USE OF ROCK PRESSUREMETER FOR DEEP FOUNDATION DESIGN

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ABSTRACT – Rock pressuremeter tests can accurately measure the in-situ deformation modulus of rock. Comparisons are made with the Rock Quality Designation (RQD) and the Rock Mass Rating (RMR). Foundation design without direct measurements of rock properties tends to be highly overconservative. A case study is presented showing lateral load analyses of deep foundation design with rock pressuremeter results.

1. Introduction

While the capacity of bedrock can be quite high, engineers often under estimate this capacity during the design phase of a project because there is very limited data available of the actual in-situ strength and deformation properties of bedrock. In-situ testing of rock is performed in a rock core hole and requires a pressuremeter that is capable of exerting very high pressures. The calibration for system compressibility is important for accurate results. Volume measurements should be made inside the probe to eliminate the correction due to tubing expansion. One can usually obtain an accurate value for the rock modulus. In good quality rock, the limit pressure may not be reached, but with a rock pressuremeter the engineer will know that the limit pressure will be at least 30 MPa. Comparisons are made between rock moduli values determined during rock pressuremeter testing and RQD and RMR evaluations of the recovered rock core.

A case study of deep foundation design using rock pressuremeter test data is presented. Rock is often heterogeneous and values of rock modulus have scatter. Our design approach to account for the heterogeneity of the rock uses a modulus design value equal to the average value minus two (2) standard deviations.

Schmertmann and Hayes (1997) performed Osterberg cell load tests on drilled shafts founded in soil and rock. The tests were taken to failure loads provided the O-cell had enough capacity. They determined that engineers tend to under-predict capacity more as the material transitioned from soil to rock as shown on Figure 1. In rock, engineers often underestimate ultimate capacity by a factor of 5 or more!

