain features. First, deformations occur due to a significant decrease in the soil repulse inside of the massif. Secondly, the system deforms under its own weight. This is clearly reflected during FEM simulation since only the force of earth gravitation is considered as an effect on the system. For numerical integration in short sections, where the deformation of the pipe with the fill-up soil does not exceed the amount of subsidence, the soil does not create resistance to deformation. Therefore, for all schemes of watering, an option is considered when the repulsive coefficient of the elastic basis is c_{y02} = 0 (fig. 5). For long watering areas B_w > 10 m (10 D_{ex}) the calculation is carried out for cases when c_{y02} = 0 and $c_{y02} = f(E^{sat}_s)$ = 274 kN/m^2. Thus, in the soil, there is resistance to deformations of the pipe when its values exceed the values of the subsidence of the base from the soil own weight.

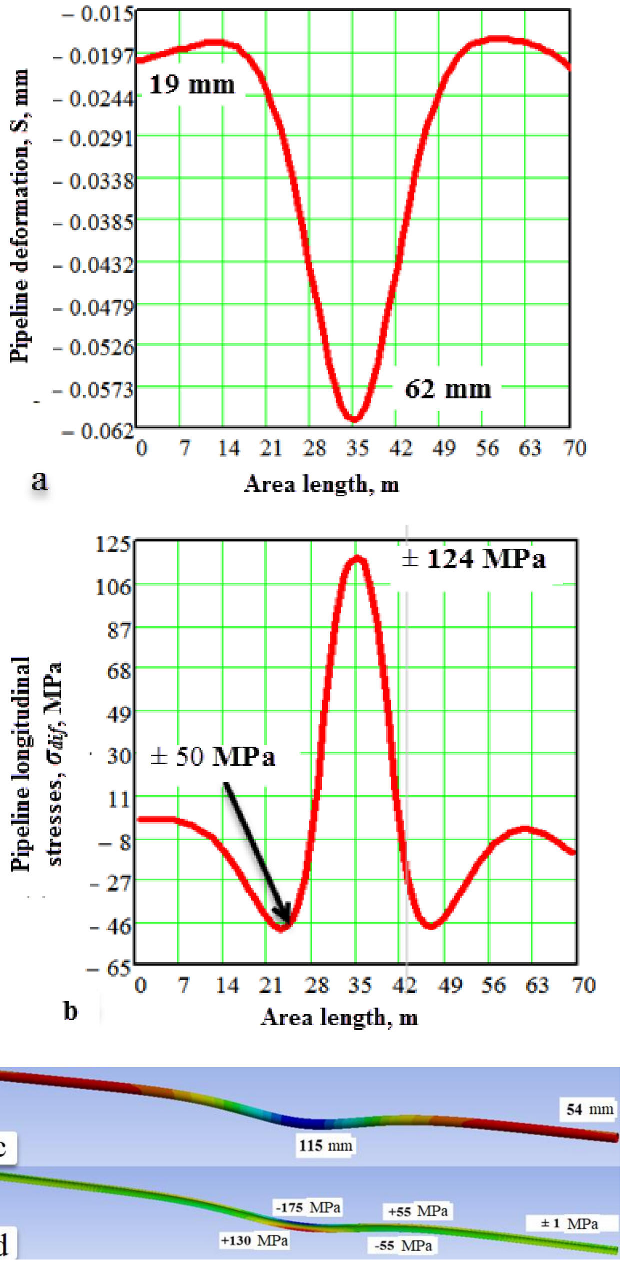

Fig. 5: Comparison of the results of numerical integration and modeling of finite element method (FEM) on the site of the local watering of the base (Dex = 1000 mm, t =10 mm, Bw = 1 – 11 m) a, c - deformation of the pipeline; b, d - longitudinal stresses

The modeling results for the characteristic areas of soil watering are summarized in Table. 4 in which it is marked: S_{slg} is the subsidence of soil from its own weight; S_{sl}^{pipe} is an uneven deformation of the pipeline; σ_{dif} *max(min)* are the maximum and minimum longitudinal stresses in the walls of the pipeline from the influence of uneven base deformations.

