Geomechanics and Engineering, *Vol. 8, No. 1 (2015) 113-132*
DOI: http://dx.doi.org/10.12989/gae.2015.8.1.113

Improvement of bearing capacity of footing on soft clay grouted with lime-silica fume mix

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(*Received March 15, 2014, Revised July 30, 2014, Accepted October 07, 2014*)

Abstract. In this study, lime (L), silica fume (SF), and lime-silica fume (L-SF) mix have been used for stabilizing and considering their effects on the soft clay soil. The improvement technique adopted in this study includes improving the behaviour, of a square footing over soft clay through grouting the clay with a slurry of lime-silica fume before and after installation of the footing. A grey-colored densified silica fume is used. Three percentages are used for lime (2%, 4% and 6%) and three percentages are used for silica fume (2.5%, 5%, 10%) and the optimum percentage of silica fume is mixed with the percentages of lime. Several tests are made to investigate the soil behaviour after adding the limeand silica fume. For grouting the soft clay underneath and around the footing, a 60 ml needle was used as a liquid tank of the lime-silica fume mix. Slurried silica fume typically contains 40 to 60% silica fume by mass. Four categories were studied to stabilize soft clay before and after footing construction and for each category, the effectiveness of grouting was investigated; the effect of injection hole spacing and depth of grout was investigated too.It was found that when the soft clay underneath or around a footing is injected by a slurry of lime-silica fume, an increase in the bearing capacity in the range of (6.58-88)% is obtained. The footing bearing capacity increases with increase of depth of grouting holes around the footing area due to increase in L-SF grout. The grouting near the footing to a distance of 0.5 B is more effective than grouting at a distance of 1.0 B due to shape of shear failure of soft clay around the footing.

Keywords: soft clay; stabilization; lime-silica fume; grouting

1. Introduction

Soft clays are recent alluvial deposits probably formed within the last 10,000 years characterized by their flat and featureless ground surface. There are several approaches which can be used in identifying and classifying soft soil. The oldest is that proposed by Terzaghi and Peck (1967) where clay is regarded as very soft if its unconfined compressive strength is less than 25 kPa and as soft when the strength is in the range of 25 to 50 kPa.

British standard (British Standard B.S: C.P. 8004 1986) defines a soil as soft if its undrained shear strength, c_u, ranges between 20 to 40 kPa. The term very soft refers to soil with c_u < 20 kPa.

Kempfert and Gebreselassie (2006) defined soft soil as clay or silty clay soil which is

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http://www.techno-press.org/?journal=gae&subpage=7 ISSN: 2005-307X (Print), 2092-6219 (Online)

geologically young and come to equilibrium under its own weight but has not undergone significant secondary or delayed consolidation since its formation.

According to the working group excavation EAB of the German Geotechnical Society (as cited by Kempfert and Gebreselassie 2006), the following criteria should be fulfilled in order to define the soil as a soft soil in terms of constructional purposes:

- Very soft to soft consistence with consistency index $I_c < 0.75$,
- fully or nearly fully saturated,
- the undrained shear strength $c_u \leq 40$ kPa,
- inclined to flow,
- light to middle plastic property,
- very sensitive to vibration, and
- thixotropic property.

2. Methods of improving the engineering properties of soft clay

All structures built on soft soil may experience uncontrollable settlement and probably critical load carrying capacity. Soil improvement and stabilizations are the keys for such problems. Soil improvement and stabilization are the collective term for any mechanical, hydrological, physicochemical, biological methods or any combination of such methods employed to improve certain properties of natural soil deposits. The purpose of ground improvement is generally to increase the strength and reduce settlement, or to change the permeability of existing soils.

Soil stabilization in its broadest sense implies the improvement of both durability and strength of soil. Soil may be improved by mechanical stabilization or in other instances cement, lime, bitumen, or special additives are used to bind or waterproof the particles of soil and so increase its strength and durability. Cement treatment is most applied to road stabilization especially when the moisture content of the subgrade is very high. Waste materials or nontraditional stabilizers, such as silica fume and fly ash, are also sometimes applied for stabilization. When the soil has been treated with any of the mixtures mentioned above, it is called stabilized soil. The soil stabilization can be divided into (Fang and Daniels 2006).

Mechanically

Such as compaction control, preloading, soil replacement, adding the deficient particle sizes to give a more satisfactory grading, etc.

Chemically

By the addition of some chemical instances such as cement, lime, asphalt, silica fume, chlorides… etc.

Multimedia (Grouting)

Adding the deficient particle sizes to give a more satisfactory grading and the same time there is chemical reaction between the grout and soil.

2.1 Lime stabilization

Calcium hydroxide (slaked lime) is most widely used for stabilization. Calcium oxide (quick

Table 1 Suggested lime content for soil stabilization (Ingles and Metcalf 1972)

Soil type	Content for modification	Content for stabilization
Fine crushed rock	2-4 percent	Not recommended
Well graded clay gravels	1-3 percent	~3 percent
Sands	Not recommended	Not recommended
Sandy clay	Not recommended	~5 percent
Silty clay	1-3 percent	2-4 percent
Heavy clay	1-3 percent	3-8 percent
Very heavy clay	1-3 percent	3-8 percent

lime) may be more effective in some cases; however, the quick lime will corrosively attack equipment and may cause severe skin burns to personnel. Ingles and Metcalf (1972) recommended the criteria of lime mixture as shown in Table 1.

Lime stabilization causes a significant improvement in soil texture and structure by reducing plasticity and by providing pozzolanic strength gain (Little 1999). The reduction in plasticity will happen by converting the soil to the rigid or granular mass, at the same time, the bonds between the soil particles become stronger, due to the cation exchange that takes place between the ions on the surface of clay particles and the calcium ions of the lime (Akawwi and Al-Kharabsheh 2000).

The traditional lime stabilization can be defined as lime mixed into the soil and immediately compacted without allowing the lime/soil mixture to sit for an extended period of time before compaction (Harris *et al*. 2004). Therefore, the addition of lime can significantly improve both the stiffness and resistance to permanent deformation of clay soils (Rogers *et al*. 2006). However, with the increase in lime content, there is an apparent reduction in clay content and a corresponding increase in percentage of coarse particles (Kumar *et al*. 2012).

