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Geotechnical Investigation for A Damaged Light Building Erected on A Fill Material-Rich in Expansive Soil

Adil Ozdemir[1]*, Ekrem Kalkan[1,2,3], Duygu Turhan[1]

[1]*Ataturk University, Graduate School of Natural and Applied Sciences, Department of Geological Engineering, Erzurum, Turkey*
[2]*Ataturk University, Graduate School of Natural and Applied Sciences, Department of Nanoscience and Nanoengineering , Erzurum, Turkey*
[3]*Atatürk University, Engineering Faculty, Department of Civil Engineering, Erzurum, Turkey*

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Contact
*Adil Ozdemir
adilozdemir2000@yahoo.com

ABSTRACT

The aim of this study is to investigate the causes of cracks and plaster spills observed in some walls and floor slabs of a light building, to determine whether these damages are caused by the soil, to present solution suggestions and preventive and/or remedial measures that can be taken for the damages caused by the soil, and to prepare the improvement project by evaluating the data obtained as a result of field and office studies for this purpose. For this purpose, soil investigation studies were carried out to reveal the geological and geotechnical properties of the soil on which the light building erected, and to determine whether the cracks and deformations seen in the building are caused by the soil. It was determined that the deep cracks observed in the walls and floors on the south and west sides of the light building were caused by the swelling pressures of the soil.

1. Introduction

The study area is surrounded by Çankaya District in the east, Sincan District in the west, Kazan in the north and Middle East Technical University in the south (Fig. 1). Within the scope of the study, 6 geotechnical boreholes with a total depth of 60 m (10 m each) were drilled and 3 trial pits were excavated in the study area. The Standard Penetration Test (SPT) was performed every 1.50 m in the sections where the soil was crossed in the boreholes and samples were taken. The samples were sent to the laboratory for the necessary tests and a sufficient number of tests were performed to determine the physical and engineering properties of the soils on samples showing typical soil properties and representing different soil levels.

This study covers the geological and geotechnical characteristics of the study area, data of the boreholes and field tests, laboratory tests and results, groundwater status, bearing capacity and settlement calculations, swelling calculations, geological cross-sections, improvement project, conclusions and recommendations.

2. Damages in the Light Building

The light building is a 2-storey reinforced concrete building constructed about 30 years ago. In the following years, a single-storey building was added adjacent to this building to provide services such as recreation and cafeteria. A simplified plan of the current state of these buildings is given in Fig. 2.

There is no damage to the walls or floors of the two-storey building. On the east side of the later single-storey annex building, there is significant plaster blistering, plaster flaking and plaster cracks on the exterior wall of the WC (Fig. 3).

On the north side of the same building, there are similar plaster damages (blistering, flaking and cracking) on the exterior walls of wet rooms such as shower, washbasin and WC (Fig. 4).

On the west and south sides of this building, there are large and small wall and floor cracks, especially on the exterior walls and floors of the spaces used as dining and recreation halls (Figs. 5 and 6). Since the cracks in the walls go beyond the plaster and penetrate into the wall, it is thought that the cracks in these sides are not simple plaster cracks but originate from the soil.

The vertical rain gutters used to collect and remove the rainwater accumulated on the roof of the building discharge the collected water directly onto the pavement and indirectly into the soil of the foundation (Fig. 7). There are also 28 trees on the west side of the building, approximately 2.5-3.0 m away from the wall, spaced 2.5-3.0 m apart and ranging in height from 3-4 m. There are also many trees on other sides.

