Geomechanical modelling of construction conditions for trunk pipeline underwater crossings

by D.R. Vafin*, D.A. Shatalov¹, and Z.Z. Sharafutdinov²

1 Linear Pipeline and Underwater Line Laboratory, Pipeline Transport Institute, Moscow, Russian Federation
2 Center for Construction Methods, Inspection of Buildings and Facilities, Pipeline Transport Institute, Moscow, Russian Federation

HIS PAPER EXAMINES issues of predicting technical and technological complications in the process of constructing trunk pipeline for underwater crossings using the directional drilling (also called directional boring) method, depending on the properties of the soils forming the underwater crossing. The prediction is made on the basis of geomechanical modelling of the underwater crossing, taking into account of the soil stresses arising during construction of the crossing. Basic criteria are formulated for predicting the stability of the soils along the bore during the construction of underwater crossings.

Key words: directional drilling, river crossing, geomechanical modelling, soil strength, soil stability, drilling mud

ONE OF THE TECHNICAL challenges which arises when designing an underwater crossing and selecting the construction method is the assessment of possible complications in the construction process, and the economic evaluation of attainable technical results. In the absence of a method to evaluate the potential for complications, difficulties can arise in developing appropriate solutions. As a consequence of these difficulties, mistakes can be made in the design, or existing and successfully applied modern technologies for underwater-crossing construction can be ignored.

The main complications in the process of constructing underwater crossings for trunk pipelines using the directional-drilling method in unstable soils are:

• the loss of the bore, when the soil forming the walls of the bore collapses;
• the deviation of the design profile from actually achieved by in the drilling process;
• loss of circulation of drilling fluid (mud);
• ‘balling’ of the drilling bit;
• increased wear and damage to the drilling tool;
• conditions preventing the pull-out of the drilling tool from the bore, or the pipeline while it is being pulled through.

These complications and the accidents arising from them are dictated by the geological conditions for the work, and eliminating their consequences would involve significant additional labour and financial expenditure, to the extent that the results achieved by this type of construction would be lost.

*Corresponding author’s contact details:
Email: vafindr@niitnn.transneft.ru
A geomechanical model is being developed for forecasting and modelling the construction process for directional and horizontal oil and gas wells. The model includes research into the properties of the rocks near the wellbore profile, and uses geophysical and other engineering and technical methods [1,2]. It describes the loads acting on the rocks in the near-wellbore zone and the stresses arising as a result, and also determines the condition of the wellbore walls and predicts possible complications in the process of well drilling and operation.

During construction of an underwater crossing using the directional-drilling method, geomechanical models are not currently used for the process of the crossing construction, pipeline pulling, or crossing operation. To a significant extent, this limits the effectiveness of the engineering and technical methods and evaluations used in the directional-drilling method.

In connection with this, the challenge is to formulate the basic criteria for building a geomechanical model that can be applied to the construction of underwater crossings using the directional-drilling method. This would allow the technology for constructing the crossing to be chosen, and the pipeline’s operating conditions to be assessed. Building a geomechanical model would help in drawing-up strict requirements for carrying out engineering and geological surveying and for their results, which are used to make design solutions in the construction of underwater crossings.

In their natural state, soils are in equilibrium, and the strength of the soils counterbalances the effect of gravity and tectonic forces. The stress state in the soils will appear as their strains which, in the plastic conditions, are determined by the degree to which overburden pressure exceeds strength, or by the soils’ yield strength.

Rock pressure can be subdivided into geostatic \( P_g \) and lateral pressure \( P_{lat} \) (Fig.1a). Geostatic pressure is the pressure caused by the weight of the overlying rock, and is given by Equn 1:

\[
P_g = \rho g H
\]

where \( g \) is gravitational acceleration, equalling 9.81 cm/sec\(^2\); \( \rho \) is the average density of the overlying rock in kg/m\(^3\); and \( H \) is the depositional depth of the rock in metres.

Reference 2 shows the load transfer applied vertically to the rock, in a horizontal direction:

\[
P_{lat} = P_g \lambda
\]

where \( \lambda \) is the lateral-earth-pressure factor.

