Challenges and Recommendations for Steel H-Piles Driven in Soft Rock

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ABSTRACT: The capacity of a pile driven in soft rock depends on soil confinement along the pile and rock at its toe; these are rarely known during design. This design challenge often leads to a large discrepancy between estimated and measured resistances. Results of six bridge projects completed in Wyoming, USA, are presented to highlight the challenges pertaining to present design and construction practices of driven piles in rock. The results show that static analysis methods, dynamic analysis methods, and structural analyses yield inconsistent pile resistance estimations. A recommendation considering the structure-geo-material interaction is proposed to improve the design and construction of steel H-piles driven in soft rock.

KEYWORDS: Pile, LRFD, Rock, WEAP, CAPWAP

1. INTRODUCTION

Because of their high driving durability on rock materials, steel H-piles are typically used to support bridges in Wyoming's shallow bedrock stratigraphy. The total axial resistance of these piles is a function of both shaft resistance and end bearing. To attain the resistance required for bridge support in soft overburden soil with low shaft resistance, the pile would need to rest on a solid rock material for higher end bearing.

Estimating the resistances of piles driven on rock materials is challenging, partly due to the erratic characteristics of natural rock. Even more challenging was that, in this research study, all test piles were driven in softer rock layers. According to the American Association of State Highway and Transportation Officials (AASHTO) Load and Resistance Factor Design (LRFD) Bridge Design Specifications (2014), soft rocks are not well distinguished from hard rocks or soils in the design and construction of piles.

Intermediate Geo-Material (IGM), a soft rock or stiff heavily-consolidated soil material, is normally considered for drilled shaft (i.e., bore pile or cast-in-place pile) design and construction. Defined by O'Neill and Reese (1999), cohesive IGM materials are clay shales or mudstones with unconfined compressive strengths (q_u) of 0.5 to 5 MPa. Cohesionless IGM materials are granular tills or granular residual soils with corrected Standard Penetration Test (SPT) N-values ($N1_{60}$) falling between 50 and 100 blows per 300 mm penetration. A review to improve understanding of IGMs was conducted by Mokwa and Brooks (2008) who agreed that the shear strength of IGMs is less than that of intact rock but greater than that of soil. The shear strength of IGMs depends on parent materials and geologic processes such as deposition, lithification, diagenesis, cementation, or weathering (Gannon et al. 1999). Hence, Papageorgiou (1993) recommended that the analysis and design of piles on soft rock should include both a geologic component integrating rock formation and a geotechnical component describing engineering properties. Haberfield and Seidel (1999) developed theoretical models based on the fundamental understanding of rough interface behavior to predict shear strength of soft rocks. The predictions of shear strengths have been shown to compare well with that measured from direct shear tests. Because of the high variability of IGMs, many definitions and descriptions of IGMs appear in literature (Mokwa and Brooks 2008). Soft and hard rocks are neither uniformly nor objectively defined. An appropriate definition must be determined by local conditions and experience. Due to the natural variability of soft rock, uncertainties in deep foundation design are exacerbated, leading to many construction challenges.

Pile-soil-rock interactive response, penetration depth, and detailed rock characteristics are usually not available for pile resistance estimation during a design stage (Hannigan et al. 2006). The resisting performance of these piles depends on driving observations, dynamic and static load tests, and local experience. Since an expensive and time-consuming static load test is usually not performed, these piles are typically verified using dynamic analysis methods, which are not a proof load test. A pile resistance is usually governed by its structural strength when it is driven to end bearing in rock of fair to excellent quality based on Rock Quality Designation (*RQD*) values. On the other hand, a pile supported on soft weathered rock should be designed based on pile load test results, because 1) the rock strength would govern the pile resistance, and 2) pile resistance could decrease due to relaxation in soft weathered rock near the pile toe (Thompson and Thompson, 1985). The AASHTO LRFD Bridge Design Specifications (2014) recommend treating soft rock in the same manner as soil during pile design. However, a recent study by Ng et al. (2015) concluded that current static analysis methods, originally developed for soil, provided inconsistent and potentially conservative geotechnical resistance estimations of a driven pile on soft rock. Recognizing that acceptable approaches to differentiate soft from hard rocks are not available, AASHTO (2014) suggested the application of local experience to define rock quality. Locally developed criteria based on dynamic analysis methods should be established to evaluate pile driveability, prevent pile damage, and attain the pile resistance.

Mokwa and Brooks (2008) investigated the suitability of conventional analysis methods in predicting the axial resistance of piles driven into IGM formations in Montana, USA. Research outcomes showed considerable variation between estimated pile resistances by DRIVEN (Mathias and Cribbs 1998) and measured resistances by CAse Pile Wave Analysis Program (CAPWAP). The Washington Department of Transportation (WSDOT) Gates dynamic formula provided the best match to measured resistances by CAPWAP. They concluded that CAPWAP or static load tests are the only reliable methods to determinate pile resistance into IGMs.

The Colorado Department of Transportation (CDOT), USA, uses a simplified empirical approach based on the allowable stress design philosophy to estimate pile resistance in cohesive IGM formations. This approach has the following assumptions: (1) IGMs are treated as hard rock, (2) pile resistance relies primarily on end bearing, and (3) allowable pile resistance (*R/FS*) is approximated to 25% structural steel pile yield stress (F_y) and pile tip area (A_p) given by Eq. (1). A factor of safety (*FS*) of 3.0 is assumed in the design. The pile depth is estimated based on past experience. CDOT Road and Bridge Construction Specifications (2005) require that piles be driven to a refusal (i.e., 25 mm or less of pile penetration for the final 10 hammer blows).

$$R/FS = 0.25\,F_y A_p \tag{1}$$

The characteristic lines method proposed by Serrano and Olalla (2002a) based on Hoek and Brown's non-linear failure model requires advanced rock parameters that are not readily available for an ultimate end bearing capacity (σ_{hp}) estimation using Eq. (2)

$$\sigma_{hp} = \left(m/8\right)\left(p_2 + q_2\right)q_u \tag{2}$$

where, m and s are the rock mass parameters based on the Hoek and Brown criterion (1980), p_2 and q_2 are the Lambe's variables, and q_u is the unconfined compressive strength of an intact rock. They concluded that the proposed method was acceptable for piles bearing in soft rock with q_u values less than 20 to 30 MPa. The method overestimated the ultimate bearing capacity of hard rock.

Literature on rock socketed driven piles is limited, and research studies have been mostly conducted on rock socketed drilled shafts. The knowledge gained from drilled shafts can be adopted in driven pile foundation. Pells (1999) summarized practices for design of socketed drilled shafts in rock. Drilled shaft resistances are highly dependent on construction technique and quality. It was concluded that method by Rowe and Armitage (1984) was the most satisfactory design tool. A specific limiting shaft displacement will be reached before an applied load is shared between shaft resistance and end bearing. The approach suggested by Carter and Kulhawy (1988) may be used to reasonably estimate this limiting displacement (Akguner and Kirkit 2012).

Seidel and Collingwood (2001) recognized that prediction of shaft resistance in rock is a complex problem and often based on empirical methods that may not account for important variables such as pile diameter and rock jointing (Seidel and Haberfield 1995). They developed a micromechanical simulation approach to better predict the shaft socket behavior by considering a nondimensional parameter known as the shaft resistance coefficient (SRC). However, the application of the SRC approach requires the knowledge of the rock socket roughness which can be back-calculated from load test results. Additionally, the SRC approach utilizing the ROCKET program can predict a transition from hard soils to rocks that relates the empirical relationships suggested by Kulhawy and Phoon (1993).

