Case studies to demonstrate challenges of driven piles on rock

1 Kam Ng PhD, PE

Assistant Professor, Department of Civil and Architectural Engineering,
University of Wyoming, Laramie, WY, USA (corresponding author:
kng1@uwyo.edu) (Orcid:0000-0001-5099-5454)

2 Todd Sullivan PG

Geologist, Geology Program, Wyoming Department of Transportation,
Cheyenne, WY, USA

In Wyoming, pile foundations for bridges are often driven on rock materials because of the state's shallow bedrock stratigraphy. Unfortunately, no static analysis methods are currently available for estimating the resistance of these driven piles. In this paper, two recently completed bridge projects (the Owl Creek and Woods Wardell sites) on steel H-piles in Wyoming are explicitly presented, and data from three past projects are included to highlight the limited knowledge and challenges pertaining to the present design and construction practices. Static analysis methods were used to estimate the geotechnical resistances of these piles. The wave equation analysis program and the case pile wave analysis program were used to verify their performances during construction. Structural capacities of these piles were also calculated. The results of the studies show that the static analysis methods and structural analyses yield inconsistent pile resistance estimations. Recommendations in terms of pile bracing and embedded pile length are proposed to predict better the resistances of piles on soft rock.

Notation

A_g	cross-sectional area of a pile
F_y	minimum yield strength of a steel pile
K	effective length factor
L	unbraced pile length
P_e	elastic critical buckling resistance
P_n	nominal structural capacity of a pile
P_o	equivalent nominal yield resistance
Q	applied load
q_u	uniaxial compressive strength
R	nominal pile resistance
R_p	nominal end bearing
R_s	nominal side resistance
r_s	radius of gyration about the axis normal to the place of buckling
γ	load factor
ϕ	resistance factor
ψ	slender element reduction factor

Introduction

The shallow bedrock stratigraphy in Wyoming, USA results in steel H-piles with a high driving durability on rock often being used as the foundation system to support bridges in the state. Driven steel H-piles are preferred over drilled shafts because of the local availability of steel pile materials, driving equipment and experienced contractors. Also, for a moderate structure load of a typical bridge in Wyoming, the driven steel pile system is more cost-effective than the drilled

shaft foundation system. The total axial resistance of these piles consists of a combination of side resistance and end bearing. To attain the required capacity, particularly in a soft overburden soil, the pile would have to rely on its resistance on a stiff rock or intermediate geomaterials (IGMs) by driving the pile to a refusal onto this layer. According to the Wyoming Department of Transportation (WYDOT) practice discussed below in the section headed 'WYDOT specifications and practices', a refusal blow count of 120 blows per 305 mm is recommended to prevent pile overstresses and damages. Unfortunately, there are currently no pragmatic static analysis methods available for estimating the side resistance and end bearing of a driven pile on rock. For example, the characteristic line method proposed by Serrano and Olalla (2002) based on Hoek and Brown's non-linear failure model requires advanced rock parameters that are not readily available for an ultimate end bearing estimation. IGMs and rock materials are typically characterised based only on rock quality designation (RQD) and uniaxial compressive strength (q_u). Serrano and Olalla (2002) concluded that the proposed method is acceptable for piles bearing in poor rock with q_u of values less than 20–30 MPa (418–627 kilopounds/square foot (ksf)). However, the method is believed to be unconservative for rocks with q_u values greater than 30 MPa (627 ksf).

Pile–rock contact area, penetration depth and rock quality are usually not available for pile resistance estimation during the 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. It was noted that a pile's resistance will usually be governed by its structural strength when it is driven to the end bearing on a rock of fair-to-excellent quality based on RQD values. On the other hand, a pile supported on soft weathered rock should be designed based on pile load test results because (a) the rock strength would govern the pile resistance and (b) pile resistance could decrease due to relaxation on soft weathered rock near the pile toe (Thompson and Thompson, 1985). However, a clear distinction between soft and hard rocks is not available, and they are normally differentiated based on local experiences. According to the American Association of State Highway and Transportation Officials's (Aashto) load and resistance factor design (LRFD) bridge design specifications (Aashto, 2014), soft rocks are not well distinguished from hard rocks or soils in the design and construction of driven piles. For IGM, a soft rock or stiff heavily consolidated soil material, Aashto (2014) normally considers drilled shaft (i.e. bore pile or cast-in-place pile) design and construction rather than driven piles. Defined by O'Neill and Reese (1999), cohesive IGMs are clay shales or mudstones with unconfined compressive strengths (q_u) of 0.5–5 MPa. Cohesionless IGMs are granular tills or granular residual soils with corrected standard penetration test (SPT) N values (N_{160}) falling between 50 and 100 blows per 300 mm penetration.

