Underground pipeline strength under non-one-dimensional motion

K Sultanov¹*, B Khusanov² and B Rikhsieva²
¹Tashkent Institute of Irrigation and Agricultural Mechanization Engineers, Department of Theoretical & Structural Mechanics, Tashkent, Uzbekistan
²IMSS, Academy of Sciences of the Republic of Uzbekistan, Tashkent, Uzbekistan
sultanov.karim@mail.ru

Abstract. This paper is devoted to the development of a method for determining longitudinal stresses in underground pipelines under periodic dynamic load, taking into account the complex process of pipe-soil interaction. The method is based on the solution of a two-dimensional axisymmetric unsteady wave problem for the “underground pipeline-soil” system. In this case, the pipeline and soil are taken as linearly deformable bodies. On the surface of their contact, the force of interaction (the friction) is determined from a two-stage (pre-limit and limit) law. The problem is solved numerically - by the finite difference method according to the Wilkins scheme. The changes in longitudinal stresses, velocities, and displacements over time were obtained for various sections of the pipeline and soil. An analysis of the results of numerical calculations showed that when a plane wave propagates along a pipeline with external friction, the hypothesis of flat sections is practically met, which justifies the regularity of considering similar problems in a one-dimensional statement. A multiple increase in the values of longitudinal stresses in the pipeline as compared to the same stresses in soil has been established. The maximum possible value of the stress growth is determined depending on the physic mechanical properties of the pipe material and soil. The role of the interaction force (the friction force) in the multiple increase in longitudinal dynamic stresses in underground pipelines under the effect of dynamic loads was determined.

1. Introduction
The use of underground pipelines in hydro-technical and ameliorative engineering and transport systems for water, gas, oil and oil products, pose the problem of ensuring their safe and uninterrupted operation. The reliability of underground pipelines is associated with the tasks of predicting their strength under various static and dynamic effects. Currently, intensive work is being carried out all over the world to develop the methods for more exact determining the strength of underground pipelines. The relevance of this problem is connected with potential catastrophic consequences in the case of underground pipeline damage.

Intensive construction of underground and submerged pipelines requires reliable operation under various external dynamic (seismic) loads [1-5]. Underground and submerged trunk pipelines are mainly used as transporting means for gas and liquid substances [5]. They are successfully used in hydro-technical and ameliorative engineering for water transportation [1]. The problem of ensuring their safety and reliable operation requires assessment and prediction of the strength of these underground and submerged communications under the influence of various static and dynamic loads.
According to the results of complex experimental and theoretical studies [1], strength factors and aspects of underground and submerged pipelines, the problem of the strength of main pipelines built for various purposes has attracted the attention of many researchers [2-24]. The issues of stability of underground pipelines under static and dynamic loads [6-10] were considered in [6-24]; the problems of the stress-strain state formation in [7, 11, 12]; the issues of the effect of design features on the stability of underground pipelines in [11-16]; the issues of the pipe-soil interaction in [14-24].

The main factors affecting the reliable operation of underground pipelines are listed in [1]. A method has been developed for determining longitudinal stresses in an underground pipeline based on the one-dimensional theory of wave propagation in the "underground pipeline-soil" system. When considering the one-dimensional wave problem in [1], some assumptions and simplifications were made. In fact, the process of wave propagation in the "underground pipeline-soil" system is a non-one-dimensional one. The present paper is devoted to checking the acceptability and accuracy of a one-dimensional approach in determining the stress state of an underground pipeline under dynamic impacts. The aim of the work is to determine the stress state of an underground pipeline under dynamic impacts by solving a non-one-dimensional wave problem.

The objectives of the study are:
1. The statement of a two-dimensional axisymmetric wave problem for the "underground pipeline-soil" system, taking into account the interaction forces on the surface of their contact arising from dynamic impact.
2. The development of an algorithm and program for the numerical solution of the problem and its substantiation.
3. The determination of longitudinal stresses in the pipeline depending on the problem under consideration.

