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Bearing capacity of strip footings on a stone masonry trench in clay

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Abstract. Soft clay strata can suffer significant settlement or stability problems under building loads. Among the methods proposed to strengthen weak soils is the application of a stone masonry trench (SMT) beneath RC strip foundations (as a masonry pad-stone). Although, SMTs are frequently employed in engineering practice; however, the effectiveness of SMTs on the ultimate bearing capacity improvement of a strip footing rested on a weak clay stratum has not been investigated quantitatively, yet. Therefore, the expected increase of bearing capacity of strip footings reinforced with SMTs is of interest and needs to be evaluated. This study presents a two-dimensional numerical model using the discrete element method (DEM) to capture the ultimate load-bearing capacity of a strip footing on a soft clay reinforced with a SMT. The developed DEM model was then used to perform a parametric study to investigate the effects of SMT geometry and properties on the footing bearing capacity with and without the presence of surcharge. The dimensions of the SMTs were varied to determine the optimum trench relative depth. The study showed that inclusion of a SMT of optimum dimension in a soft clay can improve the bearing capacity of a strip footing up to a factor of 3.5.

Keywords: bearing capacity; soft clay; stone masonry trench; discrete element method; strip footing

1. Introduction

In order to improve the ultimate load-bearing capacity and reduce the settlement of shallow foundations on soft to medium clay strata, some techniques have been developed and proposed in the literature. These techniques include the use of stone columns (Hughes *et al*. 1975, Dash and Bora 2013, Nazari Afshar and Ghazavi 2014), granular soil trenches (Madhav and Vitkar 1978, Bouassida *et al*. 2014), reinforcement elements such as geogrids (Chen and Abu-Farsakh 2015, Mir Mohammad Hosseini and Salehi 2015) and geotextiles (Cicek *et al*. 2015), and stone masonry trenches (SMTs). The use of SMTs beneath the strip RC footings of low-rise residential framed buildings is the most economic technique which is widely used in Iran urban areas with adequate supplies of good stone.

As shown in Fig. 1(a), stone masonry trenches are made up mostly of roughly squared rubble stones with variable dimensions laid above each other and the spaces among them are filled with a weak cement mortar. It should be pointed out that this weak mortar is mainly used to fill the voids between the stone units and increase the integrity of the SMT. In fact, SMTs' courses are of

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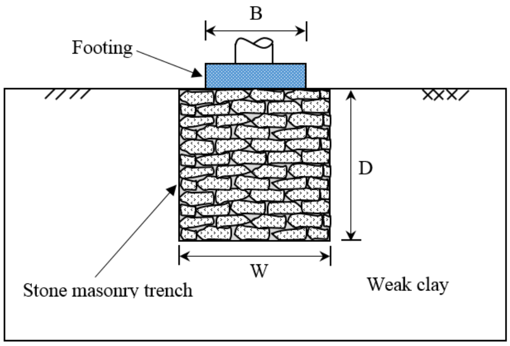
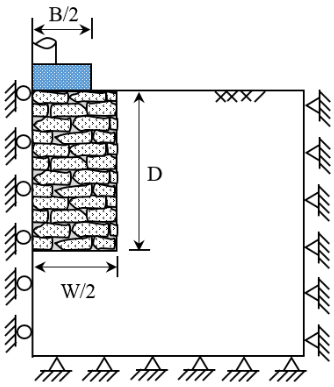
(a) A stone masonry trench before footing construction



(b) Geometric parameters



(c) Modeled domain and boundary conditions

Fig. 1 Soil improvement using a rubble stone masonry trench

different heights, but the stone units in each course are approximately squared to the same height (i.e., coursed, squared rubble stone masonry) and are roughly leveled at regular intervals. Structural bonding of SMTs is obtained by the overlapping (interlocking) of the stone masonry units which are laid above each other. The stone type used in the construction of a masonry trench to improvement weak clay soil beds beneath the residential buildings foundations is generally limestone. According to the general bearing capacity theory (Das 2009), the load-bearing ultimate capacity of a shallow strip foundation resting on a frictionless soil failing at general shear failure mode can be estimated as follows

$$q_u = cN_c \lambda_{cd} + qN_q \lambda_{qd} \tag{1}$$

where the bearing capacity factors, N_c and N_q are as 5.14 and 1.0, respectively. The depth factors (λ_{cd}, λ_{qd}) can be approximated by the following equations as proposed by Hansen (1970)