During the drilling process, part of the drilled-out rock is detached from the bed and is carried out by the drilling mud to the surface. Consequently, a free surface arises, on which strains can appear and, as a result, stresses are redistributed along the bore. The acting pressure of the drilling mud in the bore begins to counteract the lateral pressure of the rock.
The parameter which determines the possibility and extent of rock strain is the lateral-earth-pressure factor. As solids with atomic structures, monolithic rocks have high strengths which ensures a lateral-earth-pressure factor value of \( \lambda = 0.1-0.2 \) and a low capability for transferring rock pressure onto the walls of the borehole. These values of \( \lambda \) provide a stable condition in the bore. As rock ductility increases, the lateral-earth-pressure factor also increases, and the soils become unstable because rock pressure is more vigorously transferred onto the walls of the borehole.

The value of the lateral-earth-pressure factor depends on the rock properties and, when the Poisson's ratio values for the rock \( \mu = 0.13 - 0.25 \), is calculated according to the formula proposed by Dinnik (Equn 3); when the value of the internal friction angle \( \phi = 38 \text{ - } 58^\circ \), the lateral-earth-pressure factor is determined according to Krupennik's formula (Equn 4):

\[
\lambda = \frac{\mu}{1 - \mu}
\]

\[
\lambda = \frac{1 - \sin \phi}{1 + \sin \phi}
\]

Thus, for example, the lateral-earth-pressure factor for high-ductile clays is around 0.7, while for sands it is 0.3-0.4.

The pressure of the drilling fluid in the borehole \( P_h \) (Equn 5) is governed by the static \( P_{stat} \) (Equn 6) and the dynamic \( P_{dyn} \) pressures. The static pressure depends on the density of the initial drilling fluid and the equivalent circulation density of the fluid, and thus depends on the amount and density of drilled-out rocks. The dynamic pressure depends on the change in drilling-fluid pressure in the borehole during pump work or tripping operations.

\[
P_h = P_{stat} + P_{dyn}
\]

\[
P_{stat} = \rho g H
\]

The density of drilling mud with drilling-cuttings’ content (Equn 7) increases compared to the density of the initial fluid pumped in:

\[
\rho = (1 - c_s)\rho_{dm} + \rho_{np} c_s
\]

where:
- \( \rho_{np} \) is the average density of rock particles in kg/m³;
- \( \rho_{dm} \) is the density of drilling mud in kg/m³;
- \( c_s \) is the proportion of cuttings in the drilling mud, as a unit-less fraction.

An increase in drilling-mud pressure such that it exceeds lateral rock pressure can lead to an increase in tensile stresses on the wellbore walls. As a result of their impact, fractures form in the surrounding soil, which may lead to hydro-fracturing of the rock, i.e. mud spring initiation, or to the simple loss of circulation of the drilling mud. In small-diameter wells, the mud pressure also exceeds the lateral rock pressure because of the impact of dynamic pressure and the presence of cuttings in the well. In large-diameter wells, i.e. when they are reamed, the dynamic pressure has almost no impact on hydro-fracturing in the rock.

The pressure of liquid in the fractures and pores of the soil is known as the formation pressure (Equn 8) and is determined by:

\[
P_f = K_{an} \rho_w g H
\]

where:
- \( K_{an} \) is the formation pressure anomaly factor;
- \( \rho_w \) is the water density, equal to 1000 kg/m³.

The values of \( P_g \) and \( P_{his} \) as components of rock pressure characterize the natural stresses in rock as if it were monolithic, while the values \( \sigma_1 \) and \( \sigma_3 \) are the natural stresses in the body of the porous rock model [2]. The orientation of normal stresses is presented in Fig.1b.

The horizontal stress (Equn 9) is equal to:

\[
\sigma_1 = \lambda P_g + P_{fn} (1 - c)(1 - \lambda)
\]
where:

\( P_g \) is the geostatic pressure in MPa;
\( P_{fm} \) is the formation pressure in MPa;
\( c \) is the proportion of unit area (Eqn 10) in the dangerous (under examination) cross-section:

\[
c = \exp(-19.16m^2)
\] (10)

The vertical stress (Eqn 11) is given by:

\[
\sigma_z = P_{fm} + \frac{P_g - P_{fm}}{c}
\] (11)

The stress state of a wellbore is calculated in a cylindrical coordinate system, where the axis is superimposed onto the axis of the wellbore and the normal stress is denoted as \( \sigma_z \); the radial stress \( \sigma_r \) is in the direction of the radius vector; the tangential stress \( \sigma_t \) is perpendicular to the radial stress. The stresses in directional and horizontal wells are determined by two points: in the lateral part of the wellbore by point A and in the top zone by point B.