These limitations create challenges in the design and construction of pile foundations on rock as experienced by the WYDOT. A relatively high uncertainty of pile performance at sites was observed. This presented a challenge to construction management. Additionally, the estimated pile resistance based on the governed structural limit state was often not attainable during construction as demonstrated in this paper. Pile restrikes, dynamic load testing, pile extensions, increase in pile size and/or enlargement of pile caps were required to achieve the necessary pile resistance. This incurs additional construction duration and cost, which could create conflicts between owners and piling contractors. A recent study on three case studies of steel H-piles on soft rocks 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. Hence, it is important to establish methods to distinguish rock materials for driven pile design and construction as well as develop methods to predict better the resistances of piles driven on soft rocks.

Limitations and challenges pertaining to present design and construction of driven piles on rock are explicitly demonstrated by two case studies of completed bridge projects in Wyoming. The first case study was a bridge project constructed over Owl Creek in Hot Springs County, Wyoming. The geology consisted of silty sand overlying weathered shale, which in turn overlaid unweathered shale bedrock. Groundwater was encountered at approximately 2.1 m (7 ft) below the ground. The second case

study was a bridge project for Woods Wardell Road constructed over the Green River in Sublette County, Wyoming. The geology consisted of silty sand and gravel overlying weathered siltstone to claystone bedrock, which in turn overlaid unweathered siltstone to claystone bedrock. Using the results from these two case studies as well as those from three past projects, current challenges of driving piles on rock are highlighted. These additional past projects were denoted as Burns South, Casper and Torrington. The geology of Burns South and Casper was sandstone bedrock, while the geology of Torrington was claystone bedrock. The design and construction procedures and assumptions are discussed in the section headed 'WYDOT specifications and practices'. Recommendations in terms of performing pile restrikes, rigorous pile analysis using Pile Dynamics, Inc.'s case pile wave analysis program (Capwap) and modified structural analysis proposed in this study are suggested to alleviate the limitations.

Aashto's LRFD bridge design specifications

Recommendations for piles driven on rock suggested by the *AASHTO LRFD Bridge Design Specifications* (Aashto, 2014) are described as follows.

- (a) The resistance of a pile driven on soft rock shall be determined in the same manner as soil (article 10.7.3.2.2). The resistance factors of the static analysis methods and dynamic methods for driven piles are summarised in article 10.5.5.2.3.
- (b) Piles driven in hard rock shall be governed by the structural limit state described in article 6.9.4.1. The structural compressive resistance of a pile shall be considered as the smallest value based on any applicable buckling failure modes.
- (c) Local experience shall be applied to define the quality of rock, as there are no well acceptable approaches to differentiating between soft and hard rocks (article C10.7.3.2.1).
- (d) Locally developed driving criteria shall be applied to prevent pile damage.
- (e) According to article 6.5.4.2 of the Aashto specifications for a structural design, a resistance factor (ϕ) of 0.50 was recommended for steel H-piles subject to damage due to severe driving conditions where the use of a pile tip is required and a higher ϕ value of 0.60 was recommended for steel H-piles under good driving conditions where the use of a pile tip is not required.

It is important to note that the structural resistance depends on the effective length of a pile (i.e. a product of the effective length factor (K) and an unbraced pile length (L)). The effective pile length (KL) is influenced by the rotational and translational bracing along its length, contributed by the rarely-known overburden soil confinement and the rock fixity at its toe. Recommendations to address this issue are not provided by Aashto (2014). By neglecting the pile–geomaterial interaction and assuming an unrealistic KL value, the pile resistance will not be accurately estimated in terms of its structural compressive strength, leading to a potentially large discrepancy between estimated and measured resistances as demonstrated by the two case studies presented in this paper.

