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Pull-out behaviour of steel grid soil reinforcement embedded in silty sand

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ABSTRACT

This paper investigates the pull-out behaviour (particularly the bearing resistance) of a steel grid reinforcement embedded in silty sand using laboratory tests and numerical analyses. It is demonstrated that the various common analytical equations for calculating the bearing component of pull-out resistance give a wide range of calculated values, up to about 200% disparity. The disparity will increase further if the issue of whether to use the peak or critical state friction angle is brought in. Furthermore, these equations suggest that the bearing resistance factor, N_q, is only a function of soil friction angle which is not consistent with some design guidelines. In this investigation, a series of large scale laboratory pull-out tests under different test pressures were conducted. The test results unambiguously confirmed that the N_q factor is a function of test pressure. A modified equation for calculating N_q is also proposed. To have more in-depth understanding of the pull-out behaviour, the tests were modelled numerically. The input parameters for the numerical analysis were obtained from laboratory triaxial tests. The analysis results were compared with the experimental results. Good agreement between experimental and numerical results was achieved if the strain-softening behaviour from peak strength to critical state condition was captured by the soil model used.

1. Introduction

There is a re-emerging interest in using steel grid or mesh type soil reinforcement in reinforced soil walls. For grid or mesh type soil reinforcements, the pull-out resistance, R_{pull}, is the sum of the frictional resistance developed along the longitudinal members, F_f, and the bearing resistance of the soil against the transverse members, F_b [1,2]. For an inextensible grid reinforcement, such as steel grid, F_b is the dominant contributor to R_{pull}, usually about 90% of R_{pull} [1]. This paper investigates the factors affecting F_b of a steel reinforcing grid subjected to pull-out forces.

F_b is usually calculated using a bearing capacity equation as below.

$$F_b = N_q \sigma_n A_b \tag{1}$$

Here, N_q is the bearing resistance factor, σ_n the stress relevant to the analytical expression for calculating N_q, A_b is the total bearing area. In Eq. (1), σ_n is usually replaced by the average vertical stress at reinforcement level, σ_{vo}, where averaging is taken over a representative length of the reinforcement. From a design point of view, σ_{vo} is the overburden stress at reinforcement level. Thus Eq. (1) becomes

$$F_b = N_q \sigma_{vo} A_b \tag{2}$$

The expression for N_q depends on the assumed failure mechanism. After Peterson and Anderson [2], the expression of N_q for the "so-called" general shear failure mode is as follows.

$$N_q = \tan^2\left(\frac{\pi}{4} + \frac{\phi}{2}\right) e^{\pi \tan\phi} \tag{3}$$

where ϕ is the friction angle of the soil mass. The above equation can be derived based on a bearing capacity equation for a shallow footing applied in the horizontal direction and thus σ_n in Eq. (1) may be argued to be equal to the horizontal stress, σ_h.

Jewell et al. [3] calculated the bearing resistance assuming a punching failure mode against the transverse bars of grid reinforcement, with σ_n being the vertical stress (σ_v) near the boundary of the punching mechanism. This yields the expression for N_q as below.

$$N_q = \tan\left(\frac{\pi}{4} + \frac{\phi}{2}\right) e^{\left(\frac{\pi}{2} + \phi\right) \tan\phi} \tag{4}$$

Matsui et al. [4] proposed an alternative expression for N_q based on a "hybrid" failure mechanism, as shown in Eq. (5) below.

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$$N_q = e^{\pi \tan\phi} \tan\left(\frac{\pi}{4}+\frac{\phi}{2}\right)\left[\cos\left\{\left(\frac{\pi}{4}-\frac{\phi}{2}\right)\right\}+(1-\sin\phi)\sin\left\{\left(\frac{\pi}{4}-\frac{\phi}{2}\right)\right\}\right] \tag{5}$$

Eqs. (3)–(5) intrinsically assume that there are no interference between transverse bars. Palmeira and Milligan [5] showed that the interference effect becomes negligible when the value of S/B is greater than 50, where S is the spacing between transverse bars and B is the bar diameter. At smaller value of S/B, interference will lead to a reduction in the "effective" N_q value – a finding also reported later by Bergado and co-workers [6]. Even if interference is neglected, the determination of F_b for design purpose is problematic because the calculated values of N_q from different equations (i.e., Eqs. (3)–(5)) differ significantly. For example, for $\phi = 35°$, the values of N_q varies from 8.85 to 33.30 which is a very large range. Furthermore, it is not clarified in the literature whether ϕ should be taken as the peak value (ϕ_p) or critical state value (ϕ_{cv}) and this induces approximately an extra 50% difference in the calculated N_q values, although one may argue that it should not be the peak value because of the large strain involved in bearing type failure.