2.2 Silica fume

Silica fume, as defined in ACI 116R, is "very fine noncrystalline silica produced in electric arc furnaces as a by-product of the production of elemental silicon or alloys containing silicon". The silica fume, which condenses from the gases escaping from the furnaces, has a very high content of amorphous silicon dioxide and consists of very fine spherical particles typically averaging 0.1 to 0.2 μm in diameter.

Silica fume is a by-product of the manufacture of silicon and ferrosilicon alloys. Nearly 100,000 tons of silica fume, is produced each year throughout the world. Silica fume is 10 to 20 times finer than fly ash and it is pozzolanic because of its high silica contents and its high specific surface area. Before the mid-1970s, silica fume was discharged into the atmosphere. After environmental concerns necessitated the collection and landfilling of silica fume, it became economically justified to use silica fume in various applications.

Abd El-Aziz *et al*. (2004) examined the effect of adding lime and silica fume together on the engineering properties of clayey soils. A series of laboratory experiments have been conducted for varieties of samples: 1%, 3%, 5%, 7%, 9% and 11% for lime and 5%, 10% and 15% for silica fume. It was found that the engineering properties of soil have been improved by adding lime in the range of 5-9% combined with a 10% silica fume. The plasticity index decreased from 40% to

19%, when subjected to a L-SF blend of 11-15%. At L-SF 5-10%, the angle of internal friction concerning shear strength parameters would enhance from 5.80° to 24.75°. Soil cohesion increased as well from 55.52 kN/m^2 to 157.54 kN/m^2. The compressibility index (Cc) was lowered from 0.025 to 0.007. All of these results can be summarized to say that by blending lime and silica fume together, the engineering properties of clayey soils can be enhanced.

The main objectives of strengthening the soil mass are to improve stability, increase bearing capacity and reduce settlements and lateral deformation. There are several methods for improving the soil. One of the approaches is the use of geosynthetic materials. Geosynthetic is a well known technique in soil reinforcement. The use of geosynthetic three dimensions can significantly improve the soil performance and reduce costs in comparison with conventional designs. Marto *et al*. (2013) made a review of experimental test carried out by different researchers on reinforced soil with synthetic materials specially geocell had been made. Test results indicated that the inclusion of reinforcement in the sand decreased settlements and leading to an economic design of the footings.

Stone columns are found effective, feasible and economical to improving the soft and loose layered soil. Stone columns increase the unit weight and the bearing capacity of soil. It can densify the surrounding soil during construction. The improvement of a soft soil by stone columns is due to different sizes of aggregate (size between 2 to 10 mm) in the soft soil.

Fattah *et al*. (2011) conducted a testing program to study the influence of stone column number (single, two, three, and four stone columns) , *L/d* ratio and undrained shear strength of bed soil on the stress concentration ratio and the bearing improvement ratio (qtreared/ quntreated) of stone columns. The experimental tests showed that the stone columns with *L/d* = 8 provided a stress concentration ratio n of 1.4, 2.4, 2.7, and 3.1 for the soil having a shear strength c_u = 6 kPa, treated with single, two, three, and four columns, respectively. The values of n were decreased to 1.2, 2.2, 2.5, and 2.8 when the *L/d* = 6. The values of n increase when the shear strength of the treated soil was increased to 9 and 12 kPa. The value of the bearing improvement ratio decreases with increasing the shear strength of the treated soil.

Das and Pal (2013) presented the utilization of stone column to improve the load capacity of sandy silt soil with clay in naturally consolidated state. Load tests through the compression testing machine are performed on single un-encased stone column in sandy silt soil with clay (i.e., sand = 37.29%, silt = 33.00% and clay = 29.71%). Un-encased and encased (with geotextile) stone column behavior on layered soil also discussed in this investigation. In case of un-encased stone column load carrying capacity increases with the increasing diameter of the stone column but in un-encased and encased layered soil load carrying capacity decreases with the increasing diameter of the stone column. The load bearing capacity of stone column in un-encased and encased layered soil decreases with the increasing diameter of stone column.

A mixture of biomass ash (BA) and calcium carbide residual (CCR) was used by Vichan *et al*. (2013) as a cementing agent for stabilization of soft Bangkok clay. The improvement in unconfined compressive strength of stabilized clay depends on the initial soil water content, binder content (B), CCR:BA ratio, and curing time. The strength improvement can be classified into two zones: active and inert. In the active zone (B = 5-15%), the $Ca(OH)_2$ content from the CCR is high and there are insufficient natural pozzolanic materials in the clay for the reaction. The input of BA increases the SiO_2 content and induces the strength development. In the inert zone, (high binder content, B > 15%), the free lime causes unsoundness and results in insignificant strength development. The hydration products from the pozzolanic reaction (ettringite and a calcium aluminate silicate hydrate compound, gismondine) are identified by microstructural analysis via

X-ray diffraction (XRD), scanning electron microscope with energy dispersive X-ray spectroscopy (SEM with EDS), and X-ray fluorescence. SEM images show the compact morphology of the stabilized clay as the result of the increase in curing time and binder content. Over time, the cementitious products fill in the pore space, causing denser morphology and higher cementation bond strength between the clay clusters. The clay stabilized by XRD showed no reduction in the intensity of the reflection of the both quartz and kaolinite throughout the curing times. This implies that the amorphous silica from the glass of the BA is more reactive in dissolution in the CCR than the crystalline phase of quartz and kaolin in clay.

Experimental and numerical investigations into the bearing capacity of circular footing on geogrid-reinforced compacted granular fill layer overlying on natural clay deposit have been conducted by Demir *et al*. (2013). A total of 8 field tests were carried out using circular model rigid footing with a diameter of 0.30 m. 3D numerical analyses were performed to simulate soil behavior using finite element program Plaxis 3D Foundation. The results from the FE analysis were in very good agreement with the experimental observations. It was shown that the degree of improvement depends on thickness of granular fill layer and properties and configuration of geogrid layers. Parameters of the experimental and numerical analyses include depth of first reinforcement, vertical spacing of reinforcement layers. The results indicated that the use of geogrid-reinforced granular fill layers over natural clay soils has considerable effects on the bearing capacity and significantly reduces the lateral displacement and vertical displacement of the footing.

In this study, an attempt is made to stabilize the soft clay by grouting the soil by a slurry of lime-silica fume material. It is intended to examine the suitability of lime and silica fume as a stabilization material to reduce the development of desiccation cracks in compacted clayey soils and improve their strength characteristics. Thus, the objectives of this study is grouting the soft clay beneath and around a foundation model with lime-silica fume grout to investigate the optimum spacing and depth of grouting that reveal a suitable bearing capacity of the foundation.

