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FOOTING CONTACT PRESSURE OF BUILDING FOUNDED ON SOFT CLAY TREATED WITH SAND PILES

Waad Abdulsattar Zakaria

University of Diyala/ College of Engineering/ Civil Engineering Department

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ABSTRACT:- This research is devoted to the behavior of a separate single footing constructed over soft clayey soil treated with sand piles. The proposed structure is a multi-storey building having bays 4x4 m. Supporting column has large steel I-section resting on a 2.5x2.5x0.40m footing transferring a presumed load of 460kN. Several patterns of sand piles are proposed in soil foundation and each case is analyzed separately to account for the change in shear and moment in footing. The problem is analyzed numerically by the FE technique using STAAD-Pro/2004 program, while the footing is divided into a FE mesh having 100 element of semi-cubic shape. Degrees of freedom at each node are six. The distribution of soil pressure is measured using Hughes theory, which deals with the bearing capacity of sand piles. Soil modulus of subgrade reaction is assumed 20000kN/m^2.m, angle of friction for sand is 30 degrees, and the undrained cohesion of soft clay is 25kN/m^2. It is concluded that the presence of sand piles in foundation clay does not have much change in the stresses coming from shear and moment distribution in footing compared to same footing constructed on clay without sand piles treatment. Meanwhile, the ACI code failure criteria are considered valid in both cases.

Keywords: soft clay, treatment, sand piles

INTRODUCTION

The presence of soils having poor strength properties is not uncommon in landscapes. Due to the high expense of real estates, the abandon or choosing another site location for a proposed structure is not a choice nowadays. Soil improvement technique is one of the choices available to deal with soil strata possessing low strength properties. One of the common and available methods to increase the bearing capacity and reduce the settlement of soft clays is the use of sand piles (or sand columns), which are constructed, simply, by drilling bore-holes in soil then back-filled with compacted sand having known strength properties. These sand piles gain their bearing capacity by the interaction of active and passive phenomena between the sand and the surrounding clayey soil. In past when there was abandon of landscapes, if one site location does not meet the desired soil strength properties, the location was simply left, to search for another location that fits well with the desired properties. As time passed out this methodology is completely stopped, since land-estate nowadays is, in most, extremely expensive. It may be more expensive than the structure that is intended to be built on. This is a normal case for even domestic houses in Iraq. Sometimes, however, the price of one square meter of estate may be four times, or even more, than the finance of the building itself for one sq.m in cost. Therefore, there is no abandon of any site for the reason of bad soil strength properties. The solution for such critical cases where undesired soil properties are met is summarized as follows:

1- Use of large footing sections associated with heavy reinforcement to account for any high differential settlements or local spots having poor bearing capacity value. This should make the heavily reinforced footing to bridge over local poor soil positions. Of course, this choice mentioned will lead to very expensive foundation cost, in addition to extra measurements that had to be taken in the super-structure, added to that, in turn, further costs.

2- Transfer of structural load through piling system to a deeper soil stratum that possesses higher strength properties. This situation may not be possible every place since this firm soil stratum may be too deep for piles to reach or does not exists at all, such as Basrah location area, south of Iraq. In case of high rise buildings, the additional cost for using piling system is justified, but for ordinary domestic houses this foundation cost could not be reasoned.

3- The final choice is, definitely, change of soil strength properties to better. This choice needs, may be, a text as to be fulfilled and this study is not devoted for soil improvements techniques, but however, one of the soil improvements methods, in concern, is to treat weak soils with sand piles. They are columns in soft soils (usually soft clays) filled with compacted sand. Bore-holes in soft clay are drilled, then backfilled with sand receiving compaction as to form complete sand piles surrounded by soft clay. The design of these sand piles includes the knowledge of many parameters, such as, diameter, depth, and spacing of piles (pattern of distribution), density and angle of friction for sand, and the undrained cohesion of clay, and so the like.