$$\lambda_{cd} = \begin{cases} 1 + 0.4\left(\dfrac{D_f}{B}\right) & for \ \dfrac{D_f}{B} \leq 1 \\ 1 + 0.4\tan^{-1}\left(\dfrac{D_f}{B}\right) & for \ \dfrac{D_f}{B} \succ 1 \end{cases} \tag{2}$$

$$\lambda_{qd} = 1.0 \tag{3}$$

From this theory it is apparent that the use of a SMT can enhance the ultimate bearing capacity of a strip footing by increasing the footing depth. Therefore, it seems that the bearing capacity of a strip footing of width B resting on a SMT of width W (Fig. 1(b)) equal to B, may be estimated roughly by using the abovementioned depth factors replacing the SMT depth with the footing depth (i.e., $D_f = D$). However, for wider SMTs (i.e., $W > B$), this approach does not appear to guarantee the validity of the solution. This is because, a SMT is a jointed rock mass not a rigid solid box beneath the footing. In other words, stiff stone units in a SMT can slide or completely detach under the applied loads. To the best knowledge of the author, there is no experiment or numerical study on the bearing capacity of strip footings rested on SMTs in the literature and the extent of this improvement on their performance has not been investigated, yet. There are just some tests (Hamed 1986) on the bearing capacity of strip footings on a saturated soft clay medium reinforced with a granular trench. Hamed (1986) found that the bearing capacity of strip footings is increased up to a factor of 2.6 by using a granular trench with dense sand which is reached at $W/B = 3$. The experimental results comparison with the theoretical limit analysis results presented by Madhav and Vitkar (1978) also showed that the theoretical bearing capacity values are 40%-70% higher than the corresponding test results. Nevertheless, it is apparent that the results of the studies conducted on the performance of soft clay strata reinforced with granular trenches cannot be generalized to those reinforced with SMTs because of their different mechanical behavior and properties.

In order to identify the structural behavior and performance of rubble stone masonry assemblages, some experimental studies have been conducted by other researchers which deserve to be investigated. Lourenço *et al*. (2005) performed an experimental research on the structural behavior of coursed dry joint stone masonry walls and observed that the stiffness of such walls increases with the normal stress exhibiting a nonlinear elastic behavior. However, the experimental tests conducted by Vasconcelos (2005) to specify the influence of the textural arrangement (bond) on the structural performance of granitic stone masonry walls revealed that the lowest value of the modulus of elasticity pertains to rubble masonry walls and it does not depend much on the applied normal stress. According to the compression tests conducted on small rubble stone masonry specimens (i.e three-course, single-leaf specimens) by Milosevic *et al*. (2013), it was found that the specimens' failure was due to stone crushing (by stone to stone contact) and the mortar quality had small effect on the specimens' ultimate strength. This is because, crushing of weak mortar among the masonry units causes the stones to contact with each other. Therefore, the compressive strength of a rubble stone masonry prism is governed by its constituent stones' compressive strength. However, as it has been pointed out by Milosevic *et al*. (2013), the influence of mortar quality may be significant in larger rubble stone masonry specimens. Hence, it seems that the influence of mortar quality might be probably considerable in simulating the real nonlinear behavior of SMTs with large thicknesses. Therefore, for the case of SMTs it may be concluded that the mortar would act as a paste connecting the irregular stone units aggregates to each other to form a monolithic and semi-rigid strip pad over the underlying weak soil.

The purpose of this paper is to investigate the efficiency and structural performance of rubble SMTs in improving the ultimate load-bearing capacity of strip footings on a soft clay soil stratum. To achieve this end, a 2D discrete element model (DEM) is developed using the specialized discrete element software UDEC (Itasca Consulting Group Inc. 2004) for the nonlinear static analysis of strip footings rested on a soft clay stratum reinforced with a rubble SMT subjected to normal loadings. This model is then used to perform a parametric study with varying SMT's width and depth to determine the effect of their aspect ratio on improving the bearing capacity of the footings.

