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AXIAL RESPONSE OF DRILLED SHAFTS IN INTERMEDIATE GEOMATERIALS IN THE SOUTHEAST

by

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ABSTRACT

Concern regarding scour at bridge foundations has lead to deeper foundation embedment requirements and increased use of drilled shaft foundations for highway bridges in the Southeastern U.S. Very often these shafts are designed to penetrate relatively weak rock materials for which there are little load test data to provide design guidance. Data have been collected from a number of sites in the southeastern states which demonstrate a substantial amount of socket friction in relatively weak rock and dense soils. This paper presents the results of 12 axial tests in Alabama, Georgia, Mississippi, and South Carolina. Although these intermediate materials are often difficult to quantify with respect to in-place strength characteristics, these load test data provide the type of feedback necessary to develop judgement and form the basis for comparisons with proposed design methods.

INTRODUCTION

Drilled shafts are increasingly being used for highway bridge foundations in the Southeastern U.S. Design considerations for scour have led to deeper foundation embedment requirements, resulting in drilled shafts being designed as sockets in relatively weak rock materials. Very little load test data on shafts in these materials are available to provide design guidance. This lack of data has resulted in extremely conservative designs with regard to the mobilization of side friction and end bearing.

In order to provide better design guidance with respect to soft rock materials, the Alabama Department of Transportation (ALDOT) has sponsored a research program to investigate the axial capacity of drilled shafts constructed in weak rocks. These materials can range from very dense/hard soils to weak or soft rocks. By collecting data from tests conducted in materials similar to those encountered in Alabama, design guidance may be developed for use by ALDOT engineers. The results of 12 such tests are presented in this paper.

REVIEW OF DESIGN METHODS

Introduction

Most design methods in use today can be placed in one of four design approaches (Rosenberg and Journeaux, 1976). These are: 1) design based on end bearing only, 2) design based on skin friction only, 3) design based on allowable end bearing with remaining load carried in skin friction, and 4) design based on estimates of mobilized end bearing and skin friction.

The first three approaches place restrictions on the potential socket geometries available to carry a given load by disregarding all or part of the shaft's ability to mobilize one of the components of its capacity. These restrictions can lead to conservative designs since the full load carrying ability of the shaft is not considered.

The fourth approach assumes that the load is carried in both side friction and end bearing in proportions that depend on the actual load transfer occurring between the shaft and the soil or rock. An understanding of the load transfer relationship provides a means of estimating the expected values of mobilized side friction and end bearing. Predicting load transfer can be difficult since the amount of displacement needed at the top of the shaft to mobilize side friction and end bearing are often unequal. A relatively small amount of displacement is need to fully mobilize side friction, whereas relatively large displacements are necessary to completely mobilize end bearing (Osterberg, 1992). This relationship can make it difficult to determine the relative proportions of each component that are mobilized under a given load.

All of these approaches require designing the shaft based on some characteristic of the geomaterial in which the shaft is built. Various methods are based on in-situ strength tests, laboratory strength tests, elastic modulus values, or some combination of these or other characteristics. Yet, it is often difficult to quantify the in-situ strength characteristics of soft rock materials. Most design methods are therefore based on a correlation between unit side friction or unit end bearing and the unconfined compressive strength (q_u) of the rock. These correlations are common because (Horvath and Kenney, 1979): 1) the maximum potential friction at the shaft-rock interface is controlled by the shear strength of the weaker material (usually the rock), 2) the rock strength is dependent on material type, degree of weathering, extent of fractures and joints, etc., and 3) information concerning rock q_u values is readily available or easily obtained.

Methods Reviewed

A number of design methods were reviewed for the project. These include:

Rosenberg and Journeaux, 1976
Pells and Turner, 1979
Williams et al., 1980
Rowe and Armitage, 1987(a), 1987(b)
Reese and O'Neill, 1988
M^cVay et al., 1992
O'Neill, 1993
Mayne, 1993

One early correlation of side friction to rock strength through full scale load tests was presented by Rosenberg and Journeaux (1976). Their method used a correlation of side friction to rock unconfined compressive strength along with load transfer curves developed by

Osterberg and Gill (1973) to select a shaft geometry that mobilized full shaft resistance without exceeding an allowable base resistance. The work of Osterberg and Gill included developing load transfer curves of a rock socketed shaft based on an elastic finite element analysis. They found that the distribution of the load depends on the depth of embedment of the socket, the socket diameter, and the elastic modulus and Poisson's ratio of the rock.

