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Punching shear coefficients for the design of working platforms

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ABSTRACT: Working platforms provide a safe working environment for the operation of tracked plant on sites with soft clay subgrades. The ultimate bearing capacity of such platforms is determined considering a layered soil model and assumes that collapse is governed by a punching shear mode of failure. The contribution of the platform to the bearing capacity is computed using a coefficient of punching shear which reflects the mobilised shearing resistance within the granular fill from which the platform is constructed. This paper investigates the punching shear coefficients used in computing the bearing capacity of layered soils. Finite Element Limit Analysis is used to obtain upper and lower bounds on the ultimate bearing capacity of layered soil system. Punching shear coefficients are obtained by back-calculating the ultimate bearing capacity of layered soils following the punching shear model by Meyerhof. The presented coefficients predict the actual ultimate bearing capacity of working platforms for a wide range of design conditions with 5 percent error or less.

1 INTRODUCTION

Crawler cranes and other tracked plant are more frequently being utilized in heavy engineering construction and in the development of infrastructure. Sites with poor subgrades are often incapable of supporting the loads associated with such equipment and temporary working platforms are required to provide a safe working environment. The size and capacity of construction equipment has also grown with advances in technology further emphasizing the need for reliable working platforms to not only protect expensive equipment but to mitigate risk associated often with critical construction activities.

A good practice guide to the design, construction and maintenance of ground-supported working platform BR-470 has been published by Building Research Establishment (2004) to improve safety on building sites. Although the guide has been successful in providing a reliable and safe method for platform design, some users have reported unnecessarily large platform thicknesses (Corke & Gannon 2010).

In BR-470, the ultimate capacity of the platform is based upon the punching shear mechanism proposed by Meyerhof (1974) for the bearing capacity of a sand layer overlying a weak clay subgrade. This failure mechanism assumes the footing punches vertically down through the sand to impose a bearing capacity failure within the clay for a footing of the same width. This is in contrast to the load spread model of (Terzaghi & Peck 1948) in which the sand

layer spreads the load and failure in the clay is that associated with a wider footing.

The contribution of the sand layer to the bearing capacity of the footing is obtained through the resistance of sand to shearing. Assuming vertical shear planes, Meyerhof (1974) incorporated shear resistance considering passive earth pressure acting on the failure planes and the average mobilised angle of shearing resistance (δ) along the planes. The mobilised angle of shearing resistance is a function of relative strength of the two layers and the peak internal friction angle of granular materials (Hanna & Meyerhof 1980). However, this dependency has been ignored by BR-470 as it adopts a mobilised friction angle of $\delta=2/3\phi$ following Meyerhof (1974).

This paper presents the results of a parametric study investigating the punching shear coefficient. The parametric study was performed using Finite Element Limit Analysis and considered a range of parameters encountered in the design of working platforms. The presented coefficients can be used to predict the actual ultimate bearing capacity of working platforms with 5 percent error or less.

2 BACKGROUND

2.1 *Punching shear failure in layered soils*

Meyerhof (1974) proposed a method to estimate

the ultimate bearing capacity of footings resting on a thin layer of dense sand overlying soft or very soft clay. From the results of experimental tests on layered soils, Meyerhof observed that where the thickness of dense sand is comparable to the width of footing, sand mass with an approximately pyramidal shape is punched into the soft clay as it is shown in Figure 1. In the case of general shear failure, the friction angle of the sand (ϕ) and the undrained cohesion of the clay (c_u) are mobilised. Meyerhof approximated the actual curved failure plains in the upper sand layer with a vertical plane through the footing edge. It was shown by Meyerhof that the total passive earth pressure (P_p) inclined at an average angle δ acting upwards on the vertical shear planes, well approximate the forces on the actual failure surface in the sand. Under this model, bearing capacity of the clay is that of a footing of equal width and as such, the ultimate bearing capacity of a strip footing resting on the surface is given as

$$q_u = c_u N_c + \frac{\gamma H^2}{B} K_p \tan \delta \quad (1)$$

where N_c is the bearing capacity factor equal to 5.14; γ the unit weight of the sand; H the thickness of the sand layer; B the footing width; K_p the coefficient of passive earth pressure and δ is the average mobilised friction angle on the assumed vertical failure plains. As a matter of convenience, Meyerhof replaced the term ($K_p \tan \delta$) by an equivalent term of ($K_s \tan \phi$) in which K_s is known as the Coefficient of Punching Shear.