3. Experimental work

The natural soil used in this study is a brown silty clay soil brought from Baladroz site east of Baghdad in Iraq. Standard tests were performed to determine the physical and chemical properties of the soil. The grain size distribution of the soil used reveals that the soil is composed of 3.5% sand, 31.5% silt and 65% clay. The results of chemical analysis for the soil used is presented in Table 2, while Table 3 shows the physical properties of the soil used which is classified as CL according to the Unified Soil Classification System (USCS).

Table 2 Chemical analysis results

Index property	Index value
Total SO_3%	1.8
Gypsum content%	2.92
Total dissolved salts, T.D.S%	3.7
Organic matter, O.M.%	0.73
pH value	9.32

Table 3 Physical properties of the soil used

Physical properties	Index properties	Index value
Atterberg limits	Liquid limit, L.L (%)	46
	Plastic limit, P.L (%)	20
	Plasticity index, P.I (%)	26
Gain size analysis	% sand (0.075-2) mm	3.5
	% silt (0.005-0.075) mm	31.5
	% clay (< 0.005) mm	65
Activity	_	1.3
Specific Gravity G_s	_	2.65
Compaction test	Max. dry unit weight (kN/m^3)	17.1
	Optimum moisture content, %	17

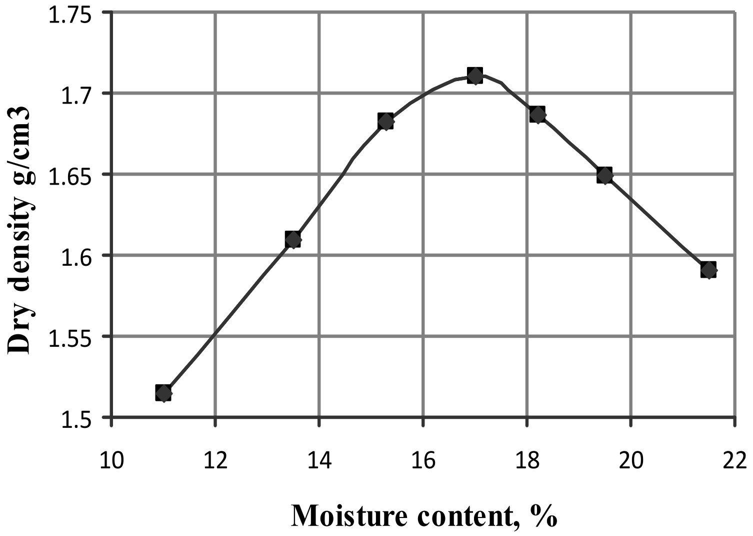

Fig. 1 Compaction test results for untreated soil

3.1 Compaction test (ASTM D698-00)

Compaction, in general, is the densification of soil by removal of air, which requires mechanical energy, and the degree of compaction of soil is measured in terms of its dry unit weight (Das 2002). In order to obtain the moisture density relations for the tested soil, standard Proctor test was carried out. The dry unit weight-moisture content relationship for the soil used is shown in Fig. 1.

3.2 The silica fume material

In this study, a grey-colored densified silica fume is used. It is a pozzolanic material which has

Table 4 Chemical analysis of silica fume

Property	Composition (%)
SiO_2	> 85%
C (free)	< 4%
S	< 1%
Fe_2O_3	< 2.5%
Al_2O_3	< 1%
CaO	< 1%
$K_2O + Na_2O$	< 3%
Cl	< 0.2%
L.O.I.	< 6%
Moisture	< 2%
Specific surface	~20 m^2/gr

Table 5 Shear strength parameters for soil stabilized with lime, silica fume and L-SF mix

Lime %	0%	2%	4%	6%
c_u, kPa	48	64	106	73
ϕ_u, (°)	3	14	16	17
SF%	0%	2.5%	5%	10%
c_u, kPa	48	77	129	116
ϕ_u, (°)	3	16	20	18
L-SF%	0%	2-5%	4-5%	6-5%
c_u, kPa	48	135	187	147
ϕ_u, (°)	3	22	26	24

a high content of amorphous silicon dioxide and consists of very fine spherical particles. Silica fume was used as an additive material to improve soil properties. The product used is called MEYCO®MS610 which contains extremely fine (0.1-0.2 μm) latently reactive silicon dioxide. The presence of this substance gives greatly improved internal cohesion, water retention and increased density when set.

The additional crystal formation and the fineness of MEYCO®MS610 produce a significantly more dense set cement matrix. The results of the chemical analysis of this material are presented in Table 4. All products of silica fume available are in densified silica fume type to make these products dense enough to be transported economically and be handled like Portland cement. The densification process greatly reduces the dust associated with the as-produced silica fume. Therefore, when used in grouting model, in this study the silica fume should be grind by pulverization machine with capacity 0.25 kg for at least 3 hours.

3.3 Unconfined compression test

The unconfined compressive strength UCS value of a compacted soil is the most common and adaptable method of evaluating the strength of stabilized soil. It is the main test recommended for the determination of the required amount of additive to be used in stabilization of soil. UCS tests were performed on natural soil and soil with (2%, 4%, and 6% lime) and soil with (2.5%, 5% and 10% SF) and soil with combination of optimum SF and different lime percents (2, 4, and 6). All the unconfined compressive tests were carried out on specimens with optimum moisture content and maximum dry unit weight. UCS testing for soil-lime, SF and L-SF mixtures was carried out after curing time at 20°C for 28 days and 100% relative humidity. The specimens remain in moulds as suggested by Little, (1995). The unconfined compressive strength test was carried out following the ASTM D 2166-00 specification. Table 5 shows the relationship between the undrained angle of internal friction and lime and silica fume content percents.

Based on the unconfined compression test results, the optimum L-SF percent was found to be 4:5% (4% lime and 5% silica fume).

4. Preparation of model test

All model tests were conducted using the setup shown in Fig. 2, which consists of a frame, soil tank, bed of soil and grouting apparatus. The vertical load was applied to the model foundation by dead weights put in the axial shaft with sensitivity 0.001 gram and two deformation dial gages with 0.01 mm sensitivity have been used for measuring displacements of the square steel footing with side length of 100 mm.

The soil tank has 0.5 m length, 0.5 m width, and 0.6 m height as shown in Fig. 2. It is supported by the frame. The dimensions of the tank were chosen so that the tank can be put inside the testing frame.

