Cracking of RC School Building Due to Soil Expansion

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ABSTRACT

The geotechnical investigations, the structural analyses and the remedial measures of a cracked reinforced concrete school building are presented in this paper. The building is located in Irbid, Jordan, where the soil is highly expansive and the volume change of the soil causes major destruction in the buildings in the area. Field and laboratory tests were performed on the foundation soil of the building to determine its swell potential and other engineering properties. The school building is modeled as a 3-D finite element model using Sap2000 code. The model is built based on results of geotechnical investigation. The results revealed structural deficiencies in building members (columns, beams and footings) due to the swelling gradient. Remedial measures were proposed and implemented to rehabilitate and strengthen the overstressed members. The repaired school has been under service since 2003. The school building performance is being under monitoring since then and has shown reliable performance.

KEYWORDS: Expansive Soil, Finite Element, Reinforced Concrete Structures, Swelling; Cracking, Jordan.

1. INTRODUCTION

Large areas of Jordan soils are expansive soils. The change of volume of the soil during wet and dry seasons brings about serious functional and structural problems in buildings, highway pavements and other engineering structures. Consequently, the volume changing clay soil comprises the most costly natural hazard to buildings in Jordan. In the present paper, a 3-storey school building was rehabilitated against structural damages as a result of being built over highly expansive soil. The building is located in Irbid city, Jordan, 200 m to the north of the Jordanian Central Bank (Baghdad Street) in an area called "Al-Masbaghaniyya". The region's elevation is lower than that of the surrounding area, which results in water accumulation; i.e., basin area. The school building which was built in 1992 is a three-storey building (300 m^2 each) plus a wing for restrooms. The latter are separated from one other by expansion joints. The main building consists of 6 units joined by expansion joints. Figure (1) shows a plan view of the building. The main destruction occurred in the southeastern part of the building. Openings between the columns along the expansion joints were up to 5 cm wide. In addition, cracks inclined at an angle of 45^o with the horizontal (shear cracks) in the walls were observed. No cracks in the columns or beams were observed. The outer walls, allover the perimeter of the building, are made of stone and are considered as bearing walls, while the inside walls are made of hollow blocks and are used as partition walls. The soil investigation report was not available at the time of this investigation. However, the structural design drawings and foundation drawings were available.

Received on 15/7/2007 and Accepted for Publication on 1/10/2007.

Test bits excavated around the exterior walls and boreholes drilled in the site showed that the exterior walls were located on a strip footing located at a depth of 2.5 m to 2.7 m below ground surface.

The southeastern part of the building suffered cracks in the walls over the windows and door corners due to change in water content over the period of the structure life. A crack width up to 5 cm was measured between the expansion joints with a tilting towards the east where the trees are planted around the perimeter (Figure 2). It was in 1999 when the cracks were observed in the building for the first time. In the first year, the cracks were insignificant and were not of concern to students and teachers. Thereafter, the cracks propagated and more tilting appeared to the southeastern part of the building which attracted the attention of the administration. In December 2000 the authors were called by the owner (Ministry of Education) to investigate the causes of the destructions and to propose appropriate remedial measures. The authors visited the building site promptly and the geotechnical investigations, structural analysis and remedial measures presented in this paper were carried out.

2. CLIMATE AND SOIL PROFILE

Jordan is located in a semi-arid area with four distinct seasons in a year. The rainy season begins in November and lasts until April. However, spring is short and followed by a hot dry summer that lasts from May to September. The annual precipitation in Irbid city, north of Jordan, varies from 200 mm to 800 mm. The clay in the eastern part of Irbid lays in an east-west, broad, elongated depression or basin, which runs across the highland plateau area of eastern and southern Irbid. The underlying bedrock is thought to be mostly basalt, which came into the area from north–east as a lava flow during the Tertiary Era. The soil profile consists of 2-3 m of dark grayish brown clay underlained by reddish brown clay down to a depth of 10 m. Then, weathered materials of large rounded boulders of basalt exist just above the clay/basalt interface (Sharif and Stevens, 1983). According to the Unified Soil Classification System, Irbid's clay is classified as high plasticity clay (CH) with a high swelling potential. Due to alternating weather changes from wet to dry, the soil expands during the wet season and shrinks during the dry one, causing damages to buildings and many other structures as well.

3. SITE INVESTIGATION

The site investigation was conducted by excavating four test pits and drilling six bore holes. *In-situ* tests were conducted in an attempt to carry out an integrated study. Undisturbed and disturbed soil samples were extracted and sent to the laboratory for extensive soil testing.

3.1 Test Pits

Four test pits were excavated to depths ranging from 2.57 m to 2.74 m. The locations of the excavated test pits are shown in Figure (1). The profiles of the soil in the four test pits consisted of a mixture of marl, cobbles of crushed limestone marly limestone and chert. A reinforced concrete layer and cement tiles covered the subsurface soils. Water was collected in the four excavated test pits to different levels below ground (tile) surface as outlined below.

