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TEHNIČKO REŠENJE I ANALIZA DISKONTINUIRANE POTPORNE KONSTRUKCIJE OD AB ŠIPOVA ZA ZAŠTITU GRAĐEVINSKE JAME OBJEKTA FLATIRON U SKOPLJU

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Sažetak:

U centralnom području grada Skoplja, firma „Adora inženering“ je projektovala i izvela stanbeno-poslovni objekat visine P+M+15+Pk. U pitanju je jedan od najatraktivnijih objekata ovoga tipa u R. Makedoniji: svojim izgledom podseća na poznati Flat Iron u Nju Jorku, po čemu je ovaj objekat i dobio svoje ime. Za obezbeđenje dovoljnog broja parking-mesta, predviđena je izgradnja dva podzemna nivoa ispod celog objekta, čime se kota fundiranja objekta spušta na -6.8 m ispod površine terena. S obzirom na to da na više segmenata oko konstrukcije su postojeći objekti, te da se na južnoj strani nalazi i gradska saobraćajnica sa autobuskim prevozom i pratećom stanicom koja doseže do oko 3 m od objekta, nameće se potreba da se na ta mesta projektuju konstrukcije za zaštitu građevinske jame. Na delu pored trotoara na autobuskoj stanici, za taj cilj usvojena je diskontinuirana konstrukcija od livenih AB šipova povezanih AB gredom na gornjem kraju. Izvršen je geostatički proračun prema više klasičnih (analitičkih i grafičkih) i numeričkih (sa konačnim elementima) metoda. Za vreme gradnje vršena je geodetska oskultacija tačaka postavljenih na gornjoj površini vezne grede. Izvršena je analiza rezultata za unutrašnje statičke veličine dobijenih primenjenim metodama, kao i upoređenje rezultata za horizontalno pomeranje najviše tačke šipa. Primećena je relativno velika razlika u rezultatima što se duguje brojnim razlozima, koji su takođe razmatrani u radu.

Ključne riječi:

AB šipovi, analiza, numeričke metode, metoda konačnih elemenata, građevinska jama

TECHNICAL SOLUTION AND ANALYSIS OF DISCOUNTINOUS RC PILE SUPPORT STRUCTURE FOR PROTECTION OF THE EXCAVATION PIT OF THE FLATIRON BUILDING IN SKOPJE

Summary:

In the downtown area of the city of Skopje, the company “Adora Engineering” designed and is building a residential-commercial building with a height of B+M+15+A. It is at the moment one of the most prestigious buildings of this type in R.Macedonia. Its shape resembles the famous Flat Iron in New York which is the reason behind the popular name of this building too. In order to accommodate for the necessary parking places, two sublevels under the whole building are to be build, which takes the foundation level to -6.8m below the current terrain. Considering that there are existing buildings at several segments of the building perimeter as well as busy city street on its south side with a bus stop that comes to about 3m of the building, it is necessary to design appropriate measures for the protection of the construction pit at this locations. As a protection of the construction pit at the location of nearby bus station, a discontinuous RC structure made of RC piles connected at the top with a RC beam is adopted and designed. A geostatical calculation has been carried out using several classical (analytical and graphical) and numerical (finite element) methods. Geodetic auscultation of points placed on the upper surface of the connection beam has been carried out during the construction. An analysis of the results for the sectional forces obtained by the analytical, graphical and numerical methods, as well as the results for the horizontal displacement at the top point of the pile has been carried out. A significant difference among these results has been observed which can be attributed to several reasons which are analyzed in this paper as well.

Key words:

RC Piles, analysis, numerical methods, finite element method, construction pit.

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1. INTRODUCTION

"Adora engineering Ltd.", construction company from Skopje, has designed and is building a residential and commercial building in the downtown area of the city of Skopje, in the "Ivan Kozarov" street. The building is of height of 1 ground floor level, 1 mezzanine level, 15 floors and an attic level. i.e. about 50m above the ground level. Because of the location attractiveness, as well as because of its shape (a shape of an iron), this biulding is known as the Flatiron Building (see Fig.1 and Fig.2). The investor created a design according to which 4403m2 of the location space is used for the building. This imposes a problem for the accommodation of the necessary number of vehicle parking spaces. The designer solved this problem by using 2 subsurface levels under the building. This solutions means that the overall height of the building is -2 subsurface floors + 1 ground floor + 1 mezzanine + 15 floors + attic level. This solution also means that the foundation level of the building is at an elevation of 236.40 m a.m.s.l. i.e. about 8.6 m under the current ground level (approximate elevation of 245 m a.m.s.l.), at which the street of "Ivan Kozarov" is also situated, from which the building will be serviced. Considering the construction pit excavation depth of about 8.80m, the existence of several building along its perimeter, as well as a nearby city bus station nearing to not more than 3m to the building, a need for design of excavation pit protection structures for protection of nearby buildings and the existing street at the location of the city bus station arises.

