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Investigation of the Suitability of a Pipe Stock Site in Terms of Geotechnical Characterization and Engineering Design Parameters Considering the Economic Aspects

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ABSTRACT

Geotechnical investigations are generally site specific and the parameters variate spatially and sometimes temporally due to meteorological events and earth dynamics. In this study, the suitability of an inert area to utilize a pipe stock site under a natural gas pipeline project was investigated, while considering the economic yield. Within the context of this study, office works and field works are performed. Geotechnical data are collected during the field work in order to assess sub-surface conditions, prevailing under the proposed loads or pipe batch structures. Under field works, drilling operations, in –situ testing are performed. Assessment of the collected sub-surface data and its interpretation are realized within the office works. In other respects, rentability is one of the main parameters controlling the feasibility of the project in engineering. In this study, the bearing capacity of the soil profile and the liquefaction potential of the investigated area are enlightened and the suitability of the study area for the project is evaluated by comparing it with the various alternatives of soil remediation techniques and the consideration of their costs. As a result of this study, it is revealed that the soil remediation costs are much more than port storage renting fees.

Keywords:
Bearing capacity; Liquefaction analysis; Geotechnical investigations

INTRODUCTION

In geotechnical engineering, soil is defined as the unaggregated natural material formed by the weathering and deposition processes of rock cycle. Due to the inhomogeneous nature and complexity of soil forming processes, physical parameters of soils variate spatially [1]. Soil moisture content variations by virtue of the rainfall runoff dynamics, a temporal variation of physical parameters of soils come into question. Additionally, inherent variability of different soil parameters include also measurement errors and model transformation uncertainties [1]. Soil liquefaction is an important event which can be occurred by an earthquake in fully saturated and cohesionless soils. In consequence of liquefaction, pore water pressure of soil increases which provoke a dramatical shear strength loss, and important decrease in bearing capacity. The results of low bearing capacity and liquefaction have critical and severe concerns in engineering. For this reason, prior to the construction stage, geotechnical characterization of soil should be investigated precisely to predict the liquefaction potential and bearing capacity problems. Under these circumstances, engineering use of lands requires a detailed ground investigation in order to assess the liquefaction potential and the bearing capacity characteristics. Geotechnical site characterization in terms of soil bearing capacity and foundation analysis are studied by many researchers [2, 3, 4, 5, 6, 7]. In these studies, obtaining of the geotechnical parameters in the most correct way with in-situ testing and laboratory testing are emphasised. Bowles [2] expressed bearing capacity considering SPT. Random field theory and elasto-plastic finite element analysis are used to determine the spatial variability of cohesion and angle of internal friction by [3, 4]. Geotechnical site characterization with in-situ testing applications are presented by [5]. Measurement errors for soil in-situ testing in a geotechnical site characterization is studied by [6]. Vanmarcke [7] modelled the engineering properties through a soil profile.

The aim of this study is to investigate the geotechnical characteristics and to determine the soil remediation costs of an inert area with a low rental cost to use as a pipe storage site for an international natural gas pipeline project. The soil remediation costs of this inert area is compared with a regular port storage area (which is expected of no soil remediation) as an alternative since its rental costs are 800% higher than the inert area.

MATERIAL AND METHODS

In terms of definition of the geotechnical problems which is assumed to be faced in the field and search for solutions; preliminary works including office and field survey/analysis and detailed site works and detailed office works are also performed throughout the project. Field pre-studies consist of investigation of detailed geological mapping of the study area and its near environ. Drilling operations, in-situ testing are conducted under detailed site works. The results of these systematic works are evaluated with detailed office works considering the profitability of subjected study area including the soil remediation costs. As a result of these engineering works, ultimate feasibility of the storage operations on the study area is determined in terms of geotechnical applicability.

Profitability of a project is one of the main issues in engineering. In this study, the soil remediation alternatives are evaluated considering their costs and construction time. In this regard, excavation of problematic layer of soil and filling it with suitable natural structural material, jet grouting, drain swamping and port storage renting fees are also compared. Within the context of this study, the engineering approaches are presented when assessing different soil remediation alternatives and project feasibility in terms of project profitability.