It should be noted that the latter, at significant subsidence, exceed the values of the annular stresses from the working pressure.

Comparison of the results of the modeling of finite element method (FEM) and numerical integration shows that the results for the short portions of the jamming are rather close, the relative difference is within 10%.

With the growth of the deformation zone, the relative difference increases (Table 4).

At the same time, the results of FEM modeling are more precise, which is explained by the qualitatively correct method of applying the imposition to the system.

Table 4: System deformations "The linear part of the trunk pipeline (LPTP) - subsidence base" and longitudinal stresses in the pipeline for different types of contact between the elements

Length of soaked area, L, m	The thickness of the pipe wall, *t*, mm	Measured value	FEM modeling for different types of contact between the pipe and the base			Missing pipe	Numerical solution of Runge-Kutta	
			Hard	Friction	Missing contact		c_{y02} = 0	$c_{y02} = f(E^{sat}_s)$
Point source 1 – 16 m	10	S_{slq}, (S_{sl}^{pipe}), mm	105 (96)	110 (96)	107(95)	155	91	
		$\sigma_{dif}^{max(min)}$, MPa	+184 -240	+161 -225	+170 -237		± 190	
	15	S_{slq}, (S_{sl}^{ppei}), mm	101(91)	105(90)	102(90)		85	
		$\sigma_{dif}^{max(min)}$, MPa	+161 -199	+141 -184	+149 -196	-	± 140	
	20	S_{slg}, mm	98 (87)	103 (83)	99 (87)		80	
		$\sigma_{dif}^{max(min)}$, MPa	+144 -174	+126 -160	+135 -170		± 115	
Plot area source 10 – 25 m	10	S_{slq}, (S_{sl}^{pipe}), mm	215 (208)	225 (198)	272 (161)	267	295	94
		$\sigma_{dif}^{max(min)}$, MPa	+295 -356	+262 -328	+155 -207		± 380	± 120
	15	S_{slq}, (S_{sl}^{pipe}), mm	205 (195)	222 (182)	258 (152)		230	90
		$\sigma_{dif}^{max(min)}$, MPa	+256 -301	+219 -263	+140 -176	-	± 293	± 107
	20	S_{slq}, (S_{sl}^{pipe}), mm	198 (186)	215 (174)	276 (145)		200	83
		$\sigma_{dif}^{max(min)}$, MPa	+228 -265	+196 -228	+128 -156		± 234	± 93

From the analysis of the results, it is possible to conclude that the most dangerous case of watering the site B_w = 10 m (10 Dex), β = 40 °.

Such scheme is appropriate to use in subsequent calculations. Table 4 analysis shows that maximum deformations occur with increasing the length of the bare zone B_w to the size of the thickness of the subsidence depth H_{sl}, 10 m for our scheme.

The obtained plot of the dependence of the size of the S_{slg} subsidence on the length of the watering site B_w (Fig. 6, item 1) is rather close to the theoretical one, and, therefore, it can be taken as a reference.

In the presence of the pipeline, the size of the subsidence is significantly reduced, for example, for the watering area B_w = 10 m – S_{slg} = 210 mm or 74% of the maximum, indicating that the stiffness of the pipeline significantly reduces the subsidence value (Fig. 6, pos. 2).

For comparison, when the pipe is absent, for B_w =10 m - the magnitude of the subsidence is 267 mm, which is 94% of the maximum, for the FEM modeling it is 284 mm. Starting from the 20 m area, the value of the base subsidence does not change in fact.

Such an effect is characteristic for point watering sources and a small site width $B_w \leq 1$ - 1.5 H_{sl} (10 Dex), since the effect disappears with increasing $B_w > 2$ H_{sl} (20 Dex).

