

# An End Bearing Method for Evaluating Instrumented Becker Penetration Test Data for Site Characterization

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**ABSTRACT:** Becker Penetration Tests (BPT) and instrumented Becker Penetration Tests (iBPT) are regularly performed to characterize the density of embankment dams and foundations that consist of sandy gravel to gravelly soils and rockfill, where other methods such as the Standard Penetration Test (SPT) and the Cone Penetration Test (CPT) are difficult to perform. An embankment dam constructed in the 1950s was recently evaluated using BPT/iBPT and sonic borings, as well as earlier large diameter in-situ ring density tests and BPTs, original construction records, and observations and in-situ testing as part of a dam modification project. Three commonly used BPT and iBPT methods for conversion of BPT/iBPT blow counts to equivalent SPT  $N_{60}$  values were evaluated. Wide differences in the resulting equivalent SPT  $N_{60}$  values between (1) the BPT-based Harder and Seed (1986) method and the two instrumented BPT-based methods of (2) Sy and Campanella (1994) and (3) DeJong et al. (2017) and Ghafghazi et al. (2017) were observed in this project. This finding is consistent with several other projects that were reviewed by the project team. These would result in significantly different estimates of expected seismic performance of the dam. The availability of 34 in situ large diameter ring density tests, and corollary laboratory tests performed to determine in situ relative density ( $D_R$ ) from the ring density tests, as well as the well-documented emplacement and compaction history of the dam embankment, presented a valuable opportunity to examine and evaluate the three BPT interpretation methods. In the current project, the interpreted equivalent SPT  $N_{60}$  values and the resulting inferred in situ relative density ( $D_r$ ) values were compared with in situ relative density values that were determined based on (1) thirty four large diameter in-situ ring density tests, and (2) construction history. Based on an evaluation of origins and development of the three BPT and iBPT interpretation methods listed above, and further analyses of available data, a fourth alternative “end bearing” method to determine site-specific equivalent SPT  $N_{60}$  values from BPT was developed. This proposed method systematically and transparently analyzes the collected BPT instrumentation data and provides equivalent SPT  $N_{60}$  values considering in-situ soil characteristics; a significant improvement to commonly used procedures. In addition to site characterization for seismic analyses, this method can be used for deep foundation design and capacity verification and in selecting the most appropriate method for BPT to SPT  $N_{60}$  conversion. The end bearing method also supported selection of the Harder and Seed (1986) method as the most appropriate method for this current dam re-evaluation project.

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## INTRODUCTION

Becker Penetration Testing (BPT) is a widely used method for site characterization for coarse-grained soils with larger particles such as gravels and cobbles. Although the BPT had been used in Canada since the 1950s, the use of BPT-based blow counts for liquefaction assessment began in the mid-1980s, after the introduction of the Harder and Seed (1986) method. Since the 1980s, two additional methods that take advantage of dynamic instrumentation have been developed. These are (1) the Sy and Campanella (1994) method and (2) the DeJong et al. (2017) and the Ghafghazi et al. (2017) method [companion papers by the same research team]. As part of the current study, site characterization data from 10 different dams [owned and operated by different Federal and Local agencies] were evaluated to compare these three different BPT interpretation methods. It was found that in all of these dams, the two instrumented methods [(1) The Sy and Campanella (1994) method and (2) the DeJong et al. (2017) and Ghafghazi et al. (2017) method] predicted equivalent  $N_{60}$  values that were about two or more times higher than equivalent  $N_{60}$  values developed using the Harder and Seed (1986) method. These large differences in equivalent SPT  $N_{60}$  values can produce significantly different results in seismic analyses of liquefaction-related expected seismic performance for engineered structures such as dams, buildings, etc.

In a recent embankment dam project, significant differences between the results from the three commonly used BPT interpretation methods were found. In order to select an appropriate BPT method for the project, a detailed review of the results from these different methods was performed, including comparing results with (1) the well documented construction history and (2) data from large-diameter in-situ ring density testing. The significant differences between the three BPT interpretation methods found for this subject embankment dam, and in other projects, and the cross-comparisons with field emplacement procedures and with large diameter in-situ ring density test data, have led to ongoing further investigation of BPT (and BPT interpretation) procedures and practices. Based on an evaluation of the BPT process and utilizing (a) data obtained from instruments and (b) commonly used empirical relationships, a fourth alternative “end bearing” method to determine site-specific equivalent SPT  $N_{60}$  values was developed.

This paper presents the evaluations of the three currently available BPT interpretation methods, and the new end bearing method in five sections: (1) discussion of the strengths and limitations of the three commonly used BPT to SPT  $N_{60}$  conversion methods, (2) methodology for an alternative “End Bearing Method” for site characterization using instrumented BPT data, (3) a back-of-the-envelope  $N_{B30}$  to  $N_{60}$  correlation, (4) site characterization for the current dam re-evaluation project using previous 1980s and more recent 2019 site investigation studies using BPT and instrumented BPTs, and in-situ ring density test results (5) additional/alternate characterization of the key dam embankment zones using the end bearing method and comparison with other methods, (5) discussion of results of nonlinear seismic deformation analyses (NDA) using the Harder and Seed (1986) method and the End Bearing Method, and (6) conclusions regarding the limitations and constraints of the current BPT and instrumented BPT methods, and the key attributes of the end bearing method for site characterization and deep foundation design.

# 1. STRENGTHS AND LIMITATIONS OF THE THREE EXISTING BPT TO SPT INTERPRETATION METHODS

## (a) The Harder and Seed Method (1986):

Harder and Seed (1986) standardized the BPT equipment and procedures and developed a relationship to convert BPT blow counts per foot of penetration [corrected for bounce chamber pressure and elevation] to equivalent SPT  $N_{60}$  values. This method was based on a framework similar to that in use at that time for performing and interpreting SPT, as the bounce chamber pressure measurements provided a basis for essentially normalizing BPT hammer energy in much the same manner that SPT hammer energy was being more closely controlled or measured, and corrected for. Harder and Seed (1986) developed their relationship based on a study conducted at three free-field sites in soils consisting of Silty Sand to clean Sand. They carefully selected these Sandy sites, as they would provide reliable in-situ SPT  $N_{60}$  values that were not impacted by potentially significant gravelly content or coarse particles (which would increase blow counts) or clayey contents (which can tend to “lubricate” the penetration process and reduce blow counts).

Figure 1 shows the Harder and Seed (1986) relationship. The left-hand figure illustrates the relationship used to normalize measured BPT blow counts based on measured BPT bounce chamber pressures (as a proxy for hammer energy), and the right-hand figure shows the relationship between normalized BPT blowcounts ( $N_{BC}$ ) vs. SPT  $N_{60}$  values.

As the Harder and Seed (1986) method was developed using BPT and adjacent SPT borings to depths of up to about 50 feet, the data generally reflect relatively lower BPT shaft friction than would develop for deeper penetrations. The method does not include a systematic consideration of shaft resistance, and as a result, the interpreted SPT  $N_{60}$  values may be un-conservative or over-conservative; depending on the shaft resistance actually encountered, as compared to the shaft resistance implicit in the initial development of the relationship. Thus, a drawback of this method is the inability to directly account for, and correct for, different levels of shaft resistance in different materials and at different overall depths of penetration.

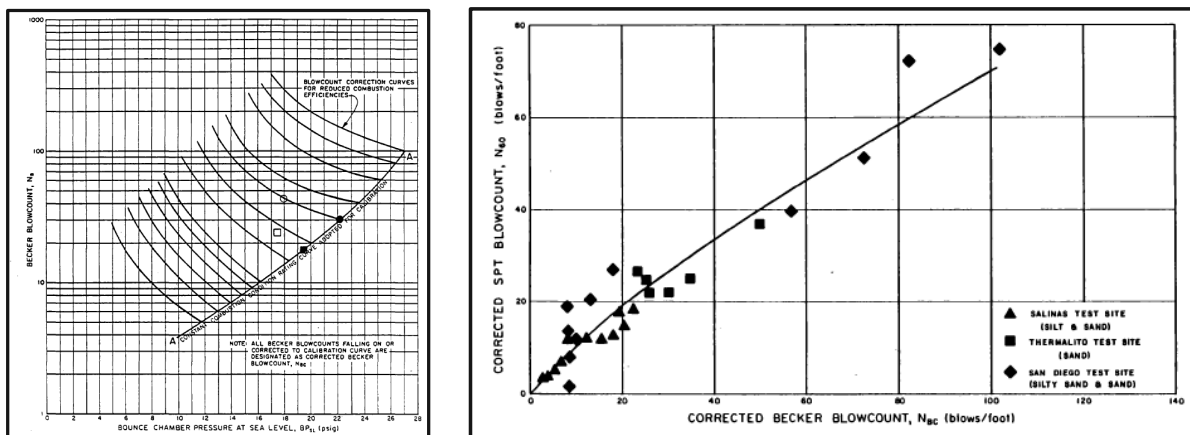


Figure 1: The Seed and Harder (1986) BPT to Equivalent SPT Interpretation Method

**(b) The Sy and Campanella Method (1994):**

The Sy and Campanella (1994) method is another of the three commonly used methods for BPT data collection and conversion to equivalent SPT  $N_{60}$  values. The Sy and Campanella method determines  $N_{B30}$  based on the measured BPT blow counts corrected to a reference maximum energy (ENTHRU) level of 30% of the manufacturer's rated energy for the ICE 180 hammer, as

$$N_{B30} = N_b \frac{ENTHRU}{30}$$

where  $N_{B30}$  is the BPT blow count normalized to 30% of the reference energy level,  $N_b$  is the measured blow count, and ENTHRU is the measured maximum energy expressed as the percentage of the rated hammer energy of 11.0 kJ. It should be noted that the term ENTHRU is generally defined in the pile instrumentation industry as the energy transferred to a pile top, whereas in Equation 1, the term ENTHRU instead refers to energy transfer ratio.

Recognizing that BPT blow counts are affected by shaft resistance, Sy and Campanella (1994) developed a set of curves to correct for shaft resistance when estimating equivalent SPT  $N_{60}$  values. These shaft resistance correction curves, shown in Figure 2, were developed using the following procedure:

- (1) CAPWAP signal matching analyses were performed for five single BPT hammer blows at five selected depths of a single BPT boring at the Annacis (British Columbia, Canada) test site. SPT and CPTs were also performed at locations adjacent to this BPT (BPT-B1).
- (2) Wave equation analyses using GRLWEAP (1992) were performed to match the CAPWAP signal matching analyses at the five selected depths. These five CAPWAP analyses allowed for calibration of the pipe and soil parameters for the wave equation analyses.
- (3) The shaft resistance component ( $R_s$ ) was then removed in the GRLWEAP-wave equation analyses to estimate BPT  $N_{B30}$  blow counts representing estimated BPT toe (or end) resistance for a "frictionless" pile. Sy and Campanella (1994) compared the resulting zero shaft resistance  $N_{B30}$  values with adjacent corrected SPT  $N_{60}$  blow counts and proposed a ratio of  $SPT N_{60} / BPT N_{B30} = 2.5$  for zero shaft resistance ( $R_s = 0$ ).
- (4) After developing the fundamental  $N_{60}/N_{B30}$  ratio for zero shaft resistance, correction curves for shaft resistance were then developed by performing GRLWEAP wave equation analyses for various assumed shaft resistance levels, i.e.  $R_s = 45, 90, 135, 180, 225, 270, 315,$  and  $360$  kN. The resulting shaft resistance correction curves are shown in Figure 2. Note that these curves are all based on the  $N_{60}/N_{B30} = 2.5$  ratio for zero shaft friction derived from the 5 CAPWAP dynamic wave equation analyses performed at the Annacis site.

In a number of recent projects from which data has been provided to this current evaluation team, the Sy and Campanella method (1994) results in significantly higher equivalent SPT  $N_{60}$  values than the Harder and Seed (1986) method; often by a factor of 2 or greater. In addition, an

evaluation of findings from a recent seismic dam evaluation project indicates that equivalent SPT  $N_{60}$  values from the Sy and Campanella (1994) also significantly overestimate  $N_{60}$  (and relative

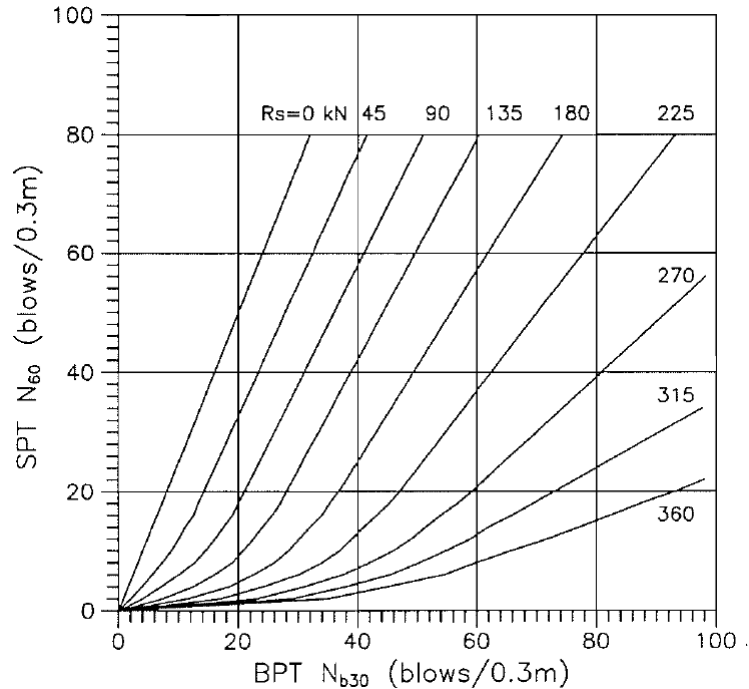


Figure 2. Relationship between BPT  $N_{B30}$  and SPT  $N_{60}$  for Different Levels of BPT Shaft Resistance ( $R_s$ ) (Sy and Campanella, 1994)

density) of the cohesionless dam embankment materials, when compared with both (1) well-documented embankment construction history and (2) data from a series of 34 site-specific large diameter in-situ ring density tests, with corollary laboratory testing to convert the ring density test data to values of in situ relative density ( $D_R$ ) (Chowdhury et al, 2020).