The mechanism of a single sand pile is rather complicated, but in general, they gain their ultimate bearing strength through the passive resistance obtained from the surrounding soil, i.e. passive resistance of clay against the sand in pile. The columns work as drains, and that's why they are called sand drains as well. In such a case, they shorten the longest drainage path for clay thus accelerating terribly the consolidation process and in the same time increase the undrained shear strength and strength properties of the clay through draining water out from them. There are numerous charts, equations, and theories concerning the ultimate bearing capacity of those piles. Nevertheless, one of most popular theories that are used in soil mechanics literatures is the Hughes et al (1975)[1] equation, which is going to be used in this study,

$$\sigma_{vg} = 6\ N\varphi\ cu$$

where,

σ_{vg} = the maximum vertical stress that can be applied to a sand pile.

$N\varphi$ = Lamb's bearing capacity factor = $(1+\sin\varphi)/(1-\sin\varphi)$ or $\tan^2(45+\varphi/2)$

cu = the undrained shear strength of clay.

Φ = the angle of shearing resistance of sand.

SIMPLE HISTORICAL CASE IN IRAQ

The treatment of soft soils with sand piles needs, in certain, large machinery and equipment, which may be available only in governmental sector or large-scale joint-venture companies or contractors. Nevertheless, in early seventies of the last century, at Basrah/ Al-Bakr seaport, there, the upper soil stratum is underlain with a layer of very soft clay (as usually are the soil condition in Basrah region). The soft clay stratum is inclined to some critical angle towards the sea, the situation that made the upper stratum to slide very slowly in the direction of the gulf. Thereafter, an extensive, large-scale program was initiated by treating the whole area with large numbers of sand piles hopping to increase the shear resistance of clay and stopping this slope stability failure and it was the case until now. Although the intention here is rather different, i.e., the purpose of using the sand piles was not to increase the bearing capacity but to increase the shear resistance, the sand piles were indeed successfully used in Iraq. On the other hand, (and due to author’s work in some local projects) the sand piles did used in Baghdad to increase the bearing capacity in the project known before as the Big-Mosque in center Baghdad. Moreover, as in Basrah, they were used with success as well in the capital. Another location where sand piles were used was in the project known as the Rahman-Mosque near Al-Mansour district in Baghdad. The situation is similar to the Big-Mosque project, i.e. to increase the bearing capacity of weak soil under footings. Good results were obtained in both projects.

MODELING OF PROBLEM

In order to have a realistic study, a two-storey building is considered in this research having equal bays of 4x4m center to center. The roofs are reinforced concrete with thickness of 150mm. Columns are steel, each having an I-section and lateral overall dimensions of 0.5 x 0.5m. Instead of assuming a DL for partitions, each roof is assumed to carry two brick-walls 0.25m thick x 3.00m height x 4m length, each. The LL is assumed 1.5kPa. Now, simple calculations lead to a total non-factored load carried by each steel I-column of approximately 460kN. The foundation soil is soft clay having modulus of subgrade reaction of 20000kN/m^2.m. and an undrained cohesion of 25kPa. The footing, which is 2.5m x 2.5m x 0.40m (thick), is of reinforced concrete having f'c=20MPa. It is sometimes reasonable to assume a low value of concrete compressive strength especially for foundations in our country. This can be attributed to the low level of work control. Also the engineer may encounter water table at shallow depths in most times. Some of the ground waters may contaminate the fresh concrete mix for footing during foundation casting. So, it is another reason for assuming a low compressive strength for concrete footing. The presumed contact pressure going to soil is 74kPa. For preliminary design purposes, presumed bearing values are given in British Standards 1986(2) being pressure range which would normally in an adequate factor of safety against shear failure for particular soil type, but with out consideration of settlement. For soft clay it is <75kPa which make soil pressure in study in its maximum limit

MECHANISM OF MOMENT AND SHEAR FAILURE

The strength of concrete in tension is an important property that greatly affects the extent and size of cracking in structures (Wang et al, 1985(3) and Ferguson 1981(4)). Tensile strength is usually determined by using the split-cylinder test. The tensile strength is a more variable property than compressive strength, and is about 10 to 15% of it. The split-cylinder tensile strength f_{ct} is found, through testing, proportional to $\sqrt{f'c}$ such that $f_{ct} = 0.5$ to $0.6 \sqrt{f'c}$ MPa. The ACI code 2008(5) has indirectly used $f_{ct} = 6.7 \sqrt{f'c}$ psi. On the other hand, the tensile strength in flexure, modulus of rupture, is also important when considering cracking &