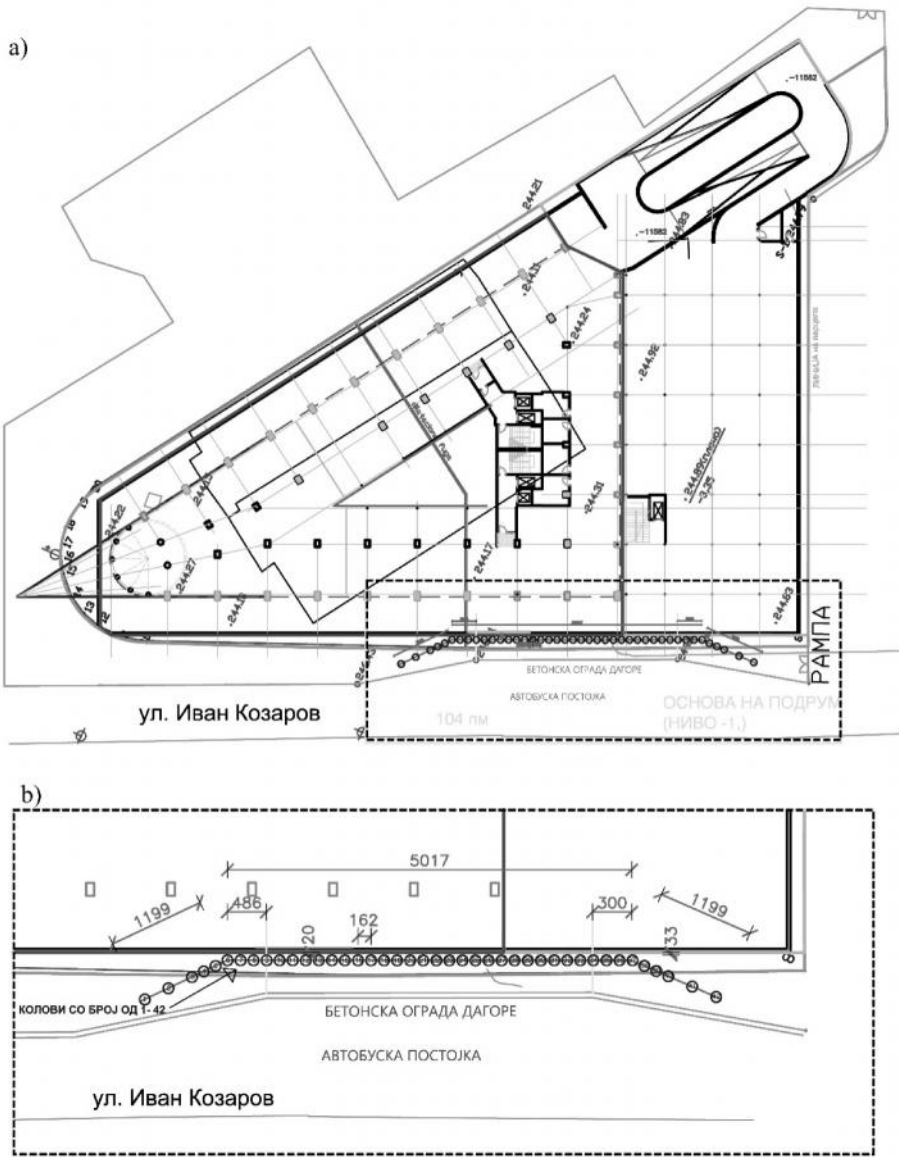

Fig.1 Plan; a) of basement level, level -1; b) of the RC pile support structure

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Fig.2 Spatial representation of the future building and its surroundings

2. GEOTECHNICAL STRUCTURE OF THE LOCATION

For the purpose of the building design, a geotechnical report has been created in March of 2015. In order to create the report field investigation at the location have been carried out, consisting of 5 boreholes with a depth of 20m. Samples for laboratory analyses have been taken from them. Tests of standard dynamic penetration (SPT) have been carried out at the same time, which lead to conclusion that the soil is in medium to good compaction consistency state. The groundwater level in all of the 5 boreholes was detected at about 9-10 m below terrain surface. The samples for laboratory analyses taken from the boreholes were used for soil classification tests as well as soil mechanical properties determination tests.