2. CHALLENGES

Limitations associated with soft rock characterizations and accurate pile resistance estimations create the following challenges in the design and construction of pile foundations on soft rock:

1) Static analysis methods are not available for the estimation of pile resistance on soft rock.
2) Clear definition of soft rock is not available.
3) Resistance factors (φ) for piles on rock in Eq. (3) are neither locally calibrated nor recommended by AASHTO (2014), where γ_i is a load factor and Q_i is an applied load.

$$\sum \gamma_i Q_i \leq \varphi R \tag{3}$$

4) The natural variability of soft rocks creates uncertainty in the subsurface condition for pile designs.
5) The static load test is rarely performed to verify the pile resistance nor is used for calibrating dynamic analysis methods.
6) Large discrepancies between estimated and measured pile resistances were observed (Ng et al. 2015). It is not unusual that these piles do not satisfy the LRFD strength limit state at EOD or occasionally at the beginning of last restrike (BOR). When pile performance is not attained during construction, possible pile extension and/or additional piles with enlarged pile caps will be proposed to achieve the required resistance. This could take more time and incur additional construction costs.
7) The uncertainty in pile performance could incur difficulty in construction management since foundation construction is a critical component of a bridge project. This uncertainty could result in higher construction bids, higher frequency of claims, and higher design safety for offsetting the challenge in construction management (Mokwa and Brooks 2008).
8) Conflicts between owners and contractors could occur, resulting in change-orders to the original contract for additional claims and time to achieve the required pile performance.

3. WYOMING PRACTICES

The Wyoming Department of Transportation (WYDOT) currently uses the AASHTO LRFD Bridge Design Specifications (2014) and applies local experience to the design and construction of pile foundations. The WYDOT Geology Program normally performs a site investigation at every bridge project to determine its subsurface profile and geomaterial properties. The SPT is the most commonly used in-situ field test in Wyoming. At the same site of the SPT, a drivepoint penetration test is conducted by driving a 50 mm diameter drivepoint into the ground using a 64 kg hammer at a drop height of 762 mm. Hammer blows to penetrate the drivepoint 305 mm into the ground are counted and recorded. The purpose of the drivepoint penetration test is to determine the depth of an adequate bearing layer, such as unweathered bedrock, for the end bearing pile. When a bedrock layer is encountered, a rock coring is taken to determine the *RQD* value, and rock samples are tested for q_u value.

The Geology Program has developed a table of typical soil material properties for pile capacity estimation. However, because locally calibrated data on unit shaft resistance and end bearing of piles on soft rock are currently not available, pile performance cannot be verified until construction.

WYDOT currently uses Wave Equation Analysis Program (WEAP) to establish pile driving criteria for all production piles. Pile Driving Analyzer (PDA) with subsequent signal matching analyses using CAPWAP is used as a construction control method on about 2% of the production piles in some bridge projects. PDA/CAPWAP is implemented to determine and verify the required pile capacity at bridge projects expecting high loads and soft rock bearing. Pile restrikes at 24 hours after the end of driving (EOD) are normally performed to further ensure that the desired pile resistance is achieved and pile performance is accepted.

This paper presents the evaluation of the axial resistances of fifteen steel H-piles driven on soft rock at six recently completed bridge projects in the state of Wyoming, USA. Geotechnical resistances estimated using five static analysis methods and WEAP were compared with measured resistances by CAPWAP. Structural capacities were determined to improve our understanding of the pile-geomaterial interaction. A new analysis method was proposed to improve the accuracy of pile resistance estimation and alleviate the discrepancy between estimated and measured resistances.

4. BRIDGE PROJECTS

Six bridge projects were chosen for this study. The bridge locations are indicated on the Wyoming map shown in Figure 1. Table 1 summarizes the test pile, hammer, and overburden soil of these fifteen test piles at the respective bridge projects. The uncorrected SPT N-value, q_u and RQD values of the rock bearing layers are summarized in Table 2. Their soil profiles and SPT N-values are shown in Figure 2, Figure 3 and Figure 4. The backgrounds of these six bridge projects are briefly described in the following subsections. Detailed descriptions of the Burns South Road, Casper Street, and Torrington Street projects can be found in the conference paper by Ng et al. (2015). Detailed descriptions of the Owl Creek, Woods Wardell, and Pine Bluffs projects can be found in a series of summary reports submitted to WYDOT by Ng (2015) as well as in a conference paper by Ng and Sullivan (2016).

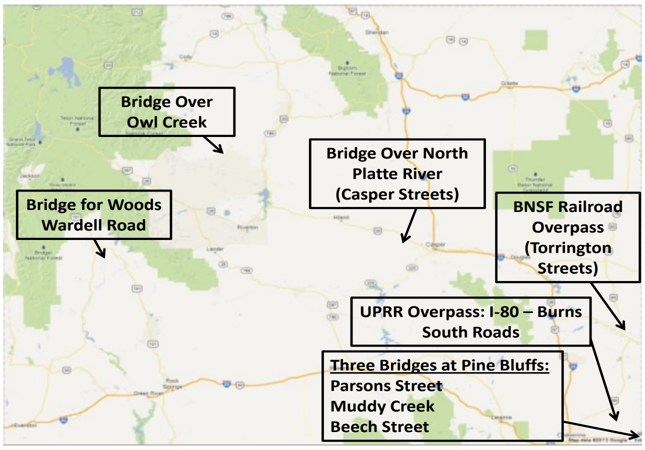

Figure 1 Six bridge projects on the map of Wyoming

Table 1 Summary of Bridge Projects

Project	Str.	H-Pile	L_e (m)	Ham.	Soil
Burns South	Pi3P1	360×109	11.9	D16-32	Silty Sand
	A1P1	360×109	22.0	D16-32	Silty Sand
Casper	A2P1	360×109	10.1	M-19	Silty Sand
Torrington	A2P1	360×109	30.5	M-19	Poor Sand
Owl	B2P5	360×109	10.4	ICE-42S	Silty Sand
Woods	Pi2P1	310×79	7.0	D19-42	Sandy Silt
PB-Parsons	A1P5	310×79	26.8	D16-32	Sandy Silt
	A2P1	310×79	22.9	D16-32	Sandy Silt
PB-Muddy Creek	A2P1	310×79	16.5	D16-32	Sandy Silt
	B2P1	310×79	10.7	D16-32	Sandy Silt
	B3P10	310×79	11.6	D16-32	Sandy Silt
PB-Beech Street	A1P1	310×79	14.3	D16-32	Sandy Silt
	A1P5	310×79	14.3	D16-32	Sandy Silt
	A2P1	310×79	13.7	D16-32	Sandy Silt
	A2P3	310×79	14.3	D16-32	Sandy Silt

PB–Pine Bluffs; Str.–Bridge Structure; Pi–Pier; B–Bent; A–Abutment; P–Test Pile; L_e–Embedded Pile Length; Ham.–Hammer; D–Delmag; M–Mississippi Valley Equipment Company; and ICE–International Construction Equipment.

Table 2 Summary of Bearing Rock Properties

Project	Structure	Bearing Rock: Type	N	q_u (MPa)	RQD (%)
Burns South	Pi3P1	Sandstone	79	n/a	n/a
	A1P1	Sandstone	57-141	n/a	n/a
Casper	A2P1	Sandstone	n/a	n/a	33
Torrin.	A2P1	Claystone	42	n/a	n/a
Owl	B2P5	Shale	n/a	0.15-0.69	11-67
Woods	Pi2P1	Siltstone to Claystone	59	1.39-3.64	92
PB-Parsons	A1P5	Siltstone	81-150	2.16	59
	A2P1	Siltstone	n/a	2.16	59
PB-Muddy Creek	A2P1	Siltstone	51	1.64	68
	B2P1	Siltstone	71	1.64	68
	B3P10	Siltstone	51	1.64	68
PB-Beech Street	A1P1	Siltstone	55	0.15	21
	A1P5	Siltstone	55	0.15	21
	A2P1	Siltstone	43	0.15	21
	A2P3	Siltstone	43	0.15	21

PB–Pine Bluffs; Pi–Pier; B–Bent; A–Abutment; P–Test Pile; N–Uncorrected SPT N-value; q_u–Unconfined Compressive Strength; RQD–Rock Quality Designation; and n/a–Not Available.