The thickness of the wall affects the size of the S_{slg} subsidence within 1 - 2% (Fig. 6, Pos. 3, 4).

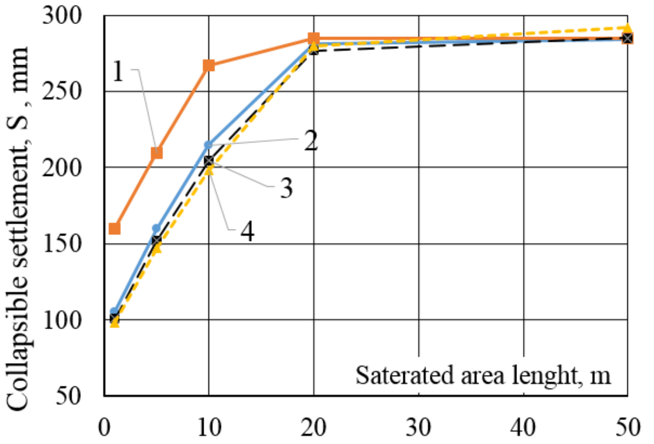

Fig. 6: Dependence of the subsidence value S_{slg} on the length of the locking area B_w 1 is the soil mass without the pipeline; 2 - 4 - bonded contact, wall thickness 10 mm, 15 mm and 20 mm respectively

From the obtained results (Table 4), it's possible to conclude that the type of Bonded contact gives the most correct deformation and resilient LPTP. While using, firstly, the values of subsidence of soil are locally reduced. And secondly, with length increasing of the watering area, if other types of contacts are used, the deformation of the S_{sl}^{pipe} pipe takes values that exceed the deformation of the base.

That is, it is modeled as if the soil under the pipe is not present at all, but the tube absorbs the load from the above-layed layers.

The rigidity of the pipeline is correctly taken into account when simulating using the Bonded contact. So, the hypothesis correctness of equivalence between the subsidence of the base and the deformations of the LPTP is proved.

For the point source of watering, the maximum longitudinal stresses in the pipeline were σ_{dif} = +184 (-240) MPa, for the area maximum stresses obtained for the source B_w = 10 m, σ_{dif} = +295 (-356) MPa, which exceeds the value of the annular stresses from internal pressure. The point source of watering with width B_w = 1 m and width plot B_w = 10 m is used for the next probabilistic FEM modeling.

2.4. Laws of distribution and statistics of soil subsidence parameters in a water-saturated state

As statistical material the data of explorations are used in Kherson city which is object 1; and on the line of the main pipeline in the Poltava region which is object 2.

The statistical material of object 1 is selected from 10 wells with a depth of 13 m. The research highlighted four elements: EGE-2 - EGE-4 are drainage layers. The data collection was 128 values, in which EGE-2 - 52, EGE-3-48, EGE-4-28, thickness - 10.1 m. Object 2 is a depth of 2.3 m in the immediate proximity of the pipeline route.

It is determined that EGE-2 is a subsidence layer.

The total sample of statistical data of the EGE-2 was 24, and the thickness was 6.2 m.

The results of the statistical analysis for each EGE in the pressure range from the soil's own weight, are shown in Fig. 7, the results of the approximation of experimental histograms according to various distribution laws are given in Table. 5.

Notational conventions: $\overline{X}$ is mathematical expectation; $\widehat{X}$ is the dispersion; σ is an average quadratic deviation; v is the coefficient of variation; A is an asymmetry factor; E is kurtosis; χ_{exp}^2 and χ_{tab}^2 are experimental and tabular value of Pearson's criterion in terms of significance α = 0,05 and the number of free degrees depending on the distribution law.

In parentheses you can see the value of the main statistics for the lognormal distribution law. Enclosed cells, where the experimental value of Pearson is less than theoretically, shows that the distribution law can be taken for aprox-simulation.