### Re-Analyses of the Sy and Campanella (1994) Annacis (British Columbia) BPT Field Test Site Data Using CAPWAP-RSA Methods

As described above, the basis of the Sy and Campanella method (1994) is field data from the Annacis site, where they performed side-by-side BPT, SPT, and CPT soundings. BPT toe and shaft resistance values were computed using single-blow CAPWAP signal matching analyses at 5 discrete depths, and these values were used to estimate an  $N_{60}$  versus  $N_{B30}$  relation for a hypothetical pile with zero shaft resistance ( $R_s = 0$ ). Their procedure is illustrated in Figure 3.

Unfortunately, at the time of Sy and Campanella's (1994) work, the modern CAPWAP technique of Residual Stress Analyses (RSA) (Holloway et al., 1979; Rausche et al. 2010), was not yet widely used. RSA involves applying a blow repeatedly to the CAPWAP model while iteratively varying model parameterization until convergence is reached. This technique is critical for accurate determination of toe resistance for structural foundation piles, and it also accurately differentiates the toe resistance from the "during driving" shaft resistance. Although the total pile

capacity (shaft resistance plus toe resistance) predicted by the earlier single-blow CAPWAP analyses can be close to CAPWAP-RSA analyses, the distribution of resistance between the shaft resistance and the toe resistance can be quite different. In the absence of the more modern multiple-blow RSA analyses, the single-blow CAPWAP analyses of Sy and Campanella (1994) for the Annacis site have potential to (1) overpredict the shaft resistance, and (2) underpredict the BPT toe resistance; and this error may have skewed their relationship between  $N_{60}$  and  $N_{B30}$ . As part of the current study, these five single blows were re-analyzed using CAPWAP-RSA multiple-blow analysis method to evaluate this hypothesis.

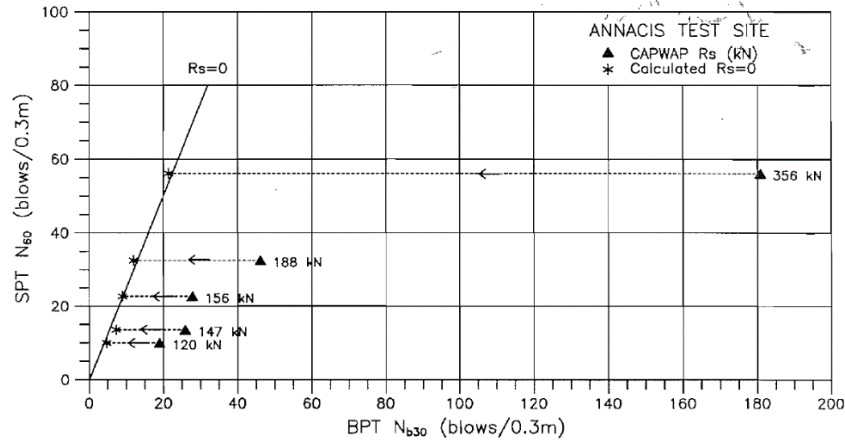


Figure 3: Effect of casing friction on BPT blow count to determine the zero friction line ( $R_s=0$  line); (Sy and Campanella, 1994)

As part of the current study, modern CAPWAP-RSA analyses for the Annacis site were performed using traces of the recorded pile head force and velocity digitized from figures in Sy (1993). The updated analyses in the current study were performed using modern residual stress analyses, which determine residual force, an important part of toe resistance (Rausche et al. 2010). The measured wave traces (blue lines) from Sy (1993) and the signal matching wave traces (red lines) from the updated CAPWAP-RSA analyses from the current study for the five blows from the Annacis site, are shown in Figure 4.

The results of this revised CAPWAP-RSA analyses for the Annacis site, are compared to the original Sy and Campanella analyses in Figure 5. It can be seen that the revised (updated) results have significantly higher toe resistances and lower shaft resistances. However, the total (overall) pile resistance (right panel) is very close to that of the Sy and Campanella (1994) analyses, providing a useful check on the digitization and signal matching process. As a further check on the revised CAPWAP-RSA analyses, the toe resistance is compared to an independent estimate based on the CPT sounding at the Annacis site, using the well-established Imperial College Pile (ICP) method for design of driven piles in sand (Jardine et al., 2005). The estimation of BPT toe resistance from CPT tip stress  $q_t$  will be discussed in greater detail in later sections. As shown in Figure 5, toe resistances from the revised CAPWAP-RSA analysis are in reasonably good agreement with the CPT-based estimates, while the original Sy and Campanella toe resistances are much lower than that the CPT-based estimates.

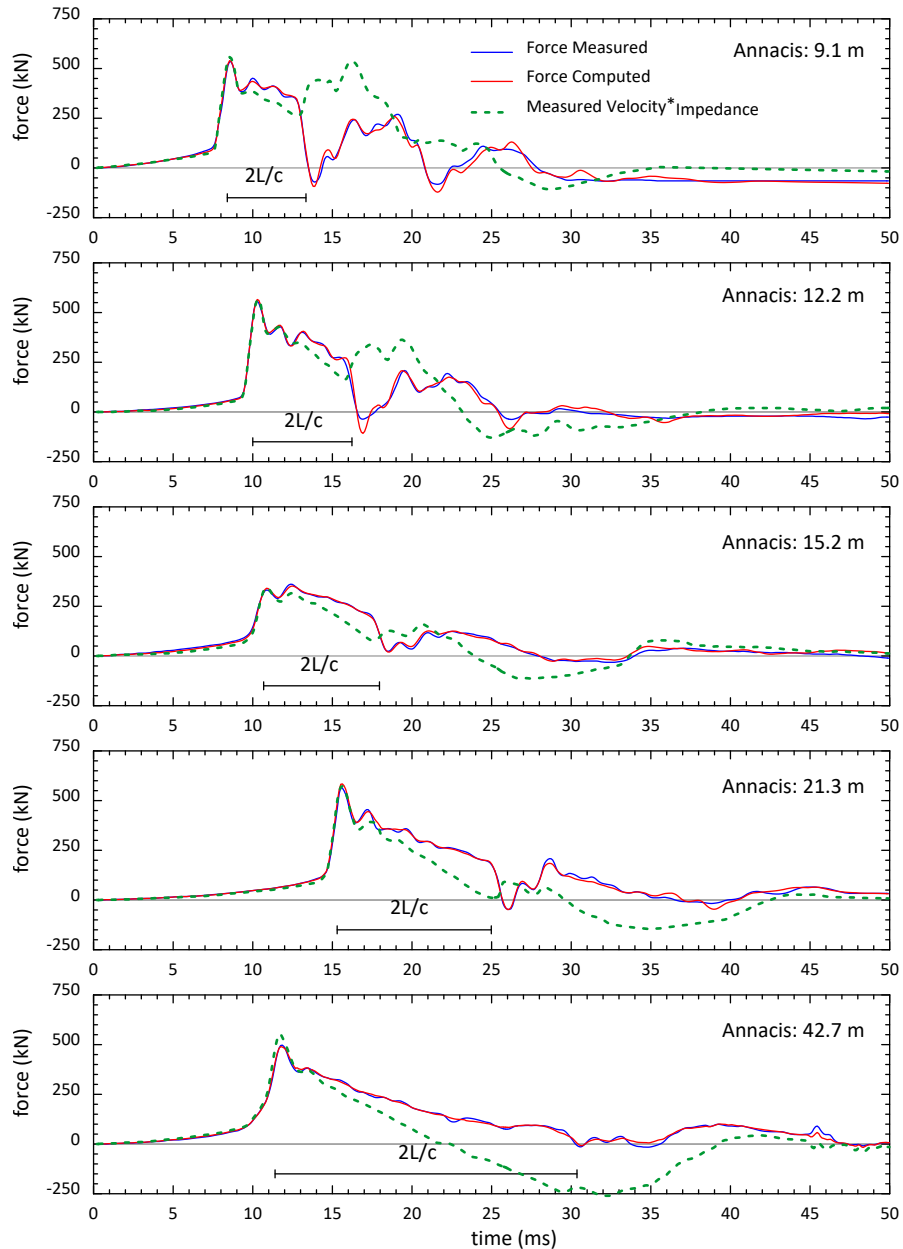


Figure 4: CAPWAP-RSA signal matching of BPT blows from Sy and Campanella (1994) Annacis (British Columbia, Canada) Site (performed for current study)

Using the revised shaft resistance estimates for the Annacis site, the Sy and Campanella (1994) estimated zero-shaft-resistance ( $R_s = 0$  kN)  $N_{60}$  versus  $N_{B30}$  relationship has been adjusted by simply scaling the  $N_{B30}$  offset for shaft friction by the ratio of revised to original shaft resistance. This approximate scaling process can be justified based on energy arguments. The adjusted results, illustrated in Figure 6, suggests an  $N_{60}/N_{B30}$  ratio of closer to 1.3 for the Annacis site, rather than the 2.5 ratio proposed by Sy and Campanella (1994). The implication of the shift of the revised  $N_{60}/N_{B30}$  line from 2.5 to 1.3 is significant, as it suggests that equivalent  $N_{60}$  values from the Sy and Campanella (1994) method could overestimate  $N_{60}$  by a factor of about

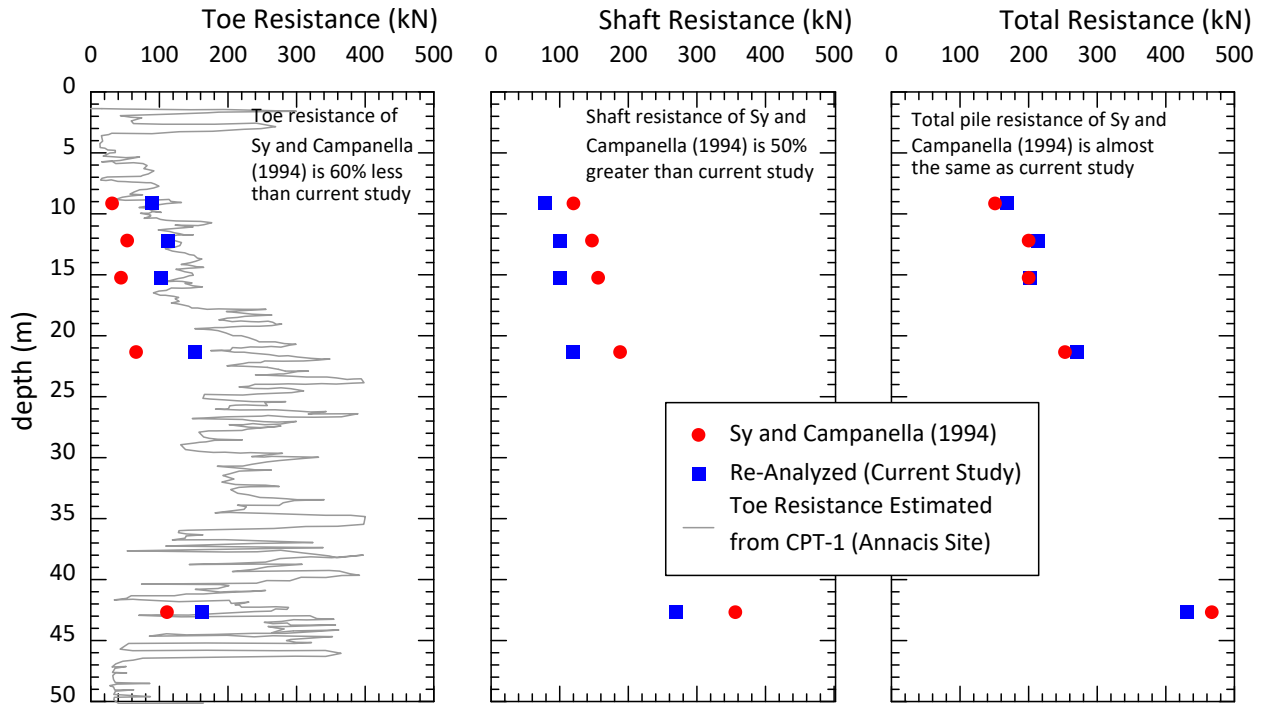


Figure 5: BPT toe resistance, shaft resistance and total resistance at Annacis Site: Original Sy and Campanella (1994) CAPWAP and re-analyzed CAPWAP-RSA (current study) comparison with adjacent CPT data from CPT-1

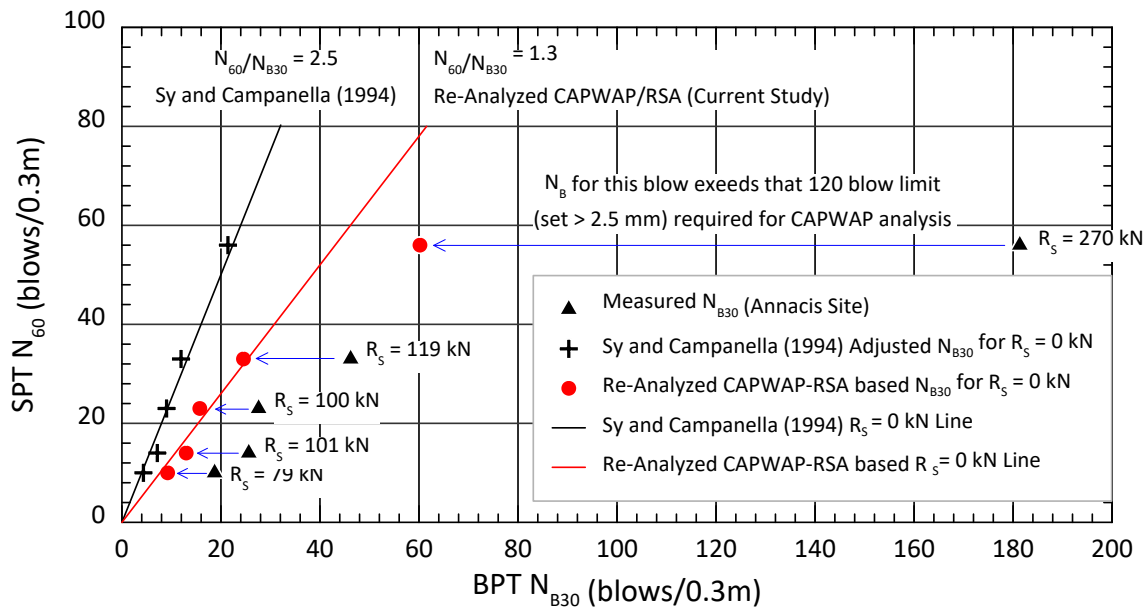


Figure 6: Adjusted zero-shaft friction BPT blow counts from Sy and Campanella (1994) for the Annacis Site data re-analyzed using CAPWAP-RSA methods (current study)

2 or so. This is consistent with the typical ratio of 2 or greater between the Sy and Campanella (1994) equivalent  $N_{60}$  values and the Harder and Seed (1986) equivalent  $N_{60}$  values, as found in several recent projects. It can be also noted that the revised  $N_{60}/N_{B30}$  ratio of 1.3 is closer to the



Harder and Seed (1986)  $N_{60}/N_{BC}$  ratio of 1.0, which includes shaft friction for the three test sites that were used to develop the relationship. With inclusion of “during driving” shaft resistance, the Sy and Campanella (1994) zero shaft resistance ( $R_s = 0$  kN) ratio of  $N_{60}/N_{BC} = 1.3$  would likely move in a direction somewhat closer to the Harder and Seed (1986)  $N_{60}/N_{BC}$  ratio of 1.0.

