A COMPARISON BETWEEN PREDICTED AND ACTUAL PERFORMANCES OF AN EMBANKMENT BUILT OVER A STONE COLUMN AREA

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By

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LIST OF ABBRVIATIONS

CPT	Cone Penetration Test
MP	Mackintosh Probe
DPT	Dynamic Penetration Test
ROW	KTMB Right of Way
RSG	Rod Settlement Gauge
SM	Settlement Marker or Surface Markers
SC	Stone Column

LIST OF SYMOLS

A	Grid area
A_c	Stone Column Area
C	Cohesion
C_h	Coefficient of horizontal consolidation
C_v	Coefficient of vertical consolidation
d	Depth of subsoil layer from ground
D_c	Constrained modulus of stone column material
D_s	Constrained modulus of subsoil
f_d	Depth factor
H	Thickness of subsoil
K_{aC}	Coefficient of active earth pressure of column material
m_v	Coefficient of volume change
n	Settlement improvement ratio
N_k	Cone factor
P_c	Pressure within stone column along the depth
P_s	Pressure within soil in tributary area
Q_c	Cone friction
R	Settlement reduction factor
S_u	Undrained shear strength
U_r	Degree of consolidation (radial only)
U_{rv}	Degree of consolidation (both radial and vertical)
U_v	Degree of consolidation (vertical only)
α	Coefficient of constrained modulus

δ_{ig}	Settlement of improved ground
δ_{og}	Settlement of unimproved ground
Φ_c	Friction angle of stone column material
Φ_s	Friction angle of subsoil
γ_S	Bulk density of subsoil
μ_s	Poisson's ratio of stone column material
σ_{vo}	Insitu overburden stress
q_u	Ultimate bearing capacity kpa
C	Soil cohesion for undrained shear strength Kpa
B	Foundation width in meters
γ_H	Effective unit weight beneath foundation base within failure zone KN/m^3
N_c,	Dimensionless bearing capacity factors for cohesion C.
N_γ ,	Dimensionless bearing capacity factors for soil weight in the failure wedge
N_q	Dimensionless bearing capacity factors for surcharge q,
δ_c,	Dimensionless correction factors for cohesion C
δ_γ	Dimensionless correction factors for soil weight in the failure wedge
δ_q	Dimensionless correction factors for surcharge q
τ_s	Shear soil strength resistance, kpa
σ_n	Normal stress on slip path, kpa
L_{sh}	Horizontal length of general shear failure
Fs	Factor of safety with respect of shear strength.

L	Length of the slip path
F	Forces acting to drive the slip
W_i	Weight of the slice.
ΔL	Width of the slice

PERBANDINGAN DIANTARA PRESTASI ANGGARAN DAN SEBENAR BAGI BENTENG YANG DIBINA DI ATAS KAWASAN BERTIANG KELIKIR

ABSTRAK

Didalam pengalaman menggunakan tiang kelikir, enapan, tempoh pengukuhan, dan lain-lain prestasi sebenar selalunya tidak sama dengan yang dianggar. Dalam kajian ini, pengukuran, terutamanya keatas enapan dan tempoh pengukuhan sebenar telah dilakukan bagi satu tanbakan yang dibina diatas suatu kawasan tiang kelikir. Lokasi kajian di Kodiang, Kedah, iaitu sebahagian daripada projek keretapi elektrik berkembar dari Ipoh ke Padang Besar, terdiri daripada Zon 1 dan Zon 2. Tapaknya terdiri daripada tanah lempung marin kawasan utara semenanjung Malaysia. Tempoh pengukuhan yang biasa diperuntukkan bagi projek keretapi berkembar adalah 45 hari daripada masa peletakkan beban terakhir benteng. Peningkatan mendadak keupayaan galas bagi tanah yang dirawat dan kestabilan benteng kerana pembaikan tanah juga dinilai. Tindakbalas lain yang diambil maklum adalah oerubahan tekanan air liang dan pergerakan sisi tanah terbeban. Penyiasatan tanah dan pengalatan yang meluas telah dijalankan dalam kajian ini terutama bagi membandingkan diantara anggaran dan prestasi sebenar benteng. Didapati bahawa enapan lapangan sebenar secara amnya kurang daripada yang dianggar. Enapan bagi Zon 1 hanya 72 % daripada yang dianggar sementara bagi Zon 2 hanya 58 % daripada yang dianggar. Anggaran masa bagi pengukuhan di Zon 1 hanya 68 % daripada sebenar sementara bagi Zon 2 hanya 84 % daripada

sebenar. Keupayaan galas bagi kawasan yang dirawat dengan tiang kelikir 4 kali lebih besar daripada kawasan tidak dirawat. Faktor keselamatan daripada kegagalan cerun benteng secara menggelongsor telah meningkat 1.4 kali daripada nilai sebelum dirawat.

