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DISTRIBUTION OF SHEAR STRESSES IN ISOLATED SQUARE CONCRETE FOUNDATION HAVING SPOTS OF VERY WEAK SOILS

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ABSTRACT:- In the usual methodology for soil investigation, boreholes locations are selected in site, machinery extract, with due care, soil samples (or specimens) and tested, and then soil strength parameters for site are calculated. The soil report may contain some of the field tests, the SPT for instance, as well. The procedure is to pass this information to the structural engineer who, in turn, may design the structure. In soil investigation principle, the small soil specimen extracted from the soil strata have complete representation of the soil body underneath the footing. In soil mechanics theory the soil is well known to be nonhomogeneous and nonisotropic. This, eventually, means that soil properties and strength parameters change not due to location only but due to change of direction as well. This situation imposes the fact that the principle of representing the whole body of soil with a small sample is practically incorrect. The principle is assumed correct only as much as the soil is more and more isotropic and homogeneous. Therefore, it is in reality not uncommon for cavities to be present in soils body due to many reasons. If those cavities are detected before footing construction, then it is assumed that there is no serious problem. On the other hand, if it is not detected, the situation imposes a serious problem to the footing and structure depending on size and location of cavity.

This study deals with cavity presence directly underneath a single separated footing of proposed multi-storey building. Location and size of cavity are changed and a finite element (FE) analysis is run for each individual case. The subgrade soil is assumed medium-dense sand with a modulus of subgrade reaction 35000 kN/cu.m. Other than cavity location, the subgrade is assumed to be homogeneous and isotropic, i.e. have same material and strength properties. Suitable graphs are used to illustrate the stresses in footing. The study, however, take into account the shear stresses in 3D only (a shear-care study) in concrete. The foundation concrete is assumed to have low compressive strength such as 20 MPa. Other properties of concrete are assumed program's default.

Keywords:- Soil, Structure, Size and location of cavity.

PURPOSE OF STUDY

The exact soil profile is difficult if not impossible to draw. The usual method is to use drilling machines to get undistributed samples. Those samples are tested and their strength properties are considered to reflect the properties of the soil body from which they were taken. The size of the soil body to sample is tremendous. The presence of gaps is impossible to detect underneath footing. Also cavities in soil can create after construction of structure due to seepage or organic roots decay. In other words, the presence of hidden cavities below a foundation can be considered in general inevitable.

AN ACTUAL CASE OF FOUNDATION HAVING SPOTS OF VERY WEAK SOILS (CAVITIES)

Very recently in Baghdad (near Palestine Street), author has experienced a case were the problem of post formation of cavity after five decades of building construction. In early sixties of past century a moderate public building was built near Palestine Street in Baghdad. There after a date-palm tree was put near the rear boundary wall. This palm tree grew up to be huge with its roots penetrating and pushing the soil underneath the bearing wall foundation. Earth worms and insects had attacked the base and roots of this tree leaving a very huge cavity under foundation which was not detected until very recently when owners of the building intended to add an annex to the old structure. When that particular wall was removed away a very large cavity was seen under old foundation (about one meter in length and more than 70cm depth).

DESCRIPTION OF PROBLEM

In this study it is desired to simulate a footing having a cavity down under. The behavior and strength of footing are analyzed using Finite Element Program [4]. The following aspects are proposed:

1- The footing is 2.5m x 2.5m carrying a usual domestic column load for 3-storey building.
2- Load is transferred to footing by a single column. No eccentricity is assumed. Column dimensions are 0.5m x 0.5m, and carrying a total unfactored load of 432 kN. In this

way the stresses induced in analysis are nominal (actual) and not designed factored loads.

3- The allowable bearing capacity of soil is assumed 70 kN/m^2, quit common to Baghdad soil, with modulus of subgrade reaction of 35000 kN/m^3, i.e., medium dense sand.
4- The square footing is divided into a finite element (FE) mesh, each element is a semi-cube with lateral dimensions of 0.25m x 0.25m and height as presumed by footing depth which is 0.3m, d=effective depth =0.25m.
5- Number of total mesh elements is thus 100 elements.
6- The column load is distributed to nine nodes, thus each node of those on footing carries a force of 48 kN.
7- The lateral dimensions of footing are X and Z, while the “depth” direction is Y and oriented as positive to top of footing.
8- A cavity directly beneath footing is presumed of a selective size and location − this is called a case.
9- In F.E., it is difficult to simulate a cavity in soil. The approach is to reduce the modulus of subgrade reaction (k) to a very low value, or to zero if possible. The k value is transformed to equivalent elastic force working on nodal points in concern.
10- Each case is analyzed separately. No reinforcement is discussed, shear stresses are considered only. The drawings are color-contour type and rather indicative.
11- The movement or enlargement of cavity is limited to nodal displacements and making use of footing symmetry where possible.
12- The analysis is focused on shear stresses Sxy, Syz in top and bottom fibers.
13- It is hoped to feel from graphs how critical is the cavity and its effect on the structure of foundation.
14- By cavity it is meant a soil having real cavity case or a soil having spots of very loose structure which is considered then as a cavity.
15- In the drawings, column is presented by gray bold lines and cavity by black bold ones.

THE MEANING OF "Cavity" IN SUBGRADE SOIL

Suitable building sites in urban areas are becoming difficult to fix, and then sites targeted for urban renewal are used. These sites can be quite hazardous from demolition of previously existing structures and backfilling of former basements during load seeping. Often, this type of backfill is done with little supervision or quality control [3], so there can be significant soil variation of these sites within few meters in any direction.

The presence of cavities in soil skeleton is not uncommon. The source of these cavities can be attributed to numerous reasons, but mainly due to the decade of organic materials such as trees roots, for instance. If there is a seepage situation in soil the water seepage force j ($j = i\gamma_w V$) may wash out the fine grains in soil leaving small holes or a cavity potential problem in soil. Where i is the hydraulic gradient and V is volume of soil in which total head loss working on. The foregoing problem may arise in case of foundation construction in levels down below the natural water table. Unsuitable design of filters may cause the immigration of fine grains with ground water. The situation come rise when dewatering of ground water, which is a necessary condition during the foundation construction. That is when the water table is higher than the foundation level. The dewatering of ground water may continue until the full construction of footing works under the water table.

From the brief foregoing discussion, it can be realized that the cavity does not mean absolute space in soil. It may mean a location in soil body where accumulations of tiny voids are present or a complete decade of trees roots leaving an organic soil of very loose structure or even an empty space. Thus, a complete empty space in soil is rather difficult to imagine although it is not uncommon. On the other hand, such a location may reduce the modulus of subgrade reaction of soil (k) to very low value. As a simulation to computer programming, the modulus is replaced by elastic springs each working out in a specified area that is the area of the element centered at the nodal point and as follows in figure (1).

How much is the k value and how much is the reduced value of k in cavity? According to Teng [5] the coefficient of subgrade reaction (or modulus of foundation, subgrade modulus) is the ratio between the pressure against the footing or mat and the settlement at a given point.

$$k=q/s$$

Where:

k = soil reaction in unit pressure required to produce a unit settlement.

q = the applied pressure, and,

s = the settlement.

In clayey soils, settlement under the load takes place over a long period and k should be determined on the basis of final (total) settlement. The modulus of foundation is independent of the applied pressure, and k has the same value for every point of the surface of footing in case of no cavity presence. Unfortunately, the k value depends on many factors. They are stated by Terzaghi as [5]:

1-Effect of size,

$k=k_1(B+1)/(2*B)$ …..(granular)

$k=k_1/B$ …..(clay)

Where:

k = plain strain coefficient of subgrade reaction for footing of width B (B in feet).

k_1= plain strain coefficient of subgrade reaction for footing of 1 foot width.

It is worth to mention here that the equations mentioned in this study, which belong to different scientists, are written some in foot-Ib system and other in SI units according to their original articles. Most of these equations are empirical ones. Thus author preferred to put them that way without revision.

2-Effect of shape,

$$k = k_s \frac{(1+\frac{B}{L})}{1.5}$$

Where:

k= coefficient of subgrade reaction for rectangular footing of width B and length L.

k_s=k for square footing (B * B).

3-Effect of depth,

$k'=k(1+2D/B)$ $(k' \leq 2k)$

Where:

k'=coefficient of subgrade reaction at depth D.

If size and depth are combined together, then,

$$k = k_1(\frac{B+1}{2*B})^2 (1+\frac{2D}{B}) \quad \leq \quad 2k_1(\frac{B+1}{2*B})^2 \qquad \text{…(B and D are in feet).}$$

On the other hand, Bowles [2] presented the Table(1) for range values of k, however, he confirms that these values should be used as a "guide" only and never as a substitute to the actual testing.

If a value of k_1=35 000 kN/m^3 is chosen, that is medium dense sand, then,

$$k = 35000(\frac{(8.2+1)}{(2*8.2)})^2 (1+2*\frac{3.28}{8.2}) = 20000\,\text{kN/m}^3$$ is to be used in FE solution. Here again, B=2.5 m = 8.2ft and D=1 m=3.28 ft.

In case of cavity location, the k value is difficult to be predicted. Of course, it may lie between a value of actual k and little more than zero. As a risk analysis to be performed, the cavity is assumed empty, leading to value k=zero (a space location). Thus in finite element (FE) solution, in which k is replaced by an equivalent elastic spring; the nodal point above cavity location is left unsupported. This situation imposes a greater load (force) at the other springs, since the total column load is distributed to the other supporting springs. In such case, the shear in footing cannot be calculated by classical methods.

It is worth to mention here that having a cavity under foundation in soil may alter the distribution of soil pressure and foundation, accordingly, may experience some eccentricity in stress distribution. This kind of problems is not handled in this study and Winkler springs of modulus of subgrade reaction are modeled in equal sense. There may be some small differences in results, but in this research it is intended to focus on behavior in a qualitative manner and not to concentrate on pure design values. Furthermore, the exact soil stress distribution under foundation with cavity is rather difficult to follow and hard to predict, since the following factors contribute in type and magnitude of exact soil pressure distribution under footing:

1-Soil never behaves elastic since it is non-isotropic and non-homogeneous.

2-Type of soil, sand, clay, peat, collapsible, expansive and the like…

3- Presence of any suspended large boulder in soil matrix.

4- Location of water table.

5-Rigidity of foundation, flexible or rigid.

6- Eccentricity in applied loading.

7-Presence of any nearby structure which may produce overlap zone of stresses and settlements as well.

8- Any vibration action may alter the stress distribution especially if soil is sandy.

SOIL PROPERTIES OF FOUNDATION

Since this research is theoretical and for the sake of simplicity the foundation soil is assumed to be sandy. The clayey soil imposes further difficulty that is the problems of the consolidation and the secondary compression settlements (or creep). If the cavity is located at one edge or one-half of footing a serious problem will arise, that is the tilt of the foundation. This is because the pressure in one side of the soil (the cavity side) is larger than the other, resulting in higher settlement in the higher-pressure side. After a period, alteration of soil properties will likely to happen consequently. In this study, the foregoing problem will not be discussed and the soil properties including the modulus of foundation will remain as assumed 35000 kN/cu.m. Calculations are focused on the shear stresses induced in footing.