In directional wells, the distribution of stress at points A (Eqn 12) and B (Eqn 13) is shown in Fig.2 and takes the following form shown in Eqs 12 and 13 (see right) where \( \alpha \) is the angle of the wellbore curvature in degrees.

The pressure of hydro-fracturing in a directional well at point A (Eqn 14) and at point B (Eqn 15) is determined by the formulae:

\[
P_{PB} = c(3\sigma_3 - \sigma_1) + P_{fm}(1 - c) + \sigma_t
\] (14)

\[
P_{PB} = 0.5c(3\sigma_3 + \sigma_1) + P_{fm}(1 - c) + \sigma_t
\] (15)

The stresses around a horizontal well are shown in Fig.3 and come to:

- at point A

\[
\begin{align*}
\sigma_z &= \sigma_1 + 2\mu(\sigma_3 - \sigma_1) \\
\sigma_r &= P_{fm} + \frac{P_h - P_{fm}}{c} \\
\sigma_t &= \sigma_3 + \sigma_1 + 2(\sigma_3 - \sigma_1) - \sigma_r
\end{align*}
\] (16)

- at point B where \( \phi = 90^\circ \)

\[
\begin{align*}
\sigma_z &= \sigma_1 - 2\mu(\sigma_3 - \sigma_1) \\
\sigma_r &= P_{fm} + \frac{P_h - P_{fm}}{c} \\
\sigma_t &= \sigma_3 + \sigma_1 - 2(\sigma_3 - \sigma_1) - \sigma_r
\end{align*}
\] (17)

The pressure of hydro-fracturing at points A (Eqn 18) and B (Eqn 19) is given by:

\[
P_{PB} = c(3\sigma_3 - \sigma_1) + P_{fm}(1 - c)
\] (18)

\[
P_{PB} = 0.5c(3\sigma_3 + \sigma_1) + P_{fm}(1 - c)
\] (19)

The difference between tangential stress in the vicinity of the well and natural stress is governed by disparities in natural stress and drilling mud pressure in the well. Admittedly, the ratio of their values is unknown. The calculation is therefore based on conditions of non-equation for every point - see Equn 20 (see right).

The use of this methodology allows the loads acting on the rock to be determined for the near-well zone of directional and horizontal wells, as well as the stresses and hydro-fracturing pressures which thus arise in the rock. Calculating the lateral-earth-pressure factor using Dinnik’s and

![Fig.2. Stress distribution in a directional borehole.](image)
Kruppenik’s formulae is limited by the values of the Poisson’s ratio and the soil’s internal friction angle and does not cover all possible values. Therefore, this problem requires its own additional solution.

Soil characteristics and elastic-state conditions

The primary geological factor determining the stability of the wellbore walls is the soil strength. The soil strength is dictated by the nature and force of the bonds between the minerals and particles from which it is formed. Depending on the type of chemical bond and on the parameters determining the soil strength, soils can be classified as:

- rocky;
- disperse consolidated;
- disperse non-consolidated.

Rocky soils are characterized by covalent and polar-covalent bonds, which provide high strength and properties of solids.

![Fig.3. Stress distribution in a horizontal borehole.](image1)

The internal friction forces, which arise in the rock from the external load, have secondary significance in rocky soils when compared to structural cohesion, and this means that the internal friction force can be ignored. These soils include granite, calcium, limestone, mica, etc. The rocky soil strength may be determined in uniaxial compression tests (Equn 21) or by punch indentation (Equn 22):

\[
\tau_u = \frac{\sigma_{ab}}{2}, \sigma_{op} = \frac{\sigma_{ab}}{2} \tag{21}
\]

\[
\tau_u = K_1 p_{aw}, \sigma_{op} = K_2 p_{aw} \tag{22}
\]

\[
\begin{align*}
\sigma_x &= \sigma_3 \left( \cos^2 \alpha - 2 \mu \sin^2 \alpha \right) + \sigma_1 \left( 1 + 2 \mu \right) \sin^2 \alpha \\
\sigma_z &= \left( 3 \sigma_3 - \sigma_1 \right) \sin^2 \alpha + 2 \sigma_1 \cos^2 \alpha - \sigma_r \\
\sigma_r &= P_{pm} + \frac{P_c - P_{pm}}{c} \tag{12}
\end{align*}
\]

at Point A:

\[
\begin{align*}
\sigma_x &= \sigma_3 \cos^2 \alpha + \left( \sigma_1 + 2 \mu \left( \sigma_3 - \sigma_1 \right) \right) \sin^2 \alpha \\
\sigma_z &= \left( 3 \sigma_3 - \sigma_1 \right) \sin^2 \alpha + 2 \sigma_1 \cos^2 \alpha - \sigma_r \\
\sigma_r &= P_{pm} + \frac{P_c - P_{pm}}{c} \tag{13}
\end{align*}
\]

\[
\begin{align*}
\text{if } |\sigma_x| > |\sigma_z| > |\sigma_r| \text{ then } \tau_{max} &= \frac{\left| \sigma_x - \sigma_z \right|}{2}, \sigma_{op} = \frac{\left| \sigma_x + \sigma_z \right|}{2} \\
\text{if } |\sigma_z| > |\sigma_x| > |\sigma_r| \text{ then } \tau_{max} &= \frac{\left| \sigma_z - \sigma_x \right|}{2}, \sigma_{op} = \frac{\left| \sigma_z + \sigma_x \right|}{2} \\
\text{if } |\sigma_r| > |\sigma_x| > |\sigma_z| \text{ then } \tau_{max} &= \frac{\left| \sigma_r - \sigma_x \right|}{2}, \sigma_{op} = \frac{\left| \sigma_r + \sigma_x \right|}{2} \tag{20}
\end{align*}
\]
where $K_1$ and $K_2$ are proportionality factors, which are dependent on the Poisson’s ratio:

$$K_1 = 0.346 - 0.109\mu$$
$$K_2 = 0.509 - 0.020\mu$$

Consolidated soils are represented by argillite, siltstone, clay, loam, and loamy sand in various consistencies in which the natural agglomerates are bound together by hydrogen bonds. The strength of the hydrogen bonds in their volume depends on the water-cut in argillaceous components. Argillaceous soils can be subdivided into solid and ductile, depending on the water-cut.

Solid clays are characterized by higher strength. In these clays, water is absent, giving the clay the ductile properties, since clay particles are bound by hydrate bonds. At low geostatic pressure, clay with particles bound by hydrate bonds behaves like a solid with atomic structure: it has strength, sometimes slowly and weakly reacting with water. The ultimate strength of solid clays is described by Equn 23.

The shear strength of ductile clays will be determined by the water content in the composition of the silica gel membrane of the clay particles, and by the condition of the hydrogen bonds in the membrane. The hydrate membrane, at the moment the load is applied, acquires a water-repellent property due to the significant induction period of bond formation, and the triaxial bond between the clay particles disappears. They become non-oriented in relation to one another and react weakly with each other.

In order to determine the design strength and evaluate the stability of consolidated soils, such characteristics as the degree of ductility and the magnitude of specific cohesion are used. Ductility indicates the clay’s capacity to absorb water without dispersing: the lower a clay’s ductility, the faster it will disperse. The ductility index itself is informative for the drilling process as an indicator evaluating the clay’s absorbing capacity and its mechanical impurity content. The disadvantage of this indicator is that, in order to determine ductility, an initial sample of clay must be dried. The advantage is that drilled-out rock cuttings are used to determine its value. Yielding characterizes the initial state of the clay due to its ductility. Physically, this is reflected as an indicator demonstrating how close the clay in its initial state is to its ductile state, and its capacity for dispersion. Where the yield index $I_l > 0$, argillaceous soils are considered to be ductile.

The process of ductile clays destruction is characterized by swelling of clays without any loss of bond between particles, and by their subsequent dispersion in the volume of the fluid. Where the yield index $I_l < 0$, argillaceous soils belong to the solid group, and the process of destruction under the impact of arising stresses occurs with a minimal quantity of water absorbed. In clays with varied degrees of ductility, porosity will be equal to zero; this is connected with the fact that the water structured in it forms part of the clay structure and is its continuation [5].

The ultimate shear stress in soils is calculated according to the Mohr-Coulomb strength theory:

$$\tau = \tau_n + \sigma_n \tan \phi$$  \hspace{1cm} (23)

$$\sigma_n = \sigma_{nmax} - \sigma_n \tan \phi$$  \hspace{1cm} (24)

where $\tau_n$ is the cohesion factor, corresponding to the ultimate shear-stress value at $\sigma_n = 0$, MPa;

$\phi$ is the internal friction angle, degrees;

$\sigma_n$ is the normal stress (Equn 24), MPa.

