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ABSTRACT

Construction of multi-story buildings with basement(s) in urban environment necessitates the installation of side support system including side walls for both protection of adjacent structures and for groundwater control. The side support walls are usually penetrated underneath the foundation level for different reason. The penetration of the side walls surrounding the construction excavations provides confinement of the soil underneath the foundation level. The influence of such confinement on the allowable pressure and in general the behavior of the foundation system is usually ignored during the design of the foundation system. This paper is a part of major research aiming to highlight such influence and hopefully to provide a practical tool to aid geotechnical engineer to include such influence in the design. This paper is concerned with foundations on granular soils. The paper presents numerical modelling of the problem using Finite Element method in the plain strain condition. A parametric analysis is carried utilizing this model to uncover the influence of some parameters on the behavior of raft foundations on soils under such conditions. The studied parameters include; raft width, embedded depth of the side walls and the state of denseness of the granular soils under the foundations. The depth of the foundation level is kept constant at 4m for one basement during the investigation. The results indicated that the sand confinement below the foundation level has a significant influence on the behavior of the foundation in such environment modifying the shear failure pattern under the raft and thus increasing the allowable pressure for shear failure and reducing the settlement.

Keywords: *Raft foundation, Sand, Soil Confinement, Sidewalls, and Finite Element.*

Introduction

Several changes are expected in the behavior of shallow foundations due to confinement of the bearing sand. Footings with side support walls (i.e., rafts surrounded by sheet-pile walls) constructed on a deep sand layer are common types of foundations resting on laterally confined sand. The side support walls are usually penetrated underneath the foundation level for different reason such as to retain earth, water or any other fill materials and reduce the piping failure at the excavation level. The effect of sand confinement on the behavior of shallow foundations has been investigated through loading confined sand specimens, testing foundation models resting on laterally or vertically confined sand, and conducting theoretical analyses. One of the early studies on the effect of lateral confinement on compressibility of sand was undertaken by Hendron (1963) through loading a specimen of laterally confined, moderately coarse uniform sand under drained condition. The measured strains or settlements were considerably less than those predicted for unconfined sand utilizing physical and mathematical models (Nova and Montrasio 1991) and commonly used equations based on results of standard penetration (e.g., Meyerhof 1965; Burland and Burbidge 1985) or cone penetration tests (e.g., Schmertmann 1970; Schmertmann et al. 1978). Similar behavior was reported by Rajagopal et al. (1999) based on results of triaxial compression tests on sand specimens confined with geocells. Several investigators have reported a

significant increase in bearing capacity of foundation models on sand confined by placing layers of geogrids horizontally (e.g., Binquet and Lee 1975; Fragaszy and Lawton 1984; Mahmoud and Abdrabbo 1989; Khing et al. 1993; Das et al. 1996; Dash et al. 2001a) or vertically (e.g., Mandal and Manjunath 1995, Dash et al. 2001b). Using more-rigid confining elements, such as metal skirts fixed to the edges or the circumferences of foundation models resting on sand, resulted in considerable improvement in their bearing capacity and stress–strain behavior (Al-Aghbari 1999, 2002; Villalobos et al. 2003; Al-Aghbari and Mohamedzein 2004).
Several authors studied of lateral confinement on compressibility of sand using different approaches have been reported. However few researchers studied the influence of such confinement on the allowable pressure and in general the behavior of the foundation system is usually ignored during the design of the foundation system. Therefore, the aim of this research was to study and to provide a practical tool to aid geotechnical engineer to include such influence in the design for diversity of parameters.

NUMERICAL MODEL

The numerical model has been validated using results reported by Youssef et al., (2003). In this research, analytical studies of the beamed wall system are performed using the finite element program (2D PLAXIS). The study is made for cohesion less soils with different properties. The hardening soil model is used in this research. The description of this model is introduced in Fig. (3-1).
In this analysis the total wall height, H, is kept constant while the driven depth, d, is varied. The driven depth of the wall is taken equal to 1.25 times the free height in the initial analysis.