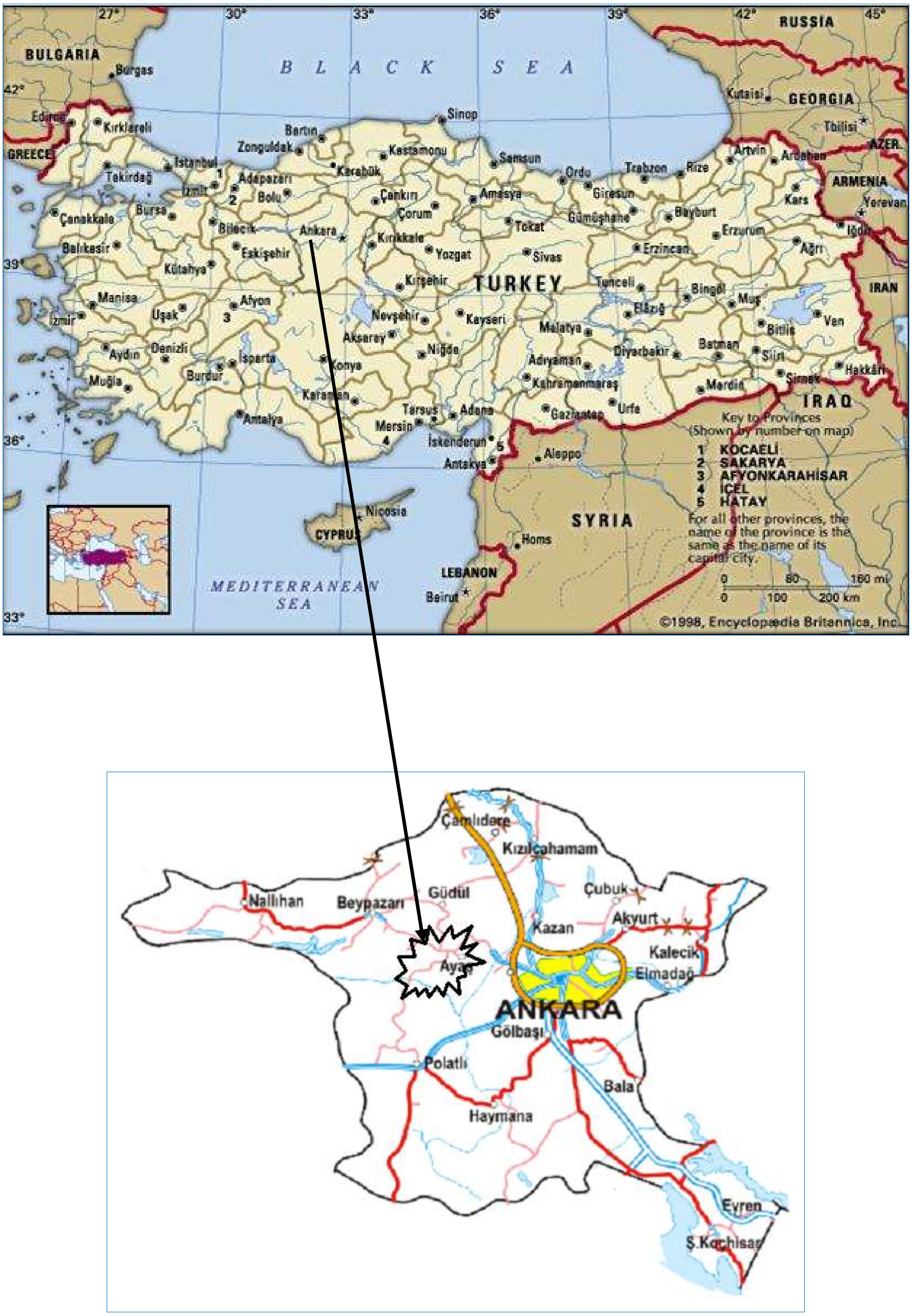

Fig. 1. Location map of the study area

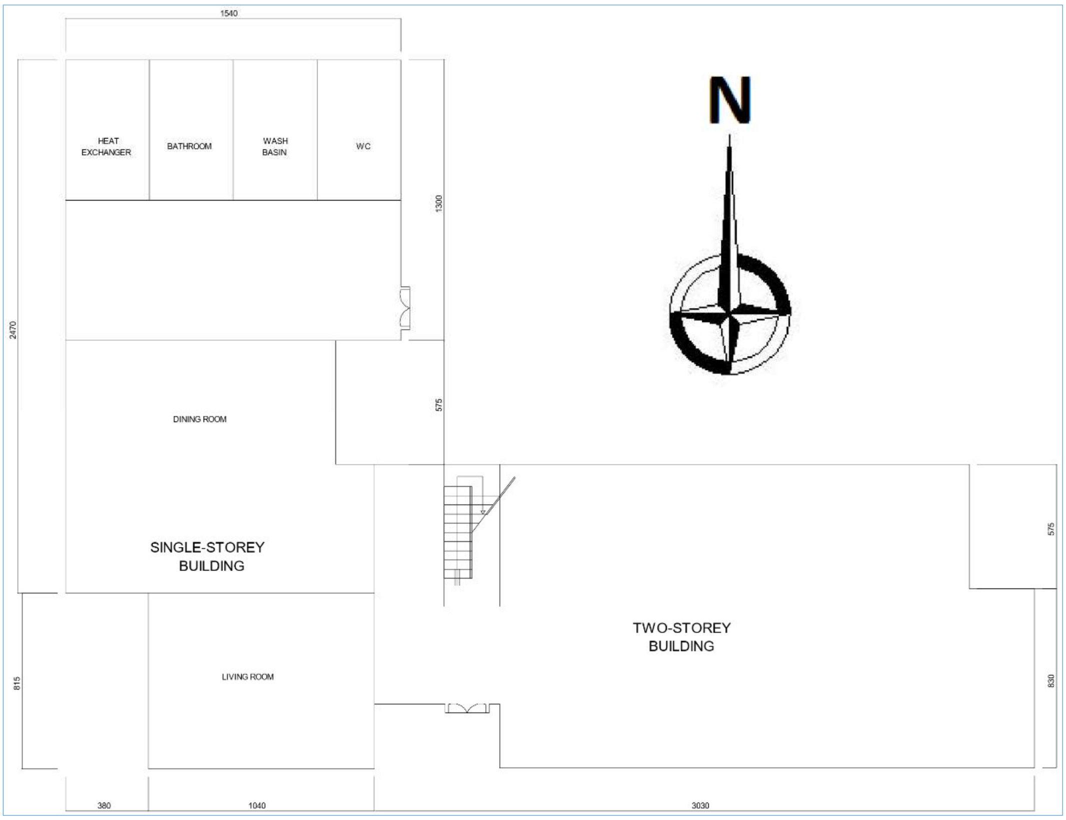

Fig. 2. Simplified plan of the building

3. Geology of the Study Area

Five main lithologic units are macroscopically distinguished within the Ankara urban area. These are epimetomorphic schists, mixed blocky series (Ankara Melange), volcanics and volcanoclastics, river and lake sediments and alluvium (Fig. 8).

Fig. 3. Plaster spills on the east side of the building

The epimetomorphic schists, which constitute the bedrock in the Ankara urban area, outcroped in the southeastern corner of the city where the topography is high and the topographic slope is steep.

This series, defined as “Ankara Melange” by Bailey and McCallien (1950, 1954), which is cleaved by deep valleys in places and which presents outcrops in the southern and southeastern parts of the urban area, has a complex appearance within the urban area, generally consisting of phyllite, greywacke, mudstone, tuffite, limestone and pillowed spilite - basalt blocks.

Fig. 4. Stucco blistering on the north side of the building

Birand (1978) defines the clayey levels as “Ankara Clay” and interprets the lime concretions as follows. These lime concretions were formed in some areas by seeping down from the surface after the surrounding limestones dissolved in the carbonic acid in the water, and in some areas by sedimentation as a result of the partial pressure drop of carbon dioxide gas in the water containing calcium ions during evaporation. On these Pliocene River and lake sediments containing lime concretions in places, brown soils characterizing Central Anatolia are located.

Fig. 5. Floor and wall cracks on the west side of the building

Fig. 6. Wall crack inside the building

According to Erol (1956), these Upper Pliocene River and lake sediments are the products of rivers emptying into shallow lakes in a semi-closed depression basin formed by the collapse of the Ankara Plain. The lake deposits seen in small areas on the andesites in the east and on the groves in the south of the urban area are the remains of the andesite and greywacke topography in the hollow areas at the end of the erosion process.

Fig. 7. Photographs showing rain gutters discharging into the pavement

The most widespread deposits in the Ankara urban area are pinkish purple-colored sediments of volcanic origin. In the southern and southeastern parts of the city area, deposits containing lime concretions originating from grayish and yellow colored greywackes are found. The volcanic and volcanoclastic rocks, generally represented by andesite and occasionally by dacite, which outcrope in the northern and eastern parts of the city area, are generally noteworthy as eroded heights separating low plains (Kasapoğlu, 2000).

The river and lake sediments, which cover about two thirds of the urban area, are generally clay, sand and gravelly levels of varying thicknesses, and occasionally, very fine calcareous levels, lime nodules and/or lime concretions. The clayey, sandy and gravelly levels within these deposits, which do not show a distinct stratification within the urban area, are mostly in the form of lenses of different sizes wedged within each other and they have no lateral continuity (Kasapoğlu, 2000). The alluvium covering the valley floor plains of the major rivers and small tributary streams within the Ankara urban area are Quaternary deposits consisting of gravel, sand and clay. The dry appearance of alluvium is very similar to that of river and lake sediments. However, alluvium has a looser, softer and homogeneous structure (Kasapoğlu, 2000).