FAILURRE CRITERION IN SHEAR

Computation of shear strength in footing is done in accordance with section 15.5/ACI 318-08, [1]. The shear strength for footing in the vicinity of concentrated loads or reactions is governed by the more severe of the two conditions: 1- The beam action for footing with critical section extending in a plane across the entire width and located at a distance d from face of column, and 2- Two-way action for footing with a critical section perpendicular to plane of slab and located so that its perimeter b_o is minimum and need not approach closer than d/2 to perimeter of column.

In case of beam action, $\mathrm{Vc} = \frac{1}{6}\sqrt{f'c} = \frac{1}{6}\sqrt{20} = 0.745\ \mathrm{N/mm^2}$

And in case of two-way action,

$$\mathrm{Vc} = 0.083\,(2 + \frac{4}{\beta_c})\sqrt{f'c} = 0.083\,(2 + \frac{4}{1})\sqrt{20} \leq \frac{1}{3}\sqrt{f'c} = 2.227 \leq 1.491$$

Now the problem that comes to mind, what is the failure criterion for footing in shear for the case of cavity existence?

Since a cavity can exists underneath a footing in any size, shape, location, and modulus of foundation value, author believes that it is almost impossible to predict the exact crack shape and its propagation due to failure. Thus the ACI-Code criterion for shear failure will continue to control with the minimum value of shear strength is used, i.e. Vc=0.745 MPa. This is so since in computer aspects the program does not provide type of failure but instead provides the stresses induced in concrete foundation body. Thus for sever precaution the minimum shear strength provided by concrete is used as a reference.

To illustrate the previous concepts, forward "jump" is done to a step to one of the analysis (Case 3 in FE solution), but a simple modification is conducted here and only here, that is the footing thickness is reduced from 0.3m to 0.25m. This step is done in order to emphasize the shear stresses in footing more thus the reader is referred to the associated Compact Disc. The centerline across the column and cavity is highlighted. Top and bottom shear is recorded for nodal shear forces in the plane xy (Sxy in FE solution) as can be seen in the figures (2, 3 and 4), A foreword finite difference (FD) is conducted by the author based on the shear nodal forces to find the shear function as a 5th degree interpolation polynomial,

$V(x) = Sxy = 5.82x^5 - 19.285x^4 + 21.926x^3 - 8.675x^2 - 0.1114x + 0.139$

Both the shear function and the linear fitting shear diagrams are shown in figure (5). It can be seen that the maximum shear in this direction occur at a distance of one nodal displacement = 0.25m which is approximately equal to the effective depth of footing d. On the other hand, if the opposite side is highlighted, the top shear for the elements 56 to 96 is as shown in figure (6).

These calculations although indicating a beam action failure, if the maximum shear is used as an indication, it cannot be yet concluded that the shear failure criterion is known in advance. The full examination of the cases all may highlight other facts.

CHOICE OF SIZE AND LOCATION OF CAVITY

As mentioned earlier, the cavity can exist, without our knowledge, at any size, shape, depth, and direction. Then endless ways to simulate the cavity exist. Nevertheless, author believes that the following combination of cases may serve to show the general behavior of footing in case for shear study. Those cases will be listed with appropriate notes and indicating figures. Other combination of cavity locations is, of coarse, possible as well.

It is worth to mention that the cavity was presented in the figures from 7 to 26 by bold black lines while column by bold gray square.

Case No. 1, No Cavity Presence

In case of no cavity existence, the three shear components Sxy, Syz, and Szx have the maximum values (±0.403, ±0.403, ±0.289 MPa) respectively which are all below the failure shear strength for beam-action (0.745 MPa). The following two figures in (Fig.7) show the shear stress Sxy in the bottom face (fiber) of footing and the upper one. It can be seen that the maximum shear stresses is circular-like envelop located near the two sides of the column at left and right directions. Their centers are about one nodal distance which is approximately the effective depth of footing.

There is no much difference in shape between the top and the bottom faces of footing. The same trend is obtained for Syz, (Fig. 8) but the maximum shear counters are located on top and bottom. Thus, it can be seen clearly that the column is "surrounded" by zones of maximum shear stresses that fit well with the ACI-failure criteria, namely, the two-way and the beam actions. However, form shear stress aspects, the footing is safe having a safety factor of 0.745/0.401 which is about 1.86. This factor is nominal and considered hardly "satisfactory" because no load factors are used for the applied load column. However, according to concrete design fundamental, the footing is safe nominally.

Case No.2

In this case the cavity begins at the centerline of column, extends 3 by 4 nodal displacements in the left direction, i.e. 0.75m x 1m, and making a cavity area 12% of the total footing area. The cavity location can be seen easily in all figures as there is no spring forces in the nodal points representing the cavity. The following two figures (Fig.9) show the Sxy in top and bottom faces of footing.

The maximum shear stresses Sxy are +0.403, -0.382 MPa. In the top face there are two zones of maximum stresses located at right and left to column, its centers are about one nodal displacement from the face of support. As can be seen, the stress zones have semi-elliptical shapes. Their magnitudes are about half the shear strength of concrete, i.e. 0.745 MPa. In the bottom face we can see three shapes of maximum stress at left and one at right. Therefore, under cavity we notice three zones of maximum Sxy stresses. They, as well, about one-half the failure shear stress.

On the other hand, the following two figures (Fig.10) show the Syz shear stresses. The maximum shear stress ranges between ±0.435 MPa.

It can be seen that there are two semi-circular shapes of maximum Syz located in top and bottom to the supporting column, with centers about one nodal displacement away from the face of support. It is noticed that the maximum shear occur not in the cavity region, but right angles to it. In this case the situation is safe and no fear of failure exists, since there is no serious increase in shear stress compared to case one.

Case No. 3

In our case here, the cavity, in its same location as before, is extended in size to 4 by 6 nodal spaces. The total cavity area is now 24% of footing area, i.e. 1m x 1.5m. The following two figures (Fig.11) show Sxy stresses and the magnitude of its maximum ranges between +0.402 and -0.352 MPa, not too much different from the case before.

The two figures are rather similar having three zones of maximum Sxy at the left and one at the right of column, all are semi-elliptical in shape. The two zones near the column are about one nodal displacement from face of support.

The two figures that follow (Fig. 12) show the Syz stress distribution throughout footing. The maximum of Syz are +-0.557 MPa that is about 75% of the shear strength for concrete, an increase in shear stress of (0.557-0.401)/0.401 = 39% for no cavity presence.

The two figures are, as can be seen, similar having two circular zones of maximum stresses located up and down the column, and about one nodal space from face of support. Again, the maximum shear stresses occur at two locations, at right angles to cavity position.

Case No.4

The cavity, which is now 3 by 6 nodal spaced, is shifted from the center of column to be now at the face of support, its area is 18% of footing size, i.e. 0.75m x 1.5m. The two figures of Sxy (Fig.13) have common similarities, its maximum values ranges between +0.402 and -0.318 MPa. We have two elliptical shapes of the maximum, located to left and right of column.

On the other hand, the Syz stresses (Fig.14) have maximum values of ±0.479 MPa, a shear increase of (0.479-0.401)/0.401=20% for no cavity case. We have, as well, two circular shapes located at right angles to support.

Again and as before, the maximum shear stresses occur at right angles to cavity position, about one nodal space from face of support.

Case No.5

Cavity is 4 by 6 nodal spaces, 1m x 1.5m, i.e. 24% of footing area, starting at face of support, see figures (15) and (16).

The maximum Sxy stresses are +0.394, -0.287 MPa, the top and bottom faces are similar having three zones of maximum at left and one at right of support. On the other hand, the maximum Syz stresses are ±0.501 MPa (a shear increase of (0.501-0.401)/0.401=25% compared to no cavity case), having two zones at up and down the column. In all figures, the maximum zones are close to face of support and below the shear strength of concrete.

Case No.6

This case has the cavity at corner, a size of 4 by 4 nodal spaces, i.e. 1m x 1m, totaling 16% of footing area. The following figures describe the case, see figures (17 & 18).

The maximum values for both Sxy and Syz stresses ranges between -0.412 and -0.382 MPa having circular shapes near the column, as can be noticed from figures. No appreciable increase in shear stresses compared to no cavity case.

Case No.7

Here the cavity is located directly under the column, 4 by 4 element spaces, i.e. 1m x 1m and 16% of total footing area, see figures (19 and 20).

The maximum Sxy and Syz stress values are ±0.467 MPa, having two circular zones of maximum shear at one element space to left and right for the case of Sxy and two, up and down column for Syz. This is expected from the symmetry of footing. There is a small shear increase in shear compared to case one, i.e. 16%.

Case No.8

It is similar to case 7 before but the cavity is much larger, 6 by 6 nodal spacing, i.e. 1.5m x 1.5m with total area of 36% of footing size, see figures (21 and 22).

The Sxy and Syz have maximum values of 0.518 Mpa, about 70 of shear strength and 29% increase in shear stress compared to case one. The trend in stress envelope and propagation is similar to the case before but with larger stresses.

Case No.9

This is a case, which considers two cavities into account are seen in figures (23 and 24). Each cavity is 3 by 4 nodal spaces, i.e. 0.75m x 1m, extending from face of support at left and right to column, thus totaling a cavity area of 24% of footing area. Although the probability of having such a situation seems to be very low, but it cannot be neutralized.

The maximum Sxy stresses are ±0.337 MPa. At the bottom face, we have three zones of maximum stresses at each cavity side. These are one nodal space from the face of support. In top face, the three zones on each side seems to converge to one big zone of maximum Sxy value on each cavity side, and starting at face of support.

The Syz have maximum magnitudes of ±0.419 MPa and consist of the zones shown in the figures related. The maximum values of both Sxy and Syz are about one-half the shear strength of concrete.

Case No.10

Now, finally yet importantly, extending the probability of two cavities presence, here, the cavities are, as well 3 by 4 each (0.75m x 1m each) but extending at right angle to each other and starting at the face of support, i.e. 23% of footing area as can be seen in figures (25 and 26). The maximum values of Sxy are 0.416 and -0.392 MPa and consist of three zones at left and one zone at right for the bottom face, for top face, two zones. They begin at face of support. Of course, no need for the Syz stresses due to symmetry but, yet it is listed.