Prior to the stage of preparation of the bed of soil, trial tests were performed to control the efficiency of the method of preparation. Control tests were carried out to check two main points of vital importance regarding the preparation of the homogeneous bed of soft soil. The first one is to determine the variation of undrained shear strength with different water contents (or at different liquidity indices).

Several trails were made and typical results are presented in Fig. 3. The shear strength of soil decreases with increasing value of liquidity index ($LI = \frac{w - PL}{LL - PL}$ where LL = liquid limit, PL = plastic limit, and w = in-situ moisture content). Furthermore, decrease in the rate of strength also decreases with increasing the value of the liquidity index. The second point is determining the variation of shear strength of the soil with time after mixing. The tests provided the time required for the remolded soil to regain strength after a rest period following the mixing process. To accomplish this point, nine samples were prepared individually and placed in five layers inside CBR moulds. Each layer was tamped gently with a special hammer to extract any entrapped air.

The samples were then covered with polythen sheet and left for a period of eight days. Each day, the undrained shear strength was measured by a vane shear device. The results of the variation of the undrained shear strength with time are shown in Fig. 4. For the range of liquidity index, the influence of time decreases with increase of liquidity index.

4.1 Preparation of the bed of soil

The test was conducted at liquidity index of 0.4 corresponding to an undrained shear strength

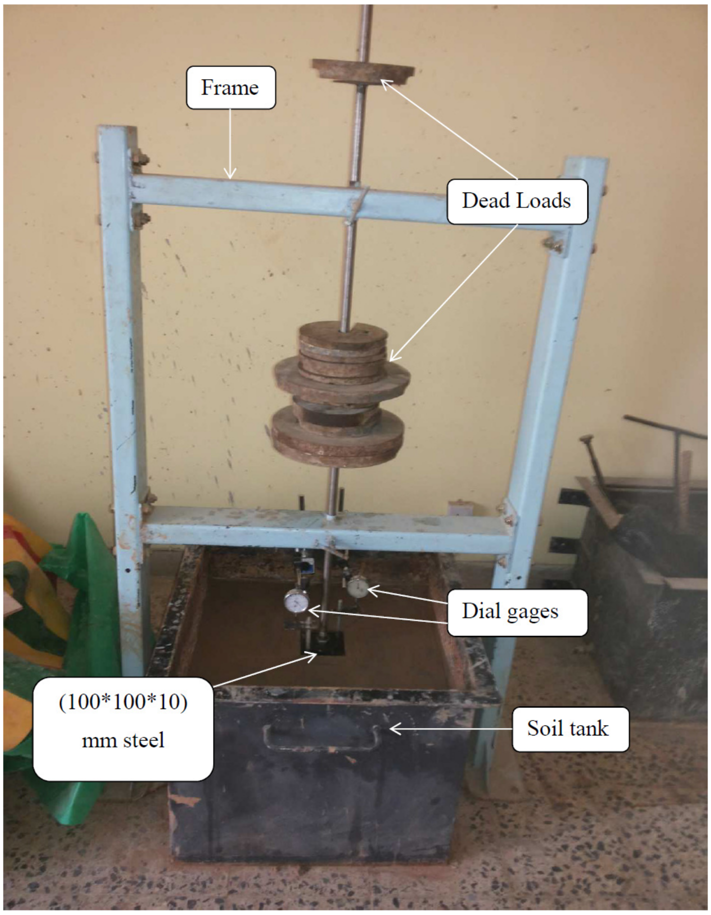

Fig. 2 Setup of the laboratory footing model

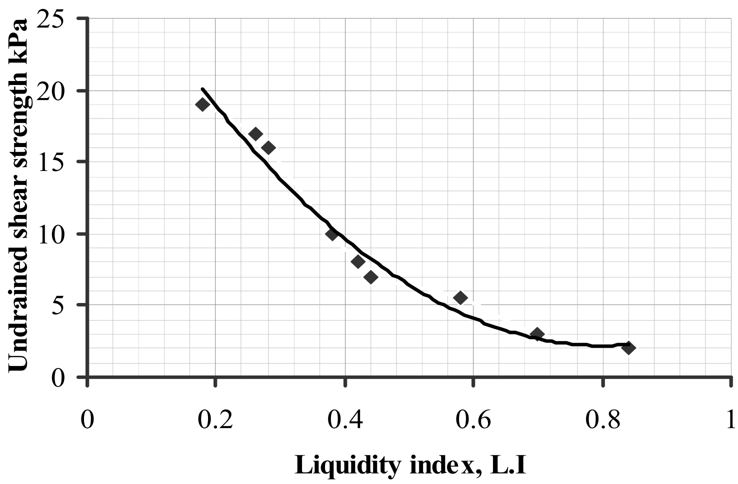

Fig. 3 Variation of the undrained shear strength with liquidity index for the remolded clay after 48 hours