The condition (Equn 25) corresponds to the soil’s elastic state, taking into account longterm strength $k_\ell$

$$|\tau_{nmax}| \leq k_\ell \tau_n$$  \hspace{1cm} (25)

The safety margin of wellbore walls at the point under examination is given by Equn 26:

$$n = \frac{\tau_n}{\tau_{nmax}}$$  \hspace{1cm} (26)
if \( n > 1 \) then the solid is in an elastic state;
if \( n < 1 \) then the solid may deform plastically or collapse.

During the construction of underwater crossings using the directional-drilling method, passing through non-consolidated soils (such as sands, gravel, and shingle) is complicated by the unstable state of the borehole shaft, and by constant collapses. Furthermore, coarse fractions are not carried out of the well, but accumulate on the lower surface of the borehole. Maintaining stability in these soils for directional boreholes with large diameters is a challenging technical task. The nature of these collapses depends on the particle-interaction processes. Interaction between soil particles is characterized by internal friction forces and structural cohesion. The shear strength of non-consolidated soils grows in proportion to the increase of intergranular contact area, which is expressed in parameters such as porosity and density.

For dense soils, the structural cohesion is ranges from 0.03 to 0.05 MPa, and for loose soils it is 0 MPa. The relationship between shear resistance \( S \) in non-consolidated soils and load \( p \) given their different density (per porosity metres: \( m_1 \) indicates a limited loose state, and \( m_4 \) a dense state), and the relationship between the angle of internal friction of a medium-grained sand and its porosity \( m \) are presented in Figs 4 and 5 [6]. Structural bonds have elastic character, determining the extent of rock deformability and its compaction. This accounts for a sand friability on the surface of the earth. In the subsurface, the soil is in a compacted state; however, when the stratum is drilled, decompaction occurs, i.e. the grains of rock become loosened, ruining the structural strength of the soil.

During the drilling process, the drilling fluid vigorously reacts with penetrated soils. Clay dispersion, having a low content of clay particles, seeps into the soil and wets its particles, thus leading to its disintegration. The structural bonds between the particles ruptures, and the soil begins to flow. Up to a certain soil-moisture level, the angle of internal friction hardly changes. However, the angle of internal friction tends towards rapid reduction with oversaturation of the rock with water. This is a direct consequence of the increased water content in non-consolidated soil, accompanied by the reduction in the number of contacts between soil particles [6].
The maximum safety margin for non-consolidated soil in a dense condition is the relationship between $t_u$ and $s_u$, which is given by Eqns 23 and 26.

Rocky soils have sufficient strength to support a borehole in a stable state. The strength condition is determined by uniaxial compression and punch indentation tests. A stable state in argillaceous soils is achieved when clay is in a solid or semi-solid state, while where it is in a plastic state, the clay will be extruded into the borehole. The clay's state is determined according to its yield strength and structural cohesion value. The stability of non-consolidated soils is determined by the degree of their compaction in situ, and by its structural cohesion and the angle of internal friction.

On the basis of the calculations presented here and their comparison according to Equn 26, it is possible to predict the stability of soils along a borehole. Where the condition $n > 1$ is not fulfilled for a particular interval, the borehole walls are liable to collapse and will be unstable.

The interaction of dispersed non-consolidated soils with drilling fluid

Drilling fluid (mud) is a poly-dispersed structure linked by hydrogen bonds, which are created by the chemical properties of its constituents. Therefore, the behaviour of the drilling fluid is determined by the strength of these hydrogen bonds and their degree of distribution throughout the fluid. The strength of the drilling-fluid structure, in turn, depends on the level of polarity and concentration of constituents in the fluid. Where there is a uniform distribution of bonds' strength in the fluid, the mud will display clearly manifested pseudo-plastic properties. Where there is a less-uniform level of distribution of hydrogen bonds' strength throughout the fluid, the drilling mud will acquire the properties of a visco-plastic liquid.

Drilling fluid is filtered through permeable channels in a non-consolidated soil, whose size is much bigger than the particles of the mud's solid phase. Filtration takes place until the moment when the differential pressure and pressure losses for the mud movement are equalized; at that point, the mud structure does not disintegrate. Where the depth of the drilling mud penetration into the permeable bed is achieved, and depending on the energy input leading to its structural collapse, part of the water is removed from the mud's disintegrated structure, and is accompanied by accumulation of the solid particles into the mud.

The solid particles either form a united structure or simply accumulate as the solid phase (Fig.6), depending on the state of the water around them, and the quality of the bonds within it. When permeable channels become smaller than the size of the solid-phase particles, the structure of the mud disintegrates under the impact of differential pressure and it separates into its constituents, which clog the permeable channels, forming a barrier in the near-borehole zone.

