IOSR Journal of Mechanical and Civil Engineering (IOSR-JMCE)
e-ISSN: 2278-1684,p-ISSN: 2320-334X, Volume 12, Issue 2 Ver. I (Mar - Apr. 2015), PP 05-12
www.iosrjournals.org

The Effect of Replacement of Soil by More Competent Granular Fill

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Abstract: *In all Civil Engineering Structures, preparation of foundations for construction of structures assumes a lot of significance from the point of view of strength and stability. Due attention is required to be paid to this aspect. Ground Improvement techniques and Ground replacement are adopted for improving the strength characteristics. It is quite possible that a combination of these methods may also be used. Also rapid urban and industrial growth demands more land for further development. In order to meet this demand, land reclamation and utilization of unsuitable and environmentally affected lands by adopting one or more ground improvement techniques. The Boussinesq method uses elastic theory to calculate stress distribution in an elastic half space due to a point load. By integrating point loads over a specified area, the stress distribution in an elastic half space can be obtained under foundations and footings of different shapes. The main disadvantage of the Boussinesq method is that it assumes a homogeneous material. But, in general, the soil is stratified. For soil layers with high stiffness contrast, the Boussinesq solution can produce large errors in stress. The Westergaard equation (1938) are thought to provide better results for layered media, however the solution essentially smear out the effect of the different layers so that large errors will still occur for a finite number of layers with high stiffness contrast. The cardinal aim of the present work is to understand the effect of replacement of soil by more competent granular fill consisting of combination of 6mm metal and sand in different proportions. Accordingly, the investigation considers the granular material in different combinations of sand. The granular material is used in conjunction with four different types of soils in order to bring out relative effects on stress distribution.*
Keywords: *Elastic moduli, granular fill, ground improvement, layered soil, soil-structure interaction, stiffness*

I. Introduction

A civil engineer has to deal with soils in their diverse roles. Every civil engineering structure, whether it is a building, a bridge, a tower, an embankment, a road pavement, a railway line, a tunnel or a dam, has to be founded on the soil (assuming that a rock strata is not available) and thus shall transmit the dead and live loads to the soil stratum. Soil is, therefore the ultimate foundation material which supports the structure. The proper functioning of the structure will, therefore, depend critically on the success of the foundation element resting on the subsoil. Here the term foundation is used in the conventional sense, namely, a structure that distributes the load to the ultimate foundation, namely, the soil.

Soil is also the most abundantly available construction material. From ancient times, man has used soil for the construction of tombs, monuments, dwellings, and barrages for storing water. In modern times, the use of earth for building dams, for constructing pavements, for highways and airfields is an important aspect of civil engineering.

In the design and construction of underground structures such as tunnels, conduits, power houses, bracing for excavations and earth retaining structures, the role of soil is again very crucial. Since the soil is in direct contact with the structure, it acts as a medium of load transfer and hence for any analysis of forces acting on such structures, one has to consider the aspect of stress distribution through the soil. This, however, cannot be done by considering the behavior of the structure in isolation of the soil or by treating the soil independently of the structure. The structure, too, causes stresses and strains in the soil, while the stability of the structure itself is affected by soil behavior.

The classes of problems where the structure and soil mutually interact are known as soil – structure interaction problems. The existing soil at a construction site may not always be totally suitable for supporting structures such as building, bridges, highways, and dams. For example, in granular soil deposits, the in-situ soil may be very loose and indicate a large elastic settlement. In such a case, the soils need to be densified to increase its unit weight and thus the shear strength.

Sometimes the top layers of soil are undesirable and must be removed and replaced with better soil on which the structural foundation can be built. The soil used as fill should be well compacted to sustain the desired structural load. Compacted fills may also be required in low-lying areas to raise the ground elevations for foundation construction.

Soft saturated clay layers are often encountered at shallow depths below the foundations. Depending on the structural load and the depth of the clay layers, unusually large consolidation settlement may occur. Special soil-improvement techniques are required to minimize settlement.