2. DEM modeling of strip footings on a stone masonry trench

Depending on the level of accuracy, there are different modeling approaches in the literature to simulate the nonlinear behavior of such an assembly (i.e., a stone masonry trench and its surrounding soil medium). The first approach is the limit analysis (Chen 1975) using the upper-bound and lower-bound theorems of plasticity theory. This approach has been used successfully to predict the ultimate bearing capacity of strip footings on a granular trench (Madhav and Vitkar 1978 and Bouassida *et al*. 2014). Although, limit analysis estimates the bearing capacity, however, it cannot predict the displacement of the system at failure. The second approach is the finite element method (FEM) which is a continuum-based numerical method. In this approach, masonry trench units are modeled by elastic/inelastic continuum elements connected with zero-thickness interface finite elements to each other accounting for potential or slip planes. The surrounding soil can also be modeled as an inelastic continuum medium. The third approach is the discrete/distinct element method (DEM) which can be utilized as an alternative to the FEM to simulate the nonlinear behavior of masonry trenches rested on a soft soil. The DEM is a powerful method which has been developed by Cundall (1971) especially for the static and dynamic analysis of jointed discrete blocks. In the DEM, shear sliding, large displacements, joints openings and also automatic detection of new contact points are allowed as the analysis proceeds. In this method, it is possible to model the masonry units as either rigid or deformable units with elastic/inelastic behavior. The DEM has been used successfully in the literature to simulate the nonlinear behavior of masonry buildings (Mohebkhah and Sarv-Cheraghi 2014), masonry infilled frames (Mohebkhah *et al*. 2008, Mohebkhah and Sarhosis 2016, and Sarhosis *et al*. 2014), stone masonry colonnades (Sarhosis *et al*. 2016a,b) and a rubble stone masonry arch bridge (Tran *et al*. 2014) among others. Comprehensive investigation on the advantages and disadvantages of different DEM and FEM procedures available in the literature for numerical modeling of historic masonry structures has been recently reported in the book chapter by Asteris *et al*. (2015). In this paper, the DEM is utilized to simulate nonlinear behavior of strip footings on a SMT as described in the following subsections.

2.1 Geometry

In order to achieve the intended goal of the paper, a rough strip footing of width B (equal to 4 m) placed on the surface of a semi-infinite frictionless soil medium reinforced with a SMT of width W and depth D is considered as shown in Fig. 1(b). Because of the symmetry of the loading and geometry of the footing, only one-half of the system is modeled with the correct induced boundary conditions as shown in Fig. 1(c).

As it was pointed out earlier, SMTs are made with cement mortar of variable thickness between irregular stone pieces. However, in this study for the sake of modeling simplicity, the same sizes of rectangular stone units (500×400×200 mm) laid in running bond pattern are assumed for the structural bonds between the stone masonry units through the width of a SMT (plane strain condition). The stone masonry trench medium was modeled at a semi-detailed level in which the joints are simulated as interface elements of zero thickness.

2.2 Stone masonry trench material properties

In performing such an analysis, careful attention should be paid to the stone masonry modeling and to the material properties in order to assure the representation of real SMTs commonly used in practice. The stone masonry trench with limestone units was assumed to be inelastic isotropic material. Due to the lack of appropriate experimental data concerning the elastic and inelastic properties of rubble limestone masonry units, the compressive strength, modulus of elasticity and Poisson's ratio of 30 MPa, 30 GPa (Como 2013), and 0.2 were adopted for the limestone units, respectively. The unit weight of the limestone masonry trench with cement mortar was considered to be 27 kN/m^3. The limestone units are built using the Mohr-Coulomb material model. The internal friction angle of limestone units is taken to be 35°.

The interface's normal and shear stiffness parameters (i.e., k_n and k_s) can be estimated from the stiffness of the real joint under the assumption of stack bond and uniform stress distributions both in the unit and mortar. Lourenço *et al*. (2005) proposed the following equations to determine the joint normal and shear stiffnesses of dry joint stone masonry walls

$$k_n = \frac{1}{h_s\left(\frac{1}{E_{wall}} - \frac{1}{E_{stone}}\right)} \tag{4}$$

$$k_s = \frac{k_n}{2(1+\nu)} \tag{5}$$

where h_s = height of the stone unit; ν = Poisson's ratio of wall; and E_{wall} and E_{stone} are the Young's modulus of the wall and the stone unit, respectively. Although, the abovementioned relations have been given for dry joint masonry, however, it is believed that they can be used for rubble stone masonry units, as well. Because, as it was pointed out earlier, the weak cement mortar between the limestones in a SMT crushes under compression causing the stone units to contact with each other like the situation of a dry joint stone masonry wall. According to the experiments conducted by Vasconcelos (2005) on the compressive strength and stiffness of rubble stone masonry walls, it has been shown that the modulus of elasticity of such assemblages is approximately 0.12 times the modulus of elasticity of their constituent stone units (i.e., $E_{wall} \approx 0.12\ E_{stone}$). Therefore, the normal and shear stiffnesses of the stone masonry joints were calculated as 20.45 N/mm^3 and 8.52 N/mm^3, respectively. To simulate the nonlinear behavior of zero thickness interfaces (the SMT joints between the limestone units), a Mohr-Coulomb slip model is utilized. Considering a weak mortar between the stone units, the angle of internal friction and cohesion of these interfaces (φ_j, c_j) are taken to be 30° and 0.0 MPa, respectively.