CONCLUSIONS

1- The presence of a cavity is not uncommon, and this cavity may go from complete space to some loose structure of soil particles. The size and location of cavity are random in nature and depend on soil structure and history. The cavity may be created due to decay of trees roots, some organisms, or due to some seepage problems causing small soil particles immigration.
2- The subgrade reaction of soil in this case will vary from a value near zero to a value little less than the actual modulus reaction of soil. In this study, the soil is assumed sandy (medium dense) with soil modulus value of 35 000 kN/cu.m.
3- The shear failure criterion of soil is easy to predict for normal footing situations, which are defined in the ACI code, namely, the beam action and the punching shear. In the presence of a cavity, the failure criterion is difficult to be predicted. However, the FE calculations have shown that the maximum shear stresses do occur in locations ranging between the face of support and a distance of one effective depth (d) away from the face of support depending on the location and size of cavity. This can be noticed clearly by observing the case studies in this research.
4- The cavity itself is chosen in locations, believed to be of critical places. Those chosen places are selected to provide full scope in range for cavity presence under footing. Each case is analyzed separately.
5- In case of cavity at one side of footing, the maximum shear stresses occur not at the cavity zone but at right angles to the cavity place. Almost all zones of maximum shear stresses are shown to be circular or elliptical in shape of contours type.
6- If the size of the cavity is small, no appreciable changes in shear stresses occur. The cavity positions beside/under the face of support impose the largest increase in shear stresses, i.e. about 40% compared to no cavity case.
7- Due to the low applied load, no factorization exists on dead and live loads, and nominal strength properties used in study, no failure exists in all cases. It is the aim in this study to consider shear variations in footing.
8- The case of corner cavity location has imposes the least interaction among the other cases.

9- If the loads applied to footing are factorized, the design stresses may be on verge of failure. By these pervious findings, author may recommend to increase footing thickness in sites coming from demolition of previously existing structures, or in sites that may have potential hazards for such problem of cavity location.

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Table(1): the range values of k.

Soils	**k in kcf**	**k in kN/m^3**
Sandy Soils		
Loose sand	30-100	4800-16000
Medium-dense sand	60-300	9600-80000
Dense sand	400-800	64000-128000
Clay medium dense sand	200-500	32000-80000
Silty medium dense sand	150-300	24000-48000
Clayey soils		
qu≤200kPa	75-150	12000-24000
200≤qu≤400kPa	150-300	24000-48000
q_u>400kPa	>300	>48000

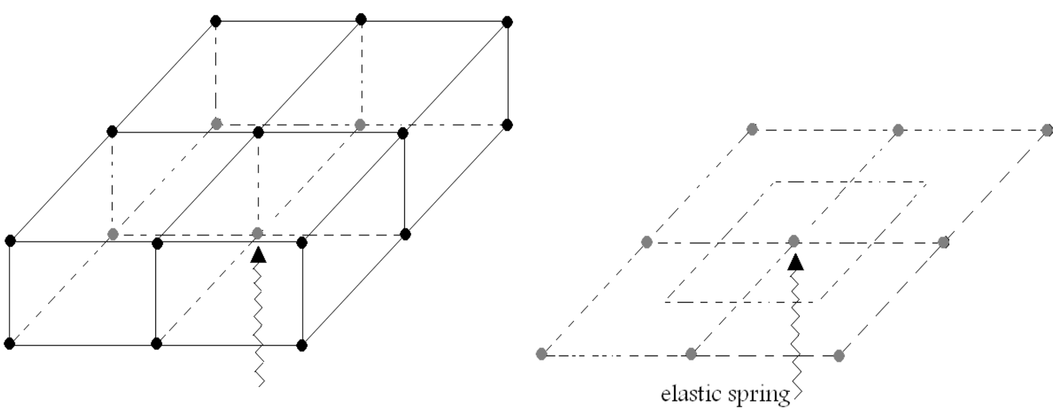

Fig.(1): Soil-spring simulation in Finite Element.

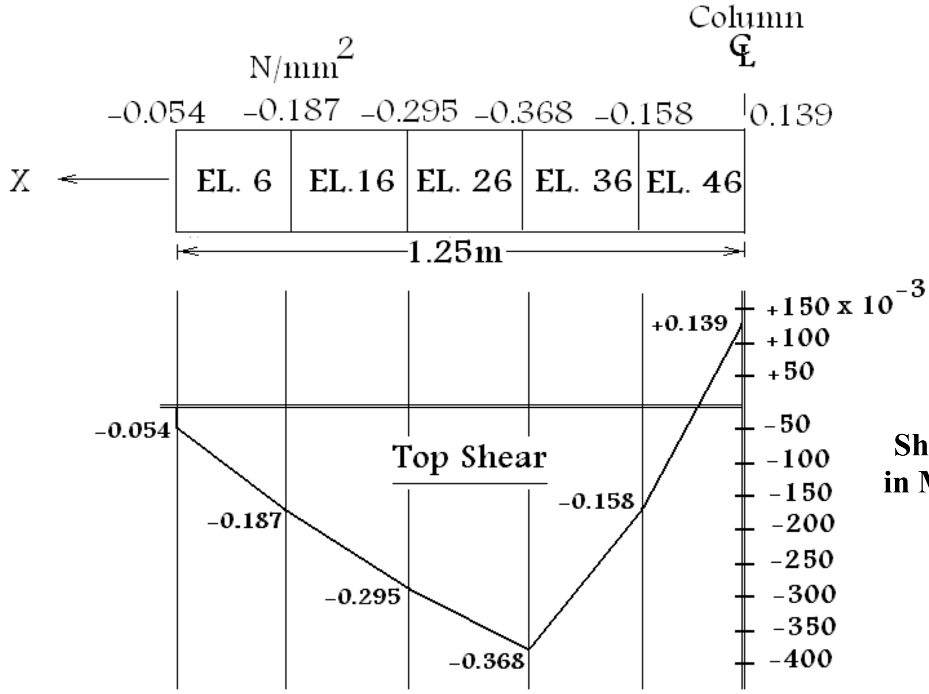

Fig.(5): Shear stress diagram for finite element (FE) analysis of top footing face.

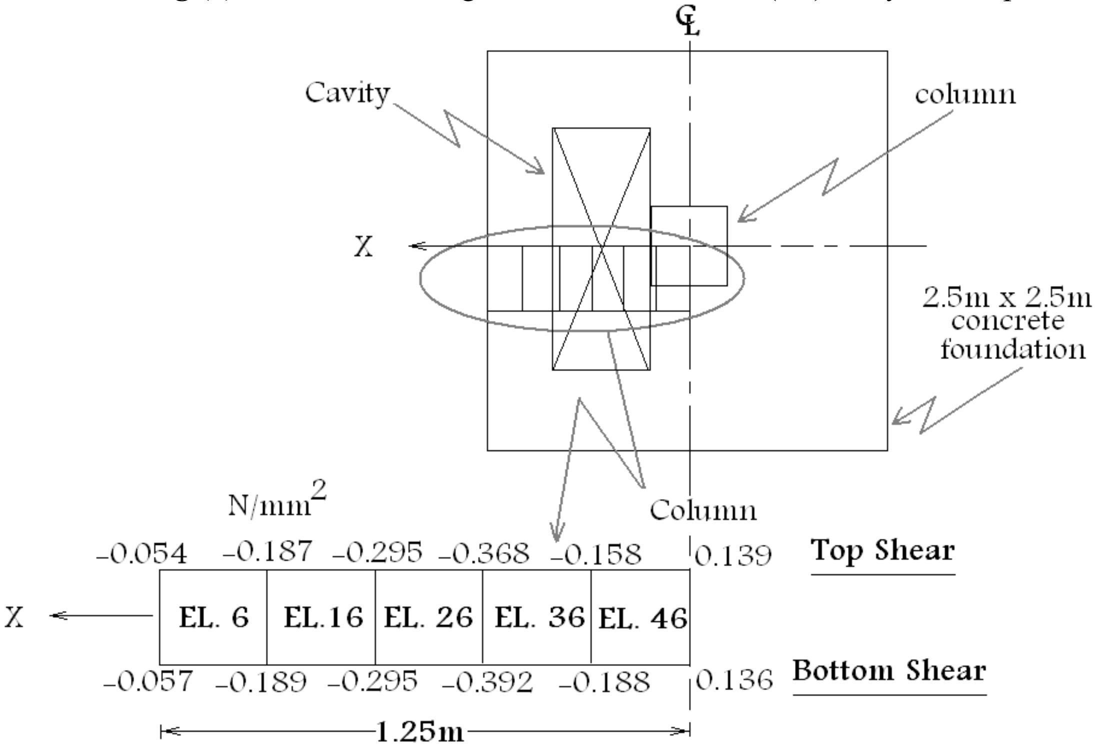

Fig.(6): Finite element (FE) analysis for top and bottom shear fiber stresses.

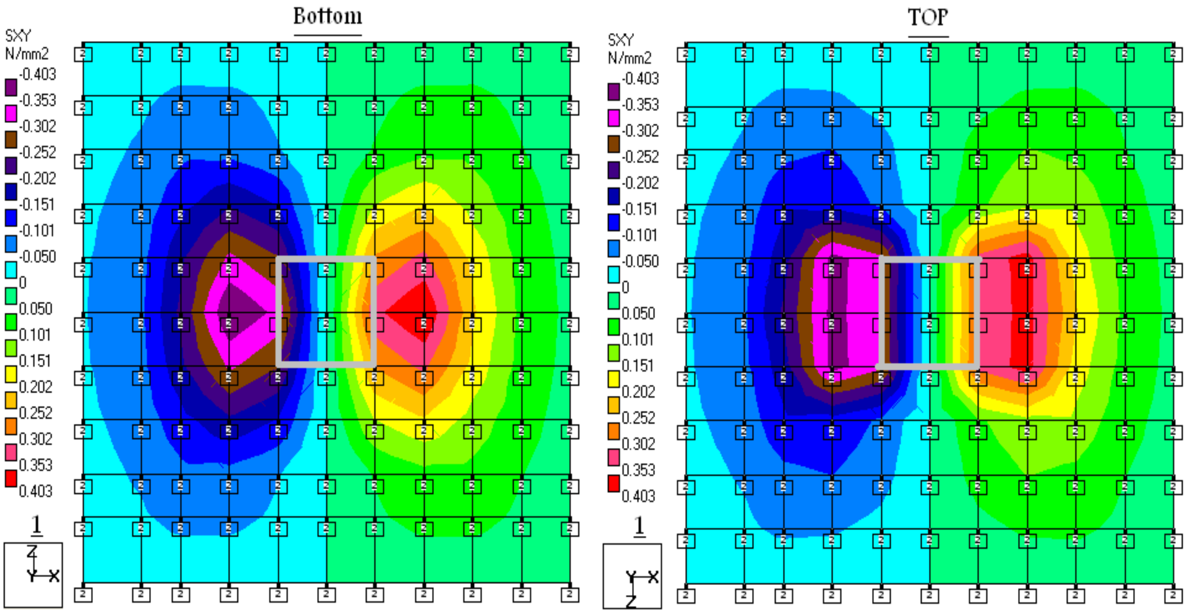

Fig.(7): Distribution of shear stresses Sxy at the bottom and at the top of footing (case 1).

