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ASSESMENT OF LIQUEFACTION POTENTIAL RELEVANT TO CHOICE OF TYPE AND DEPTH OF FOUNDATIONS IN SEISMICALLY ACTIVE AREAS

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ABSTRACT

There is evidence of great increase of pore pressures in saturated sand soils during cyclic loading caused by earthquakes. These increased pore pressures can often increase to effective stresses in soil. Dependant on sand density, this can lead to a total loss of shear strength, liquefaction or greater deformability of soil. Emergence of liquefaction or great deformations within soil can cause significant damage or total destruction of constructions on the ground, even when they have been correctly designed. For this reason, it is very important to perform detailed geotechnical and seismic investigations of ground conditions and evaluate liquefaction potential for saturated sand soil in seismically active terrain. It is not possible to design stable constructions in certain types of terrain without the analyses of liquefaction potential.

This paper refers to the comparative cost-analyses of two possible ways of the foundations of the business complex: shallow foundations with stabilization of potentially liquefiable sand deposit using vertical gravel drains versus deep pile foundation on unliquefiable soil.

INTRODUCTION

Soil liquefaction is a major concern for structures constructed on sand or sandy soils. To evaluate the potential for soil liquefaction it is important to determine the soil stratigraphy and in-situ state of the deposits. The CPT is an ideal in-situ test to evaluate the potential for soil liquefaction because of its repeatability, reliability, continuous data and cost effectiveness.

Based on the extensive and detailed geotechnical and seismic investigations of the ground at the site for the future Energy Business Centre of the Electricity Board of Belgrade in New Belgrade (Report, 2009), it has been determined that in case of a strong seismic impulse and appearance of shear cyclic loading, development of liquefaction may occur in some of the sand soil deposits.

Two variants of foundations has been performed due to this, shallow – on foundation slab with vertical sand drains which would stop the liquefaction from occurring; and deep – with bored piles which would be driven deeper than the zone of possible liquefaction.

GEOTECHNIAL CHARACTERISTICS OF THE SOIL

Soil in the investigation area is composed of deposits of varying geological age. Typical soil stratigraphy in the investigation area is shown in table 1.

Terrain in the wider area has been artificially levelled by sand up to elevation 75-76 meters above sea-level. At the time of conduct of additional planned soil investigations, the underground water level has been established around elevation 67.30 meters above sea-level, (June 2008), while during the maximum water-levels of the rivers, due to the direct hydraulic link, underground water can be expected around elevation 74 (Report, 2009).

Geotechnical characteristics of lythological elements comprising the construction of the upper soil (up to the depths of around 20 meters) significant for the analysis of liquefaction potential: content of certain grains, mean grain diameter value (D_{50}), resistance to cone penetration (q_{C1N}),

Table 1. Soil stratigraphy in the investigation area

Soil and depth	Lithology and description of soils
1 1.50m	**Man-maid deposit**, sand, SF_S, very heterogenous in density, loose to dense, gray to brown. **Man-made deposit,** silty to sandy clay, waste deposit, with very different grain size, unconsolidated
2 6.00m	**Clay,** sandy and silty , CL/CI, gray to brown gray (old ground surface)
3 7.50m	**Silt,** ML, sandy**,** with clay lenses, very loose to loose, very deformable, brown
4 19.0m	**Sand,** SU, medium-grained sands with presence of fine layers, loose to medium-compacted, red to red-brown
5 27.0m	**Sand,** gravely, (SP/GP) medium-grained to coarse, with silty and clayey layers, dense, gray
6 30.0m	**Gravel,** sandy, GW/SW with different silt and clay content, dense, gray
7 ?	**Clay,** marly, CH, overconsolidated, stiff, non-deformable, gray

relative density (D_r), ratio of effective vertical overburden stress (σ_0') and total vertical overburden stress (σ_0) are given in Table 2.

SEISMIC REGIME

Seismic regime at the investigation micro-location is defined by triple exponential and third Gumbel distribution.

Magnitude of quake is given by formula

$$M(D,R) = M_{max} - (M_{max} - U)\left[-\frac{n_t}{D}\ln(1-R)\right]^{\lambda} \qquad (1)$$

whose parameters for central parts of Serbia have been empirically determined by N. Grujić and S. Radovanović (1989).

$$n_t = 2 \qquad M_{max} = 7.3 \qquad U = 41 \qquad \lambda = 0.238$$

$$M(100:0.1) = 7.3 - (7.3 - 4.1)\left[-\frac{2}{100}\ln(1-0.1)\right]^{0.238}$$

Expected maximum magnitude (by both methods) for Serbian seismogenic zone, for period of D=100 years and risk R=0.1 is: M_{max}=6.56. According to Fig. 1, for magnitude $M=6_{1/2}$, equivalent number of stress cycles n ≅ 8.