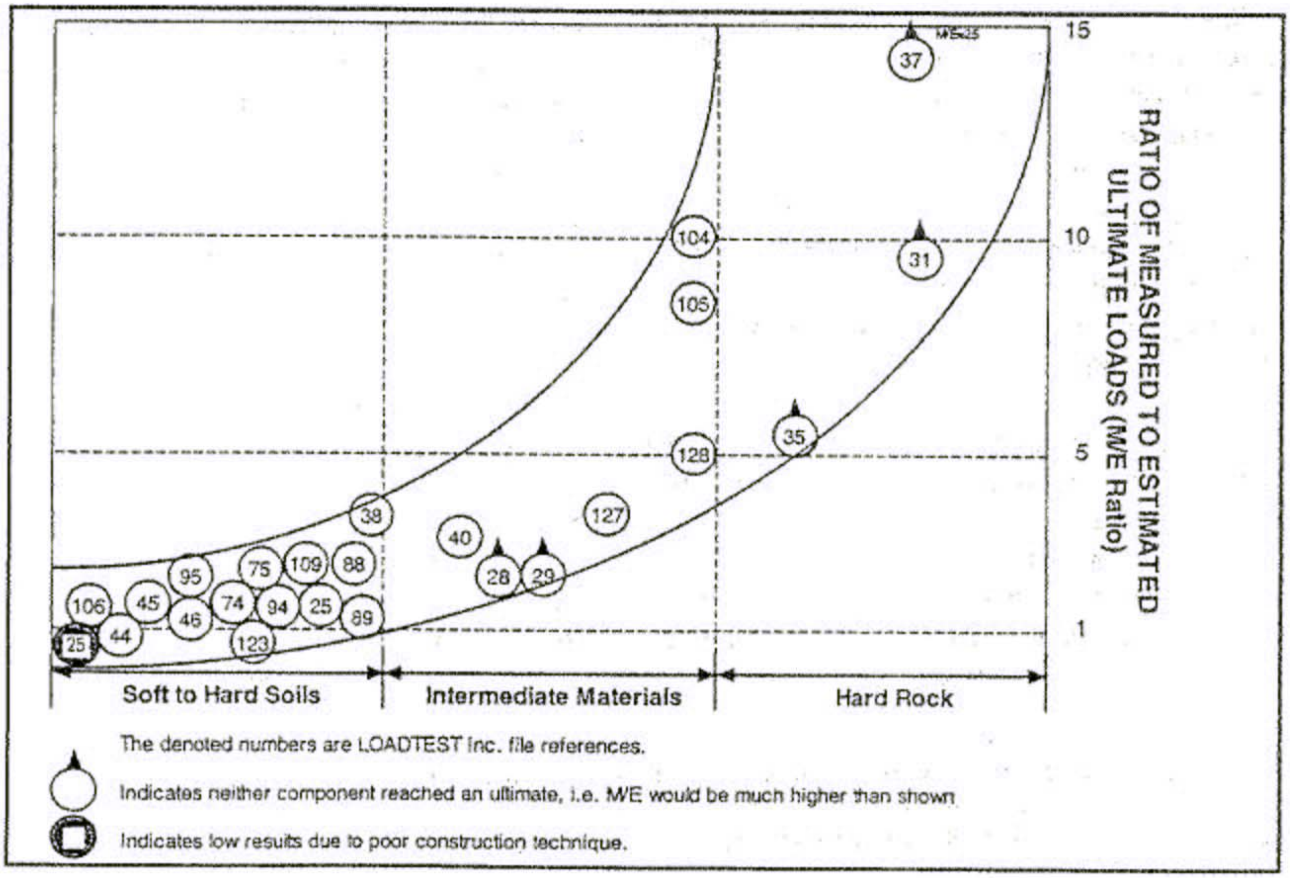

Figure 1. Underestimating rock capacity (Schmertmann and Hayes)

2. Rock Pressuremeter Test Procedure

NX size rock coring was performed in rock. A "Probex" rock pressuremeter manufactured by Roctest was lowered into the cored hole. Pressures were applied by pumping a hydraulic jack that extends a piston downward inside the probe. This piston pressurizes the ethanol glycol-water mixture inside the membrane causing the membrane to expand. The hydraulic pressure is measured at the pump with a pressure gauge and a transducer. A linear variable displacement transducer (LVDT) is mounted inside the probe and measures the movement of the hydraulic piston. The volume of the expanded pressuremeter is computed by multiplying the displacement by the piston area. To get accurate and stable data, readings are recorded with a data logger one (1) minute after initially applying the pressure. The test is performed as a stress controlled test, applying approximately 10 pressure increments and recording the corresponding volume. The tests were stopped either after 30 MPa of pressure was applied or a volume of about 300 cm^3 was injected. Based on our experience, we note that when volumes of more than 300 cm^3 are applied the membrane could rupture. This is usually caused when the probe bulged locally due to a clay seam or other weak zone in the rock. Although the membranes were robust, they were quite costly (about $2500 U.S.).

The pressuremeter was calibrated for system compressibility and membrane resistance. Because the volume was measured inside the probe, the expansion of the tubing in a conventional pressuremeter was eliminated. Typically, the system compressibility was about 1 cm^3/Mpa. The membrane resistance was about 2.5 MPa at 400 cm^3. A photo of the rock pressuremeter being lowered into the rock-cored borehole is shown below. The hydraulic pump and data logger are shown in the front.