Based on the field and laboratory testing, it was determined that the location is built of mainly two materials. The first layer with a thickness of 1.70-2.50m over the whole location is determined to be a deposited rubble, below which, to the investigated depth of 20m, a sendy gravel, with medium to good compaction, is to be found. The granulometric composition of the gravel is given in Fig.3.

Laboratory triaxial test of shear strength are conducted only for the layer determined as sandy clay (GP). For dry soil compaction of 20.40-21.62 kN/m^3 and water content of 5.47-9.10%, values of 38.35°-38.98° for the angle of internal friction and 0.00kPa for the cohesion have been determined.

Deformability properties have been calculated in correlation with the results of the standard dynamic penetration test (SPT) [3]. The results of these 8 calculations for the compressibility modulus,, which are concentrated at a depth of about 10m, are in the range of 136-598x102kPa.

Based on the mechanical and deformability properties, an allowable soil bearing capacity of 380 kPa is adopted and the calculated building settlements is 14cm.

3. ADOPTION OF TYPE OF EXCAVATION PIT PROTECTION STRUCTURE

As it was stated in section 1, the considered location and building, as well as their surrounding, impose a need for protection of the excavation pit at several segments of the pit perimeter. During the analyses for the selection of the possible type of structure for the protection of the separate segments of the excavation pit, the segment near the city bus station on the “Ivan Kozarov” street was also analysed. The excavation pit and the bus station at this segment create the greatest conflict. Namely, the buses stopping at the bus station are nearing the excavation pit edge to just about 3m. This condition demands that the supporting structure should be vertical and

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it should take as less as possible space on the location. One positive circumstance here is that the street is fenced from the location and supported with a small retaining wall. This enables excavation and removal of a small quantity of soil that will “decrease” the depth of the excavation pit or the height of its protecting structure (Fig.4). Another positive circumstance is that the groundwater level is lower (233.20m a.m.s.l.) than the foundation level of the building (236.40 m a.m.s.l.). This allows discontinuities in the excavation pit protection structure and demands no water evacuation. The next circumstance that influenced the selection of the excavation pit protection structure is that the construction companies in R.Macedonia have a relatively limited number and type of equipment for execution of excavation put protection structures in absence of groundwater. In the city of Skopje, where at the most part the groundwater is at the depth of 6m or more, the construction companies have gathered considerable experience in excavation pit protection with discontinuous RC pile wall build with RC cast piles mostly of a diameter of Ø800mm or Ø600mm. The RC piles are connected at the top with RC beam which ensures that they work as a whole. They are usually freestanding, i.e. cantilever, cast with sufficient depth below the pit bottom or anchored in the upper parts if the pit is with a greater depth and if the geotechnical and infrastructural surroundings of the excavation pit allows for the anchors to penetrate with enough length in the “neighbourhood” without risk.

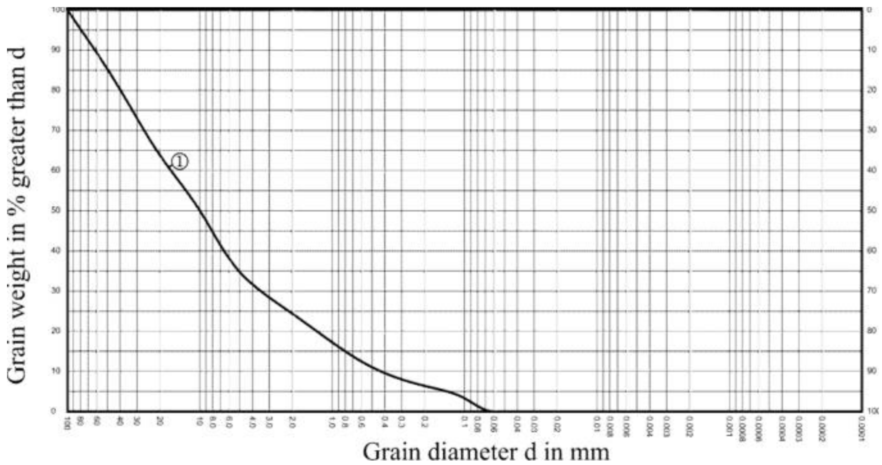

Fig.3 Granulometric composition curve

Considering all of the circumstances stated above, a discontinuous RC pile wall of RC cast piles connected at the top with RC beam was adopted and designed [5] as a protection structure for the segment of excavation pit at the proximity of the city bus station. Because of the high infrastructure density at that location and because of the relatively acceptable depth of the excavation pit, as well as because of the difficulties that usually arise with the installation of geotechnical anchors, the RC pile structure is unanchored. It is secured by anchoring the piles in the soil below the foundation level.

