Design project of the structure of a residential building in the city of Castellón street Paseo Ribalta n◦1

ANNEX Nº1: GEOTECHNICAL STUDY

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1. Purpose

The purpose of this annex is to obtain the geological and geotechnical characterization of the area in which the project is located. A geotechnical study was used that requested the Castellón de la plana council for the work of Maset Blau. This report was made in 2006 by the company "Maestrat Global S.L."

With all this, the necessary geotechnical characteristics will be proceeded to estimate and the most suitable types of foundations will be determined.

The actions generated by the earthquake have not been considered in the calculations.

2. Geological location

The area where the site to be studied is geologically located is the 1: 50,000 scale sheet of the IGME nº 641 (Castellón de la Plana).

The city of Castellón is based on quaternary streams made up of red clays with ridges and crusts.

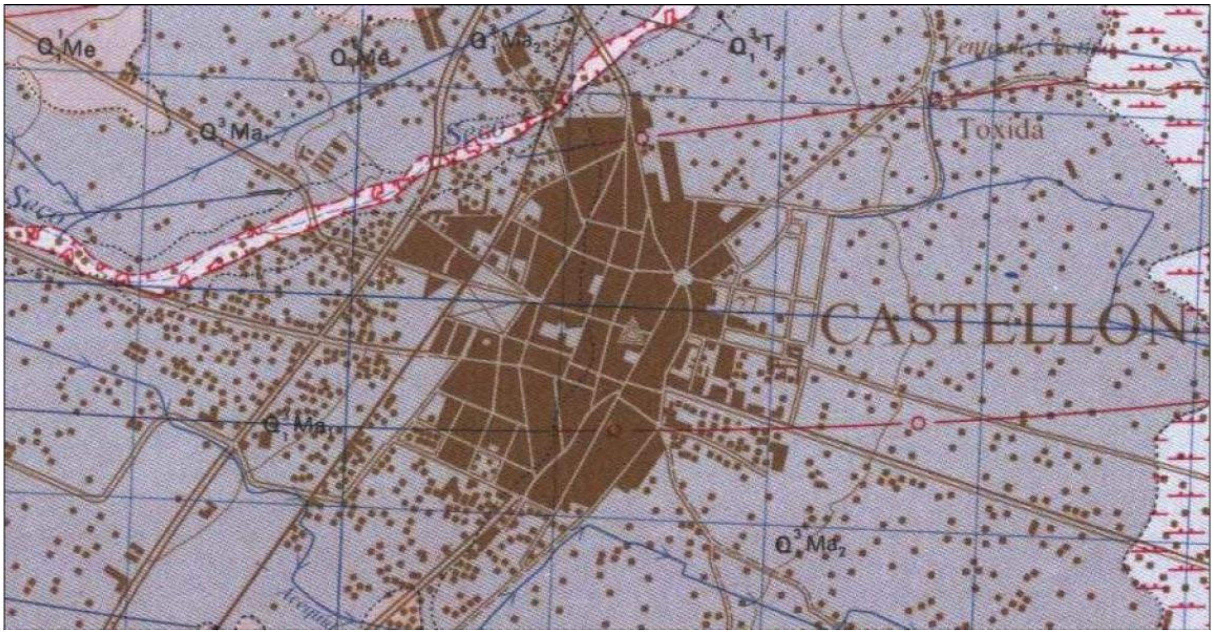

Figure 2.1 Geological map of Castellón de la Plana 1

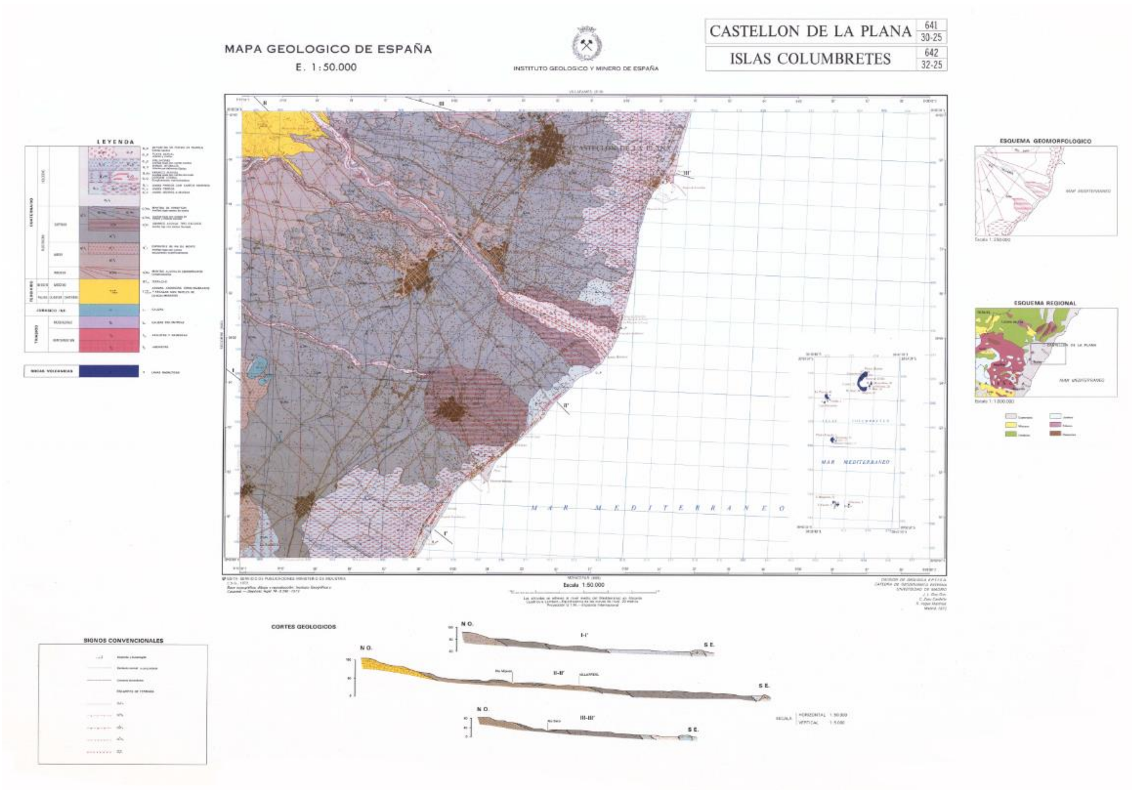

Figure 2.2 Geological map of Castellón de la Plana 2

3. Borings

The field geotechnical campaign consisted four rotating boring with continuous core extraction and two dynamic penetrations.

The probes were drilled using a probe Rolatec RL-46 mounted on tracks. The boring was carried out by means of simple sampler tubes of 101 and 116 mm in diameter.

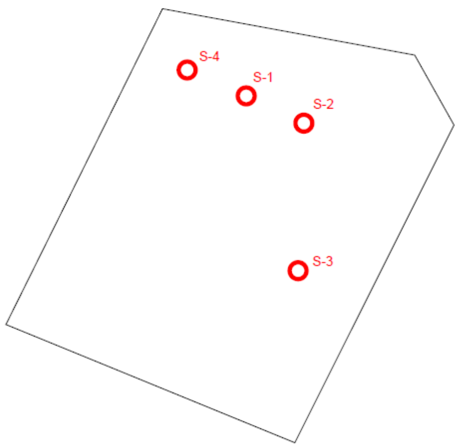

Figure 3.1 Localization of the samples

The depths reached by the surveys are indicated in the attached table:

Table 3.1 Boring depth (m)

Boring Nº	Depth (m)
S-1	6,60
S-2	0,60
S-3	0,60
S-4	0,60

Below there are all the samples taken, their depths, the hits obtained and the norm of application in each case:

Table 3.2 Sample depth (m)

Boring Nº	Sample	Depth (m)	N_{SPT}	Norm
S-1	SPT-1	2,00-2,40	R	UNE-EN ISO 22476-3:2006
	SPT-2	4,00-4,60	33	
	SPT-3	6,00-6,60	11	

The dynamic penetrations were carried out with the super heavy DPSH type.

