Bedrock characterization and design considerations for rock socketed caissons in the Greater Boston area

Caractérisation du substratum rocheux et conception de caissons foncés dans la roche dans la région de Boston

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SYNOPSIS: This paper suggests the use of a deformation-based design method for rock socketed caissons in highly variable weak rock conditions such as those existing in the Boston area.

I. INTRODUCTION

Demand for high-rise buildings and transportation infrastructure has created the need for high capacity rock socketed caissons (drilled shafts) in Cambridge Argillite, the soft bedrock underlying the greater Boston area. The traditional design method in this region for rock socketed caissons has been based on assumed end bearing and side resistance values incorporated into a force equilibrium equation. Since end bearing and side resistance are mobilized at different levels of deformation in a rock socketed caisson, the validity of the force-equilibrium method has been more strongly questioned recently. This paper suggests the use of a design approach (Rowe and Armitage, 1987a, 1987b) that is more appropriate for highly variable soft rock conditions such as those encountered in the Boston area. The traditional design procedure is reviewed and the proposed displacement-based design procedure is described.

II. GEOLOGY OF BOSTON AREA

Boston, Massachusetts lies in the Boston Basin and is underlain by bedrock divided into two principal formations: the younger Cambridge Argillite overlying the older Roxbury Conglomerate. A wide range of igneous intrusions in the form of dikes and sills are commonly encountered (Figure 1) (Humphrey and Soydemir, 1991). The Boston Basin is a topographic as well as a structural depression which had been subjected to glaciation during the Pleistocene Epoch, resulting in deposition of glacial and post-glacial marine and estuarine sediments. A typical soil overburden profile consists of (from ground surface down) recent fill, organic silt, marine sand, marine silty clay, glacial outwash sand and gravel, glacial till and bedrock (Figure 1).

The Cambridge Argillite is a shale, locally and weakly metamorphosed, and occasionally encountered with reworked tuffaceous material. It is generally hard and competent due to its poorly developed bedding planes and general lack of fissility. However, localized zones exist where alteration of the bedrock has produced zones of varying widths of kaolin,

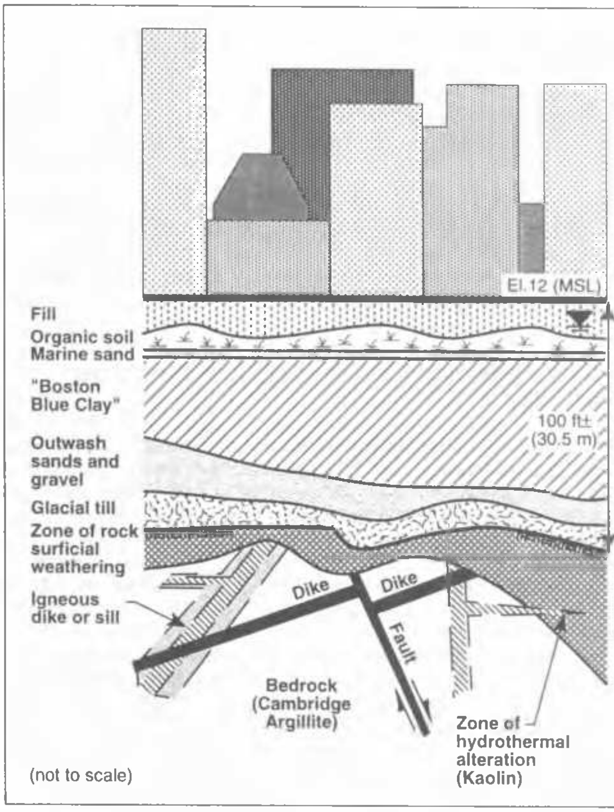

Figure 1. Typical Geologic Conditions

a "soil-like" material composed essentially of the clay mineral kaolinite. The abrupt and unpredictable change from the sound Argillite to the kaolinitic weak Argillite occurs in very short distances. To further complicate the situation, preglacial surficial weathering has created a variable thickness of overlying weathered rock that appears to have characteristics similar to those of the kaolinized zones. Kaye's (1967) study of thin-sections from the altered zone revealed that the commonly present rock minerals, including quartz, have been replaced by sericite and kaolinite at varying levels. This observation led to the possible explanation that the younger igneous intrusions have hydrothermally altered the adjacent weaker rocks and created the zones of highly decomposed kaolinite-rich, clay-like "soil"