A discontinuous RC structure of RC piles with a diameter of 80cm and distance between individual piles of 1m, connected at the top with a RC beam of 70/50 cm was adopted for the analysis (Fig.4).

4. GEOSTATIC CALCULATION OF THE DISCONTINUOUS RC STRUCTURE

Geostatical calculations are carried out using several methods - classical and numerical for which a geostatic model of the structure was firstly defined (Fig.4). In that order, based on the results of the geotechnical investigations, a lithological build of the halfspace was constructed, the mechanical properties of the soil layers were adopted and at the same time a certain simplification of the influence of the external loads from the bus traffic and from the load of the soil on the part of the excavation between the RC structure and the small retaining wall was made.

As it was previously stated, there was a need for excavation pit protection at several location on the pit perimeter, but in this paper only the analysis of the segment at the location of the bus station on the “Ivan Kozarov” street will be presented. The length of this segment is about 50 m. Analytical, graphical and numerical calculation was carried out during the geostatic analysis of the geostatic model (Fig. 4b).

4.1. Analytical calculation

In the analytical calculation, the load from the bus traffic and the soil above 244.50 m a.m.s.l. are included as an equivalent soil layer (which is on the safety side), and the angle of friction in the soil-concrete interface is taken as 2/3 φ. In this way, the geostatical scheme for the calculation of: (1) The anchoring depth of the RC discontinuous structure and (2) the value of the bending moment would be as it is shown in Fig. 5. The active soil pressure is calculated according to Coulomb and the passive one according to Rankine [1].

The drive depth of the pile wall is obtained from the base overturning condition about point D (Fig.5). For a safety coefficient of 1.2 (for a temporary structure) the necessary anchorage depth is 5.2 m.

The location of the maximal bending moment in the supporting RC structure is at the ordinate z where the sum of the transversal forces vanishes [2]. From this condition a value of 10.1 m for the ordinate z is obtained. A maximal value for the bending moment of 630 kN/m' is obtained at this point. The axial force is 233 kN/m'.

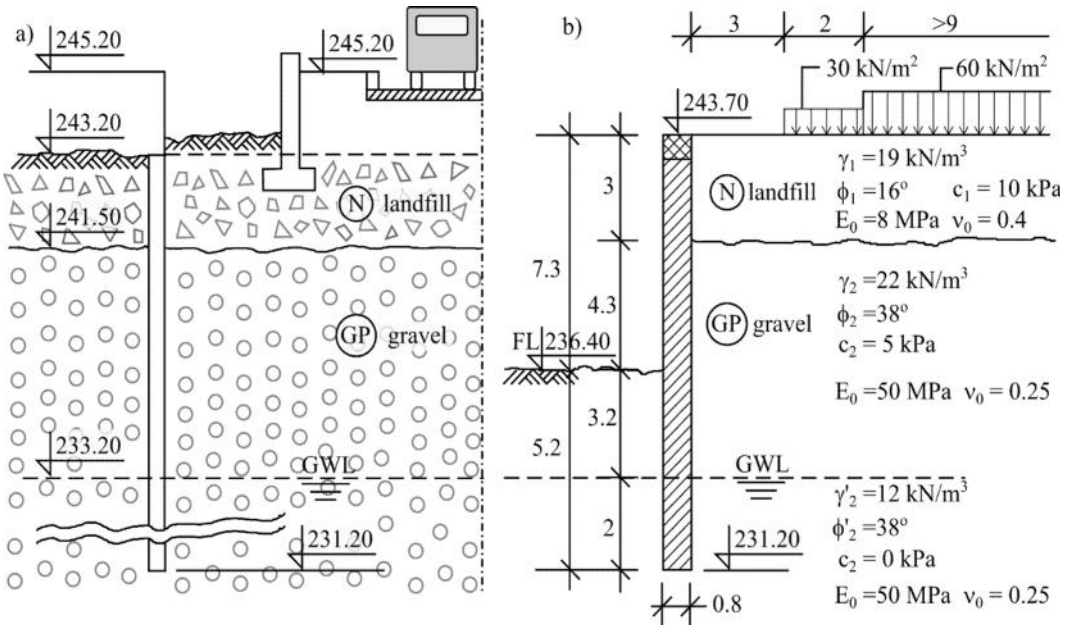

Fig.4 Geotechnical profile of the location (a) and geostatic model of the RC structure (b)