STUDY AREA

Study area is located in İskenderun (Mediterranean Region) in which the climate is characterized by dry summers and mild rainy winters. İskenderun is an important county of Hatay city with a huge seaport and industrial potential. The average temperature during the hottest month is around 32 to 34°C whereas the average temperature during the coldest month is around 10 to 12°C. Mean annual rainfall is 850 mm with an irregular regime. This precipitation regime causes an inharmonic fluctuation of water table considering the elevation of the study area which is 2.5 m on average. This situation affects the soil dynamics in terms of soil moisture content and liquefaction potential.

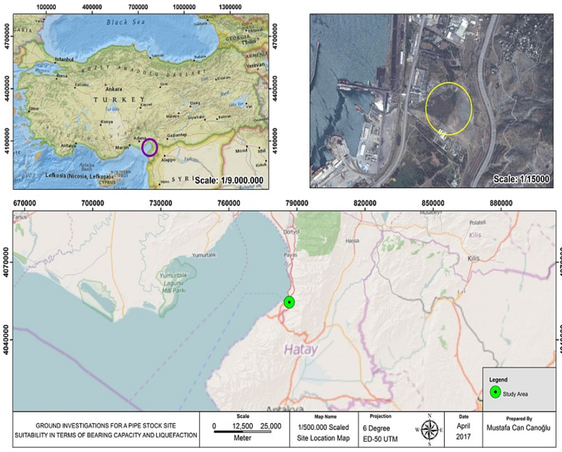

Figure 1. Site location map of the study area

Figure 2. General view of study area

The study area is identified in 1/25000 scaled Hatay O36-d3 topographic map prepared by National Mapping Agency of Turkey [8]. Investigations are conducted in the swamp region that is located on the western side of İskenderun-Payas state road. İskenderun-Ceyhan highway is located on the eastern side of the study area where, there is a military area on the southern side. Additionally, İskenderun-Payas state road is located on the western side of study area where FIL Filter Factory and warehouses of logistic companies are located on the northern side. Transportation is provided by highways for four seasons (Fig. 1 - 2).

Geological Setting of Study Area

In the study area and its near environ the Mesozoic and Cenozoic aged geological units are dominantly observed. Triassic-Jurassic aged Demirkazık Formation (TJd) are founded in the base of stratigraphic array. Demirkazık Formation is covered discordantly by Cretaceous aged Kızıldağ Ophiolites (Ck) [9]. Finally, Quaternary aged alluvium (Qal) and slope debris (Qsw) cover all these units discordantly (Fig. 3 - 4).

Demirkazık Formation (TJd)

The eastern side of İskenderun depression and along the line of Amanos Mountain, the formation consists of micritic limestone. This formation is light- dark gray colored,

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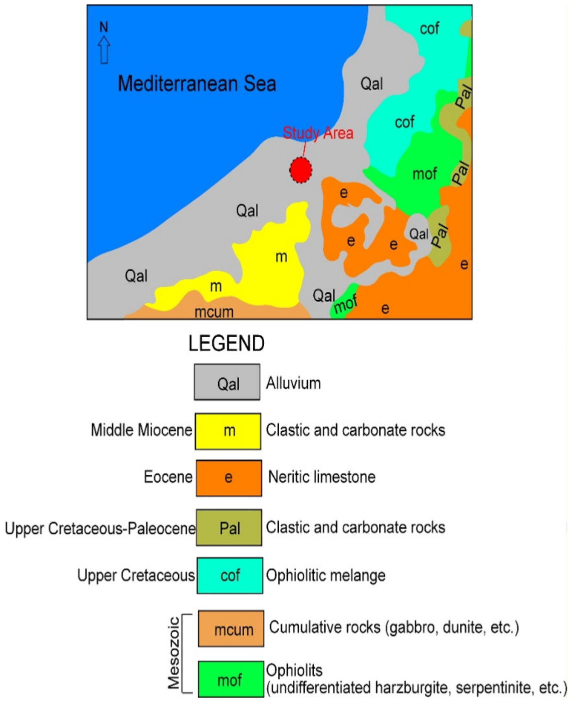