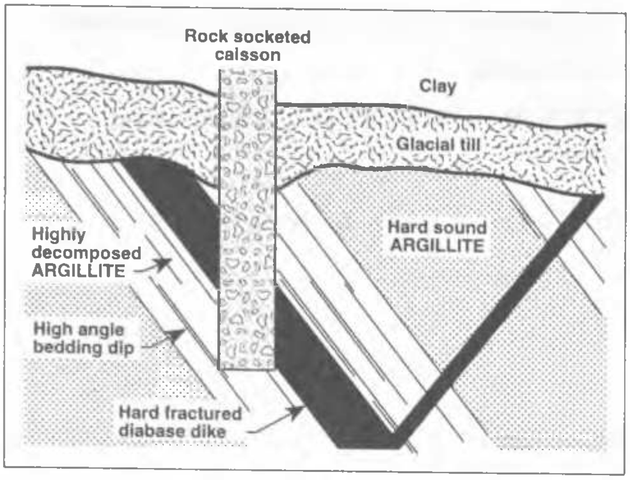

Figure 2. Caisson Socketed in Cambridge Argillite

adjacent and running parallel to the intrusions.

The erratic occurrence of the weathered and altered zones, in conjunction with the steeply dipping bedding planes typical of the Argillite, causes difficulty in characterizing the bedrock for engineering purposes. Without test borings at each caisson location, it is difficult to predict the quality of the rock within which a foundation unit will bear (Figure 2).

III. ENGINEERING PROPERTIES OF CAMBRIDGE ARGILLITE

The highly variable nature of the Cambridge Argillite makes it difficult to select an appropriate set of properties for design which represents the range of rock conditions that may be encountered in a project. In recent projects, it has been concluded that engineering properties could be defined for specific ranges in weathering.

Engineering properties that must be evaluated for rock socketed caisson design include the rock mass modulus, side resistance (adhesion), and end bearing capacity. The governing parameter for a displacement-based design method is the rock mass modulus, which defines the deformation characteristics of the rock formation. Rock mass modulus may be estimated by load tests, pressuremeter tests and empirical correlations.

Based on available data and empirical relationships, typical ranges of rock mass modulus for Cambridge Argillite are listed in Table 1. As depicted in the table, the range in modulus is relatively small for highly weathered rock. As the weathering decreases, the range in modulus increases, suggesting that structural features (e.g., fracturing) play a more dominant role in the modulus of fresher rock.

Rowe and Armitage (1987b) suggested empirical correlations, based on the measured performance of test caissons, to estimate side shear resistance and rock mass modulus from the average unconfined compressive strength of rock. Rowe and Armitage also recommended reduction factors be applied to parameters derived from empirical relationships to account for rock variability, and further recommended using

Table 1. Typical Argillite Properties

Weathering	Rock Mass Modulus kPa (ksf)	Average Unconfined Compressive Strength kPa (ksf)
Very severely to completely weathered	192,000-1,680,000 (4,000-35,000)	320 (7)
Moderately severely to severely weathered	144,000-886,000 (3,000-18,500)	7,000 (150)
Fresh to moderately weathered	24,000-192,000 (500-4,000)	24,000 (500)

parameters determined directly from load tests in the design area if available. Load test experience in the Boston area suggests that while the Rowe and Armitage side resistance correlation overestimates the available side resistance by as much as 3.5 times, the modulus correlation is appropriate.

IV. LOCAL DESIGN PRACTICE

The evolution of design practice for rock socketed caissons in the Boston area has been essentially empirical up through the early 1980s. The City of Boston Building Code, which was in effect until 1975, specified allowable design stresses for end bearing in rock and for bond between rock and concrete. Thus, proportioning of the caisson socket followed a force equilibrium procedure.

