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Prediction of settlement for vertically loaded strip footings on clay

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ABSTRACT: Accurate predictions of settlements are imperative for the efficient serviceability design of strip footings. Whilst the elastic response of strip footings on clay is understood, the pressure-settlement curves of strip footings are inherently non-linear and have been less explored. Despite field testing arguably being the best way to obtain a "real" non-linear pressure-settlement curve, this can be costly and site specific. Similarly, two- or three-dimensional numerical analysis can be time-consuming for early-stage design and require parameters that may be hard to calibrate. Alternatively, simplified analytical solutions can provide reasonable predictions of settlements with easy to calibrate soil parameters (e.g., from a standard triaxial test). Therefore, firstly, this paper investigates a similarity-based approach, akin to methods already employed for circular footings on clay. This approach assumes the pressure-settlement curve of a surface footing is similar in shape to a stress-strain curve of a representative soil element test. Similarity factors, to scale the soil element test to predict the pressure-settlement curve, are derived from elasticity assumptions. Secondly, a cone model approach is used to derive closed form non-linear expressions for the pressure-settlement response of strip footings. This model is combined with a hyperbolic non-linear simplified soil constitutive model. Finally, these simplified solutions are verified using the finite difference software FLAC 2D. As anticipated for strip footings, the solutions are dependent on the depth to bedrock.

KEYWORDS: Surface Footings, Settlement, Foundation Response, Plane-strain, Similarity, FLAC 2D.

1 INTRODUCTION

Strip footings are frequently used in geotechnical design to support linear structures, such as load bearing walls. The design of these foundations is often governed by footing settlement, based on serviceability limit state (SLS) design considerations. Therefore, accurate estimation of settlements under specific loading is imperative for efficient foundation design.

Approximate solutions in closed form are available for the settlement response of flexible rectangular footings on an elastic soil layer above a rigid base (Steinbrenner, 1934). In the presence of a rigid layer at a certain depth, this solution can be used for a strip footing of infinite length. It should be noted that this problem is more complex than that of a circular or square footing due to the solution being singular if no rigid base (i.e., a layer of rock) is found beneath the elastic material (Davis and Selvadurai, 1996). Despite this solution being limited to elastic material, Terzaghi in 1943 suggested it is "accurate enough for every practical purpose". However, it is well known that soil is inherently non-linear, and therefore, so too is the footing response. Obtaining an accurate estimate of the non-linear response of a strip footing is crucial in modern foundation design especially where settlements govern the design.

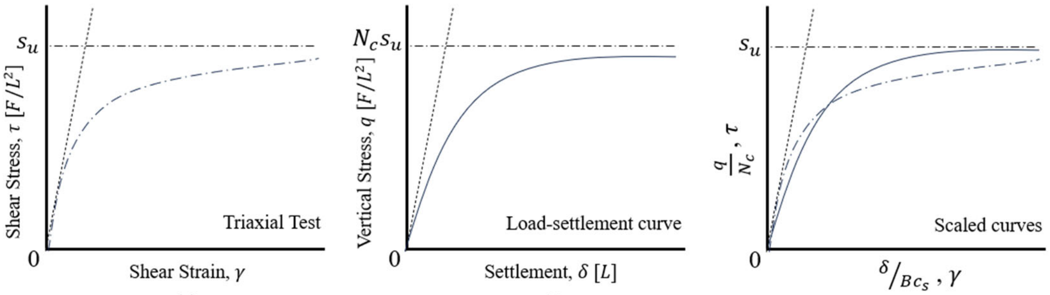
Arguably the best way to predict the non-linear settlement of the footing under a given load is using field testing; however, this approach is expensive and yields site specific results. Numerical approaches are becoming increasingly popular and are generally significantly cheaper, but often have hard to calibrate parameters (e.g., Raja et al., 2023). Alternatively, analytical or semi-analytical solutions use easy to calibrate parameters, are simple to employ in practice and have clear and easy to understand assumptions. This type of solution is particularly useful early in the design process to obtain a reasonable first estimate of the non-linear response and is, therefore, the focus of this work.