The rejection depths reached in the penetrations are indicated in the attached table:

Table 3.3 Dinamic penetration depth (m)

Boring Nº	Depth (m)
P-1	2,70
P-2	2,90

4. Groundwater level

In the days in which the field works were carried out and for the depths reached, the presence of groundwater was not observed.

5. Aggressiveness to concrete

Instruction EHE-08 classifies soils detected as non-aggressive towards concrete, and therefore it will not be necessary to use cements that are particularly resistant to sulphates for the manufacture of concrete

6. Description of the leyers

Table 6.1 Layers of the soil

Scale	Lithology	Description	High	Samples	Nº sample (Hit N30)	Water table	Casagrande	Aggressiveness to concrete
0.50 1.00		LEVEL I: CONCRETE (30 cm, sub-level IA), ANTRÓPIC FILLERS constituted by gravels and skittles (20 cm, sub-level IB) and old VEGETAL TERRAIN (70 cm, sub-level IC).	1.20				Nula 0.30	
1.50 2.00 2.50 3.00 3.50 4.00 4.50 5.00 5.50		LEVEL II: SANDY GRAVES, Medium to very dense compactness.	5.80	2.00 2.40 4.00 4.60	SPT-1 (R) 2.00 SPT-2 (33) 4.00		GP-GM 2.00	
6.00		LEVEL III: CLAYS SILT with some Scattered gravel. Orange brown color. Firm consistency	6.60	6.00 6.60	SPT-3 (11) 6.00			

7. Summary of results

In order to obtain the geotechnical data necessary to determine the type of soil where the study structure is going to be based, an analyze of the information from the boring S-1 have been done.

The S-1 survey shows the soil divided into 3 levels.

Level 1 consists of:

-leyer of concrete from the old work from 0,00m to 0,30m

- leyer of gravels from 0,30m to 0,50m

- leyer of vegetal terrain from 0,50m to 1,2m

Level 2 consists of sand-gravel mix from 1,2m to 5,8m

Level 3 consists of clays loams from 5,8m to 6,6m

According to the aggressiveness tests carried out on the soil samples and consulting the EHE-08, it is determined that the% SO3 present is not considered as aggressive for concrete.

At the maximum level of drilling no groundwater level was detected, so concludes that the water will not affect the structure.

