Improvement of Handy soils by Micropile method

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***Abstract*— This research was conducted by micropile method in order to study the handy and non-aggregate soil on the east side of the Kan River, in west of Tehran. In this area a tent structure is located on a relatively deep gable roof of coarse grain non- aggregate soil (handy soil), in around of this structure, deeply cracks have been observed. Survey and study of existing cracks around this structure indicate the presence of non-aggregate surface soil subsidence and unstable gable roofs, micropile method is used for control subsidence and structure movement as well as soil improvement. These areas located in river deposits and for geotechnical studies 31 boreholes and 10 handy wells have been drilled. Types of soil are GC and SC.**

With analyses by GeoStudio software, micropile implementation for stability of gable roof and stabilizing structure in the entire system was confirmed. After implementation of design, the gable roof movement and subsidence was controlled by continuum camera and resulted stop movement and general subsidence of gable roof.

Keywords—component; slip, micropile, GeoStudio software

I. INTRODUCTION

Every year, as a result of earthquake, more bodily and financial damages caused by gable roofs sliding and structures subsidence and structures that are constructed on the Problematic lands and handy soils with low bearing capacity, comes in the different regions of the world. So we have to consider special preparations for stability of gable roofs, tranches, deeps, also verification of construction site soil. Gable roofs stability type, as well as optimization and verification of soil in different areas depending on the type of land use, climatic conditions, land quality, ease and speed of execution as well as source access to materials, commonly done with different manners, and generally the considered method should be justifiable in terms of value of engineering.

Restore and rehabilitate streams and urban watercourses due to numerous environmental is an effective step to creating sustainable urban natural green spaces. Kan river tourism studies and designing urban furniture between Hemat and Hakim highway is being implemented as well as the west side of kan river. Based on field observations and comparison of existing topographic maps in the 1384 and 1390 years, a large amounts of non- aggregate soil masses on the Earth surface is covered a large part of this area. Also in the area, a tent structure was on a relatively deep gable roof of coarse grain non-aggregate soil (handy soil) and in the around of this structure deep cracks have been observed on surface, that demonstrate subsidence and movement of this structure toward down of gable roof. In this study, soil improvement solutions (tent structure area) are provided in terms of safety against wedge slips and subsidence control.

One of the methods for trenches stability is using of micropile. Due to the limitations of using deep piles in urban and residential areas, using of micropiles can be a good alternative for rural areas with limited space. In the last two decades of extensive research in the world scientific and Professional community on micropiles and investigating their dynamic and static behaviors, we can refer to the doing of “Forever” national project in the France about surveying micropiles behavior, or design guidelines codification and conducting micropiles in the Highways Department of United States (khargabi and izadpanah, 1391).

There are different improvement methods, depending on executive operations, changing mechanisms of soil characters, grading type, … divided in several basic groups (litkohi, 1381). In this study of micropiles, we use reinforcement methods.

Micropiles:

Micropile is a type of pile with small diameter (between 100 until 300 millimeter), its performance is based on injection in the ground. Micropiles in comparison with commonly piles have smaller dimensions, and often in their implementation is used light steel reinforcement and Cement slurry injection and concrete with high efficiency. Since implementation of micropile, like implementation of commonly piles due structure overhead transfer to the lower resistant layers of soil and limiting subsidence, in addition, result stability of slope and wall of excavation and tranches, seismic optimization, reinforce damaged foundations and the ground beneath the foundation, optimization of Problematic soils and control of soil liquefaction during earthquake, optimize the mechanicals properties of the soil surrounding the project site as well as micropile side And soil mechanical parameters optimize such as: density, bearing capacity, permeability and subsidence modulus. domain of Micropiles usage are relatively wide but in most cases, are related to soils bearing capacity optimization in two horizontal and vertical orientations. Main forces that are spread in micropiles, mainly are: compressive and tensile forces. Bending in case of a single micropile or a group of micropiles that are vertically, can be substantial. Into the soil-micropile interaction, essentially is a friction because the force of a single micropile tip, in stressful positions, because of the small diameter it is often negligible. In addition, the loose lands that have little lateral resistance, there is the possibility of micropiles buckling (khargabi and izadpanah, 1391).