The following are observations regarding factors that may contribute to higher equivalent SPT  $N_{60}$  values that are produced using the Sy and Campanella (1994) BPT interpretation method for both low shaft friction and high shaft friction conditions. These observations have been developed based on revised CAPWAP-RSA analyses performed using the dynamic wave traces for the Annacis site from Sy (1993) and literature review.

Table 1 presents results of the toe resistance estimated by Sy and Campanella (1994) and results of the re-evaluation of tip resistance using (1) adjacent CPTs from Annacis site, and (2) revised CAPWAP analyses. The estimated toe resistances from (1) CPT and (2) re-analyses using updated CAPWAP and RSA are in good agreement with each other, however, these values are 45% to 67% higher than toe resistance estimated by Sy and Campanella (1994).

While CAPWAP analyses can provide accurate overall pile capacity estimates, RSA is necessary to accurately distinguish between (1) toe resistance and (2) shaft resistance. RSA typically requires more calculation effort, and CAPWAP-RSA was in its infancy in the early 1990s when the Sy and Campanella (1994) method was developed. If residual force is not properly accounted for, it may result in significant underestimation of tip resistance. While the total capacity calculation is now, and has been for many years been reliably calculated, small differences in the match quality of the signal matching process can make a potentially significant difference between more or less accurate resistance distribution results, particularly near the pile toe. The recent state of practice in CAPWAP and RSA has evolved by taking advantage of the powerful computing systems, which now facilitate more thorough search for the best wave-matching results in a short time.

The failure to perform RSA to properly evaluate residual force (or locked-in force) was an important factor introducing both inaccuracy and uncertainty into the Sy and Campanella work. The magnitude of this error cannot be fully quantified, based on the data as published, but it could be as high as the difference between the CPT-based tip resistance and tip resistance determined by Sy and Campanella (1994). The impact of this omission of locked-in force would be to under-predict BPT end bearing or tip resistance. Relationships based on these underpredicted toe values would result in development of systematically higher equivalent SPT- $N_{60}$  for forward analyses, which would be unconservative. It is also important to understand the limitations of signal matching when separating side friction and end bearing, especially near the tip of the BPT pipe pile.

Table 2 presents (1) results of the shaft resistance estimated by Sy and Campanella (1994) and (2) results of the re-evaluation of shaft resistance using revised CAPWAP and RSA analyses. The estimated shaft resistances from re-analyses using CAPWAP and RSA are 31% to 65% lower than shaft resistances estimated by Sy and Campanella (1994).

Table 1: Comparison of toe resistance estimates of Sy and Campanella (1994) vs. estimates from adjacent CPT and from re-analyzed CAPWAP-RSA: Anacis Site

Toe resistance from Sy and Campanella (1994) Table 1			Toe resistance from adjacent CPT and re-analyzed CAPWAP-RSA using digitized wave traces			
Pile embedment (m)	Toe resistance (kN)	Measured BPT $N_{b30}$	BPT Toe resistance estimated from Annacis CPT (kN)	Toe-resistance under-prediction of Sy and Campanella (1994) compared to adjacent CPT [2] versus [4]	Toe resistance from re-analyzed CAPWAP-RSA (kN)	Toe resistance under-prediction of Sy and Campanella (1994) Compared to CAPWAP-RSA [2] versus [6]
[1]	[2]	[3]	[4]	[5]	[6]	[7]
9.1	31	19	95	67%	89	65%
12.2	53	26	120	56%	113	53%
15.2	44	28	130	66%	102	57%
21.3	66	46	175	62%	152	57%
42.7	111	181	200	45%	162	31%

Table 2. Comparison of shaft resistance estimates of Sy and Campanella (1994) to estimates from re-analyzed CAPWAP-RSA: Annacis Site

Shaft resistance from Sy and Campanella (1994) Table 1				Shaft resistance from adjacent CPT and re-analyzed CAPWAP-RSA using digitized wave traces	
Pile embedment (m)	Shaft resistance (kN)	Total resistance (kN)	Measured BPT $N_{b30}$	Shaft resistance from re-analyzed CAPWAP-RSA (kN)	Shaft resistance over-prediction of Sy and Campanella (1994) Compared to CAPWAP-RSA [2] versus [5]
[1]	[2]	[3]	[4]	[5]	[6]
9.1	120	151	19	79	51%
12.2	147	200	26	101	46%
15.2	156	200	28	100	56%
21.3	188	254	46	119	69%
42.7	356	467	181	270	86%

The potential impact of (1) the over-prediction of the total shaft resistance, and (2) under-prediction of the end bearing can be significant. As discussed, the zero shaft friction line of Sy and Campanella (1994) ( $N_{60}/N_{B30} = 2.5$ ) is significantly steeper than zero friction line based on re-evaluation as part of this study ( $N_{60}/N_{B30} = 1.3$ ). For example, a BPT value of  $N_{B30} = 20$  blows/ft

using the Sy and Campanella (1994) method would result in equivalent SPT value of  $N_{60} = 50$  blows/ft for the  $R_s = 0$  condition, while the current study indicates the equivalent SPT  $N_{60}$  value would be approximately 26 blows/ft.

**(c) The DeJong et al. (2017) and Ghafghazi et al. (2017) Method:**

DeJong et al. (2017) and Ghafghazi et al. (2017) developed a third procedure for BPT data collection utilizing instrumentation placed at the toe of a BPT pile. The use of instrumentation data at the toe is advantageous as (a) the data are not as influenced by the shaft resistance and (b) some dynamic wave-related analytical complexities and/or uncertainties can be reduced. DeJong et al. (2017) use residual energy at the tip to calculate  $N_{B30}$  values from toe-instrumented BPT data, and then use Figure 7 to determine equivalent SPT  $N_{60}$  in accordance with the Ghafghazi et al. (2017) relationship, which recommends equivalent SPT  $N_{60} = 1.8 N_{B30}$ . The DeJong et al. (2017) and Ghafghazi et al. (2017) method (one method, with two sets of reference publications) were developed based on field investigations at four embankment dams owned by the Los Angeles Department of Water and Power (LADWP). DeJong et al. (2017) define  $N_{B30}$  as follows:

$$N_{B30} = N_B \frac{E_{res,TIP}}{30 (\%)}$$

where  $E_{res,TIP}$  = residual energy transferred to the instrumented section above the BPT drill string tip at the end of each blow, expressed as a percentage of the rated ICE 180 hammer energy (10.85 kJ), and normalized to a reference 30% hammer energy efficiency.

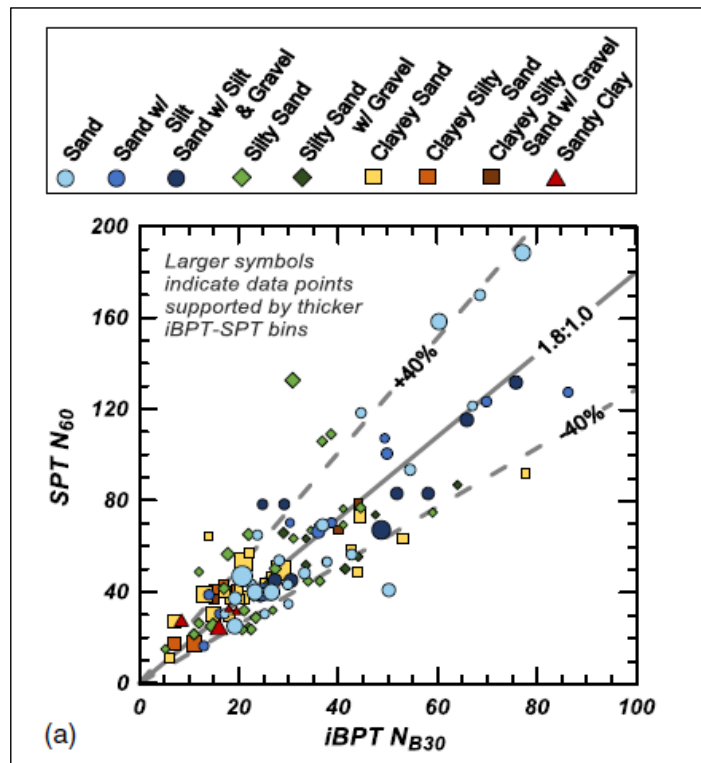


Figure 7: Relationship between BPT  $N_{B30}$  and SPT  $N_{60}$  using the Ghafghazi et al. (2017) method

As discussed previously, the Sy and Campanella (1994) method has limitations which may result in higher than actual equivalent SPT  $N_{60}$  values. However, the DeJong et al. (2017) and the Ghafghazi et al. (2017) method also results in similarly higher equivalent SPT  $N_{60}$  blow counts (Chowdhury et al., 2020). Chowdhury et al. (2020) have found that the equivalent SPT- $N_{60}$  values for coarse-grained soils from the Sy and Campanella (1994) and the DeJong et al. (2017) and the Ghafghazi et al. (2017) methods are generally relatively close to each other with regard to interpreted equivalent SPT  $N_{60}$  values. The previous section discussed reasons why the Sy and Campanella method appears to produce high  $N_{60}$  values. This section will examine the DeJong et al. (2017) and Ghafghazi et al. (2017) method.

The DeJong et al. (2017) and Ghafghazi et al. (2017) method may produce similar interpreted  $N_{60}$  results as the Sy and Campanella method, but for different reasons. These two instrumented methods utilize different definitions and relationships. As part of this study, a review of the development and data collection and analysis of the DeJong et al. (2017) and the Ghafghazi et al. (2017) procedure was performed. The following are observations regarding factors that may contribute to higher equivalent SPT  $N_{60}$  values using the DeJong et al. (2017) and Ghafghazi et al. (2017) method.

**a. Large scatter in the BPT to SPT relationship, and selection of site materials tested:**

As shown in Figure 7, there is large scatter in the BPT  $N_{B30}$  versus SPT  $N_{60}$  plot. The inset in this figure indicates symbols used to represent different soil types. The soil types used in developing the Ghafghazi et al. (2017) relationship ranged from clayey soils to silty and sandy soils to gravelly soils. Data included in Figure 7 fall largely within a cone between +60% to -60% above and below the selected 1.8:1 line [multiply and divide 1.8:1 slope line by 1.6]. This appears to reflect, in part, effects of SPT data influenced by differing soil types ranging from gravelly soils [which may increase nearby measured SPT blow counts] to clayey soils [which may reduce measured nearby SPT  $N_{60}$  blows]. The full scatter seen in Figure 7 suggests that a single straight-line relationship for all soil types is not appropriate. The full scatter would be within a cone of +240% and minus 200% from the 1.8:1 line selected as the basis for the interpretive BPT  $N_{B30}$  to equivalent SPT  $N_{60}$  relationship recommended.

As discussed previously, the Harder and Seed (1986) BPT  $N_{B30}$  to equivalent SPT  $N_{60}$  relationship, and the Sy and Campanella (1994) BPT  $N_{B30}$  to equivalent SPT  $N_{60}$  relationship, were both developed based on side by side SPT and BPT testing in sandy and silty sand soils that did not (1) have potential issues with regard to effects of clayey fines on SPT  $N_{60}$  values measured or (2) have potential issues with regard to larger/coarser particles impeding SPT penetration (and thus increasing measured SPT penetration resistances).

As part of this current study, the exercise of developing the relationship shown in Figure 7 was repeated, but this time the four “classes” of materials representing “clayey” soil mixes in Figure 7 were omitted. Instead, only sandy and silty sand soil data points from the four LADWP sites have been included. Figure 8 shows a plot of this reduced set of only sand and silty sand data points used in developing the recommended equivalent SPT  $N_{60} = 1.8$  BPT  $N_{B30}$  relationship of Figure 7. There are 40 data points in Figure 8. It can be seen from Figure 8 that only 8 of these

40 sand to silty sand data points plot above the Ghafghazi et al. (2017) recommended relationship, with 32 out of 40 (80%) of the data points plotting below.

Based on an evaluation of Figure 8, it can be reasonably concluded that the recommended relationship would likely overestimate equivalent SPT  $N_{60}$  values for sandy to silty sand soils. The recommended relationship might be somewhat more appropriate for a data set that includes more cohesive/clayey soils, but those types of more cohesive soils are not so commonly the subject of BPT investigations.