Based on the data obtained from the NCSE-02 Earthquake Regulation, it has been determined that, due to that the basic seismic acceleration of the municipality of Castellón is less than what is established, it is not necessary to consider the earthquake for the calculation of the structure.

The underground garage is seted on the hight -3,15m. The layer of the concrete floor of the garage is 15cm and below there is situated 30cm of the balast ground for the concrete. The hight of the spread footing will be between 60-70cm. Taking these information into account the foundation will initially be set on 4,20m with the possibility of changing after a later determination of the preliminary dimensions of the foundation.

From this study the following conclusions are drawn:

Table 7.1 Basic fundation and soil information

Foundation support height	-4,20m
Attack on concrete	No
Affectation of the water table	No
Affection of the earthquake	No

Table 7.2 Characteristic parameter of the soil

h [m]	Leyer	I_D / I_L	ρ_s [t/m^3]	ρ [t/m^3]	γ [kN/ m3]	γ' [kN/ m^3]	C [kPa]	Φ [^{o}]	E_0 [kPa]	E [kPa]	M_0 [kPa]	M [kPa]
1,2 5,8	Sand-gravel mix (Po)	0,75	2,65	1,82	18,2	18,2	0	40,5	22 500	190 000	210 000	210 000
5,8 6,6	Clay (Cl)	0,20	2,72	2,00	20,0	20,0	16	15	57 000	21 000	29 000	48333

8. Spread footing

8.1. Gathering forces for the foundation

The values of forces from the columns were read from the Cype computer program

Table 8.1 Forces in the bottm of the colums

No. of column	N (kN)	Mxx (kNm)	Myy (kN·m)	Vx (kN)	Vy (kN)
P1	1262.10	52.80	-13.20	-14.00	-42.10
P2	2290.50	-19.30	-28.70	-23.40	19.20
P3	1559.60	18.30	6.90	8.00	-11.50
P4	1438.60	16.40	6.40	8.20	-5.80
P5	1868.40	44.70	-45.20	-42.30	-34.00
P6	1049.40	-45.00	-13.80	-14.30	36.00
P7	2156.90	22.40	-16.70	-12.00	-22.10
P8	4427.00	-33.30	-34.20	-20.20	34.30
P9	3737.40	20.10	52.90	47.50	-17.40
P10	4948.40	-6.50	53.20	47.90	7.50
P11	3953.30	-22.10	-8.40	-5.10	17.20
P12	2011.50	-30.50	-17.80	-13.10	31.30
P13	1854.60	36.20	7.50	7.50	-30.60
P14	3550.70	-10.00	11.60	7.20	11.60
P15	3659.30	32.00	1.10	3.00	-22.50
P16	3579.10	-8.50	17.60	13.80	9.40
P17	3124.60	34.20	-1.40	2.40	-24.60
P18	1651.90	-16.90	12.40	10.10	17.10
P19	1840.10	18.60	-23.10	-18.40	-18.10

P20	2827.10	-11.20	17.90	13.50	12.20
P21	2261.60	14.80	-0.80	0.70	-9.60
P22	2407.10	12.40	0.20	1.40	-7.20
P23	2451.40	17.90	-3.50	-1.20	-12.70
P24	1633.70	-10.50	-21.00	-16.10	10.90
P25	1859.90	20.10	16.70	14.20	-19.80
P26	2918.70	-27.80	-18.00	-14.40	22.00
P27	2351.30	15.30	-8.90	-7.30	-9.80
P28	2696.60	21.90	-14.30	-12.10	-12.80
P29	2554.90	19.00	-7.90	-6.90	-13.50
P30	1677.50	-13.10	13.60	11.30	13.50
P31	1857.20	22.10	-26.20	-21.40	-21.20
P32	3718.10	11.80	-11.70	-8.60	-2.30
P33	3321.90	24.00	-6.90	-4.20	-14.30
P34	3494.70	23.00	-5.10	-3.00	-13.40
P35	3226.50	27.00	-2.30	-0.70	-17.30
P36	1428.00	-13.60	-27.10	-19.80	13.30
P37	1385.70	20.00	26.90	24.40	-20.10
P38	2270.50	-7.00	29.00	26.40	6.60
P39	1883.70	-7.50	20.20	17.90	7.10
P40	1669.60	5.80	19.30	17.00	-4.50
P41	1831.50	6.10	21.70	19.20	-4.80
P42	1098.80	-12.20	17.60	15.30	12.90

The following calculations were done for column P1.

8.2. Determining the minimum height of the foundation

The height of the foundation must be greater than the anchorage length of the post reinforcement bars.

Calculation of the anchorage length

Anchorage length l_{bd}:

$$l_{bd} = \propto_1 \cdot \propto_2 \cdot \propto_3 \cdot \propto_4 \cdot \propto_5 \cdot l_{b,rqd}$$

$$\propto_1, \propto_2, \propto_3, \propto_4, \propto_5 = 1{,}0$$

$$l_{b,rqd} = \frac{\phi}{4} \cdot \frac{\sigma_{sd}}{f_{bd}}$$

Przyjęto: $\sigma_{sd} = f_{yd} = 435MPa$

$f_{bd} = 2{,}25 \cdot \eta_1 \cdot \eta_2 \cdot f_{ctd} = 2{,}25 \cdot 1{,}0 \cdot 1{,}0 \cdot 1{,}29 = 2{,}9MPa$

- Column

$$l_{b,rqd} = \frac{2{,}0}{4} \cdot \frac{435}{2{,}9} = 75cm$$

$$l_{bd} = 75cm$$

The horizontal anchorage length of 30cm was assumed so the minimum veritical anchorage lenght is:

$$l_{bd,ver} = 75 - 30 = 45$$

The assumed height of the foundation footing h=60cm.