2.3 The soft clay soil medium properties

A 10 m deep undrained clay soil stratum loaded by a 4 m wide perfectly rough and rigid strip footing is considered. The width and depth of the soil domain were taken to be 40 m and 10 m, respectively. It has been stated by Chen (1975) that a high value of Poission's ratio such as 0.48 has to be used to obtain a reasonable approximation for an incompressible soil elastic behavior under undrained conditions. The material properties utilized for the soil stratum is assumed as (Density = 15 kN/m^3, shear modulus (G) = 100 MPa, Bulk modulus (K) = 200 MPa, cohesion (c) = 10 kPa and internal friction angle (φ) = 0). The soil nonlinear behavior is modeled using the Mohr-

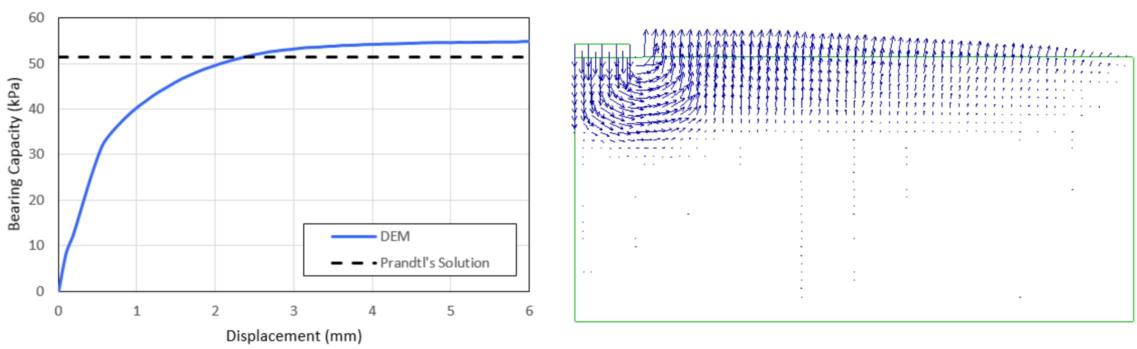
Coulomb material model. Deformable soil medium is subdivided into a mesh of finite-difference triangular elements. The formulation of these elements is similar to the constant strain triangle (CST) finite element formulation. The CST element is suitable for the analysis of plastic problems in plane strain conditions (Itasca Consulting Group Inc. 2004).

2.4 Verification of the modeling technique

It should be kept in mind that the DEM model developed here are highly idealized due to the considered assumptions for the soil stratum beneath the footing (e.g., elastic-perfectly plastic and homogeneous material) and regular rubble stones. However, it is believed that such a model can be used to obtain a rough insight and qualitative information to investigate the efficiency of using SMTs beneath the strip footings on a soft undrained clay soil (Tresca-type soil).

Due to the lack of experimental tests on the bearing capacity of strip footings on a SMT, the developed DEM model is utilized here to simulate the nonlinear behavior of a strip footing on a plain purely cohesive soil (i.e., without any SMT). The exact bearing capacity of this idealized punch indentation problem known as "*Prandtl's wedge problem*" is $(2+\pi)c$ = 51.4 kPa which has been derived by Prandtl in 1921 using a slip-line solution (Chen and Han 2007). To analyze the nonlinear behavior of such a system up to its collapse load, it is often better to use displacement-controlled boundary conditions rather than force-controlled. Therefore, the top boundary of the rigid footing was subjected to an incremental vertical displacement with a downward velocity of 1.0×10^{-3} m/sec. The numerically obtained load-displacement capacity curve of the problem is shown in Fig. 2(a) along with the exact theoretical *Prandtl's soultion*. As it can be seen, the agreement between the numerical and theoretical bearing capacities (i.e., 54.6 versus 51.4 kPa) is satisfactory with an error of 6.2%. Fig. 2(b) shows the velocity field of the soil stratum at the collapse load. As it is observed, a wedge beneath the rigid footing is identified which moves downward. This velocity field is consistent with the general shear failure mechanism considered by Prandtl to derive the exact solution.