The geographical location of the study area

The study area (javanmardan park) located in northwest of the Tehran, in Iran. Its geographical coordinates are 35º 45' 14''north and 51º 16' 14''east. The park is located in the kan river that from north leads to hemat highway and from south lead to hakim highway (figure 1). The park has a length of nearly 1800 meter along the north- south orientation along kan river and nearly 300 meter width in the west- east. The eastern and western coast of the river is included kan, and in this research the eastern coast of this park is studied.

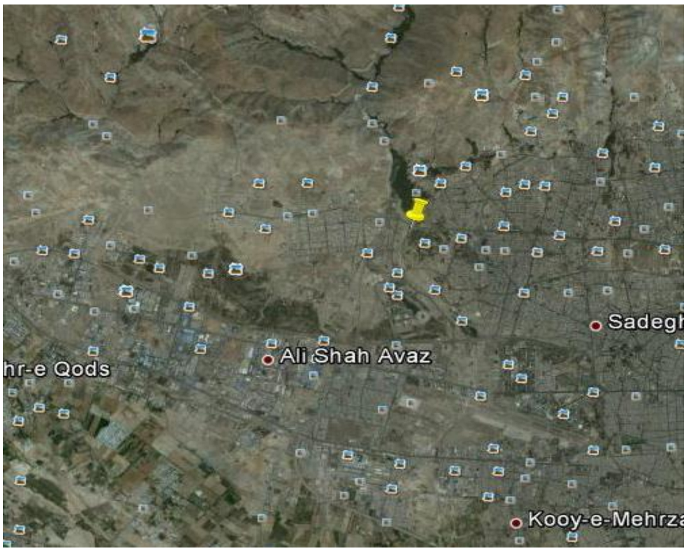

Figure 1: plan location in Tehran.(́Adopted by Google Earth)

Geotechnical studies in the area

In order to achieve the projects goals, boreholes and handy wells have been drilled in two phases in the years 1389 and 1390. In the first phase, 26 research borehole with a maximum depth of 40 meter above ground level have been drilled that data achieved from 17 boreholes has been appropriate. In the second phase in order to complete geotechnical data, 5 research borehole and another 10 handy wells have been drilled (Noandishan plan consulting engineers, 1390) (kazemi, H., 1392).

To determine the relative resistance of the soil, impact and standard penetration test (SPT) was performed according to ASTM D1586 standard. Results indicate relative density in the gravel and sand strata and tightness in the clay strata. It should be noted that in the coarse and dense strata is used of the cone-shaped tip in the end of SPT tube.

To determine the soil physical and mechanical properties, classification, resistive and deformation tests have been conducted on representative samples.

Done permeability tests (lufran) indicate lufran number 2 meter/second at shallow depths, with increasing depth to 20-meter reach to 0.18, from a depth of 20 to 30 meter increased with low slope so that in 30 m reaches to 0.8 m/s.

Direct shear test in accordance with ASTM D3080 standard was performed, for slow and fast shear on the reconstructed samples with 10 ×10 cm for obtained samples from boreholes and 30 × 30 cm for sinks samples. Result of mentioned tests are presented in table 1. Based on conducted tests in fast case, for fine –grained soils, the amount of viscosity (c) was between 0.1 and 0.22 kg/cm^2 and the amount of their internal friction angle (φ) are between 21 to 34 degrees, whereas for fine grained soils in the slow case, the amount of viscosity (c) was between 0.09 to 0.15 kg/cm^2 and the amount of their internal friction angle (φ) are between 20 to 29 degrees. While that for coarse –grained soils, the amount of samples viscosity was between 0/09 to 0.2 kg/cm^2 and the amount of their internal friction angle (φ) obtained 33.8 to 41 degrees. It is worth noting that differences in fine- grained soils is due to difference in the test speed. Because the tests are performed on reconstructed samples, the amount of soils real viscosity is more than obtained viscosity.

$$x = \frac{-b \pm \sqrt{b^2-4ac}}{2a} \quad (1)$$

to determine soil physical characteristics, grading tests, hydrometric and atterberg limits, density and moisture content was carried out on the received samples. Soil layers have been classified based on soil united classification system (USCS).