8.3. The assumed spread footing dimensions and design assumptions

- Sand-gravel mix (Po)
- Humid ground
- Degree of plasticity: $\mathrm{I_L} = 0{,}75 - soft\ plastic$
- Angle of internal friction: $\phi = 40{,}5°$
- Cohesion : c=0
- Weight of the ground: $\gamma_g = 18{,}2\ kN/m^3$

- Dimension B: 2,35m
- Dimension L: 2,35m
- Hight of the spread footing h_f=0,60cm
- Depth of level of foundation D=3,15+0,15+0,30=3,60m
- Decreasing factor $\gamma_R = 1{,}4$
- Column width a=0,35m
- Column length b=0,35m
- Height of the offset h_1=0,30m
- Hight of the concrete floor h_2=0,15m
- Hight of the coulmn h_3=3,15+0,15+0,30+12,89=16,49m

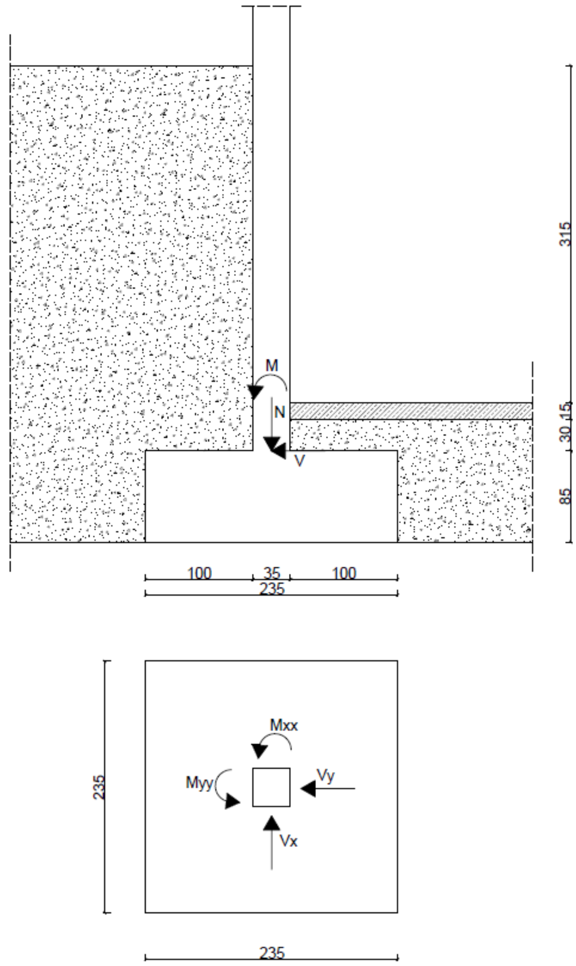

Figure 8.1 Plan and section of the spread footing

8.4. Calculation of load capacity

8.4.1. Calculation of the weight of the ground and of the spread footing

$\mathrm{G_f}$ – weight of the spread footing

$$\mathrm{G_f} = \gamma_\mathrm{f} \cdot \mathrm{B} \cdot \mathrm{L} \cdot \mathrm{h_f} \cdot \gamma = 1{,}35 \cdot 2{,}35 \cdot 2{,}35 \cdot 0{,}60 \cdot 25 = 111{,}87\ \mathrm{kN}$$

$\mathrm{G_g}$ – weight of the ground over the offset and of the concrete floor

$$\mathrm{G_g} = \gamma_\mathrm{f} \cdot \left(\frac{(\mathrm{B-a})\cdot(\mathrm{L-b})}{2} \cdot h_1 \cdot \gamma_g + \frac{(\mathrm{B-a})\cdot(\mathrm{L-b})}{2} \cdot h_2 \cdot \gamma + \frac{(\mathrm{B-a})\cdot(\mathrm{L-b})}{2} \cdot D \cdot \gamma_g\right) = 1{,}35 \cdot \left(\frac{(2{,}35-0{,}35)\cdot(2{,}35-0{,}35)}{2} \cdot 0{,}3 \cdot 18{,}20 + \frac{(2{,}35-0{,}35)\cdot(2{,}35-0{,}35)}{2} \cdot 0{,}15 \cdot 25{,}0 + \frac{(2{,}35-0{,}35)\cdot(2{,}35-0{,}35)}{2} \cdot 3{,}60 \cdot 18{,}20\right) = 272{,}39\ \mathrm{kN}$$

$\mathrm{G_c}$ – weight of the column

$$\mathrm{G_c} = \gamma_\mathrm{f} \cdot \mathrm{a} \cdot \mathrm{b} \cdot h_3 \cdot \gamma = 1{,}35 \cdot 0{,}35 \cdot 0{,}35 \cdot 16{,}49 = 68{,}18\ \mathrm{kN}$$