The first significant implementation of the Boston Building Code relative to rock socketed caissons was in the design of the foundations of Boston's Prudential Center (Ball, 1962). In accordance with the Boston Building Code requirements, the rock socketed caissons were proportioned by the force equilibrium procedure using an end bearing stress of 6.70 MPa (140 ksf) and a bond stress of 0.69 MPa (14.4 ksf) between concrete and the Cambridge Argillite. The design end bearing stress was controlled by the compressive strength of the concrete (6.20 MPa; 130 ksf).

The first reported foundation project including rock socketed caissons where "altered" Cambridge Argillite was encountered was the Boston Company Building. Johnson (1973) described that "in certain zones the rock is highly altered and weakened to the point that the material can be crumbled between the fingers." Boston Building Code at the time allowed 4.8 MPa (100 ksf) for end bearing in rock and 0.69 MPa (14.4 ksf) for bond stress around the rock socket shaft. Ultimately it was decided to support each of the four corner loads of 66.6 MN (15,000 kips) by a large diameter pier relying exclusively on an allowable end bearing stress of 2.88 MPa (60 ksf).

During the 1980s, with increased demand for high-rise buildings in downtown Boston, the Cambridge Argillite was more fully investigated to support larger foundation loads. Extensive subsurface investigations and laboratory testing have shown the wide ranging quality of the

Cambridge Argillite as a result of different degrees of geologic weathering and alteration. Relatively poor rock conditions encountered at several sites made it obvious that foundation support exclusively relying on end bearing in rock would not be economical, and side resistance mobilized through rock socketed caissons to greater depths was established as the primary source for support. Primarily based on experience and engineering judgement, side resistance values ranging from 190 kPa (4 ksf) to 950 kPa (20 ksf) were adopted in design at different sites across the city.

Relative to the use of the force equilibrium method in designing rock socketed caissons, Johnson (1983) observed that "the development of ultimate bearing capacity of the base may require several inches of movement while the shaft resistance may be fully mobilized at fractions of an inch. Settlement estimates must be made and may govern the design." Settlement analyses inherently required elastic solutions based on a rigorous evaluation of the distribution of the load between side resistance and end bearing (Poulos and Davis, 1974; Pells and Turner, 1979). More often, however, a 70 percent side resistance and 30 percent end bearing distribution was simply assumed. This approach has often been challenged by contractors who argued that the end bearing component was being underutilized, making the use of rock socketed caissons less competitive.

V. SUGGESTED DESIGN METHOD

In an effort to improve on local practice and to design more efficient rock socket geometries, the Rowe and Armitage (1987a, 1987b) approach is recommended herein for the design of rock socketed caissons in Cambridge Argillite. This approach defines rock socket geometries that will limit vertical deformation to a selected design value. It differs from traditional design practice which develops socket geometries based on an assumed load distribution between side resistance and end bearing, and then attempts to estimate settlement through an independent analysis. The Rowe and Armitage approach is rock mass modulus based and includes design charts derived from finite element modeling of varying rock conditions along the shaft and at the base (Figure 3). Design charts are presented for various ratios of rock modulus along the shaft to rock modulus at the base (E_r/E_b) and the pier (caisson) modulus to rock modulus along the shaft (E_p/E_r). Data from field performance of rock socketed caissons were included in the development of the design charts.

Rowe and Armitage (1987b) assumed that mobilization of side resistance (i.e., rock-shaft adhesion) follows elasto-plastic behavior. That is, load applied to the top of the rock socket must induce shear stresses exceeding the available adhesion prior to yielding of the shaft in the socket, which results in plastic deformation and significant load transfer to the base. This plastic deformation or "slip" occurs only when the applied load exceeds the available adhesion; however, adhesion is maintained after the slip. Accordingly, the Rowe and Armitage (1987b)