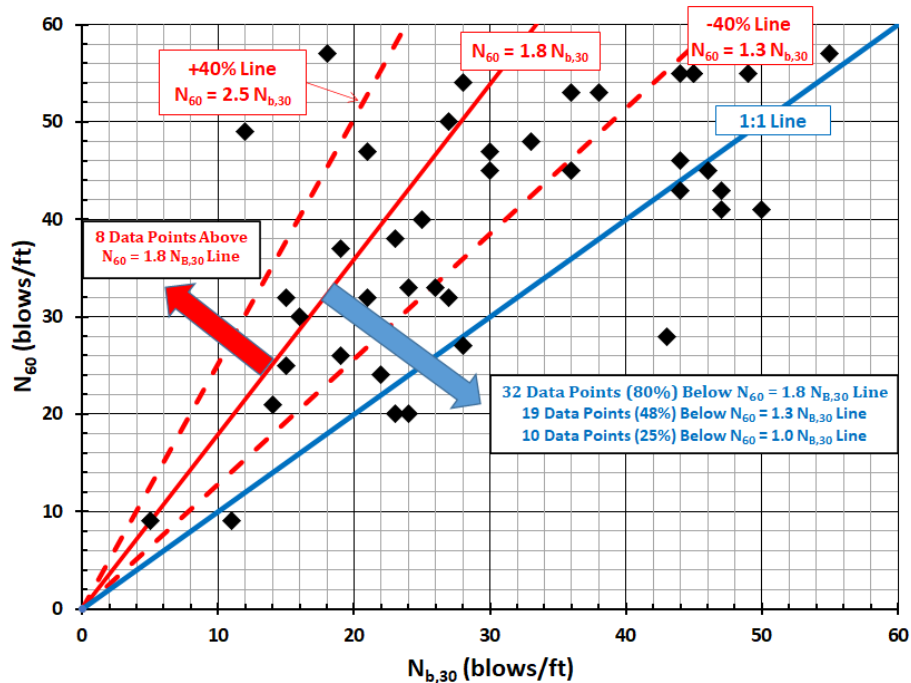


Figure 8: Forty (40) sand and silty sand data points from the DeJong et al. (2017) and Ghafghazi et al. (2017) database showing 82 percent of the data below the recommended  $N_{60} = 1.8 N_{B30}$  line.

**b. Uncertainty in data collection:**

DeJong et al. (2017) and Ghafghazi et al. (2017) correctly recognized the importance of residual force in proper determination of  $N_{B30}$  values. The DeJong et al. (2017) and Ghafgahzi et al. (2017) methods are dependent on direct measurements of residual force from the BPT pile tip. Such direct measurement of residual force is complex as it depends on driving conditions, temperature at pile tip, strain accumulation, drift, etc. At the time of this writing, the details of the instrumentation and procedures used for the residual force determination by the DeJong et al. (2017) method are not available for detailed examination, nor for industry-wide use. It was therefore not possible to evaluate these with regard to the accuracy and reliability of residual force determination. A possible additional reason for scatter between  $N_{B30}$  and SPT  $N_{60}$  values could be attributed to potential instrument errors in determining residual force. This should be clarified and/or investigated.

## **2. AN END BEARING METHOD FOR INSTRUMENTED BPT DATA EVALUATION AND CONVERSION TO EQUIVALENT SPT N<sub>60</sub>**

The significant differences between the results of the three BPT interpretation methods, and these cross-comparisons, have led to ongoing further investigation of BPT (and BPT interpretation) procedures and practices. Based on an evaluation of the instrumented data collection and evaluation techniques, constraints of the existing instrumented BPT methods, and common State of Practice site characterization relationships, an alternative method, identified as the “End Bearing Method” has been developed as part of this project. The basic approach of the End Bearing Method can be summarized as below:

Becker Penetration Testing is performed by advancing a 6.6-inches (16.8 cm) diameter closed end steel pipe pile. The instrumentation and monitoring of closed-end pipe piles during driving is a common practice in the deep foundation industry, and so also is the use of CAPWAP-RSA dynamic wave equation analyses. There are empirical relationships to convert CPT or SPT data to equivalent end bearing values for closed-end pipe piles. In an instrumented BPT data collection method, end bearing force at each depth can be measured directly. The end bearing forces at each depth can then be converted to an equivalent CPT or SPT values using commonly used and widely accepted empirical relationships.

The End Bearing Method (1) utilizes the direct end bearing measurement, (2) is not dependent on a limited database to develop a method for converting BPT/iBPT N<sub>B30</sub> values to equivalent SPT N<sub>60</sub> values, and (3) conversion to SPT N<sub>60</sub> can be performed using the commonly used soil characteristics-based relationships widely employed in conventional site characterization and pile design.

As more accurate determination of end bearing and shaft resistance is required in the end bearing method, use of instrumented BPT can be beneficial for deep foundation design along with site characterization.

The following are the steps of the End Bearing Method:

### Step 1: CAPWAP-RSA Signal Matching of Instrumented Becker Penetration Records

Dynamic load testing is a widely used technique to estimate bearing capacity of piles from dynamic measurements taken during pile driving. The preferred method for dynamic load testing is signal matching based on dynamic wave equation analyses, using programs such as CAPWAP. The key output obtained from dynamic load testing is the bearing capacity, including both end bearing and shaft resistance of the pile (Rausche et al., 2010).

CAPWAP–RSA signal matching is performed using force and velocity time histories recorded at the pile head during ram impact. These are complimentary quantities, related to each other by the pile characteristics and the soil parameters. A CAPWAP model is set up with the known physical properties of the pile and an initial assumed set of soil properties (end bearing and shaft resistance). An analysis is then performed using one of the measured quantities (typically velocity) as input and then calculating the complementary quantity (typically force) with the

analytical model. Soil properties are then adjusted iteratively so that the computed quantity matches its measured equivalent value. In this way, a complete self-consistent model for the non-linear dynamic response of the pile-soil system is obtained.

For CAPWAP-RSA analysis of instrumented Becker penetration test measurements, the instrumentation at the pile toe provides an additional set of velocity and force records that the model should also be able to match. These extra constraints on the soil properties give added confidence that an accurate, consistent model for the pile-soil system has been constructed. In the End Bearing Method, measured force and velocity is first used for CAPWAP analysis and the resulting toe resistance parameters together with the top measurements are utilized to determine the shaft resistance under consideration of the residual stresses in the pile. This second phase of the analysis also determines the additional residual resistance force at the pile toe. Note that both the measured force at the pile toe and the CAPWAP result, unless performed with residual stress consideration, misses the initial residual force component.

*Residual Force Determination.* The pile compresses during impact and then during rebound the shaft resistance prevents the pile's full extension, thereby maintaining a compressive force in the pile shaft and against the pile toe, even after the hammer blow is finished and the pile has come to rest (PDI, 2014). The residual force at the tip is calculated in CAPWAP by performing, after an initial analysis with zero initial conditions, consecutive analyses with initial conditions obtained from the end of the previous trial. Between the consecutive analyses an equilibrium condition is established. Once convergence in the residual values of consecutive analyses is achieved, typically after three or four trials, the analysis for one blow is finished [Rausche et al. (2010)]. This residual stress (RSA)-CAPWAP analysis is a well-established procedure in the deep foundation industry, and it is the basis of the end bearing method for instrumented BPT data interpretation.

Even though the total capacity of the pile (i.e. 6.6 inches diameter BPT pile) from each blow may be the same from CAPWAP analyses, an accurate distribution between shaft resistance and end bearing would require proper accounting for the residual force, which can be accomplished by the RSA procedure. The initial apparent shaft resistance will be reduced and the end bearing will be increased, after accounting for the residual force. As the purpose of the instrumented BPT is to determine the end bearing capacity of the pile, it is very important to properly account for the residual force. However, the magnitude of the residual force is site-condition dependent, and expected to be low in sites where during driving shaft resistance is low during driving (e.g. through looser soil layers such as some of the potentially liquefiable soils in the embankment dam site used in this study).

The use of CAPWAP analyses employing both toe and top measurements, and accounting for residual force by RSA analyses, results in significant improvement in estimating the end bearing values compared to methods that rely only on the energy of a single blow.

Step 2: Convert instrumented BPT end-bearing to equivalent CPT end-bearing, incorporating size Effects

The CAPWAP signal matching analysis in Step 1, extracts an end-bearing force from the dynamic instrumented BPT measurements. This force (converted to a stress) is then converted to an equivalent CPT tip stress. A driven BPT pipe pile tip or an SPT tip or a CPT tip each create a plunging cone of influence while driven or advanced through soil layers by mobilizing end bearing of soils. Recognizing this similarity between a driven pipe pile and a CPT, early CPT-based pile capacity methods for sands assumed direct equivalence between pile end-bearing stress and CPT tip stress (Meyerhof, 1976). However, more recent methods adjust the pile end-bearing stress to account for a size effect difference between the larger diameter pile and the smaller diameter CPT. Size effect relations were developed based on pile load testing results in different types of soils. For instance, the Imperial College Pile (ICP) method for driven piles in silica sand (Jardine et al., 2005) applies the relation

$$\frac{q_b}{q_c} = 1 - 0.5 \log(B/B_{CPT}) \geq 0.3$$

where  $q_b$  is the pile end bearing stress,  $q_c$  is the CPT tip bearing stress,  $B$  is the diameter of the pile and  $B_{CPT}$  is the diameter of the CPT probe. For the typical BPT diameter of 16.8 cm (6.6-inches) and the most common CPT cone size of 15 cm<sup>2</sup> with 43.7 mm diameter, the ratio of BPT end bearing stress to CPT tip stress is 0.71.

For the proposed end bearing method, the size effect relation is applied in the reverse order, as the CPT tip stress  $q_c$  is estimated from the instrumented BPT end bearing stress  $q_{BPT}$

$$q_c = [1/0.71] q_{BPT}$$

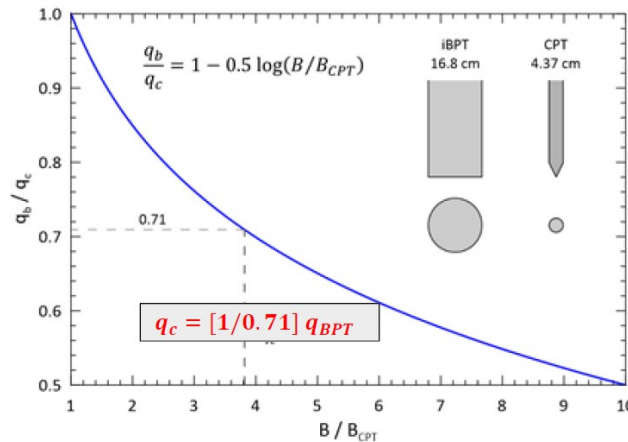


Figure 9: Jardine et al. (2005) relationship for CPT tip stress vs. tip stress for a driven closed-end pipe pile in sandy soils. The application of the formula to estimate CPT tip stress from BPT-tip resistance is illustrated.

It should be noted that the Jardine et al. (2005) method was used in the current project where the embankment shell materials consisted of sandy gravel to gravelly sand (and limited thickness of Silty Sand). Other driven pile to CPT tip resistance relationships may be appropriate for



different soil types. It is expected that the practitioners would utilize the appropriate driven pile to CPT tip resistance conversion methods considering appropriateness of the relationship for the site-specific conditions.

**Step 3:** Estimate Engineering Properties (SPT  $N_{60}$ , Relative Density, etc.) from CPT  $q_c$

CPT tip resistance ( $q_c$  values obtained from Step 2) can be utilized directly for engineering evaluations. However, SPT-based relationships are more commonly used to evaluate coarse-grained soils for embankment dams and other important structures (sometimes along with companion CPTs). In addition, comparison with currently used BPT interpretation methods requires an estimate of equivalent SPT- $N_{60}$  values. There are several commonly used relationships to convert CPT tip resistance ( $q_c$  values) to equivalent SPT  $N_{60}$  values. In the current study, the Robertson et al. (1986) and Kulhawy and Mayne (1990) methods has been used for conversion of CPT  $q_c$  to SPT  $N_{60}$ . Robertson et al. (1986) and Lunne et al. (1997) suggested  $(q_c/P_a)/N_{60}$  ratios for different soil types based on CPT soil behavior type. Table 2 presents  $(q_c/P_a)/N_{60}$  ratios proposed by Robertson et al. (1986) for soil behavior classification system from CPTU data.

Table 3: Robertson (1990) Soil Behavior Type Classifications and  $(q_c/P_a)/N_{60}$  Ratios from Robertson et al. (1986) and Lunne et al. (1997)

CPT Soil Zone	Soil Behavior Type Classification System from CPTU Data	$(q_c/P_a)/N_{60}$
1	Sensitive Fine Grained	2
2	Organic Materials	1
3	Clay	1
4	Silty Clay to Clay	1.5
5	Clayey Silt to Silty Clay	2
6	Sandy Silt to Clayey Silt	2.5
7	Silty Sand to Sandy Silt	3
8	Sand to Silty Sand	4
9	Sand	5
<b>10</b>	<b>Gravelly Sand to Sand</b>	<b>6</b>
11	Very Stiff Fine Grained	1
12	Sand to Clayey Sand	2

**Step 4:** Estimate Continuous End Bearing Profile Using the CASE Method

It may not be practical to perform CAPWAP analyses at all depths for deeper BPT borings. However, a continuous profile of end bearing force, and corresponding  $q_c$  and SPT  $N_{60}$ , can be developed using the CASE Method (Rausche et al., 1985). Rausche et al. (1985) have shown that if a pile is subjected to a sudden applied force, for which force velocity and time are measured, the soil resistance can be estimated from closed form equations. The following is a simplified explanation of the Case Method calculation procedure.

$$R_s(t) = (1 - J_c)F_{\downarrow}(t) + (1 + J_c)F_{\uparrow}(t + \frac{2L}{c})$$

where  $R_s(t)$  is the resistance at time  $t$ ,  $F_{\downarrow}(t)$  and  $F_{\uparrow}(t)$  are the measured downward and upward traveling waves at time  $t$ ,  $L$  is the pile length,  $c$  is the wave speed in the pile and  $J_c$  is the Case