3.2 Bore Holes

Six bore holes were drilled with depths ranging from 12 m to 14 m below the existing ground/tile level such that they surrounded the school building. The locations of these bore holes are shown in Figure (1). Five out of the six bore holes were drilled inside the school complex. The sixth borehole was drilled outside. Due to the difficulty of getting the boring machine around its location, a hammer was used for depths more than 9.0 m. Shelby tubes were used to extract undisturbed soil samples. Standard Penetration Test (SPT) was conducted at all drilled boreholes.

3.3 Soil Profile

The soil profile at the school site consisted of two clay layers. The first is the top silty clay layer. This is the main subsurface layer in the site. It is moist and has a stiff to hard consistency with small cobbles of limestone

Table 1: Design of Building Columns.

Column Name	Dimension (mm)	Provided Area of Steel (mm^2)	Required Area of Steel (mm^2)
C1-F1	*300*600*	*1884*	*1800*
C2-F1	*300*300*	*3200*	*1948*
C3-F1	*300*350*	*1964*	*1050*
C4-F1	*300*400*	*1524*	*1200*
C5-F1	*300*400*	*1884*	*1200*
C6-F1	*300*300*	*3200*	*900*
C7-F1	*300*400*	*1884*	*1342*
C8-F1	*300*400*	*1524*	*3282*
C1-F2	*300*600*	*1884*	*1800*
C2-F2	*300*300*	*3200*	*1948*
C3-F2	*300*350*	*1964*	*1050*
C4-F2	*300*400*	*1524*	*1200*
C5-F2	*300*400*	*1884*	*1200*
C6-F2	*300*300*	*3200*	*900*
C7-F2	*300*400*	*1884*	*1342*
C8-F2	*300*400*	*1524*	*1200*
C1-F3	*300*600*	*1884*	*1800*
C2-F3	*300*300*	*3200*	*1948*
C3-F3	*300*350*	*1964*	*1050*
C4-F3	*300*400*	*1524*	*1200*
C5-F3	*300*400*	*1884*	*1200*
C6-F3	*300*300*	*3200*	*900*
C7-F3	*300*400*	*1884*	*1342*
C8-F3	*300*400*	*1524*	*1200*

with brown to reddish color. This layer extended from the surface of ground down to a depth ranging from 10 m to12 m. The second layer encountered at the site was the clayey marl layer. This layer has a low moisture content, mixed with cobbles of limestone and a light yellow color. This layer appeared below the silty clay layer and extended to the end of boring.

4. EXPERIMENTAL INVESTIGATION AND RESULTS

4.1 Soil Properties

In order to assess the swelling behavior of the soil, it was necessary to evaluate its mineralogical composition, in addition to general physical and index properties such as natural water content, unit weight, specific gravity, Atterberg limits and grain size distribution. In Irbid clay samples clay minerals and mica were present. Of the minerals, clay was dominant and the smectite was the most abundant clay mineral. The percentage of sand, silt, kaolinite and smectite for the studied soil were 3%, 38%, 11% and 48 %, respectively.

The physical properties were obtained according to the procedures suggested by (B.S. 1377:75). Results showed that the bulk and dry unit weights of the clay were between

14.8 kN/m^3 to19.4 kN/m^3 and 13.7 kN/m^3 to 17.6 kN/m^3, respectively. The specific gravity of soil solids was around 2.8. Natural water content of the samples varied between 12% and 40%. Figure (3) shows the variation of moisture content with depth for all boreholes drilled. Atterberg limits were determined according to (B.S. 1377: 75, Test No. 2 "A", 3.4) on soil samples extracted from all boreholes. The liquid limit varies from 59% to 84%. The plastic limit varies from 27% to 35% and the plasticity index varies from 34% to 49% for all bore holes tested. According to the Unified Soil Classification System (USCS), the soil was classified as high plasticity clay (CH) with a high swelling potential. Also, according to Williams and Donaldson (1980), the soil of concern is classified as having a high swelling potential. Figures (4) and (5) show the results of Atterberg limits; liquid limit and plasticity index.

Table 2: Design of Critical Building Beams.

Beam Name	Cross Section (mm*mm)	Minimum Steel Area(mm^2) (Code)	Provided Area of Steel (mm^2)		Required Area of Steel (mm^2)	
			Top	Bottom	Top	Bottom
B1-A	250*800	704	565	1005	300	798
B2-A	250*800	704	1005	769	890	414
B3-A	250*800	704	1005	769	1566	890
B4-A	250*800	704	1005	769	1572	890
B5-A	250*800	704	1005	769	872	890
B6-A	250*800	704	565	1005	890	890
B1-B	250*700	616	565	1005	1024	850
B2-B	250*700	616	1272	769	1062	687
B3-B	250*700	616	1272	769	1730	822
B4-B	250*700	616	1272	769	1723	620
B5-B	250*700	616	1272	769	950	810
B6-B	250*700	616	1964	616	1555	780
B7-B	250*500	440	1964	616	2060	1350
B8-B	250*500	440	616	1964	2212	1100
B1-C	250*700	616	565	1005	2500	1160
B2-C	250*700	616	1272	769	2069	972
B3-C	250*700	616	1272	769	1886	891
B4-C	250*700	616	1272	769	1892	895
B5-C	250*700	616	1272	769	996	965
B6-C	250*700	616	1272	769	1270	760
B7-C	250*700	616	1900	1900	Over stressed	Over stressed
B1-D	250*500	440	461	461	350	250
B2-D	250*500	440	603	603	370	230
B1-E	250*800	704	565	1005	890	898
B2-E	250*800	704	1005	769	890	477
B3-E	200*800	576	1005	769	1566	890
B4-E	200*800	576	1005	769	1570	890
B5-E	200*800	576	1005	769	872	890
B6-E	200*800	576	565	1005	890	904
B2-1	250*475	418	1964	616	1263	597
B3-1	250*475	418	804	1964	701	1672
B1-7	250*475	418	1960	603	927	1478
B2-7	250*650	572	2165	616	1248	610
B3-7	250*650	572	2165	1570	2300	2588
B4-7	250*475	418	1964	603	1680	778