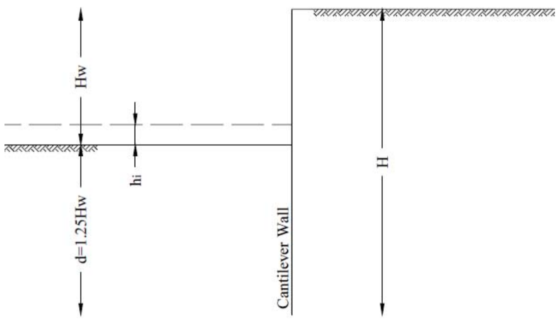

Fig. (1): Typical mesh dimensions for sheet pile wall, Youssef et al., (2003)

The soil profile consists of a medium dense sand layer with Φ=28. The sheet pile was illustrated in Table (1). The results of validation showed good agreement with the model in case of the relationship between the maximum bending moment on the wall versus increasing in the driven depth for the three wall heights and the relationship between the movement and increase in driven depth as shown in Fig. (2) and Fig. (3) respectively.

Table (1): Properties of sheet pile Wall, Youssef et al., (2003)

No.	Property	Group (1)
1	Flexural rigidity per meter	1.26E7
2	Axial stiffness per meter	3.78E5
3	Unit weight per meter	15.00
4	Poisson's Ratio	0.25
5	Equivalent wall thickness	0.60

The difference between results was convergent, which didn't exceed 6 %. Extensive numerical models of raft foundation resting on confined sand were carried out to study the influence of some parameters on the behavior of raft foundations on confined sand soils for different parameters using (2D PLAXIS) program version 8.2 that are based on Finite Element Method.

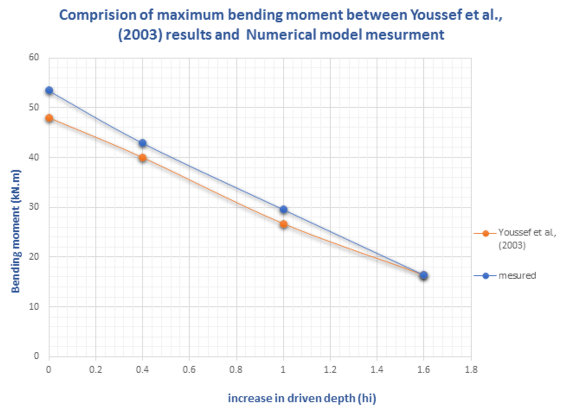

Fig. (2): Maximum Moment versus Increase in Driven Depth
(Hw=3.0m & ϕ =28°)

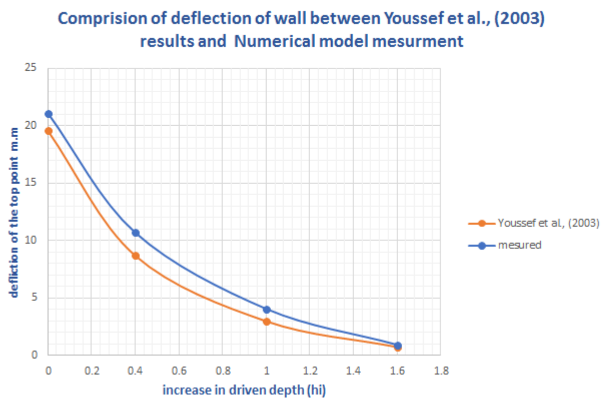

Fig. (3): Deflection of the Wall versus Increase of Driven Depth
(Hw=3.0m & ϕ =28°)

A series of models have been carried out. The plane strain models were used with the 15 nodes element. The mesh was generated by the program with (Fine) mesh as shown in Fig. (4). The subsoil is consisted of a deposit of sand layer of 100 m thickness. The Mohr-Coulomb model was considered to model elastic- plastic behavior of sand soils samples in this research. It involves five input parameters: which are Es and υ for soil elasticity; Φ and c for soil plasticity and Ψ angle of dilatancy. Since we considered sand soils in this study in the drained case, soil cohesion was set to 1*10-3 kPa to avoid errors.