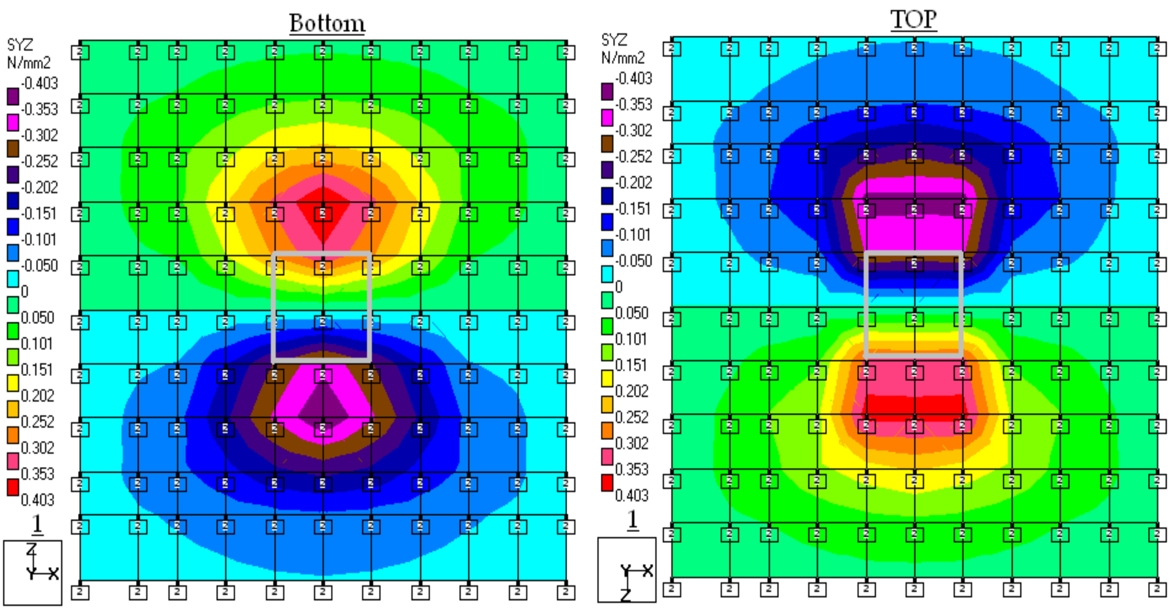

Fig.(8): Distribution of shear stresses Syz at the bottom and at the top of footing (case 1).

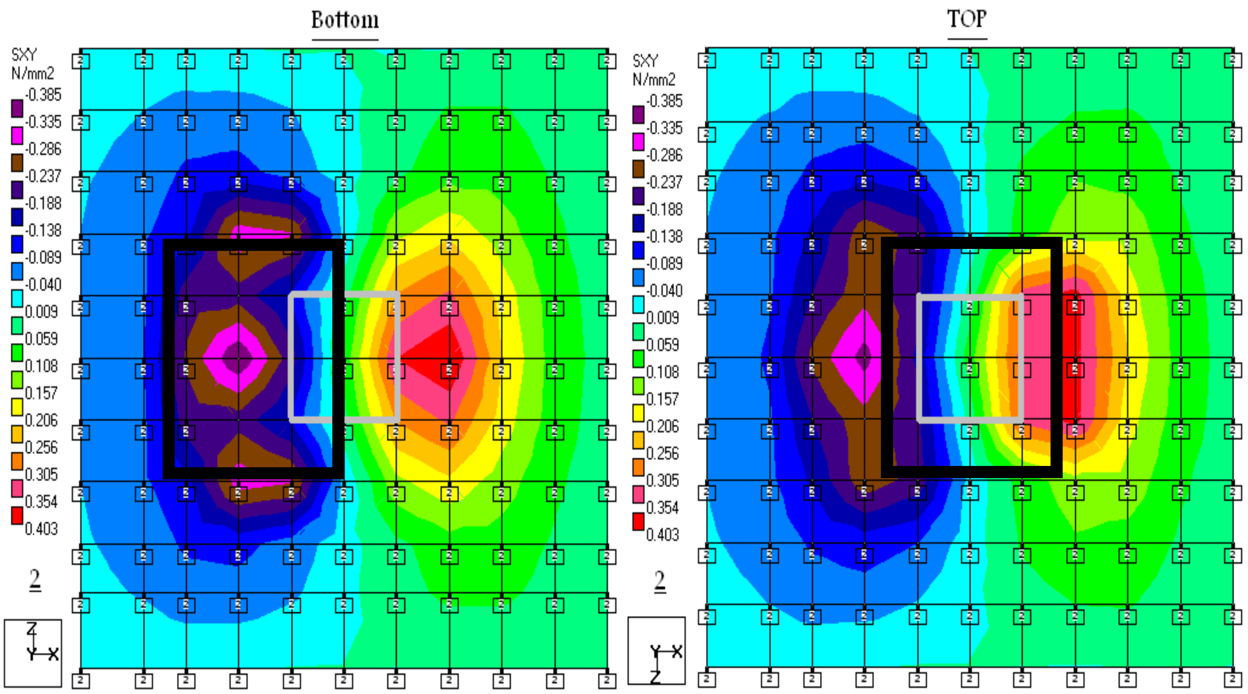

Fig. (9): Distribution of shear stresses Sxy at the bottom and at the top of footing (case 2).

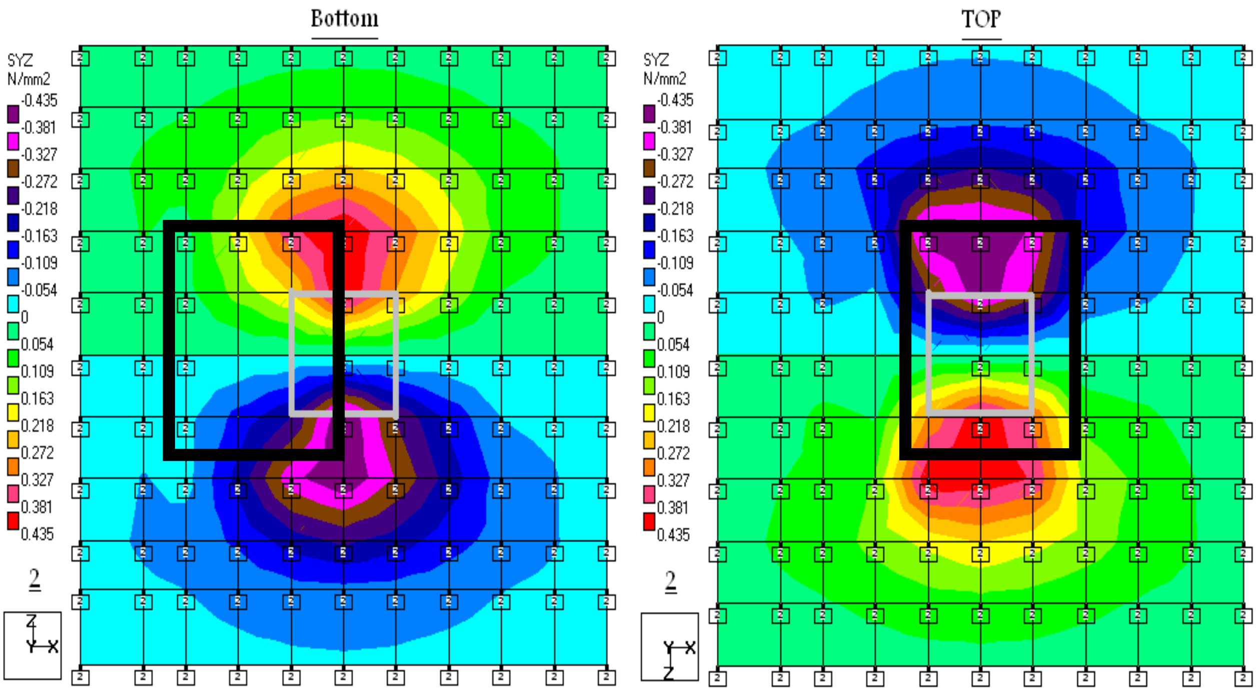

Fig. (10): Distribution of shear stresses Syz at the bottom and at the top of footing (case 2)

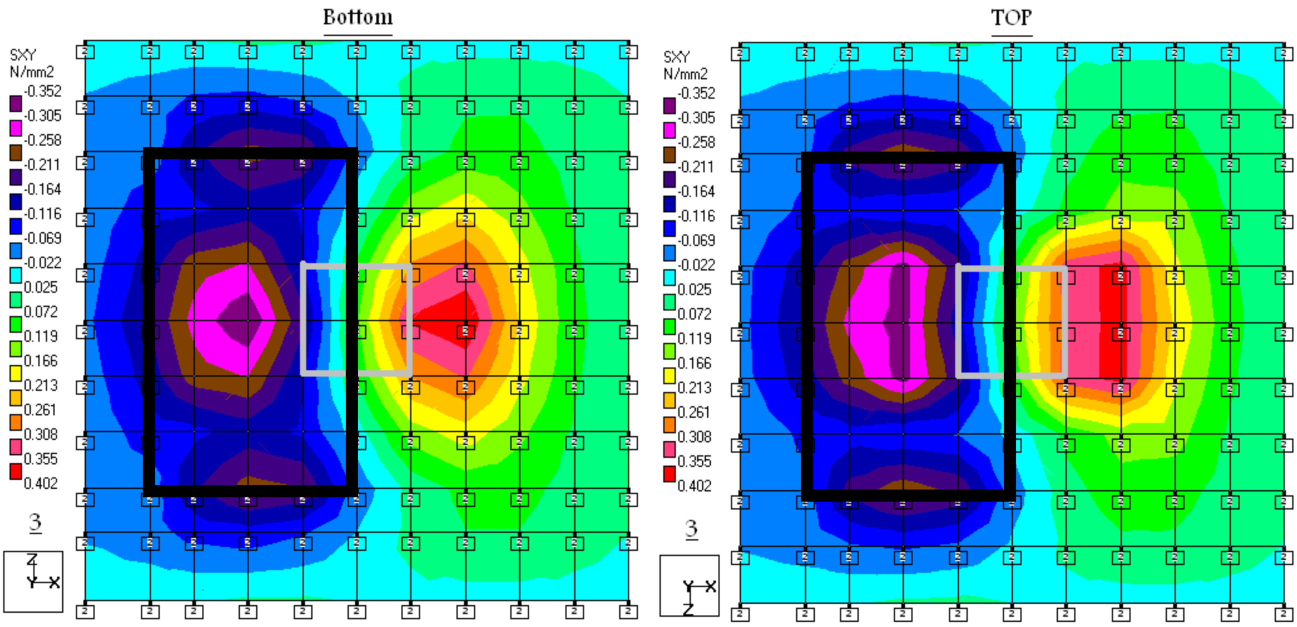

Fig.(11): Distribution of shear stresses Sxy at the bottom and at the top of footing.