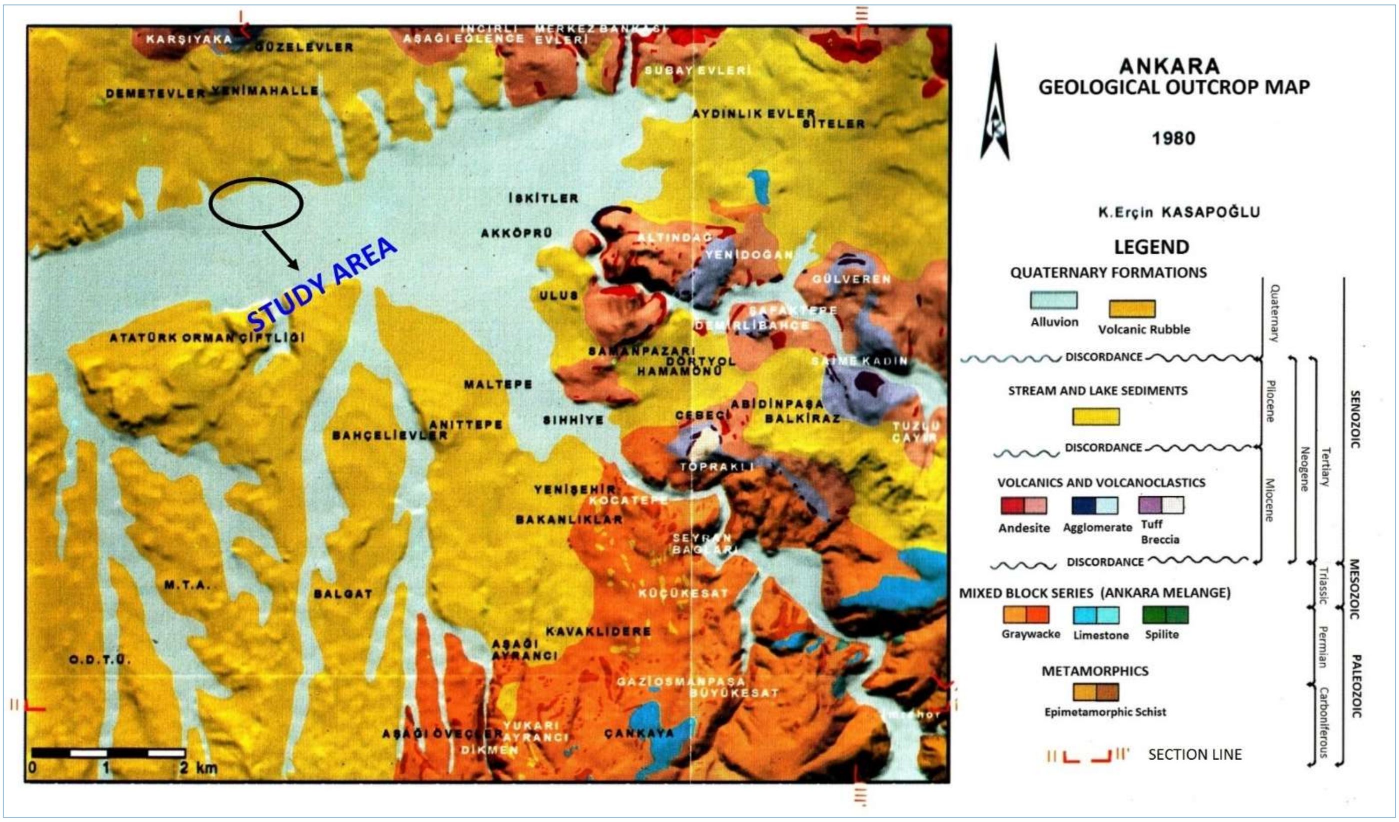

Fig. 8. Geologic outcrop map of Ankara urban area (Kasapoğlu, 2000)

4. Field Studies and Tests

Before going to the field, all previous geological-geotechnical studies in and around the study area were scanned. Field studies were guided in the light of the information obtained from this literature study. Within the scope of the study, geotechnical boreholes were made and trial pits were opened. From the trial pits, soil samples were taken using cylindrical samplers and undisturbed samples were taken for swelling tests. The samples were sent to the laboratory for the necessary experiments and a sufficient number of experiments to determine the physical and engineering properties of the soils were carried out on samples showing typical soil properties and representing different soil levels. As a result of the geotechnical parameters obtained from the study area, engineering evaluation and analysis were carried out.

4.1. Geotechnical Boreholes and Field Tests

The geotechnical boreholes in the study area were carried out to determine the lithological properties of the soils, their changes in the vertical direction, groundwater status and engineering parameters. In order to determine the geological and geotechnical properties of the foundation soil in the study areas, 6 geotechnical boreholes, each with a depth of 10 m, were carried out (Fig. 9).

Fig. 9. A view of the drilling operation

Drilling operations were carried out with a rotary drill rig mounted on a truck. In parallel with the drilling operation, SPT was performed every 1.50 m in order to check the firmness in coarse-grained soils and consistency in fine-grained soils. SPT is essentially an in-situ dynamic shear test.

The shear resistance depends on the relative stiffness of the soil in granular soils and on the strength parameters of the soil in cohesive soils. For this reason, a realistic correlation between the results of the penetration test and the bearing capacity can be established and it is also used in other geotechnical calculations (Ozdemir and Ozdemir, 2006).

The sampler is lowered to the bottom of the borehole and the sampler is pushed into the soil by falling a 63.5 kg hammer from 76.2 cm. The number of blows was recorded for every 15 cm penetration below the bottom of the borehole. The sampler was then advanced by falling the 63.5 kg hammer until 30 cm or 50 blows (excluding placement) were achieved. The number of blows required for the final 30 cm penetration was expressed as penetration resistance, N. If less than 30 cm penetration was achieved after the placement operation, the records represent the number of blows and the distance traveled. In the SPT tests, a donut hammer with 45% energy, which is mostly used in Turkey, was used. The disturbed samples taken during the SPT tests were taken with a 2” outer diameter and 13/8” inner diameter slit sampler. The samples were placed in two separate nested nylon bags, labeled and tied tightly to prevent loss of natural properties.

Groundwater was encountered at a depth of 6.00 m as a result of measurements made in the boreholes during and one day after the drilling operations.

4.2. Trial Pits

In order to determine the engineering properties of the foundation soil, 3 trial pits, each approximately 2.5 m deep, were opened in the investigation area. The locations of the trial pits were determined to coincide with the sides of the building with cracks in the walls. Cylindrical samplers were used to take undisturbed samples at a depth of approximately 1.2 m from 2 trial pits. These samples were sent to the laboratory and subjected to swelling tests. In addition, the disturbed samples were also taken from the investigation pits to determine the physical (index) properties of the soil.