These early elastic solutions to predicting shaft capacity were further refined and expanded by Pells and Turner (1979), and then again by Williams et al.(1980). Pells and Turner presented two different design methods. One method assumes full mobilization of unit side friction and end bearing. The other method uses an elastic load distribution curve to distribute the load between side friction and end bearing depending on the socket geometry and the elastic modulus of the rock.

The work of Williams et al. resulted in detailed design methods based on load tests conducted in a Silurian mudstone in the area around Melbourne, Australia. Methods for designing shafts based on side friction only, end bearing only, or combined side friction and end bearing were presented. These methods involve estimating the ultimate unit values for side friction and end bearing, and then reducing these values according to predicted elastic load distributions. These reduced values are the predicted values for the mobilized unit side friction and end bearing. Ultimate unit side friction predictions are based on rock strength (q_u), while ultimate unit base resistance predictions are based on the elastic modulus of the rock.

Another design method based on elastic analysis and load test data was developed by Rowe and Armitage (1987(a), 1987(b)). Their method is based on satisfying a specified design settlement for an overall factor of safety. Correlations of unit side friction and end bearing to rock strength (q_u) are presented as a part of the method.

The most common design method currently in use in the United States is presented by Reese and O'Neill (1988). They recommend designing the shaft for capacity in either side friction or end bearing based on a computed settlement value. If the calculated settlement is greater than 0.4 inch, the bond between the shaft and the rock is assumed to be broken, transferring the entire load to the base of the shaft. For calculated settlements less than 0.4 inch, the bond is assumed to hold such that little or no load is transferred to the base of the shaft. Both unit end bearing and unit side friction are estimated through correlations to the unconfined compressive strength of the rock.

M^c^vay et al. (1992) presented a method for estimating unit side friction through a correlation to both the unconfined compressive strength and the split tensile strength of the rock. The authors believed that the use of both strength tests more accurately defined the strength of the shaft-rock interface. A database of load tests in Florida limestone was compiled to compare a number of correlations along with the new method.

Two of the most recent investigations of the design of shafts in rock were presented by O'Neill (1993) and Mayne (1993). O'Neill published a summary of preliminary design methods proposed under a research contract with the FHWA. Two methods are presented, one for argillaceous (or clay-based) rocks and one for decomposed (granular-based) rocks.

The method for argillaceous rocks is a modification of the method presented by Williams et al. (1980). The equations and associated graphs of the Williams et al. method have been combined into a series of equations solvable without use of the graphs. Also, a factor has been included to account for the smeared shaft-rock interfaces that can occur in excavations in some rocks of this type.

The method for decomposed rock was developed by Mayne (1993) through a load test program conducted on the campus of The Georgia Institute of Technology. Estimates of unit side friction and end bearing are made with correlations to the effective angle of internal friction and the undrained shear strength of the material, respectively. O'Neill suggests that this method is probably conservative for most decomposed rock.

Summary

The methods reviewed all rely to some extent on an elastic analysis of the rock-shaft interface. All of their predictions of unit side friction yield average values over the length of the rock socket based on peak load transfer. The use of the unconfined compressive strength to characterize the rock strength is also a common feature. Unit end bearing is determined by either an elastic analysis of the controlling settlement or is taken as a simple linear correlation to the q_u value.

A more detailed review of each of the above methods is presented in Thompson (1994).

LOAD TEST DATA

Load tests provide the most potentially reliable method to verify design parameters or methods. The results of a number of axial load tests conducted in soft rock formations in the Southeastern U.S. have been collected for this project. Some of the tests were conducted using a conventional static load test set-up consisting of loading the shaft at the ground surface. Other tests were conducted using the Osterberg Cell loading device (Osterberg, 1989). Table 1 lists the location, geology, test type, and source of each test.