1.1 Stresses In Layered Medium

The stresses inside a homogeneous elastic medium due to various loading conditions. In actual cases of soil deposits it is possible to encounter layered soils, each with a different modulus of elasticity. A case of practical importance is that of a stiff soil layer on top of a softer layer as shown in Fig-1 For a given loading condition, the effect of the top stiff layer will be to reduce the stress concentration in the lower layer. Burmister[1] (1943) worked on such problems involving two-and three-layer flexible systems. This was later developed by Fox[2] (1948), Burmister[3] (1958), Jones[4] (1962) and Peattie[[5] (1962).

A new method, called the 'method of images' is presented by S.Vijayakumar[6] et.al.(2007). This method allows for accurate calculation of stresses in layered materials where a large stiffness contrasts exist between layers. It is shown that the method of images is more accurate than the Boussinesq method and much simpler than the finite element method. A simplified and rational approach to evaluate stresses in two layered soil system has been developed by K.Nagendra Prasad[7] et.al.(2012).

Arvind Dewangan[8] et.al. have presented the stress distribution on the kaolinite layer at the kaolinite - geotextile or kaolinite furnace ash interface. The stress distribution is measured with increases in footing pressure in order to assess the load dispersion angle over the soil layer. The predicted load dispersion angle is then used to estimate the bearing capacity factors of the soil layer with increases in footing deformation. This paper also focuses on typical vertical stress distributions measured below the interface (on the top surface of the kaolin layer) for a fill thickness of 110mm with different footing pressures .

The effect of the reduction of stress concentration due to the presence of a stiff top layer is demonstrated in Fig-1. Consider a flexible circular area of radius '**b**' subjected to a loading of '**q**' per unit area at the surface of a two–layered system as shown in Fig-1, E_1 and E_2 are the moduli of elasticity of the top and the bottom layer, respectively, with $E_1 > E_2$: and '**h**' is the thickness of the top layer. For h=b, the elasticity solution for the vertical stress σ_z at various depths below the center of the loaded area can be obtained from Fig.1. The curves of $\boldsymbol{\sigma_z/q}$ against **z/b** for $\mathbf{E_1/E_2 = 1}$ is the simple Boussinesq case, which is obtained by using classical equation. However, for $\mathbf{E_1/E_2>1}$, the value of $\boldsymbol{\sigma_z/q}$ for a given **z/b** decreases with the increase of E_1/E_2. It must be pointed out that in obtaining these results it is assumed that there is no slippage at the interface.

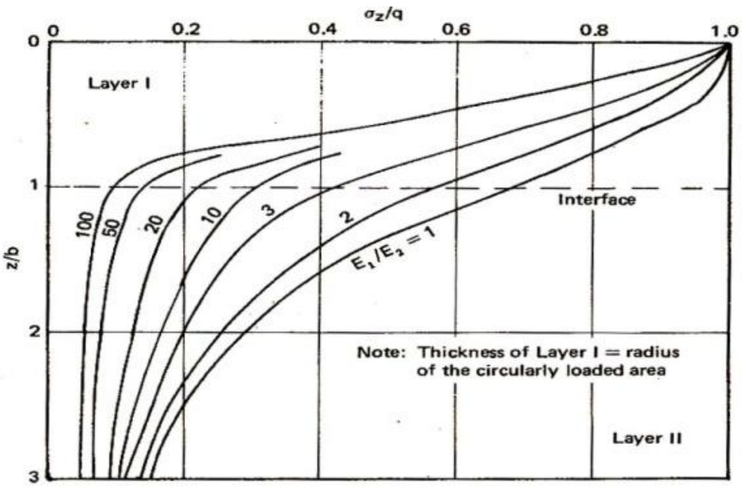

Figure -1: Vertical stress below the center line of a uniformly loaded (vertical) circular area in a two-layered system

II. Materials Tested

White metal / Black metal (6mm) and sand in the ratios of 1:1, 2:1, 3:1, 4:1 and 5:1 have been considered in the first series of investigation. The grain size distribution of the materials tested is shown table-I, the bulk unit weights of material are shown in table-II and the particle size distribution curves for various metals tested as shown in Fig.2, Fig.3 & Fig.4.

