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Features of road design in difficult soil conditions

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ABSTRACT: One of the most important tasks facing the road industry is to improve the methodology and technology of road design and construction. Road design is a complicated complex of research works, which includes geological studies, determination of technical parameters, selection of design, and technological solutions. Designing in difficult ground conditions is especially difficult. The paper presents the peculiarities of design and construction of auto roads, the results of laboratory and field soil tests, a choice of road design considering the requirements of normative documentation. The recommendations for the choice of measures to ensure the reliability and durability of road pavement and the high level of reliability are presented.

1 INTRODUCTION

Improving the quality and efficiency of construction on subsidence soils largely depends on properly assessing their properties. The choice of methods for improving subsidence foundations and making the appropriate structural and technological solution plays a key role in the design. Traditionally, in the construction of buildings and structures, as well as road structures on weak and collapsing soils, the priority belongs to pile foundations. The expediency of improving subsidence foundations with these foundations is due to the high load-bearing capacity, reliability, and economic and technological efficiency. The proper determination of the bearing capacity of piles is one of the most critical stages in the design of foundations on collapsing soil bases. This parameter is often set experimentally from the condition of the piles on the ground. It is also necessary to improve the mechanical properties of collapsible soils by reinforcing them with geotextile materials.

The development of reliable methods for improving soil foundations in subsidence areas is an urgent task today. A distinctive feature of collapsible soils is their ability when stressed by their weight or the foundation's external load, to give additional settlements, called collapses, when moisture increases – soaking (Houston, S.L. 2002). Collapsible soils include loess, loess-like sand – clays, loams, and clays, some types of head loams and sand–clays, as well as, in some cases, fine and dusty sands with high structural strength, bulk clay soils, industrial wastes (grate dust, ash, ash deposits, etc.) Loess rocks (synonym of the term "dusty rocks") occupy a place between sandy and clayey rocks in the series of uncemented sedimentary rocks and, like them, have a polygenetic origin. The term "loess rocks" has two notions: "loess" and "loess-like" rocks (Gaaver, 2012). Loess is a rock with collapsing under natural pressure and additional loads; it has a homogeneous (non-laminated) texture and weak water-resistant structural bonds; the structure is often dusty-filmy and granular-filmy (Jiang, M. 2012).

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The main features of loess soils are (Opukumo, A.W. *et al.* 2022):

1. Yellow-brown and pale-yellow coloration.
2. High dustiness (the content of dusty fraction (0.05-0.005 mm) over 50% with a small number of clay particles.
3. The raised porosity (40-55%) with a network of macropores in size 1-3 mm visible to the naked eye caused by the presence of thin vertical tubules.
4. Low natural humidity from 0.04 to 0.18 (Sr = 0.5).
5. Ability to keep a vertical slope of considerable height in a dry condition.
6. High carbonate content.

Loess-like soils are brown and red-brown, with many fine-dust particles, fewer macropores, and granular-aggregate and aggregate structures.

Today the world practice different methods (Bellil, Abbeche and Bahloul, 2018) and ways of eliminating subsidence properties of soils by their compaction or consolidation; the device of soil cushions is used (Akbari Garakani *et al.* 2019). To consolidate the subsidence soils, use the methods of one-solution silicification or thermal firing. The device of ground cushions creates a layer of non-subsidence soil in the foundation base (Zhussupbekov,A.Zh. *et al.* 2019; Tulebekova *et al.*, 2022). Another practice presented the construction of fixed soil columns and piles in weak dusty-clayey soils when combinations of jetting and boring methods (Awwad, T. 2019; Zhussupbekov A.Zh., Utepov, Ye.B. 2015) or combination of jetting technology with the immersion of ready reinforced concrete elements are also promising (Akhazhanov,S. 2020; Yessentay,D. 2021). Also, one of the effective methods of designing structures on subsidence soils is the reinforcement of foundation soils. One of the basic concepts of soil reinforcement is reduced to the scheme when the weak soil mass is reinforced with high-strength elements and diaphragms placed in the ground. In this case, both vertical and horizontal reinforcement is possible, which in each case has a different effect on the stress-strain state of foundation soils and the operation of foundations. The improvement of ground and soil base reinforcement is directly related to the materials used for reinforcement (Zhussupbekov, A.Zh., Zhankina, A.2021).

2 INVESTIGATION CONSTRUCTION SITE

The planned section of the road (km 240 + 00-km 440 + 00) is in the Karasay and Ili districts of the Almaty region (Figure 1).