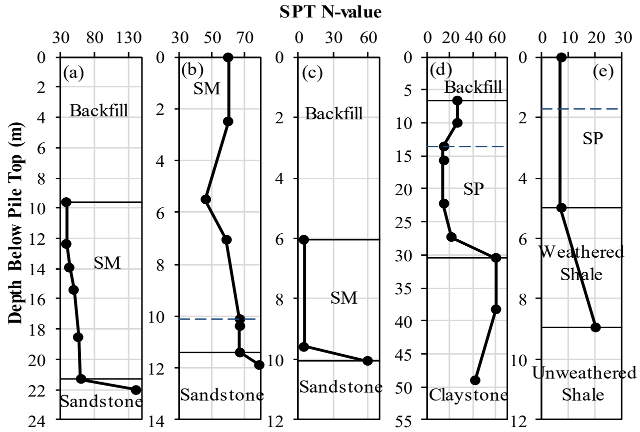

Figure 2 Soil profiles of (a) Burns South (A1P1); (b) Burns South (Pi3P1); (c) Casper (A2P1); (d) Torrington (A2P1); and (e) Owl Creek (B2P5)

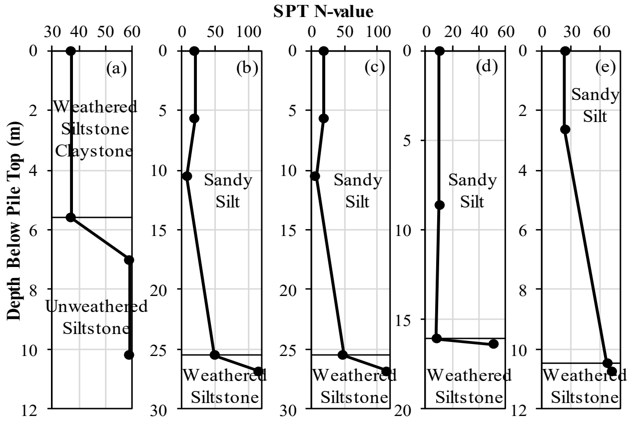

Figure 3 Soil profiles of (a) Woods Wardell (Pi2P1); (b) PB-Parsons (A1P5); (c) PB-Parsons (A2P1); (d) PB-Muddy (A2P1); and (e) PB-Muddy (B2P1)

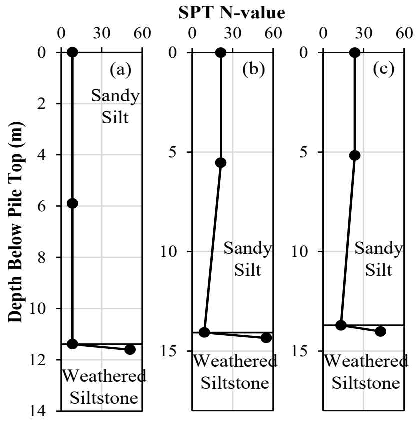

Figure 4 Soil profiles of (a) PB-Muddy (B3P10); (b) PB-Beech (A1P1 & A1P5); (c) PB-Beech (A2P1 & A2P3)

4.1 Burns South Road

A 131-m four span reinforced concrete bridge was constructed over an existing Union Pacific Rail Road (UPRR) near the intersection of Interstate 80 and Burns South Road in Laramie County. The bridge consisted of two abutments and three piers. Grade 50 HP 360×109 piles were installed at all abutment and pier locations using a

Delmag D16-32 single acting diesel hammer. Pile No. 1 at Abutment No.1 and Pile No. 1 at Pier No. 3 were selected for dynamic load tests and 24-hour restrikes. The subsurface profile is generally silty sand (SM) overlying a dense to very dense silty, fine grained, non-cemented sandstone.

4.2 Casper Street

A 154-m three span reinforced concrete bridge was constructed over the North Platte River along Casper Street in Natrona County. The bridge consists of two abutments and two piers. Grade 50 HP 360×109 piles were installed at each abutment while Grade 50 HP 360×132 piles were installed at each pier location. A Mississippi Valley Equipment company (MVE) M-19 single acting diesel hammer was used to install all piles. Pile No. 1 at Abutment No. 2 was selected for dynamic load tests and 24-hour restrikes. The subsurface profile at Abutment No. 2 is colluvium, loose to dense, silty, pea-gravelly sand overlaying un-weathered sandstone bedrock.

4.3 Torrington Street

The Torrington Streets bridge project consisted of a new 91.4-m four span reinforced concrete bridge over the Burlington Northern & Sante Fe Railroad. This overpass connects US 85 to US 26 near F Street in downtown Torrington. The bridge has two abutments and three piers. Each abutment was supported by Grade 50 HP 360×109 piles while each pier was supported by a shallow foundation. A MVE M-19 single acting diesel hammer was used to install the piles. Pile No. 1 at Abutment No. 2 was selected for dynamic load tests and 24-hour restrikes. The subsurface profile from the ground consists 6.6 m backfill, 7 m well graded sand (SW), 13.7 m poorly graded sand (SP), and 10.8 m well graded gravel (GW) overlaying a weathered claystone.

4.4 Owl Creek

A 43-m span bridge for WYO170 with two abutments and two bents was constructed over Owl Creek in Hot Springs County. Grade 50 HP 310×79 piles were installed at each abutment while Grade 50 HP 360×109 piles were installed at each bent. An International Construction Equipment (ICE) 42S diesel hammer was used to install piles. Pile No. 5 at Bent 2 was selected for dynamic load tests and 24-hour restrikes. The subsurface consists loose to very dense silty sand and gravel overlaying weathered shale, which in turn overlays very hard, unweathered shale bedrock.

4.5 Woods Wardell Road

A 64-m span bridge for Woods Wardell Road was constructed over the Green River in Sublette County. The bridge consisted of two abutments and two piers. Grade 50 HP 310×79 piles were installed at all abutments and piers. An American Pile driving Equipment Inc. (APE) D19-42 single acting diesel hammer was used to install piles. Pile No. 1 at Pier No. 2 was selected for dynamic load tests and 24-hour restrikes. The subsurface consists of saturated silty sand and gravel (river deposits) overlaying hard, dry, weathered siltstone and claystone bedrock, which in turn overlays very hard, dry, un-weathered siltstone and claystone bedrock.

4.6 Pine Bluffs

The Pine Bluffs project located in Laramie County involved the construction of three bridges at the Parsons Street Interchange, over Muddy Creek and for Beech Street Separation, respectively. All driven steel H-piles were Grade 50 HP 310×79 and installed using a Delmag D16-32 single acting diesel hammer. The 55-m span bridge for the Parsons Street Interchange consisted of two abutments and two bents. Steel H-piles were installed at each abutment while 1.2-m diameter drilled shafts were installed at each bent. Abutment No. 1 Pile No. 5 (A1P5) and Abutment No. 2 Pile No. 1 (A2P1) were selected for dynamic load tests and 24-hour restrikes. The subsurface consists mainly medium dense silty sand with minor gravel and sandy silt overlaying a very dense weathered siltstone.

The 75.3-m span bridge over Muddy Creek consisted of two abutments and three bents. Steel H-piles were installed at all abutments and bents. Abutment No. 2 Pile No. 1 (A2P1), Bent No. 2 Pile No. 1 (B2P1) and Bent No. 3 Pile No. 10 (B3P10) were selected for dynamic load tests. Restrikes were not performed on these test piles. The subsurface consists mainly medium dense sandy silt (embankment) and medium dense sandy silt with gravel overlaying very dense silt to weak siltstone.