WYDOT specifications and practices

The *AASHTO LRFD Bridge Design Specifications* (Aashto, 2014) and local experiences are being adopted by WYDOT for pile designs and constructions. A site investigation is performed at every bridge project to determine its subsurface profile and geomaterial properties. SPT is the most commonly used in situ field test in Wyoming. At the same borehole for the SPT test, a drivepoint penetration test is performed by driving a 50 mm dia. (2 inch dia.) drivepoint, as shown in Figure 1, into the ground using a 63.5 kg (140 lb) hammer at a drop height of 760 mm (30 inches). The hammer blows to penetrate the drivepoint 305 mm (1 ft) into the ground are recorded. The main purpose of the drivepoint penetration test is to determine the depth of an adequate bearing layer, such as unweathered bedrock, for end-bearing piles. Based on experience, a bearing layer and its depth can be identified when the drivepoint's hammer blow count exceeds 100 blows per 100 mm penetration. Soil samples collected from the site investigation will be classified in accordance with the Unified Soil Classification System (USCS) (ASTM D 2487 (ASTM, 2011) and the Aashto classification system (M 145 (Aashto, 2012)). Occasionally, unconfined compressive tests will be performed on cohesive soil samples to determine their respective undrained shear strengths. If a bedrock layer is encountered, a rock coring will be performed to determine the RQD value, and collected rock samples will be tested for the q_u values.

Based on past pile load test data, WYDOT developed a table of typical properties of compacted soils, which facilitates the quantification of the geotechnical resistance of piles. However, the locally calibrated unit shaft resistance and the unit end bearing of piles driven on rock are not available. The Nordlund (1963) method is frequently used to estimate the geotechnical resistance of piles in cohesionless soils, while the cone penetration test method by Nottingham and Schmertmann (1975) is used for cohesive soils. Since these static analysis methods were developed for soil materials, the estimated geotechnical resistances are often less than the applied loads, and the LRFD strength limit state requirements

are usually not satisfied during the design stage. Based on a drivepoint result which determines the depth of a bearing layer, a pile penetration depth can be estimated for contract document preparation and cost estimation. This design procedure is being adopted by assuming that a sufficient pile resistance can be achieved and the LRFD strength limit state can be satisfied through its structural capacity during the construction stage. In other words, the pile structural capacity is assumed to govern the pile performance during the construction stage when the pile is driven to a refusal.

The WYDOT's (2012) *Standard Specifications for Road and Bridge Construction* and section 504 of WYDOT's (2014) *Construction Manual* describe the following construction control requirements for furnishing and driving steel bearing piles.

- (a) The adequacy of a pile hammer will be evaluated using a wave equation analysis method.
- (b) Pile driving stresses shall not exceed 90% of the minimum yield strength of a steel pile.
- (c) A dynamic formula, wave equation or both will be used to establish pile driving criteria. Pile driveability analysis using the wave equation analysis program (Weap) will be performed to avoid potential pile damage while penetrating the pile further into a good bearing layer to attain the target resistance. If necessary, a pile driving analyser (PDA) with subsequent signal matching analyses using Capwap will be used to determine and verify the required pile resistance during construction. Pile restrikes at 24 h after the end of driving (EOD) will be required to ensure further that the desired pile resistance is achieved.

Pile driving is mostly performed using locally available diesel hammers. A refusal blow count of 120 blows per 305 mm is used by WYDOT to prevent overstressing and damage to the pile. Pile driving will be terminated when a target nominal pile resistance is achieved at the planned depth, verified using the Weap and/or the PDA with Capwap analysis. PDA with Capwap is often used as a

Figure 1. 50 mm dia. drivepoint and extension steel rod

group of five grade 50 HP 360 $\times$ 109 (HP 14 $\times$ 73) piles was installed at each bent. Piles were designed in accordance with the *AASHTO LRFD Bridge Design Specifications* (Aashto, 2014). To satisfy the LRFD strength limit state ($\gamma Q \leq \phi R$) requirement, the factored axial resistance of each pile (ϕR) at the abutment shall be greater than the specified factored load (γQ) of 694 kN (156 kips). Likewise, the factored resistance of each pile at the bent shall be greater than 1103 kN (248 kips). An International Construction Equipment 42S diesel hammer was used to install the piles and perform a restrrike test. Pile driving criteria based on bearing graphs generated by Weap were used to verify pile performance. All steel piles were driven to refusal in unweathered shale bedrock, and the pile toe elevation was estimated at 1570 m (5150 ft). Pile 5 at bent 2 (B2P5) was selected as the test pile for dynamic load testing