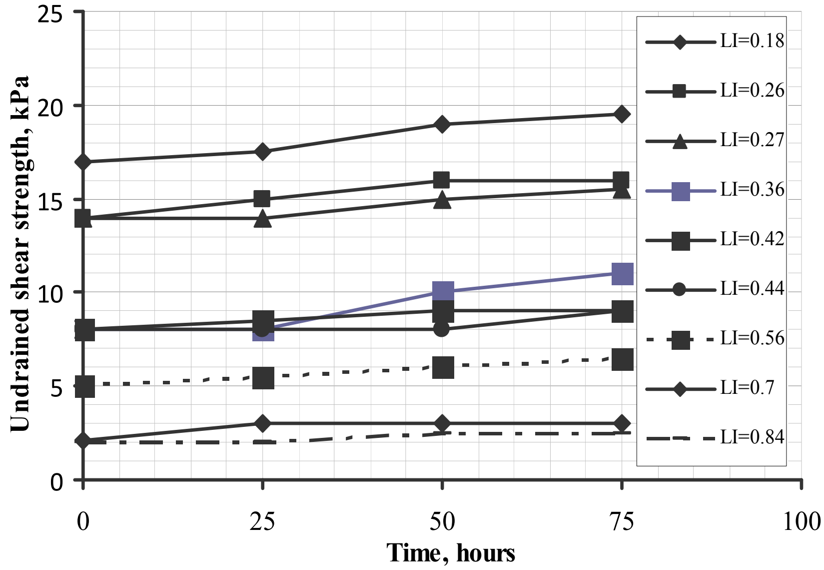

Fig. 4 Variation of the undrained shear strength with time for the remolded clay

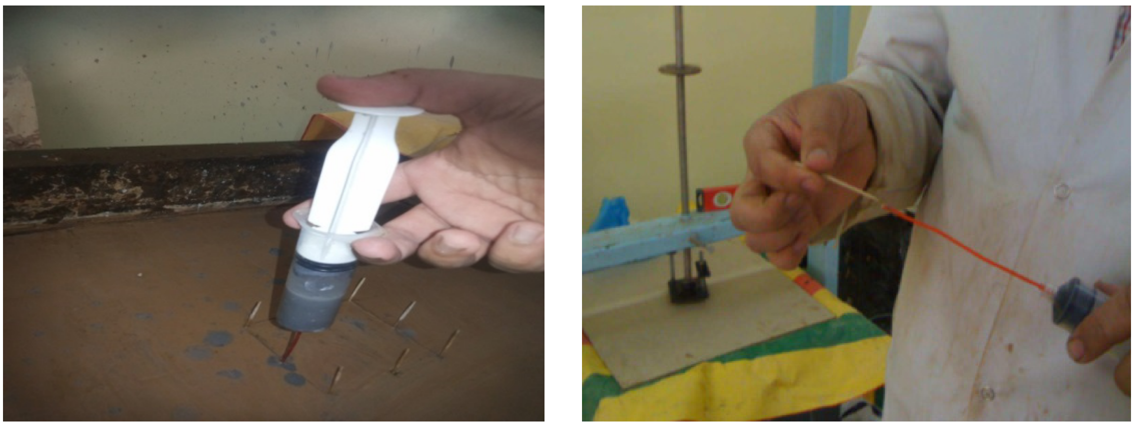

Fig. 5 Grouting apparatus

$c_u = 9$ kPa. A quantity of 225 kg of natural soil was mixed with enough quantity of water to get the desired consistency. The mixing operation was conducted using a large mixer (120 ℓ) capacity manufactured for this purpose, each 25 kg of dry soil was mixed separately till completing the whole quantity. After thorough mixing, the wet soil was kept inside tightened polythen bags for a period of two days to get uniform moisture content. After that, the soil was placed in the soil tank in ten layers, each layer was leveled gently using a wooden tamper, then the leveled layer was tamped gently with a metal hammer of 9.87 kg and dimensions of (150 × 150) mm in order to remove any entrapped air. This process was repeated for the ten layers till reaching a thickness of 400 mm of soil in the steel container. After completing the final layer, the top surface was scraped and leveled to get as near as possible a flat surface, then covered with polythen sheet to prevent any loss of moisture.

4.1.1 Grouting apparatus

A simple apparatus was designed for the purpose of this study as shown in Fig. 8. The apparatus consists of two parts as follows:

4.1.2 Needle pressure

A 60 ml needle was used as a liquid tank of the lime-silica fume mix and the maximum pressure which can be applied by hand is about 3 kN/m^2.

4.1.3 Injection pipe

The grout is injected into the soil through a pipe which has a diameter of 1 mm and length of 150 mm. One side is closed and four perforations in different directions are made. The slurry flows through the perforations and penetrates the soil, as shown in Fig. 5.

4.2 Lime- silica fume grout mixes

From the UCS test, the optimum percent of L-SF was 4% L and 5% SF. Therefore, it was suggested to prepare L-SF grout mixes of the same optimum percent and ordinary drinking water. Slurried silica fume typically contains 40 to 60% silica fume by mass and slurry of 50% solids content will contain about 700 kg/m^3 and dry material versus 130 to 430 kg/m^3 (ACI 234R-06 2012). However, there is uncertainty of the optimum percent of the water.

Three soil samples in CBR mould were grouted by 40 ml of the L-SF mixes with different water contents (40%, 50%, and 60%) with the same depth and curing time of 7 days. Each soil sample was subjected to 10 kg load applied on a circular footing 60 mm in diameter and the settlement was recorded. The mould water content which provided minimum settlement was chosen.

4.3 Procedure of grouting the model

Small quantity of lime-silica fume slurry was prepared because the pozzolanic reaction between lime and silica fume causes aggregation of the slurried particles and close the pipe injection. Therefore, 40 g of lime and 50 g of silica fume were mixed to be homogenous, and grouted to the bed of soil in specified points following a uniform pattern, Fig. 6. Then a quantity of water (60%) by weight was added to the mix and the slurry was striked down by a plastic table spoon for a gel time of 20 seconds.

Injection pipe was penetrated through the soil to the desired depth then injection started by hand pressured needle. The control of grouting pressure is vital to the success of any grouting operation. Two cases were studied to stabilize soft clay before and after footing construction and for each case, the effectiveness of grouting spacing and depth of grout was investigated.

4.3.1 Before footing construction

Four different spacings were investigated for the depth of grout equal to B (footing width), Fig. 7.

Optimum spacing of grouting points is investigated by using spacing of 0.5 B and 1.0 B holes around the footing to a depth of 1.0 B, Fig. 8.

Fig. 6 Spacing of grouted points in the soil

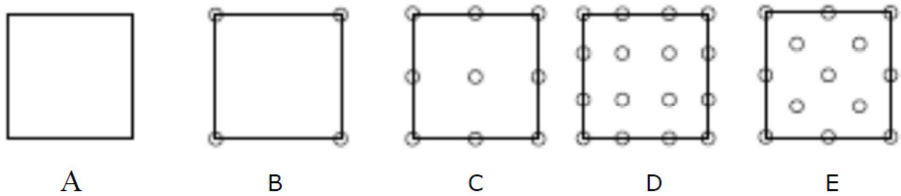

Fig. 7 Grouting spacing before footing construction with a depth of grouting equals to B