Existing non-linear pressure-settlement curves have been derived for circular footings. These derivations include (1) the use of a cone model approach to develop closed form solutions for pressure-settlement curves (Bateman et al., 2025); (2) a cavity expansion approach that uses an energy method to derive non-linear pressure-settlement curves (McMahon et al., 2013); and, (3) an energy method used to weigh the contributions of an elastic and an elastoplastic mobilisable strength design mechanism (Klar and Osman, 2008). However, none of these methods have been applied to strip footings.

An alternative, simplified approach to predict non-linear pressure-settlement curves directly from soil element tests is the similarity method, originally introduced by Skempton (1951) for circular footings. To employ this method, there is an assumption of similarity in shape between a stress-strain curve from a soil element test and a pressure-settlement curve of a strip footing. This would mean, a standard site-specific soil element test could be scaled linearly along both axes to directly predict a non-linear pressure-settlement curve. This approach was employed by Osman et al. (2007), Agaiby and Ahmed (2022) and Bateman et al. (2025) for circular footings. However, to use this approach for a strip footing, the linear transformation factors for each axis are, as yet, unknown. It should be noted that this method is approximate and accuracy in the results is not guaranteed; however, the benefits from such a simplified method based on a site-specific soil element test cannot be overstated.

Therefore, motivated by the lack of non-linear pressure-settlement curves for a strip footing and the evident benefits of the similarity approach, this paper:

- Applies the similarity method to strip footings and defines the required linear transformation factors.
- Examines the existing approximate elastic solution for strip footings and derives the corresponding elastic linear transformation factor.



(a) (b) (c)

Figure 1. (a) Idealised stress-strain curve from undrained triaxial test, (b) Idealised load-settlement curve of a foundation, (c) Normalised and scaled curves seeking similarity.

- Employs the cone model approach to derive closed form pressure-settlement curves using an elastic and a hyperbolic soil constitutive model. These are then used to determine similarity factors.
- Verifies the results with numerical analysis in FLAC 2D.

Notably, these solutions are all dependent on the depth to bedrock, as would be expected for a strip footing solution. An early version of this work can be found in Sabzevari (2024).

2 SIMILARITY METHOD

In 1951, Skempton proposed that similarity in shape exists between (1) a stress-strain curve from a soil element test on clay (e.g., Triaxial test) and (2) the pressure-settlement curve from a circular surface footing on the same clay. If this approach is applied to a strip foundation, the stress-strain curve from a soil element test can be "stretched" or "compressed" using two linear transformation factors on each axis, to directly predict the pressure-settlement curve (shown in Figure 1). The transformation factor of the y-axis can be readily obtained by comparing their respective ultimate capacities, which in this case is the undrained shear strength, s_u, and bearing capacity, q_u. For undrained clays, the bearing capacity can be given by $q_u = N_c s_u$ (where N_c is the bearing capacity factor equal to $N_c = 2 + \pi \approx 5.14$ for a strip footing, attributed to Prandtl, 1921). Therefore, the transformation factor of the y-axis is simply N_c. Secondly, the x-axis of the stress-strain curve should be scaled by a characteristic dimension, selected to be proportional to the foundation width, B. Therefore, the scaling of the x-axis can be expressed by:

$$\delta = \gamma c_s B \tag{1}$$

where δ is the foundation settlement, γ is the representative shear strain in the soil element test and c_s is the linear transformation factor.

If this similarity exists, the load-settlement curve for the strip footing can be obtained simply from a standard laboratory test. To employ this approach in design: (1) a soil element test must be undertaken on a representative soil sample, and (2) values for the normalisation factors, N_c and c_s must be selected. Once these are obtained, the following simple steps can be followed to predict the settlement under a certain load:

1. A known pressure acting on the foundation, q can be normalised by N_c to get an equivalent shear stress τ acting on the representative soil sample.
2. Given this shear stress τ, the representative shear strain γ can be obtained using the stress-strain curve from the soil element test.
3. The shear strain γ can be scaled using Eq. (1) to predict the foundation settlement.