Figure 3. Simplified geological map of the study area

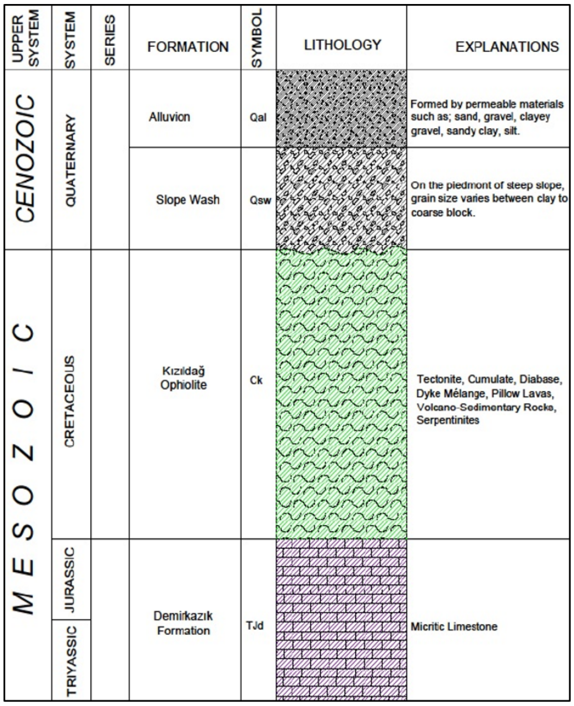

Figure 4. : Columnar stratigraphic section in the study area [modified from 9]

medium to coarse grained bedded and microfossiliferous [9].

Kızıldağ Ophiolite (Ck)

The outcrop of Kızıldağ Ophiolite is observed at the eastern side of the project area through a north-south (NS) direction. This allochthone unit composes the section of Amanos Mountain's southwest margin. Kızıldağ Ophiolite is generally formed by; from youngest to oldest, tectonite, cumulate, diabase, dyke mélange, pillow lavas and volcano-sedimentary rocks, respectively. In the investigation area, olivine serpentinization is observed for different rock types of Kızıldağ Ophiolite. Isoclinal folding is specifically characterized for Kızıldağ Ophiolites. This folding can be evaluated as the trace of an active tectonism which can cause liquefaction potential in the study area during a possible earthquake. The outcrops of this unit are generally weathered and can be classified as soil [9].

Slope Debris (Qsw)

Blocky structured slope debris are formed by the weathered bedrock and transported to piedmont. For this reason, slope debris is constituted by coarse gravel and clay sized material and the cementation is observed. The color of slope debris is generally light brown-brown, greenish gray and white, its grain size varies between clay to coarse gravel. Pebbles and blocks are angular, semiangular or rounded and the origin is serpentine. The slope debris thickness can reach up to 1,5 m based on field observations.

Alluvium (Qal)

Quaternary aged alluviums are formed by the sediments transported by the Yarıkkaya River by erosion, transportation and deposition processes. This unit is constituted by gravel, sand, silt and clay sized material. According to the drilling works, the thickness of the alluvium is varies between 2.50 - 5.00 m in the study area.

GEOTECHNICAL CHARACTERIZATION OF STUDY AREA

In the study area 11 ground investigation boreholes are drilled in order to determine the geotechnical parameters and the prediction of potential risks. These investigations reveal two different zones in the study area in terms of different soil bearing capacity characteristics under dynamic loads (Fig. 5).

In the study area, some traces of active tectonism are observed, this part consist of normal faulting component presumably developed in the East Anatolian Fault Zone (EAFZ) [10]. In Fig 6, the fault which is found in the study area is presented. That fault is crossing the quaternary aged alluvium which indicates that the fault is potentially active. As liquefaction needs dynamic movements to be occurred in soft soils, this fault could be evaluated as the triggering effect of a potential liquefaction in the study area.

