Soil Dynamics and Earthquake Engineering 78 (2015) 154–174

Contents lists available at ScienceDirect

Soil Dynamics and Earthquake Engineering

journal homepage: www.elsevier.com/locate/soildyn

Generalized failure envelope for caisson foundations in cohesive soil: Static and dynamic loading

Nikos Gerolymos [a,*], Athanasios Zafeirakos [a], Konstantinos Karapiperis [b]

[a] National Technical University, Athens, Greece
[b] University of Davis, CA, United States

ARTICLE INFO

Article history:
Received 10 February 2015
Received in revised form
23 July 2015
Accepted 25 July 2015
Available online 25 August 2015

Keywords:
Caisson foundation
3D finite element analysis
Cohesive soil
Failure envelope
Upper bound solution
Nonlinear interface
Plastic potential function
Dynamic loading

ABSTRACT

The response of massive caisson foundations to combined vertical (N), horizontal (Q) and moment (M) loading is investigated parametrically by a series of three-dimensional finite element analyses. The study considers foundations in cohesive soil, with due consideration to the caisson-soil contact interface conditions. The ultimate limit states are presented by failure envelopes in dimensionless and normalized forms and the effects of the embedment ratio, vertical load and interface friction on the bearing capacity are studied in detail. Particular emphasis is given on the physical and geometrical interpretation of the kinematic mechanisms that accompany failure, with respect to the loading ratio M/Q. Exploiting the numerical results, analytical expressions are derived for the capacities under pure horizontal, moment and vertical loading, for certain conditions. For the case of fully bonded interface conditions, comparison is given with upper bound limit equilibrium solutions based on Brinch Hansen theory for the ultimate lateral soil reaction. A generalized closed-form expression for the failure envelope in M–Q–N space is then proposed and validated for all cases examined. It is shown that the incremental displacement vector of the caisson at failure follows an associated flow rule, with respect to the envelope, irrespective of: (a) the caisson geometry, and (b) the interface conditions. A simplified geometrical explanation and physical interpretation of the associativity in M-Q load space is also provided. Finally, the derived failure envelope is validated against low (0.67 Hz) and high frequency (5 Hz) dynamic loading tests and the role of radiation damping on the response of the caisson at near failure conditions is unraveled.

1. Introduction

Large caissons are often employed as foundations for long bridges spanning over land or water. Though primarily used to transfer vertical loads safely into soft ground, caissons may also be subjected to significant, statically or dynamically imposed, horizontal and overturning moment loads from severe environmental or seismic events [54–56]. The response under these loads involves complex three-dimensional stress distribution along the caisson shaft and shear stresses at the circumference [1–5]. Moreover, due to their massive dimensions, as they deform they tend to mobilize a significant amount of the surrounding soil mass, mainly in the form of active and passive wedges, depending on the loading magnitude, the nature of the soil and the caisson-soil interface conditions.

Currently, the State-of-the-Art in foundation design involves expressing the ultimate limit states of a foundation subjected to combined N-Q-M loading by three-dimensional failure envelopes, or bearing strength surfaces e.g. [6–11,52]. The advantages of this method of analysis over the classical bearing capacity solutions have been thoroughly advocated by Gottardi and Butterfield [9], Gourvenec and Randolph [11], and Gourvenec and Barnett [12] among others.

For the determination of N–Q–M failure envelopes, three main approaches are followed: experimental, analytical and numerical. Owing to the contemporary advances in technical computing, the numerical finite element methods for defining the failure envelope gain increasing popularity. Interestingly, although notable literature exists regarding the combined capacity of foundations, it exclusively refers to shallow and skirted foundations, with embedment ratios (embedment depth, D, to foundation width, B) $D/B \leq 1$ e.g. [12–29]. Moreover, due to the complexity of the shape of the N–Q–M failure envelopes, stemming from the dependence on the foundation geometry, soil profile, embedment depth and soil-foundation interface conditions, few of these studies propose a closed-form representation of the envelopes for use in routine design. For deeply embedded foundations, with $D/B \geq 1$, the majority of solutions still rely on simple horizontal reaction concepts, developed for piles e.g. [30–32]. To the best of authors' knowledge, a three-dimensional failure envelope for deeply embedded foundations, with $D/B \geq 1$, is missing from the literature.

* Corresponding author. Tel.: +30 210 7724211; fax: +30 772 2405.
E-mail address: gerolymos@gmail.com (N. Gerolymos).

http://dx.doi.org/10.1016/j.soildyn.2015.07.012
0267-7261/

To this end, the novelty of present study lies in the thorough investigation of the response of deeply embedded (cuboid) foundations ($1 \leq D/B \leq 3$) under combined N–Q–M loading, by means of an extensive series of finite element analyses in 3-D. The analysis is performed for foundations in cohesive soil, with due consideration to material (soil) and soil-foundation interface nonlinearities. A generalized failure criterion (bearing strength surface) in the N–Q–M space is then proposed, along with closed–form expressions for the ultimate uniaxial capacities. For the case of fully bonded interface conditions, the prediction of the finite element results is verified against upper bound limit equilibrium solution based on Hansen [30] theory for the ultimate lateral soil reaction [30]. Furthermore, the paper provides a detailed analysis of the kinematic mechanisms that accompany failure of the foundation under combined loading. It is shown that the incremental displacement vector of the caisson head at near failure and post-failure conditions, follows a potential function that is associative to the derived failure surface (associated plastic flow rule). Finally, the response of a massless caisson with $D/B=1$ to externally applied dynamic loading, is also investigated, and the resulting load paths with respect to the top of the caisson are contrasted with the corresponding failure envelopes for static loading.