2. Methods

Engineering methods for determining longitudinal stresses in underground pipelines were analyzed in [1]. These methods are rather simple in their implementation. But they are based on rough assumptions and hypotheses, noted in [1]. Therefore, the longitudinal stresses in the underground pipeline determined using engineering methods are very approximate ones.

The method considered in [1] is based on solving a one-dimensional unsteady wave problem for an underground pipeline interacting with soil. In this case, the underground pipeline is modeled as a rod, and the soil around the pipeline is also modeled as a cylindrical rod. Soil medium and the pipeline material are considered as viscous-elastic bodies. At the boundary of their contact, various interaction laws are considered: the Hooke type, the Kelvin-Voigt type, the Eyring model type, and the Coulomb law [1]. As can be seen, even in a one-dimensional statement under the above assumptions, the problem of determining longitudinal stresses is quite complicated.

Based on numerical solutions [1], longitudinal stresses in the pipeline under dynamic loading were determined and the mechanisms of the stress state formation in the pipeline were revealed. The fact of a multiple increase in longitudinal stress in the pipeline with respect to the maximum value of acting load was found. The main reason for this growth is the friction force (interaction) that occurs on the contact surface of an underground pipeline with soil.

Consider the simplest case of a one-dimensional problem. The pipeline material and soil are considered to be elastically deformable bodies:

\[
\sigma_{zz}^p = \lambda^p \dot{\epsilon}^p + 2G^p \frac{\partial U_p^p}{\partial z}, \quad \sigma_{rr}^p = \lambda^p \dot{\epsilon}^p + 2G^p \frac{\partial U_p^p}{\partial r},
\]

\[
\sigma_{\phi\phi}^p = \lambda^p \dot{\epsilon}^p + 2G^p \frac{U_p^p}{r}, \quad \sigma_{\phi r}^p = G^p \left( \frac{\partial U_p^p}{\partial r} + \frac{\partial U_r^p}{\partial z} \right)
\]
\[ \sigma_{zz}^{gr} = \lambda^{gr} \frac{\partial U_{z}^{gr}}{\partial z} + 2G^{gr} \frac{\partial U_{z}^{gr}}{\partial r}, \quad \sigma_{rr}^{gr} = \lambda^{gr} \frac{\partial U_{r}^{gr}}{\partial r} + 2G^{gr} \frac{\partial U_{r}^{gr}}{\partial z}, \]
\[ \sigma_{\phi\phi}^{gr} = \lambda^{gr} \frac{\partial U_{\phi}^{gr}}{\partial \phi} + 2G^{gr} \frac{U_{r}^{gr}}{r}, \quad \sigma_{\tau r}^{gr} = G^{gr} \left( \frac{\partial U_{z}^{gr}}{\partial r} + \frac{\partial U_{r}^{gr}}{\partial z} \right), \]

(2)

where \( \sigma_{jk}^{p}, U_{r}^{p}, \lambda^{p}, G^{p} \) and \( \sigma_{jk}^{gr}, U_{r}^{gr}, \lambda^{gr}, G^{gr} \) are the components of the stress tensor, longitudinal velocity, radial velocity, Lame coefficients of the pipeline and soil, respectively. A dot over the parameters means the time derivative. Here, the expression for the rate of volume strains \( \dot{\theta}^{p} \) and \( \dot{\theta}^{gr} \) correspond to the continuity equation (for pipe and soil, respectively):

\[ \dot{\theta}^{p} = \frac{\partial U_{r}^{p}}{\partial z} + \frac{1}{r} \frac{\partial \left( r U_{r}^{p} \right)}{\partial r}, \quad \dot{\theta}^{gr} = \frac{\partial U_{z}^{gr}}{\partial z} + \frac{1}{r} \frac{\partial \left( r U_{r}^{gr} \right)}{\partial r} \]

(3)