During field observations, we have discovered some areas having very low bearing capacity which was proved

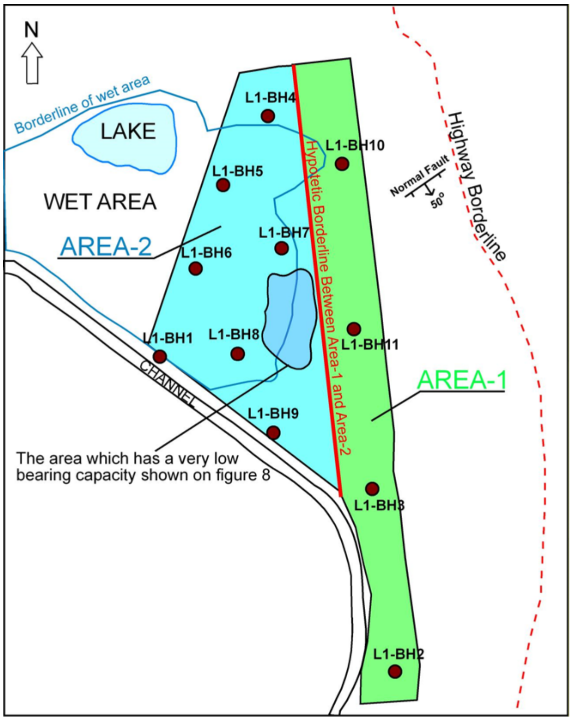

Figure 5. Zonation plan of the project area

Figure 6. Normal faulting observed in the study area, the arrows show the direction of the movement along the fault scarp (red line)

Figure 7. A photograph showing the very low soil bearing capacity of area-2

by a picture shown in Fig 7. For this reason, the study area is separated into two different area; area-1 and area-2 namely.

In Area-1, the soil is cohesionless and the depth varies between 7.0 – 16.5 m. For the borehole L1-BH3, the bedrock extends to 20 m depth. For the boreholes, L1-BH2, L1-BH10 and L1-BH11 the bedrock (serpentine) is located under the subjected cohesionless soil. In Area-2 a peat type soil is encountered which is known with its low bearing capacity. The depth of peat varies between 2.0 – 11.7 m and under the peat bedrock depth is deeper than 20 m (Fig. 8). 8 types of

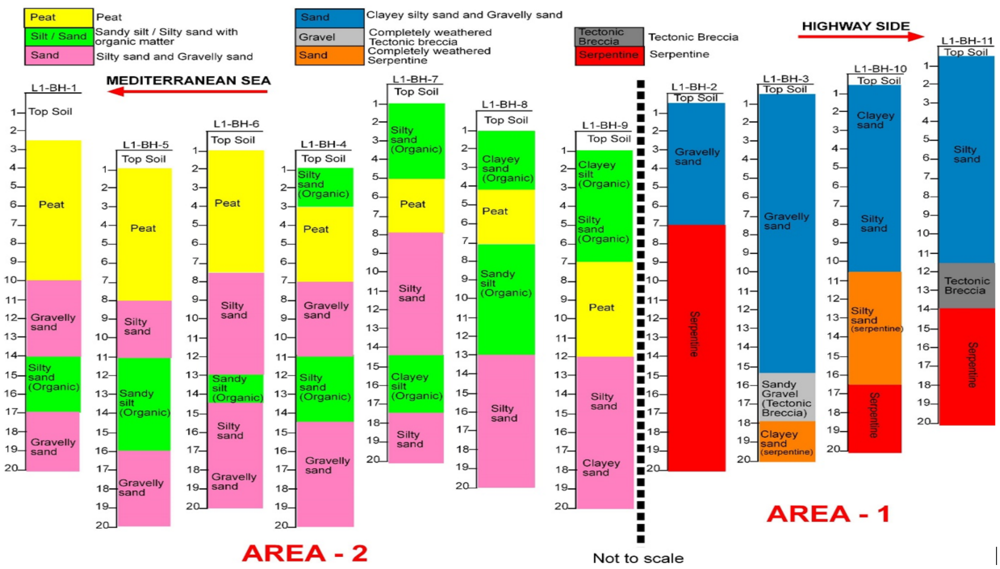

Figure 8. Stratigraphic correlation of the lithological logs collected from the study area

ground is sampled during drilling operations and laboratory tests that has been performed to determine their geotechnical characteristics.

Top Soil

The thickness of top soil layer observed in all boreholes (L1-BH-4 is the exception) is up to 0.70 meters from the ground level. It is planned to remove top soil layer by stripping excavations. For this reason, any of laboratory tests were carried out for this layer.

Filling material

This layer constitute of the residual soil subjected to a recent fill material and decided to be removed by stripping excavations due to its soft character and low bearing capacity.