In order to present the data collected, the tests have been grouped into two categories: argillaceous rocks and granular-based rocks. Tables are presented below that compare the results of each test to the predicted values for side friction and end bearing as computed by several different methods. For the tests in argillaceous rocks, predicted values from the methods of Williams et al. (1980), Rowe and Armitage (1987), Reese and O'Neill (1988), and O'Neill (1993a) are given. Predicted values form the methods of Mayne (1993) and Reese and O'Neill (1988) are given for the tests in granular based rocks. M[c]Vay's method is considered separately.

Table 2 gives the side friction data for the argillaceous rock tests. The value of unit side friction given is the average for the portion of the shaft socketed into the subject geomaterial. The deflection (δ) given is the deflection at which the average side friction was mobilized. Deflections from Osterberg Cell tests are given as positive, representing the upward movement of the shaft.

From these data, it appears that the method of Reese and O'Neill generally underpredicts unit side friction while the other three methods generally overpredict unit skin friction. At both the

Table 1. Load test summary

LOCATION	GEOLOGY	TYPE	REFERENCE
Andalusia, AL	Claystone	Conventional	Bhate Eng. Corp., 1992
Blount Co., AL	Shale	Osterberg Cell	Hwy. Rsch. Ctr., 1994
Montgomery, AL	Very dense sand	Conventional	Brown, 1994
Tuscaloosa, AL	Very dense sand	Osterberg Cell	Loadtest, 1992
Wilsonville, AL	Shale	Osterberg Cell	Loadtest, 1994a
Atlanta, GA	Weath. Granite	Conventional	Mayne, 1993
Coewta Co., GA	Weath. granite	Conventional	O'Neill, 1993
Owensboro, KY	Shale	Osterbrg Cell	Goodwin, 1993
Leake Co., MS	Clay/chalk	Osterberg Cell	Loadtest, 1994b
Mt. Pleasant, SC	Marl	Conventional	Law Engineering, 1991

Table 2. Side friction data for argillaceous rocks

LOCATION	ROCK	AVG f_s (tsf)	δ @ f_s (in)	Predicted f_s (tsf)			
				WILLMS. ET AL	ROWE & ARMTGE	REESE & O'NEILL	O'NEILL
Andalusia Alabama	Claystone	4.8	-0.13	5.7	6.7	3.1	6.0
Andalusia Alabama	Claystone	3.5	-0.61	5.7	6.7	3.1	6.0
Blount Co. Alabama	Shale	>11.5	0.07	8.5	11.7	5.4	7.5
Wilsonville Alabama	Shale	3.2	0.66	3.2	3.1	0.7	4.4
Owensboro Kentucky	Shale	>9.9	0.36	5.8	7.0	3.2	6.1
Leake Co. Mississippi	Clay/ Chalk	1.6	0.18	3.6	3.6	0.9	4.6
Mt. Pleasant S. Carolina	Marl	1.8	-0.15	2.1	1.7	0.2	3.5
Mt. Pleasant S. Carolina	Marl	1.8	-0.10	2.1	1.7	0.2	3.5

Blount County and Owensboro tests, the capacity of the Osterberg cells was reached before the sockets failed. It appears that for both of these tests, all four methods are conservative.

The data for end bearing of the argillaceous rocks is shown in Table 3. The rock strength given for the Wilsonville test is an estimate based on the available SPT blow count. The blow count was used to estimate an undrained shear strength from which q_u was estimated. The end bearing measured at a deflection of 2 percent of the shaft diameter (B) is used as the basis of comparison. Although a legitimate argument could be made that additional end bearing is available at larger deflections, this value represents one that would ordinarily be considered "large displacement" for drilled shafts associated with bridge foundations and most such structures are not capable of mobilizing larger displacements without severe structural distress. Comparisons could be made at any chosen percentage.