The seepage depth of the fluid depends on the following conditions: the seepage depth will decrease at constant differential pressure, with higher shear stress in the liquid, while the seepage depth will increase at a constant effective shear stress and higher differential pressure. That is, the uniformity of distribution and the seepage depth of the drilling mud are affected by its pressure at pumping and by the strength of the bonds in the mud volume, i.e., if the mud
constitutes a single structured system, and is not broken into separate structured clusters, then filtration will take place. A significant role in the filtration process belongs to the strength of the bonds in the liquid’s volume, and this can be seen especially clearly where components of the liquid satisfy Abrams’ criteria [7], i.e. the effective diameter of particles is less than one-third of the diameter of the permeable channel. The results of well-known studies into filtration processes were analysed on the basis of a similar approach to drilling-mud filtration in non-consolidated soils.

Supported by the data [9] for determining the fractional content of clayey suspensions and muds, and also by the results of their filtration through non-consolidated soils of various fractional content [8], results were studied for the process of drilling-mud filtration through clayey suspensions and muds (Fig.7). It was understood from this that the possibility of successful drilling-mud filtration into a porous medium is determined by the ratio of the permeable channel size ($D_w$) to the drilling mud particles size ($D_T$) and by the pressure gradient between the well and the bed. This can be achieved given the following conditions:

- where $D_w / D_T < 1$, drilling-mud filtration does not occur, and the solid phase of the mud accumulates on the surface of the filter;
- where $1 < D_w / D_T < 6$, drilling-mud filtration occurs with infilling of the pore space in the non-consolidated soil, i.e. with forming a clogged zone;
- where $D_w / D_T \geq 6$, the filtration depth is determined by the condition of the bonds in the mud structure and by the pressure gradient between the mud volume and the filter.

An analysis of the results of research [8] into the process of muds filtration in sand samples of various grain-particle size showed that at shear stresses from 0.2 to 0.7 MPa, filtration of the mud extends to the full length of the sample while keeping its geometrical dimensions where ductility factor values are 800-3000 s$^{-1}$ (Fig.7). In order to guarantee the process of the drilling-mud filtration through samples based on fine sand with an effective particle diameter of 0.142 mm, the necessary DSS (dynamic shear stress) value should be 120 dPa. In order to guarantee the process of filtration through samples of un cemented soil.

![Fig.7. The impact of the ductility factor on the dispersal of various fractions on the degree of their filtration through samples of un cemented soil.](image-url)
through a porous medium with large effective particle diameter, the DSS value of the drilling mud must be increased. Where a value of $\tau/\eta > 1300$ was guaranteed, not only was drilling-mud filtration successful, but non-consolidated soil samples were also strengthened due to the strength of bonds between the drilling mud’s constituents and the soil particles.

Limiting values were thus determined for successful drilling-mud filtration with strengthening of the porous medium, based on studies and experience of constructing large-diameter directional boreholes. These limiting values are: the ratio of permeable channel size to drilling mud particle size ($D_w/D_t$) > 6; dynamic shear stress $\geq 300$ dPa; the ductility factor $\geq 1300$ s$^{-1}$. These values for the ductility factor allow control of the condition for non-consolidated soils to improve their strength in order to form stable borehole walls when passing through unstable soils during the borehole drilling.

Interaction of argillaceous soil with drilling mud

The process of clay disintegration in a borehole where there is a reaction with water includes two stages. In the first stage, the clay absorbs water; and in the second, which limits the process of clay disintegration, hydrated particles of clay disperse. The factors which determine the process of clay disintegration are presented in Fig.8.

The disintegration of clay can occur during hydration and swelling of solid non-ductile clays. The increase in clay volume during the hydration process, i.e. the appearance of strains, contributes to the occurrence of stresses in the rock at the borehole walls, leading to its collapse. All these processes are characteristic, above all, for clays with high ductilities and low yield indicators. For a clay capable of absorbing a large quantity of water, the process of its disintegration is characteristically accompanied by a significant increase in volume.

With an increase of water content in the clay and the consequent increase in its yield indicator, the role of the degree of clay volume increase will decline, and there will be a drop in the magnitude of stresses capable of leading to the occurrence of the forces which disintegrate the clay at the borehole wall. At the same time, there will be an increase in the possibility of complications with key seals, wall-packing rings and other technical components. It will also be necessary to prevent dispersion of argillaceous cuttings in the high-velocity flow of the drilling mud.