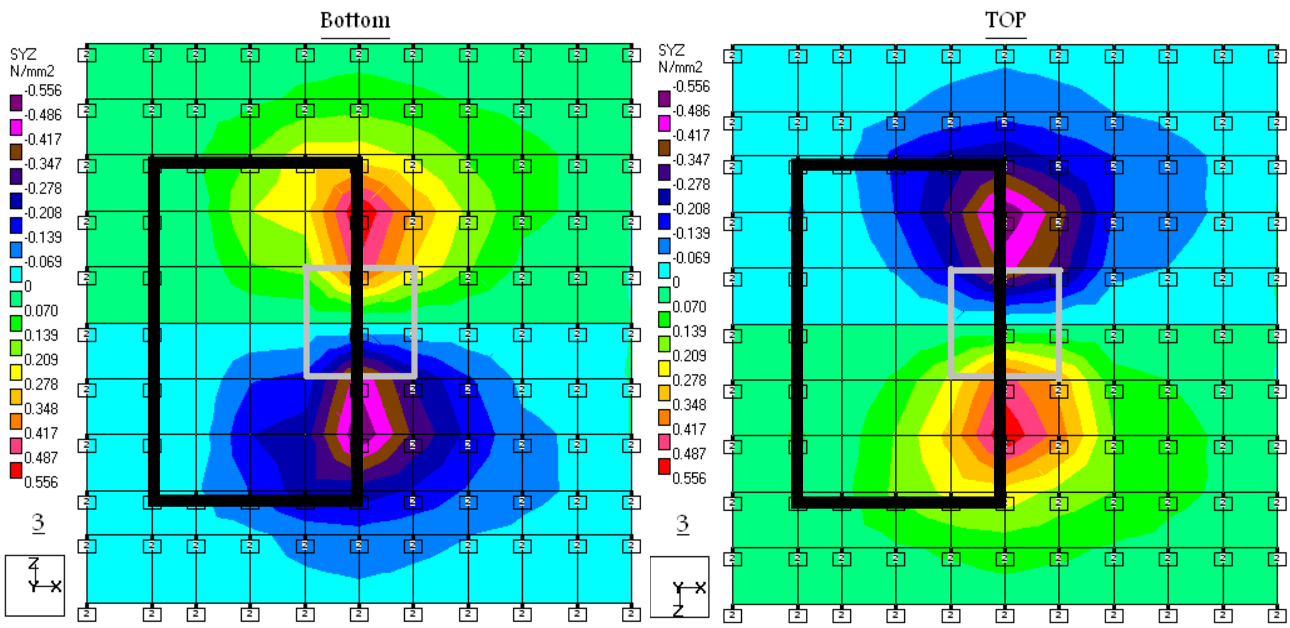

Fig.(12): Distribution of shear stresses Sxy at the bottom and at the top of footing (case 3).

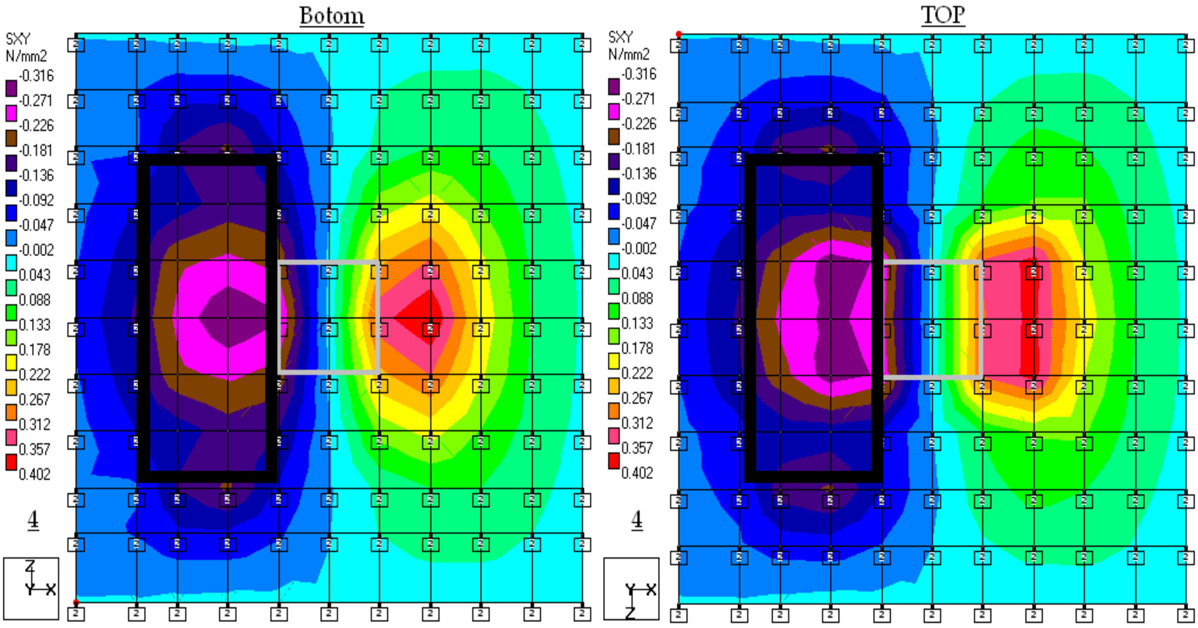

Fig.(13): Distribution of shear stresses Sxy at the bottom and at the top of footing (case 4).

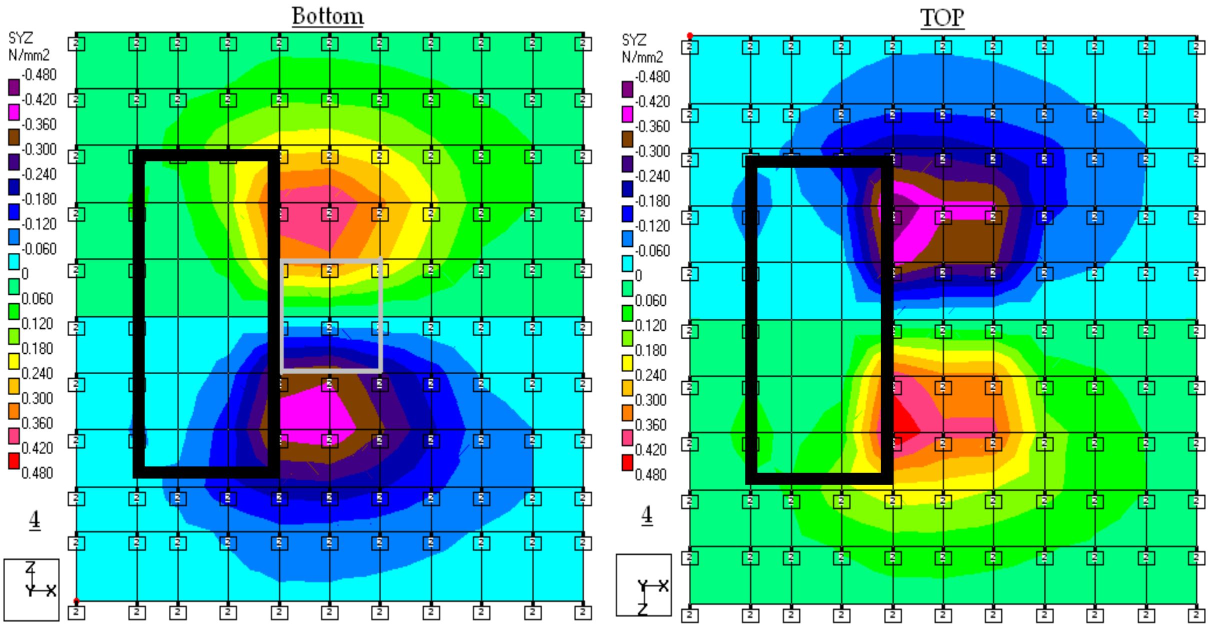

Fig.(14): Distribution of shear stresses Syz at the bottom and at the top of footing (case 4).

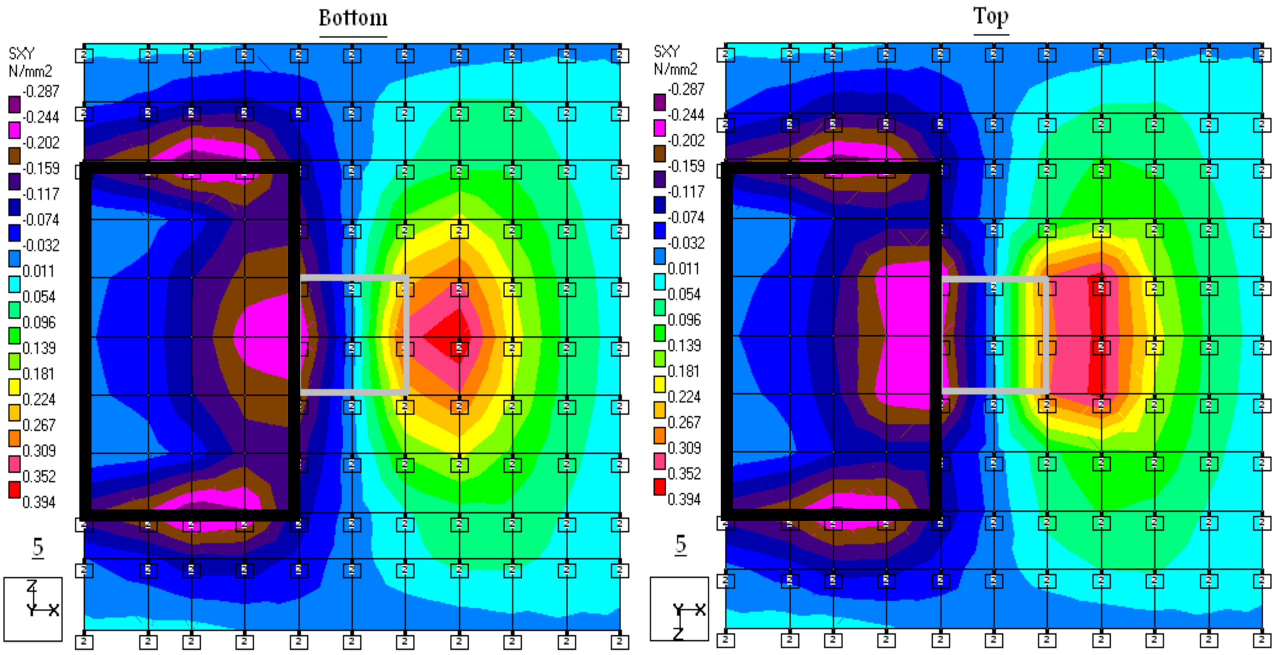

Fig.(15): Distribution of shear stresses Sxy at the bottom and at the top of footing (case 5).

