Reliability level of the buried main pipelines linear part

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ABSTRACT: This paper discuss loads and influences, which cause hoop and longitudinal stresses in the walls of the main pipeline linear part. Engineering-geological conditions, where is reasonable account differential settlements and respective longitudinal stresses, are substantiate. Natural and water saturated condition of the loessial loam bilinear models are improved. Stress strain state of the pipeline in the condition of soil local soaking is obtained. Probability density function and other statistics of the input random variables, such as operating pressure, temperature difference, deformation modulus of water saturated soil are developed Probabilistic parameters of the output random variables are obtained with help of Monte Carlo Simulation method. Such output random variables are differential settlements, hoop and longitudinal stresses. Pipeline reliability level is determined in the random variables technique, with help of linearization method.

1 PIPELINE STRENGTH CALCULATION

Hoop σ_h, longitudinal σ_l and radial σ_{rad} stresses make impact in the main pipeline linear part (MPLP). Radial stresses have relatively small values in the thin-walled high-pressure pipelines, so it used do not take into account (Yong Bai 2001, ASME B31.8-2003, Eurocode 3 2007, SNiP 2.05.06-85. 1988).

Hoop stress are calculated as follow

$$\sigma_h = \frac{nPD_{in}}{2t} \tag{1}$$

where P = internal operating pressure in the pipeline; n = the design (usage) factor for operating pressure (ASME B31.8-2003, Eurocode 3 2007, SNiP 2.05.06-85. 1988); D_{in} = pipeline internal diameter; t = pipeline wall thickness.

Calculation of the pipeline wall thickness is almost the same for different codes. The hoop stress σ_h criterion limits the characteristic tensile hoop stress, according to the pipeline steel Specified Minimum Yield Strength (*SMYS*) with accounting of the design (usage) factors, which values are specific for each code.

$$\sigma_h \le \gamma_i SMYS \tag{2}$$

where γ_i = the design (usage) factor specific for each code (ASME B31.8-2003, Eurocode 3 2007, SNiP 2.05.06-85 1988).

Longitudinal stress σ_l value in the MPLP is determined by three main factors: operating pressure P, influence of the temperature deformations and stresses, which caused by MPLP curvature

$$\sigma_l = \mu\sigma_h \pm \alpha E_p \Delta t \pm \sigma_{bend} \tag{3}$$

ɰhere μ= Poisson's ratio of the pipe steel; α= linear expansion factor of metal pipes; E_p = pipe steel Young's modulus; Δt= calculating temperature difference, which is extremal difference between MPLP wall temperature during the exploitation and in the moment when pipeline design scheme fixing; σ_{bend} = bending stress in the MPLP.

Bending stress in the MPLP σ_{bend} is composed of stresses caused by elastic bend of the pipeline sections (MPLP follows to the terrain relief) and by stresses caused by differential settlements of the MPLP soil base σ_{dif}

$$\sigma_{bend} = \pm \frac{E_p D_{ex}}{2\rho} \pm \sigma_{dif} \tag{4}$$

ɰhere D_{ex} = pipeline external diameter; ρ = pipeline axis curvature radius, which maximal values for each diameter are substantiate in the codes (SNiP 2.05.06-85 1988).

It should be noted, that hoop, temperature and stresses caused by elastic bending are sufficiently analyzed in the Ukrainian and international codes. It has analytical equations and simplified expressions for determination of their values, but stresses caused by differential settlements of the MPLP soil base σ_{dif}

haven't such equations. For example,Ukrainian codes proposed to determine efforts from design loads and influences with help of structural mechanics methods for statically indeterminate systems (SNiP 2.05.06-85 1988). Also there is recommended to applysoftware, but there are not specific equations for determining σ_{dif} stresses.

Limit values of the MPLP soil basis different settlements are also not regulated. Instead of Ukrainian and USA, Europe codes (Gresnigt 1986, Eurocode 3 2007) recommend limit value of the soil basis different settlement that is 10 cm on the wavelength 40 m Figure 1.

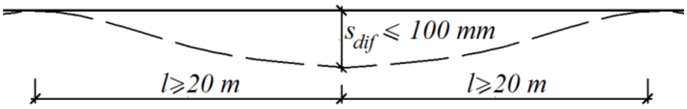

Figure 1. Limit value of the soil basis different settlement according to EUROCODE3-4-3.