A COMPARISON BETWEEN PREDICTED AND ACTUAL PERFORMANCES OF AN EMBANKMENT BUILT OVER STONE COLUMN AREA

ABSTRACT

In stone column experiences, actual settlements, consolidation times, and other performance indicators often disagree with the predicted ones. In this study, measurements particularly on actual settlements and consolidation times were made on an embankment built over stone columns. The area of study, situated at Kodiang, Kedah, as a part of the electrified double track project between Ipoh to Padang Besar, consisted of Zone 1 and Zone 2. The embankment over Zone 1 was 4 m thick, while that of Zone 2 was 2 m thick. The underlying soil material was marine clay of northern region of Peninsular Malaysia. The consolidation time allowed for embankments in the project was 45 days after achieving top surcharge levels. The rapid gain in bearing capacity of the treated soil and embankment stability achieved due to the improved soil were also assessed. Other behaviors noted were change in pore pressures and lateral displacements of soil under loads. Extensive soil investigation and instrumentation have been carried out in the study particularly to compare between actual and predicted behavior of the embankments. It was found that actual field settlements were generally lesser than predicted. The measured field settlement at Zone 1 was only equivalent to 72% of the predicted, while at Zone 2 the measured was only 58% of the predicted. Also, it was found that the predicted time required for

consolidation at Zone 1 was equivalent to 68% of the actual time, while at Zone 2, the predicted time was 84% of the actual. The ultimate bearing capacity of site treated with stone column was 4 times greater than the untreated site. The factor of safety against sliding failure of slope treated with stone column has increased by 1.4 times of the factor of safety of untreated slope.

CHAPTER 1

INTRODUCTION

1.0 Introduction

The stone column method of ground treatment involves partial replacement of soft soil with a vertical column made of compacted gravel, penetrating the soft stratum. Soft strata are common features in construction sites – they exist in the forms of soft clay, silty clay and loose silty sand. Thus for the column, the soft soil is replaced by the compacted stones. That replacement may amount to between 15 % and 35 % of the soft soil volume under a treatment area. This variation depends on design parameters such as column diameter and spacing (Bachus and Barksdale, 1983).

Stone column installation could be carried out in two different methods – the bottom feed method and the top feed method - which are distinguished from each other by the way a hole is formed and stone is fed into the hole.

The bottom feeding method, also called the dry method, is carried out by first inserting the probe to the desire depth via pushing the vibro-probe or poker. Vibration and compressed air mechanisms come together with the vibro-probe. This dry method is more suitable for soft layer of soil – of undrained shear strength about 6 to 7 kPa - with high water table. Once brought to the desired depth, the vibro-probe is gradually drawn out, and during the process, stone is filled almost continuously into the hole via a chute or pipe attached to the vibro-probe. A stone container or hopper attached to the mast holding the vibro-probe continually feed the gravel into the chute and down into the created void at the bottom of the column while the hopper itself is continually re-filled with stone using a loader. The stone in this case should be relatively finer in size such as about 40 mm in diameter. The compaction of each batch of stone is achieved through repeated withdrawal and insertion – pulling and pushing – of the vibro-probe. The compacted gravel interlocks with the surrounding soil forming a strong column – the stone column, (Bryan et al., 2007).

The wet or top feeding method is also carried out by first inserting the probe to the desire depth via pushing the vibro-probe. Instead of using compressed air to create the hollow, compressed water is used to flush out the soft soil during vibro-probe descent. This wet method is more suitable for relatively harder layer of soil. Once brought to the desired depth, the vibro-probe is drawn, out creating a column of void. Stone is fed into it in batches and the compaction of each batch is again achieved with the pushing actions of the vibrating probe. The stone in this case can be coarser in size such as between 40 mm and 70 mm in diameter. This method is favorable for a relatively harder soil in order to avoid the collapse of unsupported hole after retrieving the vibro-probe and before filling the cavity with stone. In finished form, again, a stone column will have compacted stone interlocks with the surrounding soil forming a strong column – the stone column (Raju et. al., 1997).

The success of stone column concept was first demonstrated in France in 1830 in order to treat the soft soils. Then in Europe in late 1950's and in USA in 1972 the method was re-adapted to the construction industry. In Malaysia, the method was utilized effectively in early 1980's. This type of ground treatment has continued to spread successfully until today and become the most commonly used technique due to the rapid gaining of bearing capacity and the crucial role of accelerating the consolidation settlement. The decreased settlement time makes the stone column method more advantageous over the prefabricated vertical drain which requires relatively longer time for the same settlement amount to take place and only minor gain in bearing capacity. Stone column ground treatment method is also considered more favorable over piled embankment ground treatment method due to mainly economical factor (Bachus and Barksdale, 1983).

The stone column is especially unique in the sense that it creates a composite ground that has higher shear strength than the native soil. As such, more stress will be concentrated at each stone column. The design load on a stone column could vary between 20 to 50 tons (Bachus and Barksdale, 1983).

1.1 Problem statement

Malaysia has significant areas of marine clay that are undergoing development especially in the northern region. Stone column is one of the most common ground treatment methods applied for the areas, and yet not a single stone column trial embankment has been studied, describing and analyzing the comprehensive behavior of a group of stone columns. There were instead studies pointing out only to single or detached aspect of stone column behavior.

In one instance, the measured settlements of a 10 m high bridge embankment in Shah Alam – for the Shah Alam expressway - which was founded at a similar mining pond area treated with similar stone columns were found to be 100 mm at a particular area and 250 mm at another. These settlements not only differed too much but also much lesser than predicted (Raju et. al., 1997). Consequently, the settlement times were 2.5 months for the first area and 6 months for the second. The particular difference between the predicted and measured settlements has been puzzling but not been discussed or analyzed. For the double track project between Rawang and Ipoh, field settlements were found 50% lesser than predicted for some embankments founded on stone columns (Raman 2006). Apparently, actual field settlements being lesser than predicted are a common occurrence which actually could be a good indication of the success of stone column procedure, but these haven't been completely addressed.