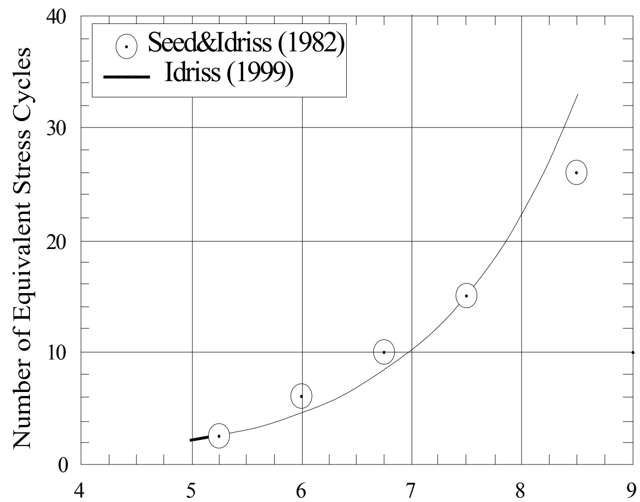

Fig. 1. Number of equivalent stress cycles vs. earthquake magnitude

Maximum seismic acceleration of soil at the location is:

$$\frac{a_{max}}{g} = 6.7\left[e^{1.05M + \frac{1.65}{M}}\right]\left[X + 35 + 0.17e^{0.65M}\right]^{-2.56} \qquad (2)$$

Investigated location is found at X=35 km north of Lazarevac focus zone, so that for M_{max}=6.56 and X=35 km follows: a_{max}/g =0.106. Calculated value is increased by 50 % and as referent acceleration we adopt:

$$\frac{a_{max}}{g} = 0.160$$

Conventionally, this acceleration is considered appropriate for seismic intensity VIII° MCS.

ASSESSMENT OF LIQUEFACTION POTENTIAL

Evaluation of liquefaction potential has been performed by using two methods. Firstly, based on the simplified Seed and Idriss procedure (1971), it has been assessed that there are natural and artificial sediments in the soil construction, which, depending on the strength and duration of the strong phase of the earthquake and value of initial effective vertical overburden stress can get to liquefaction state. Those are: man-made sands deposit on the ground surface and alluvial sands below the old soil surface (up to the depths of 4-16 meters from the present day soil surface). Analysis of liquefaction potential in further text relates only to the natural soils – alluvial sands. Assessment of the liquefaction potential of the foundation soils has been performed by using in-situ measurements, Seed *et al* (1983), based on the results of field tests of static penetration (CPT), Liam Finn, 1988.

Table 2. Liquefaction's possibility with respect to grain size

Soil	Sample	< 0.006 (mm)	0.70 (mm)	D_{50} (mm)	Liquefiable
2	ED-3(4.50)	28	2	0.018	Yes/no
	ED-4 (5.10)	28	2	0.020	Yes/no
	ED-2 (5.30)	25	1	0.022	Yes/no
	ED-1 (5.60)	29	0	0.015	Yes/no
	ED-4 (5.90)	15	1	0.035	Yes/no
	ED-1 (6.50)	18	2	0.025	Yes/no
3	ED-3 (5.50)	12	0	0.040	Yes/no
	ED-2 (6.30)	11	0	0.060	Yes/no
	ED-3 (7.20)	12	0	0.050	Yes/no
	ED-3 (7.50)	14	0	0.035	Yes/no
4	ED-2 (7.80)	9	0	0.070	Yes
	ED-2 (8.50)	5	0	0.090	Yes
	ED-3 (8.50)	15	0	0.028	Yes/no
	ED-1 (8.60)	1	0	0.150	Yes
	ED-4 (9.80)	20	0	0.030	Yes/no
	E-1 (10.50)	3	1	0.120	Yes
	B-9 (13.00)	7	0	0.250	Yes
	B-9 (15.00)	10	0	0.250	Yes
	B-9 (16.00)	3	30	0.420	Yes/no