Based on the results of the laboratory, fixed and engineering judgment, physical and mechanical properties of soil different layers are given in table 2.

According to 2800 Regulation instruction of design for buildings against earthquakes (publications of building and housing research center, No- 253) the study area located within the area one that are places with relatively high risk. The amount of acceleration on the design recommended a=0.35g. also the soil type for determining structure reflection coefficient is estimated Ш type.

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Table 1: direct shear tests (kazemi, H., 1392)

C (kg/cm2)	ɸ (deg)	Density (gr/cm3)	Shear spead	Box (cm)	Soil type	Specimen type	(M) Depth	Borehole
0.09	28	1.94	Slow	10*10	SC	Remolded	13	BH-1
0.1	34	1.94	Fast	10*10	SC	Remolded	13	
0.22	29	1.95	Fast	10*10	CL	Remolded	22	
0.15	20	1.94	Slow	10*10	CL	Remolded	8	BH-2
0.2	21	1.94	Fast	10*10	CL	Remolded	8	
0.1	29	1.94	Slow	10*10	SC	Remolded	4	BH-5
0.14	32	1.94	Fast	10*10	SC	Remolded	4	
0.21	34.2	1.85	Fast	30*30	GC-GM	Remolded	1.5-3.5	TP-1
0.23	34.5	1.88	Fast	30*30	GC	Remolded	3.5	TP-2
0.16	38.7	2.1	Fast	30*30	GC	Remolded	3	TP-3
0.2	33.8	1.72	Fast	30*30	GC-GM	Remolded	3	TP-4
0.15	39.3	2.3	Fast	30*30	GC-GM	Remolded	3	TP-5
0.14	37.6	2.16	Fast	30*30	GC-GM	Remolded	3	TP-6
0.18	35.7	1.91	Fast	30*30	GW	Remolded	3	TP-7
0.12	39	2.23	Fast	30*30	GC-GM	Remolded	3	TP-8
0.09	41	2.23	Fast	30*30	GW	Remolded	3	TP-9
0.17	37.1	2	Fast	30*30	GC-GM	Remolded	3	TP-10

CRACKS OCCURRED IN THE STUDY AREA

The results of continuum monitoring of predetermined points by mapping camera as well as measurement of disruption existing cracks around the tent structure on a relatively deep gable roof indicate the together occurrence of surface non- aggregate soil subsidence and instability in gable roof.
The cause of cracks in the surface of the ground could have been argued, that some of these cracks occurred because of sufficient density in handy bulwark and another batch occurred because of slide outbreak and instability in gable roofs. In fact, the incidence of atmospheric precipitation caused soil surface resistance and create non- aggregate soil subsidence that cause subsidence in ground different places. On the other hand, due to insufficient viscosity of soil and increasing in Pore water pressure, instability slides occurred in the gable roofs. That bunch of cracks that often are located downstream of the tent structure have appeared for this reason.

AREA OF GEOTECHNICAL SECTIONS

Given the importance and criticality on this part compared to other parts of area, A, B, C and D geotechnical sections were drawn One of the sections cross of the tent structure and three other has passed around it. Stabilization solutions in the plan has been investigated for more critical section that passes under the tent structure (section B) (figure 2).

Table 2. physical and mechanical properties of existing natural soil strata in the area (kazemi, H., 1392).