2 DIFFERENTIALSETTLEMENTSOFTHEPIPELINESOILBASIS

2.1 *Causes of thepipeline differentialsettlements*

MPLP soil basis differential settlements lead to additional longitudinal stresses in the pipeline walls, the destruction of anti-corrosion coating, which significantly reduces pipeline durability (Bolotin1969, Palmer 1972, Gresnigt 1986, Faeli 2010). In addition, MPLP large deflection may cause violation if the operating condition, which again confirms necessityof the different settlements regulation Figure 1.

It is necessary to understand in such soil conditions impact of the differential settlements significant, and it causes additional longitudinal stresses in the pipes wall. For example, in normal homogeneous soils differential settlements values are very small, and its impact on the MPLP strength and reliability is insignificant (Pichugin 2009). For pipeline, which basis consist of homogeneous fine sand additional longitudinal stress from different settlements is less than 10 MPa. This is because in normal conditions soil doesn't settling from its own weight, and pipeline weight in most case is less than the weight of excavated soil.

Large values of the MPLP differential settlements is typical for pipeline laying in non-standard soil conditions. Non-standard soil conditions it is when pipeline layor design in areas with the following characteristic features (Zaripov 2000, Palmer 1972, SNiP 2.05.06-85 1988):swamp or flooded areas, areas with underground cavities of various nature (mining and mine construction zones, areas with karst cavities, etc.), thawing permafrost areas, landslide territories, seismic zones.

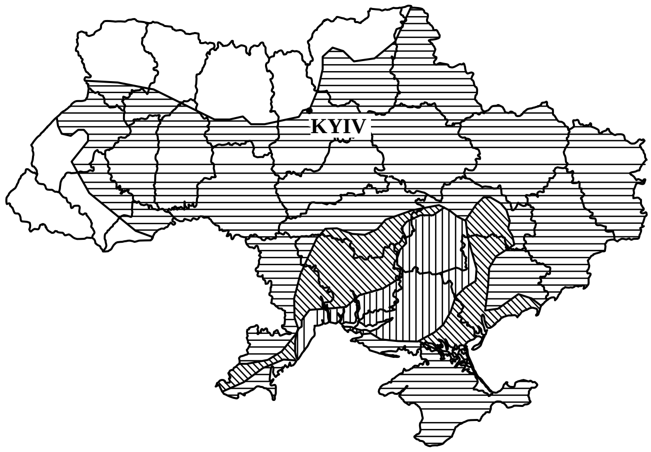

Figure 2. Broadening of the loessial soils through the Ukraine territory

For the Ukraine loessial collapsible soils is one of the most common problem, because such soil occupy 65-70% of the territory. Such problem is especially urgent for the southern region, where loessial layer reaches 45…50 m, and the value of the soil collapse from its own weight may occur 1…2 m (Shokarev 2007).

2.2 *Loessial soil collapse effect under the pipeline*

Soil – pipeline interaction in the most cases is modeled by beam on elastic Winkler foundation (Bolotin 1969, Palmer 1972, Pichugin 2009). In such formulation soil execute function of the space spring supports, which set limits on the MPLP movements. Movement resistance mostly depends on soil deformation modulus E_s. Nevertheless, such formulation is suitable only for standard soil conditions, where pipeline settlements directly proportional to the soil base resistance.

Collapsible soil in natural state is quite well foundation for buildings and structures, but strength and deformation soil characteristics have property to reduce significantly as result of the soaking.Water saturated soil has a collapse property – soil volume reducing under additional loads or even its own weight. Soil in the natural condition has follow compressive curve Figure 3 a. Compressive curve has 3 sections: close to linear section AB – it has small reducing of the porosity ratio (consolidation isn't occur); curve section BD – it corresponds to soil consolidation under pressure that exceeds soi structural strength; section DF – unloading (Zhuk2006).

Saturated loessial soil has typical compressive curve with 4 sections Figure 3 b: section AB – settling in the natural state; section BC – soil collapse as result of soaking; section CD – soil settling with broken structural bonds (further consolidation); section DF – unloading.Should be noted that soil collapse occurs only by soil characteristicsreducing, without additional load applying.

We propose to limit soil deformation diagram in the second section (point C), if take into account MPLP laying features and soil deformation, which cause longitudinal stresses in the pipeline wall. It is reasonable to consider case when soaking occurs after pipeline laying and additional load is absence. In addition, it is necessary to consider the most adverse soil soaking case, when soaking happen in the some points of the pipeline trace, and it is absent in the other – local soaking.