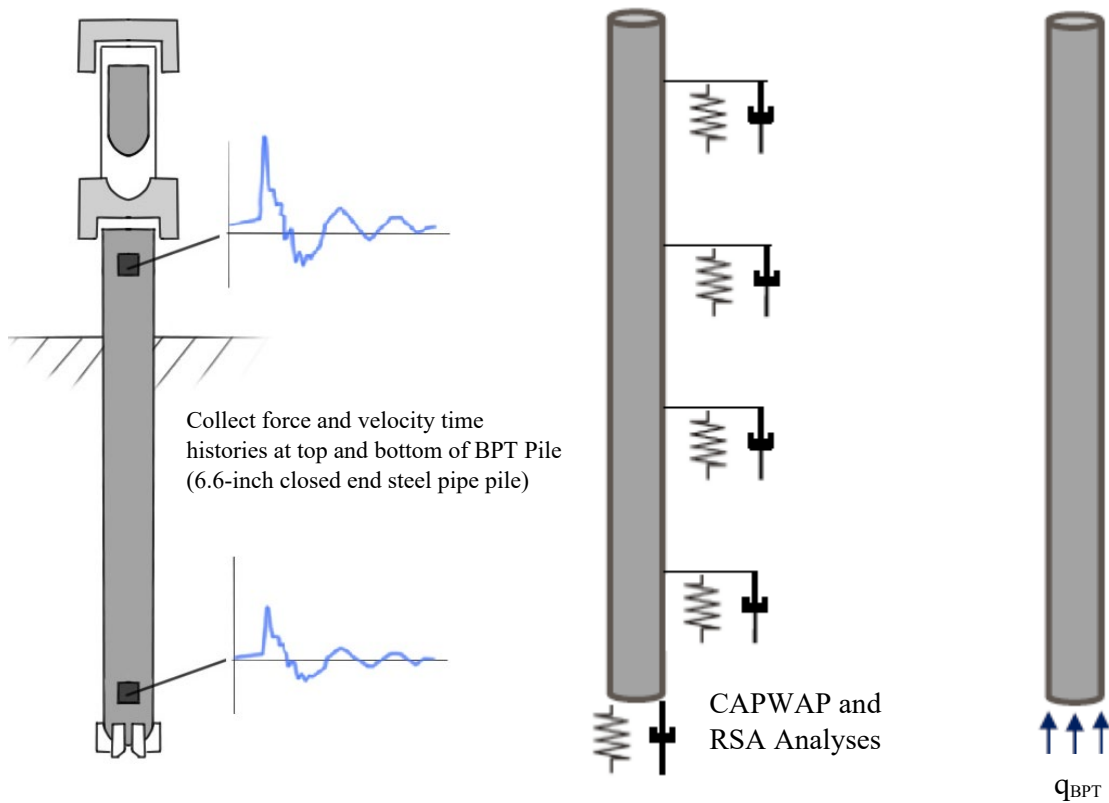
Method damping coefficient.  $R_{XJ}$  ( $R_{X0}$ ) is the maximum  $R_s$  value, determined by calculating  $R_s$  at all times after the first force and velocity peak for a non-zero  $J_c$  value ( $J_c=0$  value).  $SF_0$ , the shaft resistance with no damping, is estimated as 2 times  $F_{\uparrow}(t)$  at a time of  $2L/c$  after the onset of impact, i.e., immediately prior to end bearing activation.  $SFR$  is the shaft resistance considering damping. It is calculated by proportionality, i.e.,  $SFR = SF_0(R_{XJ}/R_{X0})$ . End bearing  $EBR$  is then  $R_{XJ} - SFR$ . Since the Case Method evaluation takes much less time than CAPWAP, the best estimate  $J_c$  value for a soil layer is obtained by comparison with the result from CAPWAP analysis and applied to records not analyzed by CAPWAP. Case Method and CAPWAP can be applied to measurements obtained both near the rod top and at the tip. Thus, when tip instrumentation is available,  $R_{XJ}$  equals  $EBR$  for the toe. The directly calculated end bearing by CAPWAP from tip measurements is used as a boundary condition for CAPWAP using top measurements. In this way, the CAPWAP shaft resistance calculation has the same reliability as the total CAPWAP capacity calculations. In the closed form equations for CASE method, the source of biggest uncertainty is the damping constant. In the BPT method, as more detailed CAPWAP-RSA based end bearing force values are available at selected depths, results from the CASE method can be compared to validate and select the damping values appropriate for the soil conditions encountered. In developing a continuous end bearing force profile for a BPT boring, steps 2 and 3 could be performed to determine engineering properties for subsurface layers.

Figure 10 presents a summary of the different steps of this proposed end bearing method to utilize instrumented BPT data to characterize soil conditions.

### **Recommendation for Performing BPT in High Shaft Resistance Conditions During Driving**

A limitation of the end bearing method as well as other instrumented or non-instrumented BPT methods can occur when refusal conditions are encountered in borings that develop high shaft friction values during driving. With the BPT exploration method, mobilization of the end bearing conditions of the soil is an important criterion to accurately determine toe resistance. Without activation or mobilization of the end bearing of the soil, the end bearing values using high blow counts could be erroneous.

In the pile driving industry, 2.5 mm or 0.1 inch is considered as the minimum pile displacement that is required to activate soil behavior from elastic to plastic (commonly known as quake for coarse-grained soils). This quake of 2.5 mm represents roughly 120 blows per foot. However, in BPT-based investigation, it is important to ensure that a high blow count is due to soil's resistance, not due to high shaft resistance restricting mobilization of end bearing of soil. The impact of high shaft resistance artificially restricting soil mobilization can be reduced by pulling out the BPT pile casing for few feet and re-driving. In some cases, a full pullout, followed by an open bit drilling to the required depth, and followed again by closed bit BPT drilling may allow accurate measurement of the end bearing capacity of soil. The end bearing method is a useful tool to evaluate the usefulness of the blows, as it systematically differentiates the “during driving” shaft resistance and the end bearing.



**Step 1:** Perform CAPWAP-RSA signal matching analysis, matching pile top and toe measurements to determine the “end bearing force” for the BPT pile tip ( $q_{BPT}$ )

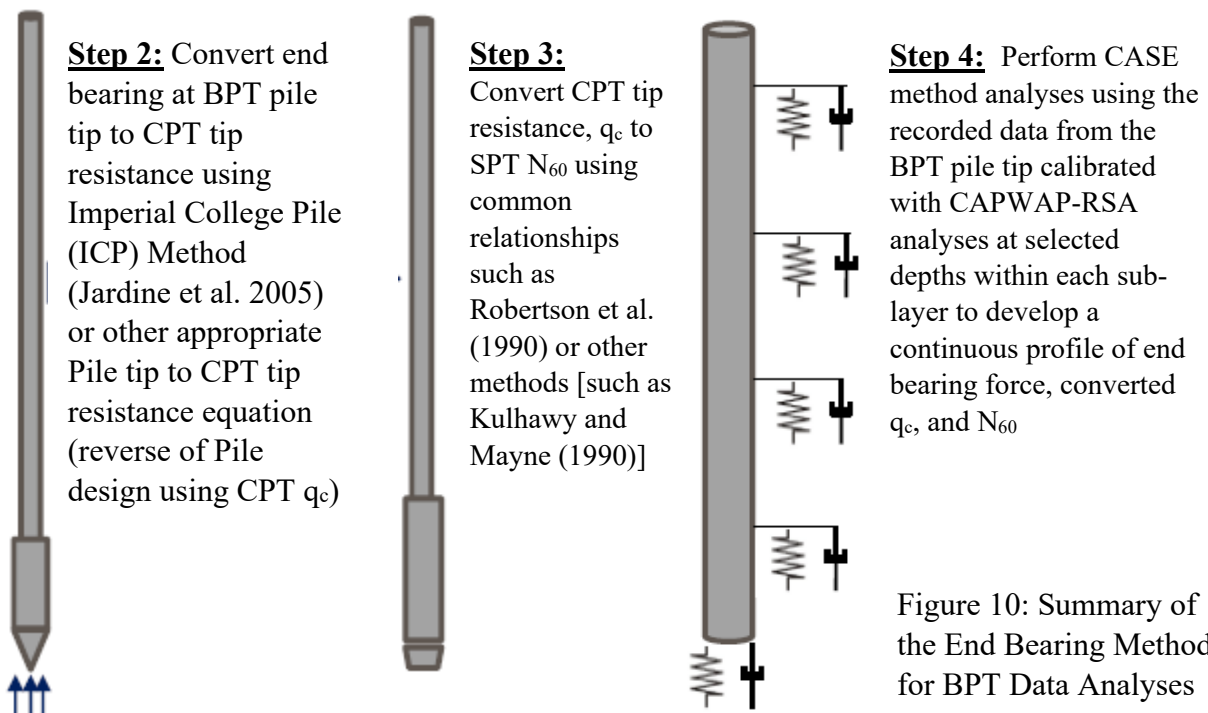


Figure 10: Summary of the End Bearing Method for BPT Data Analyses

### 3. BACK-OF-THE-ENVELOPE BPT- $N_{B30}$ to SPT- $N_{60}$ CORRELATION

Using the ideas behind the end bearing method, it is possible to make an estimate of the expected correlation between  $N_{B30}$  and  $N_{60}$ . This exercise is helpful in interpreting field trials as it suggests a range of values likely, and identifies factors expected to affect the correlation. An example of this back-of-the-envelope estimate is summarized in Table 4 below.

$N_{B30}$  as defined by DeJong et al. (2017) is an energy-normalized BPT blow count such that the residual or final work done by the pile tip during each blow is 30% of the nominal rated energy of an ICE 180 hammer. The symbol  $E_{30}$  has been utilized for this work [ $E_{30} = 3.3$  kJ (2.4 kip-ft)]. Under good driving conditions (the permanent displacement of each blow is not too small) most of this toe work is done by the displacement of the pile tip advancing against the static end bearing force, with the remaining work done by dynamic effects. If a static work (i.e. energy) ratio  $\beta$  is defined as the fraction of the total toe work done by the static end bearing force [as shown in Figure 11 example], then the energy balance at the toe for one blow is:

$$\beta E_{30} = Q_{BPT} (L_N / N_{B30})$$

where  $Q_{BPT}$  is the end bearing force and the displacement of the pile tip is  $L_N / N_{B30}$ . Here  $L_N$  is the length over which blows are counted ( $L_N = 1$  ft or 0.3 m). The end bearing force can then be expressed in terms of  $N_{B30}$  as

$$Q_{BPT} = \beta E_{30} N_{B30} / L_N$$

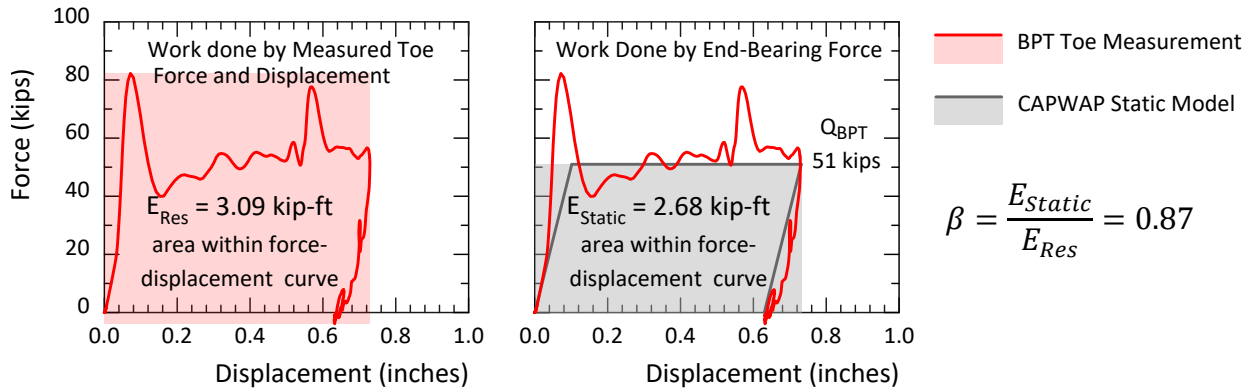


Figure 11: Computation of the static work (energy) ratio  $\beta$  for a BPT blow. The end bearing force is estimated from a CAPWAP analysis. Blow at 47-ft depth, BPT US-1.

The end bearing stress  $q_{BPT}$  is computed by dividing by the area of the pile tip (0.239 ft<sup>2</sup> or 0.0222 m<sup>2</sup>). An equivalent CPT tip stress  $q_c$  is then obtained by multiplying the pile tip stress by the Imperial College Pile (ICP) size effect parameter (e.g. 1/0.71 for silica sands). Finally, the equivalent CPT tip stress can be converted to an estimated SPT  $N_{60}$  value using the correlation of Robertson et al. (1990), which is a function of soil type. This approximate exercise suggests that using a single value of  $N_{60}/N_{B30}$  for all soil types is not appropriate. It also suggests that a correlation between  $N_{B30}$  vs. CPT  $q_c$  might be less noisy. Table 4 and Figure 12 have been

developed for 4 different soil types for  $\beta$  values of 0.75 to 0.95, a range developed based on an evaluation of data from the current study for  $N_{B30} \leq 60$ .

Table 4: Back-of-the-Envelope  $N_{B30}$ - $N_{60}$  Correlation to Estimate Expected Values

Soil Type	Static Energy Ratio $\beta$	BPT Pile Toe Stress $q_{BPT}/N_{B30}$ (ksf)	ICP Size Effect $q_c/q_{BPT}$	CPT Tip Stress $q_c/N_{B30}$ (ksf)	Robertson et al. (1990) CPT-SPT Factor $(q_c/P_a)/N_{60}$	$N_{60}/N_{B30}$
Gravelly Sand to Sand	0.75 - 0.95	7.5 - 9.5	1/0.71	10.6 - 13.4	6	0.8 - 1.1
Sand	0.75 - 0.95	7.5 - 9.5	1/0.71	10.6 - 13.4	5	1.0 - 1.3
Sand to Silty Sand	0.75 - 0.95	7.5 - 9.5	1/0.71	10.6 - 13.4	4	1.3 - 1.6
Silty Sand to Sandy Silt	0.75 - 0.95	7.5 - 9.5	1/0.71	10.6 - 13.4	3	1.7 - 2.1

NOTES: (1) The approximate  $N_{60}/N_{B30}$  values listed above apply only for  $N_B < 60$  blows/ft.  
(2) The ICP size effect and the  $(q_c/P_a)/N_{60}$  ratio are soil type dependent parameters.