Where a moisture-content indicator characterizing the reduction of its strength characteristics is reached, the clay will tend towards flowing under the impact of geostatic pressure (quick clay). However, it must be understood that where clay is significantly diluted by inert components (fragments of rock, sand, etc.), it is these components that will make up the structural body of the argillaceous sediments and determine their behaviour under the impact of geostatic pressure. In
this case, the stability of the borehole walls will be determined by the inert components of argillaceous sediments.

Clays which have a low ductility index values will be characterized by another behaviour in situ, i.e. in the case when their hydro-silicate membrane has low absorbing capacity relative to water. For the separate minerals of clay, the ductility number is determined by the value of $I_p$ approx.10-30 [9] according to Grim’s data. It is the boundary value characterising the reduced moisture retention capacity of clay (i.e. its hydrophilic).

Where there is contact with water and absorption of a small quantity of water, similar clays, increasing in volume, will destroy the borehole walls. But their relatively low hydrophilic forces this clay immediately to disperse, regardless of their moisture-retention state, i.e. the yield indicator. The role of a factor such as an increase in volume will be less significant for these clays due to the speed of their hydration process and the low quantity of water absorbed by them. The dilution of these clays with filler material only accelerates and aggravates their disintegration process.

A similar speed for processes of hydration softening and disintegration is dictated by the fact that the hydro-silicate membrane of clay undergoes significant changes during the bleeding processes, and has low ductility and flexibility. Therefore, it is not capable of absorbing a significant quantity of water. The drilling process in such a clay is characterized by intensive clay dispersion, borehole collapse, higher recovery of cuttings, etc. In order to stabilize the borehole in similar argillaceous sediments, technical actions should be taken, depending on the impact on clays and their ductility indicators.

**Geomechanical model of an underwater crossing for a trunk pipeline**

When constructing a geomechanical model for a trunk pipeline underwater crossing using directional drilling, the following points need to be considered:

- analysis of the conditions in which work is to be carried out (acquisition of geological, geophysical, and petrophysical data; analysis of drilling and reaming the boreholes for underwater crossing, together with analysis of the drilling muds used in similar geological conditions; analysis of complications which might arise during the construction of underwater crossings; identification of any additional data required for modelling; summarizing the analysis results);
- analysis of the initial stress-strain state of the soils making up the borehole walls (calculation of the loads acting on the rock in the near-borehole zone, and the stresses thereby arising; assessment of the condition of the borehole walls; prediction of possible complications);
- hydrodynamic modelling with the impact of drilling mud, or other fluid, on the state of the rock and its strength characteristics (choice of rheological parameters for drilling mud; calculation for filtration of drilling or fluid into the bed; borehole flushing)
- analysis of the stress state of the soils making up the borehole wall, taking into account data from hydrodynamic modelling;
- summing-up and making recommendations for improvement to the technology for constructing underwater crossings, drilling trajectory, drilling-fluid formulation, the basis for using the directional-drilling method regarding the conditions of crossing construction.

Based on the methodology presented above, geomechanical models have been constructed for the underwater crossings of trunk oil pipelines in differing geological conditions across the Suvorosch and...
Kuvash Rivers. Profiles of the boreholes and the geological conditions are shown in Figs 9 and 10.

An analysis of the works carried out at the Suvoroshch and Kuvash Rivers showed that the greatest difficulties during borehole drilling and reaming will be due to the following complications: instability of the borehole walls, lost circulation of the drilling mud, insufficient cuttings recovery from the borehole, narrowing of the borehole, and absorption of the clay into the mud.

A preliminary analysis of the stressed...
Condition of soils making up the borehole walls in the Suvoroshch River crossing revealed two dangerous areas with very high active pressures: sandy silts at a depth of up to 12 m (compressive stresses were up to 6 MPa, and tensile stress up to 0.08 MPa) and very soft loamy soil at a depth of 5 m (tensile stresses were up to 0.18 MPa).

**Fig. 11.** Soil safety margin for the Suvoroshch River crossing.

**Fig. 12.** Soil safety margin for the Kuvash River crossing.
Pipeline Science and Technology

78

to 0.04 MPa and compressive stresses up to 2.5 MPa). Throughout the length of the entire borehole, the maximum shear stresses are two times higher than the ultimate strength of the soils forming the borehole.