Another series of tests have been conducted on soil samples collected from four locations of Tirupati region. The locations from where samples have been obtained are designated as Cherlopalli, Ramachandrapuram, Vidyanagar and Perur.

Table-1: Grain Size Distribution Of The Materials Tested

Sieve Size, mm	Percent finer, N		
	White metal	Black metal	Sand
9.5	100	100	100
6.3	97.17	95.31	100
4.75	43.88	55.66	100
2.36	3.117	19.27	99.71
0.6	0.917	0.31	40.32
0.075	0	0	0

Table-2: Bulk Unit Weights Of The Materials Tested

Description	**Value**		
	White metal	**Black metal**	**Sand**
Unit Weight, g/cm^3	1.380	1.538	1.325

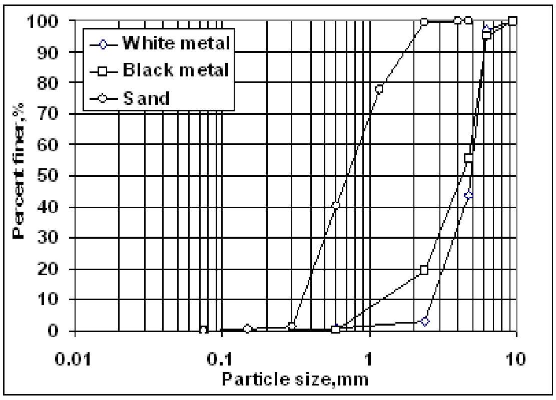

Figure-2 : Particle size distribution curves for the materials tested

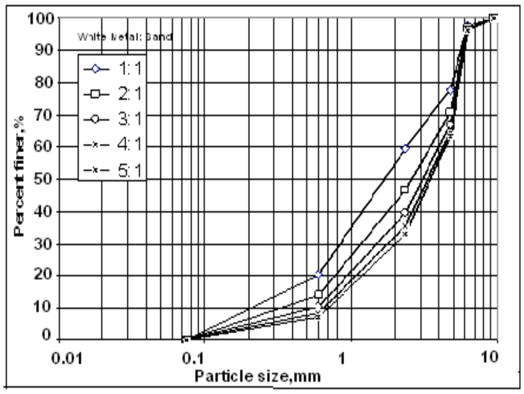

Figure-3: Particle size gradations for different proportions of white metal and sand

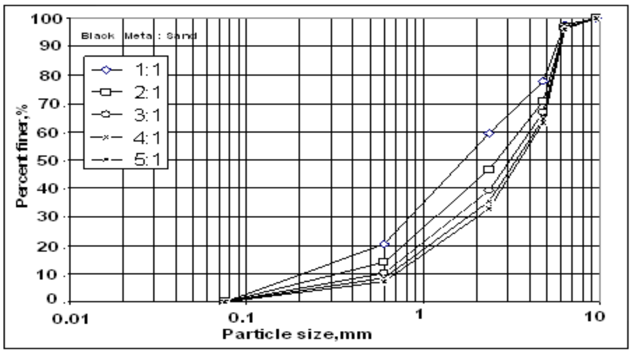

Figure-4: Particle size gradations for different proportions of black metal and sand

2.1 Gradations Considered

The gradations considered for the present investigation are 1:1, 2:1, 3:1, 4:1 & 5:1 of white metal or black metal and sand. Two metals have been considered to find the relative effects on strength of the mixtures. The gradation characteristics of the mixtures as represented by coefficient of curvature are shown in table-3. It may be seen from the table-3 that the range of coefficient of curvature conforms to well grade classification.