Evidently, if the similarity between the two curves exists, this method would offer numerous advantages due to its simplicity in deriving non-linear pressure-settlement curves from site specific testing. Despite extensive use of the similarity method in various geotechnical engineering applications, no c_s values are available in the literature. Therefore, the focus of this work is on determining values of c_s and validating these against pertinent numerical analysis in FLAC 2D.

3 ELASTICITY

Steinbrenner (1934) provides a solution for the settlement at the corner of a rectangular flexible footing overlaying an elastic clay layer on bedrock for a given applied vertical pressure. The Steinbrenner approximation is based on calculating the difference between the vertical settlement at the surface and at a certain depth, to evaluate the influence of presence of rock at the specific depth. Therefore, the solution is dependent on the depth to bedrock, H, and becomes singular for a strip footing founded on an infinite domain. Despite being well referenced, the Steinbrenner (1934) paper is in a very obscure publication, making it one of the "most widely unread papers" in geotechnical engineering (Kausel 2010).

Superposition of this solution can be employed to predict the settlement at the centre of a strip footing (i.e. the maximum settlement) by dividing it into four equal sections. Therefore, the settlement at the centre of a flexible strip footing on homogenous soil of finite depth is given by (Terzaghi, 1943, note the correction to F_1 provided by Bozozuk 1963):

$$\frac{\delta}{B} = \left[\frac{I_p}{G(1+\nu)}\right] q \tag{2}$$

where G and ν are the elastic shear modulus and Poisson's ratio of the clay, respectively ($\nu = 0.5$ for undrained conditions) and I_p is the settlement influence factor, given by:

$$I_p = (1-\nu^2)F_1 + (1-\nu-2\nu^2)F_2 \tag{3a}$$

where F_1 and F_2 are given by:

$$F_1 = \frac{1}{\pi}\left[l \ln\left(\frac{\left(1+\sqrt{l^2+1}\right)\sqrt{l^2+h^2}}{l\left(1+\sqrt{l^2+h^2+1}\right)}\right) + \ln\left(\frac{\left(l+\sqrt{l^2+1}\right)\sqrt{1+h^2}}{l+\sqrt{l^2+h^2+1}}\right)\right] \tag{3b}$$

$$F_2 = \frac{h}{2\pi}\arctan\left(\frac{l}{h\sqrt{l^2+h^2+1}}\right) \tag{3c}$$

where $l = L/B$ is the shape footing factor (L is the length of the foundation, $l = \infty$ for strip footings) and $h = 2H/B$ is the depth factor (H is the depth to bedrock). For the limit $L/B \to \infty$ and $\nu = 0.5$, Eqs. (2) and (3) simplify to:

$$\frac{\delta}{B} = \left[\frac{1}{4\pi G}\ln\left(\left(\frac{2H}{B}\right)^2 + 1\right)\right] q \qquad (4)$$

The linear-elastic soil constitutive model can be normalised by the undrained shear strength, s_u:

$$\frac{\tau}{s_u} = \frac{G}{s_u}\gamma \qquad (5)$$

Following the similarity method logic (Eq. (1)), Eq. (4) can be normalised by the bearing capacity ($q_u = N_c s_u$) and compared with Eq. (5) to derive an analytical solution for the transformation factor c_s:

$$c_s = \frac{I_p N_c}{1+\nu} = \frac{2+\pi}{4\pi}\ln\left(\left(\frac{2H}{B}\right)^2 + 1\right) \qquad (6)$$

Since c_s is dependent on H, the depth to bedrock, Figure 2a illustrates the relationship between c_s and H/B. Using Figure 2a, one can read what factor is needed to multiply the shear strain to obtain δ/B. For example, where H/B equals 10 then $c_s \approx 2.5$, and a settlement of one hundredth of the footing breadth ($\delta = 0.01\,B$) would imply a representative shear strain of 4×10^{-3}. For comparison, Bateman et al. (2025) derived a transformation factor using similar assumptions for circular footings on a clay surface, which was equal to 1.2. Figure 2a shows a similar c_s value at $H/B \approx 2$.