Hard clay with low plasticity range	Semi-hard clay with low plasticity range	clayey sand (Very dense)	Silty or clayey sand (semi-intensive)	Sandy or silty clay(dense and very dense)	***Type of soil***
22-28	20-35	32-36	33-36	35-38	***Drained Angle of friction (□$_d$) Deg***
0/5-1	*0/15-0/5*	0/05-0/5	0/1-0/8	0/15-1	***Drained cohesive coefficent (C_d) Kg/cm^2***
0-5	0-5	-	-	-	***unDrained Angle of friction (□$_u$) Deg***
1/5-2/5	*0/5-1*	-	-	-	***unDrained cohesive coefficent (C_u) Kg/cm^2***
1/9-2	*1/8-1/9*	1/85-1/95	1/9-2/1	1/95-2/15	***Special Weight (□) gr/cm^3***
400-700	*200-350*	300-400	500-700	600-800	***Modulus of elasticity of rock (E) Kg/cm^2***
0/4	*0/4*	0/35	0/3	0/3	***Poisson's (□) ratio***

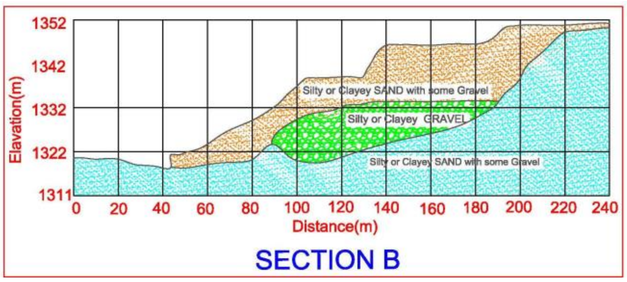

Figure 2: B geotechnical section (Kazemi, H., 1392).

Analyze of the data

In this section, we discussed the bearing capacity and design of done structure stability methods on tent structure in the study area. As previously mentioned, stabilization solutions in the plan for more critical section has been investigated that passes under the tent structure (geotechnical section B).

In order to estimating soil allowable resistance, shear failure factors by Hansen method and subsidence has been studied and examined together. Results of calculations shows that for single foundations with 1, 2, 5 and 10 length to width ratios by limiting allowable subsidence to 2.5, the allowable bearing capacity of foundations is equal 4.5, 3.5, 3 and 2.8kg/cm^2, respectively. And for spread foundations with 10 and 15 m length these number are 7.8 and 5.7 kg/cm^2, respectively (Kazemi, H., 1392).

To determine floor reaction coefficient for foundation with slightly dimensions from the corresponding graphs, by dividing the soil allowable stress (qa) on the amount of subsidence (s), the numerical coefficient of soil reaction (ks) are achieved. For example, for stirp foundation with L/B=2 and width of 3 m, under 5.4 kg/cm^2 allowable shear, approximately floor reaction coefficient will be achieved 2.10 kg/cm^3 (Kazemi, H., 1392).

In order to determine the forces applied from soil to the structure for design of retaining walls (if necessary to build in the future), Ka stimulus, K0 rest and Kp resistant lateral pressure coefficients, in both static and dynamic mode is recommended as follows (tables 3 and 4) (Kazemi, H., 1392).

Table 3: soil lateral pressure coefficient in the static mode- rankin method

Hard clay with low plasticity range	Semi-hard clay with low plasticity range	clayey sand (Very dense)	Silty or clayey sand (semi-intensive)	Sandy or silty clay (dense and very dense)	*static lateral pressure coefficients*
0.36-0.45	0.41-0.49	0.26-0.31	0.26-0.29	0.24-0.27	*Active pressure coefficent (K_a)*
0.53-0.63	0.58-0.66	0.41-0.47	0.41-0.46	0.38-0.42	*static pressure coefficient (k0)*
2.2-2.77	2.04-2.46	3.25-3.85	3.39-3.85	3.69-4.2	*passive pressure coefficient (K_p)*

Table 4: soil lateral pressure coefficient in the dynamic mode- rankin method.