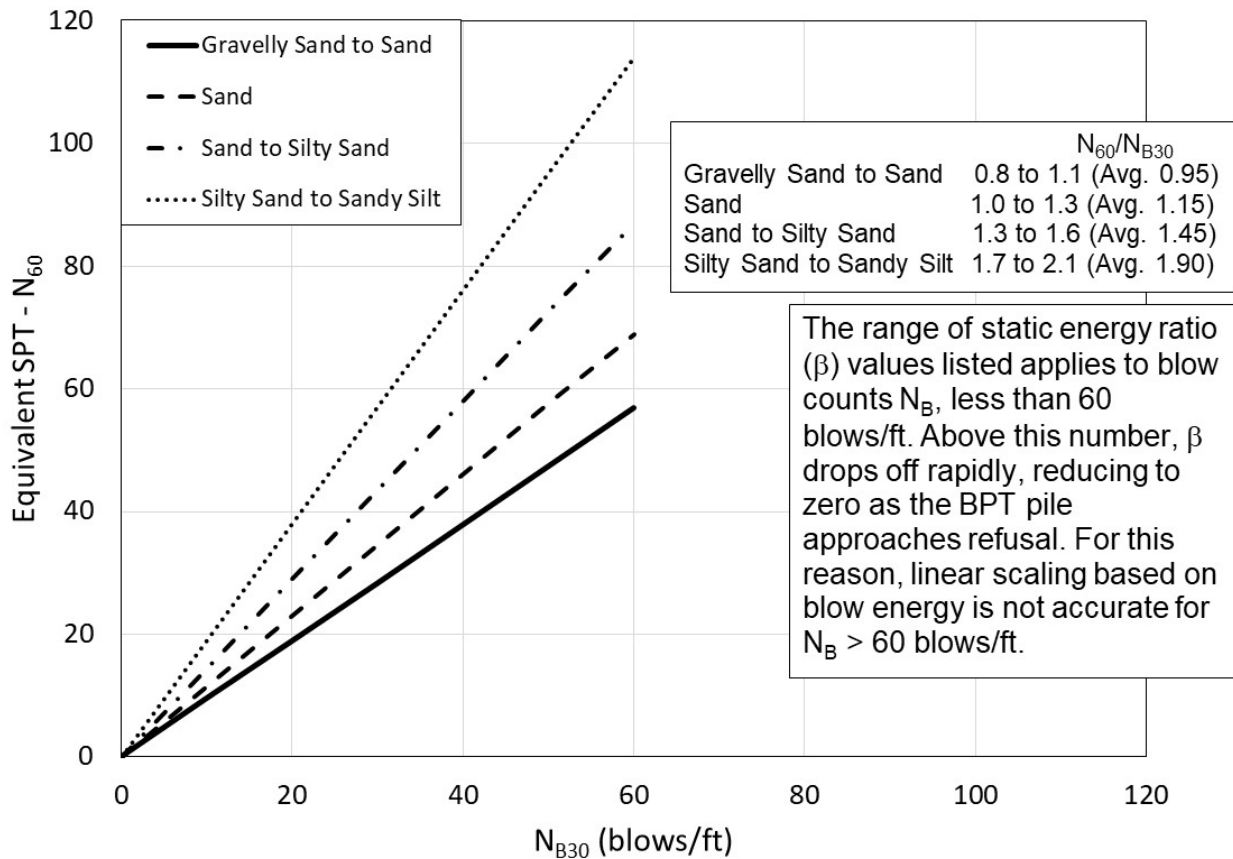


Figure 12:  $N_{B30}$  to  $N_{60}$  relationships for different coarse-grained soil types estimated using the back-of-the-envelope approximation

#### 4. SITE CHARACTERIZATION PART 1: INITIAL EVALUATION OF BPT AND IN-SITU RING DENSITY TEST RESULTS FROM A 1980s STUDY

A total of six BPT borings were performed in the 1980s in two areas on the downstream side of the dam under re-evaluation. The BPT blow counts were converted to equivalent SPT  $N_{60}$  values using the Harder and Seed (1986) method and then to SPT  $N_{1,60}$  values using the Seed et al. (1986) method. Figure 13(a) shows equivalent SPT  $N_{1,60}$  values from two of these 1980s BPTs located (longitudinally along the embankment) near the seismic analysis cross-section discussed in this paper. These two BPT's are located about 35 feet apart, and they showed similar trends and ranges in  $N_{1,60}$  values. Figure 13(b) shows equivalent relative density for these two BPT borings as estimated using the Terzaghi and Peck (1948) equation of  $D_R(\%) = \text{SQRT}[N_{1,60}/C_d]$ , where a  $C_d$  value of 60 was used for the sandy gravel to gravelly sand transition and shell zone materials considering Skempton (1986) for coarse sands, and Cubrinovski and Ishihara (1999) for cohesionless materials with  $e_{\max} - e_{\min} = 0.40$ , for an adjusted SPT hammer energy ratio of 60%.

These relative density values can be cross-compared with the results of the 34 large diameter (4-foot diameter) in situ ring density tests that were performed in the sandy gravel and gravelly sand materials of the shell and transition zones on the downstream side of the dam embankment in two different segments of the dam. Figure 14 shows the results of these large diameter in situ ring density tests. The left side figure shows the results of 16 in situ ring density tests performed on the embankment segment where the seismic deformation analyses presented in the paper by Chowdhury et al. (2020) were performed, and the figure on the right shows the results of 18 in situ density tests performed on a different segment of the dam embankment. The Geotechnical Laboratory of the USACE Waterways Experiment Station (WES) performed the corollary laboratory tests for determining the maximum density (and  $e_{\min}$ ) and minimum density (and  $e_{\max}$ ) for purposes of determining relative density ( $D_R$ ). The Maximum density tests were performed using both impact and vibratory compaction applied to samples in a range of mold sizes from 11 inches to 36 inches in diameter. Soil particles greater than 3 inches in size were scalped for this maximum and minimum density laboratory testing.

The two 1980s BPT borings shown in Figure 13 show the presence of several loose sub-layers, which is consistent with both construction history and the results of the in situ large diameter ring density tests.

An important additional (third) cross-check of the BPT-based  $N_{1,60}$  values, and the values from the 34 in situ ring density tests, was made based on the well-documented field placement and compaction history. The cohesionless shell and transition zone materials were loosely dumped and spread without significant deliberate compactive effort, just one full "levelling" pass by a D-8 Caterpillar dozer. Based on this, the engineering team and the board of consultants concluded that the expected resulting densities would be mainly between  $D_R \approx 40\%$  to  $65\%$ , with some denser values resulting from (irregular) over-passage of construction equipment traffic delivering and spreading fill materials; values of  $D_R$  less than  $30\%$  would be rare, and values of  $D_R$  greater than  $90\%$  would also be rare in the absence of significant deliberative vibratory compactive effort. Figures 14(a) and (b) are annotated with dashed red lines indicting the range of  $D_R = 40\%$  to  $65\%$ , and  $D_R = 90\%$ , corresponding to these estimates. As shown in Figure 14, the results of the large diameter in situ ring density tests correlate well with the field placement history. This is important,

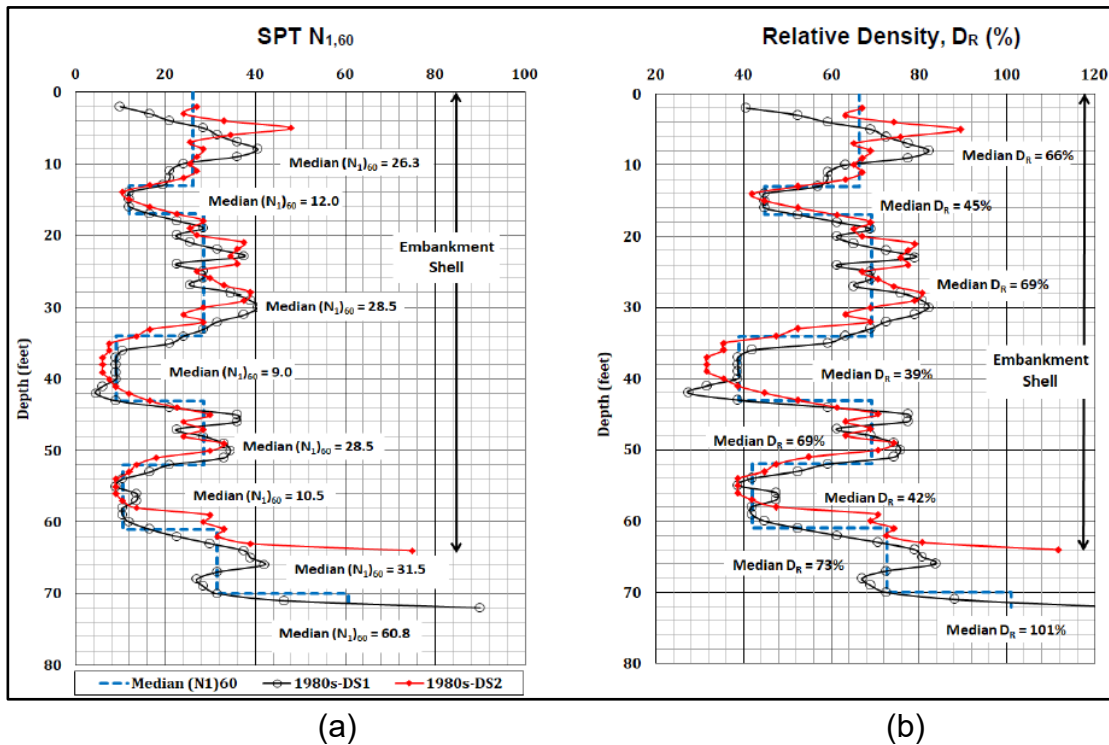


Figure 13: Equivalent SPT  $N_{1,60}$  profiles from two 1980's U/S BPT borings using the Harder and Seed (1986) method and relative density ( $D_R$ ) using  $C_d = 60$  for coarse-grained sandy gravelly soils

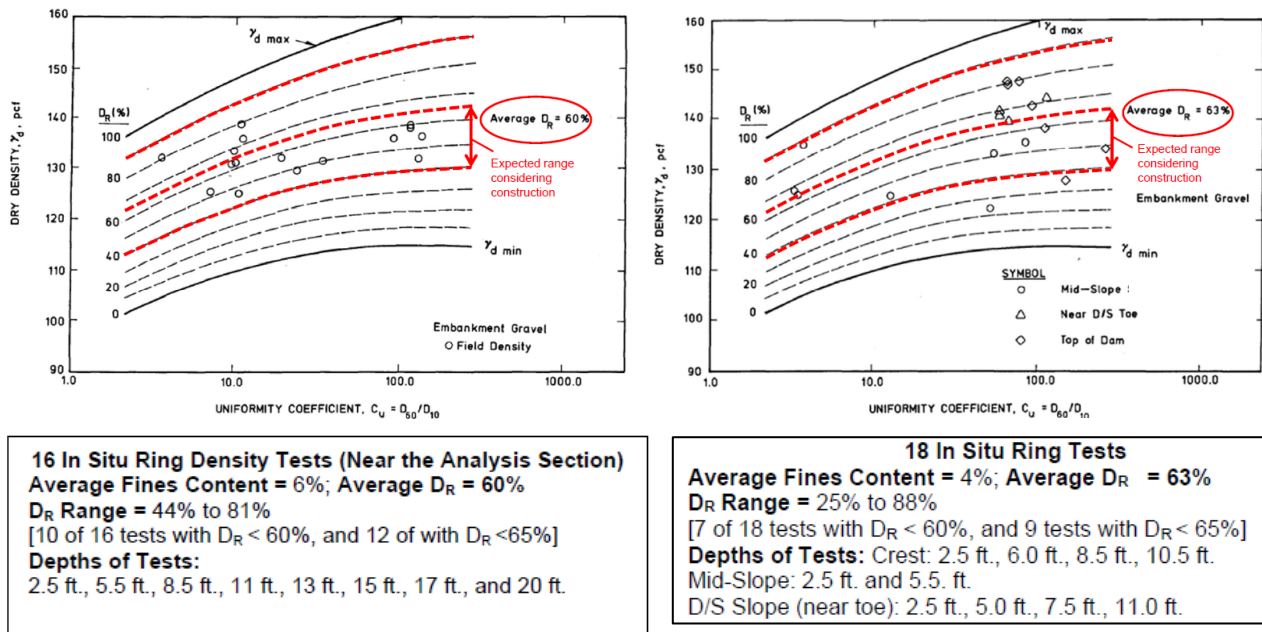


Figure 14: Results of 34 large in-situ diameter ring density tests performed at 4 different areas of the dam and at different depths (two tests per depth), and the results of the supporting laboratory maximum and minimum density tests to determine  $D_R$

as together, the in-situ  $D_R$  tests and the estimates of expected  $D_R$  based on placement and compaction history, provide a good basis for assessment of overall conditions. The subsequent BPT investigations in 2019 were performed for four purposes: as an additional cross-check, as an additional basis for inferring SPT  $N_{1,60}$  values, to examine whether there are discernable “looser” and “denser” strata within the shell and transition zone fills to refine modeling of these units, and to determine consistency between upstream and downstream shell materials.

## **UPDATED SITE CHARACTERIZATION PART 2: EVALUATION OF NEW BPT/iBPT DATA FROM THE 2019 STUDY**

In 2019, a total of six new BPT borings were performed, with instrumentation and recording at the top and bottom of the closed-ended BPT “pipe pile” in a manner that would allow for the analysis of the driving process by various instrumentation-based methods. Interpretations could then be made by (1) the Sy and Campanella (1994) method, (2) the DeJong et al. (2017) and Ghafghazi et al. (2017) method, and (3) the Harder and Seed (1986) method. GRL Engineers performed the instrumentation and obtained and analyzed data. Two of these new instrumented BPTs (iBPTs) were performed on the upstream side of the dam, and the other four new iBPTs were performed on the downstream side. Figure 15 shows a comparison of  $N_{60}$  values using the three commonly used BPT to SPT conversion methods for two new BPT borings in the upstream side of the embankment performed as part of the 2019 investigation. Comparisons for the other four new (additional) 2019 iBPTs are similar. These three sets of iBPT-based  $N_{1,60}$  values, based on three different interpretation methods, again can all be converted to relative density ( $D_R$ ), using the same conversion previously employed.