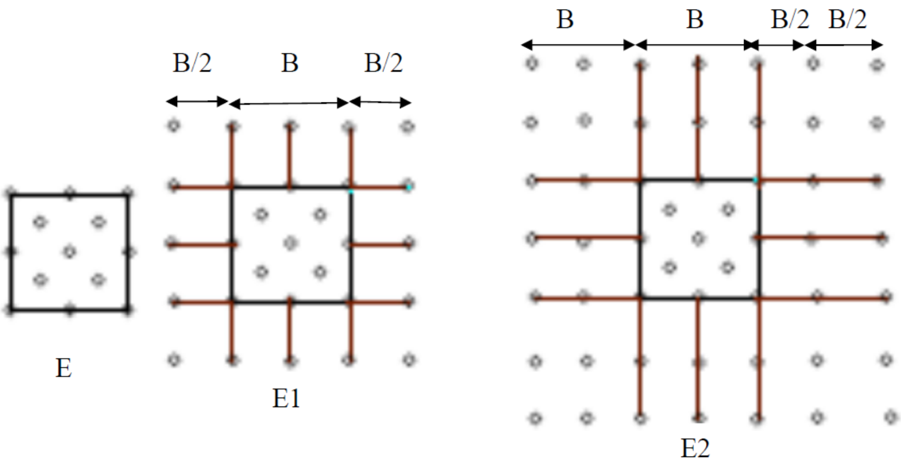

Fig. 8 Spacing of grouting points with depth of grouting equals to B

It is intended to study the effect of the depth of grouting, therefore models E, E1 and E2, are grouted to a depth of 1.5 B.

4.3.2 After footing construction

Grouting can be used successfully to level an inclined building on the thick soft clay deposit with a final compensation efficiency of 10% (Ni and Wen 2009). Therefore, it is important to

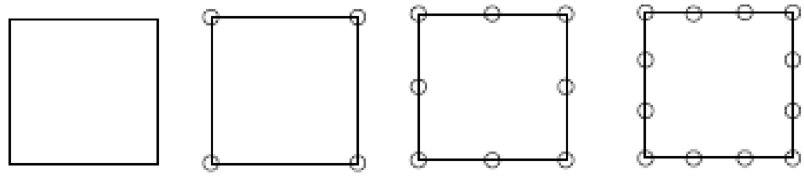

Fig. 9 Grouting points grid after footing construction, grouting to depth B

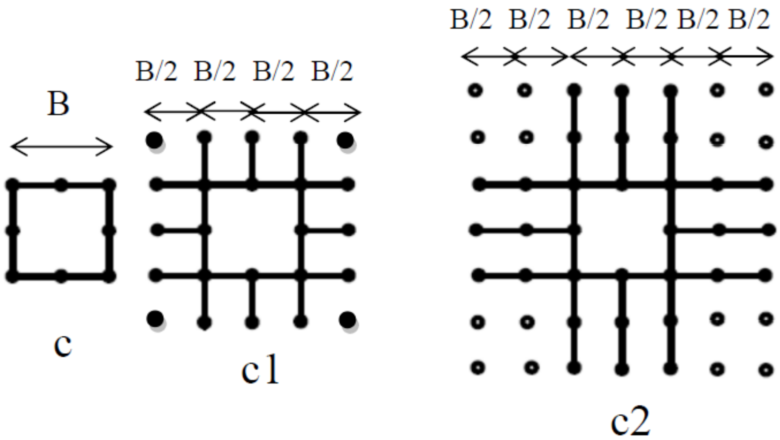

Fig. 10 Grouting around the footing to a depth of B

study the grouting effectiveness for reducing settlement in existing buildings. Three different spacings (B, B/2, B/3) were investigated to a depth equal to B, Fig. 9.
In the same way, the optimum spacing will be investigated by inserting holes around the footing at distances of 0.5 B and 1.0 B to a depth of B, Fig. 10.

where:
a untreated soil
b soil treated at spacing 10 cm, four holes to a depth of 10 cm.
c soil treated at spacing 5 cm, eight holes to a depth of 10 cm.
d soil treated at spacing 3.3 cm, twelve holes to a depth of 10 cm

It is also intended to study the effect of the depth of grouting. Therefore, models c, c1 and c1 were also grouted to a depth of 15 cm.

5. Results and discussion

To investigate the behaviour of soft clay improved by lime-silica fume grout, different model tests were performed on a shallow foundation resting on soft clay subjected to static vertical load. The investigation focuses on the influence of the spacing of grouting holes and depth of grouting. For all model tests, the failure is defined as the stress required to generate settlement corresponding to 10% of model footing width as proposed by Terzaghi (1943). The analysis of results of all model tests regarding the applied stress and the corresponding settlement is illustrated in terms of load-settlement curves. The model tests are classified into four categories.

5.1 Grouting underneath the footing area

In this category, the experiments were carried out to investigate the hole spacing inside the footing area which consists of rigid steel plate (100 × 100 × 10) mm as shown in Fig. 11 while Fig. 12 shows the load-settlement curves for footing loaded in category 1. It can be noticed that the optimum grid spacing which showed the maximum load according to the failure criterion of (10%

Fig. 11 Rigid steel footing resting on untreated soil

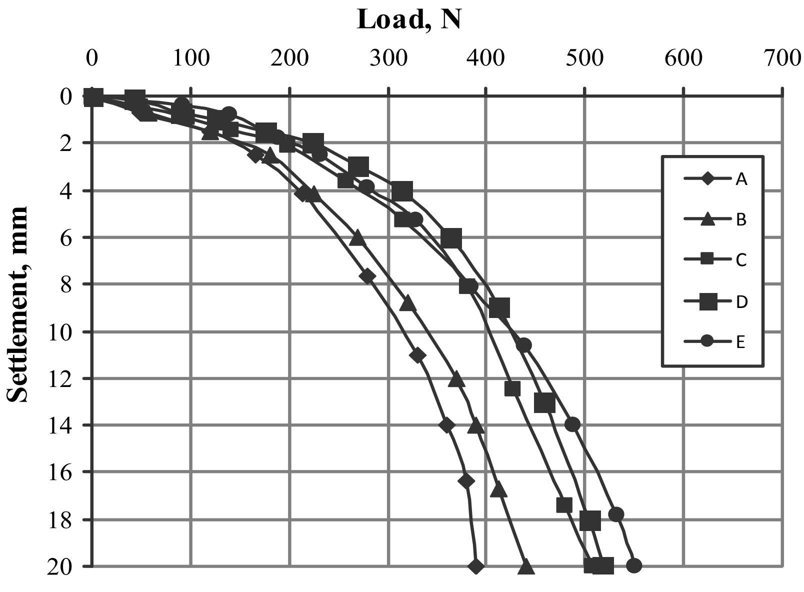

Fig. 12 Load-settlement relations for footings resting on soil grouted according to category 1