Background

group of five grade 50 HP 360 $\times$ 109 (HP 14 $\times$ 73) piles was installed at each bent. Piles were designed in accordance with the *AASHTO LRFD Bridge Design Specifications* (Aashto, 2014). To satisfy the LRFD strength limit state ($\gamma Q \leq \phi R$) requirement, the factored axial resistance of each pile (ϕR) at the abutment shall be greater than the specified factored load (γQ) of 694 kN (156 kips). Likewise, the factored resistance of each pile at the bent shall be greater than 1103 kN (248 kips). An International Construction Equipment 42S diesel hammer was used to install the piles and perform a restrrike test. Pile driving criteria based on bearing graphs generated by Weap were used to verify pile performance. All steel piles were driven to refusal in unweathered shale bedrock, and the pile toe elevation was estimated at 1570 m (5150 ft). Pile 5 at bent 2 (B2P5) was selected as the test pile for dynamic load testing

using PDA. Capwap was used for subsequent signal matching analysis. A restrike at 24 h after the EOD of the test pile was specified and performed to ensure that the desired capacity had been achieved.

Subsurface

The subsurface consisted of loose-to-very dense silty sand and gravel overlying weathered shale, which, in turn, overlaid very hard, unweathered shale bedrock as shown in Figure 4. Groundwater elevation was measured at approximately 1579 m (5179 ft). Two SPT boreholes were created. The uncorrected SPT N values of the silty sand and weathered shale were 7 and 20, respectively. At each borehole location, a drivepoint penetration test was performed to determine the depth of an adequate pile bearing layer. The silty sand was classified as poorly graded sand (SP) in accordance with the USCS. A weathered shale sample was collected in a Shelby tube for a triaxial compression test. The cohesion and the friction angle were determined to be 0.14 MPa (2.91 ksf) and 12° , respectively. Unweathered shale samples were collected for uniaxial compression tests to determine q_u values, ranging from 0.15 to 0.69 MPa (3.04–14.35 ksf) with an average q_u value of 0.4 MPa (8.42 ksf). This unweathered shale could be classified as an IGM with some q_u values greater than 0.48 MPa (10 ksf) (O'Neill and Reese, 1999). Also, RQDs of the shale were between 10 and 67% with an average RQD of about 45%.

Analytical results

Geotechnical and structural analyses were performed on the test pile driven 4.97 m (16.3 ft) into the silty sand layer, 3.96 m (13.0 ft) into the weathered shale and 1.52 m (5.0 ft) into the unweathered shale bedrock with a total embedded length of

10.45 m (34.3 ft). The SPT-Meyerhof method (Meyerhof, 1976), Nordlund (1963) method, Driven method (Mathias and Cribbs, 1998) and β -method (Burland, 1973) were selected to estimate the nominal pile side resistance (R_s) in the silty sand layer. Understanding that static analysis methods are not available for estimating pile resistance in the shale layers and treating the shale as a cohesive soil, the α -method (Tomlinson, 1980) and the λ -method (Vijayvergiya and Focht, 1972) were selected to estimate both side resistance and end bearing (R_p) in the shale layers. Driven is a program that estimates pile resistance based on a combination of the Nordlund (1963) method for cohesionless soil and the α -method (Tomlinson, 1980) for cohesive soil. Four combinations of static analysis methods along with the Driven method and the WYDOT method were applied to estimate the total pile resistances as summarised in Figure 5. Dynamic tests using a PDA were performed on the test pile at the EOD on 2 July 2014 and at restrike the next day. Hammer blow counts increased from 862 blows/m (263 blows/ft) at the EOD to 1180 blows/m (360 blows/ft) at the beginning of restrike (BOR). The nominal pile resistances estimated by WYDOT and the University of Wyoming (UW) using Capwap are summarised in Figure 5 for comparison purposes. Using the driving, pile, soil and hammer information, driveability analyses were performed and bearing graphs were generated using Weap. Two soil input procedures, the soil type-based method (ST) and the SPT N value-based method (SA), were used in the Weap analysis, and their results are summarised in Figure 5.

