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Practical experience in the construction of roads in difficult soil conditions

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Abstract: One of the most important and urgent problems is constructing roads in difficult soil conditions, ensuring their strength, reliability, and normal operation. To create an efficient and competitive transport infrastructure in Kazakhstan, the State Programme of Infrastructural Development "Nurly Zhol" for 2020–2025 was developed. Its main objectives are to improve the technological, scientific, and methodological base, provide resources, and to attract "Big Transit". The paper presents the details of the survey carried out in one of road construction areas. Irrigation canals and periodic and permanent watercourses represent the hydrographic network of the construction site. The analysis of these features and field tests were included in the research. Stamp tests were performed to analyse mechanical properties of embankment soil to provide more reliable information on the mechanical properties of the soil. Structural and technological solutions were adopted based on the field tests and surveys of hydrological conditions. A numerical simulation was used to determine the stability of the road embankment, the results of which showed maximum deformations of 4.5 mm during the operation of road transport. Geosynthetic material was used to reinforce the subgrade. The results of the study have shown that the analysis of factors affecting the stability of engineering structures on difficult soil conditions helps to achieve some improvement.

Keywords: field tests, hydrological condition, road, subsidence soil, waterway

INTRODUCTION

Geotechnical designing in difficult soil conditions must be carried out considering soil specific properties. When moisture content rises above a certain level, soil conditions change rapidly leading to deformations under the pressure of external loads and own weight of soil (Houston, Houston and Lawrence, 2002).

In the geotechnical design of structures on collapsing soils, one should consider the possibility of increasing their moisture content due to (NTP RK, 2011):

– explicit waterlogging due to water soaking the soil from external sources above it and/or from below when the groundwater table rises;
– implicit waterlogging due to gradual accumulation of moisture in the soil from surface water infiltration and screening of the surface in built-up areas;
– when designing foundations based on collapsing soils, the moisture level should be taken into account. Saturation ratio $S_r = \frac{w\rho_s}{e\rho_w}$, where w = natural soil moisture content, ρ_s = density of soil particles (t·m^{-3}), ρ_w = water density, taken as 1 Mg·m^{-3}, e = void ratio. The water saturation ratio is one of the main characteristics of loessial soils. If $S_r > 0.8$, soil is considered collapsible. If it is impossible to submerge foundations, the established value of moisture W_{eq} is taken equal to the natural moisture W, if $W \geq W_p$ and moisture at the rolling border, if $W < W_p$.

Considering the life cycle, the goal of road design should be to adapt design and technological solutions to the conditions and ensure the safety of these solutions, or reduce the possible negative impact of the road on its location.

Protective measures and structures should be designed based on the following conditions:
- provision of safe and uninterrupted movement of vehicles, and
- preservation of the required durability and stability of structural elements when exposed to external loads and natural climatic factors.

Today, different methods of eliminating collapsible properties of soils are applied in the world, e.g. compaction or the development of soil cushions (Niu *et al.*, 2021). To consolidate the collapsible soil, silicification (Lv *et al.*, 2014) or thermal firing (Bellil, Abbeche and Bahloul, 2018), or fibre reinforcement (Ayeldeen, Azzam and Arab, 2022) may be used as relevant methods. Ground cushions create a layer of non-collapsible soil in the foundation base. Another promising practice is the construction of fixed soil columns and piles in weak dusty-clayey soils with the combination of jetting and boring or jetting technology and the immersion of pre-reinforced concrete elements (Hameedi, Fattah and Abd Al-Kaream, 2021). Moreover, one of the effective methods of designing structures on collapsible soil is the reinforcement of foundation (Callai, Tataranni and Sangiorgi, 2021). However, all these methods fail to give a positive result if the quality of engineering and hydrological survey of the project site is poor.

STUDY MATERIALS AND METHODS

CASE STUDY DESCRIPTION

The road under the study passes through the Almaty region, Kazakhstan. The "zero pickets" is located on the western side of the Almaty city. The end of the road is on the eastern side of Almaty. The road will finally have from 4 to 6 lanes and is designed for convey traffic of up to 150 km·h^{-1} (KGS, LTD, 2020a). The purpose of building the road is to reduce traffic in the city and significantly improve the ecological situation in the region.

The reduced traffic taken away from some streets of the city will free it from traffic jams at the entrance to Almaty. The reduced number of trucks may translate into fewer traffic accidents.

In hydrogeological terms, the southern part of the piedmont plain territory, which includes the river outflow cones, is a zone of surface runoff infiltration (from river channels, irrigation canals) and groundwater formation.

Ground water in the watershed areas lies mainly at more than 3.0–5.0 m (KGS, LTD, 2020b). It should be noted that basically all watercourses that cross the route are regulated by dams and the flow is redistributed to a large extent. Field surveys were conducted at the sites: P 57+50 dry irrigation ditch; P 65+63 dry spreading ravine; P 189+90 irrigation canal in a natural channel.