flexure. The modulus of rupture, f_r, computed from the famous flexural formula f=Mc/I gives higher values for tensile strength than the split-cylinder test. The ACI code takes $f_r = 0.62\sqrt{f'c}$. If the footing in this study is loaded with a column load, there are induced flexural stresses that may control the safety of footing. Now, from the foregoing discussion, the ACI code put the f_r as a failure limit for concrete in flexure. Nevertheless, the author believes that introducing sand columns in soil foundation do not alter these criteria. However, instead of comparing the STAAD maximum stress numbers with f_r, he feels that the f_{ct} is more safe to compare, although theory does not support that. Thus the maximum governing tensile stress in footing concrete is limited to $f_{ct} = 0.5\sqrt{f'c} = 0.5\ \sqrt{20} = 2.236$ MPa (without safety factor). Here the lower case of Wang (2) is used. On the other hand, the shear strength of footing in the vicinity of a concentrated load is governed by the more severe of the two conditions as stipulated by the ACI code.

1- The beam action with a critical section extending in a plane across the entire width located at a distance d from the face of support. This shear strength $v_c = 1/6\sqrt{f'c} = 0.745$ MPa.
2- Two-way action with a critical section perpendicular to plane of slab and located so that its perimeter b_o is a minimum but need not to approach closer than d/2 to perimeter of column, or, $v_c = 1/6\ (1+2/\beta_c)\ \sqrt{f'c} <= 1/3\ \sqrt{f'c} = 1.491$ MPa.

Moreover, as in the case of the moments, the failure criterion in case of the sand piles existence is not changed and continues to control. Thus, adopting the minimum shear strength value, v_c is 0.745 MPa. The previous suggestions about the moment and shear are completely logical since the pressure of sand piles in foundation soil will change the soil pressure distribution under the footing, other than uniform. It will no longer be uniform; instead, the soil pressure distribution will be higher in the sand piles regions than the remaining soil.

ASSUMPTIONS CONCERNING DESIGN APPROACH

For unreinforced soils, in general, two design approaches may cause differences in working stresses in footing and the analysis as well, namely, to assume a flexible footing or rigid one. In case of flexible footing, the soil is assumed to reflect a reaction against footing through simulated springs concentrated at nodal points. Each spring represents a force derived from the soil pressure multiplied by the area, which the node represents. That is to say, spring forces are not equal throughout the footing. The foregoing situation can be represented by a weighing factor, which reflects the area for that node to represent. In this case, and for the four nodes at the four corners of footing, each bears a weighing factor of 1/4. The nodes at the edge of footing, each bears a weighing factor of 1/2, while the nodes other than those, previously mentioned, each bears a factor of one.

Thus by the foregoing assumptions, the spring force in each node would reflect a reaction related to the area that the node represents. Now, in the case of rigid foundation, the situation is quit different. Here, the rigid footing is assumed to lie down on a "bed" of springs. So each spring bears on an equivalent force derived from the total load applied by column divided by the number of springs available in the whole footing. In other words, the spring's forces are all equal and that is only when the chosen K for springs are the same Ks constant below the footing. Now, which approach is more realistic? is something for the designer to decide. However, author believes that such reinforced footing is rigid enough to assume the second approach to be valid is this study. It is worth to mention here, that none of the two assumptions is totally correct because the footing is neither rigid nor flexible.

THE CASE OF SAND PILES REINFORCING THE SOIL

If sand piles are present in soil, the pressure is no longer uniform under the footing since the stiffness of sand piles is larger than the surrounding soft clay. The critical question is "how will the total load be shared between the soft soil and the sand piles?". However, with author's impression, the proportion load share is assumed as follows:

1- A single sand pile is to carry its full strength, at least eventually in future, and that is as stated earlier,

$\sigma_{vg} = 6\ N\varphi\ cu$ and,

$P_{sp} = 6\ A_{sc}\ N\varphi\ cu$ where,

P_{sp} = strength load of sand pile in kN

A_{sp} = cross sectional area of sand pile, i.e. which has a diameter of 250mm.