The equations of motion of the pipeline and soil in a cylindrical coordinate system \( r, z, \phi \) have the form:

\[ \rho^{p} \frac{dU_{r}^{p}}{dt} = \frac{\partial \sigma_{rr}^{p}}{\partial r} + \frac{\partial \sigma_{r\phi}^{p}}{\partial \phi} + \frac{\sigma_{rr}^{p} - \sigma_{\phi\phi}^{p}}{r}, \quad \rho^{gr} \frac{dU_{r}^{gr}}{dt} = \frac{\partial \sigma_{rr}^{gr}}{\partial r} + \frac{\partial \sigma_{r\phi}^{gr}}{\partial \phi} + \frac{\sigma_{rr}^{gr} - \sigma_{\phi\phi}^{gr}}{r} \]

\[ \rho^{p} \frac{dU_{z}^{p}}{dt} = \frac{\partial \sigma_{zz}^{p}}{\partial z} + \frac{\partial \sigma_{z\phi}^{p}}{\partial \phi} + \frac{\sigma_{zz}^{p} - \sigma_{\phi\phi}^{p}}{r}, \quad \rho^{gr} \frac{dU_{z}^{gr}}{dt} = \frac{\partial \sigma_{zz}^{gr}}{\partial z} + \frac{\partial \sigma_{z\phi}^{gr}}{\partial \phi} + \frac{\sigma_{zz}^{gr} - \sigma_{\phi\phi}^{gr}}{r} \]

(4)

(5)

Here \( \rho^{p} \) and \( \rho^{gr} \) are the density of the pipeline material and soil, related to volume strain through relations: \( \dot{\theta} = \dot{V}/V, \quad V = \rho_{0}/\rho, \quad \rho_{0} \) is the initial density.

The given equations (1) - (5) represent the complete system of equations for an underground pipeline and soil. The boundary conditions are established now. The interaction model is taken as in [1], considering the non-one-dimensionality of motion and taking into account the two-stage law: pre-limit (Hooke type) and limit one (Coulomb law):

\[ \tau = K_{z} \left( \sigma_{N}, I_{S} \right) \cdot \bar{u} \quad \text{at} \quad 0 \leq \bar{u} \leq \bar{u}_{*}, \quad d\bar{u}/dt > 0 \]
\[ \tau = f \cdot \sigma_{N} \quad \text{at} \quad \bar{u} > \bar{u}_{*} \]
\[ \tau = K_{R} \left( \sigma_{N}, I_{S} \right) \cdot \bar{u} \quad \text{at} \quad \bar{u} < \bar{u}_{*}, \quad d\bar{u}/dt < 0, \quad d\bar{u}/dt < 0 \]

(6)

(7)

(8)

(9)

where \( \bar{u} = U_{z}^{p}(z, R_{0}, t) - U_{z}^{gr}(z, R_{0}, t) \) is the pipeline displacement relative to soil layer, \( \bar{u}_{*} \) is the critical value of relative displacement at which the structure of soil contact layer is completely destroyed, \( R_{0} \) is the outer radius of the pipeline; the expressions and values of remaining parameters in (6)-(8) are given in [1]. The second condition on the contact surface is taken as follows:

\[ U_{r}^{p}(z, R_{0}, t) = U_{z}^{gr}(z, R_{0}, t) \]

(10)

Here in (6)-(10) \( \sigma_{N} \) and \( U_{N} \) are the normal components of velocity and stress to the contact surface, determined by expressions:

\[ \sigma_{N} = \sigma_{zz}^{gr} \sin^{2} \alpha + \sigma_{rr}^{gr} \cos^{2} \alpha - 2\sigma_{zr}^{gr} \sin \alpha \cos \alpha, \quad U_{N} = -U_{z} \sin \alpha + U_{r} \cos \alpha \]

(11)

where \( \alpha \) is the angle of inclination (under strain) of the side surface of the underground pipeline.