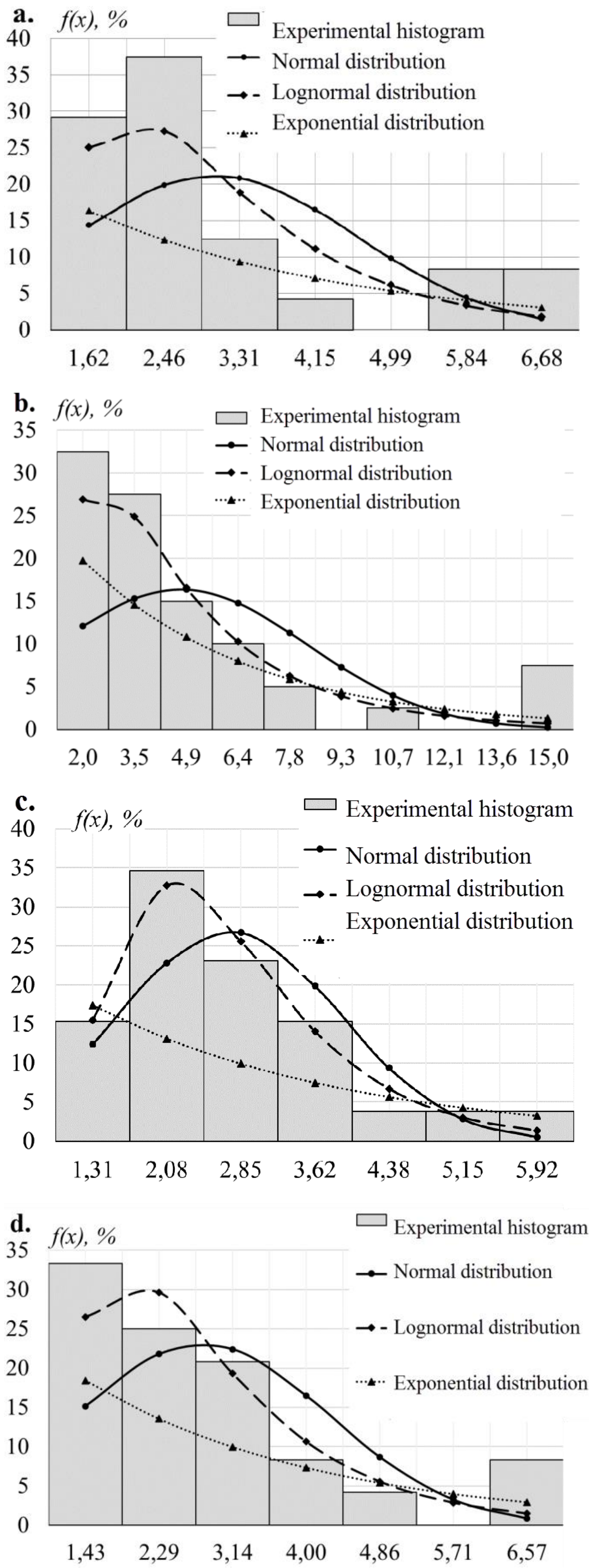

Fig. 7: The distribution of the input random variable in the system reliability function "LPTP – collapsing base": a is Object 1 EGE-2 (0,05 - 0,1 MPa); b

is EGE-3 (0,1 - 0,2 MPa); c is EGE-4 (0,2 - 0,25 MPa); d is Object 2 EGE-2 (0.1-0.15 MPa)

Table 5: Statistical parameters of experimental distributions VV deformation module of subsidence soil