Thus, $P_{sp} = 6\ (0.25)^2\ \pi/4\ (1+ \sin\varphi)/(1-\sin\varphi)\ (25) = 22.1$kN, where φ here is 30^o as assumed for the sand in pile.

2- The load carried by the clay, P_{clay}, is then equals to the number of sand piles under footing (n) multiplied by P_{sp}, subtracted from the total load

$P_{clay} = 460\ \text{kN} - n\ .\ P_{sp}$

3- The force in each spring of clay, F_{clay}, is,

$F_{clay} = P_{clay}$ / number of springs in clay = $P_{clay} / (100 - n)$

But this approach is not useful in FE modeling; instead it must be modified a little bit.

Now, how to reflect these information into the FE model? In FE modeling the predominating idea for reflecting the soil subgrade reaction is (Bowles, 1996[6], and Das, 2008[7]):-

1-the stress-strain modulus, Es, and

2-the modulus of subgrade reaction, ks.

The second choice is by far the most preferable one among engineers, and considered much easier in reflecting ks in terms of spring forces. Bowles [3] presented a simplified formula reflecting the ks value with the allowable bearing capacity qa which is based on a settlement of one inch

$$ks = 40(SF)\ qa \quad \text{where SF is s safety factor}$$

Now, again;

$\sigma_{vg} = 6\ N\varphi\ cu = 6*3*25 = 450kN/sq.m$

$ks,_{sand\text{-}pile} = 40*450 = 18000kN/m^2.m$

$F_{spring\ force\ in\ sand\ pile} = 18000*0.25*0.25 = 1125kN/m$

Here the area of sand pile is approximated by 0.25m by 0.25m. And in a similar way for clayey soil,

$qult = cu\ Nc = 5.4*25 = 135kN/sq.m$

$ks_{clay} = 40*135 = 5400kN/cu.m$

$F_{spring\ force\ in\ clay} = 5400*0.25*0.25 = 338kN/m$ = stiffness of the spring.

q_{ult} is the ultimate bearing capacity of clay. Now, the spring force in sand pile is 1125kN/m, while that in clay is 338 kN/m. The forces and degrees of freedom matrices will then be used as proposed here. The foregoing methodology for load share is used in this study and seems logical because after considerable time lag the consolidation process takes place in the clay (due to radial consolidation) leading to larger settlement in clay. This would cause a larger concentration of load on sand piles, which in turn will reach its full loading capability thereafter.

ANALYSIS AND RESULTS

In order to focus on study well, the analysis is presented in two cases and as follows:

1- <u>Case One</u>: In this case, the footing is analyzed without any treatment to soil. The shear and moment in footing is presented in STAAD standard presentation form i.e., in terms of stresses designated as Sxx, Syy, Szz, Sxy, Syz and Szx. The first three are tensile or compressive while the last three are shear stresses. The analyses are used as a reference to the other cases, the maximum stresses in footing are shown in Table (1). In case of moment study, the stress f_{ct} is used as a reference for maximum tensile stresses. There is no reference to any reinforcement since the study is considered as "pure stress analysis." The STAAD program is well equipped with the facility of providing reinforcement to footing, the case that is not going to be discussed here.

2- Case Two: The patterns of sand piles, some are shown in Figure (3), are analyzed. As in case one the study is focused on the stresses developed in footing and compares it with the stresses in case one.

Figure (1) show the FE mesh used for footing. It consists of 100 elements, each of lateral dimension of 0.25 by 0.25m and height of 0.4m which represents the footing thickness. The column load is modeled into nine nodal forces working on the nine central nodal points on top of footing and totaling a load of 460kN. Thus the FE mesh is very simple one and is far from complications. The modulus of subgrade reaction of soil is modeled into springs each having a modulus of 338kN/m as was stated earlier. Now the footing resting on such soft clay is analyzed, and two of the stress contours is shown in Figure (2) for convenience, it represents the Sxy stress in top and bottom faces of footing, please care for Table (1) as well. As can be seen that shear stresses is below the shear strength for beam action and that for punching shear (but noting that safety factors are not included !), while the tensile stresses have indeed passed the f_{ct} tensile strength, thus steel reinforcement is essential. From Figure (2) it can be seen that there are high stress contours near the support which confirms well with the aspects of the ACI code. Stresses become smaller, as can be noticed, when we move away from support, the colors are much indicative.