Trial pit-1 was excavated on the west side of the building, near the wall where the cracks were observed. In the first stage, after the pit was excavated to a depth of approximately 0.8 m, undisturbed samples were taken with cylindrical samplers. From the region where the undisturbed samples were taken, the disturbed samples were also taken and placed in nylon bags and labeled and preserved to be sent to the laboratory. After the sampling was completed, the pit was deepened to approximately 2.5 m in order to observe and log the soil profile with the naked eye. As a result of the observations, a light brown colored, very soft topsoil containing tree roots was observed between 0.00-0.30 m, and a sandy, gravelly, clayey, silty, medium solid to solid fill material containing large andesite blocks and brick fragments were observed between 0.30-2.50 m (Fig. 10).

Fig. 10. View of the fill material from Trial pit-1 and undisturbed sampling

Trial pit-2 was excavated on the south side of the building, in the immediate vicinity of the wall where the cracks were observed. The pit was first excavated to a depth of approximately 0.8 m and then undisturbed samples were taken with cylindrical samplers. The disturbed samples were also taken from the same area where the undisturbed samples were taken and placed in nylon bags, and labeled and preserved to be sent to the laboratory. After the sampling was completed, the pit was deepened to approximately 2.5 m in order to observe and log the soil profile with the naked eye.

Fig. 11. Condition of the fill material from Trial pit-2

Fig. 12. Taking undisturbed samples from Trial pit-2

As a result of the observations, a light brown colored, very soft topsoil containing tree roots was observed between 0.00-0.30 m. Between 0.30-1.00 m, a silt and clay-dominated, medium-solid to solid fill material containing large concrete blocks was observed, and between 1.00-2.50 m, a brown-black colored, very solid clay-silt mixture was observed (Figs. 11 and 12).

Trial pit-3 was excavated on the south side of the building in the area where heaving was observed on the pavement. Since the material in this pit contains a large amount of coarse sand and gravel, no sample could be taken. For this reason, the pit was excavated to a depth of 2.5 m in one stage. As a result of the observations made, a light brown colored, very soft, very soft, vegetable soil containing tree roots and occasional brick fragments was observed between 0.00-0.30 m.

Between 0.30-1.00 m, a sandy, gravelly, sandy, gravelly, medium solid-solid fill material with little clay and little silt, containing very large concrete blocks, and between 1.00-2.50 m, a brown-black colored, very solid clay-silt mixture was observed (Fig. 13).

Fig. 13. Condition of the fill material from Trial pit-3

5. Laboratory Tests
The samples were sent to the laboratory for the necessary experiments, and a sufficient number of experiments to determine the physical and engineering properties of the soils

were performed on samples showing typical soil properties and representing different soil levels. Laboratory tests such as water content, sieve analysis, and Atterberg limits were performed on the soil samples taken from the boreholes in the study area to determine the engineering properties of the soils, and soil classes were determined according to the Unified Soil Classification System (USCS) using these data (Table 1 and Fig. 14).

Table 1. Results of index and classification tests on SPT samples

Borehole No	Sample No	Depth (m)	Water Content (%)	No.4 Retained (%)	No.200 Passing (%)	LL (%)	PL (%)	PI (%)	Classification of Soil (USCS)
SK-1	SPT-1	1,50-1,95	4.2	78.2	7.5	27.8	19.3	8.5	GW-GC
SK-1	SPT-2	3,00-3,45	11.7	6.7	63.1	49.3	30.9	18.4	ML
SK-1	SPT-3	4,50-4,95	4.6	13.8	59.0	23.9	15.0	8.9	CL
SK-1	SPT-4	6,00-6,45	13.8	18.8	37.7	27.1	15.2	11.9	SC
SK-1	SPT-5	7,50-7,95	11.2	40.7	13.0	-	NP	-	SM
SK-1	SPT-6	10,00-10,45	13.7	8.1	16.8	-	NP	-	SM
SK-2	SPT-1	1,50-1,95	7.1	34.9	28.7	28.6	18.5	10.1	SC
SK-2	SPT-2	3,00-3,45	24.9	18.0	62.3	37.6	21.8	15.8	CL
SK-2	SPT-3	4,50-4,95	6.5	64.6	1.9	-	NP	-	GW
SK-2	SPT-4	6,00-6,45	17.5	0.4	15.3	-	NP	-	SM
SK-2	SPT-5	7,50-7,95	18.3	1.3	14.3	-	NP	-	SM
SK-2	SPT-6	10,00-10,45	9.1	43.1	6.6	-	NP	-	SP-SM
SK-3	SPT-1	1,50-1,95	3.2	96.1	0.2	-	NP	-	GP
SK-3	SPT-2	3,00-3,45	12.5	65.8	7.5	25.7	14.8	10.9	GW-GC
SK-3	SPT-3	4,50-4,95	10.0	48.5	5.5	-	NP	-	GW-GM
SK-3	SPT-4	6,00-6,45	8.1	51.1	5.6	-	NP	-	GW-GM
SK-3	SPT-5	7,50-7,95	14.2	11.0	18.4	-	NP	-	SM
SK-3	SPT-6	10,00-10,45	9.4	52.7	2.8	-	NP	-	GM
SK-4	SPT-1	1,50-1,95	11.4	55.2	13.3	27.7	19.0	8.7	GC
SK-4	SPT-2	3,00-3,45	24.9	6.9	66.1	41.1	23.5	17.6	CL
SK-4	SPT-3	4,50-4,95	10.9	30.2	34.3	-	NP	-	SM
SK-4	SPT-4	6,00-6,45	26.1	0.5	68.8	-	NP	-	ML
SK-4	SPT-5	7,50-7,95	10.6	37.5	39.4	-	NP	-	GM
SK-4	SPT-6	10,00-10,45	10.9	18.4	2.4	-	NP	-	SP
SK-5	SPT-2	3,00-3,45	12.2	7.6	66.4	39.8	22.5	17.3	CL
SK-5	SPT-3	4,50-4,95	8.7	46.0	3.1	-	NP	-	SP
SK-5	SPT-4	6,00-6,45	14.7	24.7	18.3	-	NP	-	SM
SK-5	SPT-5	7,50-7,95	12.2	50.4	8.0	-	NP	-	GP-GM
SK-5	SPT-6	10,00-10,45	11.0	46.9	15.5	-	NP	-	GM
SK-6	SPT-1	1,50-1,95	14.7	32.2	21.6	28.6	17.9	10.7	SC
SK-6	SPT-2	3,00-3,45	10.9	48.6	22.3	27.4	17.6	9.8	GC
SK-6	SPT-3	4,50-4,95	25.8	8.9	67.6	40.9	17.0	17	CL
SK-6	SPT-4	6,00-6,45	22.2	3.7	72.4	-	NP	-	ML
SK-6	SPT-5	7,50-7,95	11.6	31.2	36.7	-	NP	-	SM
SK-6	SPT-6	10,00-10,45	14.4	9.4	3.6	-	NP	-	SP