For the electrified double track project between Ipoh to Padang Besar, there are a lot of stretches treated with stone column between Alor Star to Arau alone, in order to treat the problem of soft ground. For these sites, the normal periods allowed for full settlement were not more than 45 days after achieving top surcharge level of the embankments. An experimental embankment was constructed in Kodiang, Kedah in order to study the overall behavior of marine clay of the area when treated with stone column procedure. This so called trial embankment or experimental embankment was divided into two zones – Zone 1 and Zone 2 – with each having different embankment height while the rest of parameters remained the same. For each zone, the following aspects were studied:

i. Predicted and actual monitored total settlements.
ii. Lateral displacements.
iii. Time required achieving degree of consolidation.
iv. Bearing capacities of composite and original grounds.
v. Slope stabilities of composite and original grounds.

The main question asked for this experimental embankment project was if the actual behavior matches with predictions. The previous experience indicates that the actual performance has always been better.

1.2 Objectives of the Study

The main objectives of the study are as follows:

i) To compare the actual performance of an embankment over stone column foundation, in terms of settlements and times associated with them, with predictions.

ii) To evaluate bearing pressure caused by an embankment over stone column foundation in working condition and the extent to which the foundation can be further loaded.

1.3 Scopes of the Study

The scope of this study is limited to the followings:

i. The study used the method by Priebe (1995) in the analyses of the stone columns.

ii. The stone column installation has been carried out by the bottom feeding, or otherwise known as the dry method.

iii. The instrumentation installed at the trial embankment has involved 44 items, as shown in Table 1.1, from which data was collected.

Table 1.1: Summary of instrumentations

Instrument	**Items**
Inclinometer	3
Horizontal Profile-meter	2
Total Pressure Cell	2
Deep Settlement Gauge	2
Rod Settlement Gauge	10
Surface Settlement Marker	8
Ground Heave Marker	4
Piezo-meter	3
Total	44

iv. Soil investigation and samplings were carried at the trial embankment for 15 occasions, as listed in Table 1.2

Table 1.2: Summary of soil investigation and samplings

Testing	**Items**
Cone Penetration Test (CPT)	2
Vane Shear Test (VS)	2
Undisturbed Sampling (UDS)	5
Mackintosh Probes (MP)	2
Dynamic Penetration Test (DPT)	3
Bore Hole (BH)	1
Total	15

CHAPTER 2

LITERATURE REVIEW

2.0 Introduction

Vibro replacement is one of the deep vibratory compaction technique applied at soft soils in order to decrease settlement significantly as well as to decrease the time required for achieving consolidation.

Stone column ground treatment also improves treated ground parameters of cohesion C and internal angle of friction ϕ which allows immediate gain on bearing capacity of treated ground. Improved ground has a combination of constrained modulus of the inserted stone column *Ec* and the original soft soil constrained modulus *Es* which indicates relative stiffness of materials used for ground treatment (Malarvizhi and Ilamparuthi, 2011).

2.1 Stone Column Design Methods

Presently available methods for calculating settlement can be classified based on the simple approach and assumption made and also sophisticated methods based on fundamental elasticity boundary conditions. All these approaches for estimating settlement assume an infinitely wide loaded area reinforced with stone column having a constant diameter and spacing. The methods of predicting the settlement are based on the unit cell concept for the loading and geometry (Priebe, 1995; Aboshi and Suematsu, 1985). The methods of designing stone column are as given next.

2.1.1 Equilibrium Method

This method has been established by Aboshi and Barksdale (1979) which offers a simple realistic engineering approach for estimating the reduction in settlement of ground improved with stone columns. The following assumptions are necessary in developing the equilibrium method.

- The extended unit cell idealization is valid
- The total vertical load applied to the unit cell equals the sum of the force carried by the stone and the soil (i.e. equilibrium is maintained within the unit cell)
- The vertical displacement of stone column and soil is equal
- A uniform vertical stress due to external loading exists throughout the length of the stone column

The consolidation settlement then will be calculated as shown at Equation 2.1:

$$S_t = \left(C_c / (1+ e_0) \right) \times \log_{10} \left((\sigma'_O + \sigma_C) / \sigma'_O \right) \times H \qquad (2.1)$$

Where,

S_t = primary consolidation settlement occurring over a distance H of stone column treated embankment

H = vertical height of stone column treated ground which settlement are being calculated.

σ'_O = average initial effective stress in the clay layer

σ_C = change in stress at clay layer due to the externally applied load

C_c = compression index from one dimensional consolidation test

e_0 = initial void ratio

2.1.2 Greenwood Method

Greenwood (1970) has presented preliminary empirical curves giving the settlement reduction due to ground treatment with stone column as a function of undrained soil strength and stone column spacing. Greenwood has incorporated the area replacement ratio into his settlement curves with the use of improvement factor.

Greenwood in the opinion that the stress concentration η decreases as the stiffness of the ground being improved increases through time due to consolidation process in proportion to the stiffness of the stone column. Therefore, the stress concentration factors greater than 15 required to develop the large level of improvement are unlikely in the firm soils.