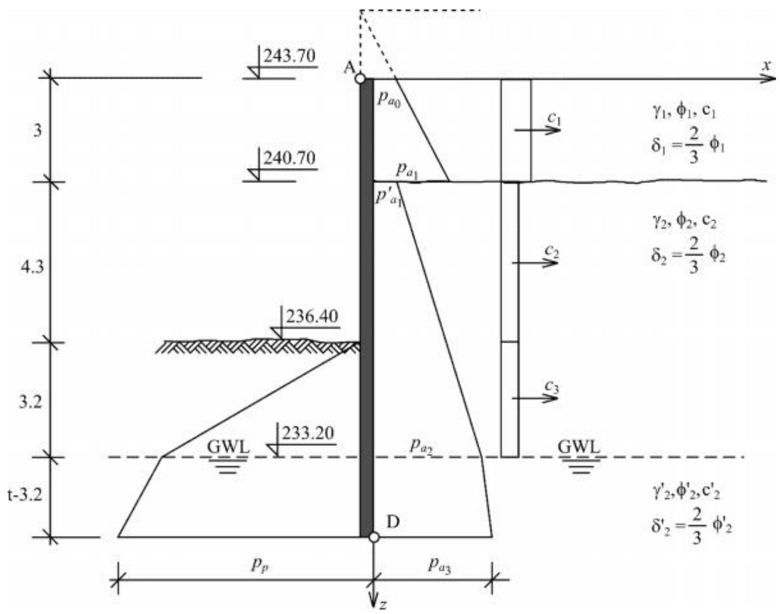

Fig.5 Loading scheme of the structure

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4.2. Graphical calculation

Using the graphical method for the calculation of the pile drive depth and the value of the maximal bending moment [6], values of 5.40m for the drive depth and 684 kN/m’ for the maximal bending moment were obtained.

4.3. Numerical calculations

The finite element method is used for the numerical calculations, i.e. the Plaxis software [3].

For the geostatical calculations using Plaxis, and based on the results of the geotechnical investigations [1], as well as the results of the calculations presented in sections 4.1 and 4.2, a geostatical model presented in Fig.6 was adopted. In order to use the plane strain state calculations, the discontinuous RC pile wall was transformed to an equivalent continuous RC wall with constant thickness and with the same stiffness, that gives an equivalent thickness of the continuous RC wall of 0.62m.