Statistical parameters	$\overline{X}$	$\tilde{X}$	$\hat{X}$	v, %	A_x	E_x	The law of distribution	χ_{exp}^2 and χ_{tab}^2
Object №1 (EGE-2 – loam subsidence)								
Module of deformation E, MPa, when the pressure σ = 0,05…0,1 MPa	4,66 (1,28)	14,93 (0,52)	3,9 (0,72)	0,8	1,5	1,2	Norm.	31,4(14,1)
							Lognorm.	7,1(14,1)
							Exp.	11,6(15,5)
Object №1 (EGE-3 – sand clay subsidence)								
Module of deformation E, MPa, when the pressure σ = 0,1…0,2 MPa	3,27 (0,96)	5,75 (0,43)	2,40 (0,66)	0,7	2,1	3,9	Norm.	53,2(14,1)
							Lognorm.	9,9(14,1)
							Exp.	25,3(15,5)
Object №1 (EGE-4 – loam subsidence)								
Module of deformation E, MPa, when the pressure σ = 0,2…0,25 MPa	2,72 (0,92)	1,29 (0,16)	1,13 (0,4)	0,4	1,0	0,8	Norm.	8,4(14,1)
							Lognorm.	1,8(14,1)
							Exp.	16,5(15,5)
Object №2 (EGE-2 – sand clay subsidence)								
Module of deformation E, MPa, when the pressure σ = 0,1…0,15 MPa	2,79 (0,9)	2,19 (0,25)	1,48 (0,50)	0,5	1,3	1,0	Norm.	7,7(9,5)
							Lognorm.	1,5(9,5)
							Exp.	9,1(11,1)

It should be mentioned that using the normal distribution law, the value of the deformation module in the water-saturated state fluctuates within 2,78 – 5,91 MPa and at all 0,89 – 1,55 MPa - for lognormal, indicating a significant decrease in the characteristics of deformability compared to the deformation module in the natural state 12 – 14 MPa. The coefficient of variation for the water-saturated state fluctuates within the limits of 53 – 89%.

2.5. Failure probability of LPTP taking into account the deformation of the collapsing base

To evaluate the work of the system "LPTP - collapsing base" the data of the surveys, described above, is used. The soaked section width of the array on the top is for the point and site source of 1 and 10 m, respectively, the width of the enclosed massif at the bottom is intended to take into account the distribution of water, depending on the type of soil and the source of soaking for different conditions, was 20 (30) m and 10 (20) m respectively (Fig. 8).
The probabilistic characteristics of capacities, as well as distribution laws and statistics of steel strength of the pipeline, are determined in accordance with Table. 1.
The mechanical properties of the system elements in the water-saturated state, which were used during the modeling, are given in Table. 5.
According to the engineering calculations, the magnitude of the uneven deformation of subsidence from its own weight for each of the calculation schemes was: Object 1 – $S_{slg}=0.342$ *m*, Object 2 – $S_{slg}=0.141$ *m*.
The developed method advantages should include the ease of obtaining the final results, i.e. to determine the probability of failure to use the formula (14).
Thus, the statistics obtained above is part of the reliability function of the pipeline by the parameter of longitudinal stresses, and the complex process of their set and subsequent statistical analysis is offset at the last stage of the research.

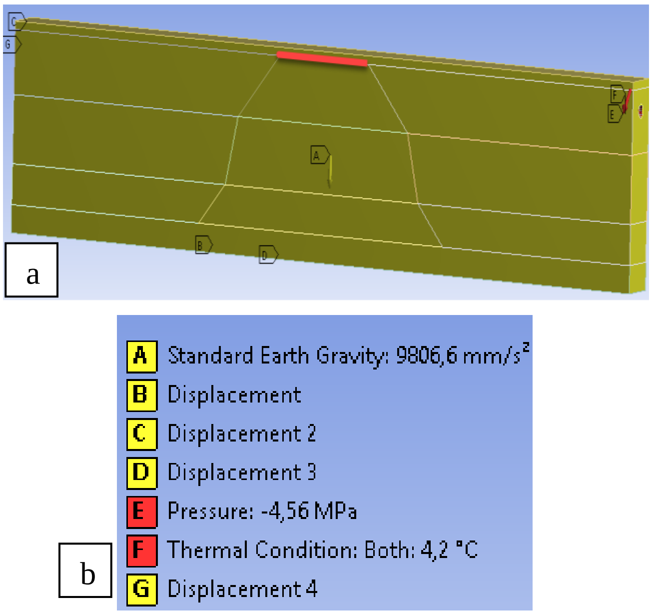

Fig. 8: The calculation scheme for the FEM modelling of "LPTP - subsidence basis" system, area source of soaking (Object 1)