Hard clay with low plasticity range	*Semi-hard clay with low plasticity range*	*clayey sand (Very dense)*	*Silty or clayey sand (semi-intensive)*	*Sandy or silty clay (dense and very dense)*	*dynamic lateral pressure coefficients*
0.47-0.57	*0.52-0.62*	*0.35-0.4*	*0.35-0.39*	*0.32-0.36*	*Active pressure coefficent (Kae)*
.96-2.51	*1.8-2.21*	*2.97-3.55*	*3.1-3.55*	*3.39-3.89*	*passive pressure coefficent (Kpe)*

Based on 2800 regulation instruction of buildings design against earthquake (publication of building and housing research center, N-253) the study area are within area one since placed with a relative high risk. The amount of design foundation acceleration is recommended a= 0.35g. the soil type to determine structure reflection coefficient is estimated of the Ш type.

In order to determine the stability of slopes (Geoslope)of praic- morgnstren method, a equilibrium - drawing extent method for determine slope stability, characterize confidence cofficient, where the balance of forces and anchores are given for sliding section with Grid and Radius method.

INVESTIGATION OF STABILIZATION SOLUTIONS IN SECTION B

In this section, stabilization options of avaiable gable roofs within the tent structure were studied (i.e, section B) and the optimal solution from standpoint of ensuring sufficient safety factor against stability, social and economic issues, ease of doing work and the run time have been investigated and better options have been introduced. In this regard, 11 options have been examined and eventually two following design are considered (kazemi, H., 1392).

Option 1- The implementation of Injection operations under the tent structure and excavation in its downstream and implementation of Concrete walls as geotextile of steel belts instead:

The injection operation is considered under a tent structure to a depth at least 12 m at intervals of 2 m, to increase the soil viscosity under the tent structure at least 50 kPa. Also, it is necessary for excavation downstream of tent structure and instead of that, concrete wall implemented as geotextile or steel belt in staircase mode. So that the least amount of equivalent viscosity and internal friction in the concrete wall should be 50 kPa and 35 degrees, respectively. In this case, the values of confidence factor in the static and dynamic condition will be 2.038 and 1.545, respectively (figures, 3 and 4).

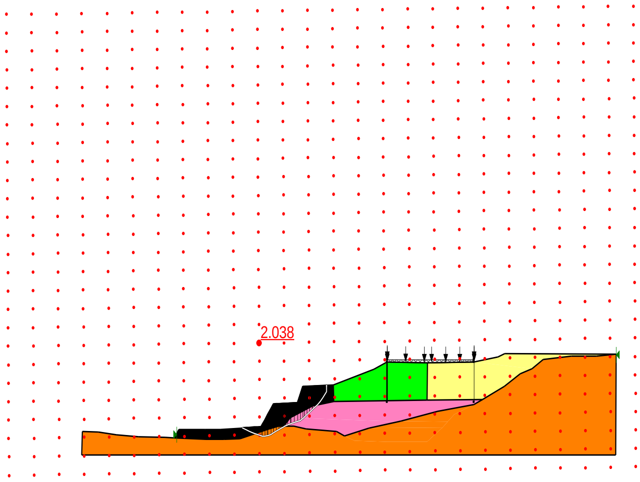

Figure 3: option 1 stabilization. Injection of tent substructure and implementation of concrete wall in the downstream of structure- static analyze- with confidence coefficient 2.038 (with Geostudio software)

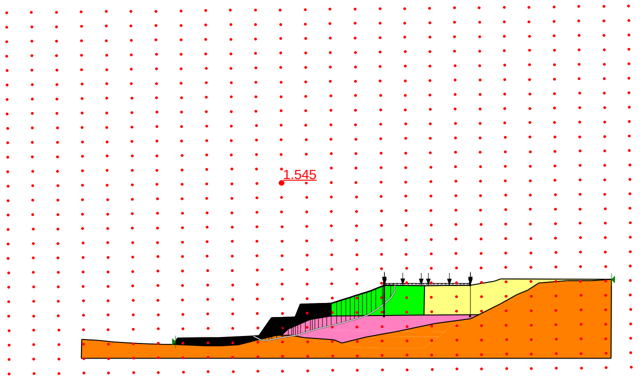

Figure 4: option 1 stabilization. Injection of tent structure and implementation of concrete wall in the downstream of structure- dynamic analyze- safty factor: (confidence coefficient) 1.545 (with Geostudio software).