Table 3. Determining Dr from ground stress conditions and CPT

Soil	Sample	Stress ratio σ_0/σ_0'	q_{c1N} MPa	Dr %
2	ED-3(4.50)	0.57	1.63	44
	ED-4 (5.10)	0.57	1.54	45
	ED-2 (5.30)	0.57	2.56	57
	ED-1 (5.60)	0.57	1.46	41
	ED-4 (5.90)	0.57	1.37	40
	ED-1 (6.50)	0.57	1.46	41
3	ED-3 (5.50)	0.55	1.40	32
	ED-2 (6.30)	0.55	2.36	57
	ED-3 (7.20)	0.55	1.40	32
	ED-3 (7.50)	0.55	1.37	40
4	ED-2 (7.80)	0.54	2.56	57
	ED-2 (8.50)	0.55	2.29	45
	ED-3 (8.50)	0.56	1.61	39
	ED-1 (8.60)	0.54	6.32	54
	ED-4 (9.80)	0.58	5.58	44
	E-1 (10.50)	0.61	6.30	54
	B-9 (13.00)	0.57	4.00	45
	B-9 (15.00)	0.56	7.00	55
	B-9 (16.00)	0.56	7.00	55

Robertson et al. (1983) have introduced the correlation of counted number of blows N and resistance to cone penetration (q_{C1N}) in sands of differing mean grain diameter (D_{50}) and have produced a diagram q_{C1N} / N_{60} vs. D_{50} (reduced to ER=55 %, commonest value in practice in USA). Based on the mentioned diagram, values of q_{C1N} / N_{60} ratio at the investigated area for mean grain diameter D_{50} are shown in Fig. 2 and Table 4.

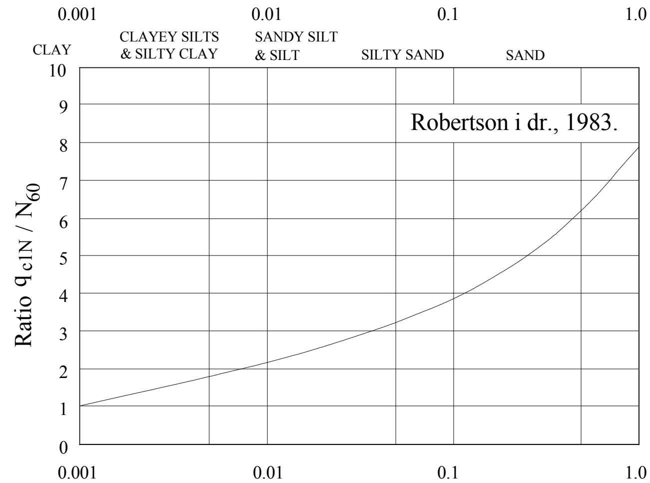

Fig. 2. Variation of q_{C1N}/ (N_{60}) with respect to D_{50}

Table 4. Variation of q_{C1N}/ (N_{60}) with respect to D_{50}

Depth (m)	D_{50} (mm)	q_{C1N}/ (N_{60})
5.5–8.5	0.015	230
6.0-10.0	0.035	300
6.0-8.0	0.070	350
8.5-10.5	0.120	400
13.0-15.0	0.250	500
16.0	0.420	600
17.5	0.700	700

FACTOR OF SAFETY

The factor of safety against liquefaction is defined as:

$$F_S = \frac{CSR_{liq}}{CSR_{eq}} \geq 1.30 \qquad (3)$$

CSR_{liq} – cyclic stress ratio which leads to liquefaction
CSR_{eq}– cyclic stress ratio caused by movements during earthquake

$$CSR_{liq} = 0.65\left(\frac{a_{max}}{g}\right)_{crit}\frac{\sigma_0}{\sigma_0^{/}}r_d \qquad (4)$$

$$CSR_{eq} = \frac{(\tau_h)_{pr}}{\sigma_o^{/}} = 0.65 \left(\frac{a_{max}}{g} \right)_0 \frac{\sigma_0}{\sigma_0^{/}} r_d \qquad (5)$$

$(\tau_h)_{pr}$ is average cyclic shear stress,

a_{max} is the maximum horizontal acceleration at the ground surface,

g = 9.81 m/s^2 is the acceleration due to gravity,

σ_0/σ_0` is ratio of total and effective overburden stresses ,

r_d is a stress reduction factor which is dependent on depth and is obtained from Fig3.