The specificity of laying LPTP is such that it is layed far from the places of artificial soaking, including water-supplying communications. It is fair to note that the appearance of subsidence deformations of its base should be considered as an emergency situation, and therefore, the combined effect of internal working pressure, temperature difference and the effects of deformation of the base is an emergency loading of capacities. Thus, the probability of failure can be set at the level $Q(\beta)=1\cdot10^{-5}$. The criterion for failure in the simulation is considered to be the excess of the fluidity normal stresses of the steel pipeline.
The results of modeling include fields of mathematical expectation of longitudinal stresses in the walls of the pipeline σ_l, appropriate distribution laws in a differential form and cumulates (Figures 9 - 10).

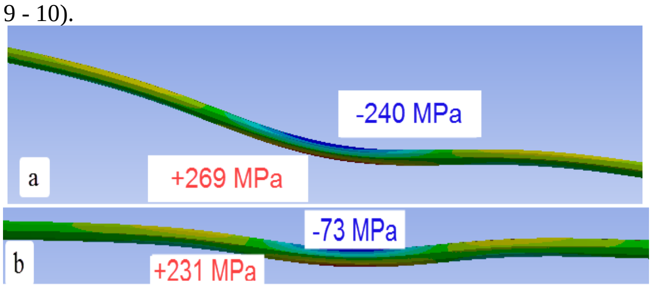

Fig. 9: Longitudinal stresses in the walls of the pipeline: a – Object 1, site watering, subsidence - 324 mm, wall - 22 mm; b - Object 2, site watering, subsidence - 176 mm, wall - 13 mm

In table 6 the longitudinal stresses, obtained as a result of the FEM modeling of the system "LPTP - a subsidence basis" for different capacities of the subsidence depth, are given.
They are used as input stochastic values of the fail-over function of the pipeline (13):

$$\tilde{Y}=\tilde{R}-\tilde{S}=\tilde{\sigma}_y-\tilde{\sigma}_h(\tilde{\sigma}_l)\geq 0 \tag{13}$$

Safety characteristics is according to the data of Table. 6

$$\beta=\frac{\overline{Y}}{\hat{Y}}=\frac{\overline{\sigma}_y-\overline{\sigma}_h(\overline{\sigma}_l)}{\sqrt{\hat{\sigma}_y{}^2+[\hat{\sigma}_h(\hat{\sigma}_l)]^2}}=\frac{586-240{,}0}{\sqrt{58{,}6^2+56{,}9^2}}=4{,}23 \tag{14}$$