Suppose that at the initial point in time a dynamic load acts on the front end of the pipeline and soil in the form
\[
\sigma_{zz} = \sigma_{\text{max}} \sin \left( \pi t / T \right), \quad \sigma_{zr} = 0 \quad \text{at} \quad z = 0, \quad t \geq 0
\]  \hspace{1cm} (12)

Before the load application, the underground structure-soil system is considered as stress-free and strain-free.

Note that, for the purpose of a stage-by-stage consideration of the problem, here the pipeline material and soil are assumed to be linearly elastic in the form given in (1)-(2). In [28–30], nonlinear models of soil strain were proposed taking into account their structural destruction under strain. Methods for determining mechanical characteristics of viscous-elastic and viscous-elastic-plastic soils were considered in [31]. The solution of non-one-dimensional unsteady wave problems for the “underground pipeline - soil” system using more complex laws of soil strain [25–31] is a problem to be solved in future. Here, we restrict ourselves to the simplest statement of the problem, based on the aim of this study.

2.1. Solution method

The main equations (1)-(5) with zero initial conditions and boundary conditions (6)-(12) were solved by the finite difference method according to the scheme proposed in [32]. A detailed substantiation of this method and its applicability in solving non-stationary wave problems were given in [32]. To the studies in [32], we can add that in the solution method under consideration, the wavefronts are smeared with artificial viscosity and therefore are not the discontinuity surfaces. The tangential discontinuities that arise, for example, when a wave propagates at the boundary of two media, must be distinguished in the calculation. Slippage should be introduced at this boundary; otherwise, a characteristic viscous layer is formed here, which has not a physical but a finite-difference nature. A certain complexity arises when conditions (6)-(9) are satisfied at the contact boundary. In calculations, these features are overcome by introducing double discrete points (bifurcation of nodes) on the contact boundary (Figure 1.a) so that the nodes of the pipeline and soil slide against each other according to the accepted condition (6)-(9). Here, one of the surfaces is considered a fixed boundary within the one-time step. The equations of motion for the pipeline are the same as the corresponding equations for the case of motion along a fixed boundary, taking into account the forces acting from the soil, determined from (6)-(9). In this case, soil cells play the role of fixed cells. Then these fixed cells on the contact surface are displaced over time, with the force field, available in the adjacent underground pipeline.

The sequence of calculations on the contact boundary is as follows:

1) The dummy cell \( B_0 \), its mass, and stress state in the soil are determined (Figure 1, b). Between points \( B_0 \), \( B_1 \) of the structure there can be any number of soil points with numbers \( k+1, k+2, \ldots \).

In the dummy cell \( B_0 \), the mass and stress components are determined

\[
M^n = \sum_{i=1}^{N} M_i^n, \quad \sigma_{ij}^n = \sum_{m=1}^{N} M_m^n \left( \sigma_{ij}^n \right)_m / M^n.
\]

Where \( N \) – is the number of soil grid cells falling into the dummy quadrangle. Mass and stress values in the quadrangle \( B_0B_1B_5B_7 \) are determined similarly;

2) To determine the stress acting in the interval \( (B_0B_1) \), formulas are derived for the velocities at points \( B_0^C \) and \( B_0^G \), which belong to the pipeline and soil at time \( l^n \). These formulas are obtained from equations (1) by integrating them over area \( A \) (for point \( B_0^C \)) and over area \( A^* \) (for point \( B_0^G \)) (Figure 1, b). They include boundary stresses acting in the intervals \( (B_0B_5) \) and \( (B_0B_7) \). Equating the normal components of the velocity at point \( B_0 \) on both sides of the contact boundary and assuming that the stress acting on this point is known from the count of the previous point, we determine the stress value for point \( B_1 \);
3) Knowing the boundary stress, we determine \( U^p_z, U^p_r \) at point \( B_0^c \) for the moment \( t^{n+1/2} \) using the difference equations [32] obtained by integrating equations (4) over the area \( A' \) (Figure 1, b). The coordinates of the point \( B_0^c \) and all other values related to the adjacent cell are determined by the general formulas [32];