2.1.3 Incremental Method

This method for estimating the settlement founded by Goughnour and Bayuk.(1979) With the use of unit cell idealization, the stone is assumed to be a non-compressible so that all volume change occurs in the clay, both radial & vertical consolidation occurs at the clay. The unit cell divided into small, horizontal increments. The vertical strain and vertical and redial stresses are calculated for each increment assuming all variables are constant over the increment.

When the stress levels are sufficiently low especially at early stage of loading, the stone column remains in the elastic range. But for most cases of design, the stone column bulges laterally yielding plastically over at least a portion of its length estimated 2 to 3 times the stone column diameter

The assumption is also made that vertical, radial and tangential stresses at the interface between the stone column and soil are principal stresses. Therefore no shear stresses are assumed to act on the vertical boundary between the stone column and soil.

In the elastic range the vertical strain is taken as the increment of vertical stress divided by the modulus of elasticity.

When failure took place at stone column under certain load; the usual assumption is made that the vertical stress in the stone equals the radial stress in the clay at the interface times the coefficient of passive pressure of the stone. Radial stress in the cohesive soil is calculated following the plastic theory developed by Kirkpatrick, Whitman, et al. and Wu et al. (1983).

The plastic theory gives the change in radial stress in the clay as a function of the change in vertical stress in the clay, the coefficient of lateral stress in the clay applicable for the stress increment.

Radial consolidation of the clay is considered using modification of the Terzaghi one-dimensional theory, but the vertical stress increased to reflect greater volume changes due to radial consolidation.

2.1.4 Finite Element Method

Balaam and Poulos (1983) have studied the stone column behavior using finite element method by a large group of stone columns using the unit cell concept. The modulus ratio of the stone to the clay was assumed to vary from 10 to 40, and the Poisson's ratio of each material was assumed to be 0.3.

A coefficient of at-rest earth pressure K_O=1 was used. Their analysis indicates that as drainage occurs, the vertical stress in the clay decreases and the stresses in the Stone increases as the clay go from the undrained to the drained state.

2.1.5 Priebe Method

It has been more than twenty years since Priebe (1995) established the theory of estimating the reduction in settlement due to stone columns ground improvement also uses the unit cell model, assuming that the stone column will be in the state of plastic equilibrium under a triaxial stress state while the soil within the unit cell is idealized as an elastic state. Meantime, Priebe method has been flexible of adapting several approaches with the consideration of many reduction factors as to suit and simulate the composite ground condition. Based on these improvement factors, the composite deformation modulus increases meanwhile the settlement decreased.

The stone columns installation densifies the soil surrounding the stone columns. In this cases, first of all the densification of the soil has to be evaluated and only then - on the basis of soil data adapted correspondingly - the design of vibro replacement follows.

The complex system of vibro replacement allows a more or less accurate evaluation only for the well-defined case of an unlimited load area on an unlimited column grid. In this case a unit cell with the area A is considered consisting of a single column with the cross section A_C and the attributable surrounding soil.

Furthermore the following idealized conditions are assumed:

i. The column is sitting on a rigid layer.
ii. The column material is uncompressible.
iii. The bulk density of column and soil is neglected.

The stone column would not fail in end bearing but settlement would take place due to bulging under loaded area at certain length of the stone column meanwhile the vertical stress would be the same along the stone column length.

The basic improvement of a soil achieved with the installation of stone columns is evaluated on the assumption that the column material develops lateral forces towards surrounding soil as such the surrounding soil reacts elastically. Furthermore, at the moment when the stone column installed by displacing the soft soil, the initial pressure difference is equal to zero therefore, coefficient of earth pressure K = 1**.** The basic improvement factor n_0 due to stone column improvement has been expressed by Equation 2.2:

$$n_0 = 1 + Ac/A \text{ x } [(½ + f (\mu_S, Ac/A) / (K_{ac} \text{ x } f (\mu_S, Ac/A)) -1] \qquad (2.2)$$

Where n_0 is influenced most with the function of Poisson's ratio and area replacement ratio as expressed at Equation 2.3:

$$f (\mu_s, Ac/A) = (1- \mu_s) \text{ x } (1- Ac/A) / (1-2 \text{ x } \mu_s + Ac/A) \qquad (2.3)$$

The coefficient of active earth pressure for the stone column material K_{ac} was found by Equation 2.4:

$$K_{ac} = \tan^2 (45^o - \phi_c/2) \qquad (2.4)$$

A Poisson's ratio of μ_S=1/3 which is adequate for the state of final settlement.