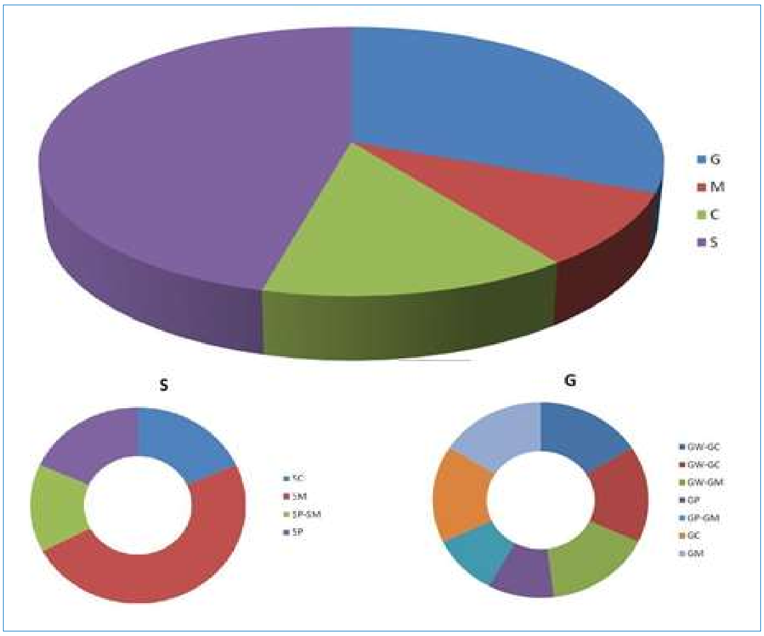

Fig. 14. Soil class distribution according to the results of the tests (USCS) performed on the samples taken from the boreholes

Table 2. Results of index and classification tests obtained on the samples

Trial Pit No.	Depth (m)	No.4 Retained (%)	No.200 Passing (%)	Percentage of clay (< 2 µm) (%)	LL (%)	PL (%)	PI (%)	Classification of Soil (USCS)
Pit-1	1.2	15.2	50.3	14.7	58.1	30.5	27.6	CH
Pit-2	1.2	8.7	67.2	24.0	67.3	31.3	36.0	CH

Table 3. Results of swelling tests obtained on undisturbed samples

Trial Pit No.	Depth (m)	Swelling percentage (%)	Swelling pressure (kPa)	Natural (initial) water content (%)	Natural unit volume weight (kN/m^3)	Dry unit volume weight, (kN/m^3)	Saturation (%)
Pit-1	1.2	3.9	37.5	20.4	17.03	14.15	63.6
Pit-2	1.2	6.9	26.3	26.3	15.68	12.42	63.1

Water content, sieve analysis, and Atterberg limits tests were performed on the soil samples taken from Trial pits-1 and 2 and soil classes were determined according to USCS (Table 2). Swelling tests were performed on the undisturbed samples (Table 3).

6. Geotechnical Assessments

Since there is no project of the buildings, there is no clear information on the foundation system. In addition, it is not known that the buildings have 1 and 2 storeys, have no basements, and are reinforced concrete. For this reason, it was assumed that the foundations were strip foundations with a depth of 0.5 meters and a width of 1 meter during the bearing capacity and settlement calculations. Since the structures are approximately 30 years old, only consolidation settlement was considered.

Due to the tightness of the soil, no undisturbed samples could be taken from the boreholes, therefore the soil parameters used in the bearing capacity and settlement calculations were obtained empirically from the SPT results. The SPTs performed in all boreholes were analyzed and the SPT values performed within the depth of influence of the foundations were determined. As a result of this evaluation, the safe SPT value within the depth of influence of the foundation was selected as 18. In the boreholes and trial pits, clay-containing fill soil was detected at a depth of approximately 1 meter from the surface and clayey units were detected below it. Therefore, the soils are accepted to be cohesive.

Empirical relationships proposed by Stroud (1974) and Stroud and Butler (1975) were used to calculate undrained shear strength and coefficient of volumetric compression from SPT, respectively. For use in these relationships, SPT values were converted to 60% energy ratio as suggested by Sivrikaya and Toğrol (2007) and no effective stress correction was made.

According to Stroud (1974), the undrained shear strength (S_u) of a soil can be calculated according to the following equation.

$$S_u = f_1.N \tag{1}$$

From the SPT-N values in Table 4, the f_1 coefficient required to determine S_u are detected (Stroud, 1974). In the calculation of undrained shear strength to be used in the bearing capacity calculations, the conversion coefficient was selected as 5 from Table 4 considering the soil properties.

Table 4. f_1 coefficients required for the determination of S_u (Stroud, 1974)

Plasticity index, PI (%)	f_1 coefficient
> 30	4.2
20 < PI < 30	4 – 5
< 20	6 – 7

According to Stroud and Butler (1975), the volumetric compression coefficient (m_v) of a soil can be determined from the following equation depending on SPT-N values and plasticity index.

$$m_v\ (kPa^{-1}) = 1/(f_2N) \tag{2}$$

The f_2 coefficient in this equation varies depending on the plasticity index of the soil and can be determined from the Table 5.

Table 5. f_2 coefficients required to determine mv (Stroud and Butler, 1975)

Plasticity index, PI (%)	f_2 coefficient
10	800
20	525
30	475
40	450

In the calculation of the volumetric compression index to be used in settlement calculations, the conversion coefficient was selected as 500 from Table 5 considering the soil properties, thus:

mv (kPa^{-1}) = 1/(f_2N) = 1/(500x18) = 0.000111 kPa^{-1} = 0.00111 cm^2/kg

6.1. Bearing Capacity

The bearing capacity was calculated separately by each of the methods of Terzaghi (1943), Meyerhof (1951), Hansen (1970) and Vesic (1975) (Table 6). In the calculations, the angle of internal friction was taken as zero ((ϕ= 0) and the factor of safety (FoS) was taken as 3. The undrained shear strength was taken as 90 kPa (Fig. 15).

$$S_u = 18 * 5 = 90\ kPa \tag{3}$$

As a result of these analyses, it was determined that the lowest factor of safety was given by the Terzaghi (1943) method and therefore the allowable bearing capacity was 174 kPa. As mentioned before, the buildings have 1 and 2 storeys. In residential buildings, a stress of 1.5 tons/m^2 per story can be obtained. Therefore, the average stress transmitted to the ground from the foundations will be 3 tons/m^2, i.e. 30 kPa for a 2-storey building. This value is considerably lower than the calculated allowable bearing capacity. Therefore, no failure of bearing capacity is expected for these structures.