The engineering-geological characteristics for artificial structures are as follows:
- route section: soil and vegetation layer of 0.2 m; below there are loams of different consistency, from hard to fluid of up to 13.0 m, as well as loam hard of 6.0 m; below there are layers of gravel-sand from 0.2 to 1.0 m;
- pipeline through the route: a soil and vegetation layer of 0.2 m; below, there is hard brown loam of up to 16.0 m, underlain by flowing loam of up to 16.2 m; and the amplitude of fluctuations 1.0 ±1.5 m;
- bridge crossing over the river: a soil and vegetation layer of 0.2 m; on the sides of the river, there are hard subsidence loams and semi-solid loams of up to 9.0 m; in the channel under flow-plastic loams, there are gravel and water-saturated sands of 3.8–9.4 m; below the depth of 22.5–28.2 m, lie tight-plastic loams; the underlying stratum of the section consists of coarse water-saturated grey sands with an uncovered thickness of up to 8.5 m; and the amplitude of fluctuations 1.0 ±1.5 m;
- bridge: a soil and vegetation layer of 0.2 m; below, it is represented by hard brown loam of up to 12.3 m, underlain by soft-plastic loam of up to 5.8 m replaced by tight-plastic loam; at the depth of 23.5 m, there is a gravel soil of grey water saturated with an uncovered thickness of up to 6.5 m.

The starting point of the picket P 0+00 starts from the settlement of Kyrgauyldy, and the route ends in the territory of the Karasai district, near the settlement of Isayevo, P 240+00. It is divided into three sections: P 0+00 – P 8+38; P 9+00 – P 45+00; P 45+00 – P 240+00. The climate of the area is sharp continental. The climate in the area is determined by latitude and the presence of orographic elements on its surface.

Indicators of physical and mechanical properties, material composition, and salinity of the variety of engineering-geological elements (EGE) of soil, selected according to the nomenclature (GOST, 2020b) were determined by laboratory methods (KGS, LTD, 2020a).

For P 0+00 – P 8+38:
- EGE-1 – topsoil layer;
- EGE-2 – asphalt-concrete (road pavement);
- EGE-3, 3a – bulk soil;
- EGE-4 – lean clay brown loessial light very stiff subsiding; according to compression tests, loam exhibits collapsible properties from additional loads; the initial collapsible pressure is 0.495 kg·cm^{-2}; the coefficient of relative collapsible at a specific pressure of 0.5 kg·cm^{-2} is 0.008–0.011; at a specific pressure of 1.0 kg·cm^{-2} is 0.012–0.020; at a specific pressure of 2.0 kg·cm^{-2} is 0.017–0.028; at a specific pressure of 3.0 kg·cm^{-2} is 0.021–0.028; engineering-geological conditions in terms of collapsible refer to type I (first).
- EGE-5 – lean clay brown loessial light very stiff non-subsiding;
- EGE-6 – lean clay brown loessial light soft non-subsiding.

For P 9+00 – P 45+00:
- EGE-1 – light hard, brown loam; according to compression tests, loams show collapsible properties from additional loads; the initial collapsible pressure is 0.1 kg·cm^{-2}; the coefficient of relative collapsible at a specific pressure of 0.5 kg·cm^{-2} is 0.055–0.081; at a specific pressure of 1.0 kg·cm^{-2} is 0.041–0.130; at a specific pressure of 2.0 kg·cm^{-2} is 0.045–0.151; at a specific pressure of 3.0 kg·cm^{-2} is 0.018–0.164; engineering and geological conditions in terms of collapsible are of the type I (first).
- EGE-2 – light tight plastic brown loam;
- EGE-3 – light soft plastic loam, brown;
- EGE-4 – coarse sand, brown, low-moisture;
- EGE-5 – pebble soil with sand.

For P 45+00 – P 240+00:
- EGE-1 – topsoil layer;
- EGE-2 – asphalt-concrete (road pavement);

– EGE-3 – bulk soil;
– EGE-4 – lean clay brown loessial light very stiff subsiding; according to compression tests, lean clays show subsiding properties from additional loadings; the initial subsiding pressure is 0.050 kg·cm^{-2}; coefficient of a relative subsiding with the specific pressure of 0.5 kg·cm^{-2} – 0.012–0.056; with the specific pressure of 1.0 kg·cm^{-2} – 0.025–0.095; with the specific pressure of 2.0 kg·cm^{-2} – 0.030-0.112; with the specific pressure of 3.0 kg·cm^{-2} – 0.058–0.132.
– EGE-5 – lean clay brown light dusty stiff non-subsiding;
– EGE-6 – lean clay brown light dusty firm consistency;
– EGE-7 – lean clay brown light soft consistency;
– EGE-8 – lean clay brown light very soft consistency;
– EGE-9 – lean clay brown light very soft consistency;
– EGE-10 – silty clay brown-grey very stiff;
– EGE-11 – sand brown average density average fineness water inundated;
– EGE-12 – sand grey average density cobble water-inundated;
– EGE-13 – gravel sand with sandy filling water inundated;
– EGE-14 – gravel soil with sandy filling slightly wet.

The standard values of physical and mechanical indicators of strength and deformation of main EGEs are presented in Table 1 (for P 0+00 – P 8+38), Table 2 (for P 9+00 – P 45+00), and Table 3 (for P 45+00 – P 240+00).

Table 1. Physical and mechanical properties of soil for P 0+00 – P 8+38

Parameter	Engineering and geological elements		
	EGE-4	EGE-5	EGE-6
Natural moisture content, %	13.5	18.8	22.0
Plasticity index, %	9.6	9.5	8.4
Index of moisture content, %	<0	0.15	0.60
Soil particles density, kg·m^{-3}	2700	2700	2700
Soil density, kg·m^{-3}	1990	2140	2000
Dry soil density, kg·m^{-3}	1750	1800	1640
Void ratio	0.754	0.502	0.653
Liquid limit, %	25.7	26.9	25.4
Plastic limit, %	16.3	17.4	17.0
Soil resistance, kPa	379	–	98

Explanations: EGE-4 = lean clay brown Loessial light very stiff subsiding, EGE-5 = lean clay brown loessial light very stiff non-subsiding, EGE-6 = lean clay brown Loessial light soft non-subsiding.
Source: KGS, LTD (2020a).