8.4.2. Statement of forces relative to the base of the foundation

$$\mathrm{N_w} = \mathrm{N} + \mathrm{G_f} + \mathrm{G_g} + G_c = 1262{,}1 + 111{,}87 + 272{,}39 + 68{,}18 = 1714{,}50\ \mathrm{kN}$$

$$\mathrm{M_{cB}} = M_{xx} + V_x \cdot \mathrm{h_1} = 52{,}80 + 14{,}00 \cdot 0{,}3 = 25{,}83\ \mathrm{kNm}$$

$$\mathrm{M_{cL}} = M_{yy} + V_y \cdot \mathrm{h_1} = 13{,}20 + 42{,}10 \cdot 0{,}3 = 74{,}70\ \mathrm{kNm}$$

8.4.3. Calculation of the eccentric

$$e_L = \frac{M_{cB}}{N_w} = \frac{25{,}83}{1743{,}25} \approx 0{,}015m < \frac{L}{6} = \frac{2{,}35}{6} = 0{,}392\ m$$

$$e_B = \frac{M_{cB}}{N_w} = \frac{74{,}70}{1525{,}76} \approx 0{,}037m < \frac{B}{6} = \frac{2{,}35}{6} = 0{,}392\ m$$

8.4.4. Reduced dimensions of foundation

$$B' = 2{,}35 - 2 \cdot 0{,}015\ m = 2{,}32\ m$$

$$L' = 2{,}35 - 2 \cdot 0{,}037\ m = 2{,}28\ m$$

$$A' = B' \cdot L' = 2{,}32 * 2{,}28 = 5{,}28\ m^2$$

8.4.5. Load coefficients

$$N_q = e^{\pi \tan \varphi'} \tan^2(45 + \frac{\varphi'}{2}) = e^{\pi \tan 30{,}8} \tan^2(45 + \frac{40{,}5}{2}) \approx 68{,}41$$

$$N_c = (N_q - 1)\ ctan\ \varphi' = (68{,}41 - 1)\ ctan\ 40{,}5° \approx 78{,}98$$

$$N_\gamma = 2(N_q - 1)\,tan\,\varphi' = 2 * (68{,}41 - 1)\tan 40{,}5° \approx 115{,}06$$

8.4.6. Coefficients of inclination of fundations bases

$$b_\gamma = b_q = (1 - \alpha tan\varphi')^2 = (1 - 0° * \tan 40{,}5\,°)^2 = 1{,}0$$

$$b_c = b_q - \frac{1 - b_q}{N_c * tan\varphi'} = 1 - \frac{1 - 1}{68{,}41 * \tan 40{,}5°} = 1{,}0$$

8.4.7. Coefficients of the shapes of foundations

$$s_q = 1 + \left(\frac{B'}{L'}\right) sin\varphi' = 1 + \left(\frac{2{,}32}{2{,}28}\right) sin40{,}5° \approx 1{,}66$$

$$s_\gamma = 1 - 0{,}3 \cdot \left(\frac{B'}{L'}\right) = 1 - 0{,}3 * \left(\frac{2{,}32}{2{,}28}\right) \approx 0{,}69$$

$$s_c = \frac{s_q N_q - 1}{N_q - 1} = \frac{1{,}66 * 68{,}41 - 1}{1{,}66 - 1} \approx 1{,}67$$

8.4.8. Coefficients of load inclination due to horizontal force

$$m = \frac{\left[2 + \left(\frac{L'}{B'}\right)\right]}{\left[1 + \left(\frac{L'}{B'}\right)\right]} = \frac{2 + 0{,}98}{1 + 0{,}98} \approx 1{,}51$$

$$i_q = [1 - \frac{N_w}{V + A'c'tan\varphi'}]^m = [1 - \frac{1743{,}42}{14{,}00 + 5{,}28 * 0 * tan\;40{,}5°}]^{1{,}51} = 1{,}00$$

$$i_\gamma = \left[1 - \frac{N_w}{V + A'c'ctan\varphi'}\right]^{m+1} = \left[1 - \frac{1743{,}42}{14{,}00 + 5{,}28 * 0 * ctan\;40{,}5°}\right]^{2{,}51} = 0{,}97$$

$$i_c = i_q - \frac{1 - i_q}{N_c tan\varphi'} = 1{,}00 - \frac{1 - 1{,}00}{78{,}98 * tan\;40{,}5} = 1{,}00$$

8.4.9. Calculation of effective stress in the foundation level

$$q' = D * \gamma_g = 3{,}6 * 18{,}2 = 65{,}52\;kPa$$

8.4.10. Calculation of the unit's ground resistance

$$\frac{R}{A'} = c \cdot N_c \cdot b_c \cdot s_c \cdot i_c + q' \cdot N_q \cdot b_q \cdot s_q \cdot i_q + 0{,}5 \cdot \gamma_g \cdot B' \cdot N_\gamma \cdot b_\gamma \cdot s_\gamma \cdot i_\gamma$$

$$\frac{R}{A'} = 0 * 78{,}98 * 1 * 1{,}67 * 1{,}00 + 65{,}52 * 68{,}41 * 1 * 1{,}66 * 1{,}00 + 0{,}5 * 18{,}2 * 2{,}27 * 115{,}06 * 1 * 0{,}69 * 0{,}97 \approx 9080{,}45\;kPa$$