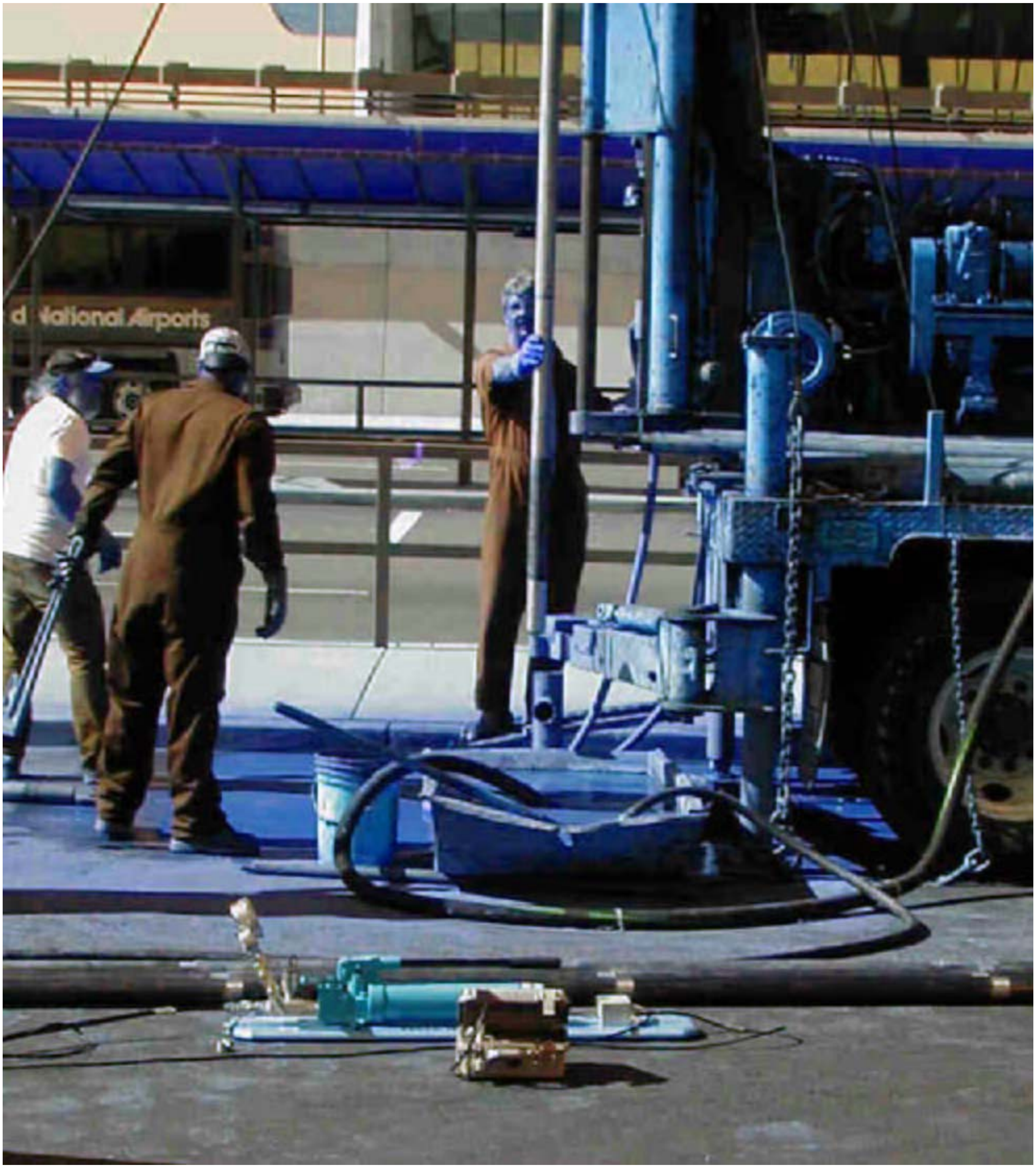

Photo 1. Rock pressuremeter being lowered into borehole

3. Site Geology

Rock pressuremeter tests were performed in various geologic formations in the eastern portion of the United States. The following sections describe the geology at the sites:

3.1 Mountaineer Race Track Road, Hancock County, West Virginia

This project is located in the Appalachian Plateaus Physiographic Province of the Central Appalachian Mountains. This region is characterized by deeply-dissected, moderate to high-relief topography formed by downcutting and rejuvenation of local rivers and streams. Surficial rock strata are typically relatively flat lying (5° or less) and consist of cyclic sandstones, shales, limestones, claystones, siltstones and coals of Pennsylvanian Age. In the project area, rock is believed to be comprised of the Allegheny Series and upper portion of the Pottsville Series. Available geologic information indicates that rock beds dip toward the west-south-west at an average rate of about 15 feet per mile. Localized variations in the rock dip are anticipated.