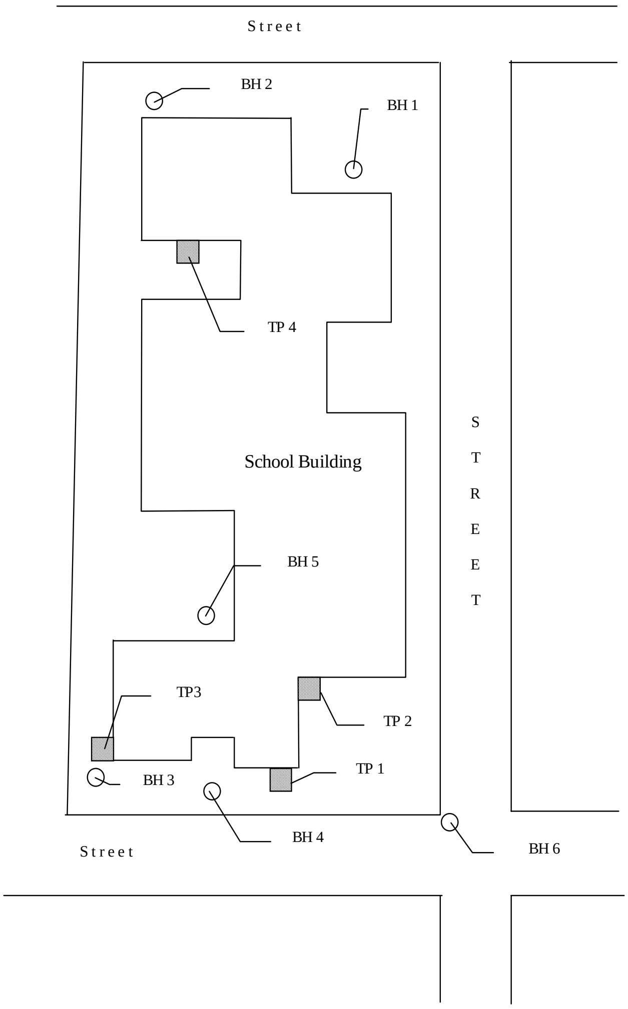

Figure (1): Plan for the school under study with locations of drilled bore holes and test pits.

4.2 Swelling Potential and Swelling Pressure

Swelling potential was determined in terms of swelling pressure, and free swell tests were conducted according to (B.S. 1377). The swelling pressure of the tested samples varies from 9.3 kPa at a water content of 35.9 % and 21.3 kPa at a water content of 28 %. Test results of the free swell for tested soil samples extracted from all drilled bore holes are shown in Figure (6).

4.3 Unconsolidated Undrained Triaxial Compression Test

This test was conducted according to (B.S. 1377) on undisturbed soil specimens extracted from the silty clay layer. The test results showed that the cohesion intercept, c, ranges from 2 kPa to 8 kPa, and the internal friction angle, ϕ, ranges from 8^o to18^o.

5. Field Test Results

Standard penetration test was conducted in all drilled boreholes at the investigated school site. The test result of the SPT showed that the number of blows/ft ranges from (15 to 24) blows/ft.