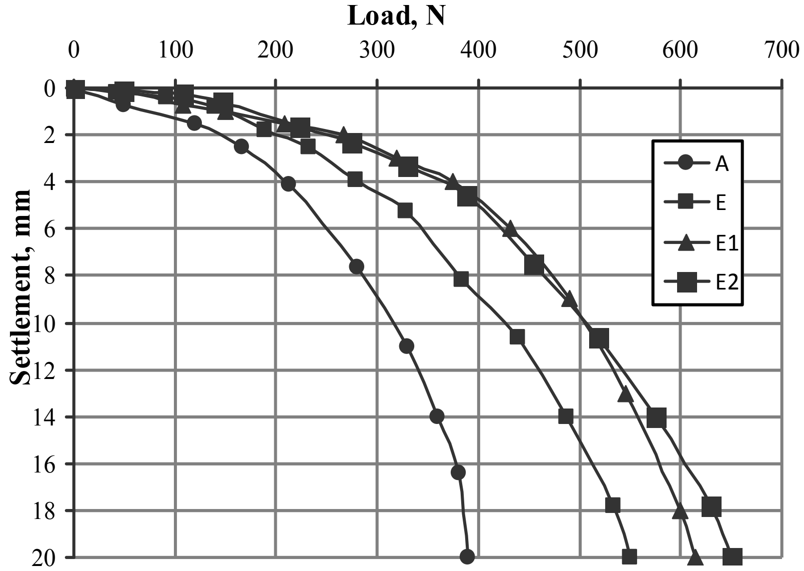

Fig. 13 Load-settlement relations for footings resting on soil grouted according to category 2

B) correspond to shapes D and E. Soil treated by grid D included 16 holes grout and soil treated by grid E included 13 holes grout. It can be decided that the shape which requires minimum hole grout (minimum L-SF grout) for economic reason is maintained by grid E.

5.2 Grouting underneath and around the footing

In this category, it is intended to study the effect of grouting underneath and around the footing to a distance of 5 cm and 10 cm for optimum grid shape E.

Fig. 13 shows the load-settlement curves for footings resting on soils stabilized within category 2. It can be seen that the grouting near the footing at a distance of 0.5 B is more effective than at 1.0 B due to the shape of shear failure of soft clay around the footing.

It is also intended to investigate the increase of grouting depth on category 2 from 1.0 B to 1.5 B. Fig. 14 shows the load-settlement curves for footing resting on soil grouted according to category 2, to a depth of 1.5 B.

It can be noticed that there is an increase in the bearing capacity of soft clay grouted to a depth of 1.5 B for all cases in Fig. 14, the bearing capacity increases because the soil is strengthened to a depth of 1.5 B where the stress induced by the applied loads is expected to spread through.

5.3 Grouting along the footing perimeter

This category includes experiments directed to investigate the effect of injection of grouts into holes located along the perimeter of the footing area. Fig. 15 shows the load-settlement curves for footing in this category. It can be noticed that the optimum grid spacing that induced the maximum load due to the failure criterion (10% B), is shape (**c**) which included 8 holes grout. However, the soil treated by grid (**d**) included more holes grout but exhibited lower bearing capacity. This may

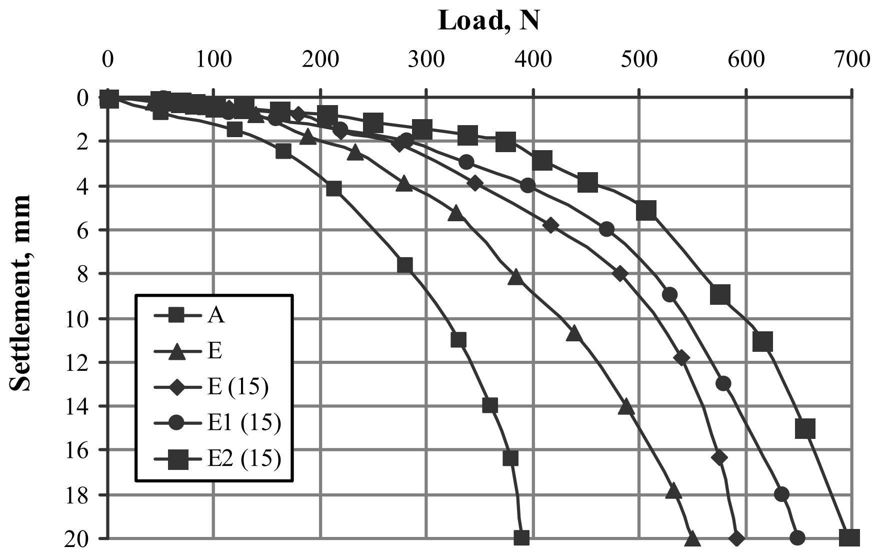

Fig. 14 Load-settlement relations of footings resting on soil grouted according category 2 to a depth of 15 cm

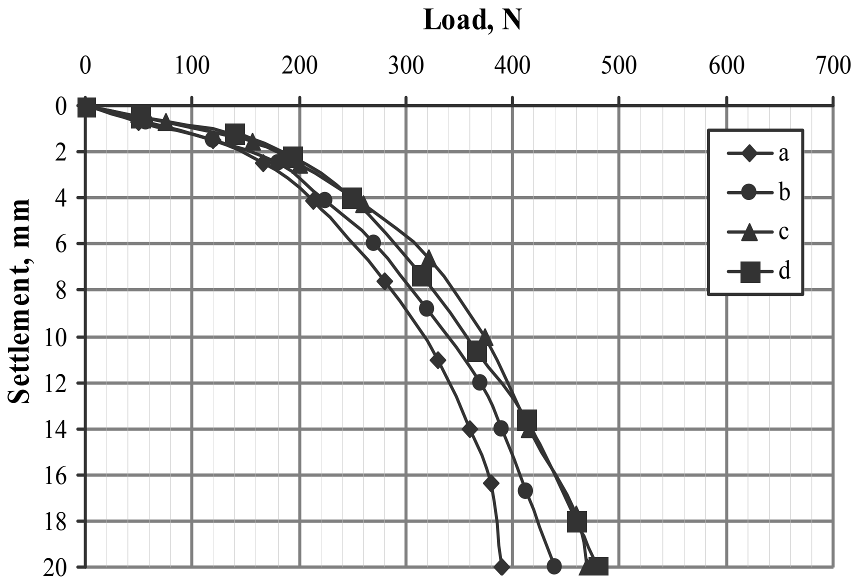

Fig. 15 Load-settlement relations for footings resting on soil grouted at the perimeter of footing area

be due to the small distance between pipes of injection in the critical zone (punching zone) which deceases the bearing capacity.