Option 9- the implementation of micropile in the whole system

In this design the implementation operation is considerd of 14 micropiles row with a total length of 13200 m (from upstream to downstream with 6 rows of 25 m, 2 rows of 20 m, 2 rows of 15 m, 2 rows of 12 m and 2 rows of 10 m, respectively) of steel tube with least diameter of 5.7 cm and a thickness of 1 cm to 2 m horizontal distance from one another along the length of the river. In this case the value of confidence coefficient in the static and dynamic condition will be 2.159 and 1.487, respectively (figure 5 and 6).

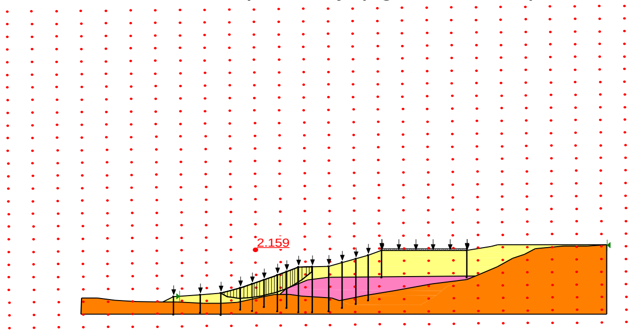

Figure 5: option 6 stabilization. The implementation of micropile in the whole system- static analyze- with safty factor: (confidence coefficient) 2.159 (with Geostudio software).

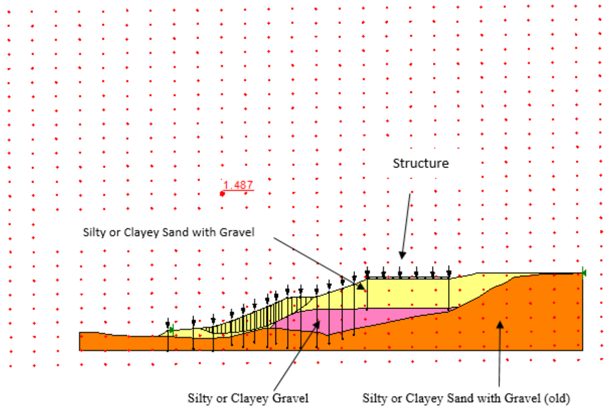

Figure 6: option 9 stabilization. The implementation of micropile in whole system- dynamic analyze- with confidence coefficient 1.487 (with Geostudio software).

Opening cracks and subsidence around structure are monitoring by mapping camera every few days, after completing of micropiles in upstream of structure (25 m micropiles) it is distinguished that slope movement and opening of cracks have not stopped completely yet. So with strengthening designs, particularly in the southwestern of structure, movement and subsidence were stopped. After completing and running micropiles show nearly complete stopping of slope movement.

Conclusion

The NE kan river improvement project located in west-northwest of Tehran and in cross of Hemat highway and kan river. From geological point of view, the design was implemented in the west of river and alluvial fan with slope to the south. In fact, the old sediments of the area are alluvial terraces. According to surveys, the existing sediments in the area mostly are coarse grain that become coarser to the north and decrease the amount of fine grain.

1. The calculations showed that for single foundations with 1, 2, 5 and 10 m length to wide ratios by limiting allowable subsidence to 2.5 cm, allowable bearing capacity of foundations are 4.5, 3.5, 3 and 2.8 kg/cm^2, respectively and for spread foundations with 10 and 15 m lengths will be 7.8 and 5.7 kg/cm^2, respectively.