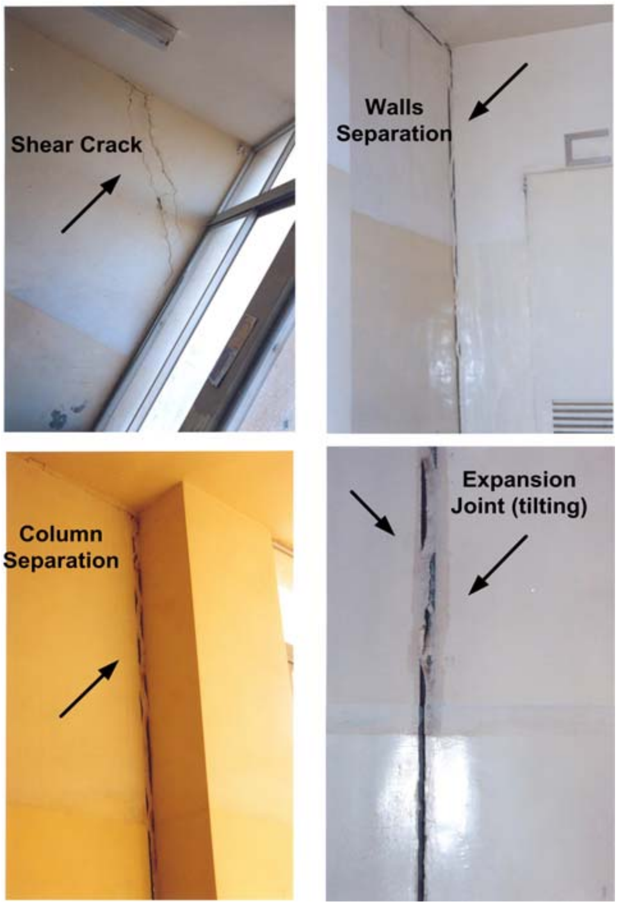

Figure (2): Illustrations of structural cracks and damage.

6. ANALYSIS OF LABORATORY AND FIELD TEST RESULTS

The foundation soil was classified as high plasticity clay (CH) with a high swelling potential according to the Unified Soil Classification System (USCS). The swelling pressure varies between 9.3 kPa and 21.3 kPa. The maximum value 21.3 kPa was measured at a water content of 28% and is much higher than the contact pressure, 15 kPa. The high value of the swelling pressure implies that soil expansiveness is the source of the problem that the superstructure of the school has been experiencing.

The consistency of the clay layer in accordance with the results of the standard penetration test ranges from 15 blows/ft to 24 blows/ft, which indicated that the consistency of the clay layer is very stiff. The results of the UU triaxial test showed that the cohesion is small compared with that obtained from the correlation with the SPT test results. Apart from the triaxial UU test and the unconfined compressive test, the reason for the discrepancy is attributed to the disturbance suffered by the triaxial soil samples during the extraction process. This disturbance caused the soil specimens to crack thus lowering the strength parameters of the tested soil specimens.

7. HEAVE-SHRINKAGE PREDICTION

The wetting-drying cycles of the foundation soil produces heave-shrink of the structure's foundation. The prediction of foundation movements due to the wetting-drying cycle is based on the moisture content-percent

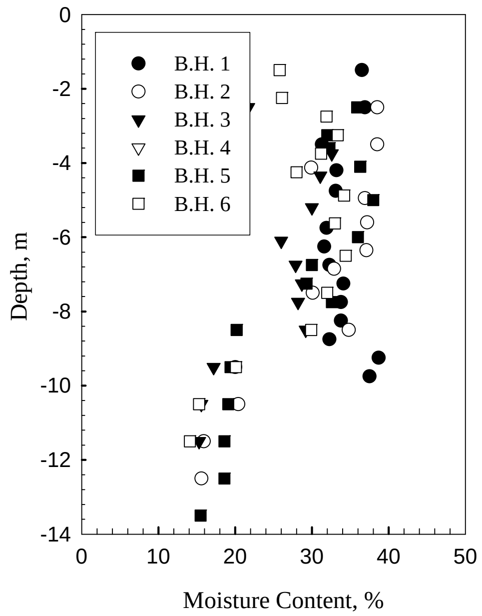

Figure 3: Variation of water content with depth.

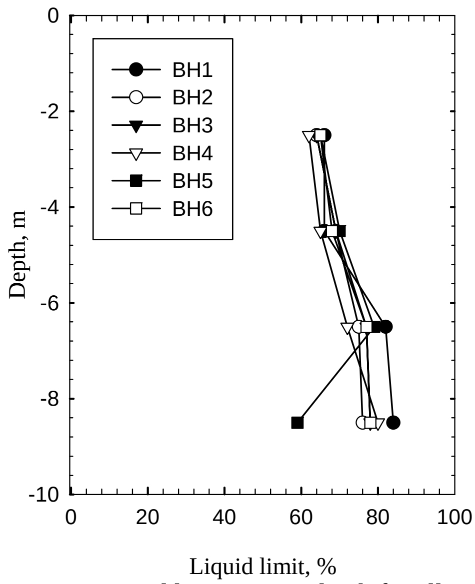

Figure 4:Liquid limit Versus depth for all soil specimen extracted from the six drilled bore holes

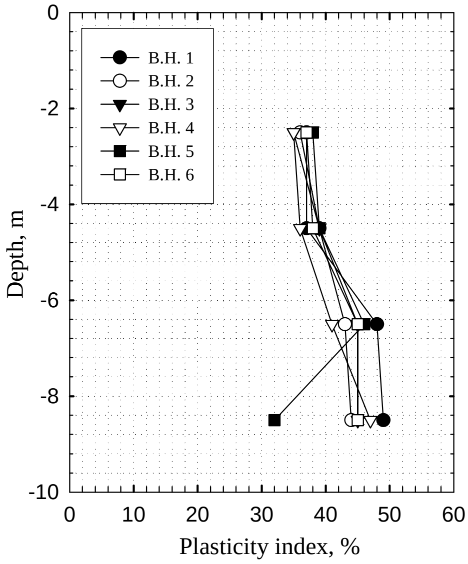

Figure 5: Plasticity index versus depth for all soil specimenvextracted from the six drilled bore holes