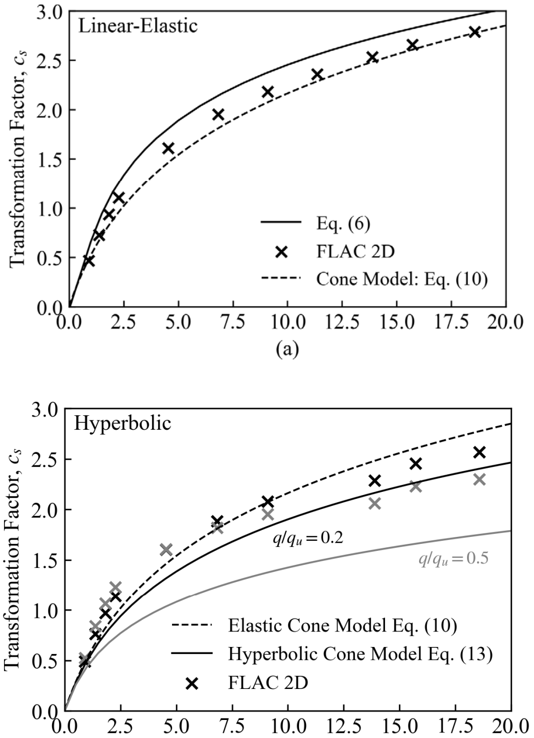


Figure 2. The change of the linear transformation factor c_s with the depth to bedrock ($m_{cone} = 0.3$)

This solution is based on applying a load to the surface of an elastic layer and, therefore, is applicable to a perfectly flexible footing. In reality, most footings will be essentially perfectly rigid, resulting in a stiffer response than that shown by this solution (yielding a lower c_s value). The authors are not aware of any solutions for rigid strip footings available in closed form.

4 NUMERICAL VALIDATION – ELASTIC RESPONSE

To validate the c_s values above, a strip footing founded on an elastic layer overlaying a rigid boundary was analysed using FLAC 2D (Itasca Consulting Group, 2011). The stiffness of the elastic pressure-settlement curves was found by applying a constant settlement rate to a rigid footing in plane-strain mode. Half the footing was analysed, assuming a line of symmetry is present along the centreline of the footing. The horizontal boundary was fixed in the vertical direction at a distance greater than twice the depth to the vertical boundary.

Comparison of stiffness of the pressure-settlement curve and the shear modulus, G, enables c_s to be determined (given $N_c = 2 + \pi$). These are plotted against the flexible footing solution in Figure 2a and, as expected, show lower c_s values (indicating a stiffer response).

5 CONE MODEL – NON-LINEAR RESPONSE

The elasticity solution yields a c_s relevant only to the elastic portion of the curve. However, the pressure-settlement curve will be non-linear by nature and understanding the full non-linear response is important to enable an efficient foundation design. Therefore, this section goes on to derive a non-linear solution based on a cone model method which can be calibrated based on the previous elastic solution.

The cone model is based on early work by Bowles (1997) and was further developed through dynamic analyses by Wolf (1994) and Wolf and Deeks (2005). This method, as employed here, extends the solution of Bateman et al. (2025) for a circular footing to a strip footing. The cone model relies on the assumption of an attenuation of normal stresses with depth beneath the footing based on the strength-of-materials assumption of "plane sections remain plane", shown in Figure 3. Although when employed in a plane-strain problem, this may be better described as a trapezoidal prism. Assuming the vertical stresses attenuate with depth such that the force remains constant but acts over a larger area, the vertical stress at depth z, q_z, can be given by:

$$q_z = \frac{qB}{B_z} = \frac{qB}{B + 2m_{cone}z} \qquad (7)$$

where B_z is the width of the cone at depth z and m_{cone} is the cone opening angle shown in Figure 3.