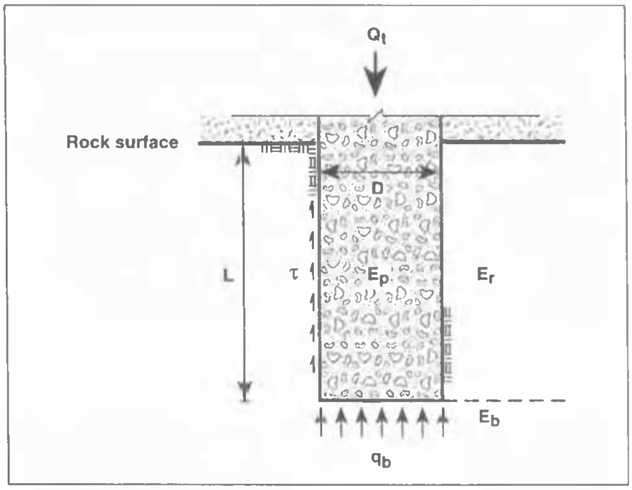

Figure 3. Notation for Rock Socketed Caisson

method better represents the actual distribution of side and end load primarily based on relative rock mass modulus at the side (E_r) and base (E_b) of the socket and thus allows for an improved estimate of settlement for a particular socket geometry.

For rock conditions where E_b/E_r varies between 0.5 and 2, the Rowe and Armitage (1987b) approach is directly applicable. Given the variability of rock weathering in the Boston area, it is possible to encounter rock at the base of the caisson that is significantly weaker than the rock along the side of the caisson. That is, E_b/E_r ratios less than 0.5 must often be considered. As the Rowe and Armitage approach only presents design charts for caissons E_b/E_r between 0.5 and 2, another method is required to calculate settlement for caissons with low modulus rock at the base.

A method to address this condition should consider the caisson loading and specific mechanisms causing deformation or settlement. These mechanisms are generally considered to be elastic shortening of the concrete caisson, plastic deformation occurring at the slip condition, and elastic compression of the rock at the base of the caisson following slip and load transfer to the base.

As load is applied, elastic shortening of the concrete shaft occurs in direct proportion to the load and length of the shaft and inversely proportional to the caisson modulus and diameter. Therefore, the amount of elastic shortening can be calculated at any applied load.

Plastic deformation occurring at the slip condition depends on other less easily defined characteristics such as the roughness of the excavated rock along the socket, the variation of rock mass modulus along the socket, and the amount of remolded rock or mineral slurry smeared along the side of the socket. It is suggested that the amount of slip movement can be approximated as 2 to 3 times the elastic shortening of the concrete within the socket. This amount of shaft movement within the socket should generally be sufficient to transfer load to the rock below the base of the caisson. Slip movement should be expected to exceed this estimate if loose material resulting from improper cleanout of the socket is left at the base of the socket.

Applied load in excess of the available side resistance capacity is resisted by rock at the base of the socket. Solutions to elastic theory may be used to estimate the resulting vertical base movement.

Therefore, it is suggested that total settlement of the caisson socketed into rock with E_b/E_r less than 0.5 can estimated by adding the calculated elastic shortening, slip movement, and elastic deformation.

VI. CLOSURE

The Greater Boston area is predominantly underlain by Cambridge Argillite, a weakly metamorphosed shale with deep and variable weathering conditions. The strength and deformation characteristics appear to be related to weathering and are thus also highly variable. Accordingly, prudent rock socketed caisson design in the Boston area should consider a range in both rock quality and design parameters appropriate for the particular site. Since the amount of bedrock data may be limited for a given project and the rock conditions may be quite variable, a statistical evaluation of the rock data would be appropriate. Statistical analyses may be used to estimate the probability of founding a caisson in a particular rock. Of particular importance is the likelihood of encountering completely weathered Argillite at depth, which could result in large settlements of a caisson unit.

It is proposed that the method and model developed by Rowe and Armitage (1987a, 1987b) for the design of caissons socketed in soft rock be adopted for the Cambridge Argillite since it can accommodate a project specific settlement criterion. Settlement estimates should consider a range of E_b/E_r from less than 0.5 to 2, due to the significant variation of rock quality over short distances both vertically and laterally. Ability of the structure to tolerate differential settlement and/or to redistribute load to adjacent foundation units bearing on more competent rock should also be considered in the design of caissons socketed in highly variable rock formations such as Cambridge Argillite.

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