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

Figure 4. Subsurface profile of Owl Creek Bridge project

Figure 5. Summary of pile resistances (Owl Creek)

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 contributed from its surrounding soil. Hence, the structural capacity (P_n) was estimated by

$$1. \quad P_n = \left[0.658^{(P_o/P_e)} \right] P_o \quad \text{if } \frac{P_e}{P_o} \geq 0.44$$

$$2. \quad P_n = 0.877 P_e \quad \text{if } \frac{P_e}{P_o} < 0.44$$

$$3. \quad P_e = \frac{\pi^2 E}{(KL/r_s)^2} A_g$$

where A_g is the cross-sectional area of a pile, F_y is the specified minimum yield strength of a steel pile, P_e is the elastic critical buckling resistance, 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 and r_s is the radius of gyration about the axis normal to the plane of buckling. 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 easily known in this study.

Tschebotarioff (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 (1 ft) into a concrete pile cap, 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, as illustrated in Figure 5. 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, the structural resistances of the test pile were calculated. The full yield strength ($F_y A_g$) of the steel test pile and its 50% value are also included for comparison.

Applying the resistance factors (ϕ) recommended by Aashto (2014) as summarised in Table 1 to the respective nominal pile resistances (R), the factored pile resistances (ϕR) were determined in Figure 5. To evaluate the LRFD strength limit state ($\gamma Q \leq \phi R$), the factored resistances were compared with the factored load (γQ) of 1103 kN (248 kips).

Large variation in nominal pile resistances was observed, ranging between 476 and 998 kN (107 and 224 kips), estimated by static analysis methods. The combination of the λ -method for side resistance and the α -method for end bearing yielded the highest nominal resistance of 998 kN (224 kips) and factored resistance of 388 kN (87 kips), which was much lower than the factored load of 1103 kN (248 kips). Hence, the LRFD strength limit state cannot be satisfied using the static analysis methods.

Table 1. Summary of resistance factors (Aashto, 2012)

Analysis method	Resistance factor (ϕ) for $\beta = 2.33$
SPT-Meyerhof	0.30
Nordlund	0.45
Driven	0.45 for cohesionless soil
β -method	0.25
Capwap	0.65
Weap	0.50
Structural capacity	0.60

β , reliability index of 2.33 for a redundant pile group

Consistent pile resistances were estimated using Capwap at the EOD based on the same blow number 894 (BN894). Although two different blow numbers (BN10 and BN31) at the BOR were used by UW, the estimated pile resistances by Capwap (i.e. 963 and 984 kN) were comparable. Resistances at the BOR estimated by UW were 6–8% lower than those estimated by WYDOT. The 1-d restrike test showed that the pile resistance increased by about 4–8%. The actual pile set-up cannot be confirmed as the hammer energy transfer ratio (ETR) values determined by PDA increased slightly from 30.2% at the EOD to 36.2% at the BOR. It was believed that apparent pile set-up was observed (Ng *et al.*, 2013). Compared with the factored load (γQ) of 1103 kN (248 kips), the pile resistance of 1050 kN (236 kips) at BOR by WYDOT using Capwap was about 5% lower as shown in Table 2. Considering the extremely small difference for piles bearing on unweathered shale bedrock and the continuous gain in pile resistance over time, the pile performance was accepted.

Pile resistances estimated by Weap based on the driveability analyses were similar to those estimated by static analysis methods. This outcome was expected as static analysis methods, such as the β -method for cohesionless soils and the α -method for cohesive soils, were adopted in the Weap driveability analysis. Nominal pile resistances at the EOD estimated by Weap based on the bearing graph approach were slightly larger than those estimated by Capwap, while the nominal pile resistances at the BOR were comparable to WYDOT's Capwap estimations. The increase in total pile resistances by Weap was expected as the hammer blow counts increased. Pile resistances estimated by Weap based on the bearing graph approach using both ST and SA input procedures were almost equal. This outcome was expected as the

pile resistance estimation was influenced chiefly by damping factors and quake values, which were similar for both procedures in this study. Since all factored resistances were smaller than the factored load, the LRFD strength limit state was not satisfied.