$$R = 9080{,}45 \cdot 5{,}28 = 47950{,}63\ kN$$

8.4.11. Calculation of shear resistance

$$R_d = \frac{R}{\gamma_R}$$

$$R_d = \frac{47950{,}63}{1{,}4} \approx 34250{,}45\ kN$$

8.4.12. Checking the condition

$$V_d \leq R_d$$

$$1714{,}50kN < 3450{,}45kN$$

Condition is met

8.4.13. Checking the stress under the foundation

$$\sigma_1 = \frac{N_w}{B \cdot L}\left(1 + \frac{6 \cdot e_L}{L}\right) = \frac{1714{,}50}{2{,}35 \cdot 2{,}35}\left(1 + \frac{6 \cdot 0{,}037}{2{,}35}\right) = 338{,}75\ kPa$$

$$\sigma_2 = \frac{N_w}{B \cdot L}\left(1 - \frac{6 \cdot e_L}{L}\right) = \frac{1714{,}50}{2{,}35 \cdot 2{,}35}\left(1 - \frac{6 \cdot 0{,}037}{2{,}35}\right) = 282{,}16\ kPa$$

$\sigma_{max} = 350kPa$

$$338{,}75\ \text{kPa} < 350\ \text{kPa}$$

Condition is met

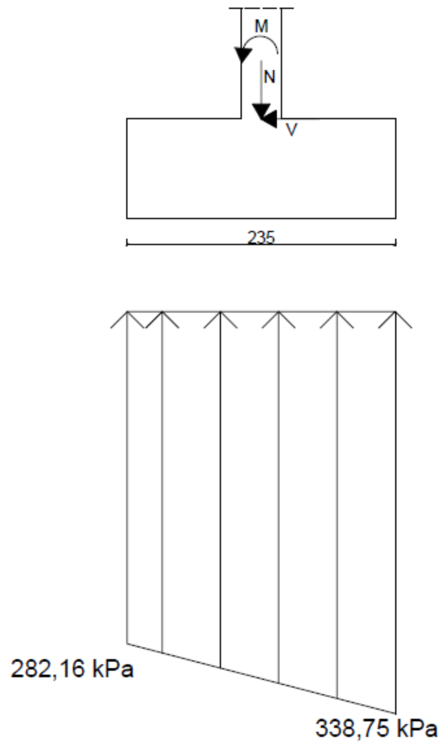

Figure 8.2 Stress under the spread footing

8.5. Grouping and dimensioning of the rest of the spread footing

Analogously, the same calculation has been made in Excel for the rest of the spreads footing. The column have been grouped according to forces that works on them, to the position they occupy on the floor and according to the dimensions of the columns.

The spread shoes where calculated for the biggest normal forces of each group of column. This assumption has been adopted to stay on the safety side of the calculations.

Table 8.2 Group of the columns

No. of group	No. of column	N (kN)	Mxx (kNm)	Myy (kN·m)	Qx (kN)	Qy (kN)
1	P1-P4-P6	1438,6	16,4	6,4	8,2	5,8
2	P2-P13-P21-P22-P23-P27-P29-P38	2554,9	19	7,9	6,9	13,5
3	P3-P5-P39-P40-P41	1883,7	7,5	20,2	17,9	7,1

4	P7-P12-P18-P19-P24-P25-P30-P31	2156,9	22,4	16,7	12	22,1
5	P8-P10	4948,4	6,5	53,2	47,9	7,5
6	P9-P11-P14-P15-P16-P32-P34	3953,3	22,1	8,4	5,1	17,2
7	P17-P33-P35	3321,9	24	6,9	4,2	14,3
8	P20-P26-P28	2918,7	27,8	18	14,4	22
9	P36-P37-P42	1428	13,6	27,1	19,8	13,3

Table 8.3 Dimensions of the spread footing for each group

No. of group	B (m)	L (m)	H (m)	$V_d \leq R_d$	$\sigma_1 \leq \sigma_{max}$
1	2,4	2,4	0,60	$1938{,}71 \leq 37060{,}89$	$337{,}40 \leq 350$
2	3,0	3,0	0,70	$2928{,}40 \leq 61049{,}41$	$330{,}67 \leq 350$
3	2,7	2,7	0,70	$2467{,}98 \leq 47828{,}27$	$344{,}65 \leq 350$
4	2,9	2,9	0,70	$2817{,}80 \leq 56269{,}0$	$342{,}63 \leq 350$
5	4,1	4,1	0,7	$5692{,}24 \leq 122576{,}82$	$342{,}89 \leq 350$
6	3,7	3,7	0,7	$4557{,}12 \leq 98231{,}42$	335,98 ≤350
7	3,4	3,4	0,7	$3802{,}71 \leq 81067{,}48$	$333{,}07 \leq 350$
8	3,2	3,2	0,70	$3403{,}75 \leq 70116{,}57$	$339{,}73 \leq 350$
9	2,4	2,4	0,60	$1873{,}18 \leq 36301{,}55$	$336{,}26 \leq 350$

All the spread footings are connected with each other by tie beams of the dimensions 40cm x 40cm.