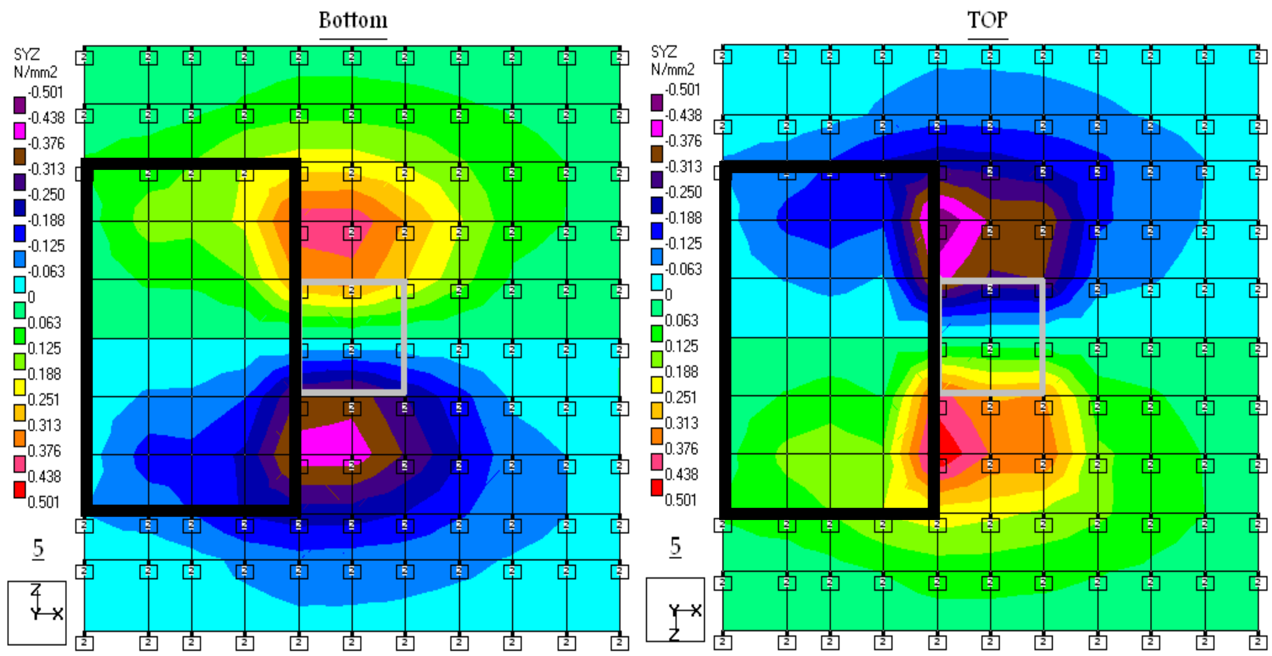

Fig.(16): Distribution of shear stresses Syz at the bottom and at the top of footing (case 5).

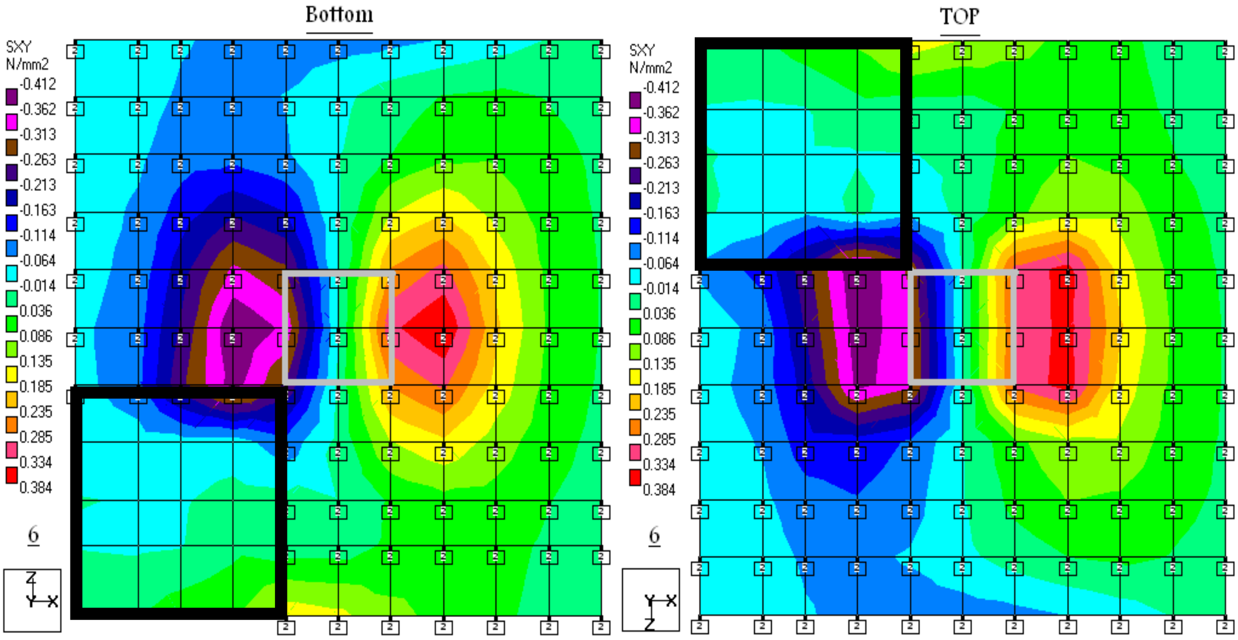

Fig.(17): Distribution of shear stresses Sxy at the bottom and at the top of footing (case 6).

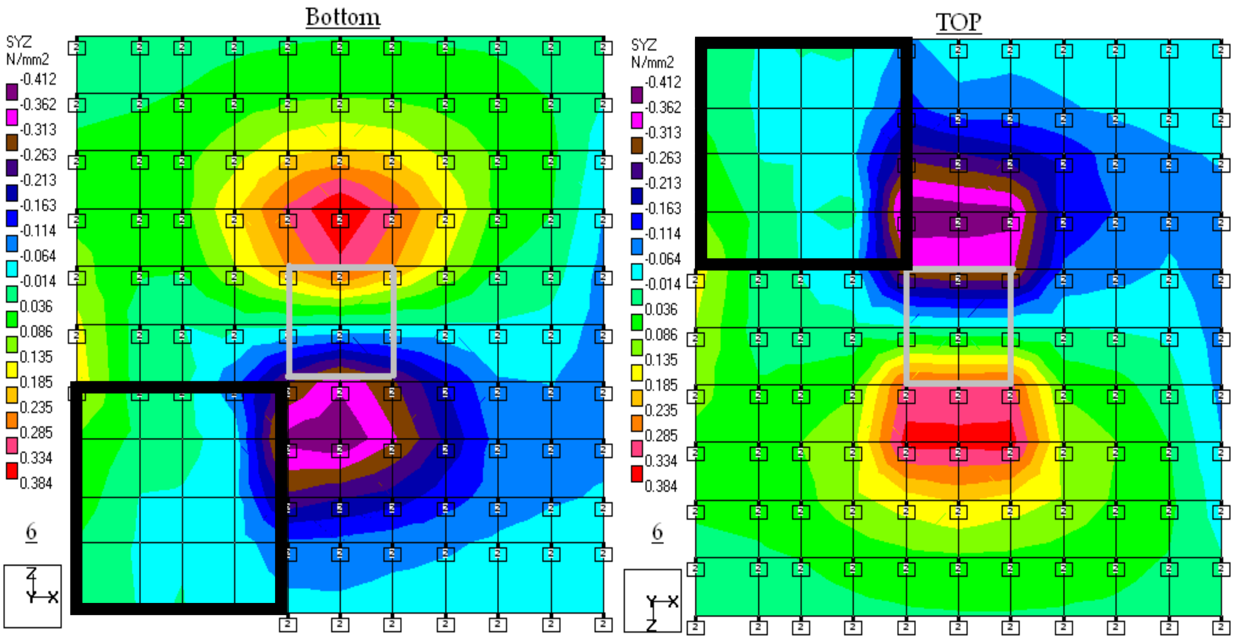

Fig.(18): Distribution of shear stresses Syz at the bottom and at the top of footing (case 6).

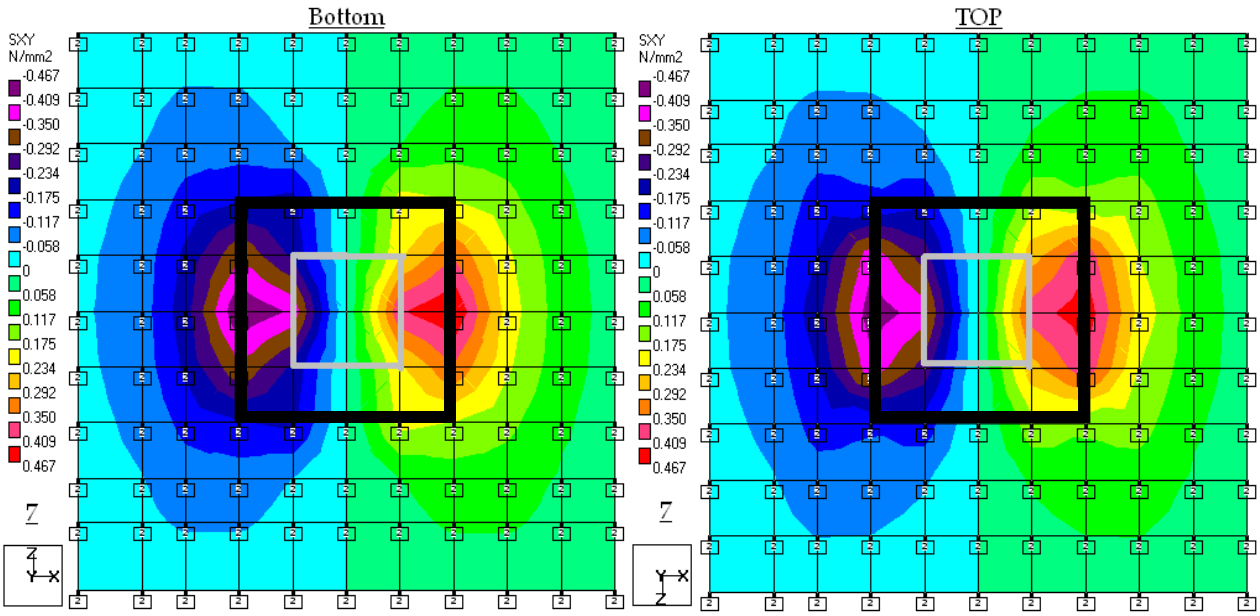

Fig.(19): Distribution of shear stresses Sxy at the bottom and at the top of footing (case 7).