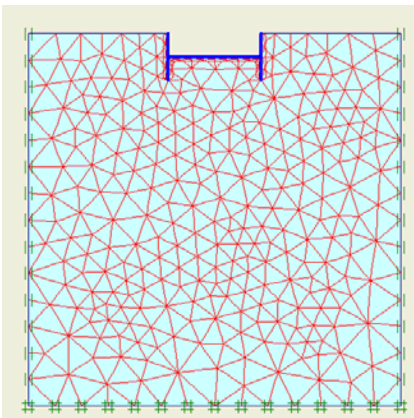

Fig. (4): Meshing for 2D PLAXIS model

Kishida and Nakai (1977) introduced empirical formulae to calculate stiffness parameters of sand soil. The soil stiffness parameters presented were function of N of SPT number and were related to the relative density (Dr) of sand soils as shown in Table (2).

$$E_S = 1.6 * N_{SPT}$$

A nonlinear analysis was assumed, so that (Es) represent a secant modulus for low load level.

Table (2): Mohr-Coulomb model parameters of sand soils

Soil code	Angle of Friction (Φ)	Dilatancy angle (ψ)	No of S.P.T Blows (N)	Relative density (Dr)	Young modulus (Es) kPa
1	28	0	5	0.20	12800
2	30	0	15	0.40	40000
3	35	5	30	0.60	72000
4	40	10	50	0.80	100000

Interfaces are special elements for the soil-foundation interaction in 2D PLAXIS program. The interface strength was defined by the parameter (Rint). Suitable values of Rint the interaction between sand soils and R.C supporting walls or raft footing was found to be 0.7. Gomes (2013)
R.C supporting walls and raft footing were made from R.C material. The unit weight of the reinforced concrete (γc) is equal to 25 kN/m3, the module of elasticity of each pile is 20*106 kN/m2 and the Poisson's ratio (υc) was taken to be 0.22.
R.C supporting walls was taken as a row of piles with spacing 1D was considered in the 2D PLAXIS model as a continuous bored piles wall with an equivalent thickness (deq) in the long direction. The equivalent thickness (deq) was determined by the program according to the following equation. Table (3) and Table (4) show the properties of supporting walls and raft footing in (2D PLAXIS) model respectively.

$$d_{eq} = \sqrt{12\frac{EI}{EA}}$$

Table (3): Supporting walls parameters in 2D PLAXIS program

Row of Piles Diameters (m)	Axial Stiffness (EA) kN/m	Flexural Rigidity (EI) kN.m2/m	Equivalent Thickness (deq) m	Weight Of plate (w) kN/m/m
1	1.73 e 7	1.08 e 6	0.86	20

Table (4): raft footing parameters in 3D PLAXIS program

Raft footing thickness (m)	Axial Stiffness (EA) kN/m	Flexural Rigidity (EI) kN.m2/m	Equivalent Thickness (deq) m	Weight Of plate (w) kN/m/m
1	20 e 6	1.89 e 6	1	25

NUMERICAL ANALYSIS

The geometry of the Finite Element soil model adopted for this analysis is 80 m in depth and in width as shown in Fig. (5). The boundary condition in all cases was evaluated to relief its effect on the results of the numerical model. Fourteen main series of tests were carried out to uncover the influence of some parameters on the behavior of raft foundations on soils under such conditions for different parameters using 2D PLAXIS models as shown in Fig. (6)