$$K_p \tan \delta = K_s \tan \phi \quad (2)$$

Hence, the bearing capacity of the layered system is given by

$$q_u = c_u N_c + \frac{\gamma H^2}{B} K_s \tan \phi \quad (3)$$

Hanna & Meyerhof (1980) calculated values of K_s using limit equilibrium technique for a range of parameters and provided design charts of K_s as functions of δ/ϕ , c_u and ϕ . The ratio of mobilised friction angle (δ/ϕ) were in turn modelled as a function of the ratio of the bearing capacity of the sand and clay layers (q_c/q_s); where q_c is the ultimate bearing capacity of the footing over uniform clay and q_s is the ultimate bearing capacity of the same footing on a deep sand layer. The bearing capacity of a strip footing on a sand layer over soft clay, calculated by Equation 1 shows fairly good agreement with the results of model tests (Meyerhof 1974). However Equation 1 tends to underestimate the bearing capacity for large friction angles.

The coefficient of punching shear defined by Meyerhof is highly dependent on the mobilised fric-

tion angle of soil (δ) on assumed failure plains. Thus the relative strength of the two layers q_c/q_s , average confining stress in the sand layer and the peak friction angle of the top layer which influence δ can significantly affect K_s . Other parameters such as undrained shear strength of the cohesive layer (c_u), the footing width (B), and the unit weight of sand (γ) which contribute to the term q_c/q_s together with the thickness of the top layer, (H) that controls the average confining stress in the sand layer, are important factors that affect K_s .

Figure 1. Failure mechanism of strong sand layer underlain by soft clay after Meyerhof (1974).

2.2 BR-470 Bearing Capacity

The punching shear model of Meyerhof (1974) is used as the basis for design in BR-470 with the ultimate bearing capacity calculated according to the equation 1. Design values for the term $K_p \tan \delta$ are specified for given friction angles of platform material for which the average mobilised friction angle (δ) has been assumed to equal $2\phi/3$ as recommended by Meyerhof. The average value of δ was shown to be between $\phi/2$ to $3\phi/4$ (Meyerhof 1974). Table 1 lists design values of $K_p \tan \delta$ specified in BR-470 (refer Table A2 and Figure A4).

While BR-470 adopted the punching shear model for bearing capacity, it did not adopt Meyerhof's coefficient of punching shear in computing the bearing capacity equation, presumably to simplify the design calculations. Table 1 lists equivalent values of the coefficient of punching shear (K_s) computed using equation (2).

Table 1. BR-470 punching shear coefficients

ϕ	$K_p \tan \delta$	K_s
35	3.1	4.4
40	5.5	6.6
42.5	7.4	8.1
45	10.0	10.0
47.5	14.0	12.8
50	20.0	16.8

The application of BR-470 design approach is limited to subgrades with undrained shear strength greater than 20 kPa. The guideline recommends us-

ing more complex methods for the design of working platforms over very soft subgrades.

2.3 Finite element limit analysis

Finite Element Limit Analysis is a finite element technique based upon the limit theorems of plasticity. Unlike traditional displacement based finite element formulations, Finite Element Limit Analysis formulations, when properly implemented, provide rigorous upper and lower bound solutions that bound the true solution. The Finite Element Limit Analysis formulations implemented in this study are based on the work initially developed by Sloan (1988), further developed by Lyamin & Sloan (2002) and later Krabbenhoft *et al.* (2005) and Krabbenhoft *et al.* (2007) are implemented using the adaptive remeshing techniques of Sloan *et al.* (2013). Key features of these methods include the use of linear finite elements to model the stress or velocity fields, and collapsed solid elements at all inter-element boundaries to simulate stress or velocity discontinuities. The solutions from the lower bound formulation always yield a statically admissible stress field, while those from the upper bound formulation result in a kinematically admissible velocity field. This ensures that the solutions preserve the important bounding properties of the limit theorems. Furthermore, adaptive meshing allows improved bounds to be obtained using less computation power.

