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Shear load transfer for rock-socketed drilled shafts based on borehole roughness and geological strength index (GSI)

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Abstract

The load distribution and deformation of rock-socketed drilled shafts subjected to axial loads were evaluated by a load-transfer approach. The emphasis was on quantifying the shear load-transfer characteristics of rock-socketed drilled shafts based on constant normal stiffness (CNS) direct shear tests performed by varying major factors influencing shaft resistance, including unconfined compressive strength, borehole roughness, initial confining stress, pile diameter, and material properties. Based on the CNS tests and the Hoek–Brown failure criterion, a nonlinear triple curve is proposed for the shear load-transfer function of rock-socketed drilled shafts. It is presented in terms of borehole roughness and the geological strength index (GSI) so that structural discontinuity and surface conditions of the rock mass can be considered. The proposed function was verified by the load test results of ten rock-socketed drilled shafts subjected to axial loads. Seven piles were constructed in completely or moderately weathered rocks of granite-gneiss, and the others were constructed in slightly weathered rocks of clayshale-limestone. Through comparisons with results of load tests, it was found that the shear load-transfer function in the present study is in good agreement with the general trend observed by *in situ* measurements, and this represents a significant improvement in the prediction of drilled shaft shear behavior.

Keywords: Rock-socketed drilled shaft; Load transfer; Shaft resistance; Constant normal stiffness; Geological strength index; Borehole roughness

1. Introduction

The current design methods for rock-socketed drilled shafts are mainly based on local knowledge derived from the observation of load tests, or empirical methods related to the unconfined compressive strength (UCS) of intact rocks. However, it is known that this design approach for piles is generally overly conservative by as much as an order of magnitude [1].

According to studies by Reese and O'Neill [2] and Ghionna et al. [3], the bearing capacity of rock-socketed drilled shafts should be determined by a serviceability limit capacity within the limit of allowable superstructure settlements rather than by ultimate bearing capacity. In addition, because the ultimate shaft resistance is generally mobilized at smaller interface displacements between the shaft and surrounding rock than ultimate toe resistance, piles typically carry most of their working load in shaft resistance. Therefore, for optimum designs of rock-socketed drilled shafts, predicting the shear load transfer from the pile into the surrounding soil or rock is as important as, or possibly more critical than, predicting the ultimate bearing capacity.

The bearing capacity and shear load-movement performance of rock-socketed drilled shafts are critically dependent on construction details and installation geometry conditions. Comprehensive studies of the details have been reported by Horvath et al. [4], O'Neill et al. [5], and Seidel and Collingwood [6]. They report that the shear behavior of rock-socketed drilled shafts is highly influenced by the following parameters: rock strength (drained intact and

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residual strength parameters are generally used), borehole roughness, rock mass modulus and Poisson's ratio, discontinuity structure and surface condition of the rock mass, pile diameter, initial normal stress between concrete and rock prior to loading, and construction practices. However, it is difficult to determine reliably, based on empirical methods, the interaction between the above mentioned factors in calculating the performance of a socketed pile due to the complexity of the interaction. A conservative approach to design is therefore pursued.

Although a few shear-load transfer functions have been proposed for rock-socketed drilled shafts [7–9], it appears that none of the methods can reliably predict shear behavior for overall rock types to a satisfactory level of accuracy due to the intrinsic characteristics of individual rock types, oversimplification of mechanisms, or subjective empirical input data. Moreover, because study test sites are mainly sandstone, limestone, and clayshale, less is known about the behavior of drilled shafts in weathered granite-gneiss, which occupies two-thirds of the total land area of the Korean peninsula.

As a consequence, various influencing factors of the pile–rock interface, including the degree of weathering and rock type, should be quantified and taken into account when designing rock-socketed drilled shafts. Toward this end, a theoretical methodology has been developed to provide a basis for a load-transfer criterion that would be applicable to drilled shafts installed in rocks. The validity of this study was tested through field case studies.

2. Shear behavior of rock-socketed drilled shafts

The load transfer from a pile to the surrounding soil ought to be considered in order to achieve structural compatibility between loads and deformation. Basically, the load-transfer method [10] and the continuum approach method [11,12] are used to calculate the load-deformation behavior of piles subjected to an axial load. Among them, the load-transfer method follows a simple analytical procedure and can be applied to any complex composition of soil layers with a nonlinear stress–strain relationship, inhomogeneous medium, and any variation in the sections along a pile. This method calculates the load–deformation relationship for drilled shafts on the basis of a load-transfer function utilizing subgrade reactions of the soil/rock surrounding the shaft. The soil is modeled by a set of localized springs, which are defined as a function of displacements at several discrete points along the pile including the pile tip: the unit shaft resistance vs local shaft displacement relations (the *t–z* or *f–w* curves) and the unit toe resistance vs local toe displacement relation (the *q–z* or *q–w* curve).

Several techniques are available for predicting the *t–z* (or *f–w*) curves in soils [13–15] and rocks [7–9]. In all of these methods, the *t–z* curves are expressed by elastic–plastic models having great differences in initial slope, although the ultimate value f_{max} is obtained in the same way as it would be for shaft friction in pile bearing capacity computations.

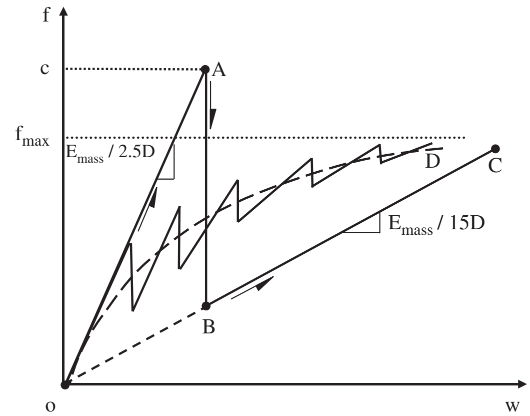

Fig. 1. Potential *t–z* relations for rock [8].

O'Neill and Hassan [8] suggested potential *t–z* behavior in rock, as shown in Fig. 1. If the pile–rock interface is clean so that the cement paste bonds to the rock, the roughness pattern is regular, and the asperities are rigid, a *t–z* relation such as OABC can be obtained. In most cases, however, the interface asperity pattern is not regular due to some degree of smear; in addition, asperities are deformable, which results in ductile, progressive failure among asperities. Therefore, they proposed an interim criterion for a hyperbolic *t–z* model in most rock types as described below until better solutions become available:

$$f = \frac{w}{(2.5D/E_m) + (w/f_{max})}, \quad (1)$$

where w is the pile movement, f_{max} the maximum unit friction, D the pile diameter, and E_m the effective Young's modulus of the rock mass.

However, Johnston [16] warns that methods such as the *t–z* curves elicit criticism because they do not explicitly consider failure mechanisms, random asperity patterns, rock stiffness, and effects of interface dilation on normal stresses.

In engineering practice, the pile–rock interface of drilled shafts consists of irregular asperities of varying heights and patterns, and hence various failure modes can be expected to occur, possibly simultaneously. For a pile–rock interface, shearing results in dilation as one asperity overrides another. If the surrounding rock mass is unable to deform sufficiently, an inevitable increase in the normal stress, $\Delta\sigma_n$, occurs during shearing. Because of the increase in stress normal to the interface produced by the constant normal stiffness (CNS) boundary condition, the frictional resistance between pile and rock increases. As a result, the shear behavior of rock-socketed drilled shafts can be modeled better under CNS conditions than constant normal load (CNL) conditions.

In order to take into account the effects of a CNS condition at the interface of a rock-socketed pile or rock joint, much research have been carried out on the shear