Table 2. Physical and mechanical properties of soil for P 9+00 – P 45+00

Parameter	Engineering and geological elements				
	EGE-1	EGE-2	EGE-3	EGE-4	EGE-5
Natural moisture content, %	11.8	19.3	21.1	–	–
Plasticity index, %	9.0	11.3	9.8	–	–
Index of moisture content, %	<0	0.38	0.53	–	–
Soil particles density, kg·m^{-3}	2700	2710	2710	–	–
Soil density, kg·m^{-3}	1830	1880	1930	1700	1750
Dry soil density, kg·m^{-3}	1650	1580	1610	–	–
Void ratio	0.64	0.718	0.686	–	–
Liquid limit, %	24.1	26.4	24.7	–	–
Plastic limit, %	14.1	15.1	14.9	–	–
Soil resistance, kPa	355	180	100	500	600

Explanations: EGE-1 = light hard, brown loam, EGE-2 = light tight plastic brown loam, EGE-3 = light soft plastic loam, brown, EGE-4 = coarse sand, brown, low-moisture, EGE-5 = pebble soil with sand.
Source: KGS, LTD (2020a).

Table 3. Physical and mechanical properties of the soil ground for P 45+00 – P 240+00

Parameter	Engineering and geological elements						
	EGE-4	EGE-5	EGE-6	EGE-7	EGE-8	EGE-9	EGE-10
Natural moisture content, %	12.8	21.8	18.9	22.4	26.8	27.4	11.3
Moisture content at the border:							
– flow	26.9	31.4	25.0	25.0	28.5	24.5	24.6
– rolling out, %	18.1	21.5	15.1	17.3	19.2	17.0	18.7
Plasticity index, %	8.8	9.9	9.9	7.7	9.3	7.5	5.9
Index of liquidity, %	–0.60	0.03	0.38	0.67	0.82	1.38	1.38
Soil stability	1.89	1.94	1.88	1.95	1.91	1.82	1.82
Soil particles density, $kg \cdot m^{-3}$	2710	2720	2720	2710	2710	2710	1770
Dry soil density, $kg \cdot m^{-3}$	1680	1590	1580	1590	1510	1430	1590
Void ratio	0.62	0.71	0.72	0.70	0.79	–	0.70
Specific cohesion, kPa	34/25[1)]	28/22[1)]	25/21[1)]	22	–	–	14/22[1)]
The angle of internal friction	25/19[1)]	23/18[1)]	21/18[1)]	18	–	–	26/22[1)]
Modulus of deformation, MPa	23.8/17.0[1)]	19.0/14.0[1)]	15.5/13.5[1)]	14.5	–	–	13.0
Design strength, kPa	362	341	160	–	–	–	294

[1)] Characteristics are given for soil at a water-saturated state.
Explanation: EGE-4 = lean clay brown loessial light very stiff subsiding, EGE-5 = lean clay brown light dusty stiff non-subsiding, EGE-6 = lean clay brown light dusty firm consistency, EGE-7 = lean clay brown light soft consistency, EGE-8 = lean clay brown light very soft consistency, EGE-9 = lean clay brown light very soft consistency, EGE-10 = silty clay brown-grey very stiff.
Source: KGS, LTD (2020a).

During the construction of the technological embankment, soil compaction was carried out followed by subsequent tests. Soil settlement was possible in some areas because EGE-4 soils had settled properties.

SITE TESTING PROCEDURES

The study of the soil bearing capacity under the overpass was carried out at 2 points of the stamp tests (for P 9+00 – P 45+00). The tests were conducted at an air temperature of +16°C (GOST, 2020a). The tests were carried out with the help of a rotary arm punching device acting according to the principle of a rocker arm of scales (Tulebekova *et al.*, 2020). For P 45+00 – P 240+00, the studying of bearing capacity of the embankment and excavation of the base soil at P 101+10, P 102+20, P 102+60 was carried out (KGS, LTD, 2020). The study of the soil bearing capacity was carried out at 6 points of stamp testing. The study was conducted at 3 points in dry conditions and at 3 points in wet conditions. The tests were conducted at an air temperature of +8°C.

Static tests (SP RK, 2013a) with indentation loads were carried out to determine the bearing capacity and deformability of the pile on the ground and to determine the dependence of pile movement in the ground under the load. Static load tests (SLT) were carried out on the bored piles No 14 and No 16 as presented in Figure 1.