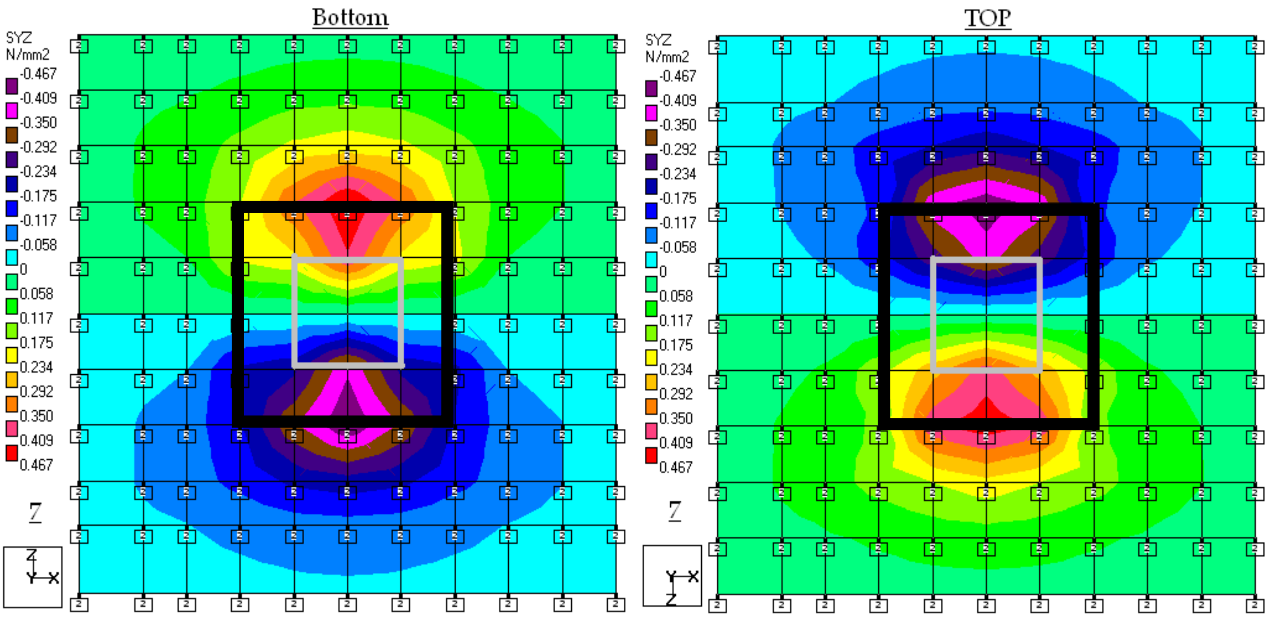

Fig.(20): Distribution of shear stresses Syz at the bottom and at the top of footing (case 7)

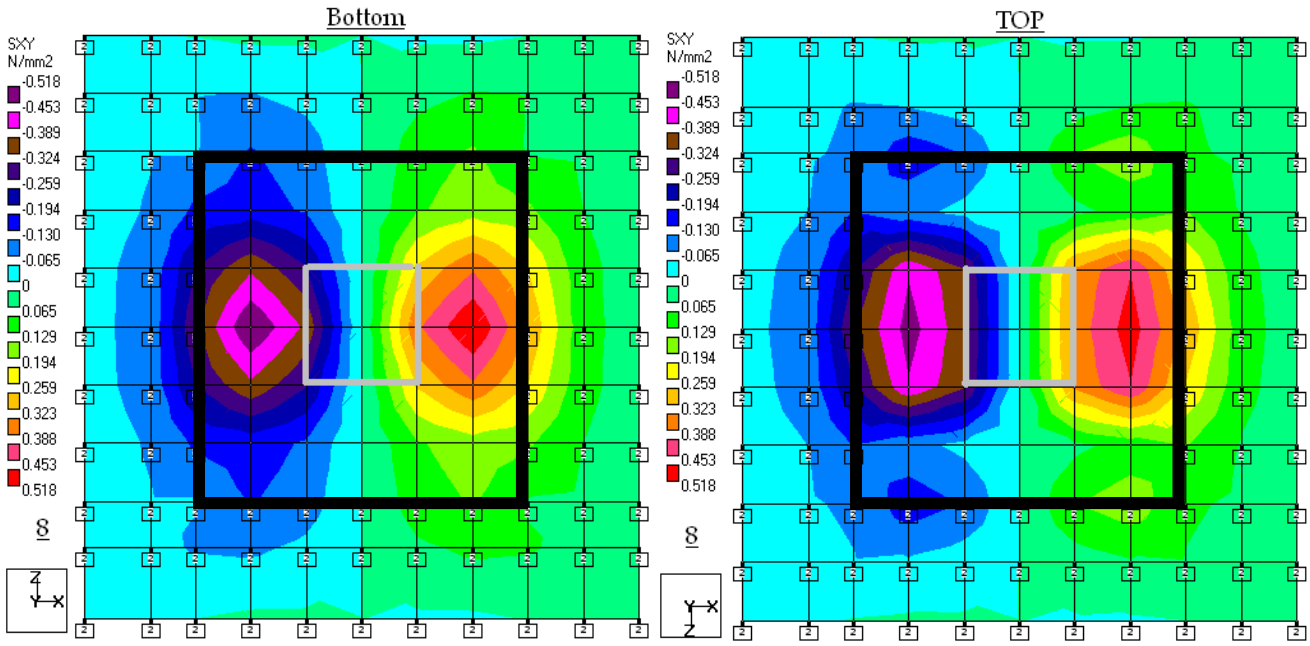

Fig.(21): Distribution of shear stresses Sxy at the bottom and at the top of footing (case 8).

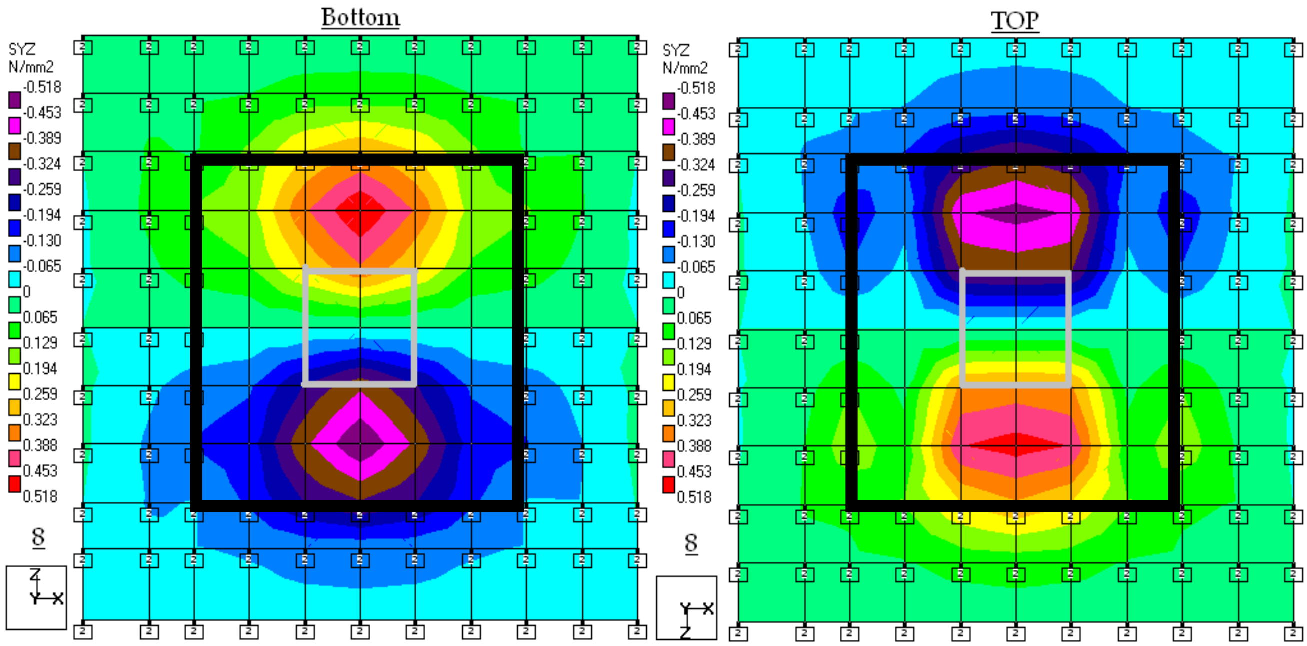

Fig.(22): Distribution of shear stresses Syz at the bottom and at the top of footing (case 8).

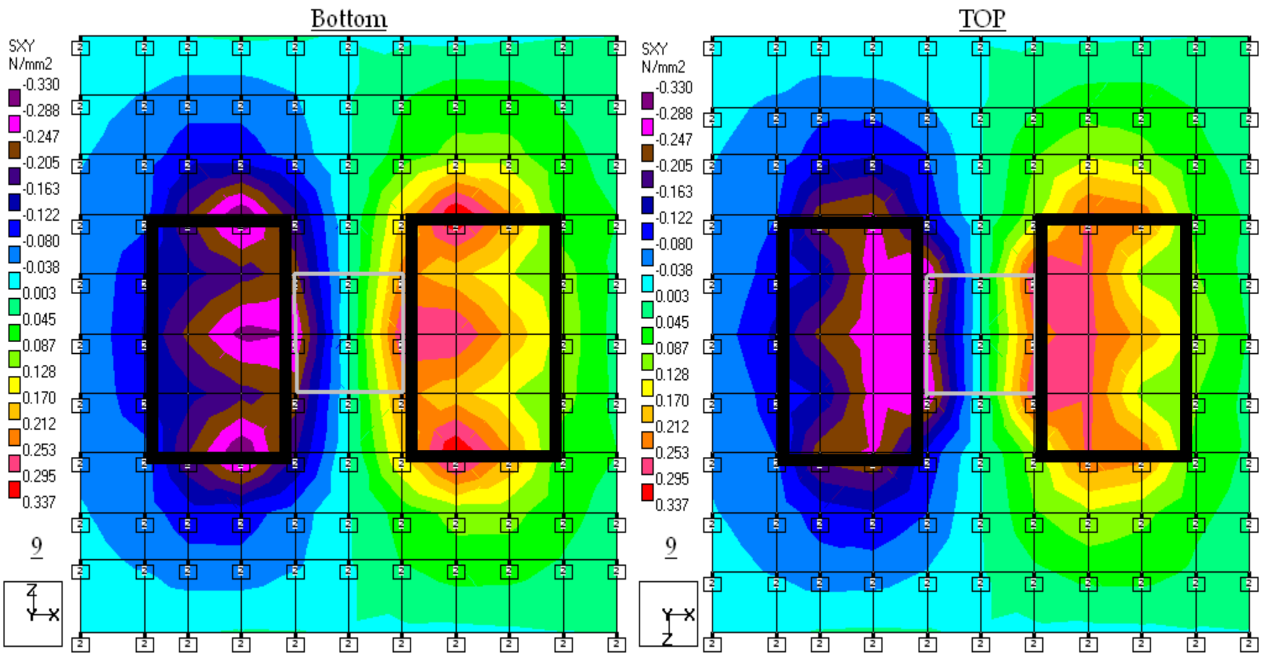

Fig.(23): Distribution of shear stresses Sxy at the bottom and at the top of footing (case 9)