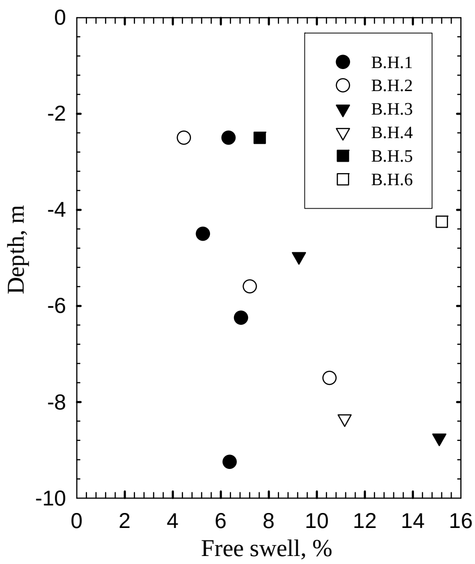

Figure 6: Percent swell Versus depth for all soil specimen extracted from the six drilled bore holes

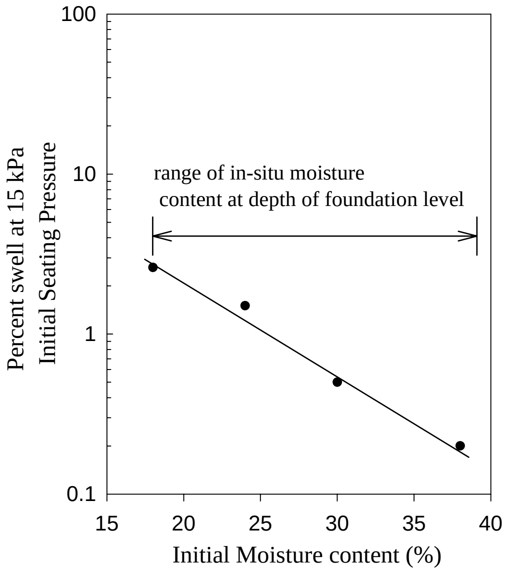

Figure 7: Moisture content versus percent swell obtained from odometer test

swell relationship shown in Figure (7). The data shown were obtained from odometer tests on remolded samples taken from the soil at the foundation level. All samples were tested at a dry unit weight of 1.48 Mg/m^3. The predicted heave for the silty clay layer under the foundation can be predicted by knowing the initial water content. The water content values varied over a wide range as shown in Figure (3). Soil samples were taken in winter and the values at depth more than 8 m were almost constant at a water content of 18 % in all extracted samples regardless of the borehole location. Therefore, the swelling zone depth was 5.5 m below the foundation level. The 18% water content value can be considered as the summer value while the value of 38 % at the top can be considered as the winter value. By using Figure (7) and taking the effective thickness of clay layer below the foundation level as 5.5 m, the maximum vertical heave under an applied pressure of 15 kPa would be 14.3cm.

8. REMEDIAL WORK

As indicated above, the building cracks were due to the cycle of heave and shrink during winter and summer seasons. The theme of remedial work consist of two parts: first, to prevent the cycle of heave and shrinkage by protecting the foundation of the building from water content changes; and second, to increase the stiffness of the building by increasing the cross-sectional area of beams and columns to account for increased bending moment and shear stress that would result due to the possible differential heave. To achieve the first goal, all trees and bushes located at a distance ranging from 1.0 to 1.5 times the height of the tree were removed, the entire area at the entrance of the school building (north side) was covered with compacted non-swelling soil and covered with cement tiles making a positive slope towards the back yard of the west side of the building. Also, the gutters were maintained, facilitating the diversion of runoff away from the building. The structural analysis was carried out by using the current loading systems as well as the new and possible loading due to the structural rotational movements resulting from the heave and shrinkage of the expansive soil that bears the footings. Additionally, soil investigation showed that the maximum relative soil movement due to drying and wetting can reach up to 7.15 cm. For safety considerations, 10 cm was used in the analysis. Therefore, it has been assumed that the far corner of the structure at (1-1, E-E) axes is settled by 20 cm, the corner at (7-7, E-E) axes settled by 10 cm, the corner at (1-1, A-A) axes settled by 10 cm, and the other columns settled relative to these values up to zero value at the opposite corner at (7-7,A-A) axes as shown in Figure (8). This generated a maximum differential settlement of 10 cm.

8.1 Analysis of Beams and Columns

The analysis was performed using several loading cases (ACI 318M, 1995) that are summarized as: Dead load, ***DL***, (1.4DL); Dead load and live load, ***DLLL***, (1.4DL+1.7LL); Dead load, live load and settlement, ***DLLLSTL***, (1.4DL+1.7LL+Settlement). In this study, the Dead Load (DL) value was computed as 7.1 kN/m^2, the Live Load (LL) was considered to be 4.0 kN/m^2 and the settlement values were taken as discussed above and shown in Figure (9).

The building under study was modeled as a 3-D model with releases in the end joints of the columns to assure the simple beam action. The 3-D Finite Element Model is shown in Figure (10). SAP2000 code was used to analyze the finite element model. The analyses produced several cases including the most critical case, which was adopted for the design and repair measures. Figure (11) is presented as an example of the bending moment diagram for the ***DLLLSTL*** case.

