To the stability of the roadbed reinforced with gabions

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Abstract. Mountain and foothill regions of Uzbekistan in the winter-spring period are the subject to erosion as a result of precipitation and mudflows. This fact led to the beginning of their strengthening with gabion structures. Gabions also improve the operation of the entire structure under the dynamic influences’ action, including the seismic ones. In the work, research and testing in the laboratory and field conditions of the erected gabions effectiveness in road construction in the Parkent district of Tashkent region. The analysis of the gabion structures main structural parameters’ influence to strengthen the subgrade on their stability and deformability has been performed. Reinforcement of the road subgrade 4P187 “Kiziltof sh. - Parkent sh. (59 km) - Kamushkon” was performed with box-shaped gabion structures. This road stretch from the stake PK80 to PK90 was blurred as a result of mudflow. To strengthen this slope, a local program for its targeted strengthening has been approved. As the initial conditions for the considered site survey, the work to determine the physical and mechanical properties of soils lying in the road slope has been carried out.

Introduction
The soil intakes’ analysis was carried out by the specialists of the “Hydroingeo” Institute of the State Committee for Geology in the Republic of Uzbekistan. The coefficients of porosity, compressibility, macro-porosity, relative subsidence and the total strain modulus were determined. The soil sampling was carried out at the Parkentsay slope at the depths of 2m and 1.5m.

Soil filtration coefficients, soil porosity and its layers’ plasticity were also determined.

According to the data received from the Tashavtoyul Parkent branch, the real road located along the Parkentsoy River has the following characteristics: road category - III; the number of lanes - 2; strip width - 3.5m; roadside width - 2.0m. The first engineering-geological element includes gray, macro-porous loam, from wet to water saturated, light to heavy, from hard to soft plastic consistency, with thin sandy loam and sand layers.

The second engineering and geological element includes the macrofragmental soil represented by gravel from sedimentary rock fragments with sand-clay aggregate. According to the soils' particle size analysis data, the aggregate is sand-clay content, which in the total mass is 27.6-51.70%. The penetrated thickness of the element is up to 7.9 m. The rated resistance of coarse soil (R0) according to KMK 2.02.01-98 (SNiP RUz) is 3.5 kgf/cm².

The calculation method
We assume that vehicles moving along the road cause longitudinal, transverse, and surface waves in the subgrade (soil base) and in the road surface. Under the influence of these waves, the soil
foundation and pavement begin to move, bending vibrations of the pavement arise, the amplitudes of the vibrations can reach the values comparable to the pavement deflection under the influence of static loads from transport.

In accordance with the norms [3] when performing the strength calculations, the dynamic effect of loads on the road surface from the moving vehicles is taken into account by introducing an empirical coefficient of dynamism \( k_d = 1.3 \). The actual value of this coefficient depends on the specific conditions for the transport interaction with the road structure and may differ significantly from the accepted one.

We will solve the dynamic problem for an elastic half-plane with the uniform movement of a load with a given intensity along its surface using the numerical method. The dynamic equilibrium of a layered half-space cut from a subgrade under the action of the intensity load \( P(t) \), applied to it, is considered. In this case, we will assume that under the action of the car wheels (Figure 1), the road surface is deflected.

![Figure 1. Sagging in traffic](image)

Taking the sagging influence line in the form \( F(t) = \frac{1}{2} F_0 (1 - \cos \theta) \) it is possible to accept \( x = Vt, \frac{2\pi V}{l} = \theta \), where \( V \) is the speed of the car. In this case, the pulse wavelength under the car wheels is set according to [4], equal to the distance between the axles. The load speed can vary up to \( V=60 \text{ km/h} \) and more. Moreover, for one wheel at \( V=60 \text{ km/h} \) the load exposure time is \( t=0.072 \text{ sec} \), \( \theta = 87.22 \text{ rad/s} \) and accordingly for the regulatory load (NK-100) the exposure time from 4 wheels will be equal: \( t=0.288 \text{ sec} \).