Table-3: Gradation Characteristics For Different Proportions

Description	**Coefficient of curvature**	
	White metal : Sand	**Black metal : Sand**
1:1	2.25	1.30
2:1	1.17	1.11
3:1	2.82	1.07
4:1	2.24	1.61
5:1	1.31	1.40

2.2 Direct Shear Test

The specimen was prepared as per specification. The required normal load was applied. The shear strain was applied at a constant rate of 0.2 mm/ mm on the upper half of the box till the failure of the specimen. The final shear was recorded through the calibrated proving ring. At the end of the test, the specimen was removed from the box and the water content at the shear zone was determined. The process was repeated by varying the normal load and 4 sets of readings were taken.

2.2.1 Analysis of Direct Shear Test Results

The direct shear test results of 6mm metal and sand mixtures are shown in Fig-5 to Fig-8. It may be seen that as the percentage 6mm metal increase the angle of internal friction increases for both the metals. However, the increase in angle of internal friction at lower percentages is quite rapid compared to higher percentages.

It means that if the sand content is just sufficient to fill all the void spaces the angle of internal friction increases with faster rate up to this optimum sand content and later on the increase is not very significant.

Therefore in order to form a stable bed material a proportion of 6 mm metal and sand in the ratio 2:1 and 3:1 would suffice. This proportion apart from producing higher friction angle also would provide greater stability. An increase in angle of internal friction from $42^o – 50^o$ is quite effective.

This magnitude of angle of internal friction provided greater stiffness to the bed material and would help in reducing the stress concentration in the lower layer. Accordingly we may have to calculate the thickness of the bed material required to arrive at reduction in the stress concentration to limit the settlements to the desired level.

The more stable black metal used also yields the same result but with increase in angle of internal friction marginally. However other parameters such as grain shape and crushing strength are to be considered to enable engineering decisions with greater confidence.