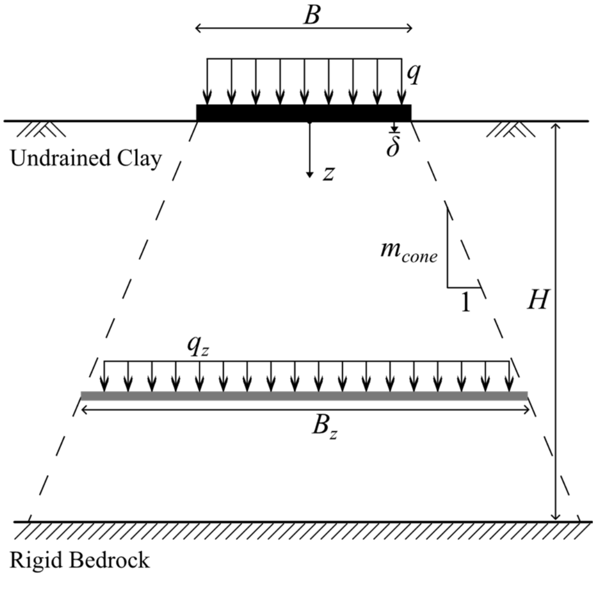


Figure 3. Illustration of the cone model setup

Using this model, the settlement of the footing, δ under an applied vertical normal pressure q can be calculated by integrating the vertical normal strain, ε, over the depth, z, between the footing and the layer of bedrock at depth H. This can be expressed in compact form as:

$$\delta = \int_0^H \varepsilon \, dz \tag{8}$$

The vertical normal strain in Eq. (8) can be obtained by selecting a soil constitutive model ($\sigma - \varepsilon$) to relate this strain with the stress in the soil, σ. The normalised vertical stress in the soil, $\sigma/(2s_u)$, can then be assumed to be equal to q_z/q_u, which is essentially a similarity assumption itself. This enables Eq. (7) to be transformed into the vertical strain required in Eq. (8). Any simplified soil constitutive model can be employed that can be given in flexibility form ($\varepsilon = \varepsilon(\sigma)$), including non-linear functions.

5.1 *Linear Elastic*

Firstly, the linear-elastic soil constitutive model is employed (provided in $\tau - \gamma$ form in Eq. (5)) to validate the cone model solution with previous sections and allow a specific cone opening angle m_{cone} to be calibrated. Given the constitutive model in Eq. (5):

1. The shear stress and strains can be converted into normal stresses and strains given $\tau/s_u = \sigma/2s_u$ and $\gamma = \varepsilon(1+\nu)$.
2. The normalised vertical stress $\sigma/2s_u$ can be set equal to q_z/q_u.
3. Eq. (7) can be substituted into this function and rearranged to yield ε as a function of q/q_u.
4. This expression can be input into Eq. (8) and solved to obtain a pressure-settlement curve (δ and a function of q).

Following these steps for a linear-elastic soil constitutive model yields the following elastic response curve:

$$\frac{\delta}{B} = \left[\frac{\ln\left(2\, m_{cone}\, \frac{H}{B} + 1\right)}{2\, m_{cone}\, \frac{G}{s_u}(1+\nu)}\right] \frac{q}{N_c s_u} \tag{9}$$

q/q_u in Eq. (9) can be set equal to τ/s_u in Eq. (5) and rearranged in the form of Eq. (1) to form an analytical solution for the transformation factor, c_s.

$$c_s = \frac{\ln\left(2\, m_{cone}\, \frac{H}{B} + 1\right)}{2\, m_{cone}\,(1+\nu)} \tag{10}$$

This shows the transformation factor is independent of load intensity and soil properties but remains dependent on the depth factor H/B as expected for a strip foundation. This solution is also dependent on the cone opening angle, m_{cone}. Wolf and Deeks (2004) suggest that $m_{cone} \approx 0.64$ for circular footings on undrained clay; however, this is likely unsuitable for a strip footing. Therefore, m_{cone} is numerically solved by comparing Eq. (10) from the cone model with Eq. (6) from the elasticity solution. This result is plotted in Figure 4 for varying H/B values. It can be seen that for $1 < H/B < 20$, m_{cone} slowly increases towards approximately 0.3. Since, the elastic solution pertains to a flexible footing, a value for m_{cone} that yields a stiffer solution is desirable (a lower c_s). Therefore, $m_{cone} = 0.3$ is selected to numerically evaluate the solutions. However, more work is required to fit m_{cone} specifically to a rigid solution and verify its use in a non-linear model.