Matsui et al. [4], by comparing the proposed equation with available pull-out test data from the literature, showed that the proposed equation can reasonably represent many published pull-out test results. However, it was not clear whether the ϕ values used in these comparisons were obtained in a consistent manner; and some of the reported ϕ values appears to be selected values [2,5,7–13]. Furthermore, it was not clarified whether ϕ_p or ϕ_{cv} were used in calculating N_q. On the other hand, design practice following the empirical study published by Christopher et al. [14] suggested that N_q varied with depth i.e., depends on σ_{vo}. A number of design guides, e.g., R57 [15] and AASHTO [16] also suggest that N_q depends on σ_{vo}. These extensively used design guidelines are not consistent with the implications of Eqs. (3)–(5) which suggest N_q is only a function of ϕ.

Numerical investigations of the pull-out behaviour of different reinforcing elements can be found in the literature. However, most of the investigations [17–20] were limited to various geosynthetic reinforcing elements. In fact, the F_b and its mobilisation, whether the reinforcement was extensible or steel, were represented by non-linear springs (linking the longitudinal bar and soil) characterised by parameters that were assumed to be known. These analyses focused on the progression of bearing failure from front to rear transverse bars (for geogrid) rather than modelling the bearing mechanism. One notable exception is the published work by Khedkar and Mandal [19] which modelled the bearing members of the grid reinforcement and their interaction with surrounding soil directly. The soil elements in the analysis were modelled using Mohr–Coulomb material model. However, the investigation was limited to test pressures of 75 and 100 kPa and the dependency of the N_q factor on σ_{vo} was not investigated.

In reality, stress–strain responses of dense sandy soil show post-peak strain-softening behaviour where the friction and dilation angle of the soil reduces with post-peak strain. Although this behaviour of sandy soil may have significant influence on the pull-out behaviour of the reinforcement, at the time conducting this study, the authors were not aware of any published literature that models such behaviour in the numerical analysis of pull-out testing.

The consideration of initial horizontal stress is another challenge. Numerical investigations on pull-out testing performed by Bayoumi et al. [17], in line with most published work, used at-rest earth pressure coefficient to initialize the horizontal stresses which was considered to be a constant. However, this coefficient may also vary with the test pressure due to the compaction effects and this may affect the pull-out behaviour. This also should be taken into account and its effect should be investigated.

The investigation in this paper is presented in two sections. In the first part, a series of large scale laboratory pull-out test results under a wide range of test pressures is discussed to investigate the behaviour of N_q under different σ_{vo}. The question of whether ϕ_p or ϕ_{cv} should be used in the calculation of N_q is also investigated. The second part of the paper discusses the numerical investigations. In view of the limitations of modelling the bearing mechanism, the focus of the numerical analysis was to investigate the mobilisation and strength of the bearing component (which accounts for about 90% of R_{pull}). The numerically computed results are compared with experimental results. The effect of varying initial earth pressure coefficient and strain-softening behaviour of the soil are also investigated in this section.

2. Experimental investigation

2.1. Tested soil

Well-graded silty sand from a construction site containing about 17% of non-plastic fines was used for this study. The particle size distribution (PSD) curve of the soil is presented in Fig. 1. The maximum dry density (MDD) and the optimum moisture content (OMC) of the soil determined from standard proctor tests were 1980 kg/m^3 and 9.58% respectively. A series of consolidated drained triaxial compression tests were conducted at different initial mean effective stresses (p'_0) ranging from 30 to 200 kPa. The triaxial test specimens were compacted to the same dry density (i.e., 95% of MDD) as of the pull-out tests. The samples were then saturated prior to shearing. This provided a family of stress–strain curves for the objective determination of the strength and deformation parameters. The measured ϕ_p reduced from 44.9° to 41.9° as p'_0 increased from 30 to 200 kPa, whereas ϕ_{cv} was 36.5°. The peak dilation angles, ψ_p were found to be in the range of 15° and 11.7° and showed a similar decreasing trend as of ϕ_p against p'_0. Further synthesis of the triaxial test results will be discussed in a later section on numerical modelling.

2.2. Reinforcing element

A galvanised steel grid with four longitudinal bars and five transverse bars was used as the reinforcement specimen. The diameters of longitudinal and transverse bars were 9.015 mm and 6.35 mm respectively. Additional details of the reinforcing grid are presented in Fig. 2. A thread of about 170 mm was made at the

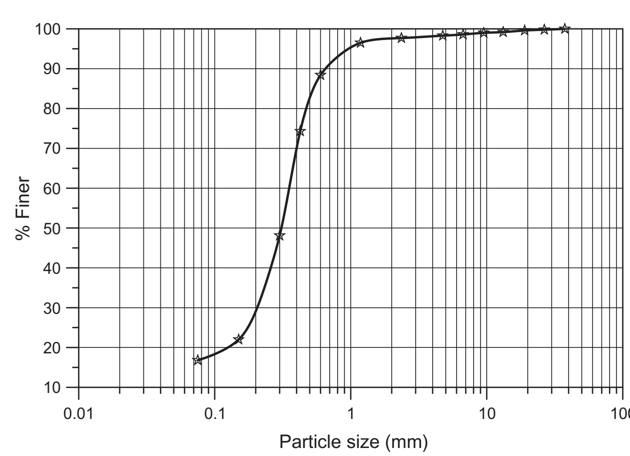

Fig. 1. Particle size distribution curve of the tested soil.