As for the structural elements, they were analyzed using the slope-deflection method and the moments were examined for design requirements. The loads were computed by distributing the slab loads. The analysis results and the steel design for the columns and the beams are shown in Table (1) and Table (2), respectively. The analysis and the required steel showed that some of the columns were over-stressed and require a larger cross-sectional area. Specifically, the results showed that column C8 at axis 2-C was potentially overstressed. Figures (12) and (13) show the remedial work suggested to increase the cross-sectional area of column C8. Although column C1 at axis 7-A, axis 7 -E and axis 1-A were not overstressed, a larger cross-sectional area is recommended due to some cracks. On the other hand, some of the beams were under-reinforced and either did not satisfy the minimum steel ratio of 0.004 for concrete compressive strength, $f_c^{'}$, of 20 MPa and steel yield strength, f_y, of 285 MPa or the anticipated settlements that require more reinforcements.

8.2 Analysis of Footing

Based on the structural analysis using the 3 cases of loading, the following results were found:

a. Maximum ultimate axial load of any column along strip footings at axes A-A and E-E was 780 kN.
b. Maximum ultimate axial load of any two adjacent columns along strip footing at axes B-B or C-C was 2445 kN.

Therefore, the strip footing A-A cross-section can be evaluated for stress requirements as:

Allowable pressure = 780/ (1.55*1.3*2.7) = 143 kN/m^2
= 1.43 kg/cm^2 <1.5 kg/cm^2 indicating a safe design.

Additionally, the strip footing B-B cross-section is re-checked for stress requirements as:

Allowable pressure = 2445/(1.55*3.8*2.7) = 153 kN/m^2
= 1.53 kg/cm^2 ~1.5 kg/cm^2, which reveals a safe design within 10%.

Moreover, the strip footing A-A for bunching is evaluated as:

$\boldsymbol{q}_{ult}$ = 780 / 1.3*2.7 = 222.2 kN/m^2.
$\boldsymbol{V}_{ult}$ = 222.22 [3.51 – (0.575*0.675)] = 695 kN.
$\boldsymbol{V}c$ = 0.85/3 $(20)^{1/2}$ 275*[(2*575)+(2*675)] = 871 kN.
$\boldsymbol{V}_{ult} < \boldsymbol{V}c$ *(Safe).*

And the strip footing B-B for bunching is examined:
Note: This check was done after redesigning column (C8).

$\boldsymbol{q}_{ult}$ = 2445 / 3.8*2.7 = 238.4 kN/m^2.
$\boldsymbol{V}_{ult}$ = 238.4 [10.28 – (0.575*0.975)- (0.575*0.675)] = 2225 kN.

$$V_c = 0.85/3\ (20)^{1/2}\ 275*[(4*575)+(2*675)+(2*975)]$$
$$= 1950 \text{ kN.}$$
$V_{ult} > V_c$ *(not Safe).*

8.3 Strengthening of the Structure

8.3.1 Strengthening of Columns

The shaded column (C8) in Table (1) is recommended to be strengthened by increasing the cross-sectional area of the column as shown in Figures (12) and (13). The same procedure is to be used for strengthening the other columns mentioned earlier.

8.3.2 Strengthening of Beams

The in Table (2) shaded beams, which represent the problem beams, are decided to be strengthened by using a steel plate bolted at the top sections and at the bottom sections of the beams as shown in Figures (14) and (15).

8.3.3 Strengthening of Footings

Footing analysis showed that only strip footing B-B is overstressed and needed to be strengthened against bunching. The remedial measures shown in Figure (16) were suggested and implemented.

9. SUMMARY AND CONCLUSION

In the present study, the causes of cracking of a three-storey building in Irbid, Jordan were presented and discussed. The site and laboratory investigations showed that the main geotechnical and structural problems in the building of concern are due to the high swelling potential of the silty clay layer which extends up to a depth of 12.5 m. Geotechnical and structural remedial measures were suggested to stop the heave and shrinkage problem. **Geotechnical measures**: It was recommended that the tall trees that existed in the eastern part of the school site should be removed and the trees and bushes which are located within a distance away from the structure ranging from 1 to 1.5 times the height of the tree should be removed too. The ground water tank should be maintained and carefully observed to ensure that no leakage of water takes place. The entire area at the entrance of the school building (north side), needs to be filled with compacted non-swelling soil and covered with cement tiles to make a positive slope from the structure towards the back yard in the west side of the building. The accumulated water in the basin area (west area) needs to be drained away from the building towards the street. Gutters around the roof should be kept open to maintain the diversity of runoff away from the structure.

Structural remedial work based on a 3-D finite element model using SAP2000 (Wilson and Habibullah, 1989) finite element code was performed. Some columns and beams were strengthened by increasing the cross-section and using steel plates, respectively. The remedial measures suggested in the present study were implemented two years ago. The building has been under continuous surveillance, and no significant deformation either as cracks in the structure or in the vicinity of the window and door openings had been observed since then.