4) To determine velocities and coordinates of soil boundary, the boundary stress is re-interpolated from the pipeline to soil. Based on the obtained boundary stresses for \( B_0 \) and \( B_1 \) (Figure 1, c), \( U^{gr}_z, U^{gr}_r \) are determined for the moment \( t^{n+1/2} \);

5) Using these velocity components, the normal component of the pipeline velocity along the boundary is determined by the second formula (11). The tangential component of the velocity is determined by interpolation from the points of the pipeline boundary to the soil points. From these two components, the velocities and coordinates of soil at point \( B_0^c \) are restored. Note that this is introduced to prevent, if possible, the divergence or intersection of the boundary points at the pipeline and soil contact.

An important aspect of calculations is that the parameters at the pipeline boundary are connected only with this material, and that the soil forming a fixed boundary is considered so that for it this boundary would be an external surface with acting forces (6)-(9) and normal stress determined during the counting process. The satisfaction of stability conditions of the scheme is carried out as follows: a
time step is determined for the pipeline and soil, then the smallest time step is selected in the counting process, which ensures the stability of the overall scheme.

Here, along with the non-one-dimensionality of motion, it is believed that the friction force arising on the contact surface directly depends on the parameters of waves propagating in soil and pipeline. The non-linearity of the problem is due to the fact that the values of the friction force are determined from the wave parameters and stress state of soil and pipe; besides, the direction of action of this interaction force depends on the kinematic parameters of underground pipeline and the external medium - soil. This nonlinearity can be considered one of the types of structural nonlinearity. Structural nonlinearity is considered a separate nonlinearity, after geometrical and physical nonlinearity [26-29]. Also, structural nonlinearity significantly complicates obtaining analytical solutions to the problems considered. Analytical solutions of the problem in a one-dimensional statement, a review of which is given in [1], were obtained under various assumptions leading to linear problems. In some cases, the friction force was determined by the constant normal pressures of the external medium - soil, in other cases, the directions of the friction force were considered as known.

In the case of underground pipelines - soil interaction, as shown by the results of experiments conducted in [1], the strength of interaction depends on the strain characteristics of the soil. Often there may be cases of a change in the direction of force action. Then the force of interaction (the friction force) plays an unusual role and turns into an active force. Besides, under the action of long waves, which correspond to seismic waves, the first stage of interaction plays an essential role [1]. Namely, at this stage of interaction, structural destruction of the near-contact soil layer occurs [30]. The second stage of interaction, which coincides with the Coulomb law, differs greatly from the interaction of two elastic media. Here, in fact, an interaction occurs between the soil particles; the coefficient of friction, both in value and in meaning, is the coefficient of internal friction of the contact layer of soil [30]. It should be emphasized that the pattern of interaction in the second stage corresponds to the state of plastic strain (flow) of soil. In this case, the shear stress is proportional to the normal stress acting on this site and does not depend on shear strain [30].

### 3. Results and discussion

Initial data are taken as in [1]: the initial density of the pipeline material is 7800 kg·m⁻³, the longitudinal wave propagation in the pipe is 5000 m·s⁻¹, the Poisson's ratio is 0.3; for soil, these characteristics are 1800 kg·m⁻³, 1000 m·s⁻¹ and 0.3, respectively. An underground pipeline is in the form of a cylindrical body with a diameter of 0.2 m. In this case, there is no free inner surface on which (due to radial strains) radial stress can be reduced in the underground pipeline. Note that in [1], depending on the external load and the interaction conditions in the underground pipeline, a multiple increase in stresses was observed over time.