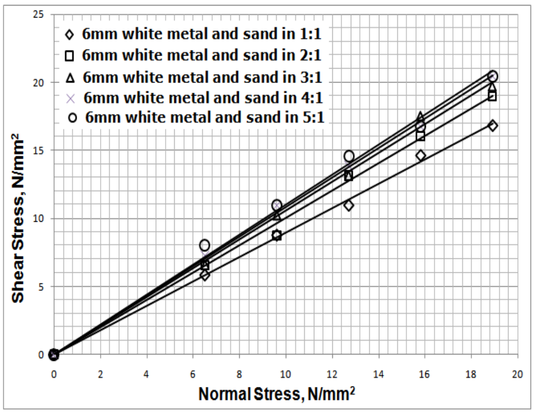

Figure-5: Direct shear test results on white metal and sand mixture

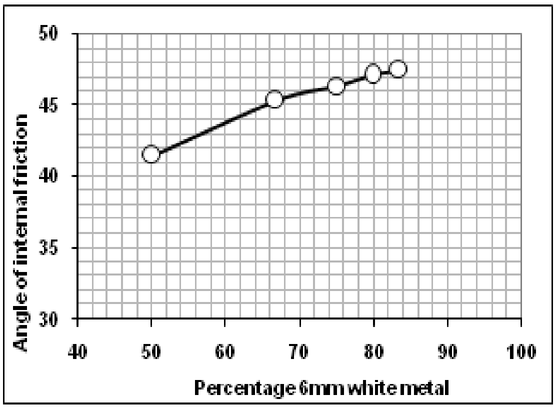

Figure-6: Variation of angle of internal friction with percentage white metal

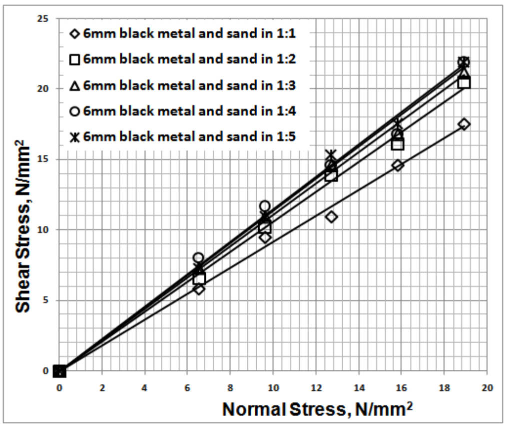

Figure-7 : Direct shear test results on black metal and sand mixture

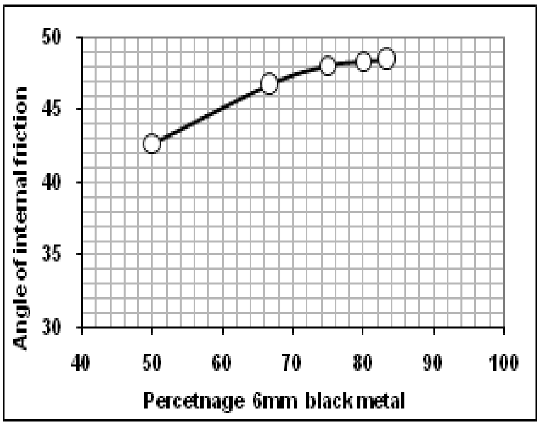

Figure-8 : Variation of angle of internal friction with percentage black metal

III. Soils Tested

The following tests have been conducted on soil samples obtained from the field. All tests have been conducted as per the relevant provisions stipulated in respective IS code procedures.

Unconfined compression test
One dimensional compression tests

The properties of soil samples tested are presented in Table-IV. It may be seen that the soils are Clayey Sand (SC) to Clay with Intermediate Compressibility (CI). The values of plasticity index from 10 to 30. However, the Young's modulus values and unconfined compression strength values differ indicating possible changes in the states of soil and grain shapes. These soils are of typical nature founded in this region. The following tests have been conducted.

3.1 Unconfined Compression Test

Another series of tests have been conducted to evaluate the modulus values of soil tested in unconfined compression test and the test results are shown in Fig-9 and Fig-10. The modulus value for Cherlopalli soil is of the order of 26.00 kg/cm^2, Ramachandra Puram soil is 64.00 kg/cm^2, Vidyanagar soil is 78.00 kg/cm^2 and Perur soil is 90.00 kg/cm^2. The respective unconfined compressive strength values are found to be 1.33 kg/cm^2, 2.68 kg/cm^2, 3.17 kg/cm^2 and 3.19 kg/cm^2. It may be seen from Fig-10 that Young's modulus values are found to be proportional to the unconfined compression strength(**q_s**) values.

Table-4: Soil Properties

S.No.	Description	Value			
		Cherlopalli	Ramachandra Puram	Vidya Nagar	Peruru
1.	% Gravel	0.80	1.90	9.20	2.80
2.	%Sand	41.20	78.60	68.00	68.70
3.	% Silt +Clay	58.00	19.50	22.80	28.50
4.	Liquid limit (%)	46.00	24.00	25.00	27.00
5.	Plastic limit (%)	17.00	14.00	14.00	15.00

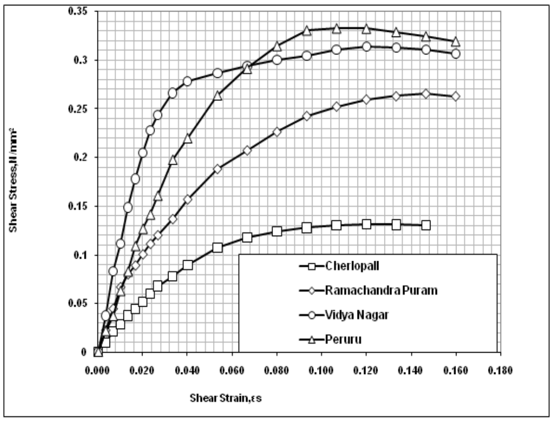

Figure-9: Stress strain response of all soils

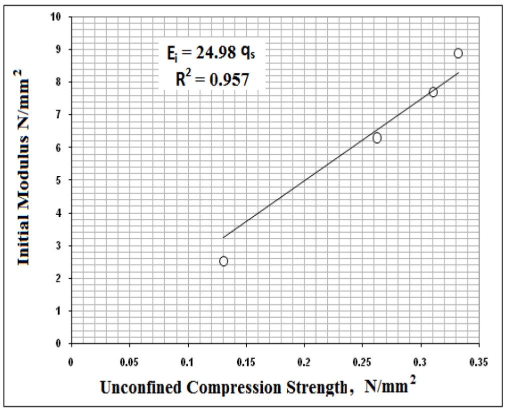

Figure-10 : Relationship between Young's modulus and unconfined compression strength of soils