The sandstone encountered at elevation 693± is believed to be the Lower Kittanning Sandstone which is relatively indistinguishable from the Clarion Sandstone located immediately beneath it. Geologic information for the area indicates that the combined thickness of these two sandstone units is approximately 85 to 90 feet. The Clarion Coal and Clay is generally located below the Clarion Sandstone; however, project borings did not encounter the coal or clay. It is believed that the Clarion Coal and Clay must have pinched out in this area since sandstone was encountered consistently from elevation 693± to elevation 574±. The lower portion of sandstone encountered from elevation 605± to 574± is believed to be the Homewood Sandstone. (Dodson, 2005)

3.2 14th and V Streets, Northwest, Washington, D.C.

The intersection of 14th and V Streets, NW in Washington, D.C. is located in the Q5 subdivision of the Quaternary deposits in Washington. This material consists of Middle Pleistocene sediments containing interlayered gravel, sand, silt, and clay. These deposits overlie the intrusive rocks of early Ordovician Age. The principle rock type in this area is the Kensington Tonalite. This unit consists of coarse-grained, weakly- to moderately-foliated, locally garnetiferous, biotite-muscovite tonalite. These rocks are intensely foliated to mylonitic biotite-muscovite granodiorite containing microcline augen. Some rocks contain hornblende.

3.3 16th Street Bridge over I-64, Huntington, West Virginia

Cabell County, West Virginia is located in the Dunkard and Monongahela Groups of Pennsylvania Age. These formations consist of interbedded and interlayered sandstone, siltstone, sandstone, shale (mudstone), limestone and coal. Of concern, from an engineering point of view, is the strength of the shales and mudstones, some of which can be particularly sensitive to the introduction of water (swell potential) and the instability of this rock foundation loads are applied. In many cases, the mudstones are only slightly harder than the clay soils from which they originate with low unconfined compressive strengths (as low as < 1tsf) and very low undrained shear strengths. Depending upon the specific mudstone strata, the material can quickly weather to a mud in as little as 24 hours. Recognition of the instability of these mudstone rocks is critical to successfully engineering foundations, highway cuts and fills, trench excavations and to provide stability in underground coal mines.

3.4 Interstate I-64 and I-295, Richmond, Virginia

The project site is located in the Piedmont Physiographic Province of Virginia. The Piedmont is characterized by low, rounded hills composed of saprolitic soils overlying folded metamorphic and igneous bedrock. Triassic-age sedimentary basins, bounded by normal faults, are also located within the Piedmont region, trending along northeast-southwest axes. Diabase dikes, trending north-south to northwest-southeast, are common in some areas of the Piedmont. These dikes are related to the extensional tectonics that formed the Triassic Basins.

Locally, the site is underlain by the Mississippian-age Petersburg Granite. The Petersburg Granite is composed of light to dark gray to pink, fine to coarse-grained granite to granodiorite. Overlying the Petersburg Granite may be varying amounts of saprolite, residual soil derived from the in-situ weathering of the bedrock. Saprolitic soils are typically silty and clayey near the surface and become coarser with depth as the soil profile approaches weathered bedrock.

3.5 Interstate I-95 over Occoquan River, Occoquan, Virginia

This project location is in the eastern Piedmont Physiographic Province of Virginia, just west of the western limit of the Coastal Plain Physiographic Province. Specifically, the site is underlain by the Quantico Formation, according to the Geologic Map of the Occoquan Quadrangle and Part of the Fort Belvoir Quadrangle, Prince William and Fairfax Counties, Virginia (Seiders and Mixon, 1981). The Quantico Formation is described by Seiders and Mixon as dark-gray to black carbonaceous pyritic slate with thin to thick graded beds of poorly sorted quartz sandstone and siltstone. Chert, mudstone, and felsic volcaniclastics may also be locally present. In addition, minor amounts of alluvial material may be present proximal to the Occoquan River. The geologic strata in this area are highly deformed, and bedding planes are nearly vertical throughout the site. Some beds may be overturned.