Based on this value of m_{cone}, c_s from the elastic cone model (Eq. (10)) is plotted in Figure 2a against the previous solutions. This suggests a good fit of this model compared with the numerical results yet alternative non-linear constitutive models can be used.

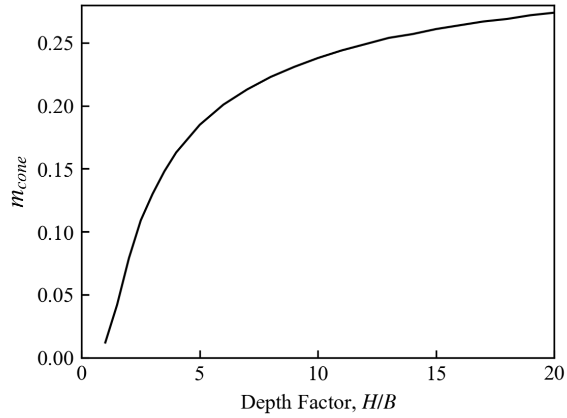


Figure 4. m_{cone} values fitted for an elastic material

5.2 *Hyperbolic*

Since a non-linear model is desirable, the commonly employed hyperbolic soil constitutive model is used in the cone model framework, given by (Kondner 1963):

$$\tau = \frac{s_u}{\frac{s_u}{G_0 \gamma} + 1} \tag{11}$$

where G_0 is the initial shear modulus of the clay.

To derive the pressure-settlement curve, the same steps as described in Section 5.1 are followed to obtain an expression relating applied stress with settlement of the strip footing:

$$\frac{\delta}{B} = \left[\frac{s_u}{2m_{cone}\, G_o(1+\nu)} \ln\left(\frac{2m_{cone}\left(\frac{H}{B}\right) - \frac{q}{q_u} + 1}{1 - \frac{q}{q_u}}\right)\right] \frac{q}{q_u} \tag{12}$$

Substituting Eq. (12) and Eq. (11) rearranged as a function of γ into (1) yields the following analytical solution for c_s:

$$c_s = \left[\frac{1 - \frac{q}{q_u}}{2\, m_{cone}\,(1+\nu)} \ln\left(\frac{2m_{cone}\left(\frac{H}{B}\right) - \frac{q}{q_u} + 1}{1 - \frac{q}{q_u}}\right)\right] \tag{13}$$

Despite this pressure-settlement curve being dependent on the soil properties, the transformation factor is independent. However, in the hyperbolic case, c_s is affected by load intensity, q/q_u. This means that perfect similarity does not exist for the hyperbolic model, a similar conclusion to that found for circular footings by Bateman et al. (2025). However, this does not necessarily mean the similarity approach is not useful but simply means the approximate loading range of the design is important. In geotechnical engineering, where safety factors are commonly in the range of 2 to 3, this means that $0.3 < q/q_u < 0.5$ would be of most interest. The relationship between q/q_u and c_s for the hyperbolic cone model solution is plotted in Figure 5 for different H/B values. Additionally, c_s for the hyperbolic model is plotted against H/B for $q/q_u = 0.2$ and 0.5 in Figure 2b.

Figure 5 shows that c_s is decreasing with increasing load intensity. In other words, as the load intensity increases, the pressure-settlement curve softens more than the stress-strain response. As the load intensity approaches 0, the curves match the elastic cone model value. Bateman et al. (2025) obtained a similar curve when applying the cone model to a hyperbolic soil constitutive model for a circular footing.