That was case one mentioned earlier. To impose the conditions of sand piles, six patterns of sand piles are considered and each is presented and analyzed separately, they are the 4, 5, 6, 8, 9, and 13 sand piles under footing. The configuration pattern for 5, 9, and 13 are shown in Figure (3) as an example. The small red spots represent the location of sand piles. In all these patterns the symmetry in footing is taken into account as not to produce a situation of non-balance that may lead to differential settlement, except for the case of 6 sand piles which is symmetric about one axis. Figure (4) shows the Sxy stresses for the top and bottom faces of footing in the case of thirteen sand piles are present under it. An important work floats to the surface now, that is the comparison between results. Referring to Table (1) again, if we take a look on numbers in table and compare between them we reach to a fact concluding that the presence of sand piles have no appreciable effect on stresses in footing. If we compare between the stresses in the case of soil with no sand piles with the case of soil with sand piles presence, we can see that the change in shear and tensile stresses in footing for all cases is really small. Thus, the use of sand piles to improve weak soils does not add appreciable change to design procedures as compared to footing resting on an untreated soil, however, this study is not devoted to foundation settlement, thus it is not included with its effects herein.

CONCLUSIONS

In summary, the presence of weak soils may impose special measures to be taken in order to deal with poor soil strength properties. One of the measures is the use of soil improvement technique, namely, the treatment of soft clay with sand piles. The measurement of piles bearing capacity is done by using well known Hughes theory. There can be two procedures to account for the soil pressure distribution under footing, depending on the presumed rigidity of the concrete. If footing is assumed perfectly rigid, the elastic springs of each nodal point representing clay will bear equal force while the nodes representing the sand piles are calculated using Hughes theory. On the other hand, if the footing is assumed to be totally flexible, then each clay spring force will bear a force which depends on the area that the nodal point represents. In this case, the spring forces are not equal while the springs representing the sand piles are, again, calculated using Hughes theory. The first approach is used in this study though author believes that it is more realistic, and the following is drawn as conclusion:

Several patterns of sand pile-configurations are used in study, namely 4, 5, 6, 8, 9, and 13 piles under footing. Symmetry is taken into account about both axes x and z, except for the case of six sand piles which has symmetry about one axis only. Nevertheless, the analysis is the same else where.

Now last but not least, “a negative result in any research is a result”, it is concluded that the presence of sand piles does not alter in footing the stress distribution and its magnitude as compared to the case of soil untreated with sand piles. And since clay assumed in this study is soft or nearly so, it can be concluded that even stiffer clay is contributed, still the effect of sand piles to stress distribution can be thought to be of negligible effect. This concept may change if a granular material having high degree of friction angle is used in the sand pile material.

And as mentioned earlier, the study does not include the settlement analysis of the case, thus the conclusions drawn here must be taken into account for the case of bearing stresses only, and it is by authors impression that the case of foundation settlement may be the essential problem in this case which must be investigated separately.

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Table (1): Summary of stresses calculated in footing.

Sand-pile pattern	**Tensile flexural stress in MPa**		**Shear stress in MPa**	
	Sxx ≈ Szz(*)	**Szz**	**Syz ≈ Sxy(**)**	**Szx**
No sand-pile	2.630	0.261	0.468	0.373
4 sand-piles	2.600	0.263	0.469	0.359
5 sand-piles	2.520	0.259	0.463	0.322
6 sand-piles($)	2.640	0.264	0.468	0.342
8 sand-piles	2.600	0.260	0.475	0.379
9 sand-piles	2.520	0.256	0.469	0.372
13 sand-piles	2.480	0.252	0.467	0.342

(*) Due to symmetry of footing around all axes these values should be equal but in fact there is slight numerical differences. The maximum tensile values are recorded since it is the most critical in concrete.

(**)

(1) These stresses are the shear stresses across the footing cross section.

(2) Same as the point (*) but the maximum absolute value is recorded since the sign in shear does not make any difference to design.

($) The footing here does not have symmetry between the x and z axis, thus the maximum values are recorded.