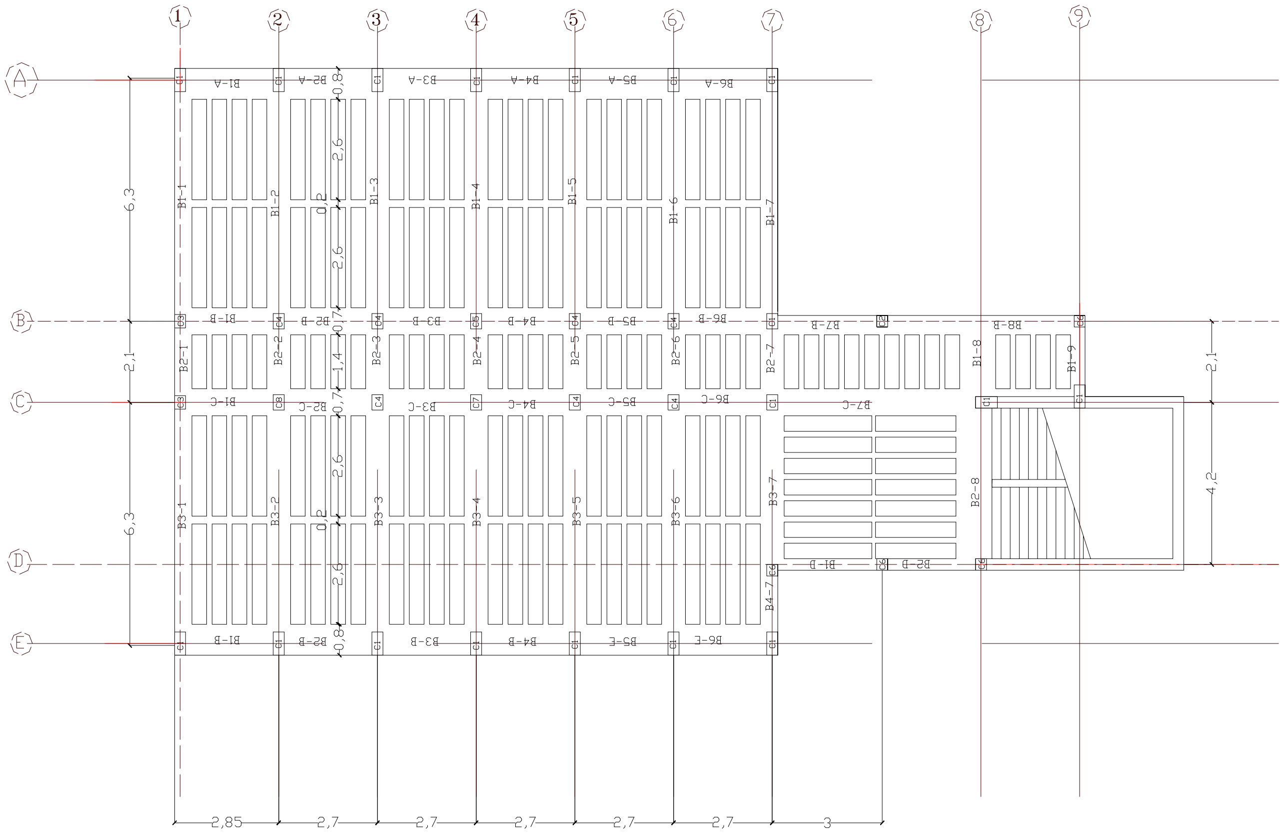

Figure (8): The plan-view of the analyzed part with beam numbers and columns used as well as the ribs and the axes.

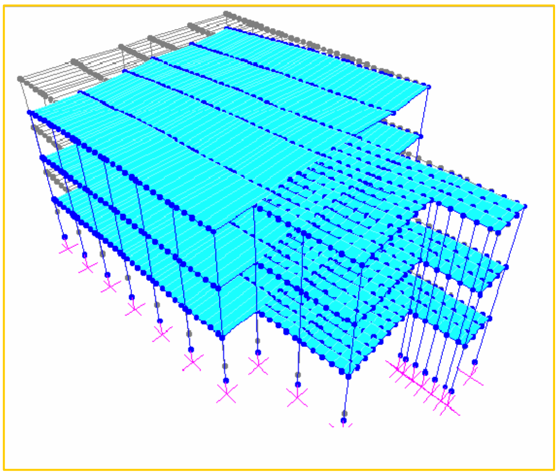

Figure (9): Deformed shape due to (DLLLST) loading case (the assumed settlements).

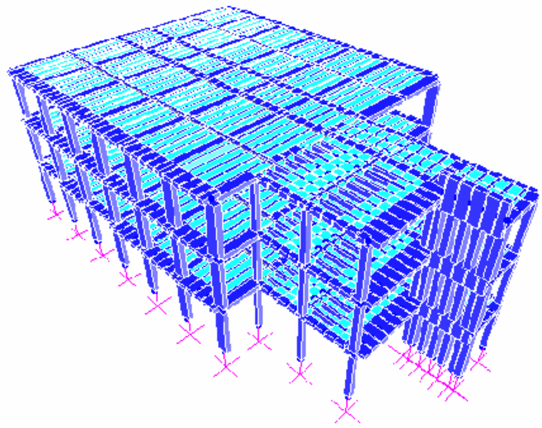

Figure (10): 3-D FEM model for analyzed part of the building.