Here are some results [1]: under the action of a load of 0.5 MPa, the stresses in the pipeline increased by 48 times; in the case \( \sigma_{\text{max}} = 1 \text{ MPa} \) –by 100 times; at \( \sigma_{\text{max}} = 2 \text{ MPa} \) –by 200 times; further, with an increase in external load values, a slight increase in stresses occurred in the pipeline. In all cases under consideration, the half-period of the load is taken equal to 0.01 sec. Let \( \sigma_{\text{max}} = 0.5 \text{ MPa} \) for the problem under consideration, and the parameters of the Hooke type interaction model are exactly the same as in [1]: \( K_N = 100 \text{ m}^{-1}; \, \alpha_S = 0.5 \) and \( \alpha_S = 1.5 \), which corresponds to \( \beta = 2 \) in [1].

The interaction model is taken in the form (6)-(9) with the transition to the second stage of interaction (\( \beta = 0.425 \)). The normal pressure included in the interaction model is defined as in [1]. The depth of the pipeline is 1 m.

The calculation results are shown in Figures 2 and 3. Figure 2 shows the changes in longitudinal stresses of the underground pipeline \( (r=0) \) and the contact layer of soil \( (r=0.21 \text{ m}) \) over time in fixed sections. It can be seen that the result obtained qualitatively reflects the result of the one-dimensional problem [1]. Stresses in the underground pipeline increase at the arrival of waves propagating in soil. The amplitude of the maximum stress values increases by 21-22 times in comparison with the amplitude of the set load. The effect of the interaction force on the stress state of soil is almost
insignificant, similar to the results in [1]. Only a relative decrease in the amplitude of longitudinal stresses is observed with a distance from the front end of the pipeline in question. Note also that the difference between the wavefronts of the pipeline and soil slightly affects the overall process of interaction.

\[
\begin{align*}
\sigma_{zz}^p, \text{MPa} & \quad \text{MPa} \\
\sigma_{zz}^{gr}, \text{MPa} & \quad \text{MPa}
\end{align*}
\]

*Figure 2. Changes in longitudinal stresses of underground structure (a) and soil (b) over time. 1 – z = 0; 2 – z = 5 m; 3 – z = 10 m; 4 – z = 15 m.*

Figure 3 shows the changes in the particle velocity of the pipe, similar to the changes in stress (Figure 2) of the pipeline and soil overtime at the same points as in Figure 2. Here, there is an increase in the particle velocity of the pipeline to level the velocities.

If we compare these results with the results obtained in [1], we conclude that the "entrainment" of energy from soil to the pipeline occurs to the leveling of particle velocities of the pipeline and soil. It follows that if the plastic properties of soil are taken into account, at least for the contact layer, the stress state of the pipeline decreases due to a decrease in the particle velocity, when compared with the result in Figure 2. Thus, for the most unfavorable case, the results show that stresses compared with a given load increase approximately by \( m = c_p \rho_p^0 / c_t^{gr} \rho_t^{gr} \) times and do not depend on the amplitude of a given load. In this case, this value is 21.667.

\[
\begin{align*}
U^p_z, \text{m/s} & \quad \text{m/s} \\
U^{gr}_z, \text{m/s} & \quad \text{m/s}
\end{align*}
\]

*Figure 3. Change in the particle velocity of an underground structure (a) and soil (b) over time. 1 – z = 0; 2 – z = 5 m; 3 – z = 10 m; 4 – z = 15 m.*

The change in relative displacement overtime at fixed points on the contact surface of the pipeline with soil is shown in Figure 4. Here, significant strains and displacements of soil particles are predominant. Figure 5 shows the dependence of shear stress on the relative displacement at the point \( z = 5 \) m on the contact surface. This dependence before the arrival of waves in soil within the given...
scale is insignificant. Further, with the arrival of waves in soil, the shear stress versus relative displacement diagram describes the OA curve, which corresponds to time intervals from 0.005 to 0.01 sec. At the moment the sign of relative displacement changes, the shear stress changes its direction. This is the beginning of a new cycle of interaction, where $\tau$ instantly descends along the AB, and changes along the BC trajectory in the intervals of 0.01-0.02 seconds. In this case, the value of $\alpha$ decreases, it characterizes the degree of change in the interaction coefficient. The next cycle is carried out similarly (CD and DE trajectories).