This idealized example demonstrates the capability of the DEM to predict the ultimate load-bearing capacity of a strip footing and model plastic flow of a continuum soil stratum. The validity of the adopted procedure to model the nonlinear behavior of jointed SMT has been proved previously in the analysis of masonry walls and infill walls in the literature (Lourenço 1996, Mohebkhah *et al*. 2008 among them).



(a) Footing load-displacement curve up to failure (b) Velocity vectors at the collapse load

Fig. 2 DEM results of Prandtl's wedge problem

3. Parametric study

The developed DEM model in the previous section is utilized here to investigate the effects of a SMT of different relative aspect ratios (i.e., W/B and D/W) and the presence of a surcharge of 20 kPa on the load-bearing capacity of a rough surface strip footing resting on a soft clay. For this purpose, a parametric study program was conducted as shown in Table 1. For convenience, these models were assigned a specific symbol as DmWn where m and n stand for the trench width and height in meter, respectively. In this parametric study, the dimensions and properties of the strip footing and the chosen surrounding domain were kept constant as described in Section 2. The other purpose of this study is to determine the optimum aspect ratio of SMTs to reach the maximum value of the ultimate bearing capacity. To compare the models with each other quantitatively, a bearing capacity ratio (BCR) was defined as follows

$$BCR = \frac{q_{ut}}{q_u} \tag{6}$$

where q_{ut} and q_u are the ultimate bearing capacity of the strip footings with and without a SMT, respectively.

4. Results and discussions

The abovementioned rough strip footing models with different SMT aspect ratios were analyzed twice -with and without the presence of surcharge- (totally 42 models) to estimate the variation of their load-bearing capacity as the SMT aspect ratio changes. The obtained results are presented and discussed in the following subsections

4.1 Effect of SMT aspect ratio

Bearing capacity ratio variations of the considered footings without any surcharge against the dimensionless width ratio of the SMT (W/B) ranging from zero to 3.0 are shown in Fig. 3.

Table 1 Parametric study models

Model	D (m)	W (m)	W/B	D/B	Model	D (m)	W (m)	W/B	D/B
D0W0	1	0	0.0	0.0	D3W4	3	4	1.0	0.75
D1W4	1	4	1.0	0.25	D3W6	3	6	1.5	0.75
D1W6	1	6	1.5	0.25	D3W8	3	8	2.0	0.75
D1W8	1	8	2.0	0.25	D3W10	3	10	2.5	0.75
D1W10	1	10	2.5	0.25	D3W12	3	12	3.0	0.75
D1W12	1	12	3.0	0.25	D4W4	4	4	1.0	1.0
D2W4	2	4	1.0	0.5	D4W6	4	6	1.5	1.0
D2W6	2	6	1.5	0.5	D4W8	4	8	2.0	1.0
D2W8	2	8	2.0	0.5	D4W10	4	10	2.5	1.0
D2W10	2	10	2.5	0.5	D4W12	4	12	3.0	1.0
D2W12	2	12	3.0	0.5					