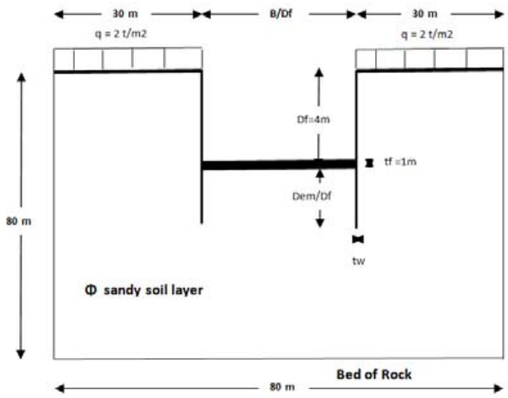

Fig. (5): The model geometry for variable parameters

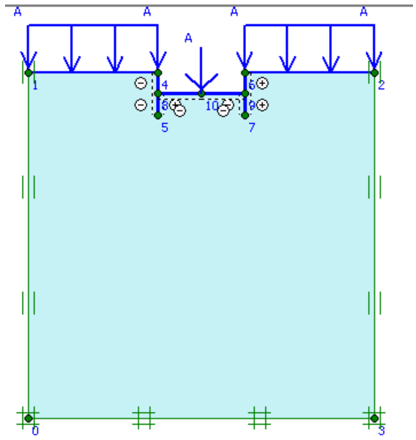

Fig. (6): The full geometry for 2D PLAXIS model

The studied parameters include; raft width ratio (B/Df), embedded depth of the side walls ratio (Dem/Df) and the state of denseness of the granular soils under the foundations that related to the angle of sand friction to the (φ). The depth of the foundation level is kept constant at 4m for one basement during the investigation as shown in Table (5).
Each serious was carried out to study the effect of one parameter while other parameters were kept constant.

Table (5): Numerical model and studied parameters

Series	**Constant parameters**	**Variable parameters**
1	Df =4m,tf=1m,tw=1m, Φ=28,B/H=5	Dem/Df =0,0.25,0.5,1,1.5,2
2	Df =4m,tf=1m,tw=1m, Φ=30, B/H=5	Dem/Df =0,0.25,0.5,1,1.5,2
3	Df =4m,tf=1m,tw=1m, Φ=35, B/H=5	Dem/Df =0,0.25,0.5,1,1.5,2
4	Df =4m,tf=1m,tw=1m, Φ=40, B/H=5	Dem/Df =0,0.25,0.5,1,1.5,2
5	Df =4m,tf=1m,tw=1m, Φ=30, Dem/Df=0.5	B/Df =2.5,5,7.5
8	Df =4m,tf=1m,tw=1m, Φ=30, Dem/Df=1.0	B/Df =2.5,5,7.5
11	Df =4m,tf=1m,tw=1m, Φ=30, Dem/Df=1.5	B/Df =2.5,5,7.5
14	Df =4m,tf=1m,tw=1m, Φ=30, Dem/Df=2.0	B/Df =2.5,5,7.5

RESULTS OF NUMERICAL ANALYSIS

Using above modeling and soil properties analysis that carried out in PLAXIS 2D for raft foundation resting on confined sand by side supporting walls, the stress-vertical settlement relationships were obtained at different investigated parameters
The bearing capacity was plotted versus the total displacement for different embedded depth of the side walls ratio (Dem/Df) of 0, 0.25, 0.5, 1, 1.5 and 2 for constant parameters of Df =4m, tf=1m, tw=1m, Φ=28 and B/H=5 as shown in the Fig. (7). This relationship was repeated for different constant parameter Φ=30, 35 and 40 as mentioned in Fig. (8), Fig. (9) and Fig. (10) receptively.
It is worth mention that the increase of embedded depth of the side walls ratio (Dem/Df) has a considerable effect in increasing the raft capacity and decreasing the settlement. That is backed to the confinement effect of vertical ribs that provided lateral constrains which prevented particles under the raft from movement to the region outside the wall as a result the raft capacity is increased.