Thus, the results presented show numerical results of solving the problems of longitudinal wave propagation in an elastic underground pipeline with active and passive external friction. Calculation results revealed the attenuation of wave propagation in the pipeline with time and distance at passive external friction and an increase in propagation at active friction. The mechanism of stress decreasing or increasing in the pipeline is explained.

4. Conclusions

1. A one-dimensional approach to underground pipelines is based on the hypothesis of flat sections and linear strain in a structure. As the calculation results show, for the applied initial data the hypothesis of flat sections is fulfilled. A slight violation was observed when a friction force acts against the underground pipeline motion. When the friction force acts as an active one, such a violation was practically not observed. The solutions obtained in a two-dimensional statement, with sufficient accuracy, coincide with one-dimensional solutions.

2. Based on the analysis of the results obtained, it was found that an increase in the maximum values of longitudinal stress in the pipeline interacting with soil under dynamic loading occurs a multiple of a number $m$ times ($m = c_1^p \rho_0^p / c_1^g \rho_0^g$, $c_1^p$, $\rho_0^p$ and $c_1^g$, $\rho_0^g$ - are the velocity of longitudinal wave propagation and the initial density of the pipeline and soil).

3. The force of interaction (the friction force) on the pipeline-soil contact surface must be determined from the "complete" law of interaction, including the pre-limit and limit stages of interaction. The manifestation of the law of interaction in one form or another occurs in the process of solving the problem; it depends on the values of wave parameters in the pipeline and soil and on the physical and mechanical characteristics of the soil.
References

[1] Sultanov K S, Kumakov J X, Loginov P V and Rikhsieva B B 2020 Strength of underground pipelines under seismic effects *Mag Civ Eng* **93**(1) pp 97–120

[2] Muravyeva L and Vatin N 2014 Risk Assessment for a Main Pipeline under Severe Soil Conditions on Exposure to Seismic Forces *Appl Mech Mater* **635–637** pp 468-471

[3] Muravyeva L and Vatin N 2014 The Safety Estimation of the Marine Pipeline *Appl Mech Mater* **633–634** pp 958-961

[4] Muravyeva L and Vatin N 2014 Application of the Risk Theory to Management Reliability of the Pipeline *Appl Mech Mater* **635–637** pp 434-438

[5] Lalin V V and Kushova D A 2014 New Results in Dynamics Stability Problems of Elastic Rods *Appl Mech Mater* **617** pp 181-186

[6] Jung J K, O’Rourke T D and Argyrou C 2016 Multi-directional force–displacement response of underground pipe in sand *Can Geotech J* **53**(11) pp 1763–1781

[7] Gao F P, Wang N, Li J and Han X T 2016 Pipe–soil interaction model for current-induced pipeline instability on a sloping sandy seabed *Can Geotech J* **53**(11) pp 1822–1830

[8] Wijewickreme D, Monroy M, Honegger D G and Nyman D J 2017 Soil restraints on buried pipelines subjected to reverse-fault displacement *Can Geotech J* **54**(10) pp 1472–1481

[9] Muravieva L and Bashirzade S 2015 Behaviour underground pipelines laid in saturated soil *Rus J Transp Eng* DOI:10.15862/01TS315

[10] Liyanage K, Dhar A S 2018 Stresses in cast iron water mains subjected to non-uniform bedding and localised concentrated forces *Int J Geotech Eng* **12**(4) pp 368–376

[11] Smith A, Dixon N and Fowmes G 2017 Monitoring buried pipe deformation using acoustic emission: quantification of attenuation *Int J Geotech Eng* **11**(4) pp 418–430