The findings of this study could also be used to support a capacity-based seismic design methodology for caisson supported bridges, as the one recently proposed by the first two authors [52]. The latter is based on a holistic approach of the bridge-caisson interaction, targeting to an optimum foundation solution that weighs performance and cost effectiveness. An integral component of this methodology is the determination of a failure envelope and plastic flow rule in the N–Q–M space.

2. Numerical modeling

For the analyses, it was assumed that the caissons were simply “wished into place” and the construction process was not simulated. Although relevant studies revealed that, following this assumption, the soil softening phenomena associated with the installation processes (e.g. [33,34]) are neglected, appropriate modifications in the soil strength and stiffness profile can be made to account for these effects when applying the results of this study. The numerical investigation involved finite element analyses [29,35] with the commercially available finite element code ABAQUS version 6.9 [36].

2.1. Caisson model

The study considers cuboid caissons of width B and embedment depth D (Fig. 1). The width is kept constant $B=10$ m in all analyses, whereas the embedment depth varies parametrically, so that three embedment ratios are obtained, namely $D/B=1$, 2 and 3. The caisson material is modeled as isotropic linear elastic with Young's modulus E_c=of 3×10^8 kPa (practically rigid), Poisson's ratio $\nu_c=0.15$ and specific unit weight of $\gamma=20$ kN/m^3.

2.2. Soil behavior

For the total stress analysis (undrained loading conditions), soil behavior is described through an elasto-plastic constitutive model [1], which is a reformulation of that originally developed by Armstrong and Frederick [37]. The model is available in the material library of ABAQUS. It uses the Von Mises failure criterion along with a nonlinear combined kinematic-isotropic hardening law and an associative plastic flow rule, capable of capturing the nonlinear soil response from the very early stages of loading. This study considers a uniform shear stress soil profile of $S_u=50$ kPa. The soil has a Shear modulus of $G_0=50$ MPa, leading to a rigidity index (at very small shear strains) of $G_0/S_u=1000$. A Poisson's ratio of $\nu \approx 0.5$ ($=0.475$ to avoid numerical instabilities) was assumed for the soil to approximate the constant volume response of the soil under undrained conditions. The cohesive soil conditions are also schematically illustrated in Fig. 2a.

The authors in [38] validated the constitutive model employed herein for the nonlinear response of embedded and caisson foundations against field tests of suction caissons [39]. From the validation process, it was deduced that the model is capable of capturing the response observed in the trials with satisfactory agreement, in both small and large amplitude lateral loading, including the development of caisson-soil interface nonlinearities (gapping).

2.3. Contact modeling

Throughout relevant literature, the interface between deep foundations and the surrounding soil is treated mainly as “fully-bonded”, thus preventing the development of any possible geometric nonlinearities (e.g. gapping and sliding along the interfaces). In this study, however, the caisson is connected to the soil with special contact surfaces (using the “Master-Slave” option), allowing for realistic simulation of possible detachment and sliding at the soil-caisson interfaces. The surface-to-surface contact interaction was modeled by exponential (“softened”) pressure-overclosure relationship through the direct constraint enforcement method that makes use of Lagrange multipliers. The slippage of the caissons along the interfaces is governed by Coulomb's friction law by appropriately assigning a friction coefficient μ [36] and assuming drained response for the interface.

2.4. Modeling Issues

2.4.1. Static loading conditions

A sensitivity analysis was carried out with respect to the extent of the FE field involving two width × ength ($L \times L$) configurations, for all D/B cases examined, namely $5B \times 5B$ and $10B \times 10B$, while the distance from caisson base to the bottom of the mesh was kept equal to $2B$. The analysis confirmed the already known from the literature, that under monotonic loading the lateral boundaries can be placed fairly close to the foundation (just outside the “pressure bulb”). Indeed, as shown in Fig. 3, the error in the computation of bearing capacity for a $D/B=3$ caisson subjected to pure horizontal displacement at its top up to failure, is less than 5% irrespective of the applied axial load. Therefore, with due consideration to the computational cost, a finite element mesh size of $5B \times 5B$ was adopted for all numerical analyses. A typical finite element mesh used in this study is shown in Fig. 1. It represents a half-caisson cut through one of the orthogonal planes of symmetry for $D/B=1$, and consists of approximately 12500 8-noded full integration (C3D8) brick elements.

The influence of relative stiffness ratio between the caisson and surrounding soil, E_c/E (where E_c and E are the elasticity modulus of the caisson and initial–at extremely small strains elasticity modulus of soil, respectively), on caisson's response, was also parametrically investigated by considering three different values of E_c and keeping E constant. It is observed in Fig. 2b that, while for small stiffness ratios (e.g. foundation in hard soil or rock) there is a considerable divergence in the response at displacement smaller than 0.30 m, the ultimate resistance is practically unaffected by the rigidity of caisson's material.

2.4.2. Dynamic loading conditions

The location and type of the lateral boundaries is a significant issue in dynamic modeling. Under transient loading waves emanating from the caisson–soil interface cannot propagate to infinity unless special absorbent boundaries (or elements) are placed at suitably large