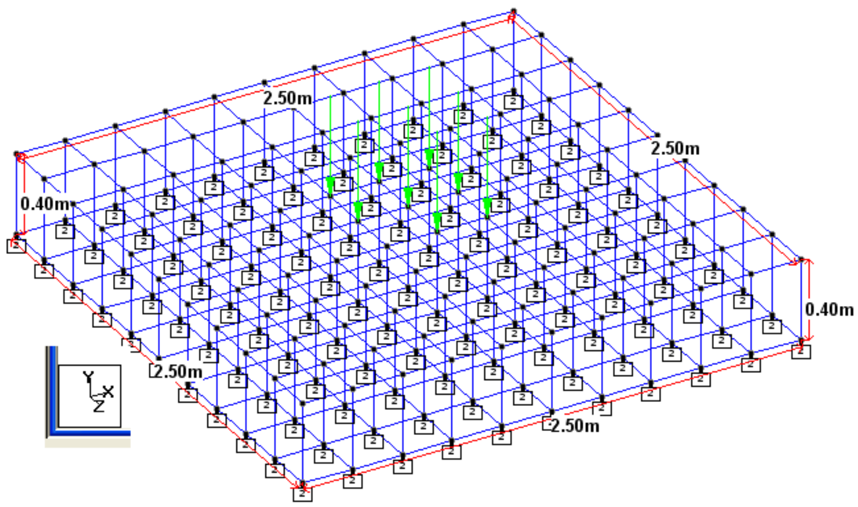

Fig.(1): The finite element mesh used for study footing.

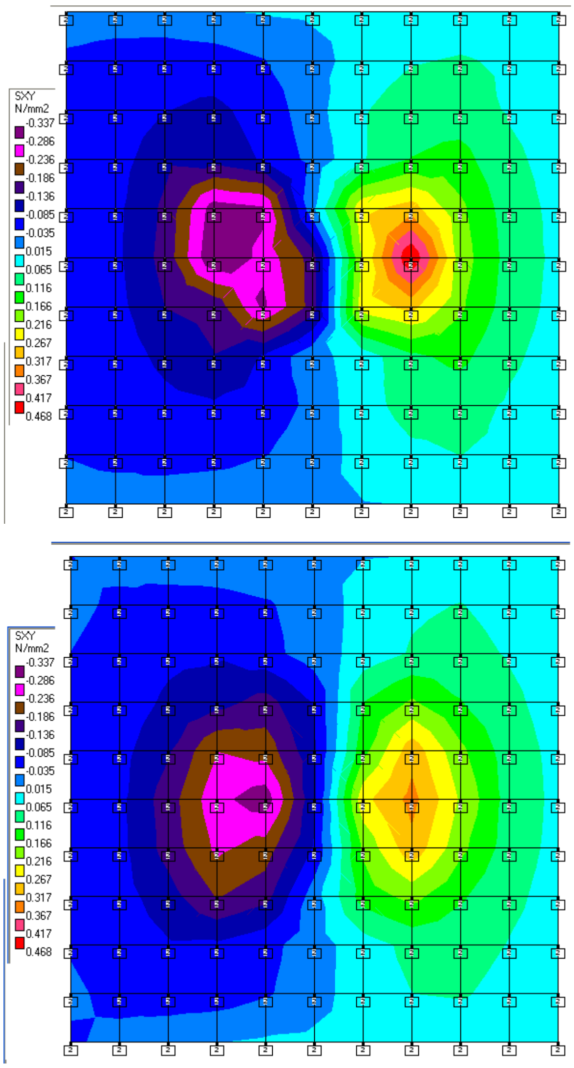

Fig.(2): STAAD Sxy stresses at top face of footing (above), and bottom face of footing (below), for foundation resting on an untreated soft clay.