The 46-m span bridge for the Beech Street Separation consisted of two abutments and two bents. Steel H-piles were installed at each abutment, while 1.2-m diameter drilled shafts were installed at each bent. Pile No. 1 (A1P1) and Pile No. 5 (A1P5) at Abutment No. 1 and Pile No. 1 (A2P1) and Pile No. 3 (A2P3) at Abutment No. 2 were selected for dynamic load tests and 24-hour restrikes. The subsurface is medium dense sandy silt with minor gravel (embankment fill) and loose to dense sandy silt with minor gravel (alluvium) overlaying a dense to very dense sandy silt to weak siltstone.

5. ANALYSES AND RESULTS

Axial geotechnical resistances of these fifteen test piles were estimated using five static analysis methods. WEAP used hammer, pile, subsurface, and driving information to estimate pile resistances for both EOD and BOR events. Since static load tests were not performed on these test piles, pile resistances obtained from the CAPWAP signal matching technique were identified as the measured pile resistances in subsequent analyses. Structural capacities of the test piles were estimated based on several boundary conditions for comparison purposes.

5.1 Static Analysis

The SPT-Meyerhof (1976), Nordlund (1963), DRIVEN (Mathias and Cribbs 1998), and β-method (Burland 1973) were selected to estimate nominal pile resistances (R). DRIVEN estimates pile resistance based on a combination of the Nordlund (1963) method for cohesionless soil and the α-method (Tomlinson 1980) for cohesive soil. However, the α-method considering soft rock updated by Kulhawy and Phoon (1993) was not included in DRIVEN. Unit pile resistances recommended by WYDOT were also used to estimate pile resistances. Since all piles were driven in cohesionless soil, "H" section was considered in the shaft resistance and end bearing estimations.

Table 3 summarizes the nominal total pile resistances estimated by these five static analysis methods. For each static analysis method, ratios of CAPWAP-measured pile resistances (R_m) at EOD (Table 9) to estimated pile resistances (R_e) (Table 3) were determined. The total sample size (N) of resistance ratios (R_m/R_e) for the WYDOT method is 11 because only 11 pile resistances (i.e., pile resistances for Owl Creek, Woods Wardell and Pine Bluffs) were estimated and compared with the measured resistances.

Statistical parameters (i.e., maximum ratio, minimum ratio, bias (λ), standard deviation (σ), and coefficient of variation (COV)) of the resistance ratios for each static analysis method are summarized in Table 4. DRIVEN, which has a bias value closest to one and a relatively lower COV value of 0.799 provided the closest match to resistances measured by CAPWAP. In contrast, the SPT proved to be the least accurate method of the five in estimating total pile resistances. On average, the SPT method underestimated total pile resistance by a factor of about 4.5. It is important to note that the WYDOT method's poor performance with a relative high λ value of 3.486 was the result of not accounting for the end bearing in the total pile resistance calculation. Comparing with the statistical parameters by Paikowsky et al. (2004) for piles driven in soil only (Table 4), all static analysis methods underestimated total pile resistances when piles are driven into rock. This comparison confirms the limitation of current static analysis methods and the unrealistic assumption of treating rock as soil material.

Table 3 Summary of Nominal Total Pile Resistances Estimated by Static Analysis Methods

Project	Test Pile	Nominal Total Pile Resistance (kN)				
		WY DOT	DRI VEN	Nord lund	β	SPT
Burns South	Pi3P1	n/a	2829	2131	1721	1210
	A1P1	n/a	2900	2500	1957	1361
Casper	A2P1	n/a	391	200	222	396
Torrin.	A2P1	n/a	4141	2576	2451	970
Owl	B2P5	480	730	n/a	n/a	n/a
Woods	Pi2P1	191	947	n/a	n/a	n/a
PB-Parsons	A1P5	1388	6361	6481	2860	939
	A2P1	1090	4075	4515	2073	703
PB-Muddy Creek	A2P1	752	1668	2037	1597	209
	B2P1	601	1157	979	623	943
	B3P10	534	876	1143	970	165
PB-Beech Street	A1P1	454	1281	1592	845	262
	A1P5	454	1259	1601	845	249
	A2P1	427	1112	1628	859	298
	A2P3	445	1192	1753	916	302

Table 4 Summary of statistical parameters of total resistance ratios

Data	Met-hod	N	Ratio of Measured to Estimated Total Resistances (R_m/R_e)				
			Max	Min	λ	σ	COV
WY-This Study	WY-DOT	11	10.465	0.994	3.486	2.612	0.749
	DRI-VEN	15	3.864	0.217	1.369	1.094	0.799
	Nord lund	13	7.556	0.213	1.439	1.957	1.359
	Beta	13	6.800	0.434	1.819	1.728	0.950
	SPT	13	18.135	1.096	4.454	4.579	1.028
P	SPT	18	n/a	n/a	0.81	0.31	0.38
	α	17	n/a	n/a	0.82	0.33	0.40
	Nord-lund	20	n/a	n/a	0.59	0.23	0.39

P−The study by Paikowsky et al. (2004) for piles driven in soil only; α−α method by Tomlinson (1980); and n/a−Not available.

Nominal shaft resistances estimated by the five static analysis methods are summarized in Table 5. Although the piles were eventually driven into relatively thin soft rock layers, overburden soils contributed the most to shaft resistances. Treating these soft rocks as soil materials in the shaft resistance estimation, the static analysis methods, which were originally developed for soil, underestimated the shaft resistances. Shaft resistance ratios were calculated by comparing the CAPWAP-measured shaft resistances at the EOD (Table 9) with the estimated shaft resistances (Table 5). Statistical parameters of the shaft resistance ratios are summarized in Table 6. The bias values range from 1.132 for the Nordlund to 3.544 for the SPT method, confirming that static analysis methods underestimated the shaft resistances. Although Nordlund and DRIVEN produced bias values closer to one, their relatively large COV values of 1.684 and 1.296, respectively, suggest a high variation in the shaft resistance estimation. The SPT method provided the least accurate shaft resistance estimation with the highest bias and a relative high COV.

Table 5 Summary of Nominal Shaft Resistances Estimated by Static Analysis Methods

Project	Test Pile	Nominal Total Pile Resistance (kN)				
		WY DOT	DRI VEN	Nord lund	β	SPT
Burns South	Pi3P1	n/a	2500	1962	1286	1005
	A1P1	n/a	2651	2264	1512	1072
Casper	A2P1	n/a	142	120	85	93
Torrin.	A2P1	n/a	4043	2531	2108	747
Owl	B2P5	480	707	n/a	n/a	n/a
Woods	Pi2P1	191	374	n/a	n/a	n/a
PB-Parsons	A1P5	1388	6143	6281	2669	801
	A2P1	1090	3959	4404	1908	601
PB-Muddy Creek	A2P1	752	1343	1721	841	165
	B2P1	601	899	721	351	899
	B3P10	534	552	818	601	129
PB-Beech Street	A1P1	454	1183	1499	743	249
	A1P5	454	1165	1477	738	240
	A2P1	427	1063	1579	761	280
	A2P3	445	1148	1699	814	285

Nominal end bearings estimated by the static analysis methods are summarized in Table 7. Although piles were driven into soft rocks, they were treated in the same manner as soil in the end bearing estimation. End bearings on soft rocks were neither estimated nor accounted for by WYDOT. End bearing ratios were calculated by comparing the CAPWAP-measured end bearings at the EOD (Table 9) with the estimated end bearings (Table 7).