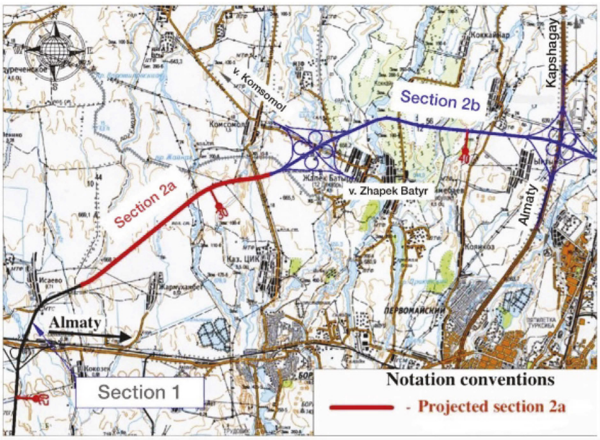


Figure 1. Section road for investigation.

2.1 *Engineering and geological conditions*

Geomorphologically, the construction site is located the foothills of the Zailiskiy Alatau (KGS 2019, 2020).

The climate of the district is sharply continental. The climate features of the region are determined by the latitude and presence of orographic elements on its surface. The combination of climate-forming factors causes the predominance of hot, dry weather with sharp seasonal and daily fluctuations in air temperatures. Summers are hot, and winters are moderately cold and mild. Heavy rains are noted in spring and summer.

The Tien Shan region is a complex mountain country formed due to repeated changes in tectonic conditions. The oldest geosyncline stage of tectonic development continued from the Archaea in places to the Ordovician. Orographically, the region is represented by a complex system of ridges and cavities. Precambrian and Caledonian fold structures form the Tien Shan region. Alpine folding began in the Upper Cretaceous era and continues intermittently to the present.

There is a general relief slope to the north. The relief was formed due to mudflows and the intense activity of river waters. It is a weakly hilly plain with river valleys. The remains of the I and II floodplain terraces are observed on it.

Hydrographically, the investigated section is located between the Kaskelen and Talgar rivers, including their feeders, and the planned route passes through them.

The planned route directly crosses the network of watercourses. Because the route of the designed road passes through these streams, they will directly affect the formation of engineering and geological conditions of the projected facility (Tulebekova,A.S. 2020; Zhussupbekov, A.Zh 2020).

The area's geological structure is characterized by loose Quaternary deposits lying on rocks of the Paleozoic bedrock. The indigenous sediments are represented by granitoid, granodiorite, and porphyrite intrusions in river basins crossing the Zailiskiy Alatau ridge. These rocks will not have a decisive impact on geotechnical profiteering, so a detailed description of the bedrock rocks is not given—soft deposits of medium- and modern-quaternary age form alluvial-proluvial complex of stones.

To detail the geological-lithological section along the axis of the route, 51 exploration wells were drilled to a depth of 6.0m. In total 199.0m were penetrated. Absolute elevations of wells are 631.59-680.70 m.

Route section from the surface is represented by topsoil with a thickness of 0.2 m. Lams from solid to fluid is penetrated with a thickness of up to 6.0 m below and sandy loams up to 1.5 m. There are layers of medium-grained sand and coarse penetrated with a thickness of 0.6 m below.

Filling soil is found on intersected existing roads. It is represented by semi-solid loam containing gravel and pebbles. Groundwater during the survey was penetrated at a depth of 1.0 - 7.8 m. The maximum penetrated power is 3.2 m.

The physical and mechanical properties of soils are given in Table 1.

2.2 *Field tests*

The tests of soil piles were carried out according to the requirements of GOST 5686-2020 (GOST 2012). The tests were carried out by the static pressing loads of 20 piles on this site (Figure 2). Loading the tested piles was made evenly, without blows, by loading steps whose size was no more than 1/10 estimated bearing capacity.

The loading and measuring devices consist of a power stand, DG200P250 jack, hydraulic pump, deflectometer 6PAO-0.01, roulette, and manometer.

The reading of each device was taken at each step of a pile loading, the first hour with an interval of 15 min., further with an interval of 30 min, before attenuation of movement of the pile called stabilization. Observations were accepted for the conditional stabilization of a pile.

Unloading of a pile was made by the steps equal to the doubled sizes of stages of loading, with the endurance of each step within 15 min. Counting on devices to measure deformations

Table 1. Physical and mechanical properties of soils.