Since most design methods predict a maximum unit end bearing as a multiple of the unconfined compressive strength, the ratio of the measured unit end bearing to the rock strength is given. Ratios of 2 to 3 are common for most design methods. The data presented here indicate ratios which are generally well above 3. The Blount County test has a ratio of less than one. It is believed that a large amount of debris was present at the bottom of the shaft excavation when the Osterberg Cell was installed. Most of the measured downward movement of the cell was probably the result of the compression of the debris, resulting in low end bearing measurements (Highway Research Center, 1994).

Tables 4 and 5 present the side friction and end bearing data, respectively, for the granular-based rock tests. The "β" method is the method for designing shafts in granular soils presented by Reese and O'Neill (1988). This method is presented as an alternative to Mayne's method since granular-based rocks weather into granular soils. Some of the tests were conducted in very dense sands similar to weathered rocks.

From these data, there do not appear to be any general trends for either method in regards to predicting skin friction. It does appear from these data that Mayne's method under predicts end bearing while the β-method appears to over predict unit end bearing. These observations are based on unit end bearing values mobilized at shaft deflections of 2 percent of shaft diameter.

Table 6 provides a comparison of side friction values as predicted by M^{c}Vay's method to measured values. Only two of the tests had split tensile data available: Blount County, Alabama and Owensboro, Kentucky. Both of these tests reached the capacity of the Osterberg cells before the socket failed. More data in rocks other than Florida limestone (the material from which the method was derived) are needed to better review this method.

CONCLUSIONS

This project represents an initial step for more reliable predictions of shaft capacity in soft rocks encountered in Alabama. The lack of a large number of tests makes statistical analyses such as linear regression or goodness of fit impractical; however, some trends are recognized in the available data. Additional load tests will be added to the

Table 3. End bearing data for argillaceous rocks

LOCATION	q_u (tsf)	q_b (tsf)	δ=2%(B) (in)	q_b/q_u
Blount Co. Alabama	64.8	42.8	-0.64	0.66
Wilsonville, Alabama	4.5	58.2	-0.64	12.9
Owensboro, Kentucky	23.0	113.0	-1.42	4.9
Leake Co., Mississippi	6.0	8.4	-1.32	1.4
Mt. Pleasant, S.Carolina (Shaft 1)	1.4	13.5	-0.48	9.6
Mt. Pleasant, S.Carolina (Shaft 2)	1.4	10.0	-0.48	7.1

Table 4. Side friction data for granular-based rocks

LOCATION	ROCK	AVG f_s (tsf)	δ @ f_s (in)	Predicted f_s (tsf)	
				Mayne	6
Montgomery Alabama	Dense Sand	1.7	-0.85	1.2	1.3
Tuscaloosa Alabama	Dense Sand	1.6	+1.11	2.2	1.4
Atlanta Georgia (C-1)	Weathered Granite	>3.2	-1.02	2.9	1.6
Atlanta Georgia (C-2)	Dense Sand	0.7	-1.0	0.7	1.4
Coweta Co. Georgia	Weathered Granite	1.4	-1.65	2.6	0.9

Table 5. End bearing data for granular-based rocks

LOCATION	ROCK	q_b (tsf)	δ 2%(B) (in)	Predicted q_b (tsf)	
				Mayne	6
Montgomery Alabama	Dense Sand	30.6	-0.60	10.3	36.0
Tuscaloosa Alabama	Dense Sand	34.0	-0.72	22.2	45.0
Atlanta Georgia (C-1)	Weathered Granite	46.4	-0.60	24.6	45.0
Atlanta Georgia (C-2)	Dense Sand	7.1	-0.60	6.3	12.0
Coweta Co. Georgia	Weathered Granite	24.0	-0.72	21.8	45.0

Table 6. Comparisson of M^cVay's method

LOCATION	q_u (tsf)	q_t (tsf)	MEASURED f_s (tsf)	PREDICTED f_s (tsf)
Blount Co., Alabama	64.8	18.3	>11.5	17.2
Owensboro, Kentucky	23.0	4.7	>9.9	5.2

database as they are available in order for further analyses to be made. At present, no single method reviewed herein can be selected as providing a better prediction of shaft capacity than methods in use by the Alabama DOT. With the addition of more tests in the future, a method may be determined as better for the soft rocks encountered in Alabama.

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