Since a static load test was not performed, the following observations were derived based on the estimated nominal pile resistance of 1417 kN (319 kips) at the EOD using the best available dynamic load test method by Capwap. The test pile was certainly not fully unbraced as its structural capacities at both pile toe conditions were smaller. On the other hand, the nominal resistance of 1417 kN (319 kips) was about 58 and 20% lower for the 50% braced pile with a fixed-toe support and a pinned-toe support, respectively. This observation implied that the per cent bracing was lower than 50%. Since all estimated nominal pile resistances were lower than the full yield strength of 4760 kN as illustrated in Figure 5, the yield strength of the steel pile was not fully mobilised, implying that geotechnical strength rather than structural strength governed the axial pile resistance. However, there is a possibility that the pile resistance was not fully mobilised during the dynamic testing as indicated by a relatively small pile deformation of about 0.6 mm (0.025 inch) per hammer blow recorded by PDA. This could lead to the underestimation of pile resistance by Capwap. Unfortunately, a static load test was not performed to verify the test pile performance.

Case study 2 (Woods Wardell)

Background

A 64 m (210 ft) span bridge for Woods Wardell Road was constructed over the Green River in Sublette County, Wyoming, as shown in Figure 6. The construction of the bridge foundation system began in January 2015 and was completed in the summer of 2016. The bridge consists of two abutments and two piers as shown in Figure 7. Four grade 50 HP 310 × 79 (HP 12 × 53) piles were installed in a pile group at each abutment, while a pile group of 14 grade 50 HP 310 × 79 (HP 12 × 53) piles was installed at each pier. Pile foundations were designed in accordance with the *AASHTO LRFD Bridge Design Specifications* (Aashto, 2014) and the WYDOT's (2010) *Standard Specifications for Road and Bridge Construction*. To satisfy the LRFD strength limit state requirement, the factored resistance of each pile at the pier shall be

Table 2. Summary of pile performance acceptance

Project	Number of piles ^a	Weap		Capwap	
		EOD	BOR	EOD	BOR
Owl Creek (B2P5)	5	No (–30%)	No (–27%)	No (–12%)	No (–5%)
Woods Wardell (PI2P1)	14	No (–65%)	No (–63%)	No (–3%)	Yes (10%)
Burns South (PI3P1)	21	No (–41%)	No (–40%)	No (–7%)	Yes (0.4%)
Burns south (A1P1)	5	No (–36%)	No (–34%)	Yes (2%)	Yes (11%)
Casper (A2P1)	14	Yes (33%)	Yes (33%)	Yes (30%)	Yes (46%)
Torrington (A2P1)	9	No (–15%)	Yes (6%)	No (–28%)	No (–19%)

^a Number of production piles at the respective bent, pier or abutment location

B, bent number; PI, pier number; A, abutment number; P, pile number; Yes, satisfied the LRFD strength limit state; No, did not satisfy the LRFD strength limit state; %, percentage higher (positive) or lower (negative) than the factored load

Figure 6. Plan view of Woods Wardell Bridge project

Figure 7. Sectional view of Woods Wardell Bridge project

greater than 1335 kN (300 kips). An American Piledriving Equipment Inc. D19-42 single-acting diesel hammer was used to install all piles and perform a restrrike test. Pile driving criteria were established using Weap. All steel piles were driven to refusal in unweathered bedrock. Pile 1 at pier 2 (PI2P1) was selected as the test pile for dynamic load testing. A restrrike was performed at 24 h after the EOD. No static load test was conducted. The test pile top and toe elevations were identified at 2164.8 m (7102.4 ft) and 2156.9 m (7076.4 ft), respectively.

Subsurface

The subsurface consisted of saturated silty sand and gravel (river deposits) overlying hard, dry, weathered siltstone to claystone

bedrock, which, in turn, overlaid very hard, dry, unweathered siltstone to claystone bedrock as shown in Figure 8. Groundwater elevation was observed at approximately 2169 m (7117 ft) during the subsurface investigation. Two SPT boreholes were created. The uncorrected SPT N values of the silty sand/gravel ranged between 6 and 15, while the uncorrected SPT N values of the weathered bedrock ranged between 35 and 65. At each borehole location, a drivepoint penetration test was performed to determine the depth of an adequate pile bearing layer. The silty sand was classified as silty sand (SM) in accordance with USCS. Unweathered claystone samples were collected from rock cores (i.e. tests 6–11 and tests 18–20 as shown in Figure 8) for uniaxial compression tests to determine q_u values. The q_u values ranged between 3.6 and 6.6 MPa

Figure 8. Subsurface profile of Woods Wardell Bridge project

(75–138 ksf) at abutment 1 and between 0.24 and 2.6 MPa (5–55 ksf) near pier 2. An average q_u value of 3.6 MPa (76 ksf) was reported by WYDOT. This unweathered claystone bedrock was classified as an IGM, with most q_u values falling between 0.48 and 4.8 MPa (10–100 ksf) (O'Neill and Reese, 1999). Also, RQD values of the unweathered claystone bedrock were between 20 and 100% with an average RQD of about 68%.