Parameter name	EGE-2a	EGE-2b	EGE-2v	EGE-2d	EGE-3a	EGE-4v	EGE-4g
Liquid limit, %	26.9	26.3	27.1	27.3	20.7	-	-
Plastic limit, %	18.8	18.7	18.9	19.0	16.9	-	-
Plasticity index, %	8.1	7.6	8.2	8.3	3.8	-	-
Index of liquidity, %	<0	0.38	0.62	0.94	<0	-	-
Natural humidity, %	12.2	21.6	24.0	26.7	4.6	-	-
Soil particles density, g/sm3	2.71	2.70	2.71	2.71	2.69	-	-
Soil density, g/sm3	1.61	1.91	1.94	1.84	1.61	1.62	1.88
Dry soil density, g/sm3	1.45	1.57	1.57	1.45	1.54	-	-
Void ratio	0.86	0.72	0.73	0.87	0.74	-	-
Degree of humidity	-	-	-	0.97	-	-	-
Coefficient of permeability, m/d	-	-	-			7.4	-
The natural angle of slope in the dry state, deg.	-	-	-			35	38
Soil resistance, kPa	318.0	145	98	98	294	400	500

Figure 2. General view of the support prepared for static tests.

was removed after each unloading step and in 15 min observations. Observations of the movement of a pile were carried out within 60 min after complete unloading (to zero) with the removal of counting from devices every 15 min. According to the project, the movement speed in soil was no more than 0.1 mm in the last 60 min.

Stamp tests were carried out in Section 2 to study the bearing capacity of the artificial base soil for the pipe at km 43 + 225.

The bearing capacity of the base soil under the Overpass bridge was studied at 3 points of stamp testing. A stamping unit with a rotary console acting on the principle of a balance beam was used to carry out the tests.

3 RESULTS

The results of the tests are given in Table 2.

Results of the test showed that the pile during the test perceives the design load of the pile head to equal - 71.0 tf. By results of the carried-out static tests of pile No. 33 on support No. 1 (Overpass bridge through the Borolday river 2 on KM 31+265) at the tenth step of loading, the equal 106.5 tf recorded deformation (deposit) - 40.28 mm. Gain of deformations on 4.24 mm is recorded after additional endurance for 26 h. The total gain of deformations was 44.58 mm, while a stabilization deposit isn't recorded. They are objectively analyzing increment of

Table 2. Test results.

Location	No. of support	No. of pile	Design load, t	Maximum load, t	Average settlement, mm
1	2	3	4	5	6
km 24+477	3	g3	project-wise	239.0	1.48
km 29+240	1	33	project-wise	160.0	2.71
km 29+240	4	33	project-wise	135.15	2.58
km 31+245	3	43	project-wise	129.0	8.02
km 31+265	1	33	71.0	85.2	44.58
km 31+265	1	45	71.4	74.9	41.43
km 31+265	2	33	69.6	52.0	41,715
km 31+265	2	42	69.6	72.975	45.965
km 31+443	2	41	75.8	113.7	6.18
km 31+443	1	34	project-wise	83.0	24.78
km 32+730	1	67	55	83.0	29.8
km 32+730	2	33	100	102.0	38.32
km 34+169	3	56	71.7	107.6	16.49
km 34+650	1	158	109.8	165.0	6.31
km 35+622	2	5	168.3	252.5	0.96
km 35+622	4	10	167.1	250.7	3.37
km 40+100	1	24	118	177.0	5.16
km 40+131	2	9	117.8	176.7	1.33
km 40+620.37	2	11	115.8	174	6.20
km 40+620.37	1	19	115.8	174	5.22

Note – Compiled according to the source (Eurasian National University named after L.N. Gumilyov, Nur-Sultan, Kazakhstan et al., 2021; Zhankina et al., 2022)

deformations and overdue approach of stabilization to the 9th steps at the impact of efforts - 96.0 tf and schedules of the actual behavior of piles under loading perhaps reasonably to take for the extreme resistance of a pile of Fu at the pressing loadings the eighth step - 85.2 tf.

The carried-out static test of pile No. 42 on support No2 (Overpass bridge through the Borolday river 2 on personal computer 31+265), the pressing loadings showed that at the eighth step of loading, the equal 83.4 tf deformation - 45.965 mm was recorded that exceeds extreme size equal - 40 mm. According to SP RK 5.01–103–2013 (items 4.5.5), the pressing loadings take the loading of the seventh step for the extreme resistance of Fu pile- 72. 975 tf. The test results showed that the bearing capacity of pile No. 42 on support No2 on soil is sufficient to perceive design loading.

Static test of pile No33 of support No2 (Overpass bridge on personal computer 32+730) the pressing loading showed that at the eighth step of loading the equal 117 tf late stabilization (was recorded through - 34.5 hours) at the same time deformation was 38.32 mm. The seventh step loading for the extreme resistance of a pile of Fu - 102 tf is taken at the pressing loadings. The test results showed that the bearing capacity of pile No33 on support No2 on soil is sufficient to perceive a projected estimated pile load.