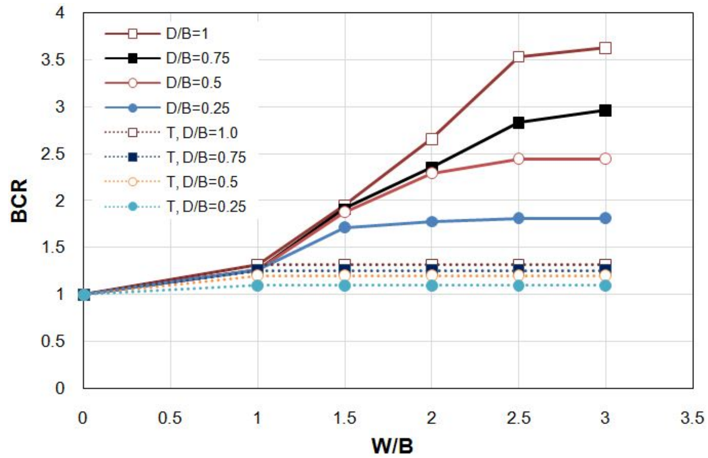


Fig. 3 DEM and theoretical bearing capacities of the strip footing versus W/B

For the cases with the trench width ratios larger than 1.0 it is seen that, the value of numerical BCR increases with ratio W/B up to a limit and then remains more or less constant for larger values of W/B. The maximum value of BCR is reached at a limiting value of $W/B \approx 1.5$, 2.0, 2.5 and 2.5 for the trench depth ratios (i.e., D/B) of 0.25, 0.5, 0.75 and 1.0, respectively. Although, the BCR value increases for some cases with W/B values greater than the abovementioned limiting (optimum) values, however, this increase is practically insignificant and can be considered constant. In addition it can be seen that the BCR value increases with the trench depth ratio D/B, as well. For example, for the optimum width ratio of 2.5, the BCR increases more than 3.5 times the one with no SMT as the trench depth ratio increases up to 1.0. Nevertheless, in order to preserve the economy of SMTs construction, it may be reasonable to use a SMT of $W/B = 2.0$ and $D/B = 0.5$ which can still lead to a BCR value of almost 2.3.

The theoretical BCR predictions ($q_u = 5.14c\lambda_{cd}$ for $D_f = D$) of the models -denoted by letter T- have also been plotted in Fig. 3 for comparison. It can be seen that while the theoretical values of BCR are close to the numerical ones for the models with $W/B = 1.0$, however, they are too conservative for the cases with $W/B > 1.0$. In other words, the theoretical depth factors (Eqs. (2)-(3)) cannot be used to include the shear resistance of the overburden soil for footings on SMTs of variable width ratio ($W/B > 1.0$).

Bearing capacity ratio variations of the considered footings against the dimensionless depth ratio of SMT (D/B) ranging from zero to 1.0 are also shown in Fig. 4. It is seen that for the optimum value of the width ratio (i.e., $W/B \geq 2.5$), the bearing capacity ratio increases almost linearly with D/B.

The obtained load-displacement curve of model $D2W8$ is shown in Fig. 5 along with the one for model $W0D0$ (footing with no SMT) for comparison. As it can be seen, inclusion of the SMT increases both stiffness and bearing capacity of the system considerably. In addition to the global load–displacement curve, a comparison in terms of the vertical displacement contour, velocity vectors, deformed geometry, cracking pattern and the failure mechanism is necessary to assess the behavior of such footings on a SMT. In Fig. 6, DEM qualitative results of model W2D8 at the collapse load (vertical displacement equal to 6 mm) are shown. From Figs. 6(a) and (c), it can be seen that the SMT undergoes some stepwise inclined cracks (in its top right corner). This means that despite the high elastic stiffness of SMTs, they do not perform as a monolithic pad beneath the