3.6 Corridor H, Scherr, West Virginia

According to the West Virginia Geological and Economic Survey's (WVGES) Geologic Map of West Virginia, the underlying rock strata in the project area are sedimentary in origin and are of the Devonian and Silurian age. The upper rock formation found along part of the project was the Wills Creek Formation. A unit that in this area is characterized by a series of gray-brown to green fissile shales, interbedded with gray, laminated limestones. These rock units are steeply dipping to the northwest. The bedrock-soil interface was observed, during drilling in this location, to be rather erratic, varying in elevation by as much as 10 feet over relatively short horizontal distances, an occurrence which can be explained by a type of karst phenomenon known as cutter-and-pinnacle karst. Cutter-and-pinnacle karst occurs when the contact between bedrock and soil overburden becomes very irregular as a result of water preferentially dissolving bedrock along planar features, such as bedding, joints, or fractures. Solutionally widened joints or bedding planes at or near the bedrock-soil interface are called cutters which are generally filled with soil. The bedrock that remains between cutters may be reduced to relatively narrow "ridges" of rock, called pinnacles, particularly where cutters are closely spaced. Cutter-and-pinnacle karst (or simply "pinnacle karst" for short) is common in many of the carbonate valleys of the Appalachians.

Underlying the Wills Creek Formation is the Tonoloway Limestone. The Tonoloway is a more resistant limestone and consists of a gray to dark-gray, non-crystalline, thinly bedded or flaggy, limestone containing some shale interbeds, and almost no fossils. Some solutional voids were noted, but generally the cutter and pinnacle phenomenon was not as apparent in the Tonoloway. (Barclay, 2005)

4. Rock Moduli Comparisons with RQD and RMR

Shown in Figure 2 is a comparison of pressuremeter rock moduli versus rock quality designations (RQD). As indicated on Figure 2, RQD alone is a poor predictor of rock modulus. For example, a claystone with a RQD of 80 has a lower modulus than other rock types with RQD values of 20. Granite with a RQD of 20 has modulus values of 5000 MPa or better. Modulus values in granite tend to be high whether the RQD is 20 or 100. Rock type, joint spacing, joint orientation and joint infill have as much importance on the rock properties as RQD. It is important to note that the determination of RQD not only includes the measurement and addition of the length of each piece of core longer than four inches but also that the rock must be "hard and sound'. Even if the individual core pieces exceed the requisite 100 mm (4 inch) length, any rock core not "hard and sound" should not be included in the determination of the RQD value. The "hard and sound" definition may not have been applied to all the boring logs in our data base.

Shown as Figure 3 is the rock modulus versus rock mass rating (RMR). Our data set was limited to the I-95 Bridge over the Occoquan River project. Although the band width is rather large, there appears to be some trending of modulus with RMR.