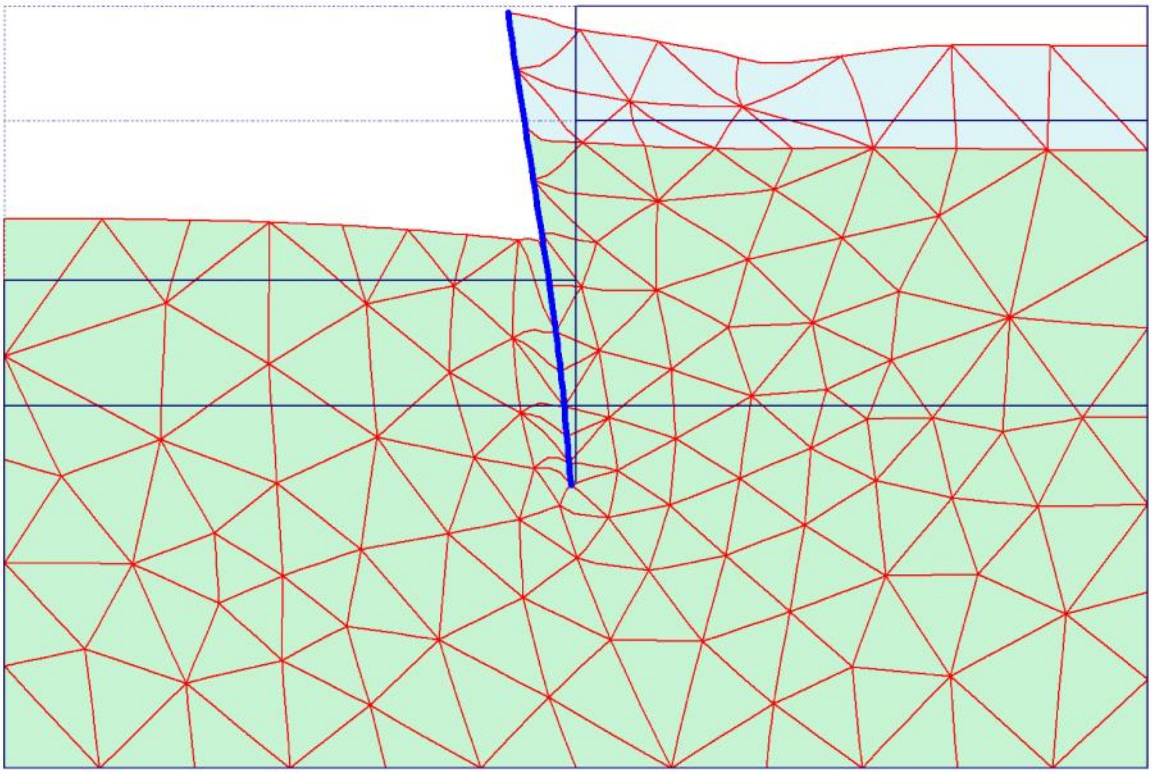

Fig.6 Geostatical calculation model in Plaxis

5. GEOSTATICAL CALCULATIONS RESULTS ANALYSIS

Analysing the results of the conducted analytical, graphical and numerical calculations, presented in section 4, it can be concluded that for the adopted “anchoring” depth of 5.2m, obtained by analytical calculations (section 4.1), which satisfies the overturning condition, significant differences in the values of the moments and the horizontal displacements compared with those obtained by the numerical calculations (section 4.3) can be observed. The values of the maximal bending moments (kNm), maximal axial force (kN) and maximal horizontal displacements (mm) at the point z=0 are given in Table 1. This is with the values of the maximal horizontal displacement obtained by analytical and graphical calculations of a statical system of the RC structure being a cantilever beam with an anchorage at point z where the condition of the vanishing transversal force sum is fulfilled (section 4.1, Fig.5), or the point z where the maximal value in the chain polygon was measured (see section 4.2).

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Table 1. Results comparison between the different calculation methods

	Analytical	Graphical	Numerical-Plaxis
Mmax (kNm)	603	684	372
Nmax (kN)	186	233	230
Displacement (mm)	24	28	36

Considering the results given in table 1, as well as the evaluation of the confidence in and quantification of the results of the geotechnical investigations (especially in the part of the qualitative evaluation of the deformability of the soil halfspace [4], as well as the points stated in section 4 and Figures 4, 5 and 6 of this paper), the dimensioning of the pile of the RC discontinuous structure is carried out using Mmax=603 kNm/m' [5] and axial force of 233 kN/m'.

6. DIMENSIONING OF A SINGLE PILE OF THE RC STRUCTURE

The dimensioning is carried out using the bending moment and the axial force values obtained by the analytical calculations. For a concrete grade of 30 and steel reinforcement RA400/500-2 and load coefficient of 1.3 (for temporary structures) a necessary reinforcement of 58.05cm2 was obtained, and 16Ø20 + 4Ø16 were adopted for the cross section of the maximal bending moment (z=10.10m) and 24.46 cm2 i.e. 8Ø20 for the cross section in the upper third part of the pile (z=4m).

The total amount of reinforcement in a single pile is 512 kg, i.e. in 1 m3 of concrete there is (512/12=) 43 kg reinforcing steel.

7. DISCUSSION OF THE RESULTS OF THE GEOSTATICAL CALCULATION

By analysing the results presented in Table 1, it can be concluded that there are significant relative differences in the internal forces (moments) as well as the horizontal displacements (the values of the axial force are almost the same). If the results of the analytical calculations (section 4.1) are adopted as conditional reference, then for the bending moments these differences are in the range of +13% to -38%. For the horizontal displacements of the top point of the pile (A), i.e. the top surface of the connecting beam these differences are in the range of +16% to +50% (for the axial force these differences are in the range of +24% to +25%). One (smaller) part of these differences are a result of the used calculation methods (adoption of different safety coefficients; appropriate "simplifications" and "rounding" in the calculation phases; certain lack of "optimism" in adoption of certain parameters that (in the literature) are recommended in wide ranges etc.). The other part of these differences is a product of the inconsistencies in the construction and definition of the geotechnical model for the geostatical calculations in the selected calculation methods, that (almost exclusively) depend on the quantitative and qualitative approach in the creation of the geotechnical investigation report.

Having this in mind, and in order to follow the horizontal displacements of the discontinuous RC structure during the building construction, it was considered in the design [2] that 9 geodetic points for geodetic observation (auscultation) to be placed at the top surface of the connecting beam.