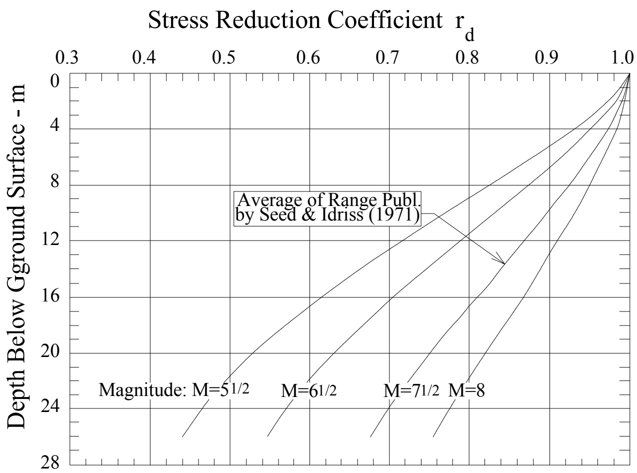

Fig.3 Variation of stress reduction coefficient with depth and earthquake magnitude (from Idriss, 1999)

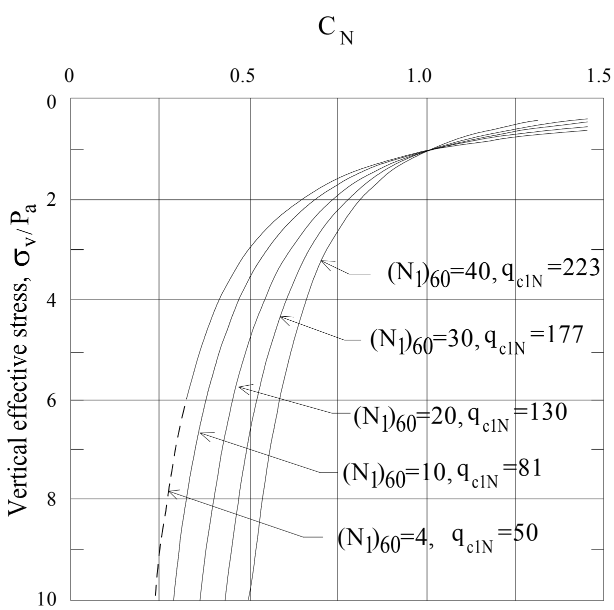

Fig. 4. Overburden normalization factor C_N (Boulanger and Idriss, 2004)

q_{c1N} - equivalent normalized cone penetration resistance from CPT.

Overburden normalization factor C_N is given as:

$$C_N = \left(\frac{P_a}{\sigma_V^{/}} \right)^{\alpha} \qquad (6)$$

$$\alpha = 1.338 - 0.249 \cdot (q_{C1N})^{0.264} \qquad (7)$$

Overburden normalization factor C_N can be obtained directly from Fig. 4.

Normalized values of N1 60 are given in Table 5.

Table 5. Normalized values N_{60}

Soil	Sample	$\frac{q_{c1N}}{N_{60}}$	N_{60} (blow/foot)	α	C_N	$(N1)_{60}$
2	ED-3(4.50)	270	6	1.05	2.23	13.4
	ED-4 (5.10)	260	6	0.96	1.89	11.3
	ED-2 (5.30)	280	9	1.02	1.97	17.7
	ED-1 (5.60)	230	6	1.06	1.81	10.9
	ED-4 (5.90)	300	5	1.07	1.75	8.7
	ED-1 (6.50)	280	5	1.06	1.58	7.9
3	ED-3 (5.50)	305	5	1.06	1.88	9.4
	ED-2 (6.30)	335	8	1.02	1.60	12.8
	ED-3 (7.20)	320	4	1.06	1.42	5.7
	ED-3 (7.50)	300	5	1.07	1.36	6.8
4	ED-2 (7.80)	350	7	1.02	1.29	9.0
	ED-2 (8.50)	380	6	1.03	1.18	7.1
	ED-3 (8.50)	285	6	1.05	1.19	7.1
	ED-1 (8.60)	420	27	0.93	1.15	31.0
	ED-4 (9.80)	290	19	0.94	1.01	19.2
	E-1 (10.50)	400	16	0.93	0.95	15.2
	B-9 (13.00)	500	8	0.98	0.69	5.5
	B-9 (15.00)	500	14	0.92	0.65	9.1
	B-9 (16.00)	600	12	0.93	0.59	10.4

Including equations (4) i (5) in equation (3), factor of safety against liquefaction can be obtained from:

$$F_S = \frac{\left(\frac{a_{max}}{g} \right)_{crit}}{\left(\frac{a_{max}}{g} \right)_0} \qquad (8)$$

$$\left(\frac{a_{max}}{g}\right)_{crit} = \frac{(N_1)_{60}}{12.9 \cdot M - 15.7} \cdot \frac{\sigma_v'}{\sigma_v} \cdot \frac{1}{0.65 \cdot r_d} \qquad (9)$$

Calculated values of factor of safety against liquefaction in the investigation area on the location of New Energy Business Centre of the Electricity Board of Belgrade in New Belgrade, are given in Table 6.