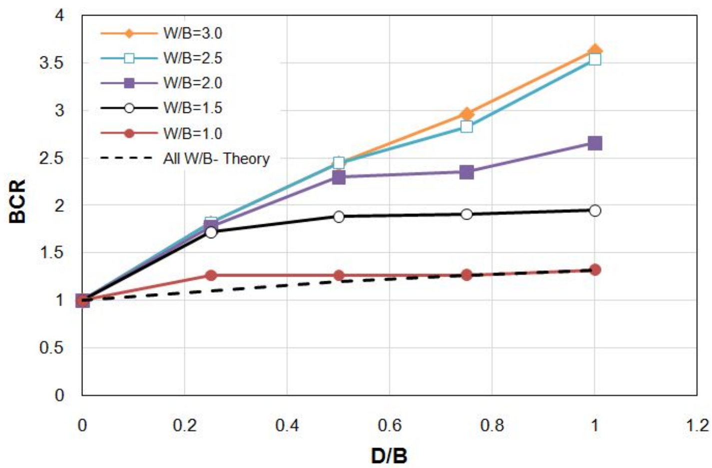


Fig. 4 DEM and theoretical bearing capacities of the strip footing versus D/B

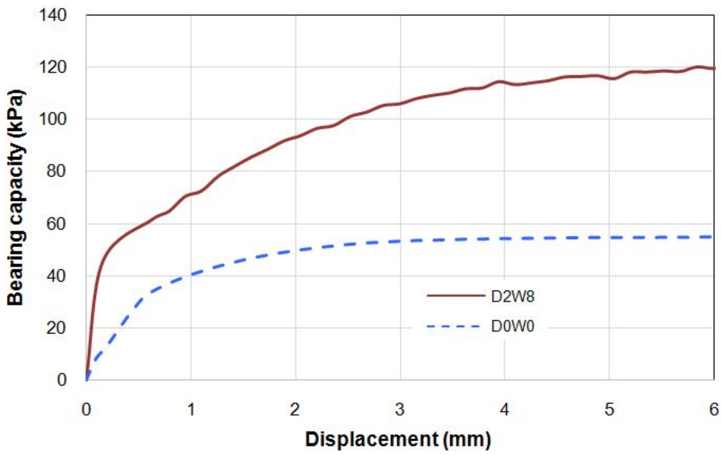
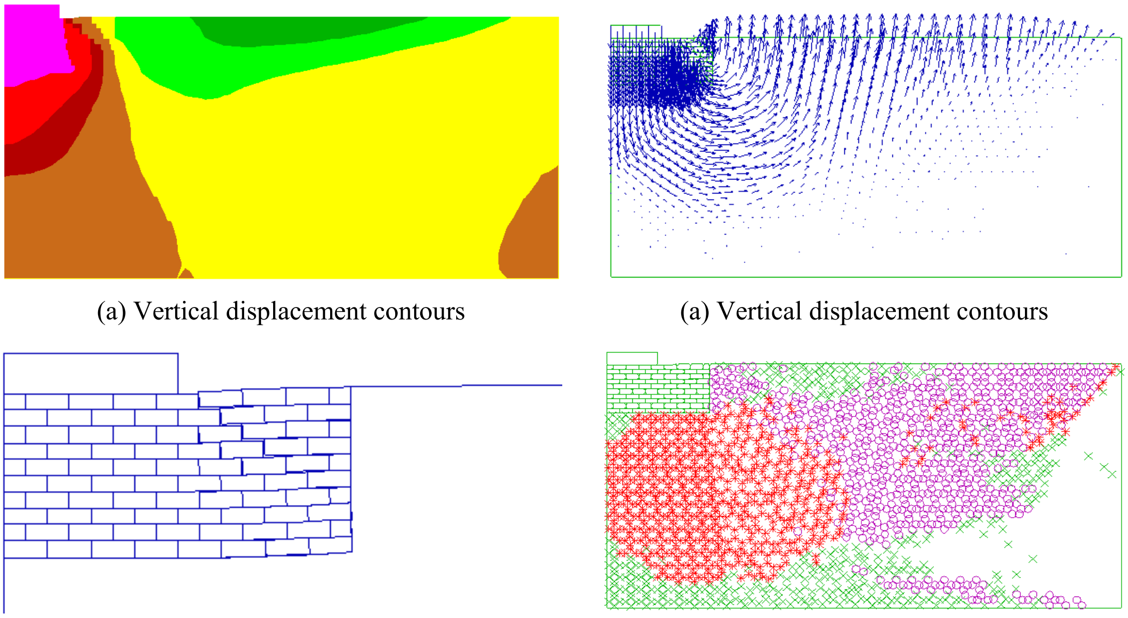


Fig. 5 Load-displacement curve for models $D0W0$ and $D2W8$

footings at the collapse load. Fig. 6(b) shows the velocity field of the domain beneath the footing (i.e., SMT and soil stratum). As it can be seen, in this case a larger wedge forms beneath the SMT moving downward. This indicates that in this case, a larger area of the soil stratum is affected due to the presence of SMT contributing to a high bearing capacity. Failure points in Fig. 6(d) show that in spite of the nonlinear behavior of SMT due to cracking, its constituent limestone units remain elastic with no failure. However, the surrounding soil medium undergoes plastic deformations indicating the formation of a deep and larger general shear failure mode in comparison with the one with no SMT (i.e., Fig. 2(b)).

Despite the considerable effect of SMTs in increasing the bearing capacity of such footings, it should be kept in mind that excessive deformations may also lead to damage or loss of function of superstructure. Therefore, the bearing capacity of such footings must be limited to the serviceability limit state that pertains to the definition of an allowable settlement as stated in Eurocode 7 (2004).



(a) Vertical displacement contours (a) Vertical displacement contours

(c) Deformed geometry (magnification factor = 10) (d) Failure points

Fig. 6 DEM results of model *W*2*D*8 at a vertical displacement of 6 mm

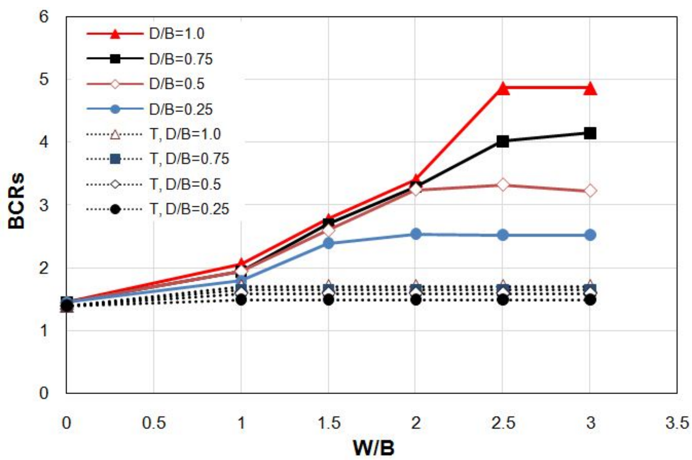


Fig. 7 DEM and theoretical bearing capacities of the strip footing with a surcharge versus *W*/*B*