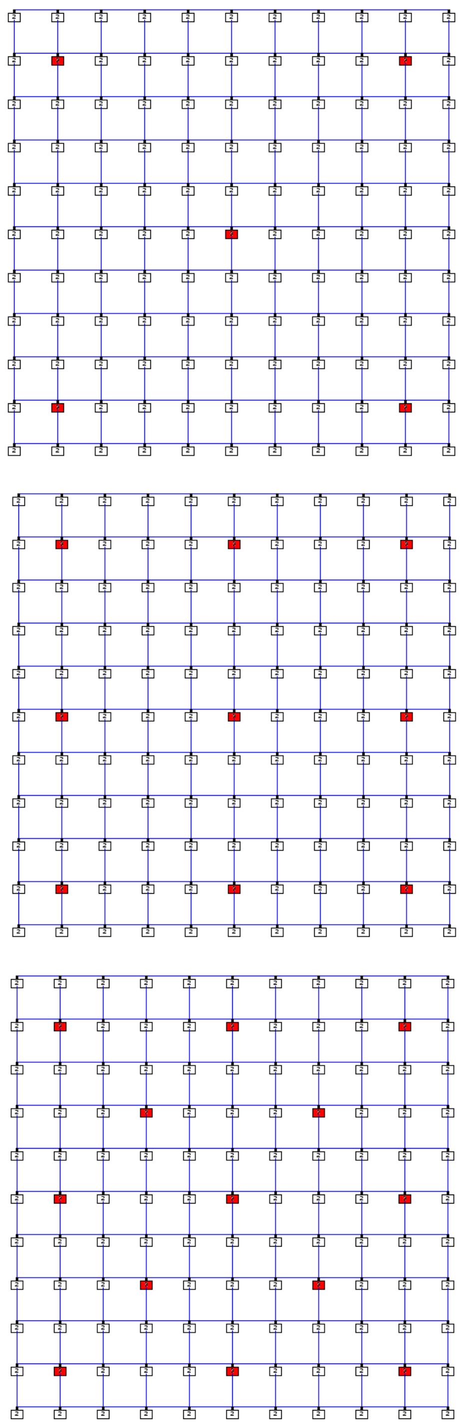

Fig.(3): Some patterns of sand piles(5, 9, and 13)under footing (B=L=2.5m for all).