Table 6. Calculated ultimate and allowable bearing capacity values

Bearing Capacity	Terzaghi (1943)	Meyerhof (1951)	Hansen (1970)	Vesic (1975)
q_u (ultimate bearing capacity), kPa	522.00	522.95	568.75	568.75
q_a (allowable bearing capacity), kPa	174.00	174.32	189.58	189.58

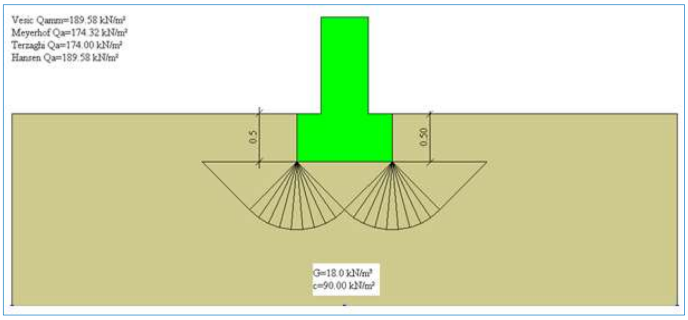

Fig. 15. Bearing capacity analysis, failure mechanism

6.2. Settlement

Due to the net foundation stress applied to the soil by a structure, the cohesive water-saturated soils first undergo rapid sudden settlement and then consolidation settlements occur as the excessive pore water pressure slowly drains. As mentioned before, only consolidation settlement was considered since the structures are about 30 years old.

In the settlement analyses, the foundation system was assumed to be a strip foundation with a depth of 0.5 meters and a width of 1 meter. Considering the foundation dimensions and soil conditions, the compressible soil thickness was selected as 15 meters.

The soil profile was divided into a total of 29 layers with a thickness of 0.5 meters. The additional stress increase caused by the net foundation stress at the midpoint of each layer was calculated using the ½ method (Table 7). In the calculations, the load on the foundation was assumed to be uniformly distributed and the consolidation settlement was calculated by the following formula.

$$S_c = \sum_{i=1}^{n} m_v \, \Delta\sigma_i \, H_i \tag{4}$$

According to these calculation results, the total consolidation settlement is of the order of 1.3 mm. Therefore, it is understood that the deformations observed in the structures in the study area are not due to settlement.

6.3. Liquefaction

Soil liquefaction is the temporary loss of strength of layers below the groundwater level and their behavior as a viscous liquid instead of a solid. In particular, clay-free sand and silt and sometimes gravel layers have the potential for liquefaction. During earthquakes, as the waves, especially shear waves, pass through water-saturated grained layers, they change the grain arrangement and cause the loose grains to settle and compress. During this settlement, if water cannot find a way between the grains and escape, the pore water pressure rises. If this pressure reaches a level close to the weight of the overlying layers, the granular layer temporarily behaves like a liquid and liquefaction occurs. The necessary conditions for liquefaction can be listed as follows.

- Shallow groundwater level,
- Sandy-silty loose soil and
- An earthquake of sufficient magnitude

The liquefaction of a soil depends mainly on its loose arrangement, the bond between the grains, the amount of clay and the inhibition of pore water drainage. The large displacements and deformations that occur in soil liquefaction also depend on the thickness of the liquefied layer, the surface slope and the loading condition.

Table 7. The results of consolidation settlement calculation

Layer No	Layer Depth (m)	Z (m)	Δσ (kPa)	Settlement (mm)
1	0,75	0,25	39,51	0,219
2	1,25	0,75	27,54	0,153
3	1,75	1,25	20,92	0,116
4	2,25	1,75	16,72	0,093
5	2,75	2,25	13,83	0,077
6	3,25	2,75	11,72	0,065
7	3,75	3,25	10,12	0,056
8	4,25	3,75	8,86	0,049
9	4,75	4,25	7,85	0,044
10	5,25	4,75	7,03	0,039
11	5,75	5,25	6,34	0,035
12	6,25	5,75	5,75	0,032
13	6,75	6,25	5,25	0,029
14	7,25	6,75	4,82	0,027
15	7,75	7,25	4,45	0,025
16	8,25	7,75	4,12	0,023
17	8,75	8,25	3,83	0,021
18	9,25	8,75	3,57	0,020
19	9,75	9,25	3,34	0,019
20	10,25	9,75	3,13	0,017
21	10,75	10,25	2,94	0,016
22	11,25	10,75	2,77	0,015
23	11,75	11,25	2,61	0,014
24	12,25	11,75	2,47	0,014
25	12,75	12,25	2,34	0,013
26	13,25	12,75	2,22	0,012
27	13,75	13,25	2,11	0,012
28	14,25	13,75	2,01	0,011
29	14,75	14,25	1,91	0,011
Total Settlement:				1,277

Generally, the liquefaction potential of recent unconsalidated sands and silts in areas with high groundwater levels is high. In addition, sands deposited by rivers have liquefaction potential due to their uniformity in size. The groundwater level closer than 10 m to the surface also increases the liquefaction hazard. On the other hand, if the groundwater level is deeper than 20 m and in dense soils, the liquefaction potential is less. The most susceptible sediments to liquefaction are the sediments deposited as a result of sedimentation processes in Holocene aged deltas, rivers, flood plains, terraces and coastal environments. This is because the dominant depositional processes in these environments cause the grains to be well-sized (grains of almost the same grain size coming together) and loosely deposited. The fact that liquefaction during earthquakes in Turkey is observed in areas with very young alluvial sediments of Holocene age supports this phenomenon. In the boreholes drilled in the study area, there are generally clayey layers up to 4 - 6 meters depth and loose - medium tight and tight sands and gravel units under these units. The groundwater level is 5 meters deep in SK 5 and about 6 meters deep in the other boreholes.