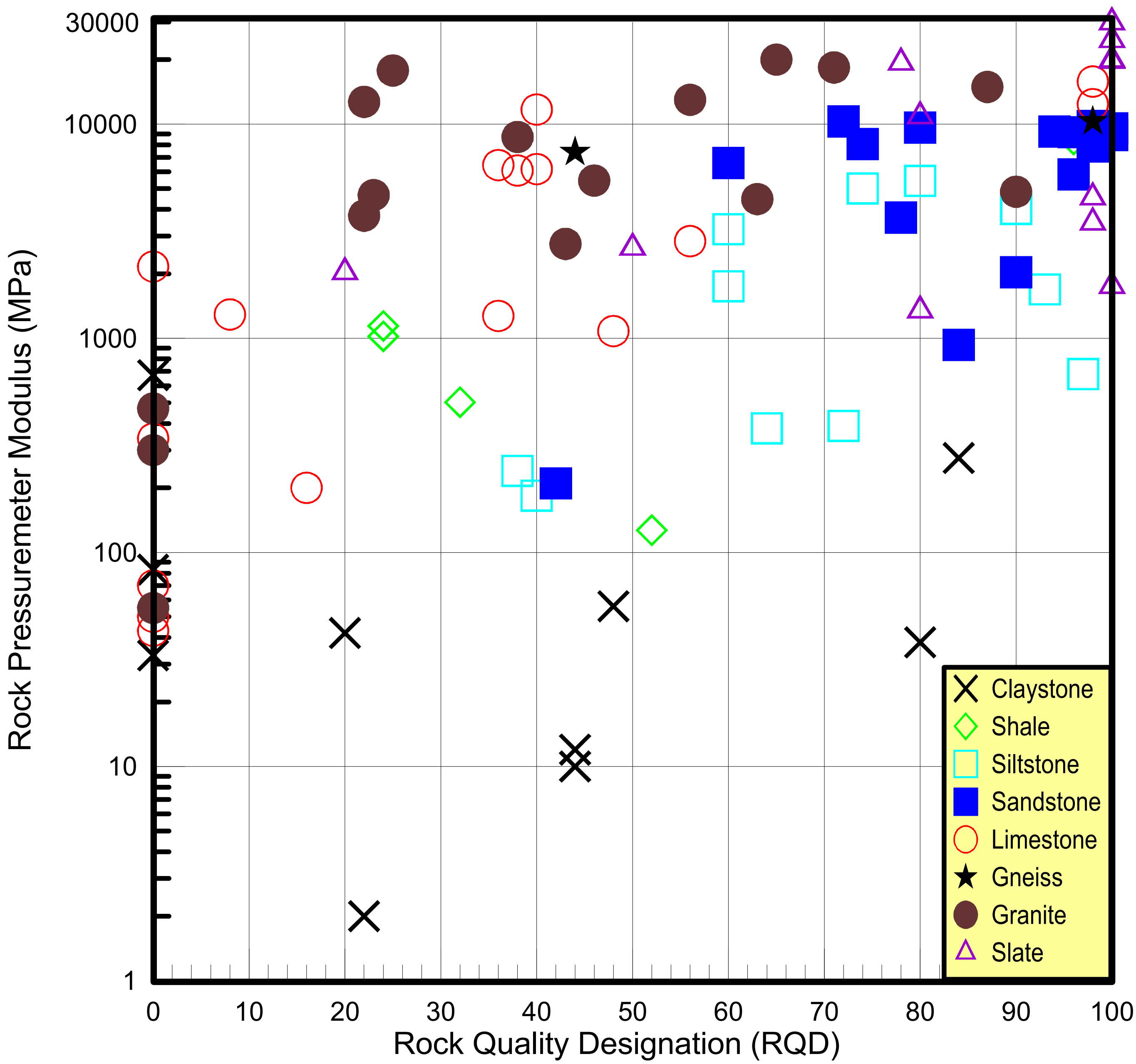

Figure 2. Pressuremeter rock modulus versus RQD comparison

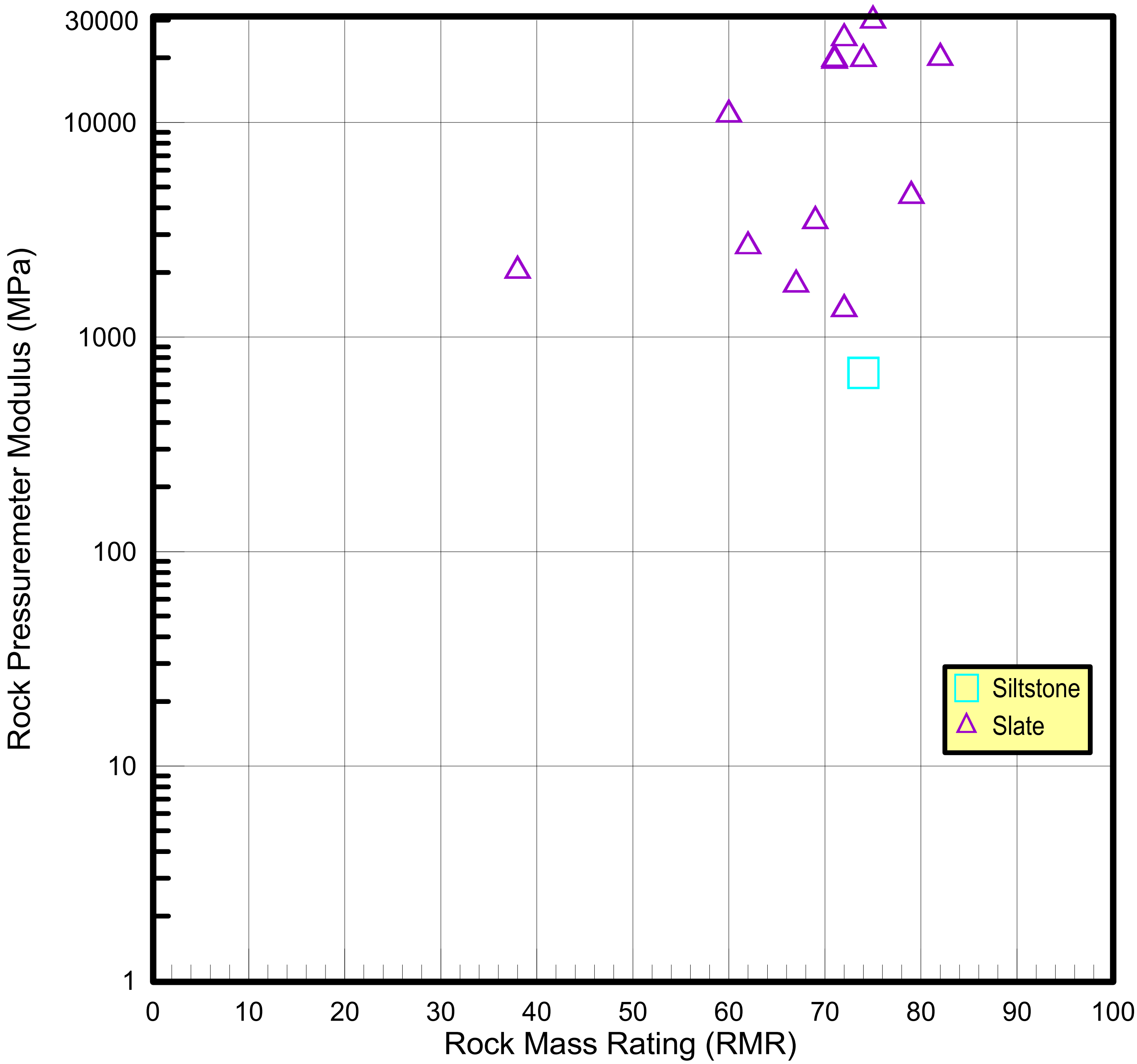

Figure 3. Pressuremeter rock modulus versus RMR comparison

5. Deep Foundation Case Study—Interstate I-95 Occoquan River

P-y curves were developed for the three different bedrock types underlying the project site for lateral deflection analyses of drilled shafts. The P-y curves were generated using a procedure outlined by Robertson, et al. (1985) and the results of 15 rock pressuremeter tests completed at the project site. Three curves were developed to encompass the differing levels of horizontal stiffness observed in the pressuremeter tests. The following general procedure was used to develop the custom P-y curves:

1. Three groups of rock were established based on differences in the rock mass modulus obtained from the pressuremeter tests and general rock classification, quality, and field observations. The groups are as follows:

Rock Classification	Rock Mass Modulus Range (MPa)	Average RQD	Average UC (MPa)	Average RMR	Comments
High Quality Slate	$E_m > 10,000$	94	60	74	Slate bedrock, fractures infrequent
Low Quality Slate	$1000 < E_m < 10,000$	78	26	64	Slate bedrock, fractures, brecciated zones, and slickensides more frequent, loss of water return common during drilling
Siltstone	$E_m < 1000$	92	43	74	Siltstone bedrock

2. Re-interpret pressuremeter results in terms of radial strain and pressure. Convert to P-y curves by multiplying radial strain component by the pile half-width and multiplying the pressure component by the pile width. The pressure component of the curve is not multiplied by an α-factor.
3. Within each established group, average the slopes of the P-y curves and calculate the standard deviation of the group. The design slope of the P-y curve is set to the average value minus two standard deviations.
4. Apply a "limiting strain" value to each design P-y curve to account for the brittle behavior of the rock mass. The limiting value is chosen within each group as the strain at the yield pressure, where non-linear behavior of the rock mass was observed to begin.

For the purposes of lateral deflection analysis, soil and rock stratigraphy was interpreted at each of the bridge abutments and piers based on the most recent subsurface data available. Our design P-y curves are shown below.