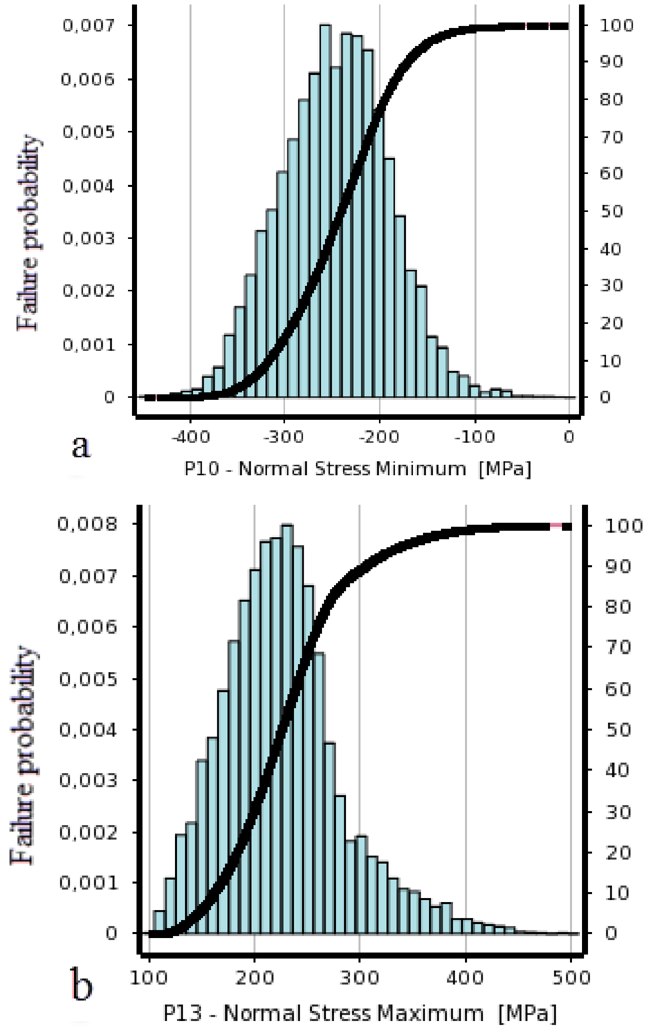

Fig. 10: Distribution laws for longitudinal stresses in a pipeline: a - object № 1, wall - 22 mm; b - object № 2, wall - 13 mm

Table 6: The results of probabilistic modelling

	Parameter		Marking	Value	Law distribution
Object № 1 t = 22 mm	The capacity in the pipeline walls	EV	$\overline{\sigma}_y$, MPa	-240,0	
		Standard	$\hat{\sigma}_y$, MPa	56,9	
	Safety characteristic		β	4,23	
	The probability of refusal		$Q(\beta)$	$1{,}0\cdot10^{-5}$	
Object № 2 t = 13 mm	Parameter		Marking	Value	Law distr.
	The capacity in the pipeline walls	EV	$\overline{\sigma}_y$, MPa	231	
		Standard	$\hat{\sigma}_y$, MPa	67	
	Safety characteristic		β	4,32	
	The probability of refusal		$Q(\beta)$	$9{,}0\cdot10^{-6}$	

Accordingly, the probability of failure is $Q(\beta)=1\cdot10^{-5}$.

The subsidence values by engineering calculation and modelling are close if values are compared at a distance of three standards from the mathematical expectation of modeling: $S_{dif}=311+3\cdot24{,}2-59=324\,мм,\ (S_{sl}=342mm)$ and $S_{dif}=115+3\cdot34{,}0-41=176\,мм,\ (S_{sl}=141mm)$.

From the analysis of Fig. 11 it can be concluded that the thickness of the wall is 10 mm, which is sufficient to ensure the reliability of the pipeline by the parameter of longitudinal stresses in the ordinary building EGE conditions, can ensure the reliable operation of the pipeline only for point soaking and subsidence size $S_{dif}=101\ mm$.