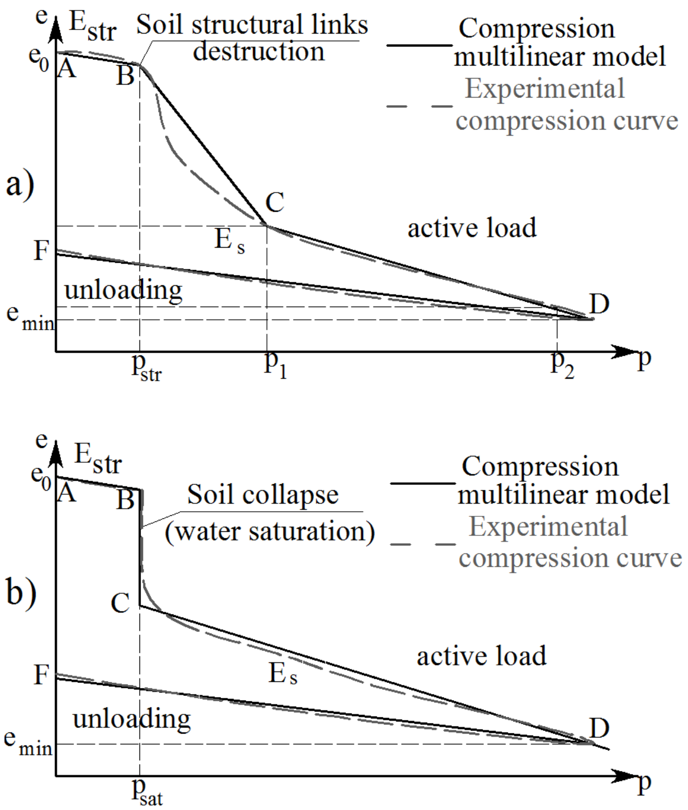

Figure3. Compressioncurveoftheloessialsoilinnatural (a) andwatersaturated (b) condition

Complicated soil models using lead to significant complication of calculations, also there is almost impossible to use analytical methods. Taken into account previously experience of pipeline strength calculation, Finite Elements Method (FEM) is reasonable to use in such problems (Seleznev 2002, Faeli 2010). Problem of the soil base collapse needs accounting of the physical and geometrical nonlinearity, so it is reasonable to use Ansys software.

2.3 *Pipeline stresses caused by soil collapse*

Soil collapse occur as result of local soaking, spot (linear) or square may berespective water sources. Main pipelines usually lay far from cities and countries, so respective water sources is follow: irrigation areas, river overflow places; MPLP crossing with under and over ground communications (sewage collectors, water and irrigation pipelines).

One of the biggest problem during FEM treatment is substantiation of the compressible strata under the pipe. In standard soil conditions, where soil hasn’t specific properties, engineering-geological surveys are conduct on the 1 meter deeper than the expect pipeline bottom. But for loessial collapsible soils is necessary to determine the characteristics of all layers that capable forcollapse. Compressible strata is reasonable to limit by layer, where collapsible property is absent Figure3(soil 5).

Soil profile typical for Poltava region is on the Figure 4. Soil profile consists of soil1– humifiedloam; soil 2 – loessialloam; soil 3 – loessialloam; soil 4 – loam; soil 5 – loam, respective characteristics is in the Table 1.

Should be noted, that collapsible soil characteristics may be different in better or worse, compressible strata may be deeper or it can consist noncollapsible layers. So obtained results from our soil profile is just some estimations for specific geological conditions, but proposed models are advisable to use.