Table 6 Summary of statistical parameters of shaft resistance ratios

Method	N	Ratio of Measured to Estimated Shaft Resistances (R_m/R_e)				
		Max	Min	λ	σ	COV
WYDOT	11	4.877	0.815	1.906	1.169	0.614
DRIVEN	15	6.007	0.164	1.210	1.569	1.296
Nordlund	13	7.154	0.180	1.132	1.907	1.684
Beta	13	10.139	0.300	1.838	2.708	1.473
SPT	13	12.499	0.423	3.544	3.742	1.056

Table 7 Summary of Nominal End Bearings Estimated by Static Analysis Methods

Project	Test Pile	Nominal Total Pile Resistance (kN)				
		WY DOT	DRI VEN	Nord lund	β	SPT
Burns South	Pi3P1	n/a	329	169	436	205
	A1P1	n/a	249	236	440	289
Casper	A2P1	n/a	249	80	142	302
Torrin.	A2P1	n/a	98	44	343	222
Owl	B2P5	n/a	22	n/a	n/a	n/a
Woods	Pi2P1	n/a	574	n/a	n/a	n/a
PB-Parsons	A1P5	n/a	218	200	191	138
	A2P1	n/a	111	116	165	98
PB-Muddy Creek	A2P1	n/a	325	320	756	40
	B2P1	n/a	254	258	276	44
	B3P10	n/a	325	325	365	36
PB-Beech Street	A1P1	n/a	98	93	102	13
	A1P5	n/a	98	125	102	9
	A2P1	n/a	49	49	98	18
	A2P3	n/a	44	49	102	18

Statistical parameters of the end bearing ratios are summarized in Table 8. Bias values range from 3.441 for the β-method to 20.506 for the SPT method, demonstrating that the end bearings were greatly underestimated by the static analysis methods. Also, the wide range of standard deviations, from 1.957 to 21.026, indicates high variation in the end bearing estimation. The relatively lower COV values are the result of high bias values. The results indicate that none of the five static analysis methods provide accurate estimations of nominal end bearing on soft rock.

Table 8 Summary of statistical parameters of end bearing ratios

Method	N	Ratio of Measured to Estimated End Bearings (R_m/R_e)				
		Max	Min	λ	σ	COV
WYDOT	0	n/a	n/a	n/a	n/a	n/a
DRIVEN	15	38.148	1.133	7.177	9.454	1.317
Nordlund	13	12.930	1.231	6.040	3.944	0.653
Beta	13	6.483	0.588	3.441	1.957	0.569
SPT	13	64.767	1.795	20.506	21.026	1.025

5.2 Dynamic Analysis

CAPWAP-measured shaft resistances, end bearings, and total resistances at both EOD and BOR events are summarized in Table 9. At EOD, approximately 56% of total pile resistance was contributed by the shaft resistance, with the remaining 44% due to end bearing. The percentage changed slightly at BOR with about 60% from shaft resistance and 40% from end bearing. These results clearly demonstrate the important contribution of end bearing on soft rock to total pile resistance. On average, 24 hours after the EOD, shaft resistance increased by 30% while end bearing decreased by 8%. Total pile resistance increased on average of 9% at the 24-hour restrike. The results showed that pile setup was totally contributed by shaft resistance. The decrease in end bearing on soft rock could be a result of relaxation. However, a long-term pile monitoring was not conducted to fully describe the possibility of pile relaxation.

Table 9 Summary of Measured Pile Resistances by CAPWAP

Project	Test Pile	R_s (kN)		R_p (kN)		R (kN)	
		E	B	E	B	E	B
Burns South	Pi3P1	649	1121	996	649	1646	1770
	A1P1	454	876	1793	1575	2246	2447
Casper	A2P1	845	1076	667	614	1512	1690
Torrin.	A2P1	663	903	400	298	1063	1201
Owl	B2P5	583	854	916	761	1499	1615
Woods	Pi2P1	934	1157	1068	1068	2002	2224
PB-Parsons	A1P5	1134	1294	245	160	1379	1450
	A2P1	890	956	356	356	1241	1308
PB-Muddy Creek	A2P1	1250	n/a	445	n/a	1695	n/a
	B2P1	1383	n/a	623	n/a	2006	n/a
	B3P10	1606	n/a	1379	n/a	2985	n/a
PB-Beech Street	A1P1	645	685	538	601	1179	1290
	A1P5	805	956	467	556	1272	1512
	A2P1	627	667	645	694	1272	1361
	A2P3	712	783	645	645	1361	1428

R_s−Shaft Resistance; R_p−End Bearing; R−Total Resistance; E−End of Driving (EOD); and B−Beginning of Restrike (BOR).

Past studies by Ng (2015) concluded that total pile resistances estimated by WEAP using bearing graphs based on either a soil type based method (ST) or a SPT N-value based method (SA) were nearly identical. This outcome was expected because pile resistance estimation was influenced chiefly by damping factors and quake values, which share similar values in both ST and SA procedures. For this reason, only results based on the SA procedure are presented in Table 10. The hammer blow counts and permanent sets at both EOD and BOR are given in Table 11. Most permanent sets exceeded 2 mm except Burns South's A1P1, Owl Creek's B2P5 and Pine Bluff-Parsons's A1P5. These driving results indicate that pile resistances, especially end bearings, were mobilized during the dynamic load testing.

Table 10 Summary of Pile Resistances Estimated by WEAP

Project	Test Pile	R_s (kN)		R_p (kN)		R (kN)	
		E	B	E	B	E	B
Burns South	Pi3P1	347	356	992	1010	1343	1366
	A1P1	467	472	1375	1415	1842	1886
Casper	A2P1	454	454	1552	1552	2011	2011
Torrin.	A2P1	961	1188	676	823	1641	2011
Owl	B2P5	676	703	703	890	1379	1592
Woods	Pi2P1	641	663	592	614	1232	1272
PB-Parsons	A1P5	1793	1842	31	31	1824	1873
	A2P1	1753	1628	40	40	1793	1668
PB-Muddy Creek	A2P1	1655	n/a	85	n/a	1735	n/a
	B2P1	1628	n/a	214	n/a	1842	n/a
	B3P10	1810	n/a	245	n/a	2055	n/a
PB-Beech Street	A1P1	1419	1597	58	62	1472	1659
	A1P5	1450	1931	58	80	1512	2006
	A2P1	1410	1695	62	76	1468	1766
	A2P3	1677	1624	67	67	1744	1690

R_s−Shaft Resistance; R_p−End Bearing; R−Total Resistance; E−End of Driving (EOD); and B−Beginning of Restrike (BOR).

Table 11 Summary of hammer blow counts and permanent sets

Project	Test Pile	Hammer Blow Count Per 300 mm Penetration		Permanent Set (mm)	
		E	B	E	B
Burns South	Pi3P1	100	108	3.1	2.8
	A1P1	452	600	0.7	0.5
Casper	A2P1	84	84	3.6	3.6
Torrin.	A2P1	68	108	4.5	2.8
Owl	B2P5	263	360	1.2	0.8
Woods	Pi2P1	128	156	2.4	2.0
PB-Parsons	A1P5	164	216	1.9	1.4
	A2P1	146	137	2.1	2.2
PB-Muddy Creek	A2P1	109	n/a	2.8	n/a
	B2P1	108	n/a	2.8	n/a
	B3P10	240	n/a	1.3	n/a
PB-Beech Street	A1P1	55	84	5.5	3.6
	A1P5	66	156	4.6	2.0
	A2P1	62	96	4.9	3.2
	A2P3	82	72	3.7	4.2

E−End of Driving (EOD); and B−Beginning of Restrike (BOR).