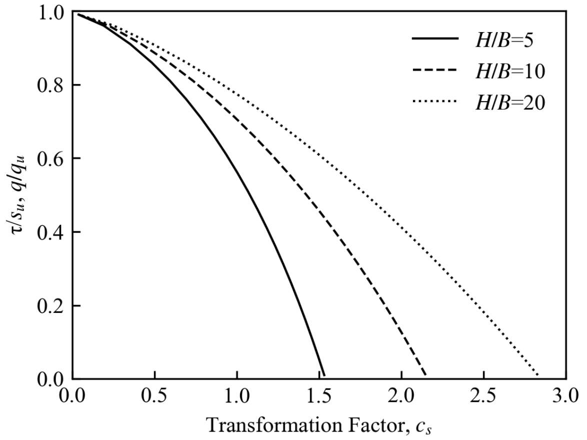


Figure 5. Transformation factor relationship with load intensity, derived from hyperbolic cone model Eq. (13) ($m_{cone} = 0.3$)

6 NUMERICAL VALIDATION – HYPERBOLIC

The FLAC analysis undertaken in Section 4 to validate the elastic cone model is extended here to a non-linear hyperbolic model (following Eq. (11)). The model is identical as that discussed above, except employs a CPPUDM (user defined) hyperbolic soil model. This model uses isotropic shear hardening with a Tresca yield surface that is defined based on the mobilised soil shear strength and the evolution of which is controlled by the hyperbolic function.

The c_s values derived from the hyperbolic FLAC model are plotted in Figure 2b. Positively, these follow a similar pattern to the cone model indicating the closed form function in Eq. (13) provides a good approximation of the c_s value.

7 CONCLUSIONS

Accurate prediction of non-linear settlement of strip footings is essential for efficient foundation design. Solutions for this problem must depend on the depth to a rigid layer (i.e., bedrock), since an infinite layer would result in infinite settlement. Simple analytical solutions are beneficial to the designer since they are easy to understand and implement. One such method is the similarity approach that can be used to predict a full, non-linear pressure-settlement curve of a strip footing from a stress-strain curve obtained from a standard soil element test. Evidently, if such a method is possible, the non-linear footing response can be predicted quickly and easily without the need for calibration of soil parameters - yet remains specific to the site at hand. However, transformation factors to employ the similarity method for strip footings are not available. Motivated by this gap in the knowledge, this paper:

- Applies the similarity method to strip footings. The transformation factors for the normalised x- and y-axes are defined as c_s and N_c (Eq. (1)), respectively. This process is illustrated in Figure 1.
- Investigates an approximate, closed form solutions proposed by Steinbrenner (1934) for strip footings on an elastic layer and a closed form expression for c_s is derived (Eq. (6)) that is dependent on depth to bedrock. However, this solution is limited to flexible footings. As an example, if H/B equals 10 then $c_s \approx 2.5$, and a settlement of one hundredth of the footing breadth ($\delta = 0.01B$) would imply a representative shear strain of about 4×10^{-3}.
- Compares this solution with a similar solution by Bateman et al. (2025) for a circular footing, which yields the same x-axis transformation when $H/B \approx 2$.
- Validates the elastic values of c_s using a model in FLAC 2D with a more realistic rigid footing. The obtained values are slightly lower than those from the flexible solution, as is expected for a stiffer result.
- Employs a cone model solution (Figure 3), firstly using an elastic soil constitutive model, to derive closed form expressions for c_s (Eq. (10)). This solution is dependent on the depth to bedrock and the cone opening angle, m_{cone}. By comparison with the previous elastic solution, m_{cone} is calibrated (shown in Figure 4) and selected as 0.3 considering the foundation rigidity.
- Extends the cone model solution to obtain a non-linear pressure-settlement curve using a hyperbolic soil constitutive model. This yields a closed form expression of c_s in Eq. (13) that is also dependent on the load intensity. This would imply that perfect similarity does not exist for this problem. Nevertheless, when a safety factor of the order of 2 to 3 is considered, a suitable c_s can still be selected.
- Validates the hyperbolic cone model solution using an identical soil constitutive model in FLAC 2D. This is shown in Figure 2b for two load intensities. This results in slightly higher c_s values than those predicted from the cone model, suggesting a wider deformed area under the footing predicted by FLAC.

8 ACKNOWLEDGEMENTS

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