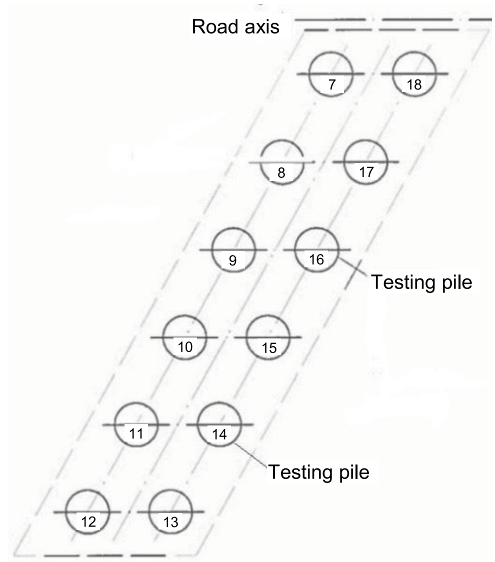

Fig. 1. Location of piles tested by static load; source: own elaboration

To ensure the application of test loads to the pile head, a loading device calibrated to the specified force and time modes was used. The loads on the pile tested were transmitted centrally and coaxially. To perform static tests, an individually designed stand was used. It included a thrust structure to absorb reactive forces. In the applied test bench design, loads on the pile head were created by the pressure in the hydraulic circuit of the jacking system. The loader design consists of an anchoring unit. The reference system consisted of two crossbars in the form of a rectangular steel section, the ends of which were welded to the sheet pile (Photo 1).

Photo 1. General view of the static load tests (photo.: *A. Zhussupbekov*)

The crossbars were additionally reinforced with inclined struts made of profiled reinforcing steel to avoid deformations under load. The fixed cross piles were located more than 2.5 m away from the pile to be tested. Standard clamps were used to fix recording devices to the crossbars in the reference system. The fixing of the deflectometer was made symmetrically to fix the pile under the test load. The calculated value of the load applied on the pile head was determined by the design of the structure. It had to comply with the provisions of GOST 5686 (GOST, 2012) and SP RK 3.03-112-2013 (SP RK, 2013b). The load safety factor for the test was 1.5.

The maximum penetration load on the tested pile was 3540 kN. The tested pile was loaded uniformly without shocks, in steps not exceeding 1/10 of its presumed bearing capacity. The force produced by the jacks in the first load step was 353 kN, and each successive load step had an increment of 353 kN.

At each stage of pile loading, readings were taken on all instruments at intervals of 30 min until the pile settlement decays and reached the so-called conditional stabilization (Zhussupbekov *et al.*, 2022).

According to the pile design, the rate of its settlement in the soil of not more than 0.1 mm during the last 60 min of observations was taken as the conditional stabilization of the pile. The first reading was made immediately after the first loading step, then the readings were taken every 30 min up to four times until the deformation at the first stage of loading. The holding time of each stage, from the first to the ninth, under indentation loads was 120 min for each stage (Zhussupbekov *et al.*, 2020).

In the last tenth stage, the holding time was 270 min. Once the pile deformation stabilised from the last step of the load increase, the decision was made to unload the tested pile. The pile was unloaded in steps equal to the double value of the loading steps, with each stage lasting for 15 min (Tulebekova *et al.*, 2016). After full unloading (up to zero), pile displacement observations took 60 min with readings taken from the instruments every 15 min. Design loading and unloading schemes are shown in Tables 4 and 5.

Table 4. Design loading schemes

Steps loading	Force by the jack, kN
1	353
2	706
3	1,059
4	1,412
5	1,765
6	2,128
7	2,471
8	2,834
9	3,187
10	3,540

Source: own elaboration.

Table 5. Design unloading schemes

Steps of unloading	Force by the jack, kN
1	2,834
2	2,128
3	1,412
4	706
5	0

Source: own elaboration.

CONSTRUCTION SOLUTION FOR EMBANKMENT

The pavement design foreseen the most suitable materials based on local resources and organization of work, appropriate dimensioning of individual layers, and their paving depth. All intersections of the designed roadway had an asphalt concrete surface with a gravel base. The thickness of asphalt concrete ranged from 0.11 to 0.38 m. The type of pavement is shown in Figure 2.

To determine the stability of the road embankment, modelling was performed in the GTS NX MIDAS software package. The road embankment and foundations were modelled as a continuous medium. The general design of the embankment of 33 m in length and 6 m in height consists of local compacted loam and a one-meter soil cushion. The soil cushion material is gravel with sandy aggregate. The gravel layer should exceed 10% of 80 to 150 mm fractions. Laying and compacting of the cushioned ground should be carried out in layers of not more than 300 mm. To achieve the required deformation modulus,

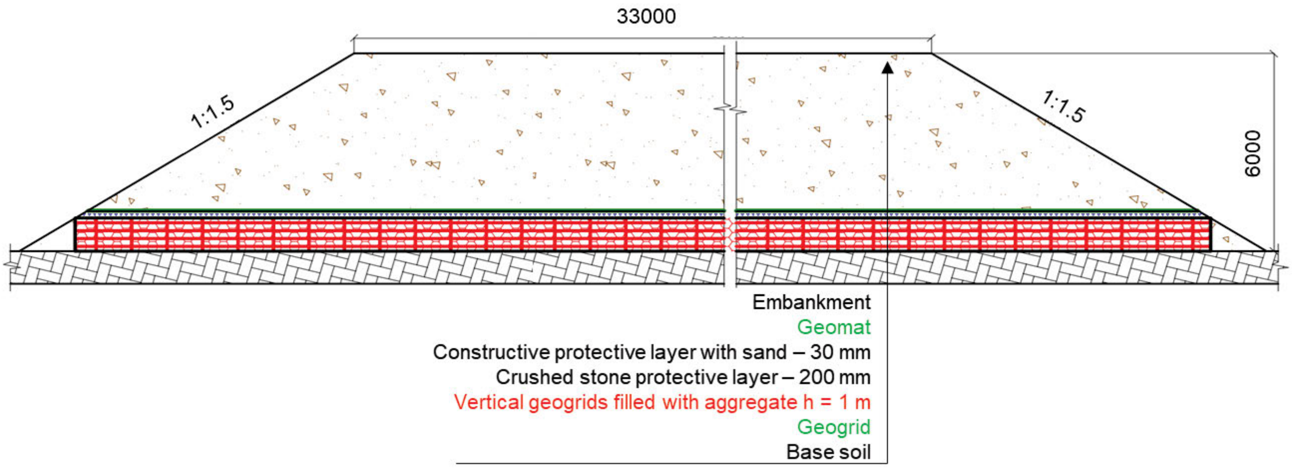

Fig. 2. Type of pavement; source: own elaboration

a vertical geogrid, which is placed in the body of the soil cushion, is used as part of the soil cushion.