Peat

This layer contains highly organic soil which is dark brown - black, with vegetable tissue, fibrous – amorphous texture, with organic odour. This layer is generally consists of 4% gravel, 37% sand and 59% fine materials and observed in the boreholes; L1-BH-1 between 2.50 – 9.65 meters, L1-BH-4 between 2.90 – 7.05 meters, L1-BH-5 between 0.90 – 7.85 meters, L1-BH-6 between 1.20 – 7.40 meters, L1-BH-7 between 4.90 – 7.90 meters, L1-BH-8 between 4.00 – 7.00 meters, L1-BH-9 between 7.20 – 11.70 meters. The Geotechnical parameters of peat layer are listed in Table 1.

Sandy silt and silty sand

This layer consists of clay-silt-sand and gravel units with different ratios. Sand and Silt units are the dominant ones. This layer contains dense organic material. Gravels and clays are observed locally. This layer is characterized by Sandy Silt – Silty Sand which consists of 10% gravel, 43% sand and 47% fine materials. The layer observed in the boreholes; L1-BH-1 between 14.00 – 17.30 meters, L1-BH-4 between 0.80 – 2.90 and 11.00 – 14.60 meters, L1-BH-5 between 11.00 – 16.00 meters, L1-BH-6 between 13.00 – 14.50 meters, L1-BH-7 between 0.90 – 4.90 and 14.70 – 17.50 meters, L1-BH-8 between 0.90 – 4.00 and 7.00 – 13.00 meters, L1-BH-9 between 1.10 – 7.20 meters. The Geotechnical parameters of sandy silt and silty sand layer are listed in Table 2.

Silty sand and gravelly sand

Silty sand and gravelly sand layer is a multi-coloured and heterogonous. It consists of silty- gravelly sand. The saturation level is approximately 90% for this layer. The mean grain size distribution is 20% gravel, 59% sand and 22% fine material and clay-silt-sand and gravel units with different ratios. Sand and Silt are dominant units. This layer contains dense organic material. Gravels and clays are observed in the boreholes; L1-BH-1 between 9.65 - 14.00 and 17.30 – 21.50 meters, L1-BH-4 between 7.05 – 11.00 and 14.60 – 20.13 meters, L1-BH-5 between 7.85 – 11.00 and 16.00 – 20.00 meters, L1-BH-6 between 7.40 – 13.00 and 14.50 – 19.95 meters, L1-BH-7 between 7.90 – 14.70 and 17.50 – 20.00 meters, L1-BH-8 between 13.00 – 20.00 and, L1-BH-9 between 11.70 – 20.00 meters. The Geotechnical parameters of silty sand and gravelly sand layer are listed in Table 3.

Clayey sand – silty sand – gravelly sand

Clayey sand – silty sand – gravelly sand layer is multi-coloured and heterogonous. It consists of loose to very dense clayey - silty – gravelly sand. The moisture con-

Table 1. Geotechnical characteristics of peat layer

Test Name				Minimum	Average	Maximum	
Moisture Content		W_n	:	32.6	141.2	301.8	%
Bulk Density		V_n	:	1.07	1.30	1.60	g/cm³
Dry Density		V_d	:	0.29	0.60	1.02	g/cm³
Specific Gravity		G_s	:	1.85	2.36	2.62	
Sieve Analysis	No:4 Retaining		:	0.0	4.4	26.1	%
	No:200 Passing		:	17.5	59.1	99.0	%
Atterberg Limits	Liquid Limit	LL	:	NP		89	%
	Plastic Limit	PL	:	NP		69	%
	Plasticity Index	PI	:	NP		37	%
Triaxial Test	Cohesion	c	:	0.07	0.26	0.33	kg/cm²
	Internal Friction Angle	Φ	:	1	2	4	o
Consolidation Test	Swelling Pressure		:		<0.10		kg/cm²
	Void Ratio	ϑ_n	:	1.567	3.538	6.703	
SPT	Field Values	N_{30}	:	0	3	11	
	Corrected Values	$N_{1,60}$	:	0	2	8	
Soil Classification			:		PT		

tent of this layer around the study area is generally 90 to 95%. The mean grain size distribution is 21% gravel, 52% sand and 27% fine material and clay-silt-sand and gravel units with different ratios. Sand and Silt units are dominant. This layer contains dense organic material. Gravels and clays are observed in the boreholes; L1-BH-2 between 0.30 – 7.00 meters, L1-BH-3 between 0.10 – 15.85 meters, L1-BH-10 between 0.30 – 10.30 meters, L1-BH-11 between 0.20 – 11.40 meters. The Geotechnical parameters of clayey sand – silty sand – gravelly sand layer are listed in Table 4.