Three comparisons between measured and estimated total pile resistances were conducted to determine their respective resistance ratios: 1) the ratio of CAPWAP-measured resistances at EOD to WEAP-estimated resistances at EOD (denoted as EOD/EOD); 2) the ratio of CAPWAP-measured resistances at EOD to WEAP-estimated resistances at BOR (denoted as EOD/BOR); and 3) the ratio of CAPWAP-measured resistances at BOR to WEAP-estimated resistances at BOR (denoted as BOR/BOR). The statistical parameters of these three resistance ratios are summarized in Table 12. WEAP provided more accurate pile resistance estimation than static analysis methods as demonstrated by bias values closer to one and lower COV values. In particular, the comparison at the EOD (i.e., EOD/EOD) yielded the best match, considering the bias value of 0.988 closest to one and lowest COV of 0.294. In general, WEAP tends to slightly overestimate the total pile resistance. Comparing the statistical parameters by Paikowsky et al. (2004) based on the US nationwide database of piles driven in soil only, this study, based on the local Wyoming data set, provided a better estimation of total pile resistances with having bias values closer to one and smaller COV values. This comparison suggests that WEAP can be suitably used to estimate total resistance of piles driven through soil into rock.

Table 12 Summary of statistical parameters of total resistances by WEAP

Data		N	Ratio of Measured to Estimated Total Resistances (R_m/R_e)				
			Max	Min	λ	σ	COV
WY-This Study	EOD/EOD	15	1.625	0.648	0.988	0.290	0.294
	EOD/BOR	12	1.573	0.529	0.879	0.299	0.341
	BOR/BOR	12	1.748	0.597	0.958	0.329	0.343
P	EOD/EOD	99	n/a	n/a	1.66	1.19	0.72

P−The study by Paikowsky et al. (2004) for piles driven in soil only; and n/a−Not available.

A similar comparison was conducted for shaft resistance as illustrated in Table 13. The comparison of shaft resistances at the EOD (i.e., EOD/EOD) yielded the best match, considering the bias value of 0.880 closest to one and lowest COV of 0.542. WEAP tended to underestimate the shaft resistance at BOR with the bias value of 1.182. When compared with total resistances, WEAP yielded a relatively higher variation in shaft resistance estimations with COV values ranging from 0.542 to 0.764.

Table 13 Summary of statistical parameters of shaft resistances by WEAP

Comparison Method	N	Ratio of Measured to Estimated Shaft Resistances (R_m/R_e)				
		Max	Min	λ	σ	COV
EOD/EOD	15	1.861	0.426	0.880	0.477	0.542
EOD/BOR	12	1.855	0.370	0.853	0.549	0.644
BOR/BOR	12	3.157	0.394	1.182	0.903	0.764

A similar comparison was conducted for end bearing as illustrated in Table 14. WEAP greatly underestimated the end bearing at both EOD and BOR events with bias values ranging from 4.588 to 4.955. Furthermore, WEAP yielded the highest variation in end bearing estimations with COV values ranging from 0.761 to 0.899. These results demonstrated that WEAP provides less accurate estimations of nominal end bearing on soft rock.

Table 14 Summary of statistical parameters of end bearing by WEAP

Comparison Method	N	Ratio of Measured to Estimated End Bearing (R_m/R_e)				
		Max	Min	λ	σ	COV
EOD/EOD	15	10.522	0.429	4.955	3.773	0.761
EOD/BOR	12	9.687	0.429	4.651	3.942	0.848
BOR/BOR	12	9.687	0.362	4.588	4.124	0.899

Table 10 shows that total pile resistance increased about 9% on average at the 24-hour restrike, which was comparable to CAPWAP's. However, WEAP's 8% and 9% increases in shaft resistance and end bearing, respectively, are less consistent with CAPWAP observations. There is a possibility that the pile resistance was not fully mobilized during the dynamic testing as indicated by a relatively small pile deformation per hammer blow recorded by PDA (Ng and Sullivan 2016). This could lead to the underestimation of pile resistance by CAPWAP. However, most permanent sets at both EOD and BOR conditions summarized in Table 11 exceed 2 mm, indicating the mobilization of pile resistance, especially end bearing. Unfortunately, a static load test was not performed to verify this test pile performance.

Dividing the CAPWAP-measured end bearing at EOD (Table 9) by the pile tip area, the unit end bearings were calculated and plotted against the SPT N-values, q_u values and RQD of the bearing soft rocks in Figure 5. A positive trend was observed between unit end bearing and SPT N-value while no clear trend was observed for the q_u and RQD.

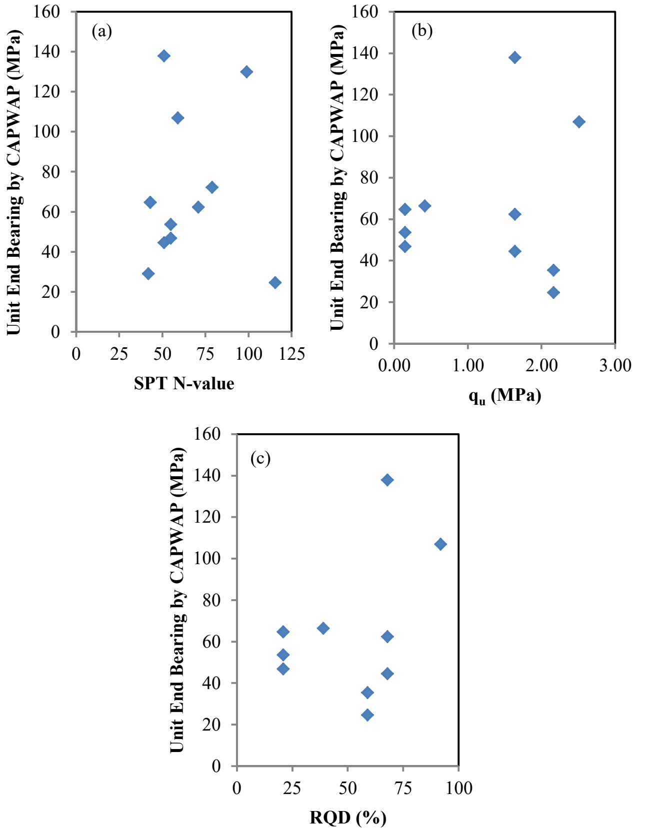

Figure 5 Relationships between CAPWAP-measured unit end bearing and (a) SPT N-value; (b) q_u value; and (c) RQD

Due to the large discrepancy between estimated and measured pile resistances, piles driven on soft rock often do not satisfy the LRFD strength limit state at EOD and occasionally at BOR as illustrated in Table 15. The total number of production piles at each structure location (i.e., bent, pier or abutment), where the test pile was selected for dynamic load testing, are included. The performance of these production piles followed the performance of the corresponding test pile. The results indicate that 80 of 104 production piles (77%) did not satisfy the LRFD strength limit state when WEAP was used as the only construction control method at EOD. Furthermore, 50 production piles (i.e., 48%) tested at the BOR did not satisfy the LRFD strength limit state.

When PDA/CAPWAP was used as the construction control method at EOD, 54 production piles (52%) were considered unacceptable. When PDA/CAPWAP was used at BOR, the number of production piles satisfying the LRFD strength limit state was reduced to 14, or 13% of total piles. However, it is important to note that WEAP was used to evaluate all production piles while PDA/CAPWAP covered only about 2% of the total production piles. These results illustrate the importance of performing restrike using PDA/CAPWAP as the construction control method.