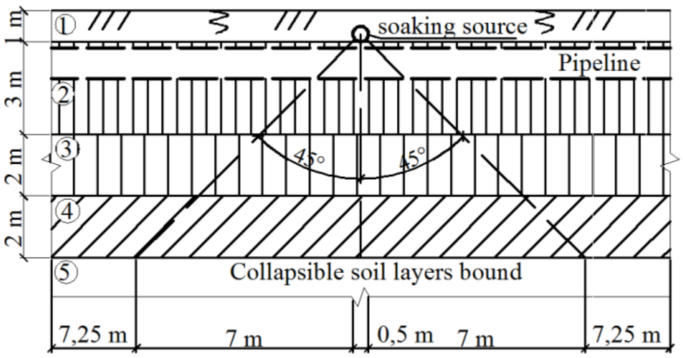

Figure 4. Soil profile typical for Poltava region

Table 1. Averagedcharacteristics of loessial collapsible soil of Poltava region

Soil characteristics		Soil 1	Soil 2	Soil 3	Soil 4	Soil 5
Layer thickness, h, m		1,0	3,0	2,0	2,0	-
Soil density, ρ, kg/m^3		1500	1500	1670	1740	1810
Dry density, ρ_d, kg/m^3		-	1310	1420	1470	1570
Saturated soil density, ρ_{sat}, kg/m^3		-	1770	1840	1870	-
Void ratio, *e*		-	1,05	0,89	0,84	0,69
Relative collapsibility, ε_{sl}, %, for pressure, P, MPa	0,05	-	1,4	0,5	0,3	-
	0,10	-	2,5	0,9	0,6	-
	0,20	-	4,4	1,5	1,0	-
	0,30	-	5,5	1,9	1,3	-
Deformation modulus in dry condition, E_s, MPa		-	8	12	14	17
Poisson ratiodry soil, μ		-	0,37	0,36	0,35	-
Poisson ratiowet soil, μ		-	0,45	0,41	0,4	-

On Figure 4 is also shows MPLP crossing with spot water source (irrigation pipelines for example). Soaking area spread from top to bottom in a cone.

The angle of the line, which limits the soaking zone in loess loam, is $\beta=45^0–55^0$ (Zhuk 2006, Shokarev 2007). Therefore, MPLP section that is necessary to consider in the calculations can be determinate, in our calculation we accept 30 m section length. We accept 3 m width of the soil strata.

Soil has been modeled in natural and water saturated state. Pressure under the MPLP is almost equal to the pressure in the soil massif from soil own weight, therefore, in the natural condition soil settlements must follow to 0.If we set dependency diagram “pressure$P$ – relative strain ε” in the linear form, with accounting only soil deformation modulus E_s, settlements from soil own weight will have significant values, it is unacceptable. Therefore, necessary to introduce parameter that eliminates soil settlements from its own weight. We propose to use soil structural strength P_{str} as such parameter for soil in natural (dry) condition. For soil 2 in dry conditionstructural strength value is P_{str} = 32 kPa. When pressure reaches P_{str} value, we propose linear dependence between pressure and settlements, where soil deformation modulus is the main parameter Figure 5.

In Ansys is absent soil direct models, but there are a lot useful models that modeling some requiredproperties, such as elasticity, bilinear or multilinear plasticity, so bilinear hardening model is sufficiently precise, and quite simple in the same time.

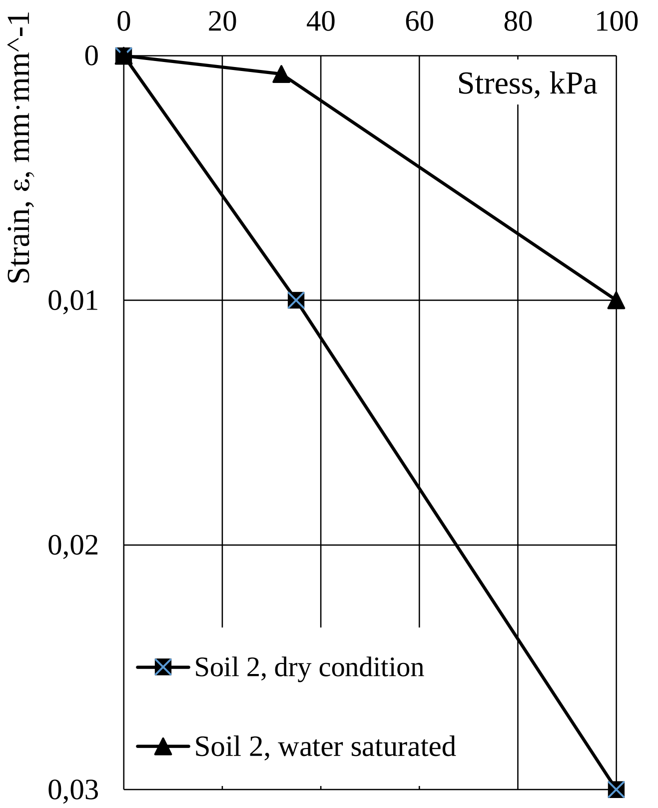

Figure 5. Bilinear dependence diagram “pressure P – relative strain ε”for soil 2 in dry condition(1) and water saturated condition (2)that used in the Ansyscalculations

Multilinear dependence diagram “pressure P – relative collapsibility strain ε_{sl}” is the most accuraterepresentcollapsible properties of the loessial soil. Such diagram is obtained during soil testing in uniaxial compression device. Multilinear diagram is unuseful for further probabilistic analysis. Therefore, it is advisable to use bilinear depending diagram “pressure P – relative collapsibility strain ε_{sl}”. In this case point of diagram fracture is very important, we propose to use average value of the pressure in the each soil layer as such point. That means soil soaking is occur when additional load is absent. For soil 2 in saturated condition diagram fracturepoint is P = 35kPa. On Figure 5 is shown that saturated soil has much larger deformations then soil in dry condition. Soil 3 and soil 4 have typical diagram from Figure 5 but with accounting data from Table 1.