The relationship between the improvement factor n_o , the reciprocal area ratio A_c/A and the friction angle of the backfill materials ϕ_c is shown in Figure 2.1 and Equation 2.2:

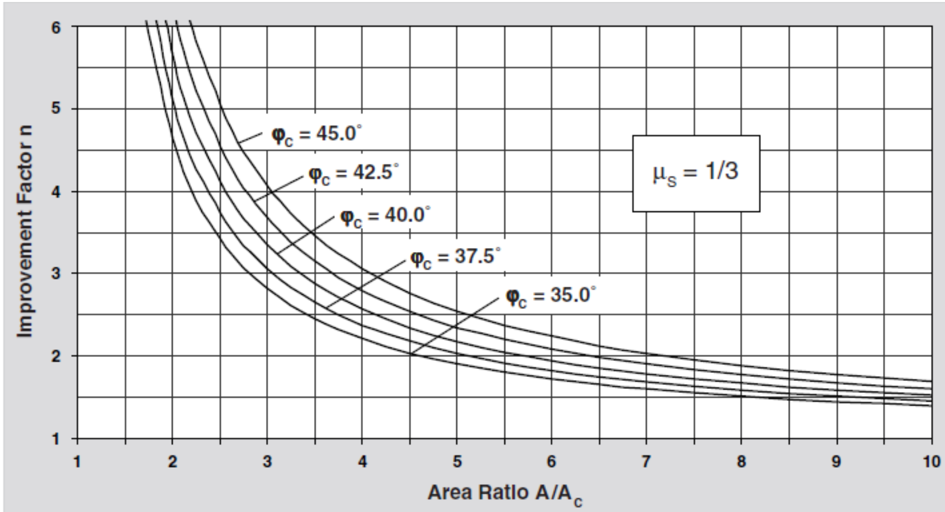

Figure 2.1: Design chart for vibro replacement (Priebe, 1995)

Where,

n_o = settlement improvement ratio
A_c = stone column area
A = grid area
μ_S = Poisson's ratio
K_{aC} = coefficient of active earth pressure for column material
ϕ_c = friction angle of column material.

Another assumption has been made by Priebe, considering the stone column backfill material used is compressible material. Therefore, the Point of concern here is any settlement caused by a load which is not related to column bulging. the actual improvement Factor correspond at best with the ratio of the constrained modulus of column material And soil Dc/Ds , as shown in Figure 2.2.

We need to know the area replacement ratio $(A_C/A)_1$ that starts corresponding to the ratio of the constrained modulus of column and Soil Dc/Ds. For example, at μ_S=1/3 , the lower positive result of the following expression with n_o= Dc/Ds delivers the area ratio $(Ac/A)_1$ as expressed at Equation 2.58:

$$(Ac/A)_1 = -(4.K_{ac}(n_o-2)+5)/(2.(4.K_{ac}-1)) \pm \tfrac{1}{2} \times [(4.K_{ac}(n_o-2)+5/(4.K_{ac}-1))^2 + (16.K_{ac}(n_o-1)/(4.K_{ac}-1))]^{1/2} \quad (2.5)$$

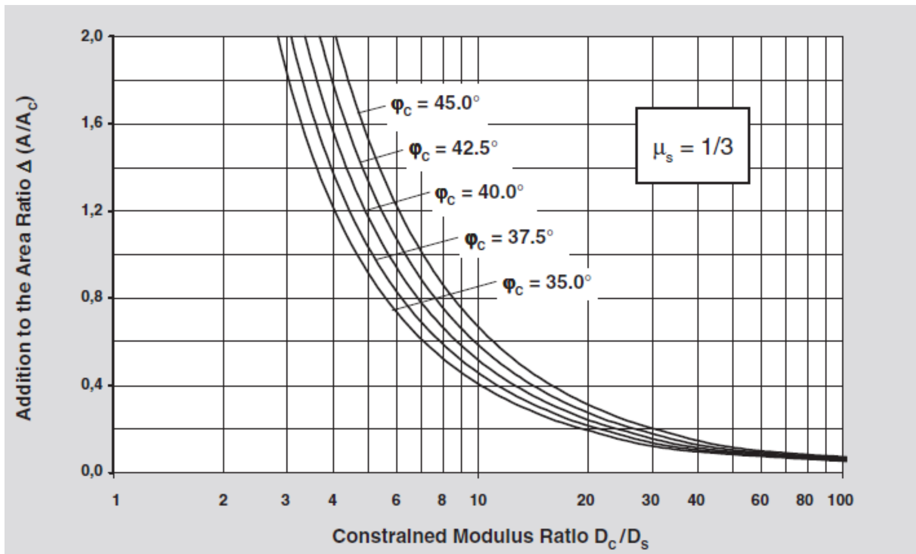

Figure 2.2: Column compressibility (Priebe, 1995)

As an assumption, the settlement caused by compressibility of the column material can be considered in using a reduced improvement factor n_1 which results from the equation developed for the basic factor n_o when the given reciprocal area ratio A_c/A is increased by additional value of $\Delta(A_c/A)$ as expressed at Equation 2.6 depending on the ratio of the constrained modulus Dc/Ds which can be extracted from Figure 2.1 below as such the new area replacement ratio (A_c/A) has been expressed by Equation 2.7 in order to get the regulated improvement factor n_1 as show at Equation 2.8.

$\Delta\,(A/A_c) = (1/(Ac/A)_1) - 1$ (2.6)

$\overline{A_c}/A \;=\; 1\,/\,(A/A_c + \Delta\,(A/A_c)$ (2.7)

$n_1 = 1 + \overline{A}_c/A\;\left[\,(0.5 + f\,(\mu_S, \overline{Ac}/A))\,/(\,K_{ac}\,.\,f\,(\mu_S, \overline{Ac}/A\,)\;)\,-1\right]$ (2.8)

Neglecting density difference between stone column and soil means that the initial pressure difference between the columns and the soil which creates bulging depends only on the load distribution of the foundation on columns and soil. This value is constant over the entire column length but the weights of the columns $W_{\mathbf{C}}$ and of the soil $W_{\mathbf{S}}$ may exceed the external loads thus, it has to be added.