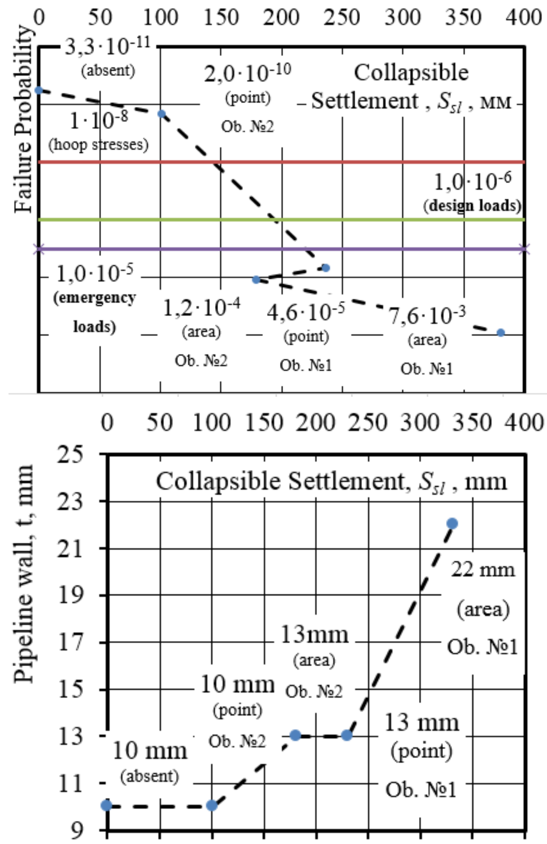

Fig. 11: LPTP dependences with increasing in size of subsidence on experimental data (Object 1 – Kherson-city, Object 2 - Poltava region): a - the probability of failure for different sizes of subsidence S_{sl}, wall- 10 mm; b - wall increasing to ensure the probability of failure at the level $1\cdot10^{-5}$

When in the watering area, the subsidence value increases ($S_{dif}=176\ mm$), therefore, it is necessary to increase the thickness of the wall to t = 13 mm.

For pipelines, that are laid in soil conditions without special properties, wall thickness, calculated according to the normative method [1], provides reliability for the parameter of longitudinal and annular stresses. The subsidence of soil up to 100 mm insignificantly affects the reliability of the pipeline and does not require an increase in the thickness of its wall.

The subsidence leads to the almost linear increase in the thickness of the wall of the pipeline; thus, the largest thickness t = 22 mm is required to provide a LPTP standard refusal rate for the size of the subsidence of $S_{dif}=324mm$.

3. Conclusions

For the first time, the probabilistic technique of random functions was used to solve the problem "LPTP - stochastic-heterogeneous base" as beams on an elastic basis. Its application is grounded for homogeneous bases without any special properties. It is determined that the stress from uneven deformation of the base does not exceed 10 – 15 MPa and uneven deformations of the system are within limits $2\cdot10^{-2}$ m.

The failure probability value of the pipeline by the parameter of longitudinal stresses on the joint action of the internal working pressure of the temperature difference and the deformation of the base without special properties are within $1{,}0\cdot10^{-10} - 1{,}2\cdot10^{-12}$.

The latter proves that laying a pipeline under normal soil conditions, the deformation of the base can be ignored as an external influence.

It has been proved for the first time that checking the strength of the pipeline wall for the action of the total longitudinal stresses, it is necessary to use the calculation scheme of the "LPTP – the collapsing base" system with the point source of soaking and the scheme where the source of the soaking is within the 1.5 - 2 height of the collapsing layer (or 15 - 20 diameters of the pipeline), with a further increase in the width of the free zone, the maximum curvature of the pipeline axis decreases. For the geometry considered in the modeling, the maximum longitudinal stresses have arisen for the case of the length of the soaked zone on the top and bottom, respectively, 20 and 30 m, and are $\sigma_{dif}{}^{max(min)}$ = +310 (-371) MPa.

The thickness of the wall, calculated according to the standard method on the action of internal pressure, is sufficient to ensure the strength and reliability of the pipeline by the parameter of longitudinal stresses with the size of the deposition to 100 mm. For the considered conditions, the probability of refusal was $2{,}0\cdot10^{-10}$. The subsidence leads to a significant increase in the probability of failure of the pipeline by the parameter of longitudinal stresses; so for the subsidence of 324 mm the probability of failure was $7{,}6\cdot10^{-3}$.

The linear dependence between increasing the size of the subsidence and the thickness of the wall of the pipeline is derived. The latter proves that laying a pipeline in collapsing massif it is necessary to recalculate the thickness of the wall, depending on the size of the subsidence of the base of the pipeline.

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