3 METHODOLOGY

The bearing capacity of a footing on a layered soil is found by application of Finite Element Limit Analysis to compute upper and lower bound estimates of the true bearing capacity. These rigorous bounds, which bracket the true bearing capacity of the footing, are computed using adaptive meshing techniques. The target solution accuracy and the refinement of the mesh have been set so that the computed bounds are within 5% of the true solution.

The soil is modeled as an associated Mohr-Coulomb material. The range of soil properties and geometries considered in the parametric study are summarised in Table 2 which reflect those considered in BR-470. The unit weights are not independent variables in the study and are correlated to the friction angle (Bolton 1986).

The coefficients of punching shear are back-calculated from the bearing capacity using Equation 3. The average of the upper and lower bounds on the bearing capacity was used in the back-calculation. Salgado *et al.* (2004) showed the upper and lower bounds of the ultimate bearing capacity of strip footing converge at a similar rate as the number of elements in the finite element mesh is increased. As such, the average bearing capacity may be consid-

ered as providing an accurate estimate of the true bearing capacity with an error of less than 5%.

Table 2. Soil properties and model dimensions

Property	Range	Values
Friction angle (ϕ)	45°-55°	45°, 47.5°, 50°, 55°
Undrained shear strength (c_u)	10 - 70 (kPa)	10, 15, 20, 25, 30, 35, 40, 50, 60, 70
Footing width (B)	0.6 - 1.8 (m)	0.6, 1.0, 1.4, 1.8
Sand thickness (H)	0.4 - 1.6 (m)	0.4, 0.8, 1.2, 1.6
Unit weight	17.5 - 20 (kN/m ³)	17.5, 18.1, 18.7, 20.0

4 RESULTS

4.1 Ultimate bearing capacity

Finite Element Limit Analysis results for a strip footing of width 0.6 m sitting on sand layer that is 0.8 m thick are plotted in Figure 2. The upper and lower bound estimates of the ultimate bearing capacity are presented for sand layers with friction angles (ϕ) of 47.5° and 50°. The plots show the bearing capacity of the footing has an almost linear variation with the undrained shear strength (c_u) of the clay sub-grade. The upper and lower results are in good agreement and provide tight bounds on the true collapse load. Figure 2 also compares results of Finite Element Limit Analysis to the ultimate bearing capacities calculated in accordance with BR-470. For the case in which $\phi = 47.5^\circ$, Figure 2a shows that BR-470 over-predicts the bearing capacity when the undrained shear strength of the clay is less than approximately 55 kPa. The over-prediction of the bearing capacity by BR-470 is more evident for platforms with higher friction angles. For a sand layer with a friction angle of 50°, Figure 2b clearly shows that bearing capacities calculated to BR-470 significantly over-estimate the bearing capacity of the layer system across the range of shear strengths relevant to the design of working platforms.

4.2 Coefficient of Punching Shear

The coefficients of punching shear (K_s) back-calculated from the finite element bearing capacities are plotted in Figure 3 for footing with B= 0.6 m and H= 0.8 m. Consistent with previous research showing K_s varies with the ratio of bearing capacities (q_c/q_s), the figure shows K_s as a function of the undrained shears strength (c_u) of the clay and friction angle (ϕ) of the sand layer. Figure 3 compares the back-calculated values of K_s to the equivalent shear coefficients (refer Table 1) based upon design to BR-470. While BR-470 is only applicable for c_u greater than 20 kPa, it is clear that the simplified design methodology provides a poor estimate of the contribution of punching shear to bearing capacity.

Figure 2. Ultimate bearing capacity of platform for a) $\phi = 47.5^\circ$ b) $\phi = 50^\circ$ ($H = 0.8$ m, $B = 0.6$ m)

The results of the parametric analysis show that the coefficient of punching shear varies very slightly by changing the footing width. Figure 4 plots coefficients of punching shear for cases in which $\phi = 50^\circ$. The figure clearly shows that coefficients back-calculated for footing widths of $B = 0.6$ m and 1.8 m are for all practical purposes equal.

Results of the entire parametric study are shown in Figure 5, which plots the coefficients of punching shear determined from the Finite Element Limit Analysis predictions of the bearing capacity. The coefficients have been computed using the average of the upper and lower bounds predictions of bearing capacity. As these bounds typically brackets the true solution with a relative error of less than 5%, bearing capacities computed using the coefficients of punching shear from Figure 5 are accurate to within a 5%.