The linear elastic and Mohr–Coulomb models were used to calculate the road embankment and subgrade composed of dispersed soils. The calculated 3D model is presented in Figure 3.

The results show that for wet soil:
- the average modulus of structure deformation for 3 point tests is E_d = 7.24 MPa;
- the average modulus of structure elasticity for all 3 point tests is E_{el} = 35.96 MPa.

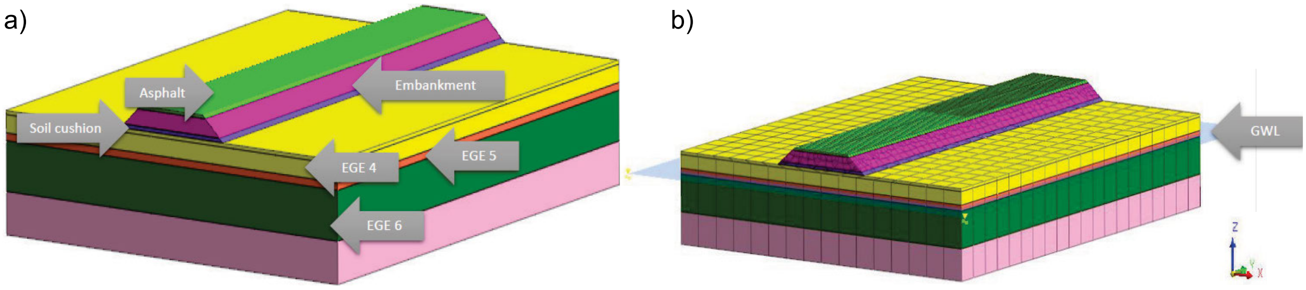

Fig. 3. Calculation model: a) with engineering-geological elements (EGE), b) with ground water level (GWL); EGE-4, EGE-5 and EGE 6 as in Tab. 3; source: own elaboration

An evenly distributed load of 5 kN·m^{-2} was set on the road embankment surface as the load from vehicles. The own weight of the soil mass was also considered. Volumetric elements were used to simulate the behaviour of the ground and massive structures, including 4-node tetrahedron, 8-node hexahedron, 10-node tetrahedron, 20-node hexahedron.

RESULTS AND DISCUSSION

Based on the stamp test results, the deformation characteristics of the soil are presented in Table 6 and Figure 4.

On dry soil: the average modulus of deformation (E_d) of the structure for 3 point tests is: E_d = 34.14 MPa; the average modulus of elasticity (E_{el}) of the structure for all 3 point tests is E_{el} = 57.65 MPa. According to the tests, the coefficient of soil layer compaction (K) is ≈0.99.

The shape of the graph for general deformations corresponds to the linear relationship between the increment of total deformations and the increase in the load on the base. There is no loss of bearing capacity.

According to the tests, the coefficient of soil layer compaction in the base fluctuates at K < 0.9. The pile static test (pile 14) showed that the pile deformation stabilised at the tenth load step with a maximum indentation load of 3540 kN, with an average settlement of about 3.8 mm as measured by the last averaged test instrument (Fig. 5a, b); for pile 16 (Fig. 5c, d) the tenth load step with a maximum indentation of 3540 kN, resulted in a stabilised pile deformation.

The pile deformation stabilization and the average settlement from the last average of the control gauge readings were about 4.05 mm. The results of the pile static test showed that the bearing capacity of the soil was sufficient for the maximum load.

The analysis of the calculation results by finite element method was performed according to (Fig. 6–7) and total embankment displacement presented in Figure 8:
- displacement components of the soil mass and structural elements;
- stress components in a continuous medium (for the conditions of spatial and axisymmetric problem and plane deformation);
- dimensions of the limit state zones (plasticity zones);
- internal forces in the structures interacting with the base.

Table 6. Characteristics of soil

Point	Modulus of deformation, MPa	Modulus of elasticity, MPa	Compression ratio
P 9+00 – P 45+00			
1	25.74	80.84	<0.95
2	39.25	105.41	0.96
P 45+00 – P 240+00			
1	46.74/10.75[1)]	39.65/39.65[1)]	0.99/0.94[1)]
2	30.64/5.54[1)]	37.08/37.08[1)]	0.99/0.9[1)]
3	25.05/5.42[1)]	41.18/31.15[1)]	0.99/0.9[1)]

[1)] Characteristics are given for soil at a water-saturated state.
Source: own study.