Table 6. Factors of safety F_s against liquefaction in the investigation area

Soil	Sample	r_d	σ_o' / σ_o	ξ	$F_s=\xi / \psi$
2	ED-3(4.50)	0.94	0.57	0.181	<1.30
	ED-4 (5.10)	0.93	0.57	0.155	<1.30
	ED-2 (5.30)	0.93	0.57	0.241	>1.30
	ED-1 (5.60)	0.92	0.57	0.151	<1.30
	ED-4 (5.90)	0.91	0.57	0.121	<1.30
	ED-1 (6.50)	0.90	0.57	0.112	<1.30
3	ED-3 (5.50)	0.90	0.55	0.128	<1.30
	ED-2 (6.30)	0.90	0.55	0.175	>1.30
	ED-3 (7.20)	0.89	0.55	0.079	<1.30
	ED-3 (7.50)	0.88	0.55	0.095	<1.30
4	ED-2 (7.80)	0.88	0.54	0.123	<1.30
	ED-2 (8.50)	0.86	0.55	0.101	<1.30
	ED-3 (8.50)	0.86	0.56	0.101	<1.30
	ED-1 (8.60)	0.86	0.54	0.435	>1.30
	ED-4 (9.80)	0.83	0.58	0.299	>1.30
	ED-1 (10.50)	0.82	0.61	0.252	>1.30
	B-9 (13.00)	0.77	0.57	0.091	<1.30
	B-9 (15.00)	0.72	0.56	0.158	<1.30
	B-9 (16.00)	0.70	0.56	0.186	>1.30

$$\xi = (a_{max} / g)_{crit} = \frac{N_1}{12.9 \cdot M - 15.7} \cdot \frac{\sigma_o'}{\sigma_0} \cdot \frac{1}{0.65 \cdot r_d}$$

$$\psi = \left(\frac{a_{max}}{g}\right)_0 = 0.160$$

From data in table 6 it can be observed that for given seismic parameters, underground water level at elevation 67.30 meters above sea-level and determined geotechnical characteristics of foundation soils at investigated area, overlying and middle parts alluvial sediments can get into liquefaction state: sandy silts (layer 3) and fine-grained to viscous and medium-grained sands (layer 4), loose to medium-compacted, at depths of 4 – 15 m, and all sediments whose measured resistance to cone penetration is less than $q_c \le 5000$ kN/m^2 .

GEOTECHNICAL CONDITION OF FOUNDATION

Taking into consideration the geological formations and structures of the ground and geotechnical properties soil, especially liquefaction possibility, there were two possible ways of foundation:

1. shallow pad foundation with drainage system of gravel blanket and vertical gravel drains, and
2. deep bored pile foundation on unliquefiable soil.

Triangular foundation pad is 1.5m tick, total surface area A=4100m^2 and total load F=430 MN.

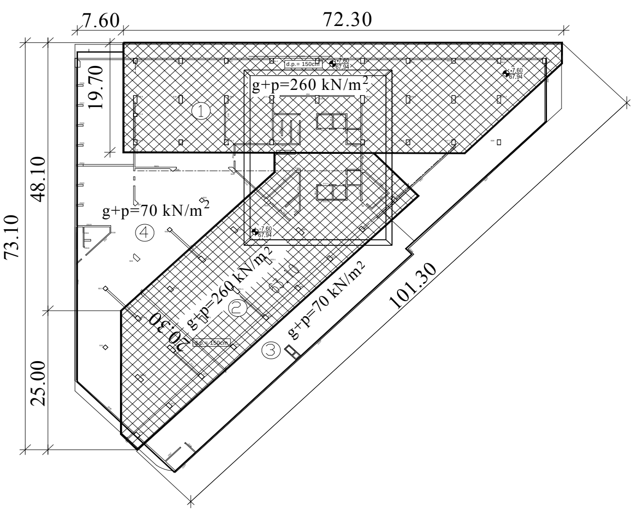

Fig.5 Plan of building New Energy Business Centre of the Electricity Board

SOIL CONSOLIDATION WITH GRAVEL DRAINS

Large grained soils, gravels, due to their greater water porosity show greater stability in respect of redistribution of stress and dissipation of pore pressures during earthquakes. Based on this, it can be concluded that the problem of lowering pore pressures during earthquakes can be solved by increasing water porosity of foundation soils. This can be achieved by constructing gravel drains if porosity in them is at least 200 times greater than that of surrounding soil (Seed, 7).