Table 15 Summary of Pile Performance Acceptances

Project	Test Pile	No. of Piles$	Pile Performance Acceptance			
			WEAP		CAPWAP	
			EOD	BOR	EOD	BOR
Burns South	Pi3 P1	21	No (-41%)	No (-40%)	No (-7%)	Yes (0.4%)
	A1 P1	5	No (-36%)	No (-34%)	Yes (2%)	Yes (11%)
Casper	A2 P1	14	Yes (33%)	Yes (33%)	Yes (30%)	Yes (46%)
Torrington	A2 P1	9	No (-15%)	Yes (6%)	No (-28%)	No (-19%)
Owl	B2 P5	5	No (-30%)	No (-27%)	No (-12%)	No (-5%)
Woods	Pi2 P1	14	No (-65%)	No (-63%)	No (-3%)	Yes (10%)
PB-Parsons	A1 P5	5	Yes (6%)	Yes (8%)	Yes (7%)	Yes (13%)
	A2 P1	5	Yes (6%)	No (-2%)	No (-4%)	Yes (2%)
PB-Muddy Creek	A2 P1	6	No (-7%)	n/a	Yes (22%)	n/a
	B2 P1	5	No (-31%)	n/a	Yes (0%)	n/a
	B3 P10	5	No (-20%)	n/a	Yes (49%)	n/a
PB-Beech Street	A1 P1	5	No (-9%)	Yes (5%)	Yes (0%)	Yes (9%)
	A1 P5		No (-6%)	Yes (24%)	Yes (8%)	Yes (28%)
	A2 P1	5	No (-11%)	Yes (8%)	Yes (8%)	Yes (16%)
	A2 P3		Yes (6%)	Yes (3%)	Yes (16%)	Yes (22%)

B−Bent No.; PI−Pier No; A−Abutment No.; P−Pile No. as the test pile; $−Number of production piles at the respective bent, pier or abutment location; Yes−Satisfied the LRFD strength limit state; and No−Did not satisfy the LRFD strength limit state; and (%)−Percent of measured factored resistance (φR) higher (positive) or lower (negative) than the required factored load (γQ).

5.3 Structural Analysis

Considering the test pile as a compression member that experienced only an axial compressive load, the nominal structural capacity of the test pile (P_n) was taken as the smallest value based on the applicable modes of flexural buckling, torsional buckling, and flexural-torsional buckling (AASHTO 2014). For a steel H-pile section without slender elements, the flexural buckling was considered while the torsional buckling was neglected due to a greater torsional resistance from the surrounding soil. Hence, the structural capacity (P_n) was estimated by Eqs. (3) through (6)

$$P_n = \left[0.658^{\left(\frac{P_o}{P_e}\right)} \right] P_o \quad \text{if } \frac{P_e}{P_o} \geq 0.44 \tag{4}$$

$$P_n = 0.877 P_e \quad \text{if } \frac{P_e}{P_o} < 0.44 \tag{5}$$

$$P_e = \frac{\pi^2 E}{\left(\frac{KL}{r_s}\right)^2} A_g \tag{6}$$

where A_g is the cross-sectional area of a pile, F_y is the specified minimum yield strength of a steel pile, P_o is the equivalent nominal yield resistance = $\psi F_y A_g$, ψ is the slender element reduction factor (taken as 1.0 for a pile without slender elements), K is the effective length factor in the plane of buckling, L is the unbraced pile length in the plane of buckling, r_s is the radius of gyration about the axis normal to the plane of buckling, and P_e is the elastic critical buckling resistance.

Among these variables, the effective pile length (KL) depends on the soil confinement along its length and rock fixity at its toe, which were not known in this study. Tscheotarioff (1973) believed that buckling of centrally-loaded vertical end-bearing piles should not be a concern as the surrounding soil or even soft clay provides adequate lateral support. Since the top of the test pile was embedded 305 mm into a concrete pilecap, the pile top-end condition was assumed to be rotation-fixed and translation-free. Two extreme pile toe-end conditions, fixed and pinned supports, were assumed with respective K values of 1.20 and 2.0. The pile length was assumed to be fully unbraced (L is the total pile length) and 50% braced (L is half of the total pile length). Based on these assumptions, structural resistances of the test pile were calculated. Table 16 summarizes the structural resistances of fifteen test piles for two pile toe conditions and two bracing conditions. The full yield strength ($F_y A_g$) of the Grade 50 steel test pile and its 25%, 50% and 75% yield strength values are summarized in Table 17 for comparison purposes.

Table 16 Summary of Nominal Structural Pile Resistances for Fully Unbraced and 50% Braced Conditions

Project	Test Pile	Nominal Structural Pile Resistance (kN)			
		Fixed Toe (K=1.2)		Pinned Toe (K=2.0)	
		Fully Braced	50% Braced	Fully Braced	50% Braced
Burns South	Pi3P1	952	3011	343	1375
	A1P1	952	1121	102	405
Casper	A2P1	952	3011	343	1375
Torrin.	A2P1	952	1121	102	405
Owl	B2P5	1237	3341	445	1779
Woods	Pi2P1	1334	2718	480	1784
PB-Parsons	A1P5	89	365	31	129
	A2P1	125	498	44	178
PB-Muddy Creek	A2P1	245	979	89	351
	B2P1	569	1975	205	818
	B3P10	489	1802	173	703
PB-Beech Street	A1P1	320	1277	116	458
	A1P5	320	1277	116	458
	A2P1	347	1392	125	503
	A2P3	325	1303	116	472

K−Effective Length Factor.

Comparing the estimated structural resistances (Table 16 and Table 17) to their respective CAPWAP-measured pile resistances (Table 19), resistance ratios for all fifteen test piles at each boundary condition were determined. Their statistical parameters are summarized in Table 18 and Table 19. The structural boundary condition with a fixed pile toe and 50% braced length summarized in Table 18 provided the best pile resistance estimation with a bias value of 1.317 closer to one and lowest COV value of 0.683. For a fully braced condition as illustrated in Table 19, pile resistance, assuming 50% yield strength ($0.5F_y A_p$), provided a bias value closer to one. The COV value of 0.363 was the same for four different yield strengths because same data set and analysis approach were used in this study. The yield strength of the steel pile was not fully mobilized since all CAPWAP-measured total pile resistances were smaller than the full yield strength ($F_y A_p$) of the steel piles. This observation suggests that geotechnical rather than structural strength governed the axial pile resistance on soft rock.

Table 17 Summary of Nominal Structural Pile Resistances for Fully Braced Conditions

Project	Test Pile	Nominal Structural Pile Resistance (kN) 0.25 F_yA_p	0.50 F_yA_p	0.75 F_yA_p	F_yA_p
Burns South	Pi3P1	1192	2380	3572	4760
	A1P1	1192	2380	3572	4760
Casper	A2P1	1192	2380	3572	4760
Torrin.	A2P1	1192	2380	3572	4760
Owl	B2P5	1192	2380	3572	4760
Woods	Pi2P1	863	1726	2584	3447
PB-Parsons	A1P5	863	1726	2584	3447
	A2P1	863	1726	2584	3447
PB-Muddy Creek	A2P1	863	1726	2584	3447
	B2P1	863	1726	2584	3447
	B3P10	863	1726	2584	3447
PB-Beech Street	A1P1	863	1726	2584	3447
	A1P5	863	1726	2584	3447
	A2P1	863	1726	2584	3447
	A2P3	863	1726	2584	3447

F_y–Yield Stress of Steel H-Pile; and A_p–Pile Toe Area.