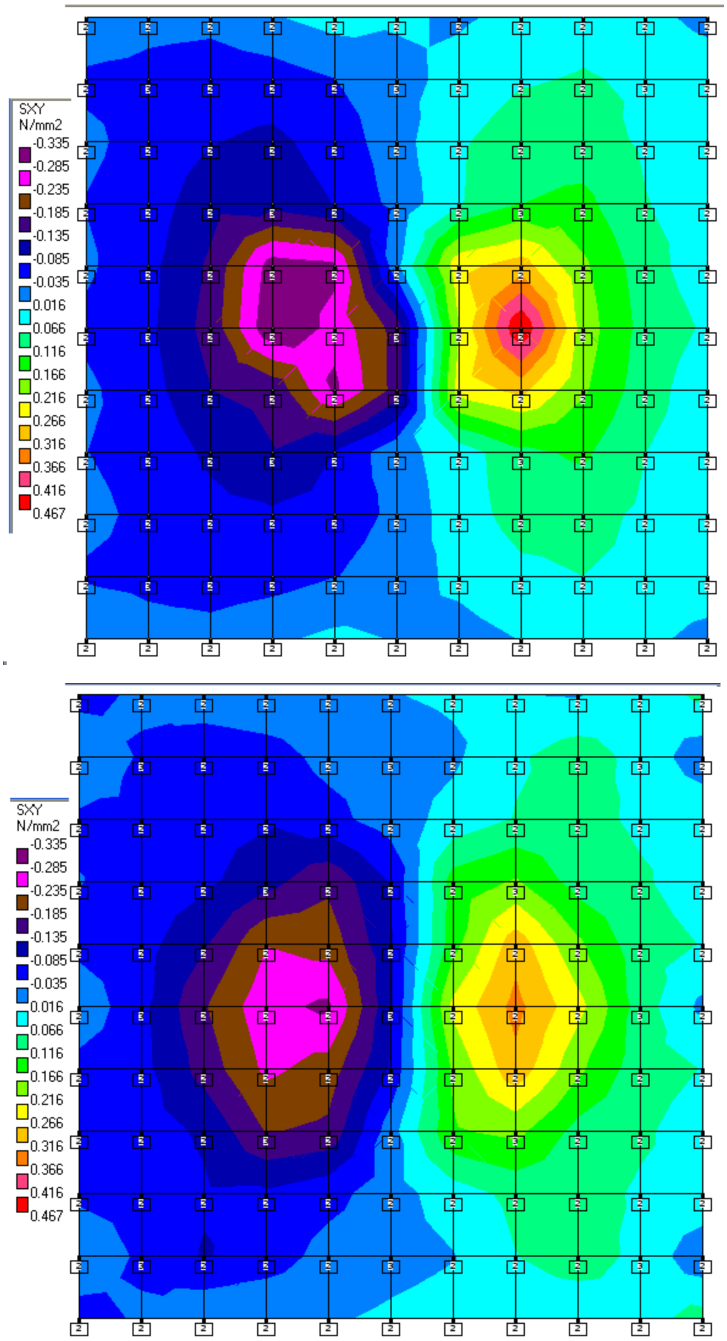

Fig.(4): STAAD Sxy stresses for top (above) and bottom (below) faces of footing resting on clay treated with thirteen sand piles.

دراسة ضغط أساس كونكريتي لبناية متعددة الطوابق منشأ فوق تربة طينية رخوة محسنة بالركائز الرملية

د.وعد عبدالستار زكريا
جامعة ديالى/ كلية الهندسة/ قسم الهندسة المدنية

الخلاصة

إن حالة وجود الترب ذات الخصائص الهندسية الضعيفة يعتبر شيء مشاع في الطبيعة. وبسبب ارتفاع أسعار العقارات والأراضي بكافة أنواعها فأن فكرة التخلي عن موقع معين أو استبداله بموقع آخر تعتبر فكرة غير مطروقة في أيامنا هذه. أن طريقة تحسين التربة لتغيير خصائصها الهندسية تعتبر أحدى الحلول في الوقت الحالي حيث إن هذه الطريقة تتعامل مع طبقات التربة الضعيفة لغرض تحسينها. أحد الطرق المستخدمة في أيامنا هذه لغرض زيادة قابلية تحمل التربة الطينية وتقليل هطولها هو استخدام الأعمدة الرملية (أو الركائز الرملية). وببساطة فأن هذه الركائز الرملية يتم إنشائها عن طريق عمل حفر في التربة الضعيفة ثم يتم إملائها برمل مع الرص وأن هذا الرمل له خصائص هندسية معلومة مسبقا. الركائز الرملية هذه تكتسب قوتها عن طريق تبادل الضغط الفعال وغير الفعال بينها وبين التربة الطينية المحيطة بها.

أن هذا البحث مخصص لدراسة تصرف أساس منفرد منشأ على تربة طينية ضعيفة تمت معالجته بالركائز الرملية. أما الثقل الذي يحمله الأساس هذا فقد تم فرضه متأتي من بناية طابقين ذات أعمدة حديدية تتباعد بنسق منتظم ٤*٤ متر والأساس أبعاده ٢.٥*٢.٥*٠.٣٠ متر وحمل العمود الواحد ٤٦٠ كيلونيوتن. تم استخدام هيئتين لتوزيع الأعمدة الرملية وكل شكل من هذه تم تحليله ومعالجته بشكل منفصل ثم المقارنة لاحقا. استخدمت طريقة العناصر المحددة– STAAD-Pro/2004 للتحليل حيث تم عمل شبكة للأساس FE-mesh من ١٠٠ عنصر، كل عنصر على شكل شبه مكعب. درجة الحرية في كل عقدة تساوي ٦.ولغرض حساب أقصى تحمل للأعمدة والتربة استخدمت نظرية Hughes أما معامل رد فعل التربة فقد تم افتراضه ٢٠٠٠٠كيلونيوتن/المتر المكعب، زاوية احتكاك الرمل ٣٠درجة وا جهاد القص غير المنظم ٢٥كيلونيوتن/المتر المربع. تم الاستنتاج بان تسليح التربة الطينية الضعيفة بواسطة الاعمدة الرملية لايؤثر كثيرا على مقادير وتوزيع الاجهادات الحاصلة بالاساس مقارنة مع بنفس الاساس فوق تربة غير مسلحة. وبنفس الوقت فأن نظريات الفشل الخاصة بالكود الأمريكي تعتبر سارية المفعول.

كلمات الدالة: طين رخو، معالجة، ركائز رملية.