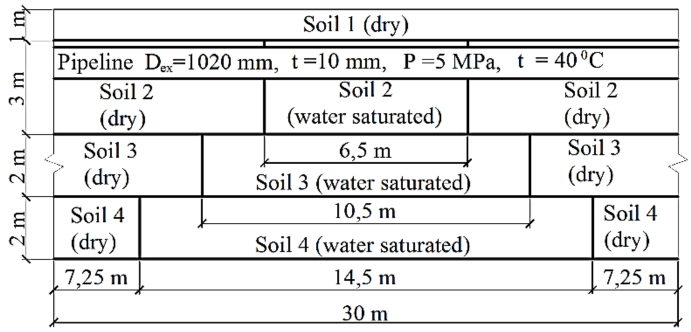

Figure6. Soil profile division on structural elements for Ansys calculation

In calculations accepted hypothesis that pipeline deformation is equal to the soil basis deformations (Bolotin 1969, Palmer 1972, Zaripov, R. 2000,Seleznev 2002, Pichugin 2009). Geometrical parameters of the MPLP is follow D_{ex} = 1020 mm, wall thickness t = 9,6mm. Internal operating pressure P = 4,9 MPa (Pichugin 2014). Substantiation of the temperature difference value of needs particular attention. We propose to use average monthly maximum temperature of the soil surface $t^{av.max}_{soil}$ = 42^0C(Kinash 2001) and average pipeline temperature $t_p \approx$ 8-12^0C, with accounting of the transported product temperature and soil at respective deep (Kinash 2001). Therefore, temperature difference value is Δt = 32^0C. Pipes are manufactured of steel grade K55, $SMYS$ = 410 MPa.

Ansys calculations were made with account of the physical and geometrical nonlinearity, deterministic results are follow Figure 6: soil collapse under the pipeline in the soaking zone is 27,8 mm, pipeline settlements in zones, where soil in natural condition, is about 1,8 mm. Settlements difference is 26 mm Figure 6 a. Hoop stresses is 268 MPa, which corresponds exactly to Equation 1, longitudinal stresses is 230,7 MPa.It is easily to obtain value of the longitudinal stresses, which caused by MPLP differential settlements. From Equation 3 such stresses is σ_{dif} = 69 MPa.

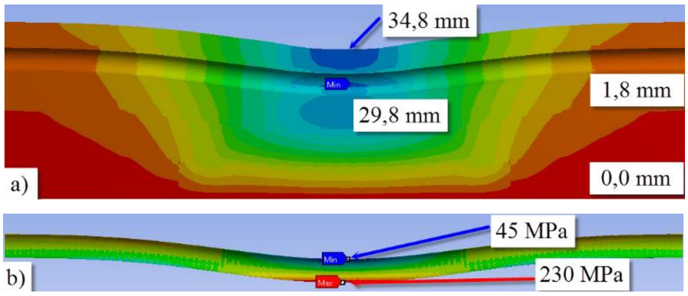

Figure 6. Pipeline local soaking in collapsible soil modelling: soil strata settlements (a) pipeline longitudinal stresses (b)

Such value of the σ_{dif} is high for relative settlements level, but it is explained by small soaking zone and very high curvature as result. Value of the σ_{dif} in the same soil profile for longer soaking zone (≈ 30 – 50 m) is about 40 MPa. Despite this, pipeline differential settlements in the collapsible soil cause significant longitudinal stresses. Short area of soaking is especially dangerous, because it leads to the formation of pipeline axis large curvature.

3 PIPELINE RELIABILITY ESTIMATION BY THE HOOP AND LONGITUDINAL STRESSES PARAMETER

The calculation of pipeline strengthis usually performed by assumption that its properties and the properties of the soil foundations are deterministic (Gresnigt 1986, Yong Bai 2001, Seleznev 2002). But this statement is not always true. Firstly, the properties of soil base depend on many factors. Those factors are not exposed directly accounting, and therefore have a random character. Secondly, external loads, material properties and geometrical dimensions of the pipe depend on a number of different, poorly controlled and difficult interacting causes that are also changing randomly (Pichugin 2009, 2014).