Under consideration of these additional loads, the initial pressure difference decreases with each depth increment and the bulging is reduced correspondingly.

Since the pressure difference is a linear parameter to the depth, the ratio of the initial pressure difference and the one depending on depth expressed as depth factor $f_{\mathbf{d}}$ delivers a value by which the improvement factor $n_{\mathbf{1}}$ increases to the final improvement factor as shown at Equation 2.14 on account of the overburden pressure.

The depth factor f_d is calculated based on the column & soil pressure differences which represented by $(p_C + \gamma_C \cdot d) \cdot Ka_C$ &$(p_S + \gamma_S \cdot d)$ $(K_S = 1)$, the coefficient of earth pressure of the columns changes from the active value $Ka_{\mathbf{C}}$ to the value at rest $K_{\mathbf{0C}}$. Up to the depth where the straight line assumed for the pressure difference, meets the actual asymptotic line, the depth factor lies on the safe side. In practical cases the treatment depth is mostly less.

The pressure ratio at the stone column p_c to the pressure at surrounding soil p_s shall be determined as per Equation 2.9:

$$Pc/Ps = (0.5 + f(\mu_s, \overline{Ac}/A)) / (K_{ac} \cdot f(\mu_s, \overline{Ac}/A)) \quad (2.9)$$

Thus, the pressure at the stone column shall be obtained from Equation 2.10:

$$Pc = P / [(\overline{Ac}/A) + (1 - (\overline{Ac}/A) / Pc/Ps)] \quad (2.10)$$

Meanwhile, weight of soil (Ws) and weight of column (Wc) shall be calculated as per Equation 2.11 and 2.12 alternatively.

$$Ws = \Sigma (\gamma_s \cdot \Delta d) \quad (2.11)$$

$$Wc = \Sigma (\gamma_c \cdot \Delta d) \quad (2.12)$$

The coefficient of column material will transformed from active to at rest (Koc) as determined by Equation 2.13

$$Koc = 1 - \sin Øc \quad (2.13)$$

Thus, the depth factor shall be calculated based on Equation 2.14

$$f_d = 1 / (1 + (Koc - Ws/Wc) x (1/Koc) x (Wc/Pc)) \quad (2.14)$$

And the regulated improvement factor n_2 shall be calculated as per Equation 2.15:

$$n_2 = f_d \times n_1 \quad (2.15)$$

The simplified diagram in Figure 2.3 considers the same bulk density $\boldsymbol{\gamma}$ for columns and soil which is not on the safe side. Therefore, for safety reasons, the lower value of the soil $\boldsymbol{\gamma_S}$ should be considered in this diagram.

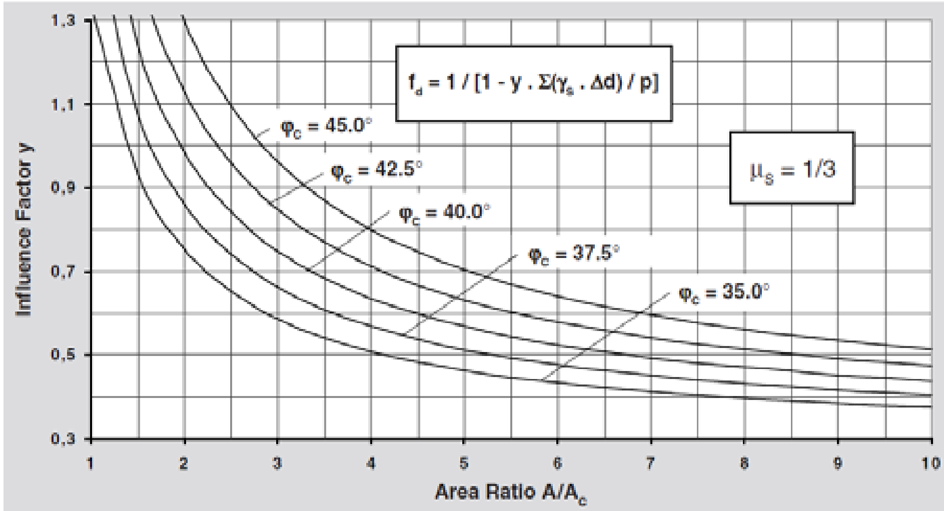

Figure 2.3: Determination of the Depth Factor (Priebe, 1995)

A step of compatibility control must be carried out in order to ascertain that the assigned load for the stone column is bearable in accordance to stone column compressibility in a step which has more simplification and approximation than mathematic calculation.

At increasing depths, the confining pressure of the soil reaches such an extent that the columns do not bulge anymore.

The depth factor will not increase to infinity based on the linear assumption of pressure difference with depth, therefore this compatibility is limiting the depth factor and subsequently load assigned to the stone column as shown at Equation (2.16) as such the settlement due to inherent compressibility would not exceed than composite system settlement.