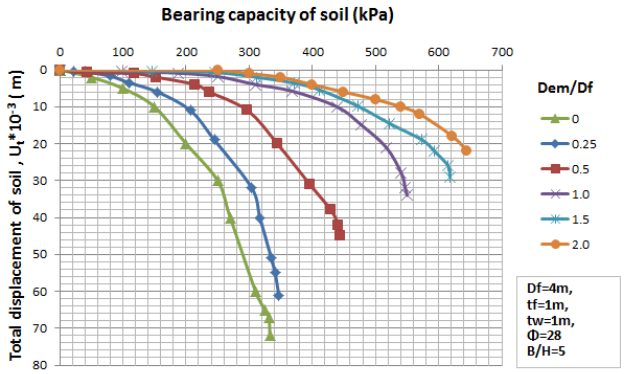

Fig. (7): The Bearing capacity of raft versus the total displacement for Φ=28

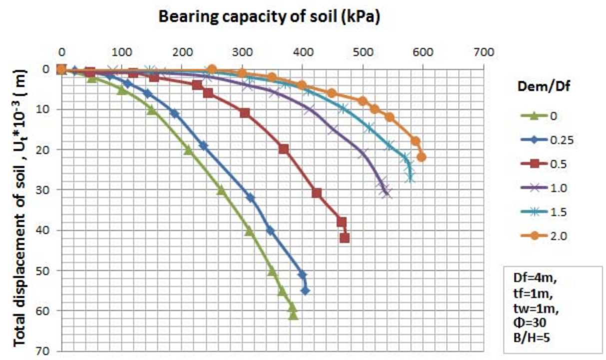

Fig. (8): The Bearing capacity of raft versus the total displacement for Φ=30

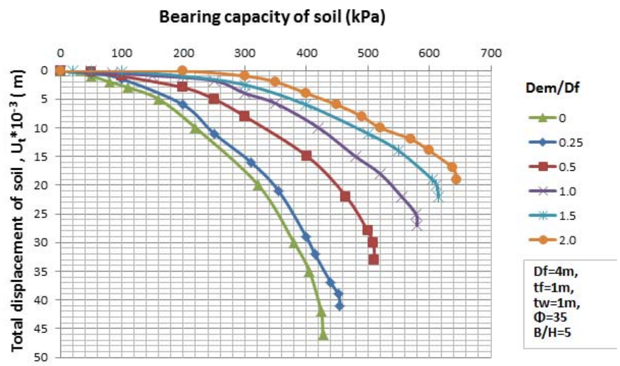

Fig. (9): The Bearing capacity of raft versus the total displacement for Φ=35

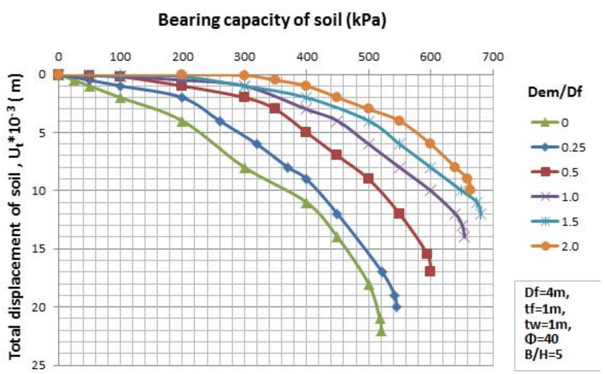

Fig. (10): The Bearing capacity of raft versus the total displacement for Φ=40

To capture the effect of raft width ratio in different cases of the embedded depth of the side walls ratio (Dem/Df) on the bearing capacity of the raft foundation, the bearing capacity was plotted versus the total displacement for different raft width ratio (B/Df) of 2.5, 5, and 7.5 for constant parameters of Df =4m, tf=1m, tw=1m, Φ=30 and Dem/Df=0.5 as shown in the Fig. (11). This charts were repeated for different constant parameter Dem/Df=1.0,1.5 and 2 as introduced in Fig. (12), Fig. (13) and Fig. (14).
It can mention that the increase of raft width ratio has a remarkable effect in decreasing the raft capacity.

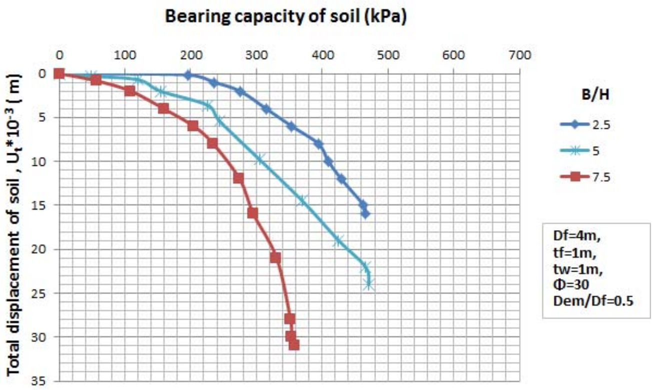

Fig. (11): The Bearing capacity of raft versus the total displacement for Dem/Df=0.5