In view of above and the current development of the construction industry, it is actual calculation of MPLP by the probabilistic methods. Stresses and deformations of MPLP laid in a static heterogeneous soil are random variables (RV).That fact must be accounting for pipeline strength calculations. Also, reliability levels by the hoop and longitudinal stresses must be obtained.

Firstly, probabilistic approach to calculate MPLP strength was used by Bolotin (Bolotin 1969). He is focused on the solution of the differential equation of the pipeline curved axis with random parameters. We propose an engineering method for calculating MPLP reliability. It is based on the idea of rational compromise between accuracy and simplicity of probabilistic calculations

Random variables aren't only possible mathematical tools for calculating pipelines strength and reliability. Standard "deterministic" apparatus in which random factors are not taken into account also may use. But it can't be forgotten that it gives approximate schematic description of the system, its some "averaged" values. In-depth study of the system such deviations are necessary to consider using the probabilistic approach.

Main difference between such approaches is given on Figure 7. There is large number of mathematical tools to estimate construction reliability: linearization; Lichov overrun method; random arguments replacement method (numerical integration); Monte Carlo Simulation; Response Surface Method, Point Estimate Method (FO – PEM & A – PEM) and First Order Reliability Method (A – FORM) (Bolotin 1969, Fenton 1997, Avsar 2004, Phoon 2008, Pichugin 2009, 2014, Zotsenko 2011).

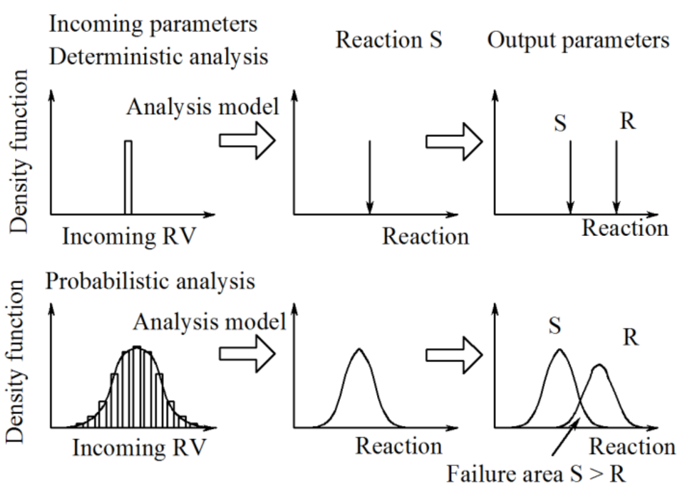

Figure 7. Difference between deterministic and probabilistic calculation method of the building constructions

Each method is convenient tool for solving a specific problem. Monte Carlo Simulation (MCS), Response Surface Method (RSM) and Point Estimate Method (PEM) are the most universal of them. Such methods use during the investigation of complex non-linear systems, including problem solution with help of FEM.

Monte Carlo Simulation consist in, that set of variables is organized with help of random number generator according to the specific density functions. Values of the output function are calculated for obtained set of variables. These values add to memory and after sorting in interval, these values forming experimental histogram. After realization of sufficiently large number of RV output function distribution density is possible to obtained.

To obtain MPLP reliability level by the parameter of hoop and longitudinal stresses is necessary to obtain output function at least from six input arguments, which are RV: operating pressure fluctuation, variations of the average monthly maximum temperature of the soil surface and average pipeline temperature, probabilistic parameters of the collapsible soil deformation characteristics. Obtained output function is extremely complicated, especially if we take into consideration correlation between input RV. But if we know input RV probabilistic parameters, output RV density function and other statistics is possible to obtain, with help of *Monte Carlo Simulation*, which is implemented in Ansys. Hoop and longitudinal stresses output RV is easily to compared with steel yield strength RV.

Internal pressure fluctuation caused by technological factors during normal pipeline operation. Those factors connect with work of the compressor equipment and hydraulic features of the whole pipeline system. Those factors has stochastic nature and cause random overloads of the pipeline construction.

Internal pressure fluctuation is obtained from main pipeline section observations for 3 month. General sampling is 3436 pressure values, which are obtained for 3 month (Pichugin 2014). Pressure has changed significantly in both upward and downward from nominal pressure value even during stationary pipeline operation.

There had been obtained 11 intervals of the pipeline stationary operation mode during the observation. To improve the reliability of the results more than 20 measuring had to get into interval, and observation period had to be over 12 hours. Observation results is follow: pressure fluctuation has normal density function Table 2. Variation ratio average value from experiment is 6,9 % (Pichugin 2014), which we will use further.