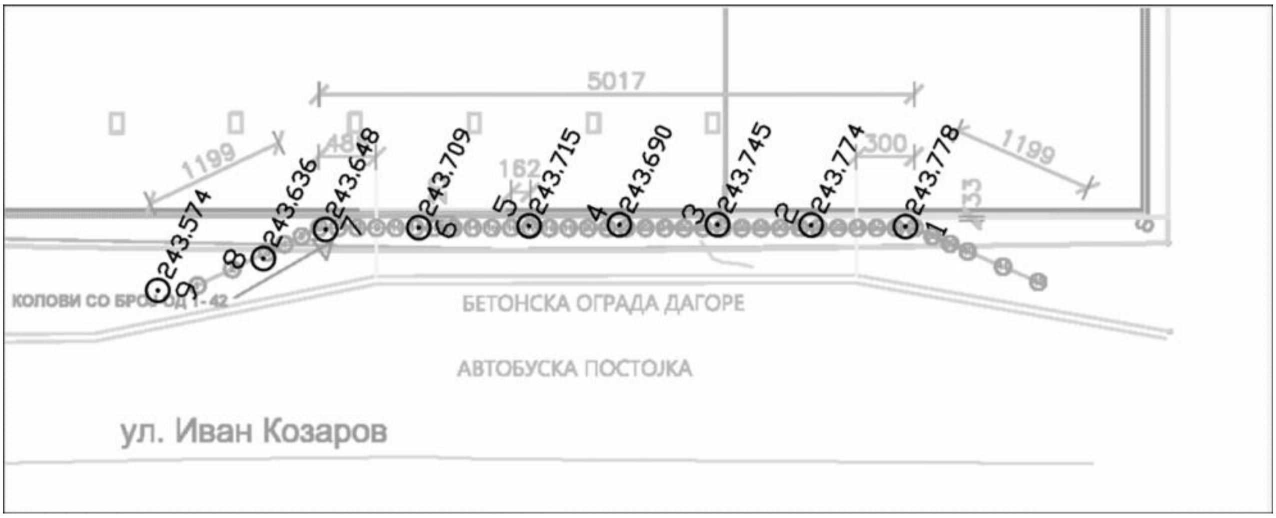

Fig.7. Disposition of the geodetic points

Fig.8 Characteristic photography of the excavated pit

8. BUILDING OF THE DISCONTINUOUS RC STRUCTURE AND ITS GEODETIC AUSCULTATION

The building of the RC support structure i.e. the building of the piles is carried out according to the usual procedure for execution of such type of structures. Namely, first the terrain was leveled to the elevation of 243.20 m a.m.s.l. and a plateau was created for unobstructed work of the mechanization for execution of bored piles. Boring depth is 12m with protection pipe that is removed after concreting. In the boring phase a previously prepared reinforcement cage is lowered, after which the concrete is poured into the pile. The concrete is poured through a fixed pipe (contactor method). After the boring reinforcing and concreting of all of the piles they are connected at elevation of 243.20m a.m.s.l to 243.70m a.m.s.l. With a connecting beam (Fig. 4b).

In order to follow the horizontal displacements of the RC structure during the excavation of the pit, as was stated in the previous section, 9 geodetic points are placed at the top surface of the connecting beam, at elevation of ≃243.7m a.m.s.l. (Fig.7).

The first, i.e. basic reading i.e. the geodetic establishment of the coordinates of these 9 points is carried out on 12.7.2015, and the results of these reading are given in Table 2.

After that date (12.7.2015) the excavation of the pit begun and it was finished in the second half of August 2015 (Fig.8). On 28.8.2015 a second geodetic reading of the 9 points was carried out and its results are given in Table 2.

The last two columns of Table 2 contain the displacement vectors of the points 1 to 9 (in plane and in space). They show that the displacement vectors are in the range of 1 to 9 mm. The largest displacement of 9mm is at point 4 which can be expected.

9. DISCUSSION OF THE RESULTS FOR THE DISPLACEMENT VECTORS OBTAINED BY GEOSTATIC CALCULATIONS AND GEODETIC AUSCULTATION

Analyzing the results of the displacement vectors in the point A (Fig.5) obtained by geostatic calculations (Table 1) and those obtained by geodetic auscultation (Table 2), it can be concluded that there are significant differences. Considering the fact that even the calculated results given in Table 1 are significantly smaller that the allowable displacements for this type of structure (which amount to about 60 mm (if an allowable displacement for cantilever beam of 1/150 to 1/200 of the beam length is adopted)) then it can be concluded that there certainly is sufficient reserve in the design of such type of structures.