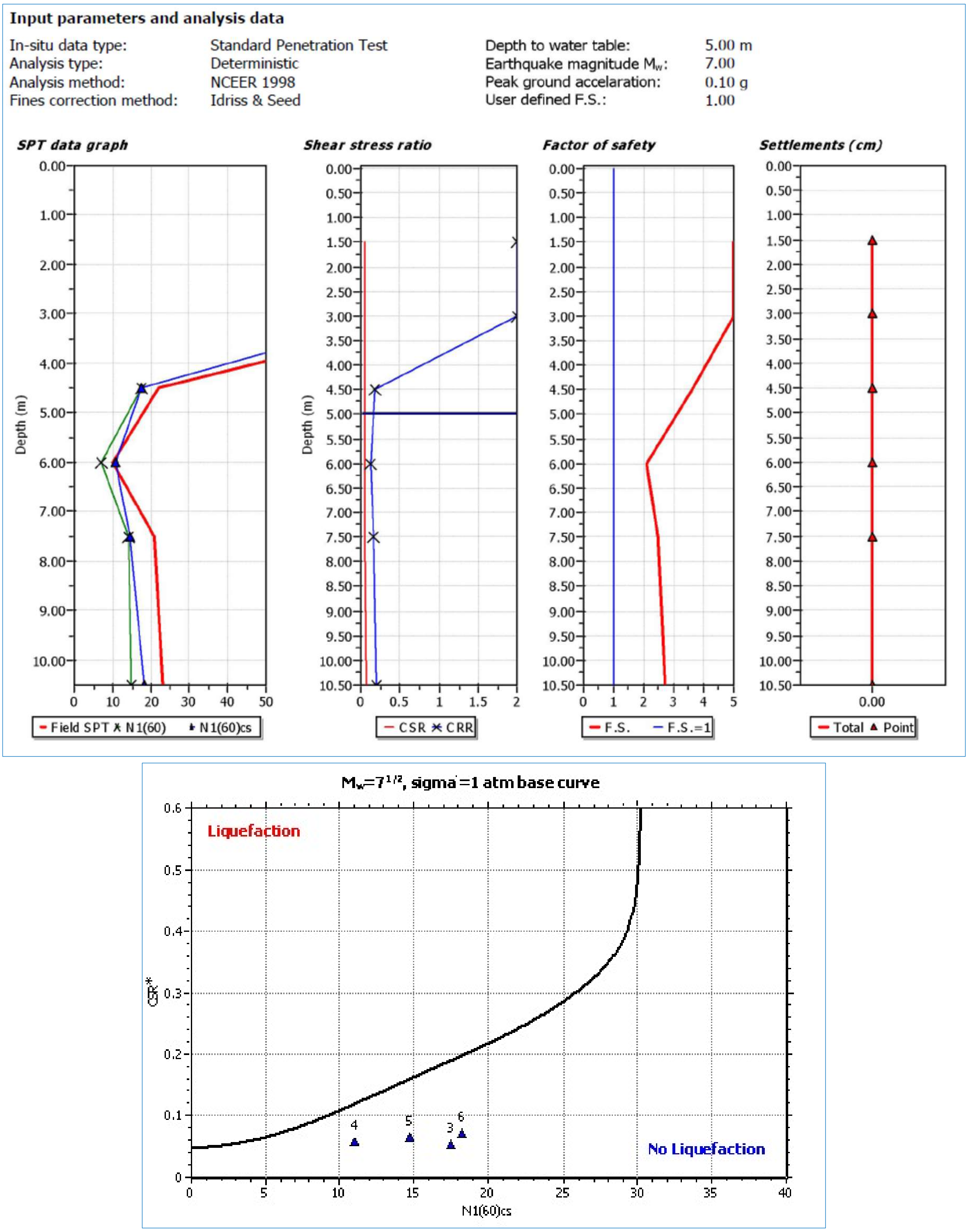

Fig. 16. Liquefaction analysis graphs

Among the boreholes, SK 5 is the borehole with the highest groundwater level and the lowest SPT values in sand - gravel levels. For this reason, the riskiest borehole SK-5 was selected for liquefaction analysis. Since the study area is a 4th degree earthquake zone, 0.1g horizontal ground acceleration was used in the liquefaction analysis and the method recommended by NCEER (1997) was used in the calculations. When Fig. 16 is examined, it is seen that there is no liquefaction in the sand and gravel levels in borehole SK-5, which is considered the most-risky in terms of liquefaction. Therefore, there is no liquefaction risk in the study area.

7. Suggested Improvement Project

As a result of the swelling tests performed on the samples taken from the trial pits, the swelling pressures of the soil were found to be 37.5 and 26.3 kPa. The building with wall and floor cracks is a single storey building and the stress transferred to the ground is approximately 15 kPa.

According to these results, it is understood that the swelling pressure of the ground is higher than the base pressure applied by the building to the ground. In this case, it is concluded that the cracks observed in the interior and exterior walls and floor slabs of the single-storey building are caused by the swelling of the ground. The absence of any cracks in the two-story parts of the building also strengthens this conclusion. Since the swelling pressure of the floor could not exceed the base stress transferred from the two-storey building to the floor, no swelling problem caused by the soil was encountered in these sections.

As a result of the investigations, it was understood that the plaster spills and blistering on the north and east sides of the building were not caused by the soil. The fact that the places where these damages are seen intensively are wet rooms such as WC, washbasin and shower indicate that these damages are caused by water leakages or leaks in the installations.

In addition, the fact that the building is an old building suggests that old type cast iron pipes were used in the plumbing. Old type cast iron pipes are oxidized over time and their cross-sections become thinner and a condition called "sweating" occurs. This idea is strengthened by the fact that the plaster blistering in the horizontal direction seen on the exterior of the building almost coincides with the level where the plumbing pipes pass. Lack of perimeter drainage around the building, rain gutters discharging directly into the foundation soil, and the presence of many trees around the building at frequent intervals are important factors that cause an increase in the water content of the soil and thus trigger swelling of the soil.