Tectonic breccia

This layer consist of yellowish beige, slightly to highly weathered (W3-W4) and fractured rock medium. Tectonic breccia unit is observed in the borehole L1-BH-11 between 11.40 – 13.80 meters. The Geotechnical parameters of tectonic breccia unit are listed in Table 5.

Serpentine

This layer constitute of dark green to greenish brown, slightly to highly weathered (W3-W4) and highly fractured rock medium. Serpentine unit is observed in the boreholes; L1-BH-2 between 7.00 – 20.00 meters, L1-BH-10 between 16.50 – 20.00 meters and L1-BH-11 between 13.80 – 20.00 meters. The Geotechnical parameters of serpentine unit are listed in Table 6.

Table 2. Geotechnical characteristics of sandy silt and silty sand layer.

Test Name				Minimum	Average	Maximum	
Moisture Content		W_n	:	11.25	35.66	83.20	%
Sieve Analysis	No:4 Retaining		:	0.0	10.4	38.0	%
	No:200 Passing		:	9.9	46.5	82.8	%
Atterberg Limits	Liquid Limit	LL	:	NP		59	%
	Plastic Limit	PL	:	NP		43	%
	Plasticity Index	PI	:	NP		21	%
SPT	Field Values	N_{30}	:	0	14	R	
	Corrected Values	$N_{1,60}$	:	0	10	>50	
Soil Classification			:		MLO/SCO		(generally)
					SMO,SCO,SM,ML		(locally)

Table 3. Geotechnical characteristics of silty sand and gravelly sand layer.

Test Name				Minimum	Average	Maximum	
Moisture Content		W_n	:	5.06	17.53	31.50	%
Sieve Analysis	No:4 Retaining		:	0.0	19.7	47.4	%
	No:200 Passing		:	3.4	21.6	48.6	%
Atterberg Limits	Liquid Limit	LL	:	NP		59	%
	Plastic Limit	PL	:	NP		34	%
	Plasticity Index	PI	:	NP		25	%
SPT	Field Values	N_{30}	:	7	32	R	
	Corrected Values	$N_{1,60}$	:	3	21	>50	
Soil Classification			:		SC/SM		

Table 4. Geotechnical characteristics of clayey sand – silty sand – gravelly sand.

Test Name				Minimum	Average	Maximum	
Moisture Content		W_n	:	8.70	17.59	31.43	%
Sieve Analysis	No:4 Retaining		:	6.8	20.6	42.5	%
	No:200 Passing		:	9.5	26.9	48.6	%
Atterberg Limits	Liquid Limit	LL	:	NP		51	%
	Plastic Limit	PL	:	NP		30	%
	Plasticity Index	PI	:	NP		21	%
SPT	Field Values	N_{30}	:	4	32	R	
	Corrected Values	$N_{1,60}$	:	2	21	>50	
Soil Classification			:		SC/SM		

Table 5. Geotechnical characteristics of tectonic breccia unit.

Test Name			Minimum	Average	Maximum	
Natural Unit Weight	V_n	:	1.19	1.88	1.85	g/cm³
Point Load Index	$I_{s(50)}$	:	0.04	0.045	0.05	MPa
Field Values	TCR	:		100		%
	RQD	:	0	25	100	%

Table 5. Geotechnical characteristics of serpentine unit.