3.2 One Dimensional Compression Tests:

3.2.1 Sample Preparation:

In order to determine the effect of replacement of soil by granular material, an attempt has been made to conduct compression tests with CBR mould effecting one dimensional compression simulating field conditions. The replacement of soil by granular material has been done systematically starting from partial replacement to total replacement of soil by granular material.

3.2.2 Analysis of Test Results:

The test results are shown in Fig-11 to Fig-19. It may be seen that the initial coefficient of sub grade reaction (**k**) from representing the stiffness value varies from 14.68 kg/cm^3 to 86.914 kg/cm^3. This indicates that the soil whose modulus is 14.68 kg/cm^3 and the metal whose stiffness is of the order of 86.90 kg/cm^3 which is

6 times stiffer than the soil material for Cherlopalli soil. The stiffness ratios for other soils tested are indicated in Table-V. It may be seen that the stiffness ratios for the materials tested range from 3-8. This turns out that the dissipation of stress in the upper layer for a stiffness ratio of 8 would be of the order of 60% (referring to Fig-1) which is quite considerable and depending on the reduction in settlement targeted, extent of replacement of the weaker material with competent material could be arrived. However greater stiffness ratios can be achieved with coarser granular materials.

These results indicate that the modulus value changes in relation to extent of replacement and are expected to dissipate stresses in the top layer quite significantly. Based on these test results, the stiffness ratios can be estimated and depending on the extent of minimizing the settlements of the soils. The thickness of stiff layer can be evaluated.