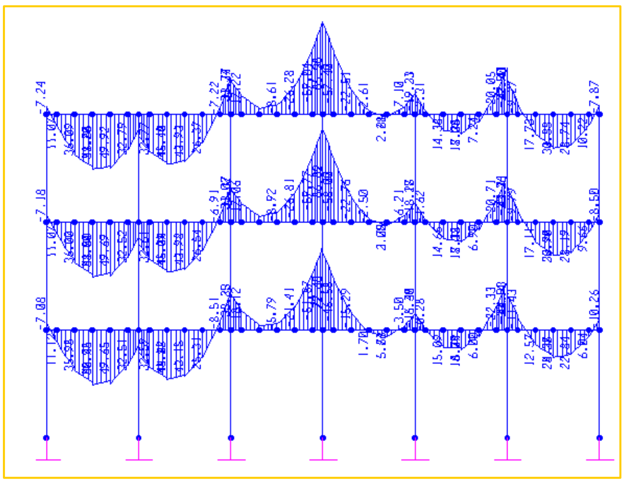

Figure (11): Moment diagram of frame E-E (DLLLST) in kN.m units.

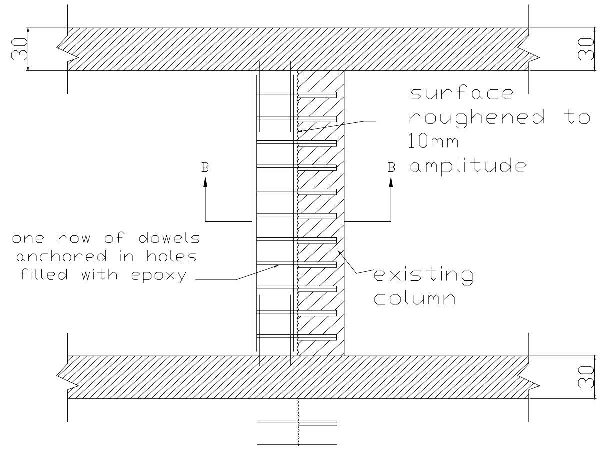

Figure (12): Method of increasing the cross-sectional area of the columns.

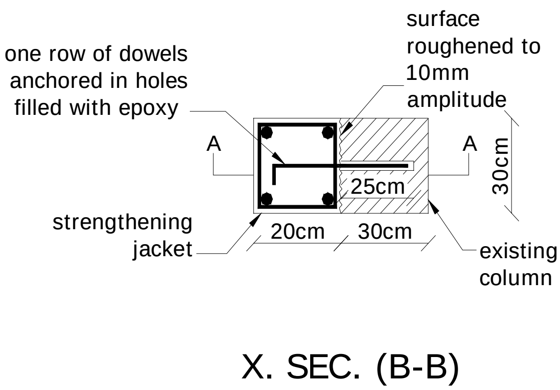

Figure (13): Plan-view of the column cross-section after repair.

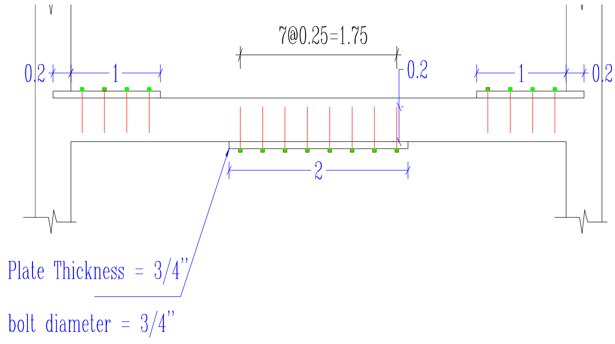

Figure (14): Beam strengthening using steel plates.

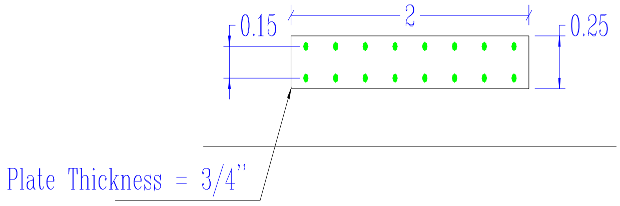

Figure (15): Top/bottom view of the steel plates attached to the beam.

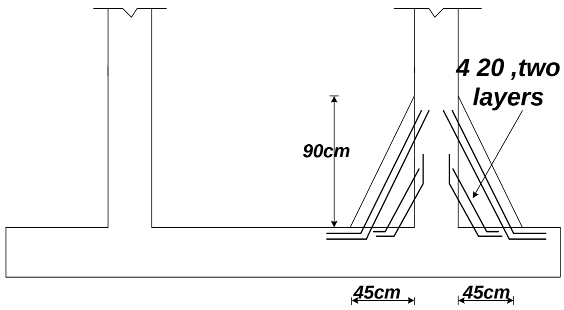

Figure (16): Strengthening of strip footing B-B against bunching shear.

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