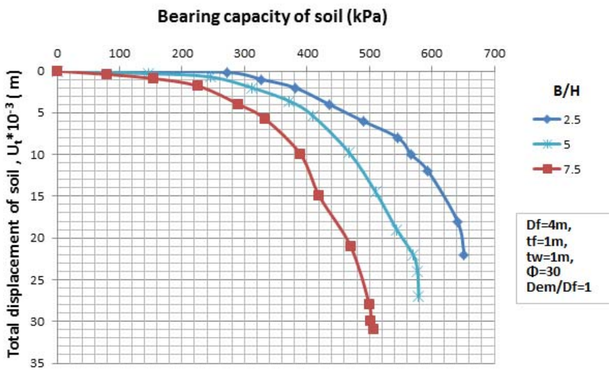

Fig. (12): The Bearing capacity of raft versus the total displacement for Dem/Df=1.0

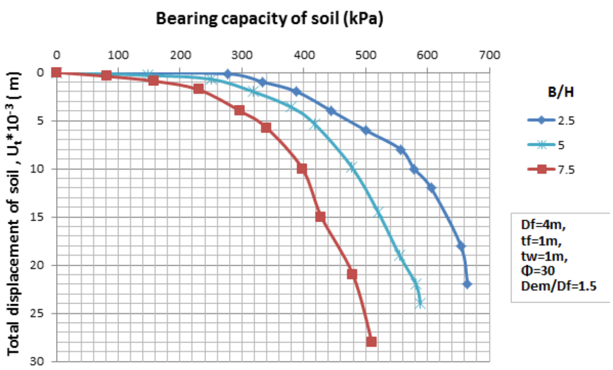

Fig. (13): The Bearing capacity of raft versus the total displacement for Dem/Df=1.5

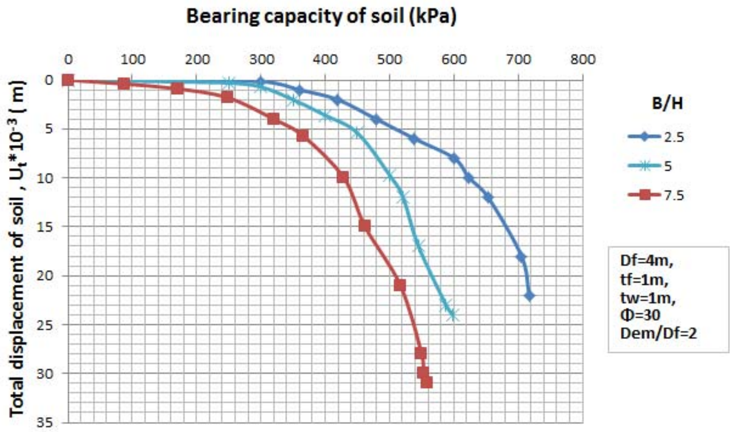

Fig. (14): The Bearing capacity of raft versus the total displacement for Dem/Df=2.0

DISCUSSIONS OF RESULTS

The effects of different parameters on the ultimate bearing capacity of raft resting on a homogeneous confined sand soil were obtained and discussed. The bearing-capacity improvements of the raft with and without ribs are represented using a non-dimensional factor, called the bearing-capacity Factor (BCF). This factor is defined as the ratio of either the footing ultimate pressure of raft resting on confined sand with embedded supporting wall depth ($q_{u\ con}$) to the footing ultimate pressure in tests of raft without any confined (Dem=0) ($q_{u\ without\ con}$) as presented by the following equation.

$$BCF = \frac{q_{u\ con}}{q_{u\ wihout\ con}}$$

Fig. (15) compare the bearing-capacity ratio (BCF) for raft with and without confinement for different embedded depth of the side walls ratio (Dem/Df) of 0, 0.25, 0.5, 1, 1.5 and 2 for constant parameters of Df =4m, tf=1m, tw=1m, and B/H=5 as shown in It can conclude that, the increase of embedded depth leads to a significant increase in the (BCR) by 240 % of its initial value without confinement. It has been found that the most optimum and economical rib depth is found to be (Dem/Df) = 1.5 because of over this rang the degree of improvement in the ultimate bearing capacity remain constant and a trivial increase is obtained.