At the Kuvash River crossing, the highest tensile stresses were found in the borehole wall during drilling through shales at a depth below 17 m. However, the ultimate strength of shales is very high, at more than 50 MPa, and thus complications do not arise when they are drilled. The greatest challenge is presented by shingle and gravel soils: where they occur, at a shallow depth of up to 7 m, the resulting stresses are not great, but the ultimate strength of such soils is minimal.

Data were thus obtained concerning the stability of the boreholes at the Suvorosch River (Fig.11) and the Kuvash River (Fig.12) crossings which indicate that the boreholes must be strengthened.

Boreholes can be strengthened using structured drilling muds or spacing-strengthening mud formulations. Preventing disintegration in argillaceous sediments is possible with the addition into the fluid of a special stabilizing chemical. The drilling mud parameters which regulate bond strength in the mud and the filtration process into the permeable channels were as follows: dynamic shear stress of 30-40 Pa and density of 1050 kg/m³. A curve describing the seeping depth of the drilling fluid into the permeable channels of the soil forming the underwater crossing of the Suvorosch River is presented in Fig.13.

An analysis of the stress state of the soils forming the borehole wall, considering the data from hydrodynamic modelling, showed that as a result of the work carried out, the safety margin of the soils forming the borehole walls was tripled along the entire length of the underwater crossing of the Suvorosch River. The minimum safety margin during drilling of sandy silts was 1.41 at a depth of 12 m, in fine sand it was 1.25, and in the area passing through very soft clays it was 1.04.

At the Kuvash River crossing, the safety margin was more than 1.5 when drilling consolidated soils, more than 88 for constituting shales, and more than 1 for gravel and shingle soils.

The geomechanical models which were constructed for the Suvorosch and Kuvash River crossings thus revealed the necessity of strengthening the soils which form the borehole walls. Soil strengthening should be carried out by using drilling mud with increased structural strength. Regarding the conditions when drilling through gravel and shingle sediments for the crossing under the Kuvash River, additional strengthening with special formulations is advisable.

**Conclusions**

Based on the results of the work completed, basic criteria were formulated for geomechanical modelling of the conditions for constructing underwater river crossings for trunk pipelines.

The methodology presented here of calculating the stressed condition of

![Fig.13. Penetration depth of drilling fluid in uncemented soil.](image-url)
soils allows loads acting on the soils to be determined for the wellbore zone of directional and horizontal boreholes, taking into account the impact of the drilling mud. The stresses thus arising can also be determined, where they may be capable of leading to hydro-fracturing in the soils.

The ultimate strength of the soils can be determined using the Mohr-Coulomb strength theory, which takes into account the nature and bond strength of the minerals and particles making up the soil.

The model’s reliability is increased by the presence of experimental data for strength and strain properties in soils and research into the influence of drilling muds on them.

An analysis of the stress state of the soils, and hydro-dynamic modelling of the impact of drilling mud on the state of the soils and their strength characteristics, must be carried out jointly in view of the redistribution of stresses within the soil when pores and fractures areas are filled with drilling fluid. This will also improve the quality of geomechanical modelling.

Within the framework of the geomechanical model developed for conditions in the construction of underwater crossings, we plan further to model the stress-strain state of the pipeline in the contact zone with soil and at its pulling through the borehole.

References

1. V.S.Voytenko, Applied geomechanics in drilling process, Moscow, Nedra, (1990) 252
2. A.N.Popov, N.N.Golovkina, Strength analysis of wellbores in porous rocks, Ufa UGNTU, (2001) 70.
3. S.G.Lekhnitsky, Stress analysis in elastic isotropic mass near vertical cylindrical circular excavation, AS USSR, ESD. 7 (1938).
4. L.A.Shreyner et al., Rock deformation behavior at high pressures and temperatures, Moscow, Nedra, (1968) 358.
5. V.M.Goldberg, N.P.Skvortsov, Permeability and filtering in clay, Moscow, Nedra, (1986) 160.
6. N.N.Maslov, Fundamentals of soil mechanics and geological engineering, Moscow, Higher School, (1968) 629.
7. N.Makovei, Drilling Hydraulics. M.Nedra, (1986) 600.
8. Z.Z.Sharaftudinov, et al., Construction of main pipeline crossings through natural and artificial obstructions, Novosibirsk, Nauka, (2013) 339.
9. R.E.Grim, Mineralogy and practical use of clay, Moscow, Mir,(1967) 510.