In the first place this control applies when the existing soil is considered pretty dense or stiff.

$$f_d \leq (D_c/D_s)/(P_c/P_s) \qquad (2.16)$$

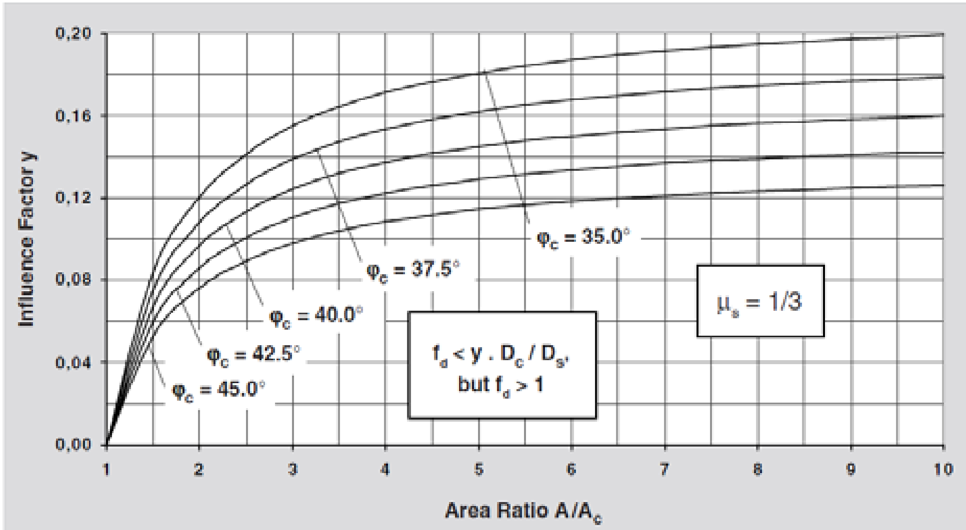

Figure 2.4: Limit value of depth factor (Priebe, 1995)

The maximum value of f_d can be extracted from the diagram in Figure 2.4.

A depth factor $f_d < 1$ should not be considered, even though it may result from the calculation.

Therefore, a second compatibility control is imperatively required which relates to the maximum value of the improvement factor. It ascertains that the column settlement inherent from their compressibility does not exceed the settlement of the surrounding soil due to its compressibility by the loads assigned to each as expressed at Equation (2.17). In the first place this second controls applies when the existing soil is encountered pretty loose or soft.

$$n_{max} = 1 + (Ac/A) \times (Dc/Ds - 1) \qquad (2.17)$$

Where A_C/A is the original replacement ratio and not the modified ratio.

There is no final formula or equation which determine the behavior performance of pad footing or strip footing founded on stone column treated ground but the available design ensues from the performance of an unlimited column grid below an unlimited load area.

The total settlement S_∞ due to the applied load on a stone column treated ground readily to determine at Equation (2.18) on the basis of the foregoing description with n_2 as an average value over the depth d.

$$S_\infty = P \times d / (Ds \times n_2) \qquad (2.18)$$

Diagrams which are given in Figure 2.5 and Figure 2.6, allow concluding from this value the settlements of single or stripping footings on groups of columns. These diagrams - with the diameter of the stone columns D as one parameter - are based on numerous calculations which considered load distribution on one side and a lower bearing capacity of the outer columns of the column group below the footing on the other side.

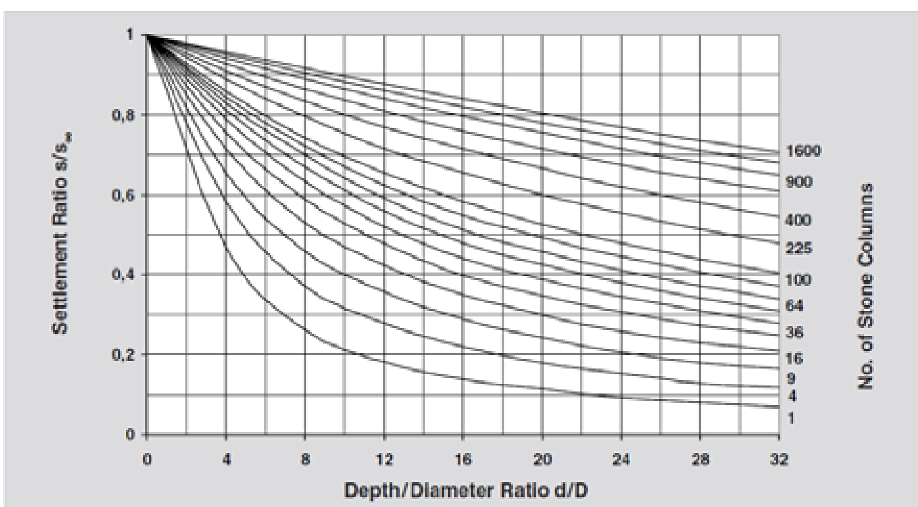

Figure2.5: Settlement of single footing (Priebe, 1995)