The dynamic analysis is based on a finite element solution of the motion equation in the following form:

\[
M \ddot{Z}(t) + K(1 - R^*) \dot{Z}(t) = \bar{F}(t) - \bar{F}(t)
\]

where \( M, K \) – define the matrix of masses and rigidity of a finite element system, \( \dot{Z}(t), \bar{F}(t) \) - are the vectors of nodes and external forces displacement. The matrix \( G \) in the case of a horizontal or vertical boundary is a diagonal matrix, in the diagonal elements of which there are resistance coefficients corresponding to the degrees of freedom through which the standard viscous boundary passes.

In this case, the equation of state in the system of equations (1) is written according to the hereditary Boltzmann-Volterra theory:

\[
\dot{\sigma} = D(1 - R^*) \dot{\epsilon}
\]
where $R^*$ is the Volterra integral operator

$$
\int_0^t R(t - \tau) f(\tau) d\tau
$$

$R(t)$ is a core of heredity having a weak Abel-type singularity

$$
R(t - \tau) = \overline{\varepsilon} e^{-\beta(t-\tau)} (t - \tau)^{\alpha-1}, \quad 0 < \alpha < 1
$$

The obtained algebraic equations are a system of ordinary integral-differential equations. At $\overline{\varepsilon}^* = 0$ they are transformed into a system of differential equations. To solve these equations, we use the step-by-step method of direct integration, $\theta$ - Wilson’s method. According to [5], it is assumed that the acceleration and the load vector change for a time $t + \theta \Delta t$. The vectors of displacement, velocity, and acceleration are written as:

$$
\begin{align*}
Z_{t+\theta \Delta t} &= \hat{Z}_t + \theta \Delta t \ddot{Z}_t + \frac{\theta^2 \Delta t^2}{6} (\dddot{Z}_t + 2 \dddot{Z}_t), \quad \ddot{Z}_{t+\theta \Delta t} = \dddot{Z}_t + \frac{\theta \Delta t}{2} (\dddot{Z}_t + \dddot{Z}_t), \\
\dddot{Z}_{t+\theta \Delta t} &= \frac{6}{\theta^2 \Delta t^2} (\dddot{Z}_t + \dddot{Z}_t) - \frac{6}{\theta \Delta t} \dddot{Z}_t - 2 \dddot{Z}_t
\end{align*}
$$

Unconditional stability of the method is ensured when $\Theta > 1.37$. To find $\dddot{Z}_{t+\theta \Delta t}$ the equation of motion (1) is written for a moment in time $t + \theta \Delta t$:

$$
M \dddot{Z}_{t+\theta \Delta t} + K(\dddot{Z}_{t+\theta \Delta t} - \int_0^{\theta \Delta t} R(t, \theta \Delta t - \tau) \dddot{Z}(\tau) d\tau) = \dddot{P}_{t+\theta \Delta t} - \overline{\Gamma} \dddot{Z}_{t+\theta \Delta t}
$$

Using the transformation proposed in [6] for the improper integral of the weakly singular kernel of Rzhanitsyn-Koltunov, we write:

$$
\overline{\varepsilon} \int_0^{\theta \Delta t} e^{\beta(t, \theta \Delta t - \tau)} (t, \theta \Delta t - \tau)^{\alpha-1} \dddot{Z}(\tau) d\tau = \frac{\theta \Delta t}{\alpha} \dddot{Z}_{t+\Delta t} + \sum_{k=1}^{i+1} B_k e^{-\beta \Delta t} \dddot{Z}_{t+\Delta t}
$$

The final equation is obtained by substituting the dependence (4) into (5)

$$
K \dddot{Z}_{t+\Delta t} = \dddot{P}_{t+\Delta t}
$$

Let us consider a test case for testing the method. Let us give an example of the plate calculation, the calculation scheme of which is shown in Figure 2. A constant acceleration of magnitude is instantly applied to the plate $a$. 
Figure 2. Continuous Acceleration Plate

The displacements along the y axis of the 43rd node are given in Figure 3. The static displacements were equal to ϵ=0.000454m. In this case, the following rheological parameters are taken: \( \bar{\varepsilon} = 0,0,1, \beta = 0,05, \alpha = 0,25 \). And Fig. 4 also shows the displacements of the 3rd node region lower point, in the case of installing the viscous impulse neutralizers. This shows that they are capable of modelling the waves outflow to infinity.