Table 5 presents a summary comparison of resulting median  $N_{1,60}$  values as well as ranges/fractiles of estimated  $D_R$  values based on (1) the construction history of the embankment dam, (2) relative density values from the 1980’s large diameter in situ relative density tests, (3)  $N_{1,60}$  values and resulting inferred relative density values from the 1980’s upstream BPTs using the Harder and Seed (1986) method, and  $N_{1,60}$  values and resulting relative density values from the six new BPT borings from the 2019 studies using (4) the Harder and Seed (1986) method, (5) the Sy and Campanella (1994) method, and (6) the DeJong et al. (2017) and Ghafghazi et al. (2017) method. As presented in Table 1, median and average SPT- $N_{1,60}$  values or  $D_R$  values from the two more recent instrumented BPT methods are very high and not realistic considering the construction history and in-situ ring density tests. It is worth noting that the iBPT interpretations based on the Sy and Campanella method and the DeJong et al. (2017) and the Ghafghazi et al. (2017) method both produced significant numbers of values of  $D_R > 100\%$ , which is not physically possible. Based on the results summarized in Table 5, the Harder and Seed (1986) BPT to SPT conversion was selected for the seismic analyses of this embankment dam, along with the new Static End Bearing Method.



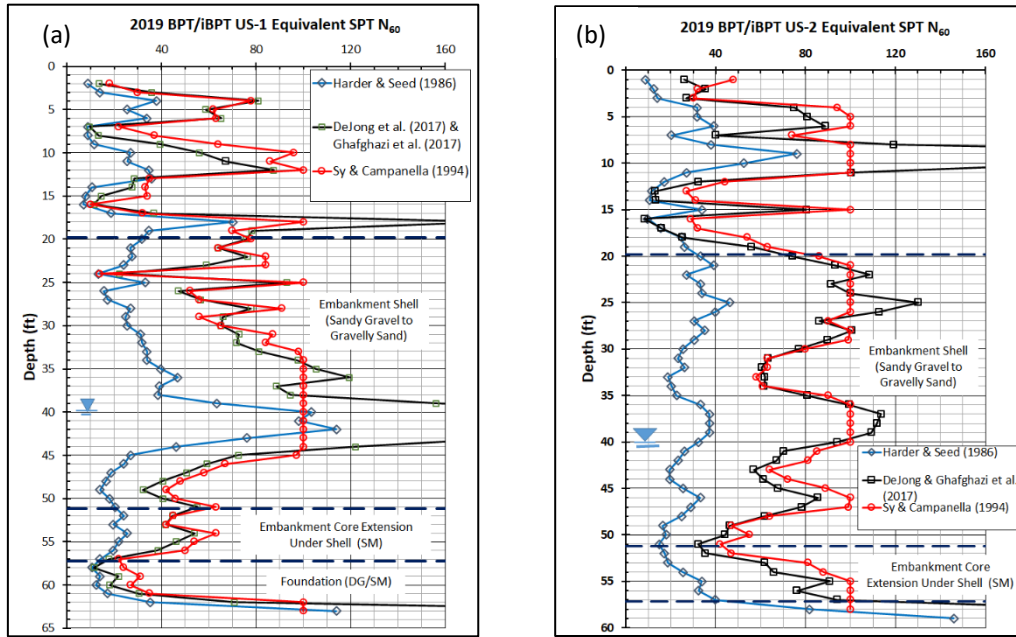


Figure 15:  $N_{60}$  values for the two upstream BPTs from the 2019 study using three commonly used BPT or instrumented BPT to SPT conversion methods

**Table 5**

Comparison of Soil Type, Construction History, and Relative Density Values from the 1980s and 2019 USACE Studies

Zone	Soil Type	Maximum lift thickness (ft) Equipment (Number of passes)	Relative Density ( $D_R$ ) from Thirty Four 4-foot Diameter In Situ Ring Density Tests	Relative Density ( $D_R$ ) from six 1980s BPT's using Harder and Seed (1990)	Relative Density ( $D_R$ ) from six 2019 BPT's using Harder and Seed (1990)	Relative Density ( $D_R$ ) from six 2019 instrumented BPT's using Sy and Campanella (1994)	Relative Density ( $D_R$ ) from six 2019 instrumented BPT's using DeJong et al. (2017) and Ghafghazi et al. (2017)
Shell And Transition Zone	Shell: Sandy Gravel to Gravelly Sand With Occasional Larger Particles Transition Zone: Gravelly Sand to Sandy Gravel	Shell: 2 feet to 4 feet to 12 feet Transition: 1 to 2 feet D-8 Cat. Tractor (1 "full" coverage – estimated to be 3 or 4 parallel passes)	Average $D_R$ : 60.9%	Average $D_R$ : 60.7%	Average $D_R$ : 66.3%	Average $D_R$ : 96.6%	Average $D_R$ : 95.6%
			Median $D_R$ : 61.0%	Median $D_R$ : 63.2%	Median $D_R$ : 62.9%	Median $D_R$ : 97.1%	Median $D_R$ : 91.5%
			Median $N_{1,60}$ : ~ 22.3	Median $N_{1,60}$ : 23.9	Median $N_{1,60}$ : 23.7	Median $N_{1,60}$ : 56.5	Median $N_{1,60}$ : 49.8
			$D_R = 40-80\%$ : 88.2%	$D_R = 40-80\%$ : 78.4%	$D_R = 40-80\%$ : 67.4%	$D_R = 40-80\%$ : 22.3%	$D_R = 40-80\%$ : 33.2%
			$D_R < 65\%$ : 61.8%	$D_R < 65\%$ : 50.0%	$D_R < 65\%$ : 54.9%	$D_R < 65\%$ : 11.9%	$D_R < 65\%$ : 22.0%
			$D_R > 100\%$ : 0% of 34 data points	$D_R > 100\%$ : 1.5% of 134 data points	$D_R > 100\%$ : 8.2% of 328 data points	$D_R > 100\%$ : 44.8% of 328 data points	$D_R > 100\%$ : 39.6% of 328 data points

## 5. CHARACTERIZATION OF U/S AND D/S EMBANKMENT ZONES USING THE END BEARING METHOD

Instrumented data from all six 2019 BPT borings were analyzed using the End Bearing method. A total of 95 CAPWAP-RSA analyses were performed at selected depths. Residual force at each of these depths was determined based on residual stress analyses (RSA) (Rausche et al, 2010). Equivalent CPT- $q_c$  values were then determined using the end bearing values from CAPWAP-RSA analyses, and equivalent SPT  $N_{60}$  values were determined using  $(q_c/P_a)/N_{60} = 6$ , appropriate for gravelly sand to sand, as per Robertson et al. (1986). Continuous profiles of end bearing force and equivalent SPT  $N_{60}$  for all these six 2019 BPT borings were developed using the CASE method. Figure 16 shows a comparison of  $N_{60}$  values using the End Bearing and the Harder and Seed (1986) BPT to SPT conversion methods in six BPT borings in upstream and downstream of the embankment dam. Figure 16 shows that the results of the Harder and Seed (1986) method match well with the results of the End Bearing method. Table 6 presents a comparison of  $N_{1,60}$  and  $D_R$  values from the Harder and Seed (1986) and the End Bearing Method.

Figure 16 shows that the results of the Harder and Seed (1986) method match well with the results of the End Bearing method. However, as the End Bearing method more accurately accounts for shaft resistance, the End Bearing Method was selected for characterizing the upstream soil layers in the seismic deformation analyses of the embankment dam.

It should be noted that the re-evaluation of the Annasis site data using the instrumented BPT data from Sy (1993) that resulted in a good match with the adjacent CPT (as presented previously in this paper) was performed in accordance with the end bearing method.

Table 6. Summary of  $N_{1,60}$  and relative density ( $D_R$ ) values from upstream BPTs and instrumented BPTs (US-1 and US-2)

Soil Layer	Harder and Seed (1986)				End Bearing Method			
	Median $N_{1,60}$	$D_R$ (%)	33rd Percentile $N_{1,60}$	$D_R$ (%)	Median $N_{1,60}$	$D_R$ (%)	33rd Percentile $N_{1,60}$	$D_R$ (%)
US-Embankment 1	22.5	61%	19.2	57%	25.1	65%	23.2	62%
US-Embankment 2	24.5	64%	23.9	63%	32.0	73%	31.0	72%
US-Embankment 3	18.3	55%	15.9	51%	22.3	61%	20.9	59%
US-Embankment 4	10.9	43%	10.5	42%	14.1	48%	12.2	45%
US-SM Core Zone	12.1	52%	11.1	50%	15.1	58%	11.5	51%

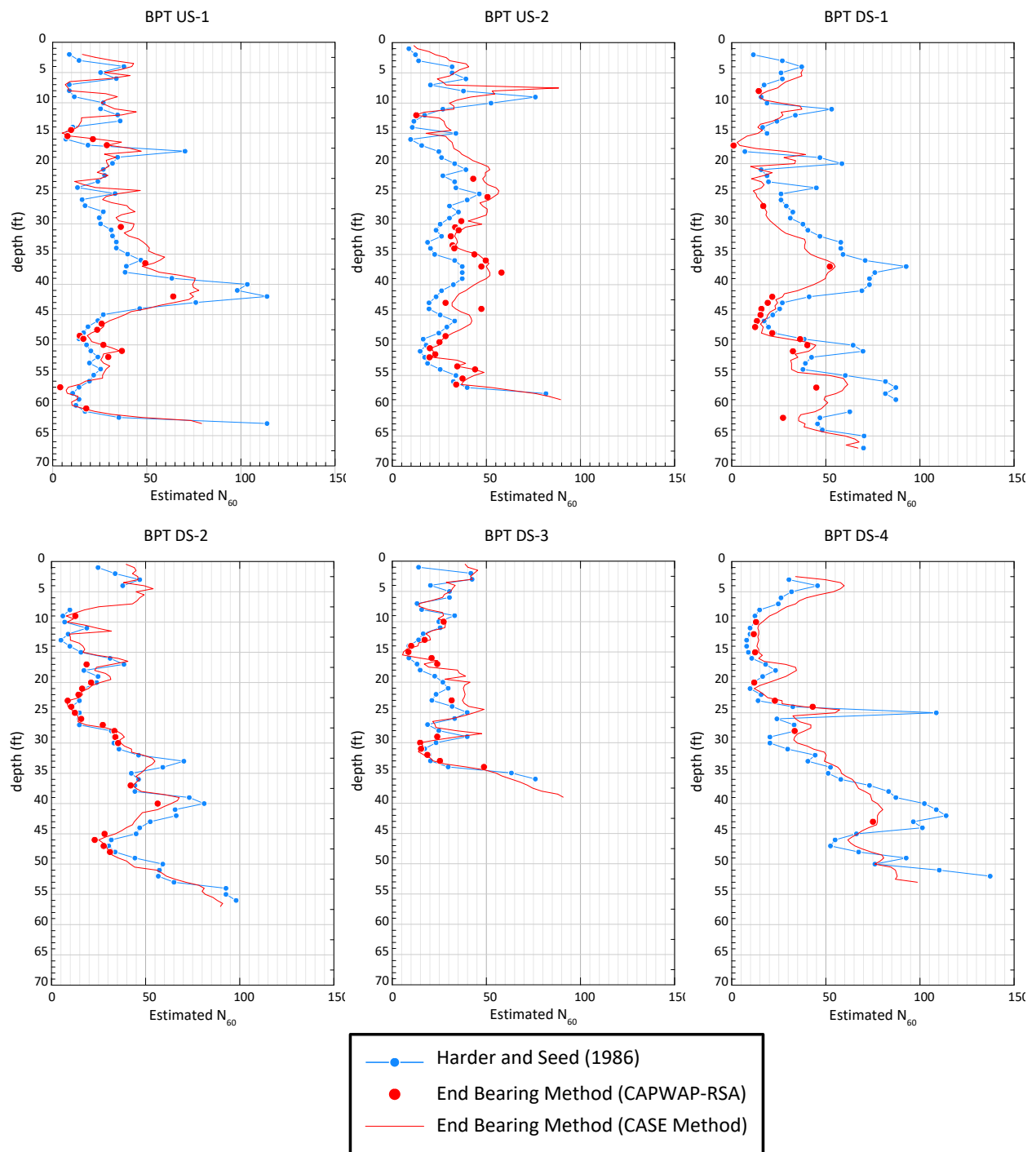


Figure 16:  $N_{60}$  Values for Upstream and Downstream BPTs using the End Bearing Method Compared to the Harder and Seed (1986) Method.

## **6. USE OF BACK-OF-THE-ENVELOPE APPROXIMATION FOR CURRENT PROJECT AND POTENTIAL USE TO UPDATE EXISTING INSTRUMENTED BPT TO SPT DATABASE**

The applicability of the Back-of-the-Envelope approximation, as described in Table 4, is explored in Figures 17 and 18. First, Back-of-the-Envelope  $N_{60}$  are computed using blow-specific static energy ratios ( $\beta$ ) from the corresponding CAPWAP analyses. As shown in Figure 17, using the blow-specific static energy ratio ( $\beta$ ),  $N_{60}$  values are almost coincidental with the detailed CAPWAP-RSA based  $N_{60}$ . These identical values validate the theoretical basis of the static-energy ratio ( $\beta$ ) based Back-of-the-Envelope  $N_{60}$  approximation. This is as expected, since the two methods are simply different expression of the same energy balance equations.

Secondly, Figure 18 shows Back-of-the-Envelope  $N_{60}$  values computed using a single fixed static energy ratio,  $\beta = 0.85$ , the typical value found from the 95 CAPWAP analyses. The estimated  $N_{60}$  are generally in good agreement with End Bearing  $N_{60}$ , although there are a few dramatic outliers. Note that for  $\beta = 0.85$ , the Back-of-the-Envelope approximation gives  $N_{60}/N_{B30} = 0.95$  for the gravelly sands or sandy gravels comprising most of the embankment material (see Table 4).

Both the detailed End Bearing Method and the Back-of-the-Envelope approximations could be used to update evaluations of instrumented data collected at the toe and top of BPT piles in different projects. These updates using the End Bearing Method and the Back-of-the-Envelope approximation would allow to systematically account for static energy ratio and BPT- $N_B$  to SPT- $N_{60}$  conversion considering soil characteristics.