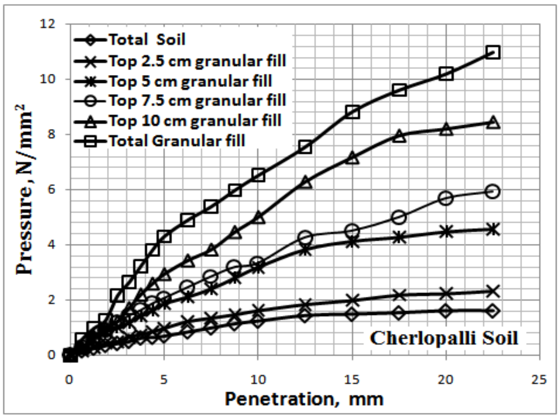

Figure-11: Response in one-dimensional compression

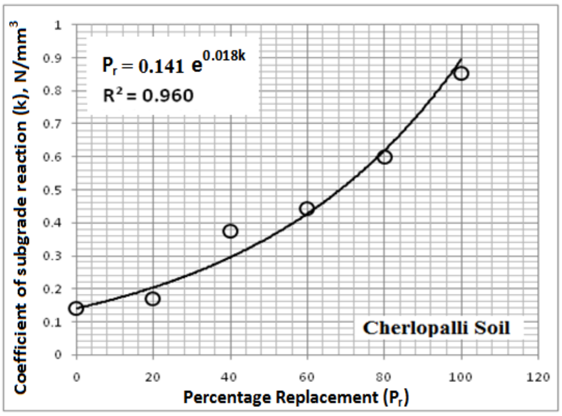

Figure-12: Effect of replacement on coefficient of subgrade reaction

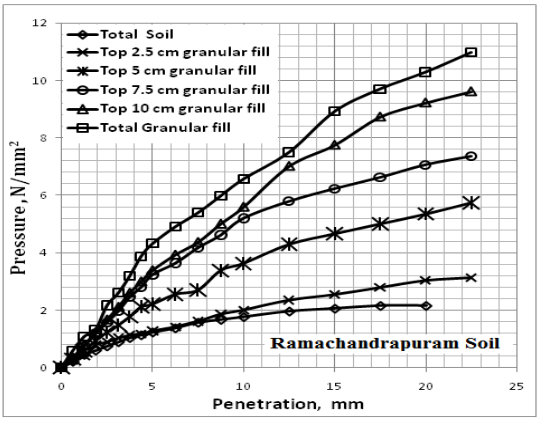

Figure-13: Response in one-dimensional compression

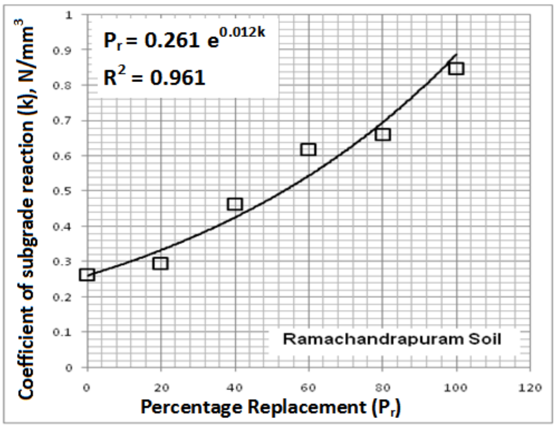

Figure-14: Effect of replacement on coefficient of subgrade reaction

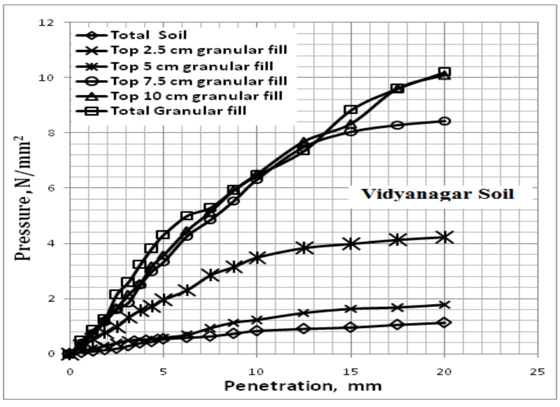

Figure-15: Response in one-dimensional compression

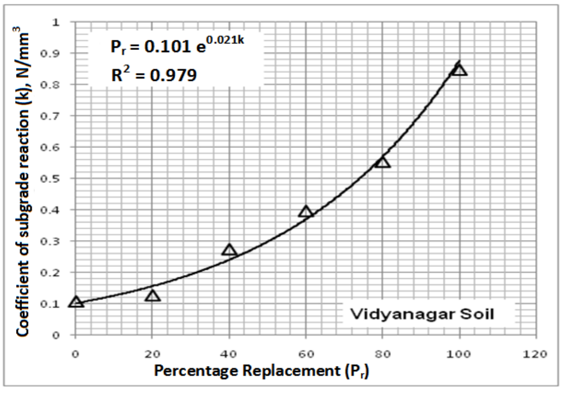

Figure-16 : Effect of replacement on coefficient of subgrade reaction

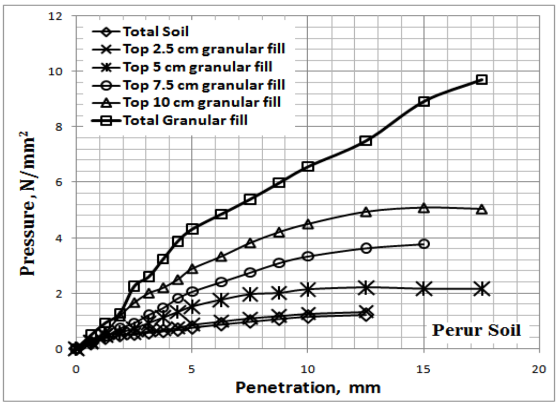

Figure-17: Response in one-dimensional compression

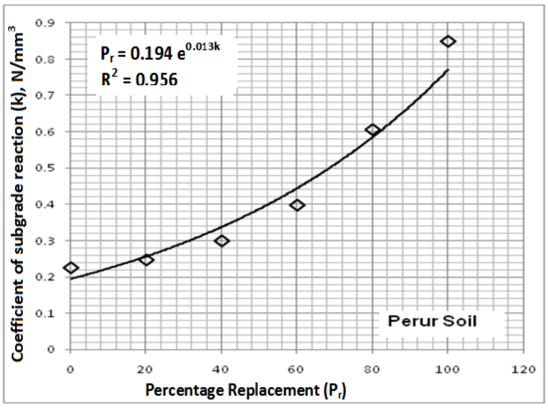

Figure-18: Effect of replacement on coefficient of Sub-grade reaction

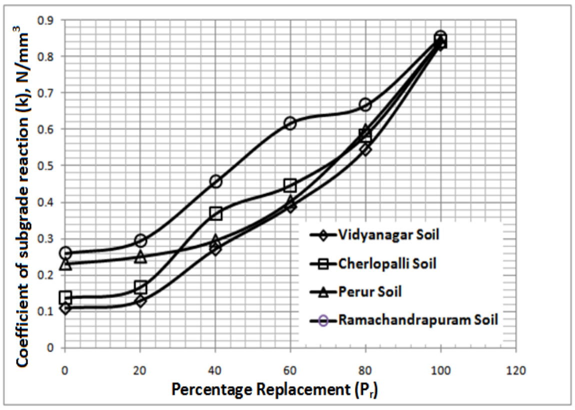

Figure-19: Effect of percentage replacement of granular material on coefficient of sub-grade reaction.

Table-5: Effect Of Percentage Replacement Of Granular Material On Stiffness Properties

Percentage of replacement (P_r)	Coefficient of Sub grade reaction (k) kg/cm^3			
	Cherlopalli	Ramachandra Puram	Vidya Nagar	Perur
0	14.68	27	11	23
20	17.08	30	13	25
40	37.57	47	28	30
60	45.34	63	40	41
80	59.70	68	56	61
100	86.91	86	86	86
Stiffness ratio Soil versus Granular material	6	3	8	4

Concluding Remarks

Based on limited experimental investigation taken up to study the strength characteristics of granular bed material the following concluding remarks may be made

- Angle of internal friction increases with the increase in the percentage 6mm metal for both the metals considered.
- The increase in angle of internal friction at lower percentages is quite rapid compared to higher percentages.
- The sand content required to fill the void space in the metal seems to govern the strength of the mixture
- A stable bed material with 6mm metal and sand may be formed in the ratio 2:1 and 3:1. This proportion apart from producing higher friction angle also would provide greater stability.