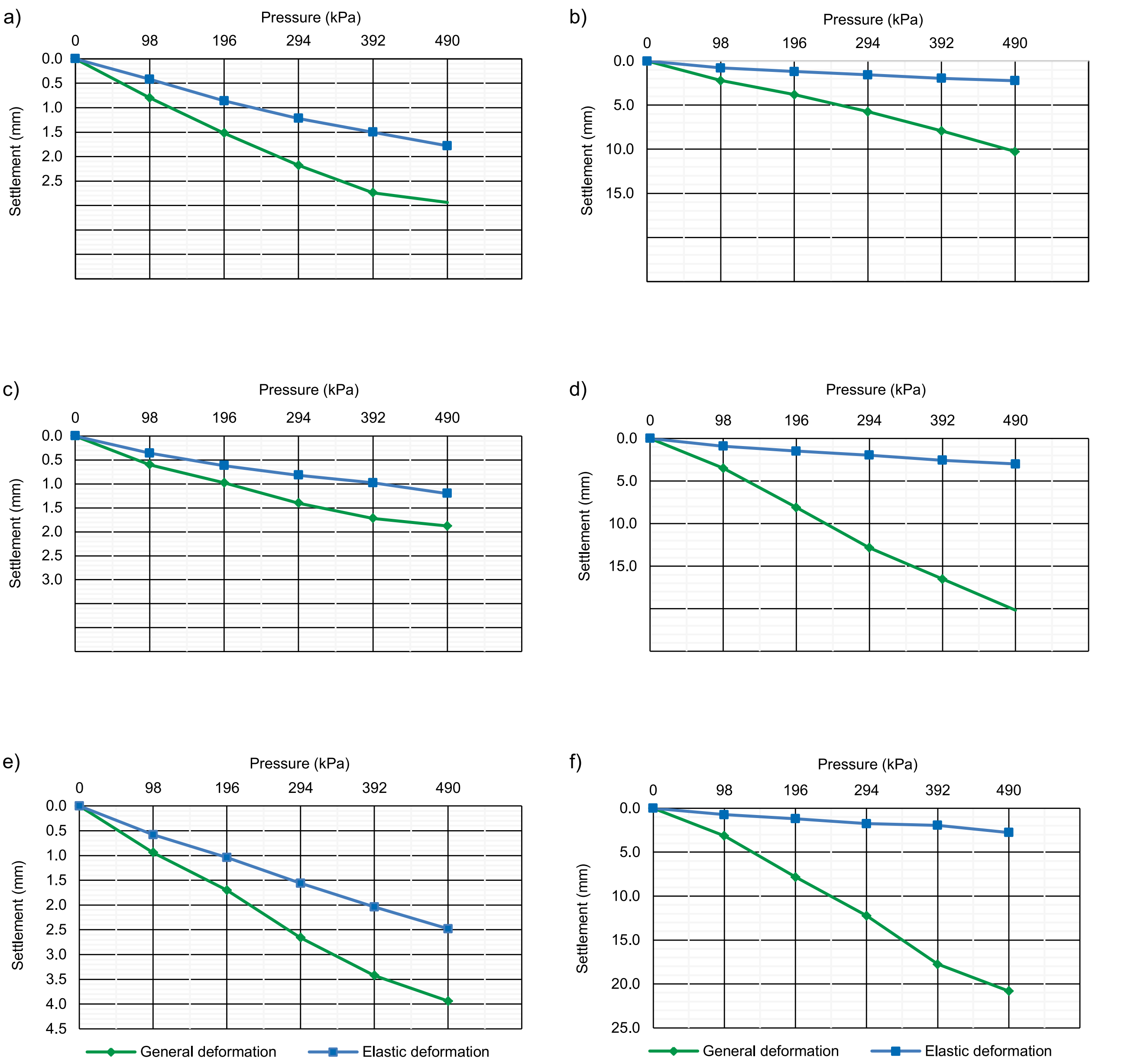

Fig. 4. Change of sample height and soaking: a) point 1 (dry soil), b) point 1.1 (wet soil), c) point 2 (dry soil), d) point 2.1 (wet soil), e) point 3 (dry soil), f) point 3.1 (dry soil); source: own elaboration