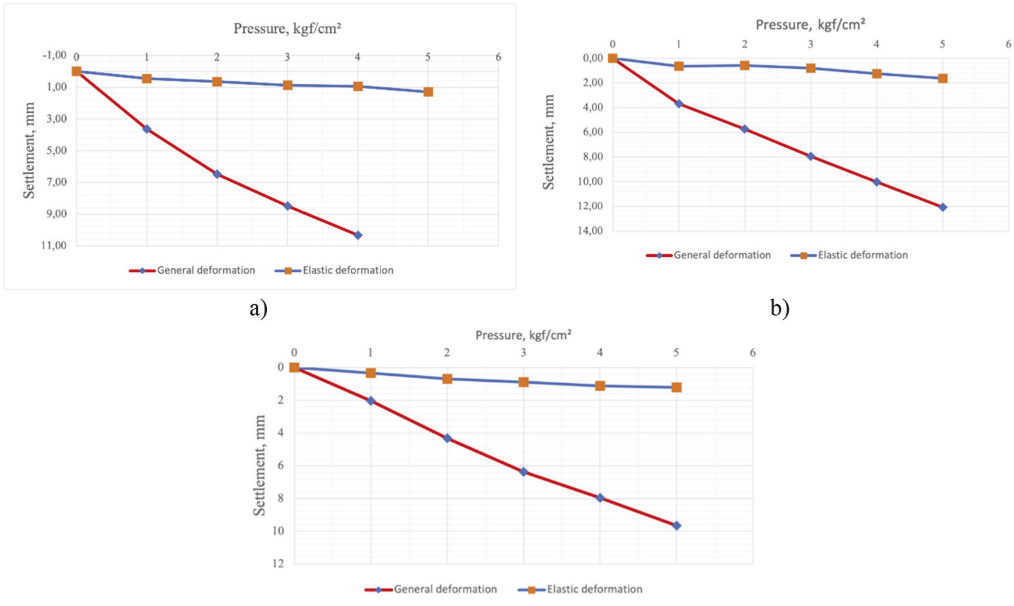
Test of pile No45 of support No1 (Overpass bridge through the Borolday river 2 on KM 31 +265), the following result was issued by the pressing loadings: at the eighth step of loading 85.6 tf, 41.43 mm settlement was recorded, at the same time stabilization of deformation didn't happen. The results showed that the bearing capacity of pile No45 on support No1 on soil is insufficient to perceive the maximum pressing design load. The actual bearing capacity doesn't exceed 74.9 tf, which is less than the projected capacity.

Static test of pile No. 33 of support No2 by the pressing loadings showed that the deformation - 41.715 mm was recorded at the sixth step of loading equal to 62.4 tf, at the same time, stabilization of deposit was not registered. The fifth step, 52.0 tf is taken for the extreme resistance of pile Fu at the pressing loadings. The tests showed that the bearing capacity of pile No. 33 on support No. 2 on soil is insufficient to perceive the maximum pressing design load. The actual bearing capacity doesn't exceed 52.0 tf, and there is less than the projected capacity.

The following conclusions can be drawn based on the results of stamp tests of artificial soil:

Point 1: Equivalent modulus of deformation of the structure E_d = 8.15 MPa.

The modulus of elasticity of the structure is not considered correctly due to extremely small elastic deformations and accumulation of residual deformations. No loss of bearing capacity occurs. Mean base modulus ratio Ee/E_d = 9.1 (Point1); Ee/Ed = 8.19 (Point 2); Ee/Ed = 6.85 (Point 3) presented in Figure 3.



a) b) c)

Figure 3. Stamping test results: a) Point 1; b) Point 2; c) Point 3.

4 CONCLUSIONS

According to the results of the study, it is concluded that:

1. The survey area is located within the IV road climatic zone.
2. Excavations with 25.0 m groundwater were penetrated at a depth from 0.4 m to 15.9 m from the day surface. The amplitude of seasonal variation of the groundwater level is + 1.0-1.2 m.
3. The standard depth of loam freezing is 0.79 m; loam, sandy, dusty and fine sands - 0.96 m; gravelly, coarse, and medium-grained sands - 1.03 m, coarse clastic sands - 1.17 m.
4. The area is in the zone with seismic hazard (according to SP RK 2.03-30-2017) [87] - 9 (nine) points.
5. The following conclusions can be drawn based on the results of the tests:

The average modulus of the structure deformation at 3 test points is following: E_d = 8.83 MPa. It is recommended to perform more thorough compaction, following the average ratio of the base modules Ee/E_d = 9.09 at the 1st point; Ee/E_d = 8.19 at the 2nd point; Ee/E_d = 6.85 at 3rd point.

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