According to existing of tent structure in the study area that have a special importance based on social and political issues, four geology profile were drawn within the area, which B section passed under the tent. That's why the stabilization of gable roof related to this section have been investigated and optimization solution was introduced. For stabilization of B section, eleven design are planned in the static and dynamic condition with a confidence coefficient obtained for each case and any of viewpoints were evaluated, scored and weighted. Finally, the design 9 (I.e., the implementation of micropile) was conducted on the entire system.

Suggestions

If before the implementation of micropile, the soils of area be bruised in 1 to 2 m maximum depth and after that the pilling plan was implemented, better outcome was acquired and would prevent the local subsidence that sometimes occurs around the tent structure.

Instead of injection operations that were done for the entire length of the bore, it was better to be in step by step mode, for example be done from 5m length of bore and from up to bottom.

The reasons for using of geophysics method: ensure of effectiveness of and accuracy of injection operations and identifying probably remained holes in the depths, in this case, a plan must be considered for that (this proposal because that recently been seen some local subsidence in some places).

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REFERENCES

[1] Asadinia, H and mirzeynali, H., (2009) “applied engineering geology,” payam noor publications..

[2] Aghanabati, A. (2004), geological survey of iran, Iran’s geological survey and mineral exploration.

[3] Javaheri, A and pakniat, A (2009), static and dynamic analyses of soil dams using Geostudio, publication of civil engineering, tehran.

[4] Bozorgi, A. pourjafar, M. bemanian, M. (2005), planning process for restoring Tehran valleys with river, journal of geography, N-76, (pages 53 to 77).

[5] National database of geoscience (2005), iran fault maps, scale 1:280000.

[6] National database of geoscience (2001), Tehran geological map, scale 1:100000.

[7] Pedrami, M. (1976), briefly subjects about quaternary sedimentsaround Tehran, geological survey of iran. 18 pages.

[8] Pahlavan, B. (2002), study of coarse grain of Tehran alluvial using pressuremeter, thesis of geological engineering doctoral, tarbiat modares university.

[9] Poutkermani, M. and arin, M. (1998), seismicity of iran, publications of shahid beheshti university.

[10] Tavakuli, B. (1999), principles of engineering geology, publication of payam noor university.

[11] Cheshmi, A. (2006), study of mechanical properties of Tehran coarse grain alluvial based on geological combination and mechanical experiments, engineering geology doctoral thesis, tarbiat modares university.

[12] Khanlari, KH. And moumeni, A. and abdilar, Y. (2007), geotechnic and geological engineering, publications of bu ali sina university.

[13] Khosrovtehrani, KH. (2004), geology of iran, publications of payam noor university.

[14] Das, Braja am. (2010), the volume of geotechnical engineering, soil mechanic, translated by shapour tahoni, the twentieth edition, pars aien publications.

[15] Das, Braja am. (2009), the second volume of geotechnical engineering, foundation engineering, translated by shapor tahoni, the fifteenth edition, pars aien publications.

[16] Darvishzade, A. (2003), geology of iran, publications of amirkabir university.

[17] Abasi, M. (2011), reports of study structural geology and new tectonic of the niayesh tunnel area, 89 pages.

[18] Kharghabi, A. and izadpanah, S. (2012), micropiles and their roles in soil optimization, nemad golestan magazine, pages 36 to 43.

[19] Fathi, A. (2010), surveying the appropriate excavation methods in Tehran coarse grain alluvial, engineering geology master thesis, Islamic azad university, science and research branch of Tehran.

[20] Ghobadi, M. (2006), principles of engineering geology (for geology students), publications of bu ali sina university.

[21] Kazemi, H. (2013) optimization of handy soils by micropile method, engineering geology master thesis, Islamic azad university, the branch of science and research.

[22] Lithkuhi, S. (2002), optimization of soil in past, now and future, irans third international conference on soil mechanic and geotechnical engineering, institute of energy, Tehran.

[23] The study of NE part of darrekan river, noandishan design consulting engineers- water and environment (2011).