4.2 Effect of surcharge

The models introduced in Table 1, are analyzed again in this part considering a uniformly distributed surcharge q = 20 kPa. Bearing capacity ratio (BCR_s) variations of the footings against the dimensionless width ratio of SMT (W/B) ranging from zero to 3.0 are shown in Fig. 7.

For the cases with the trench width ratios larger than 1.0 it is seen again that, the value of numerical BCRs increases with ratio W/B up to a limit and then remains more or less constant for larger values of W/B. The maximum value of BCR_s is reached again at a limiting value of $W/B \approx$ 1.5, 2.0, 2.5 and 2.5 for the trench depth ratios (D/B) of 0.25, 0.5, 0.75 and 1.0, respectively. Similar to the cases with no surcharge, in order to preserve the economy of SMT construction, it may be again reasonable to use a SMT of W/B = 2.0 and D/B = 0.5 which leads to the BCR_s value of more than 3.2.

The theoretical BCR_s predictions ($q_u = 5.14c\ \lambda_{cd} + q\lambda_{qd}$ for $D_f = D$) of the models -denoted by letter T- have also been plotted in Fig. 7 for comparison. It can be seen that while the theoretical values of BCR_s are close to the numerical one for the model with no SMT (W/B = 0), however, they are too conservative for the cases with W/B > 1.0. In fact, the theoretical depth factors (Eqs. (2)-(3)) cannot be used to include the effects of shear resistance of the overburden soil as well as surcharge for footings on SMTs of variable width ratio. The difference between the DEM and theoretical values of BCR_s can probably attributed to the fact that the effects of soil cohesion c (N_c) and surcharge q (N_c) are directly superimposed in Eq. (1) as suggested by Terzaghi (1943). Although, plasticity analyses has shown that this superposition is accurate for a frictionless soil (Vesić 1973), however, the numerical results presented here indicate that it does not hold for the bearing capacity of a soft clay reinforced by a SMT.

5. Conclusions

In this paper a 2D discrete element model developed for the inelastic nonlinear analysis of strip footings resting on a soft clay stratum reinforced by a rubble limestone masonry trench (SMT). The SMT was modeled using a micro-modeling strategy at a semi-detailed level in which the mortar joint is modeled as an interface with zero thickness. This strategy provides a better simulation of crack propagation and sliding in a SMT joints. The comparison of the numerical and theoretical analysis of a strip footing resting on a plain soft clay stratum indicates that the developed model can successfully predict the ultimate load bearing capacity and failure mechanism of the footing. The model was then used to perform a parametric study to investigate the influence of SMT dimensions and properties on the ultimate load-bearing capacity of strip footings on a clay reinforced with a limestone masonry trench.

The numerical parametric study showed that the use of SMTs beneath the strip footings increases the load-bearing capacity of soft clay strata. The optimum depth of a SMT to realize the maximum ultimate bearing capacity of the strip footings increases as the SMT depth increases. The increase in bearing capacity of the strip footings can be attributed to the increase in the overburden soil height. Nevertheless, it was found that the dimensionless depth factors in the general bearing capacity equation cannot be used to include the effect of shear resistance of overburden on the bearing capacity of strip footings on a SMT. The numerical results indicate that inclusion of a SMT of large dimensions in a soft clay can improve the bearing capacity of a strip footing up to a factor of 3.5. However, for economical considerations of SMTs construction, the dimensionless width and depth of W/B = 2.0 and D/B = 0.5, respectively, are proposed which can increase the bearing capacity up to a factor of 2.3.

Despite the obtained behavior of strip footings on a SMT by the developed DEM model in this study, it is obvious that these findings are not yet conclusive and further numerical and laboratory studies must be carried out to validate the predicted results. Furthermore, more numerical analyses with varying the soil cohesion is needed to reach a general conclusion.

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