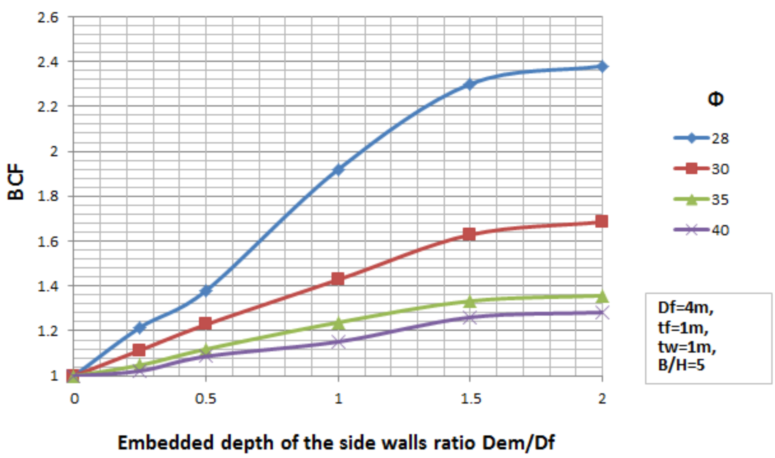

Fig. (15): The bearing-capacity Factor (BCF) versus the embedded depth of the side walls ratio (Dem/Df)

On the other hand, the bearing-capacity Factor (BCF) was plotted versus the raft width (B/Df) of 2.5,5 and 7.5 for constant variables Df =4m, tf=1m, tw=1m and Φ=30 as shown in Fig. (16). From the result, it is found that, the increase in the raft width (B/Df)leads to a remarkable decrease in the (BCF) by as much as 138% of its initial value without any confinement.

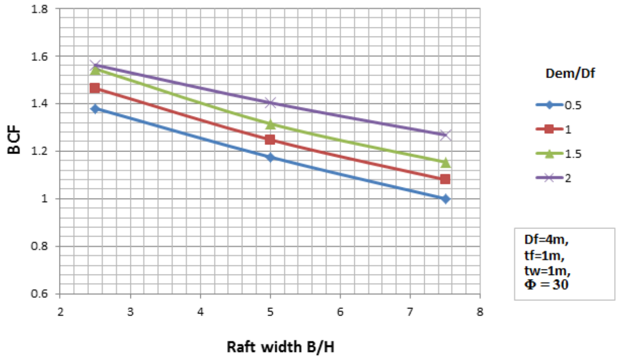

Fig. (16): The bearing-capacity Factor (BCF) versus the raft width ratio (B/Df)

Moreover, the influence of the confinement of penetration of the side walls underneath the foundation level on the allowable pressure that estimated using the Egyptian code equation for non-confinement case can be investigated as shown in Table (6).

Table (6): Relation between the BCF of the FEM and the Egyptian code equation for non-confinement cases

Series	Variable parameters	qu of FEM for non-confinement cases (kPa)	qu of Egyptian code for non-confinement cases (kPa)	qu of Terzage Method for non-confinement cases (kPa)	Average (BCF) of FEM confinement cases
Df =4m,tf=1m,tw=1m, Φ=28,B/H=5	Dem/Df =0.25,0.5,1,1.5,2	300	282	305	210%
Df =4m,tf=1m,tw=1m, Φ=30, B/H=5		360	341	365	150%
Df =4m,tf=1m,tw=1m, Φ=35, B/H=5		405	385	402	135%
Df =4m,tf=1m,tw=1m, Φ=40, B/H=5		510	465	492	120%
Df =4m,tf=1m,tw=1m, Φ=30,Dem/Df=0.5,1,1.5 and 2	B/Df =2.5	360	321	366	143%
	B/Df =5				138%
	B/Df =7.5				136%

CONCLUSIONS

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