[12] Meyer V, Langford T and White D J 2016 Physical modelling of pipe embedment and equalisation in clay *Géotechnique* **66**(7) pp 602–609

[13] Feng W, Huang R, Liu J, Xu X and Luo M 2015 Large-scale field trial to explore landslide and pipeline interaction *Soils Found* **55**(6) pp 1466–1473

[14] Matsuhashi M, Tsushima I, Fukatani W and Yokota T 2014 Damage to sewage systems caused by the Great East Japan Earthquake, and governmental policy *Soils Found* **54**(4) pp 902–909

[15] Lam S Y, Haigh S K and Bolton M D 2014 Understanding ground deformation mechanisms for multi-propped excavation in soft clay *Soils Found* **54**(3) pp 296–312

[16] Zhang Z and Zhang M 2013 Mechanical effects of tunneling on adjacent pipelines based on Galerkin solution and layered transfer matrix solution *Soils Found* **53**(4) pp 557–568

[17] Akimov M P, Mordovskoy S D and Starostin N P 2014 Calculating thermal insulation thickness and embedment depth of underground heat supply pipeline for permafrost soils *Mag Civ Eng* **46**(2) pp 14–23

[18] Kalugina J A, Keck D and Pronozin Y A 2017 Determination of soil deformation moduli after National Building Codes of Russia and Germany *Mag Civ Eng* **75**(7) pp 139-149

[19] Samarlin O D 2018 The temperature waves motion in hollow thick-walled cylinder *Mag Civ Eng* **78**(2) pp 161-168

[20] Loktionova E A and Miftakhova D R 2017 Fluid filtration in the clogged pressure pipelines *Mag Civ Eng* **79**(8) pp 214–224

[21] Singh M., Viladkar M N and Samadhiya N K 2016 Seismic response of metro underground tunnels *Int J Geotech Eng* doi:10.1080/19386362.2016.1201881

[22] Chernyshova N V, Kolosova G S and Rozin L A 2016 Combined Method of 3d Analysis for Underground Structures in View of Surrounding Infinite Homogeneous and Inhomogeneous Medium *Mag Civ Eng* **62**(2) pp 83–91

[23] Lanzano G, Bilotta E, Russo G and Silvestri F 2015 Experimental and numerical study on circular tunnels under seismic loading *European J Environm Civ Eng* **19**(5) pp 539–563

[24] Chang D W, Cheng S H and Wang Y L 2014 One-dimensional wave equation analyses for pile responses subjected to seismic horizontal ground motions *Soils Found* **54**(3) pp 313–328
[25] Mirsaidov M and Troyanovskii I E 1976 Forced axisymmetric oscillations of a viscoelastic cylindrical shell Polym Mech 11 pp 953-955
[26] Mirsaidov M 2019 An account of the foundation in assessment of earth structure dynamics Web Conf 97 04015 doi:10.1051/e3sconf/20199704015
[27] Mirsaidov M M, Abdikarimov R A and Khodzhaev D A 2019 Dynamics of a viscoelastic plate carrying concentrated mass with account of physical nonlinearity of material PNRPU Mech Bull 2 pp 143-155
[28] Sultanov K S 1998 A non-linear law of the deformation of soft soils J Appl Math Mech 62 pp 465-472
[29] Sultanov K S 2002 The attenuation of longitudinal waves in non-linear viscoelastic media J Appl Math Mech 66 pp 115-122
[30] Bakhodirov A A, Ismailova S I and Sultanov K S 2015 Dynamic deformation of the contact layer when there is shear interaction between a body and the soil J Appl Math Mech 79 pp 587-595
[31] Sultanov K S, Loginov P V, Ismoilova S I and Salikhova Z R 2019 Quasistaticity of the process of dynamic strain of soils Mag Civ Eng 85(1) pp 71–91
[32] Wilkins M L 1999 Computer Simulation of Dynamic Phenomena (Berlin, Heidelberg: Springer Berlin Heidelberg)