Process of soil consolidation is accelerated by appropriate spacing of gravel drains, and increase of pore pressures during the quakes can be maintained within acceptable limits.
Drain-to-drain distance has been determined in accordance with the paper by Seed & Booker (1977). Dominant influence in application of gravel drains is radial soil drainage, and drains are positioned in a network of equilateral triangles. Quotient of volume compressibility m_v of the soil is constant for the whole layer, and water permeability quotient k is determined by test pumping.

For stated magnitude M=6.56 quake duration is Td=25 s, and corresponding quake intensity, expressed in equivalent number of cycles of amplitude of shear strength Neq≈8 Foundation soil has the following characteristics: volume compression m_v=5.5x10^{-5} m^2/kN, water-permeability quotient k=10^{-5} m/s and non-dimensional quotient has the value:

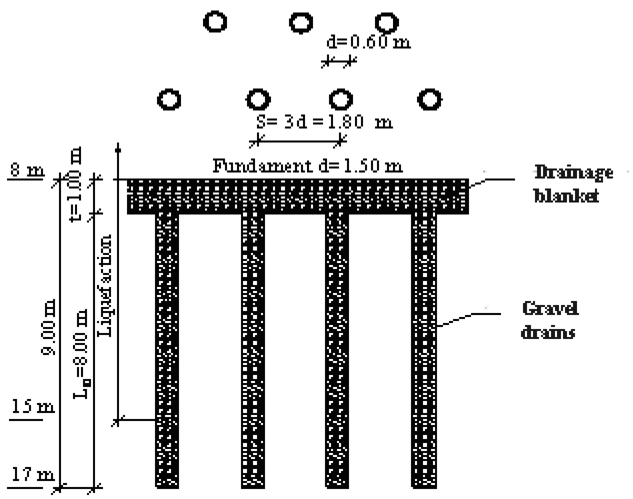

Fig. 6. Gravel drains disposition

$$T_{ad} = \left(\frac{k}{\gamma_w}\right)\cdot\left(\frac{t_d}{m_{v3}\cdot a^2}\right) = \left(\frac{10^{-5}}{9.8}\right)\cdot\left(\frac{25}{5.5\cdot 10^{-5}\cdot 0.3^2}\right) \approx 5.0$$

For maximum value of pore pressure quotient r_u=0.6, the suggested parameters of gravel drains are: diameter D=600mm and axial distance 3b=1.8m. Adopted length of gravel drains is Ld ≈ 8.0m (Figure 6).

ECONOMIC ANALYSIS

Bearing in mind that it has been assessed that soil contains layers that are prone to liquefaction in powerful earthquake conditions, cost analysis of different foundation methods that would maintain stability of the liquefiable soil has been performed. Two variants that were suggested were: shallow foundation on a mat with placement of drainage blanket and gravel drains or deep pile foundations.

For adopted pattern of equilateral triangles of gravel drains placement and total surface area of the construction of P≈4100 m^2, number of needed gravel drains is N_d≈1260. Total length of gravel drains is 10080 meters. Considering that the total cost of constructing 1 meter of gravel drains is € 100.00, construction of all gravel drains within the building would amount to € 1.008.000,00. Construction of drainage blanket 1.00 meters thick, with widening below the foundation slab costs approximately € 86.000,00, totalling € 1.094. 000,00.

Deep foundation analysis has been conducted with condition that the piles are placed under the pillars of the construction and in the same disposition as gravel drains. Piles would be performed by CFA technology. The bearing capacity of pile is Qa=900 kN, and real load is Q=745 kN. Bored piles of large diameter Ø 600 mm, 12 meters long, with adopted depth of clamping into the soil that is not liable to liquefaction of 6.0 meters, whose allowed loading is greater than designed, and which settles less than it is allowed have been suggested.

For conditions listed beforehand, total length of bored piles would be 15.120 meters, and total cost of € 2.268.000,00 excluding the cost of reinforced foundation pad.

The analysis that was performed has shown that consolidation of liquefaction potential by way of constructing gravel drains is an economical procedure.

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