- An increase in angle of internal friction from 42 – 50 degrees with the use of 6mm metal is quite effective.
- This magnitude of angle of internal friction provided greater stiffness to the bed material and would help in reducing the stress concentration in the lower layer. Accordingly we may have to calculate the thickness of the bed material required to arrive at reduction in the stress concentration to limit the settlements to the desired level.
- The more stable black metal used also yields the same result but with increase in angle of internal friction marginally. However other parameters such as grain shape and strength are to be considered for commenting on the effect of stable metal on angle of internal friction with greater confidence.
- Young's modulus values are found to be proportional to the unconfined compression strength values for the soils tested
- The test results of 1-D Compression indicate that the modulus value changes in relation to extent of replacement in an exponential form and are expected to dissipate stresses in the top layer quite significantly.
- The stiffness ratios can be determined using 1-D Compression tests.
- Depending on the extent of minimizing the settlements of the soils. The thickness of stiff layer can be evaluated.
- Use of coarser granular material may be considered for producing greater stiffness.
- It may be seen that E1/E2 ratios have pronounced effect in dissipating the stresses.
- If the replacement is effected up to 0.5 times the width of the footing the stress dissipation with increase in stress ratios is quite rapid.
- When the replacement is effected to the extent of width and 1.5 times the width of the footing the dissipation is quite rapid. For example, it is shown that for granular fill of 0.5B intensity the dissipation of stresses is relatively lower as compared to granular fill of 1.0B thickness for which case the dissipation at shallow depths is much faster and more effective of the order of 75% greater.

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