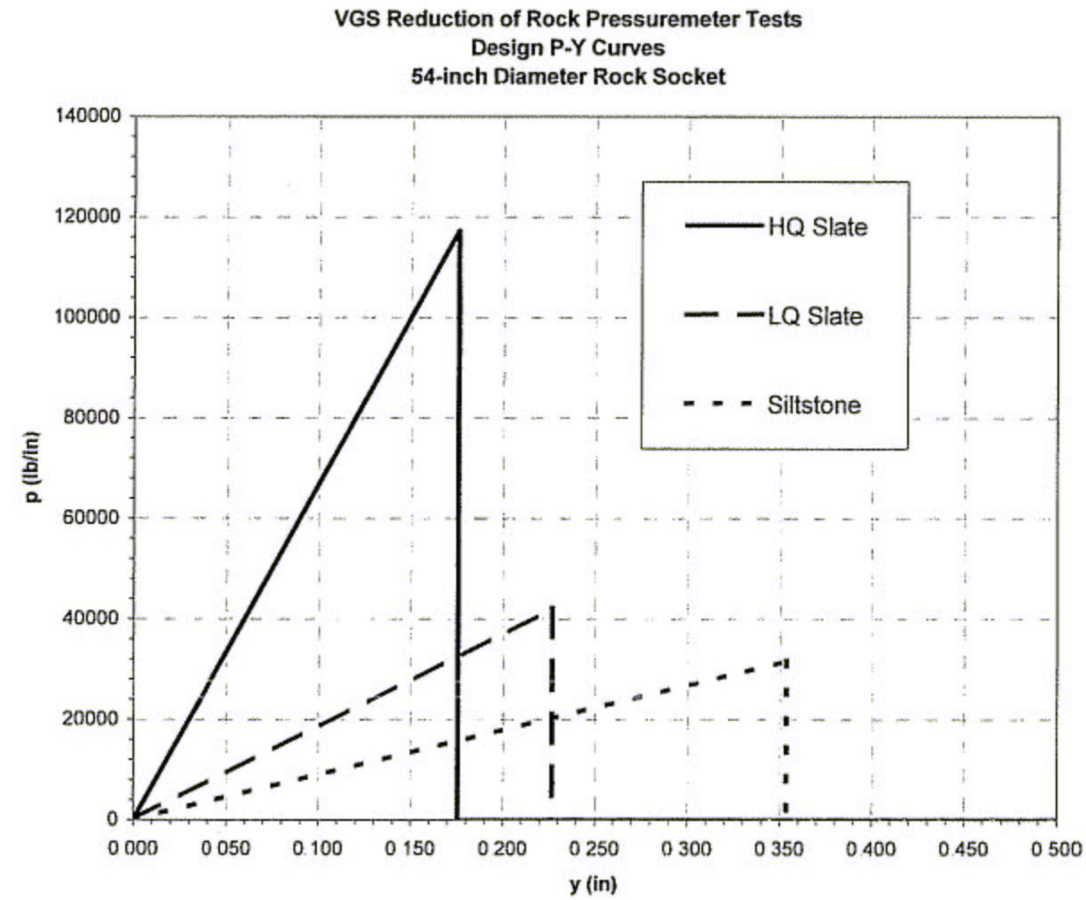

Figure 4. Design P-y curves based on rock pressuremeter tests

6. Conclusions

1. Pressuremeter rock modulus values can be successfully used to improve the lateral load design of drilled shafts in rock.

2. Rock pressuremeter tests impose sufficient strain on the rock mass to be considered indicative on rock mass behavior as opposed to “intact rock” behavior. The value of modulus determined from high pressure pressuremeter tests in rock is a reasonable representation of the “rock mass modulus” at the testing location and does not need to be further reduced.

3. Rock is quite heterogeneous. Design values for modulus should be the average value minus “x” number of standard deviations. We chose 2 standard deviations for our design, which gives a probability of 95.45% (normal distribution) that the modulus will be equal or greater than the design value.

4. Rock type was as important to modulus as RQD or RMR.

5. From our data set, we found no meaningful correlation between rock modulus and RQD.

6. Correlations may be possible between modulus and RMR, but our data set was limited.

7. References

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