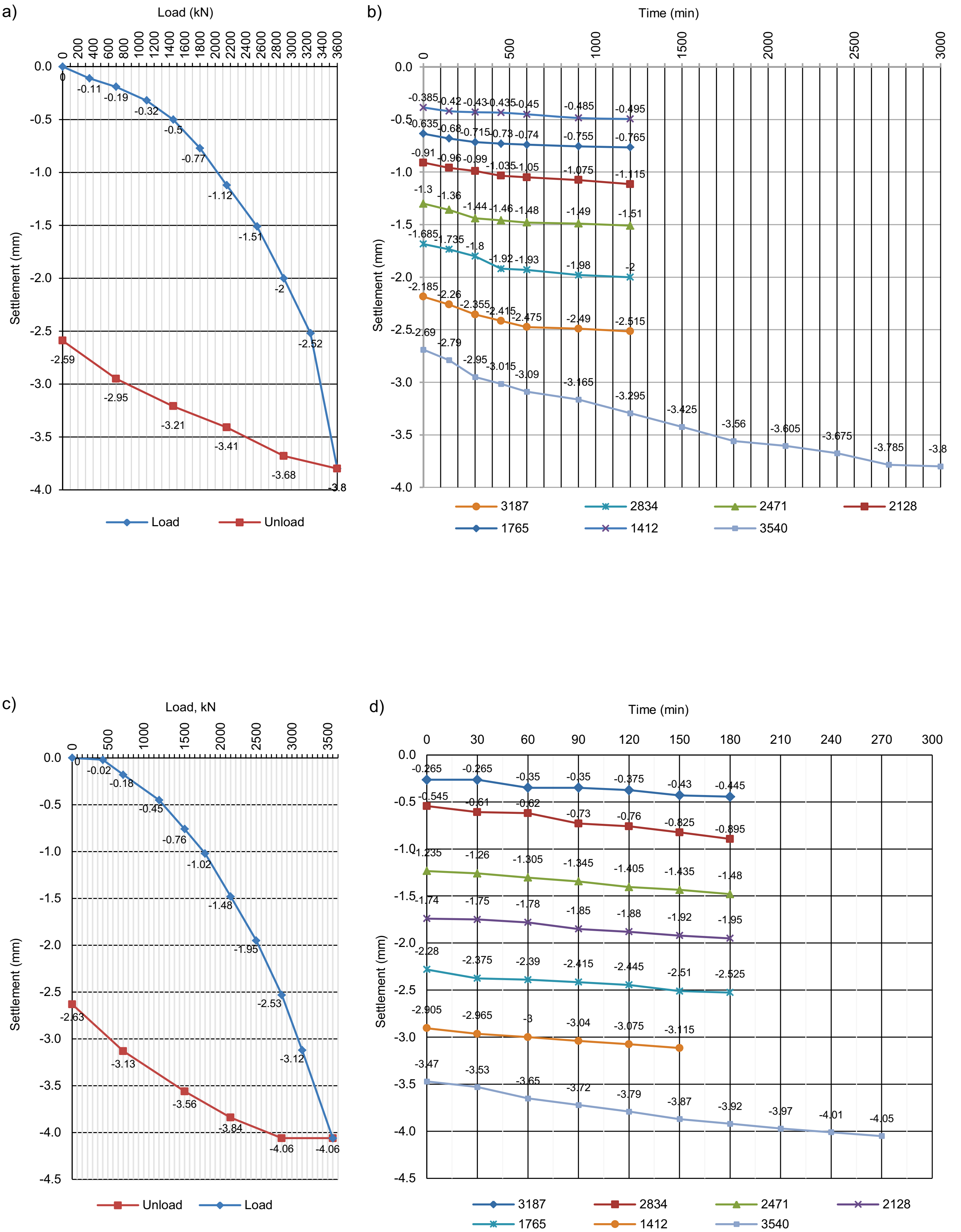

Fig. 5. Correlation of loading with settlement for pile 14: a) total settlement, b) by loading stages; for pile 16: c) total settlement, d) by loading stages; source: own study