Mostly of the temperature difference RV probabilistic parameters is possible to obtain as result of processing of the meteorological stations long-term climate observations (Kinash 2001) or direct measurements methods. Temperature statistics are in the Table 2.

Table 2. Probabilistic characteristics of input random variables – external loads and influences

Input probabilistic parameters / random variables	Density function	Mathematical expectation	Standard deviation	Variation ratio
Operating pressure, P	Normal	4,9 MPa	0,35 MPa	6,9 %
Soil surface temperature, $t^{av.max}_{soil}$	Normal	42 ^0C	3,1 ^0C	10,0 %
Pipeline temperature operating condition, t_p	Normal	10 ^0C	1,0 ^0C	10,0 %
Saturated soil 2 initial deformation modulus, E_s	Normal	3,5 MPa	0,875 MPa	25,0 %
Saturated soil 3 initial deformation modulus, E_s	Normal	4,5 MPa	1,125 MPa	25,0 %
Saturated soil 4 initial deformation modulus, E_s	Normal	8,0 MPa	2,0 MPa	25,0%

Existing experience of the geotechnical problems probabilistic study shows that RV of the soil mechanical characteristics even in the natural condition has high variation ratio 20 –30 %, also it usually has lognormal density function (Fenton 1997, Phoon 2008, Zotsenko 2011). Soil in saturated condition has even higher variation ratio, because it soaking unevenly. In our research, we apply simplified model of the soil mechanical characteristics RV, Table 2. It is made to simplify calculations and to obtain illustrative representation of results. In future, we will develop soil mechanical characteristics probabilistic model.

Output RV density functions is shown in Figure 7, Table 3. It obtained with help of *Monte Carlo Simulation.*

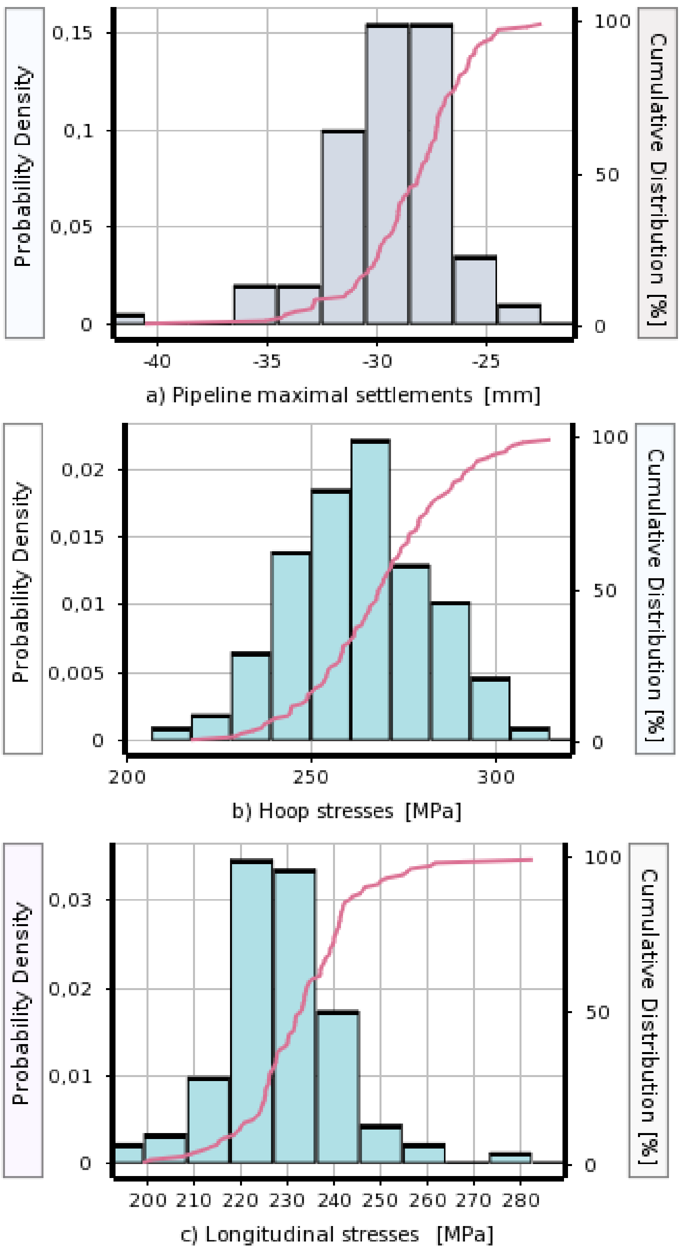

Figure 8. Probability density and cumulative distributionofpipeline reliability output random variables: pipeline maximal settlements (a), hoop (b) and longitudinal stresses