Test Name			Minimum	Average	Maximum	
Natural Unit Weight	V_n	:	1.72	2.10	2.38	g/cm³
Point Load Index	$I_{s(50)}$	:	0.01	0.07	0.17	MPa
Field Values	TCR	:	33	79	100	%
	RQD	:	0	7	100	%

RESULTS AND CONCLUSION

In this study, the spatial variations of the soil geotechnical parameters of the study area are determined by the systematic analyses of drilling, in-situ test and geotechnical laboratory surveys. As discussed earlier, the study area is divided into two areas namely area-1 and area-2 in terms of their geotechnical properties. Within the project, It is predicted that area-1 will have approximately 250 kPa service load in total (live loads; ground pressure of crawler crane is 109 kPa for this project, fill loads; max. height of fill is approximately 4.00 m. and pipe loads; the weight of a long pipe is 15 ton, 3 rows were stacked, load is distributed from 2 points). The soil strength of the area-1 is enough to bear these service loads under existing conditions. However, this area is located on a flooding gully and Ground Water Level (GWL) can rise up to ground level during a heavy rains. For this reason, area-1 tends to liquefy in case of seismic loads as there is an active fault in and close proximity of the study area. Additionally, the soil material in this area can be subjected to liquefaction under dynamic loads which is controlled by the active tectonism of EAFZ. Under seismic loading, bearing capacity would be decreased dramatically (Fig 7).

For shallow footings, an embankment with a thickness of minimum 50 cm formed by cleaned granular material is proposed for Area-1. This granular fill should be covered by a lean concrete of minimum 20 cm thick. Placement of an appropriate geotextile under this granular fill, to prevent the water infiltration, can be considered as another alternative.

Area-1 and area-2 are compared in terms of bearing capacity values and possible soil improvement techniques. Area-1 have poor soil conditions with a groundwater level (GWL) deeper than 7m. The liquefaction potential of Area-2 is higher than Area-1; but for some local parts of Area-1 can be evaluated as susceptible for liquefaction. In Area-2 the groundwater level is shallow and the general topography of the Area-2 can be evaluated as flooding gully. Deep foundations are obligatory for Area-2. However the costs of deep foundations are not profitable for this project.

The soil improvement techniques are assessed for Area-1 and Area-2 in order to find out the most suitable and rentable technique. With this context, removing the problematic soil layer and replacing it with rock fill is evaluated. But the soil thickness planned to be removed by the stripping excavation is approximately 3.5m. In this case, approximately 550 000 m^3 of excavation is predicted. In consideration of unit prices for excavation which is evaluated by General Directorate of State Hydraulic Works (DSI) and General Directorate of Highways (KGM), the costs will exceed the economic limits of the project. Additionally, in case of heavy rainfall, GWL can rise up to ground level and soil tend to liquefy under even slightest dynamic load caused by an earthquake or a dynamic vibration during the handling operation. Consolidation (preloading + drainage) and vibro-replacement stone column alternatives are also considered but these techniques consume time and not suitable for the soil type of the study area. For the dynamic compaction alternative, Atterberg limits could be fulfilled. However non-plastic (NP) soil is not dominated the whole area. At least Jet grouting and/or pile foundation are recommended in order to remediate the soil of the pipe storage area. It is emphasized that with this remediation load bearing capacity which is controlled by only pile point strata and frictional resistance will be improved since liquefaction potential of soil is ignored. Under this circumstance, pile foundation and jet grouting alternatives can be applied to augment the bearing capacity but these techniques are not effective for a potential liquefaction event. In conclusion, excavation and rock infilling, pile foundation, jet grouting, vibro replacement stone, drain swamping, utilizing geotextile are compared as the different alternatives of soil remediation in terms of feasibility and profitability. On the other hand, the port storage alternative (renting fee is 8 times more expensive) which has no need of soil remediation, is also evaluated. In this context, need of 550 000 m^3 excavation is not rentable based on the unit prices valuated by DSI and KGM. Pile foundation and jet grouting techniques are also neglected due to their ineffectiveness in prevention from liquefaction. Vibro replacement stone and drain swamping alternatives are also evaluated within the scope of the project. However, these techniques are time consuming and do not meet the urgency criterion of the project. An embankment of gravel (minimum 50 cm thick) and concreting on it and protect this construction with an impermeable geotextile is also recommended as an alternative. But this alternative is also time consuming and exceed the economic limits of the project. For this reason, the port storage is the most suitable alternative regarding the project schedule, and economic aspects in spite of its 800% more expensive renting fee. The remediation costs of the study area will be also more expensive than the port storage renting fees.

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