## **7. SEISMIC DEFORMATION ANALYSES USING THE END BEARING AND THE HARDER AND SEED (1986) METHODS**

As part of the overall seismic evaluation of the embankment dam, a total of fourteen non-linear seismic deformation analyses (NDA) were performed for the 2,450-year, 9,950-year, and 30,000-year return period seismic scenario events, as determined from probabilistic and deterministic seismic hazard analyses. These non-linear seismic deformation analyses were performed using the finite difference analysis program FLAC, Version 8 (Itasca, 2018). These NDA analyses were performed using (1) different reservoir pool elevations, (2) different input ground motions [three longer period motions and one shorter period motion], (3) two different analytical models for potentially liquefiable soils (the Roth model and the UBCSAND model), (4) two different soil liquefaction triggering and cyclic pore pressure generation relationships (Youd et al., 2001 and Cetin et al., 2018), (5) Weber et al. (2015) post-liquefaction residual strength, and (6) two different BPT to SPT  $N_{1,60}$  interpretation protocols: the Harder and Seed (1986) method and the End Bearing Method. In the first set of analyses, soil parameters in the upstream and downstream using the Harder and Seed (1986) method were utilized. In the second set of analyses, soil parameters in the upstream side using the end bearing method were utilized. In both sets of analyses, large upstream flow slide or large deformations were developed.

Chowdhury et al. (2020 and 2021) presents findings from these seismic deformation analyses in detail.

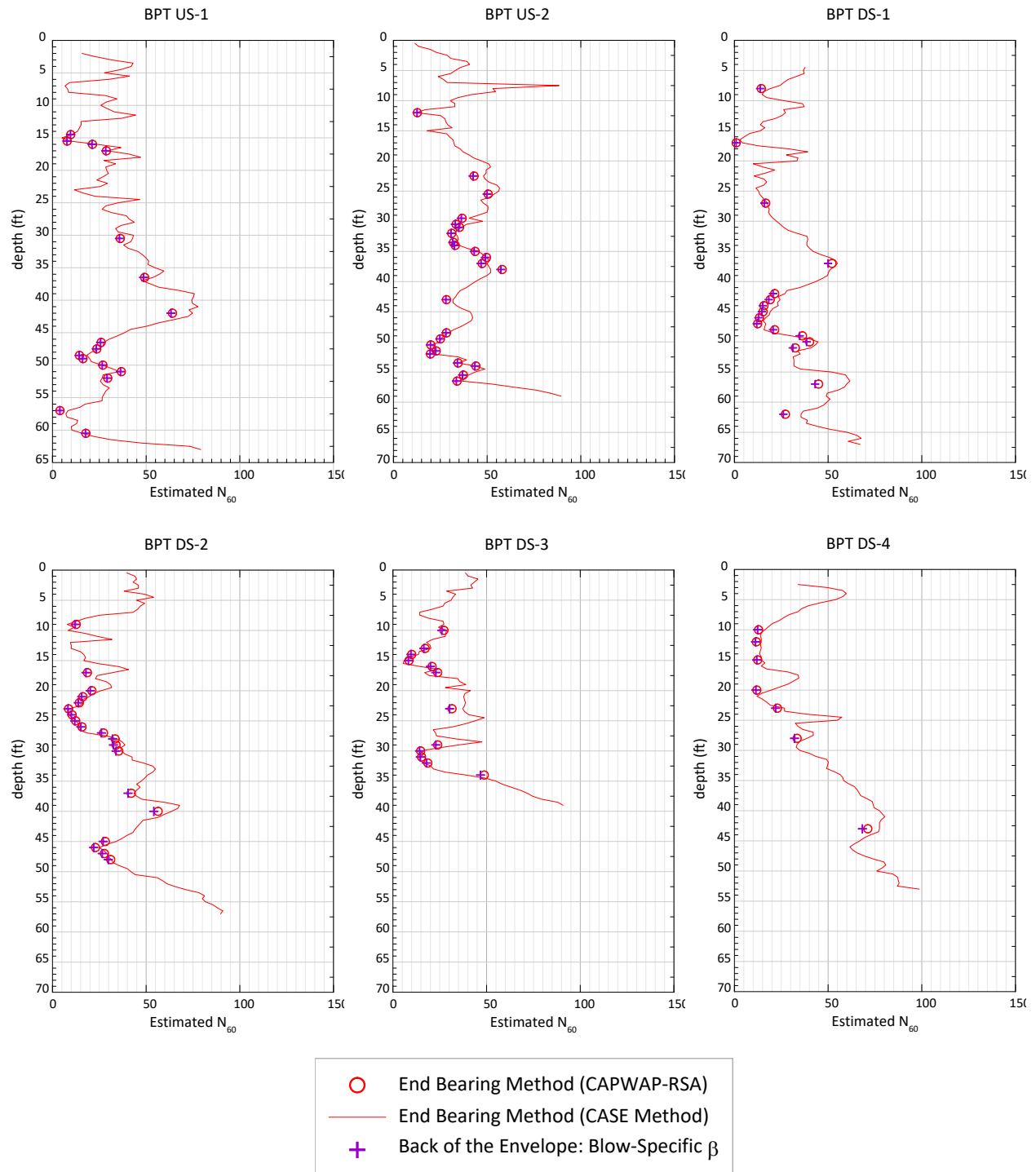


Figure 17:  $N_{60}$  Values for End Bearing Method Compared to Back-of-the-Envelope Calculation using Blow-Specific  $\beta$  value from CAPWAP-RSA analyses

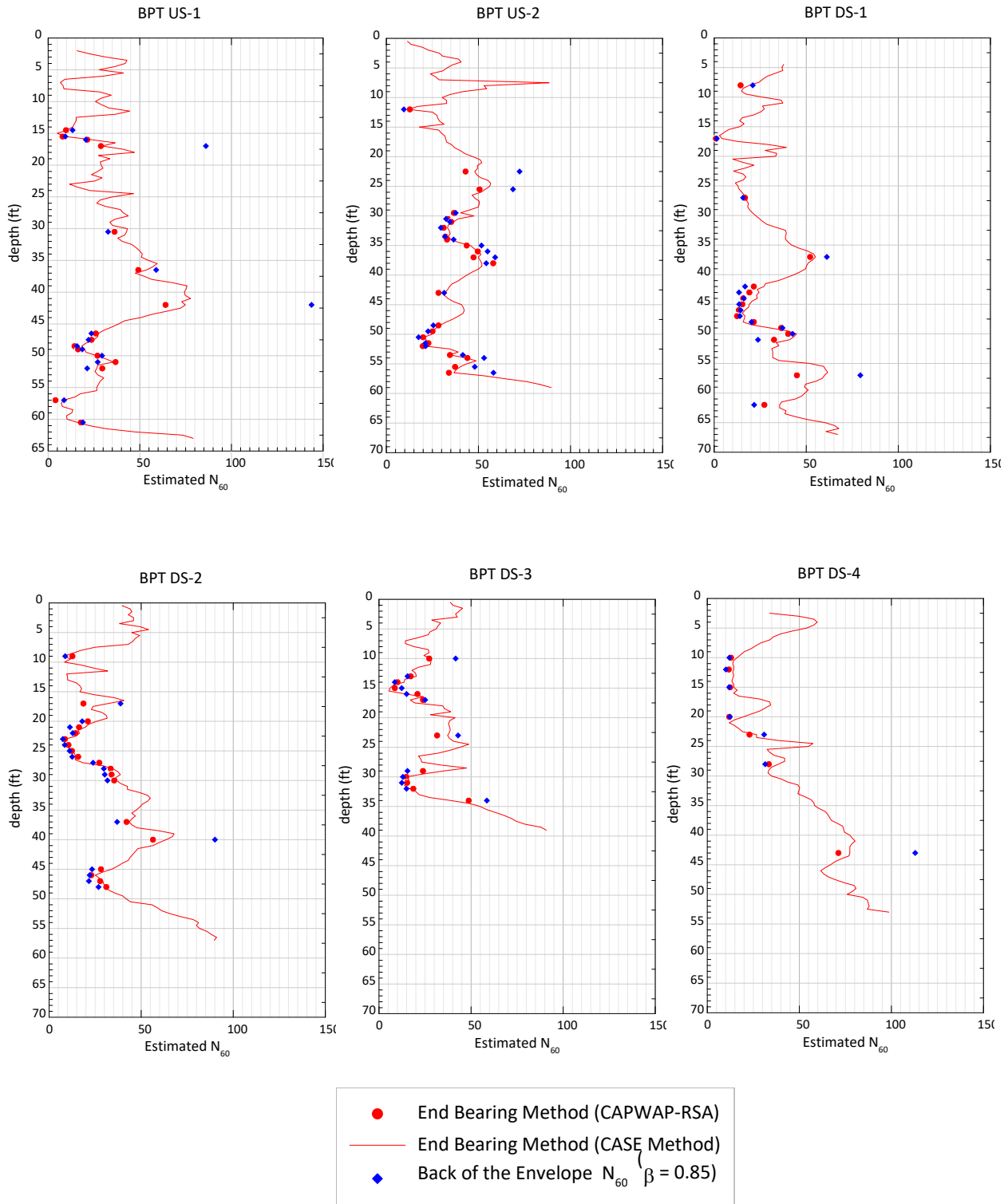


Figure 18:  $N_{60}$  Values for End Bearing Method Compared to Back-of-the-Envelope Calculation using Static Energy Ratio  $\beta = 0.85$  (typical value from CAPWAP analyses, for  $N_{B60} < 60$ )

## CONCLUSIONS

The proposed end bearing method provides an approach based on systematic treatment of instrumented data and widely used site characterization relationships. The following are conclusions based on an evaluation of the existing BPT and instrumented BPT methods.

- The Harder and Seed (1986) method does not include a systematic consideration of shaft resistance, and as a result, the interpreted SPT  $N_{60}$  values may be un-conservative or over-conservative; depending on the shaft resistance actually encountered, as compared to the shaft resistance implicit in the initial development of the relationship. Thus, a drawback of this method is the inability to directly account for, and correct for, different levels of shaft resistance in different materials and at different overall depths of penetration. However, in site conditions with lower “during driving” shaft friction (such as looser embankment soil layers in the embankment dam used in the current study, and at low to moderate depths), the results of the Harder and Seed (1986) method may be reasonable. In the current embankment dam project, results from the Harder and Seed (1986) method match well with construction history and results from in-situ ring density tests. This method may not be as reliable at greater depths, unless special steps are taken to reduce shaft friction (e.g. over-reaming to depth before commencing BPT, etc.)
- A re-analysis of wave traces used in developing the Sy and Campanella (1994) method using modern CAPWAP-RSA analyses indicates that Sy and Campanella (1994) underestimated toe resistance and overestimated shaft resistance in analyzing five blows at the Annacis site using CAPWAP analyses, however, not accounting for residual stress. This error led to steeper shaft resistance-based relationships to convert  $N_{B30}$  from BPT to equivalent  $N_{60}$  values in the Sy and Campanella (1994) method. As the recommended curves are based on underestimated toe resistances for the boring used for relationship development, it unconservatively overestimates the toe resistance for existing conditions that need to be evaluated for forward engineering analyses. Toe resistance values estimated from the re-analyses performed by CAPWAP-RSA under this study match well with the adjacent CPT in Annacis site. These re-analyses indicate that the Sy and Campanella (1994) method has potential to overestimate toe resistance of coarse-grained soils by a factor of two or higher than actual  $N_{60}$ .
- The recommended SPT  $N_{60} = 1.8 N_{B30}$  by Ghafghazi et al. (2017) using the DeJong et al. (2017) method is not appropriate for all soil types, in particular for coarse-grained soils, where BPTs are used for site characterization. The data used to develop this recommended relationship have unusually high scatter due to the presence of gravelly to clayey soils. The recommended relationship overestimates equivalent SPT  $N_{60}$  values for sandy to gravelly soil conditions based on data from four dams that were used for developing the Ghafghazi et al. (2017) relationship and thus, would result in unconservative estimates of  $N_{60}$ . In addition, the residual force determination procedure is not available for industry-wide use and thus an evaluation on accuracy has not been performed.

The following are major attributes and potential advantages of the proposed end bearing method:

- The proposed end bearing method systematically analyzes the instrumented data collected at the tip (and top) of the BPT pile. The entire process is transparent and can be performed by the existing instrumentation industry including practicing engineers.
- The proposed end bearing method utilizes direct measurements of the BPT pile tip force measurements. This method avoids noise in instrumented measurements, as it relies on direct measurement of force.
- CAPWAP-RSA and CASE method analyses using the BPT pile tip data result in significant improvement over analyses using the BPT pile top measurements, as data is not as impacted by the shaft friction parameters.
- The proposed method can account for different soil types. This is a significant improvement from the commonly used instrumented BPT methods, which are not easily adaptable for site conditions with different soil types.
- A Back-of-the-Envelope end bearing approximation has demonstrated the importance of the static energy ratio and different soil types regarding the determination of equivalent SPT-N<sub>60</sub> values.
- A limitation of the end bearing method, as well as other instrumented and non-instrumented BPT methods, can occur when refusal conditions are encountered in borings that develop high shaft friction during driving. With the BPT site characterization method, mobilization of end bearing conditions is an important criterion to accurately determine toe resistance. Without mobilization of the end bearing of the soil, the end bearing values using high blow counts could be erroneous. The end bearing method is a useful tool to evaluate the correctness of the blows representing end bearing, as it systematically differentiates the “during driving” shaft resistance and the end bearing.
- Both the detailed End Bearing Method and the Back-of-the-Envelope approximations could be used to update evaluations of instrumented data collected at the toe and top of BPT piles in different projects. These updates using the End Bearing Method and the Back-of-the-Envelope approximation would allow to systematically account for static energy ratio and BPT-N<sub>B</sub> to SPT-N<sub>60</sub> conversion considering soil characteristics.
- The proposed end bearing method utilizes established knowledge and State of Practice procedures in deep pile foundation design to improve site characterizations using BPT pile driving data. The proposed method allows engineers to utilize the appropriate Pile to CPT relationships based on soil conditions. As the proposed method is based on fundamentals of soil mechanics and instrumentation measurements, it can be potentially utilized in two major practice areas in geotechnical engineering
  1. Use of instrumented BPT piles and instrumented closed end pipe piles in subsurface investigations or post-construction verification studies to obtain site-specific subsurface characterization, and
  2. Use of instrumented BPT piles and instrumented closed end pipe piles for dynamic pile load testing to obtain a more accurate determination of pile capacity.



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