Table 2. Geodetic readings and results

	Reading on 28.08.2015			Reading on 12.07.2015			Differences			plane	space
Point	x [536000+]	y [649000+]	z [240+]	x [536000+]	y [649000+]	z [240+]	Δx	Δy	Δz	Δx-y [$\times 10^{-3}$]	Δx-y-z [$\times 10^{-3}$]
1	760.107	098.31	3.773	760.104	098.304	3.778	0.003	0.006	-0.005	6.71	8.37
2	758.060	102.791	3.770	758.059	102.791	3.774	0.001	0	-0.004	1.00	4.12
3	755.978	107.213	3.742	755.976	107.209	3.745	0.002	0.004	-0.003	4.47	5.39
4	753.800	111.832	3.690	753.791	111.831	3.690	0.009	0.001	0	9.06	9.06
5	751.713	116.069	3.714	751.713	116.069	3.715	0	0	-0.001	0.00	1.00
6	749.162	121.24	3.710	749.162	121.233	3.709	0	0.007	0.001	7.00	7.07
7	746.984	125.579	3.650	746.984	125.579	3.648	0	0	0.002	0.00	2.00
8	744.292	127.871	3.637	744.292	127.871	3.636	0	0	0.001	0.00	1.00
9	740.358	132.134	3.573	740.351	132.134	3.574	0.007	0	-0.001	7.00	7.07

10. THE COST OF THE RC DISCONTINUOUS STRUCTURE

The cost of the adopted RC discontinuous support structure is consisted of just 3 positions: (i) Excavation (boring) of the piles (with or without protecting pipe), (ii) preparation of reinforcement cages and their lowering in the bored piles and (iii) concreting of the piles. The cost of 1 m length of the pile or 1 m^2 of the pile wall for excavation pit protection will be given here.

a) Boring of a single pile of 12.50m in length (at every на 1m of the supporting structure there is 1 pile).

12.50 m x 50 €=.................... 625 €

b) Preparation and lowering of reinforcement cage for a single pile, accoring to reinforcement specification given in the Design [2].

512 kg x 0.80 €=................... 410 €

c) Concreting of a single pile

12.50 m x 0.50 m^3 x 80 €=... 500 €

d) Unforeseen expenses

5-10 %................................. 115 €

Total……………………...... 1650 €

The cost of $1m^2$ of the pile wall for excavation pit protection would be:

1650 €/7.30m(the depth of the protected pit) = 226 €/m^2

It can be concluded from the presented calculations that according to the present experience with the use of different methods for protection of excavation pits in urban areas, this price can still be considered as acceptable, especially due to the fact that the construction companies in R. Macedonia have small offering of appropriate technical equipment for execution of these type of works. In this regard, the structural companies acquired (and must have acquired) certain experience. As it was stated in section 3 of this paper, that experience begun to be acquired on the basis of accidental condition as well (the (relatively uniform) geotechnical composition of the terrain, especially at the wider city centre area, relatively low groundwater level (often more than 6m below the terrain surface), the begining of acquirement of necessary equipment for bored piles without protection pipe etc.). All of this contributed to that that the used technical solution for the protection of the excavation pit presented in this paper to be oftenly used at the locations in the dense urban zones in the central city area of Skopje.

11. CONCLUSIONS

Based on the previously stated, the following conclusions can be drawn:

Analysing the results given in Table 1 and Table 2 it may be clearly seen that there are relative differences in the values of the bending moments and the horizontal displacements obtained by the classical and numerical procedures. At the same time there are significant differences in the values of the horizontal displacements obtained by the calculations and those obtained by geodetic auscultations. One part of these differences is due to the used calculation procedures (different safety coefficients; rounding of certain parameters which are given in the literature in wide ranges (for ex. The friction coefficient in the concrete-soil interface) etc.). The other part of these differences is a product of the inconsistencies in the construction and definition of the geotechnical calculation model, that (almost exclusively) depends on the quantitative and qualitative approach in the creation of the geotechnical report.

According to the obtained values for the horizontal displacement in the point A (Fig.5) of 24-36mm using the calculation procedures and only 9 mm obtained by the geodetic auscultation, and comparing them with the allowable horizontal displacement, which can approximately be adopted at 50mm (to max of 60 mm) (obtained as 1/200*L or max(1/150)*L) it can be concluded that a relatively stiff structure has been adopted. That means that a smaller diameter of the piles of 80 cm could have been adopted or the distance between the existing piles could have been increased. In that case, a smaller amount of concrete would have been needed, but the amount of reinforcement would have increased. This means that the structure would not necessarily be cheaper, which would depend on the current prices of the concrete and the reinforcing steel.

The relatively small values of the reinforcement ratio obtained in the calculations given in section 6 of max 1.16% and 0.58% practically confirm the previous conclusion.

The conducted geodetic measurements of the horizontal displacements of the RC supporting structure and their results, which are several times smaller than the allowable ones, infuse optimism in solving future such and similar problems in a direction of lowering the financial cost of these solutions. At the end, the analyses conducted in this study clearly show that geodetic occultations during the building of such structures, as well as the analysis of the results of the geostatical calculations and the measured horizontal displacements during the building should be compulsory.

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