Table 3. Probabilisticcharacteristicsofpipeline reliability output random variables

Output probabilistic parameters / random variables	Density function	Mathematical expectation	Standard deviation	Variation ratio
Steel yield strength, R	Normal	490	49	10,0 %
Pipeline maximal settlements, S, mm	Normal	-28,5 mm	2,7 mm	9,5 %
Pipeline hoop stresses, σ_h, MPa	Normal	232,9 MPa	12,6 MPa	5,4 %
Pipeline longitudinal stresses, σ_l, MPa	Normal	268,5 MPa	18,8 MPa	7,0 %

Obtained density functions for output hoop and longitudinal stresses RV allow estimating pipeline reliability easily in the random variables technique. At this stage, linearization method is reasonable, because final equations is quite simple. All of the input functions has normal density function, respectively output functions has normal density too. Therefore, pipeline reliability is estimating as follow

$$\tilde{Y} = \tilde{R} - \tilde{S} = \tilde{\sigma}_y - \tilde{\sigma}_h(\tilde{\sigma}_l) \geq 0 \quad (5)$$

where $\tilde{R}$ = pipeline steel strength distribution function; $\tilde{S}$ = hoop (longitudinal) stresses distribution function.

Mathematical expextation and standard deviation respectively:

$$\bar{Y} = \bar{R} - \bar{S} = \bar{\sigma}_y - \bar{\sigma}_h(\bar{\sigma}_l) \quad (6)$$

$$\hat{Y} = \hat{R} - \hat{S} = \hat{\sigma}_y - \hat{\sigma}_h(\hat{\sigma}_l) \quad (7)$$

Safety characteristic:

$$\beta = \frac{\bar{Y}}{\hat{Y}} = \frac{\bar{\sigma}_y - \bar{\sigma}_h(\bar{\sigma}_l}{\sqrt{\hat{\sigma}_y - \hat{\sigma}_h(\hat{\sigma}_l)}} \quad (8)$$

Reliability level:

$$P(\beta) = 0{,}5 + \Phi(\beta) \quad (9)$$

where $\Phi(\beta)$ = Laplace function safety characteristics respective value β.

Table4. Pipeline reliability level by the hoop and longitudinal stresses

Pipeline reliability parameters	Longitudinal stresses, σ_l	Hoop stresses, σ_h
Mathematical expectation of reliability function, $\bar{Y}$	257,1 MPa	221,5 MPa
Standard deviation of reliability function, $\hat{Y}$	50,6 MPa	52,5 MPa
Safety characteristic, β	5,08	4,2
Failure probability, $Q(\beta)$	$1{,}7 \cdot 10^{-7}$	$1{,}33 \cdot 10^{-5}$
Reliability level, $P(\beta)$	0,9999993	0,9999966

Should be noted, that the most important building objects in Ukraine must have failure probability less then $Q = 1 \cdot 10^{-6}$ according to the national codes (DBN V.1.2-14-2009). But if the construction failure leads only to economic losses, there are possible to reduce reliability level based on the conditions to minimize the total cost of manufacture, installation, operation and the elimination of losses from possible failure. Therefore, MPLP has high enough reliability level according to obtained results from Table 4.

As we mentioned, considered soil profile, which is typical for Poltava region, is some average pattern of the possible soil conditions. Compressible strata may be much deeper (Shokarev 2007), and soil characteristics may change in the wide range. So, soil collapse value may be much higher.

Conclusions

Pipeline differential settlements and respective longitudinal stresses is reasonable to account only in the non-standard soil condition, especially in the loessial collapsible soils.

Full collapsible strata must be taken into consideration, when differential settlements are calculating. Soil bilinear models is the most appropriate for deterministic and probabilistic differential settlements calculations.

Pipeline differential settlements in the soil profile, which is typical for Poltava region, is 26 mm, and it cause 69 MPa longitudinal stresses. It indicates about significant impact of the differential settlements on the pipeline strength. Short area of soaking is especially dangerous.

Pipeline failure probability by the longitudinal and hoop stresses parameters is $Q(\beta) = 1{,}7 \cdot 10^{-7}$ and $Q(\beta) = 1{,}33 \cdot 10^{-5}$ respectively, which indicates that pipeline high enough reliability level in such soil conditions. Nevertheless, it can be significantly reduce when soil condition is worse.

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