Figure 3. The 43rd node move: 1) at \( \bar{\varepsilon} = 0 \), 2) at \( \bar{\varepsilon} = 0,1, \beta = 0,05, \alpha = 0,25 \)
**Figure 4.** 1) Moving the 3rd node with $\varepsilon = 0.1$, $\beta = 0.5$, $\alpha = 0.25$; 2) at the 43rd node at $\varepsilon = 0$

**Analysis and assessment of the gabions influence on the road deformability**

Now, when calculating the vibro-dynamic load, we will take the car location into account in two cases: when located at a certain distance from the road slope and closer to it. First, we evaluate the stress-strain state of the array without installing the gabions. According to the work [7,8], the following rheological parameters for the gravel-sandstone loam are $\varepsilon = 0.0674$, $\beta = 0.0000013$, $\alpha = 0.2$.

**Figure 5.** Cross section of a two-lane highway

1,2,3,4 - points where the displacements are determined when calculating the array

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-0.001 0 0.001 0.002 0.003 0.004 0.005 0.006 0.007 0.008 0.009 0.01
0 0.2 0.4 0.6 0.8 1.0 1.2 1.4 1.6 1.8 2.0

Moving, m

Time, s
```
Figure 6. The movements amplitude in time when the load at the road edge:
a) point 1, b) point 2, c) point 3, d) point 4 (blue line horizontal movements, red line vertical movements)

Figure 7 shows the calculation results taking into account the installed gabions.
Figure 7. The movements amplitude in time, with the load on the road side, taking into account the gabion reinforcement: a) point 1, b) point 2, c) point 3, d) point 4 (blue line horizontal movements, red line vertical movements)

A comparison of the results shows a difference in the reduction direction, both displacements and stresses when taking gabions into account on the road slope. Thus, the installation of gabions to strengthen the slope of the road is fully justified.

It is interesting to evaluate the stress-strain state of the array taking into account the effects of the seismic waves. To do this, let us suppose that a seismic impact passes to the left side of the array in the form of a damped sine wave: \( \sigma(t) = \rho V_p u e^{-\eta t} (\cos \theta t) \), where \( \rho, V_p, u, \eta, \theta \) define the density of the medium, the longitudinal waves’ velocity in the medium, the speed of the soil movement on the verge, the attenuation coefficient of the waves in the medium, the exposure frequency. We set all the external action parameters in the form: \( \rho = 1810 \frac{kg}{m^3} \), \( V_p = 170 \frac{m}{s} \), and \( u = 3 \frac{m}{s} \), \( \eta = 0.8 \), \( \theta = 31.4 \frac{rad}{s} \). Figure 8 shows the calculation result for the region without taking into account the gabion structure, and Figure 9 shows the calculation result taking into account the gabion design.

Figure 8 Amplitude of the displacements in time during seismic impact: point 1 - blue line vertical movements, red line horizontal movements
Figure 9 Amplitude of the displacements in time during seismic impact: point 1 - blue line of vertical movement, red horizontal line of movement

The experimental studies were carried out on the considered road section with the reinforced gabions slopes. The single-component seismic receivers OSP-2M were used. As a result of the experiments, it was determined that the time during which a passing car affects the soil oscillations on the slope no more than for 25 seconds. The number of oscillation measurements per second was about 240. Oscillations at all 4 levels of the gabion structure are close among themselves and differ from the calculated ones not exceeding 15%.

Discussions and Results
As a result of the study, the numerical modeling taking into account the structure’s interaction with the base under seismic effects has been carried out; a test procedure to study the interaction of various types of gabion structures and incoherent soils has been developed; the analysis of the gabion structures’ main structural parameters influence on the reinforcement, stability and deformability of the subgrade has been performed; the effectiveness of the erected gabions in the Parkent district of Tashkent region has been tested in laboratory and real field conditions of the road construction.

Based on the experiments’ results, the currently existing standard solutions for strengthening the subgrade, drainage devices and other structures of roads and overpasses have been improved and expanded. The design and technological solutions for the individual design of road and bridge structures using the gabion structures on weak soil foundations have been developed.

Summary
Regulatory and working documentation for the widespread use of proposals and technical solutions for the existing and newly built roads’ reconstruction has been developed.

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