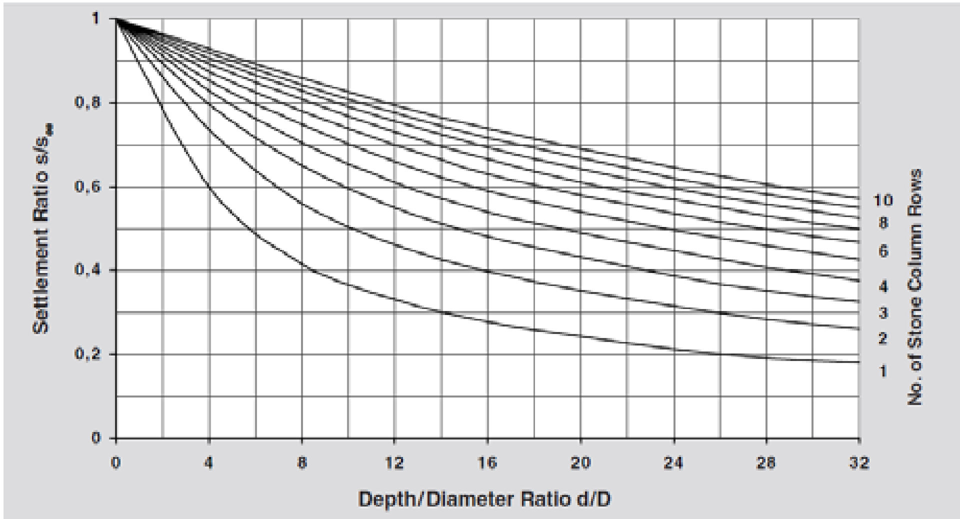

Figure 2.6: Settlement of strip footing (Priebe, 1995)

The diagrams do not refer directly to footing extensions as to be expected. The exits of a pad footing will mark indirectly the grid area A and the improvement factor n has to be obtained beside number of stone columns required at the above said grid area A. For example, the settlement reduction for a larger footing under the same load shall be compensated with a lower improvement ratio which results from a high area replacement. The approximation given for the diagrams by this assumed compensation seems to be acceptable for usually considered area ratios, i. e. up to some $A/A_C = 10$.

Obviously the above diagrams are valid for homogeneous conditions only and refer to the settlement s up to a depth d which the second parameter is counting from foundation level.

The shear performance of treated ground with stone column is an essential part to be investigated. While the shear stress increases due to foundation load, a bearing wedge element try to break out therefore, stone columns continues to deform until any further load increment would be transferred to adjacent columns and most ideal case comply with the aforementioned is the landslide or slip failure which will not occur before the bearing capacity of the total group of columns installed has been activated. The stone columns receive an increased portion of the total load m which depends on the area ratio A_C/A and the improvement factor n, thus m is expressed at Equation (2.19):

$$m = (n - 1 + (Ac/A)) /n \qquad (2.19)$$

In order to simplified the design which is not taking under consideration the decrease in volume of the surrounding soil to stone column due to bulging. Thus and especially at a high area ratio, the soil in fact are receiving a higher portion of the total load than the design has assumed. In order not to overestimate the shear resistance of the columns based on sharing basis of load distribution between columns and soil, the proportional load on the columns has to be reduced as determined by Equation (2.20):

$$m' = (n - 1) /n \qquad (2.20)$$

The diagram in Figure 2.7 shows in solid lines the proportional load of the columns m´ and in dashed lines the not reduced one m.

Based on the proportional load assigned to the column and soil, the average internal friction for the composite ground system shall be calculated as per Equation (2.21):

$$\tan Ø' = m'.\tan Ø_c + (1- m') . \tan Ø_s \qquad (2.21)$$

Due to the damages of soil structure accompanied the stone column installation, the soil cohesion shall be reduced for safety reason and the composite system cohesion shall be considered as a load proportional as per Equation (2.22):

$$C' = (1-m').\ Cs \qquad (2.22)$$

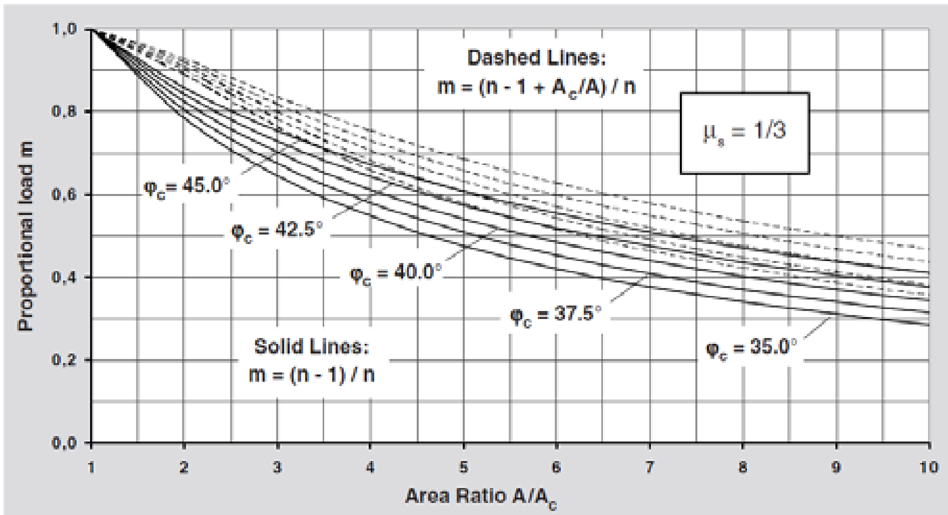

Figure 2.7: Proportional load on stone columns (Priebe, 1995)

2.2 Unit Cell Concept

The unit cell concept is fundamental to the analysis of stone column. To begin with, consider the tributary area of soil surrounding each stone column as illustrated in Figure 2.8.