8.6. Calculation of settlement

8.6.1. Produced settlement

The calculation of settlement will be done for the spread footing of the biggest dimensions because it can cause the biggest settlement. The spread footing of the group 5 of the dimensions 4,1m x 4,1m will be taken into consideration.

Eurocode 7 recommends use of Schmertmann Method (CPT) for the calculations of the settlement in non-cohesive soils. It's a semi-empirical method, which uses the influence factor of the unit deformation (Iz) and the static penetration test results qc.

Settlement of spread footing using CPT tests according to Schmertmann theory is based on the formula:

$$s = C_1 \cdot \sigma_{ol,neta} \cdot \sum_{1}^{N} \frac{I_{zi} \cdot h_i}{\chi \cdot q_{ci}}$$

Where:

s - settlement of spread footing

C_1 - correction factor for footing depth

$\sigma_{ol,neta}$ - stress in the footing bottom

I_{zi} - strain influence factor at the center of the ith sublayer

h_i -thickness of the ith sublayer

χ - modulus factor

q_{ci} - average value of cone penetration resistance in the ith sublayer

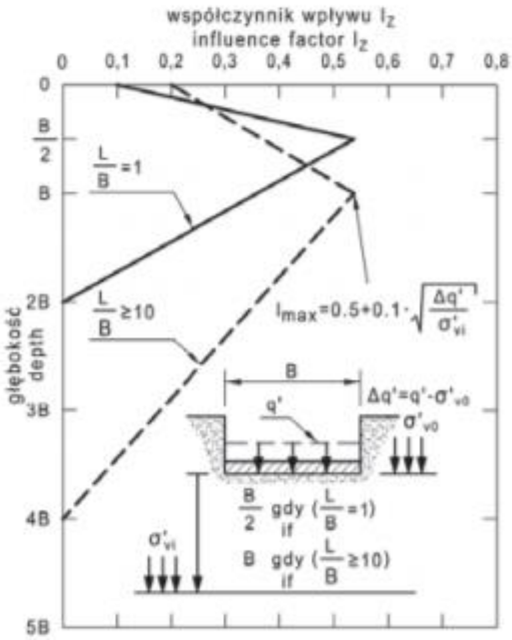

Figure 8.3 Determination of Iz factor (Schmertmann, 1978)

For a spread footing

For $\frac{L}{B} = 1 \rightarrow \chi = 2{,}5$

$$s = C_1 \cdot \sigma_{ol,neta} \cdot \sum_{1}^{2B} \frac{I_{zi} \cdot h_i}{2{,}5 \cdot q_{ci}}$$

$$C_1 = 1 - 0{,}5(\frac{\sigma_{or}}{\sigma_{ol,neta}})$$

σ_{or} - geostatic stress in the footing bottom

$$I_{zp} = 0{,}5 + 0{,}1\sqrt{\frac{\sigma_{ol,neta}}{\sigma_{zp}}}$$

σzp - geostatic stress in the depth of the center of the ith sublayer

The value of $q_{c,}$ which is the average value of cone penetration resistance in the ith sublayer, can be related to the hits obtained N_{SPT} of the standard test by:

Table 8.4 Value of q_c/N for different soils

Type of the ground	q_c/N [kPa/cm^2]
soft clay, peat	2
silt	3

fine silty sand	3-4
medium sand	4-5
gross sand	5-8
gravel	8-12

Table that relates the type of soil and the relationship between the resistance to static penetration and the test standard.

Data:

$$\sigma_{or} = 65{,}52\frac{kN}{m^2}$$

$$\sigma_{ol} = \frac{N_w}{L' \cdot B'} = \frac{5692{,}24}{4{,}08 \cdot 4{,}08} = 341{,}95\frac{kN}{m^2}$$

$$\sigma_{ol,neta} = 341{,}95 - 65{,}52 = 276{,}43\ \frac{kN}{m^2}$$

$$C_1 = 1 - 0{,}5\left(\frac{65{,}52}{276{,}43}\right) = 0{,}881$$

Considered ground is sandy gravel, which doesn't appear in the table that relates the type of soil and the relationship between the resistance to static penetration and the test standard. The lowest value for the gravel will be approved as the closest value to the sandy gravel ground.

For gravel in the layer 1 h=(0m;5,8m) q_c/N=8

$$N = 33\frac{kg}{cm^2} = 3300\frac{kN}{m^2}$$

$$q_c = 3300 \cdot 8 = 26400\frac{kN}{m^2}$$

$$I_{zp} = 0{,}5 + 0{,}1\sqrt{\frac{276{,}43}{2{,}9 \cdot 3300}} = 0{,}517$$

For clay in the layer 2 h=(5,8m;8,2) q_c/N=2

$$N = 11\frac{kg}{cm^2} = 1100\frac{kN}{m^2}$$

$$q_c = 1100 \cdot 2 = 2200\frac{kN}{m^2}$$

$$I_{zp} = 0{,}5 + 0{,}1\sqrt{\frac{276{,}43}{7{,}00 \cdot 1100}} = 0{,}519$$

$$\sum_{1}^{2B} \frac{I_{zi} \cdot h_i}{2{,}5 \cdot q_{ci}} = \sum_{1}^{8{,}2} \frac{I_{zi} \cdot h_i}{2{,}5 \cdot q_{ci}} = \sum_{1}^{5{,}8} \frac{0{,}517 \cdot 5{,}8}{2{,}5 \cdot 26400} + \sum_{5{,}8}^{8{,}2} \frac{0{,}519 \cdot 2{,}4}{2{,}5 \cdot 2200} = 8{,}543 \cdot 10^{-5}$$