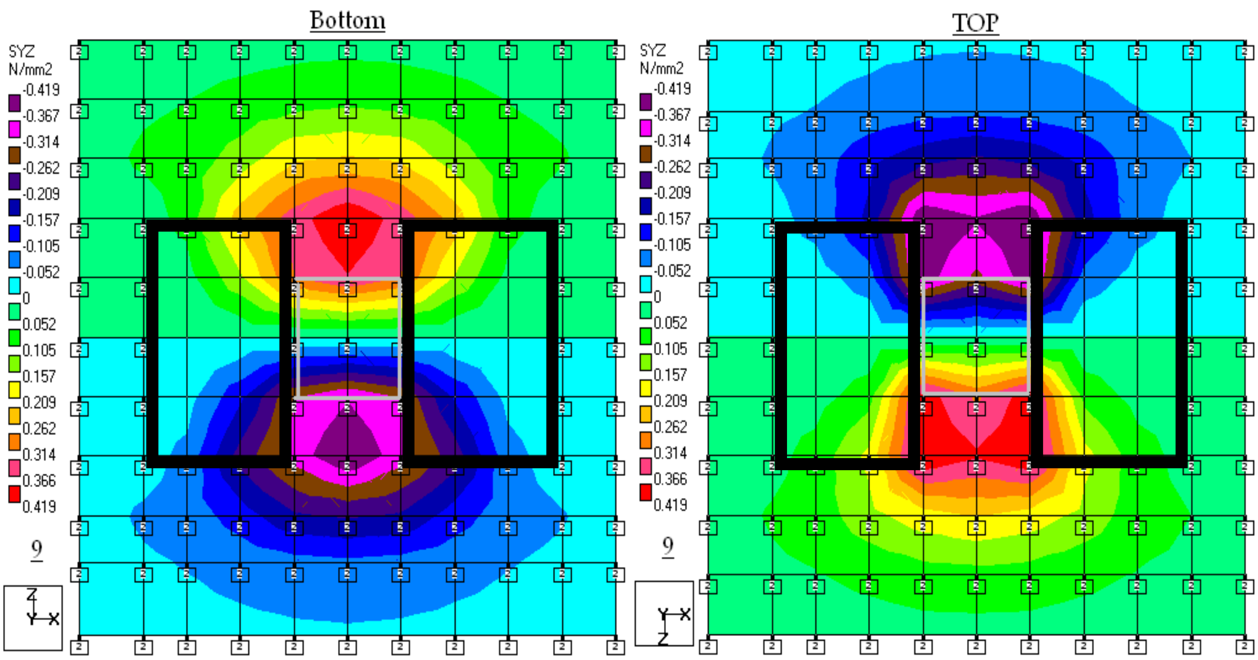

Fig.(24): Distribution of shear stresses Syz at the bottom and at the top of footing (case 9).

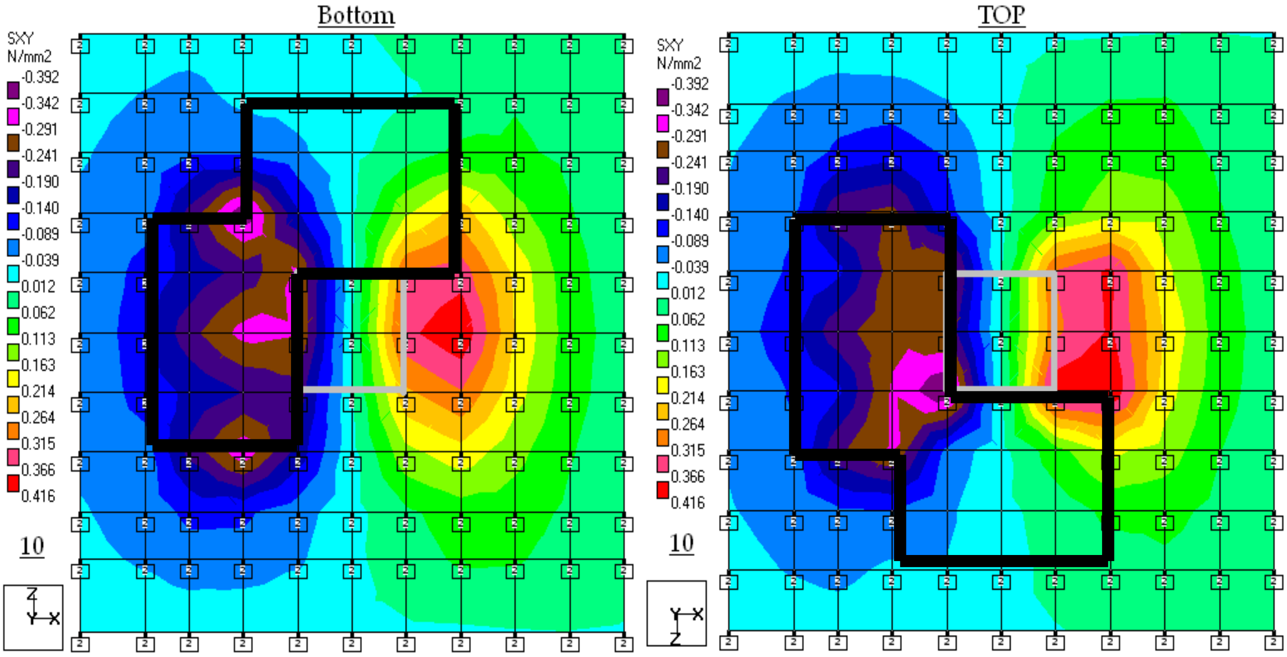

Fig.(25): Distribution of shear stresses Sxy at the bottom and at the top of footing (case 10).

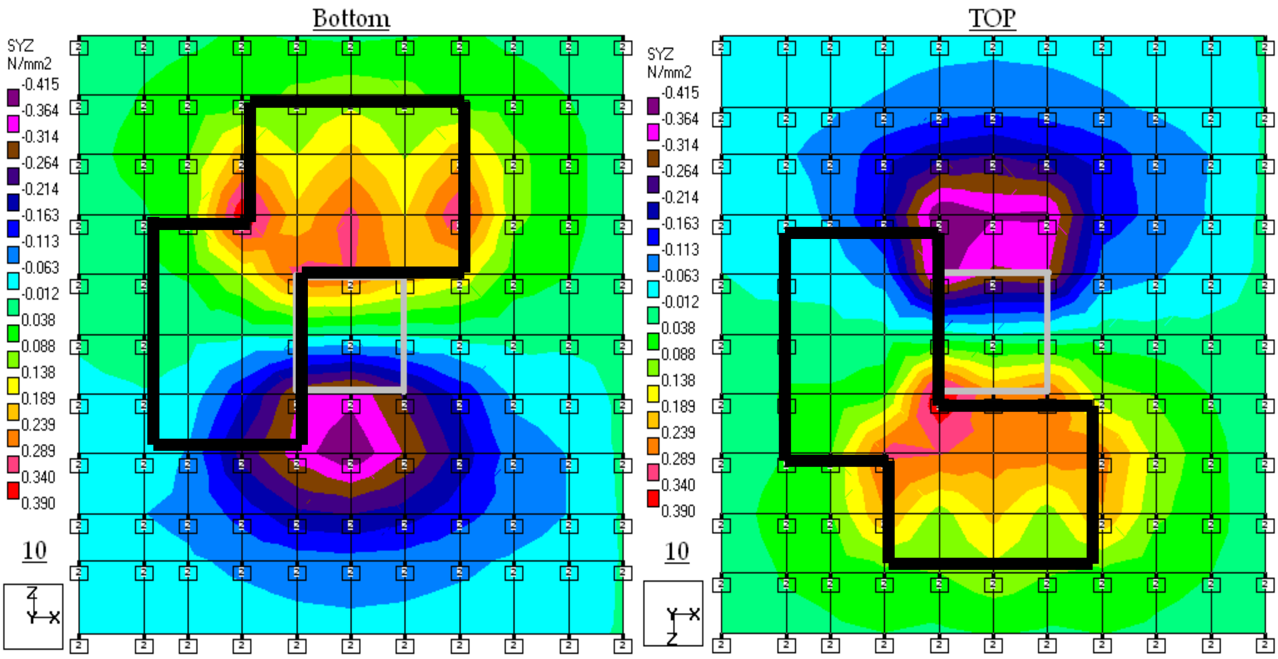

Fig.(26): Distribution of shear stresses Syz at the bottom and at the top of footing (case 10).

توزيع اجهادات القص في الاساس الخرساني عند ظهور تربة ضعيفة تحته

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الخلاصة

في الطرق الاعتيادية لتحريات التربة يتم اختيار مواقع للحفر الاختبارية في الموقع ثم بأستخدام المكائن يتم استخراج نماذج التربة مع اتخاذ الخطوات اللازمة للحصول على نماذج غير مشوشة قدر الامكان. و عند فحص هذه النماذج نحصل على خصائص القوة للتربة. ان تقرير فحص التربة قد يحوي على نتائج فحوص موقعية مثل SPT و هذه المعلومات يحتاجها المهندس الانشائي لغرض التصميم. ان مبدأ تحريات التربة مبني على ان النموذج الصغير المستخرج من الحفر الاختبارية يمثل بشكل كامل و تام طبقة التربة المأخوذ منها و ان نظرية ميكانيك التربة المتقدمة تفرض ان التربة غير متجانسة و غير موحدة الخواص و بهذا فأن خواص القوة للتربة تتغير ليس بتغيير الموقع بل بتغيير اتجاه النموذج للتربة ايضا و ان فرضية تمثيل نموذج صغير من التربة بجسم التربة الاصلي الكبيرهو غير دقيق او يمكن ان نقول بأنه صحيح حسب صحة تطابق خصائص النموذج مع جسم التربة. و من هذا يمكن ان نقول بأن وجود الفجوات او الفراغات بمختلف احجامها في التربة شئ مقبول عمليا. اذا تم اكتشاف هذه الفراغات قبل الانشاء فلا توجد عندئذ مشكلة لكن المشكلة تكمن في عدم معرفة هذه الفجوات لا قبل الانشاء و لا بعده فتكون حجم المشكلة معتمدا على حجم الفجوات و مكانها.

ان هذا البحث يختص بدراسة سلوك الاساس الكونكريتي في حالة وجود فجوات في التربة مباشرة تحت الاساس. و الاساس هو افتراضي يمثل حمل عمود منفرد لبناية ذات ثلاث طوابق بتحميل خفيف. تم تغيير موقع الفجوة (او الفجوات) و حجمها تحت الاساس و بأستخدام نظرية العناصر المحددة ثم تحليل كل حالة على انفراد. التربة تم افتراضها رملية Medium Dense ذات معامل ارتداد قيمته $k = 35000\ kN/m^3$. و لغرض تسهيل الحسابات تم افتراض التربة تحت الاساس متجانسة و سوية الخواص الهندسية. استخدمت اشكال ملونة لتعطي فكرة واضحة على اجهاد القص الموجود في الاساس الكونكريتي حيث ان الدراسة تختص فقط بدراسة اجهادات القص. اما قوة تحمل الكونكريت فتم فرضها 20 MPa.