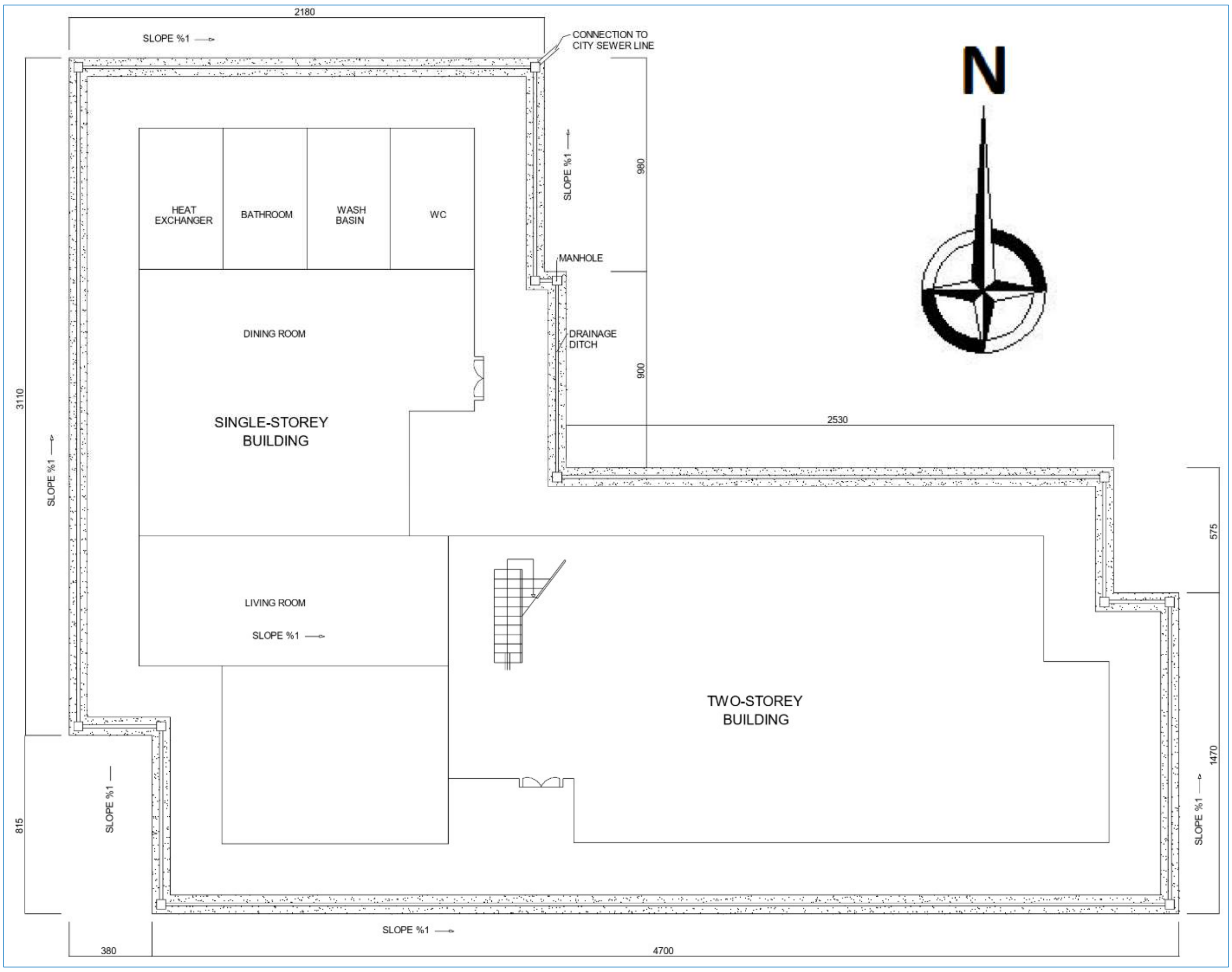

Fig. 17. Suggested perimeter drainage project (length of total drainage line: 180 m)

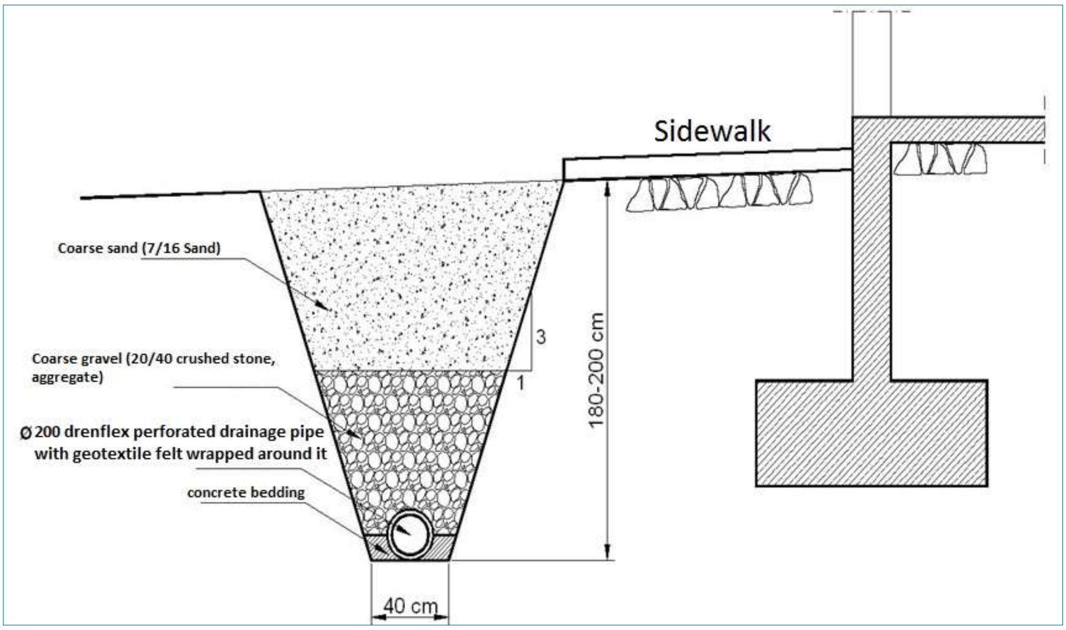

Fig. 18. Detail of the drainage trench (cross-sectional area of the trench is approximately 1.85 m^2)

In order to prevent soil swelling, it is necessary to prevent or minimize the change in water content. Therefore, the measures that can be taken to prevent or minimize the swelling problem in the investigated building can be summarized as; (1) an effective perimeter drainage that goes down to below the foundation level (2) emptying rain gutters into manholes and (3) avoiding excessive irrigation for the purpose of watering the grass or trees around/near the building.

The depth of the perimeter drainage should be designed to be at least 30 cm lower than the foundation depth. However, since the exact foundation depth of the building is not known, it is thought that the depth of the drainage should be planned as 2.0 m (Figs. 17 and 18).

8. Conclusion and Recommendations

In this study, the causes of cracks and plaster spalling observed in some walls and floor slabs of a light building were investigated. For this purpose, the data obtained as a result of field and office studies were evaluated. As a result of the field observations and field/laboratory tests, it was understood that the plaster spills and heaving observed on the north and east sides of the building were not caused by the soil. The fact that these damages were intensively observed on the exterior walls of wet rooms such as WC, washbasin and shower indicates that these damages were caused by leaks or leaks in the plumbing system. In addition, the fact that the building is an old building suggests that old type cast iron pipes were used in the plumbing. Old type cast iron pipes are oxidized over time and their cross-sections become thinner and a condition called "sweating" occurs. This idea is strengthened by the fact that the plaster blistering in the horizontal direction seen on the exterior of the building almost coincides with the level where the plumbing pipes pass.

In order to prevent plaster blistering and flaking caused by defects in the plumbing system, it may be recommended to renew the water pipes, and if cast iron pipes are used, these pipes should be replaced with PVC based water pipes that do not sweat.

As a result of the swelling tests performed on the undisturbed samples taken from the trial pits, it was determined that the swelling pressure of the soil varied between 37.5 and 26.3 kPa. The base stress transferred from the single-storey building to the soil is approximately 15 kPa. According to these data, it was determined that the deep cracks observed in the walls and floors of the single-storey building on the south and west sides were caused by the swelling pressures of the ground. In addition, the vertical rain gutters in the building discharging into the pavement and thus into the foundation soil, the lack of perimeter drainage around the building, and the presence of many trees at frequent intervals near the building have effects that increase the water content of the soil and cause swelling.

In order to prevent and/or mitigate damage to the structure caused by swelling of the soil, it may be recommended to (1) provide effective perimeter drainage down to below the foundation level, (2) discharge rain gutters into manholes, and (3) avoid excessive irrigation of lawns or trees around/near the building.

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