$$s = C_1 \cdot \sigma_{ol,neta} \cdot \sum_{1}^{2B} \frac{I_{zi} \cdot h_i}{2{,}5 \cdot q_{ci}} = 0{,}881 \cdot 276{,}43 \cdot 8{,}543 \cdot 10^{-5} = 0{,}0208m = 2{,}08cm$$

$$\boldsymbol{s = 2{,}08cm}$$

8.6.2. Settlement admissible

According to the Eurocode using the formulas indicated above, we should have a maximum settlement of 25mm:

$$s_{max} = 2{,}5cm$$

$$2{,}08cm \leq 2{,}5cm$$

Condition is met

9. Change of spread footing for mat foundation

After putting the dimensions of all the spread footing in Cype it is visible that they occupy more that 50% of the area of the building.

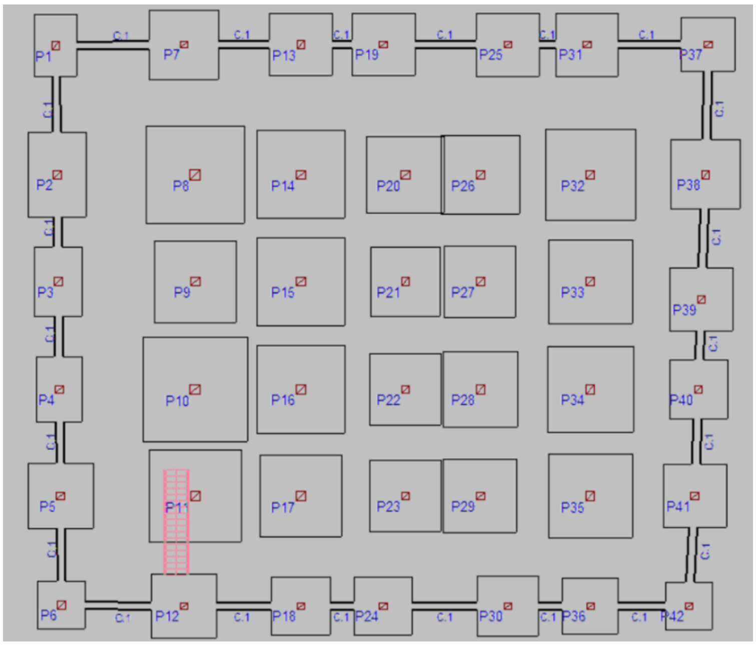

Figure 9.1 Spread footing occupacy of the building area

Therefore it is necessary to change for the mat foundation.

The model of interaction of the ground- foundation-structure is the following:

Flexible foundation, deformation proportional to the pressure transmitted to ground

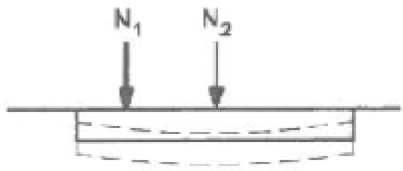

Figure 9.2 Obtained from the presentation of “Losas y emparrillados de cimentación“ from the class “Estructuras de cimentación y contención” of Universitat Politècnica de València

The numerical analysis with soil-structure interaction Winkler model was used to calculate the mat foundation.

The soil is treated as a series of elastic supports on the supporting foundation.

The pressure exerted by the ground follows the law: $P = K \cdot w$

K - ballast coefficient. Ratio between tension applied on a surface and the displacement produced.

w – settlement

For non-cohesive soils:

$$K_{s,rectangular} = 0{,}667 K_{s,square} \cdot (1 + \frac{B}{2L})$$

$$K_{s,square} = K_{30} \cdot [\frac{(B + 0{,}30)}{2B}]^2$$

K_{30} – ballast coefficient for a plate of 30 x 30 cm

$K_{s,rectangular}$ – ballast coefficient for a rectangular slab

$K_{s,square}$ –ballast coefficient for a square slab

B – widh of the slab

L – lenght of the slab

According to CTE Codigo Técnico de la Edificación for the soil compact sandy gravel:

$$K_{30} = \{120; 300\} \, [\frac{MN}{m^3}]$$

K_{30}=135 MN/m^3 – medium gravel with gross sand

B=25,8m

L=30,12m

$$K_{s,square} = 135000 \cdot [\frac{(25,8 + 0,30)}{2 \cdot 25,8}]^2 = 34539\ \frac{kN}{m^3}$$

$$K_{s,rectangular} = 0,667 \cdot 34539 \cdot (1 + \frac{25,8}{2 \cdot 30,12})$$

$$K_{s,rectangular} = 32888\ \frac{kN}{m^3}$$

The value of $K_{s,rectangular}$ was applied to the Cype and the height of the slab of 0,8m was obtained.

The ground bearing pressures was checked. The uplift of the foundation is 0 mm.

Table 9.1 Summary of the foundations

Foundation type	Mat foundation
Fundation height	0,80m
Foundation support height	-4,10m