Figure 3. Coefficients of punching shear for $B = 0.6$ m, $H = 0.8$ m.

Figure 4. Coefficients of Punching Shear K_s for $\phi = 50^\circ$.

Figure 5. Coefficient of punching shear.

Figure 6. (a) Power dissipation intensity and (b) velocity plot for $\phi = 50^\circ$, $c_u/\gamma H = 2.0$ ($B = 0.6$ m, $H = 0.8$ m, $c_u = 30$ kPa).

Figure 7. (a) Power dissipation intensity and (b) velocity plot $\phi = 45^\circ$, $c_u/\gamma H = 7.14$ ($B = 0.6$ m, $H = 0.4$ m, $c_u = 50$ kPa).

Figure 5, shows for friction angles of 50° , 47.5° and 55° that the coefficients of punching shear increase with increasing $c_u/\gamma H$. In these cases, collapse of the footing resembles the punching shear mechanism of Meyerhof (1974) with the block of soil directly beneath the footing moving vertically downwards. Figure 6 shows the power dissipation intensity and the velocities for a footing with $\phi = 50^\circ$ and $c_u/\gamma H = 2.0$ which clearly shows the punching failure mechanism with shear planes extending close to vertical below the edges of the footing. For $\phi = 45^\circ$, Figure 5 shows that coefficients of punching shear do not increase monotonically, instead reaching a maximum value at $c_u/\gamma H = 5$. In this case, the footing collapse is associated with a punching shear failure mechanism that involves shear planes that angle back under the footing. This is clearly shown in Figure 7 for the footing with $\phi = 45^\circ$ and $c_u/\gamma H = 7.14$.

The results presented in Figure 5 are based on the assumption that the materials used in the model follow the associative flow rule (normality rule) that is

not true for the sand in the top layer. The influence of using frictional soils in Limit Analysis on the limit loads has been discussed initially by (Davis 1968). It was concluded by Davis (1968) that such influence is not major unless the problem is strongly constrained in a kinematic sense. However most geotechnical collapse loads are not strongly constrained as they involve freely deforming ground surface and a semi-infinite domain Sloan (2013). This condition also applies to the existing problem. Davis (1968) has concluded that in such cases the Limit bounds can be considered as an acceptable estimation of the true limit loads (Sloan 2013).

5 SUMMARY AND CONCLUSION

The ultimate bearing capacity of temporary working platforms has been investigated using Finite Element Limit Analysis. A parametric study was conducted that considered a range of soil parameters and geometry typically encountered in the design of work-

ing platforms. Following the punching shear model proposed by Meyerhof (1974) for the bearing capacity of layered soils, the coefficient of punching shear was back-calculated from the limit analysis results.

Results show that for the range of parameters analyzed, the coefficient of punching shear is not dependent upon the footing width (B), at least for practical purposes, but is instead a function of the dimensionless parameter ($c_u/\gamma H$). Most previous studies have not recognized this dependency. Burd & Frydman (1997) identified that the bearing capacity of a layered soil is a function of ($c_u/\gamma H$) in considering a load spread model for bearing capacity of layered soils.

Finite Element Limit Analysis provided rigorous bounds that bracketed the true bearing capacity of layered soils to within 5%. Consequently, back-calculated values of the coefficient of punching shear can be used to accurately predict bearing capacity with less than 5% error. The results are summarised as a chart plotting the coefficient of punching shear as a function of friction angle of the sand, undrained cohesion of the clay and depth of the sand layer.

The punching shear model of Meyerhof (1974) assumes a simple mechanism in which the footing punches vertically through the sand layer. Results of Finite Element Limit Analysis showing the upper bound failure mechanisms are not presented in this paper. However, these show that the shear planes beneath the footing drift from vertical encroaching under the footing. Further research to investigate the mechanisms associated with the bearing capacity of the layered soils is being conducted.

Finally, the coefficients of punching shear adopted in the BR-470 guideline for the design of working platforms do not adequately model the variation of the coefficient. Results of Finite Element Limit Analysis showed that BR-470 over-predicts ultimate bearing capacities for sites with very soft clay subgrades. Also due to its simplistic nature, the coefficients provided in BR-470 may under-estimate the ultimate bearing capacity in other design conditions especially in lower sand friction angles and stiffer subgrades.

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