5.4 Grouting outside the footing area

This category simulates the condition of grouting the soil when the foundation is constructed and it is not possible to inject the grout below the footing area. It is intended to study the effect of grouting around the footing area to a distance extended to 0.5 B and 1.0 B. Fig. 16 shows the load-settlement curves for footings on soil grouted according to this category.

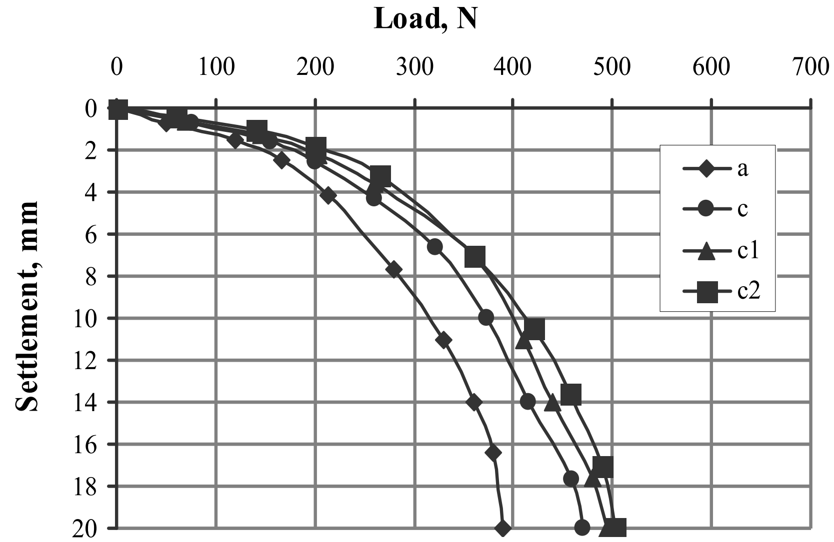

Fig. 16 Load-settlement relations for footings resting on soil grouted outside the footing area to a depth of B

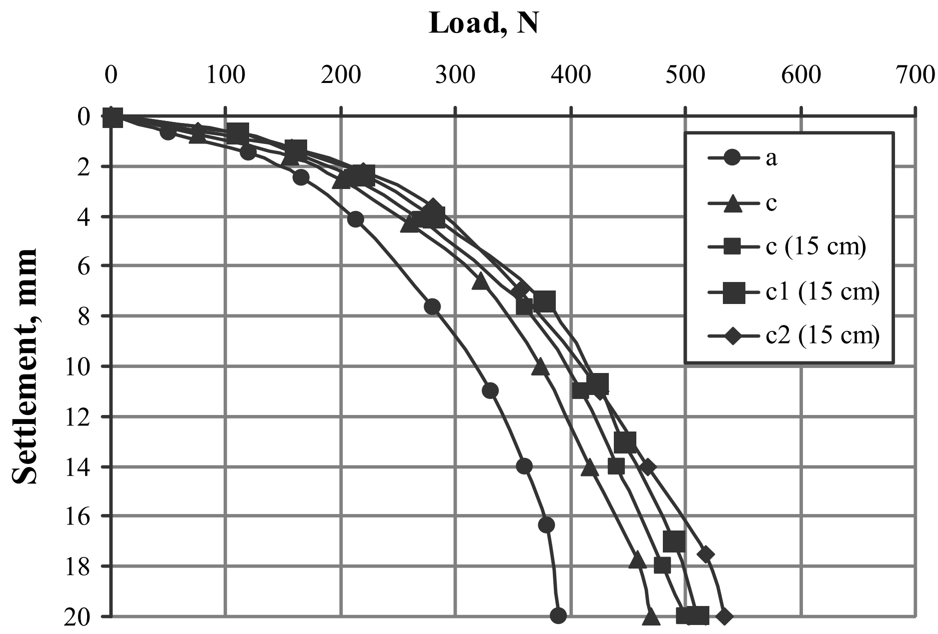

Fig. 17 Load-settlement relation of footing resting on soil grouted outside the footing to a depth 15 cm

Table 6 Bearing capacity of footings resting on grouted soil by one of the four categories

Cases	Depth of grouting, (cm)	Footing bearing capacity, q (kN/m^2)	% increase in bearing capacity
Untreated soil (A)	0	31.9	—
Case B	10	34	6.58
Case C	10	40.3	26.33
Case D	10	42.5	33.22
Case E	10	42.5	33.22
Case a	10	50.5	58.3
Case b	10	52	63.0
Case c	10	50	56.73
Case d	10	54.5	70.84
Case J	15	60	88.0
Case L	10	37.5	17.55
Case M	10	35.5	11.28
Case N	10	40	25.4
Case O	10	41.5	30.0
Case P	15	40	25.4
Case Q	15	41	28.52
Case R	15	42.5	33.22

It can be seen that the bearing capacity increases with increasing the number of grouting holes around the footing area due to increasing in L-SF grout. Fig. 17 shows the load-settlement relations for footings resting on soil grouted outside the footing area to a depth of 15 cm.

In the same way, the bearing capacity increases with increase of depth of grouting holes around the footing area due to increasing in L-SF grout. Table 6 summarizes the increase in bearing capacity for the four categories according to the following expression

$$q_{inc} = (q_{treated} - q_{untreated}) / q_{untreated} \qquad (1)$$

where $q_{treated}$ is the bearing capacity of footing on treated soil, and
$q_{untreated}$ is the bearing capacity of footing on untreated soil

It can be concluded that grid E2 within category 2 reveals the best improvement in footing bearing capacity. This grid involved grouting beneath the footing area and outside to a distance of 1.0 B. The depth of grouting is extended to 1.5 B.

6. Conclusions

The optimum percent of lime and silica fume as a stabilizer to soft clayey soil for compression and compressibility characteristics is between 4:5% (4% blime and 5% silica fume).

When the soft clay underneath or around a footing is injected by a slurry of lime-silica fume, an increase in the bearing capacity in the range of (6.58-88)% is obtained. The bearing capacity increases with increase of depth of grouting holes around the footing area due to increasing in L-SF grout. The grouting near the footing to a distance of 0.5 B is more effective than grouting at a distance of 1.0 B due to shape of shear failure of soft clay around the footing.

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