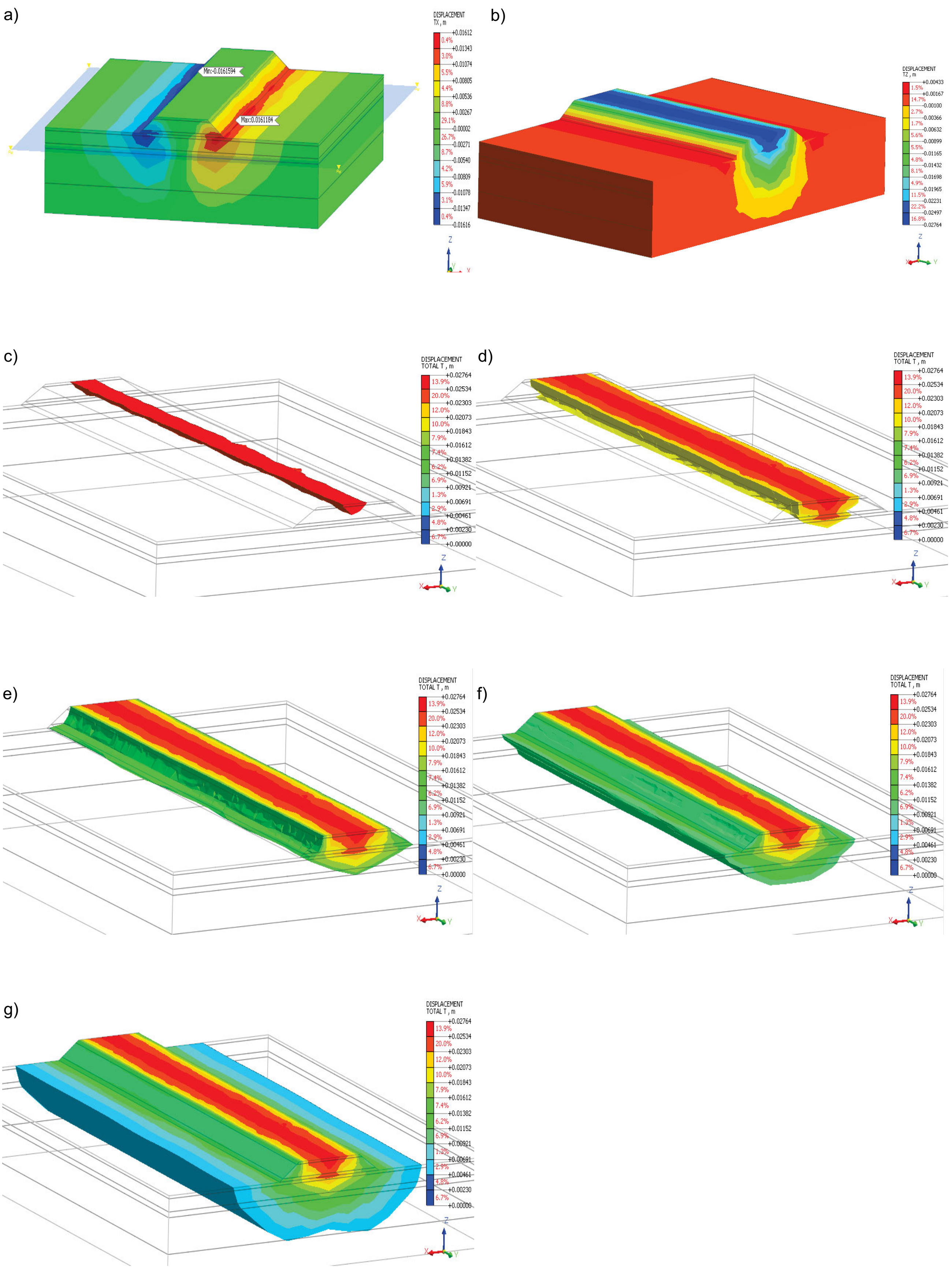

Fig. 6. Displacement of the embankment: a) *x*-axis, b) *y*-axis, c) at ≥26 mm, d) at ≥20 mm, e) at ≥15 mm, f) at ≥10 mm, g) at ≥5 mm; source: own study

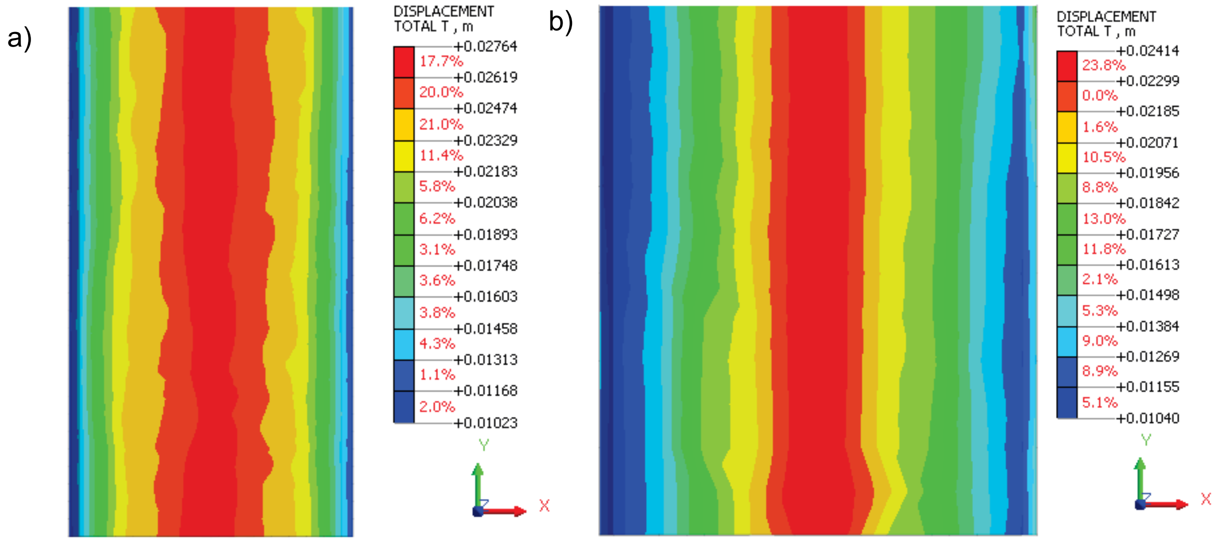

Fig. 7. General deformities: a) pavement, b) soil cushion; source: own study

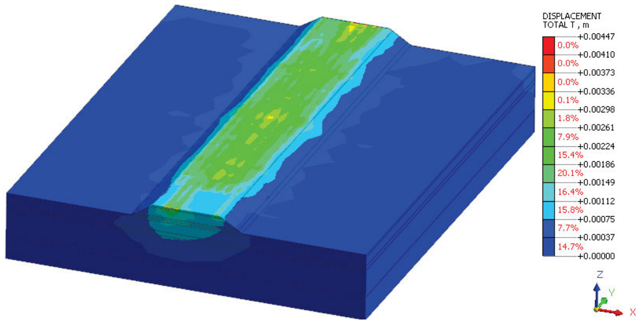

Fig. 8. Total embankment displacement due to road traffic; source: own study

CONCLUSIONS

The characteristics of designing structures in difficult soil conditions were explored in this study. It is important to consider the possibility of increasing soil moisture content by soaking the soil from the top by external sources (rainwater, meltwater). Taking this into account, it is necessary to provide a set of measures, including the elimination of collapsible properties. Regarding the investigation, the following results can be highlighted:

1) to determine the bearing capacity of soil foundations for the highway, one should rely on the results of field tests (stamp experience); the results are as follows: for soil at natural moisture, deformation modulus E_d = 34.14 MPa, for water-saturated soil, deformation modulus E_d = 7.24 MPa;

2) in collapsible soils, it is most appropriate to use pile foundations; when using piles, it is important that these are placed at a depth sufficient to provide bearing capacity and avoid collapsing, as presented in the satisfactory results of the research;

3) it is important to understand the physical state of the foundation mass and conduct calculations for the construction stage analysis; a numerical simulation of the road embankment stability has shown that the maximum displacement of the embankment is 28 mm when the soil cushion is installed; the maximum deformation due to road traffic is 4.5 mm.

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CONFLICT OF INTERESTS

All authors declare that they have no conflict of interests.

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