Table 18 Summary of statistical parameters of structural resistances for both fixed and pinned toe conditions

Toe Condition (Pile Bracing)	N	Ratio of Measured to Estimated Total Resistances (R_m/R_e) Max	Min	λ	σ	COV
Fixed (Fully Unbraced)	15	15.137	1.117	4.448	3.841	0.864
Fixed (50% Braced)	15	3.785	0.449	1.317	0.899	0.683
Fixed (Proposed)	15	2.029	0.542	0.969	0.385	0.397
Pinned (Fully Unbraced)	15	42.062	3.373	13.902	10.493	0.755
Pinned (50% Braced)	15	10.512	0.843	3.481	2.618	0.752
Pinned (Proposed)	15	1.357	0.400	0.633	0.249	0.393

Table 19 Summary of statistical parameters of structural resistances for a fully braced length condition

Percent of Yield Strength	N	Ratio of Measured to Estimated Total Resistances (R_m/R_e) Max	Min	λ	σ	COV
25%	15	3.463	0.893	1.714	0.623	0.363
50%	15	1.732	0.447	0.857	0.311	0.363
75%	15	1.154	0.298	0.571	0.208	0.363
100%	15	0.866	0.223	0.429	0.156	0.363

6. RECOMMENDATION

To facilitate the resistance estimation of piles driven on soft rock materials, it is beneficial to evaluate the pile-geo-material interaction in terms of the amounts of pile bracing from the surrounding geo-material and boundary conditions. Matching the nominal total pile resistance estimated from CAPWAP at EOD to its structural compressive capacity calculated using Eqs. (4) through (6), a required pile bracing factor (ξ) (a ratio of braced length to total embedded pile length (L_e)), was determined for each test pile for the two pile toe conditions of pinned and fixed supports. A relationship of braced pile length and embedded pile length was established in Figure 6 for steel H-piles driven primarily in silty sand and on soft rock materials. The rationale of matching the resistances assumes that the pile resistance will be governed by its structural strength, although this is not always the case, while the geotechnical strength will be indirectly accounted for in terms of the pile bracing factor (ξ). It is believed that this approach will improve current pile resistance estimation and alleviate the discrepancy between estimated and measured resistances prior to resorting to an extensive research study.

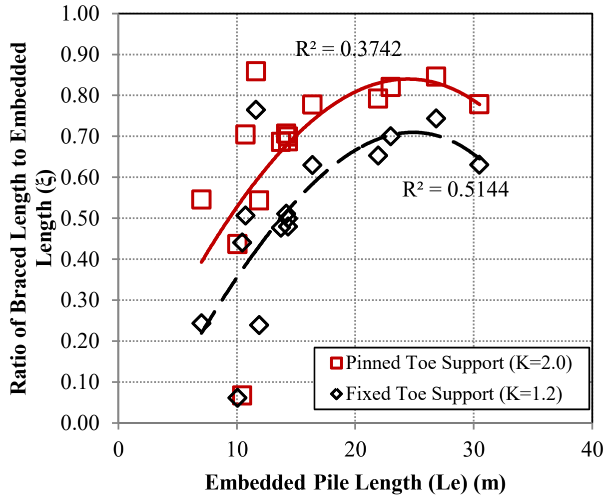

Figure 6 A relationship between pile bracing (ξ) and embedded pile length (L_e)

Recognizing that limited data were available for this study, the following observations and recommendations provide the basis for future investigations and should be further validated when more pile data become available:

(1) Pile bracing increases with increasing embedded pile length up to about 24 m and decreases thereafter. This implies that the overall contribution of surrounding soil/rock confinement to the structural pile analysis decreases for an embedded pile length greater than 24 m;

(2) The required bracing based on a pinned-toe support is about 0.14 times larger than that of a fixed-toe support at the embedded pile length of 24 m;

(3) The percent bracing contributed from the surrounding geo-material for an embedded pile length less than 6 m or greater than 31 m is not available;

(4) The maximum possible bracing based on the regression curves increases from 0.70 for a fixed-toe support to 0.84 for a pinned-toe support;

(5) Regression equations to define the pile bracing for both fixed- and pinned-toe supports are shown in Eqs. (7) and (8), respectively. The unbraced pile length (L) required in the calculation of P_e by Eq. (6) can be determined in terms of known embedded pile length (L_e) using Eq. (9). Pile resistance can be estimated using either Eq. (4) or Eq. (5) based on the estimated unbraced pile length (L);

(6) Pile resistances (R_e) estimated using the proposed method were compared with CAPWAP-measured total pile resistances (R_m) at EOD (see Table) in terms of total resistance ratios (R_m/R_e). The statistical parameters of the resistance ratios for both pile toe conditions are included in Table 18 for comparison. The proposed method based on the fixed-toe support provides a better match with the bias value of 0.969 closest to one and a relatively low COV of 0.397. When compared with the statistical parameters of the static analysis methods presented in Table 4, the proposed method provides a more accurate total pile resistance estimation.

(7) The pile toe fixity depends on the rock quality and the length of pile penetration into the rock bearing layer. For similar COV values, the relatively higher bias value of 0.969 for the fixed-toe support than the bias value of 0.663 for pinned-toe support summarized in Table 18 suggests that the pile toe behaves more like a fixed than a pinned support.

$$\xi = -2\times10^{-5}(L_e)^3 - 0.0003(L_e)^2 + 0.0546(L_e) - 0.1425 \tag{7}$$

$$\xi = -9\times10^{-6}(L_e)^3 - 0.0009(L_e)^2 + 0.0631(L_e) \tag{8}$$

$$L = (1-\xi)\times L_e \tag{9}$$

7. CONCLUSIONS

Recognizing the design and construction challenges WYDOT faces with steel H-piles driven on soft rock, detailed analyses of fifteen test piles at six bridge projects were conducted with the goal of improving current pile design and construction control procedures. The study drew the following conclusions:

(1) Current static analysis methods provide inconsistent and potentially conservative geotechnical resistance estimations of piles driven on soft rock. Among the five methods discussed, DRIVEN provides the most accurate estimates of total pile resistance. SPT provides the least accuracy. All static analysis methods underestimated shaft resistance and end bearing.

(2) The end bearing of piles driven on soft rock contributes about 40% of the total pile resistance;

(3) On average, at 24 hours after EOD, total pile resistance and shaft resistance increased by about 9% and 30%, respectively, while the end bearing decreased by about 8%. The results showed that pile setup was totally contributed by the shaft resistance. The decrease in end bearing on soft rock could be a result of relaxation.

(4) WEAP provided more accurate estimations of pile resistance than static analysis methods. Generally, WEAP tends to slightly overestimate the total pile resistance and shaft resistance but underestimate the end bearing.

(5) A positive trend was observed between unit end bearing and SPT N-value. However, no clear trend was observed for unconfined compressive strength and rock quality designation of soft rock.

(6) Pile performances were mostly satisfied when PDA/CAPWAP was used as the construction control and 24-hour restrike tests were performed. Thus, it is a good practice to evaluate the pile performance using PDA/CAPWAP and include restrikes in the testing program;

(7) Piles driven through overburden soil and into rock were neither fully unbraced nor braced. Among the eight structural conditions discussed, the structural boundary condition with a fixed pile toe and 50% braced length provided the best estimation of pile resistance. Furthermore, the yield strength of the steel pile was not fully mobilized since all CAPWAP-measured total pile resistances were smaller than the full yield strengths (F_yA_p) of the steel piles. This observation suggests that geotechnical strength rather than structural strength governed axial pile resistance on soft rock.

(8) Pile bracing increases with increasing embedded pile length up to about 24 m and decreases thereafter. The bracing factor (ξ) for the pinned-toe support is higher than that of the fixed-toe support. However, the bracing factor (ξ) is currently not available for piles with embedded lengths shorter than 6 m and longer than 31 m.

(9) To improve the accuracy of resistance estimations of piles driven on soft rock, a new method was proposed. This method assumes that structural strength governs the design and indirectly accounts for geotechnical strength in terms of pile bracing. Relationships between the bracing factor (ξ) and embedded pile length (L_e) for both fixed-toe and pinned-toe supports were established. The results show that the proposed method based on fixed-toe support provides a better match than that based on pinned-toe support. This also implies that the pile toe condition behaves more like a fixed than a pinned support. Statistical parameters indicate that the proposed method provides a more accurate total pile resistance estimation than static analysis methods.

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