Analytical results

The test pile was driven 5.6 m (18.4 ft) into the weathered claystone (layer 1) and 1.4 m (4.6 ft) into the unweathered claystone bedrock (layer 2) with a total embedded pile length of 7 m (23 ft). Since the pile was completely embedded in cohesive geomaterials, the SPT method, the Nordlund method and β -method were not chosen to estimate the nominal pile side resistance. Understanding that static analysis methods are not readily available for estimating pile resistance in a claystone bedrock layer and treating the claystone as a cohesive soil, the α -method, Driven method and λ -method were selected to estimate both side resistance and end bearing. The total pile resistances estimated by four static analysis methods are summarised in Figure 9. Dynamic tests were performed on the test pile at the EOD on 6 January 2015 and at restrike the next day. Hammer blow counts increased from 420 blows/m (128 blows/ft) at the EOD to 511 blows/m (156 blows/ft) at the BOR. The nominal pile resistances estimated by GRL, Inc. and UW at different blow numbers using Capwap are summarised in Figure 9 for comparison purposes. Using the driving, pile, soil and hammer information, pile resistances estimated using Weap are summarised in Figure 9. Using the approach described in the subsection headed 'Analytical results' under the section headed 'Case study 1 (Owl Creek)', structural capacities are presented in Figure 9 for comparison purposes. Applying the ϕ values summarised in Table 1 to the respective nominal pile resistances (R), the factored pile resistances (ϕR) were included in Figure 9.

Figure 9 shows that the combination of the λ -method for side resistance and the α -method for end bearing yielded the highest nominal resistance, while the WYDOT method yielded the lowest resistances. No static analysis methods satisfied the LRFD strength limit state since all factored resistances were smaller than 1335 kN (300 kips). Compared with dynamic analysis methods, static analysis methods generally underestimated the pile resistance. This comparison concluded that presently available static analysis methods cannot accurately estimate the axial resistance of a pile driven in materials harder than soils.

Although different blow numbers were chosen, pile resistances estimated using Capwap by GRL and UW were comparable. The pile resistances by UW were about 2.4 and 1.8% higher than those by GRL for the EOD and BOR conditions, respectively. Nominal pile resistances increased about 5–9% from the EOD to BOR, although the ETR values decreased slightly from 38.1% at the EOD to 35.7% at the BOR. Only pile resistances estimated at the BOR by Capwap satisfied the LRFD strength limit state. This observation revealed the importance of performing a 24-h restrike test and the subsequent analysis by Capwap.

Pile resistances estimated by Weap's driveability analyses were similar to those based on static analysis methods that were adopted in the analysis. Nominal pile resistances at the EOD and BOR conditions estimated by Weap using the bearing graph approach were lower than those estimated by Capwap. Increases in pile resistances by Weap were also expected as hammer blow counts increased from the EOD to BOR. If Weap was the only construction control method, the LRFD strength limit state would not have been satisfied.

The structural capacities were compared with the best estimated nominal pile resistance of 2001 kN (450 kips) at the EOD using Capwap. The test pile was certainly not fully unbraced as its structural capacities at both pile toe conditions were smaller. On

Figure 9. Summary of pile resistance (Woods Wardell)

the other hand, the resistance of 2001 kN (450 kips) was about 26% lower and 12% higher for the 50% braced pile with a fixed-toe support and a pinned-toe support, respectively. This observation implied that if the pile was truly 50% braced, the pile toe would behave in between a fixed and a pinned condition. However, the per cent bracing will be lower than 50% if the pile toe was fixed and higher than 50% if the pile toe was pinned. The yield strength of the steel pile was not fully mobilised, implying that geotechnical strength rather than structural strength governed the axial pile resistance. Similar to the test pile at the Owl Creek Bridge project, the pile resistance may not have been fully mobilised during the dynamic testing as indicated by a relatively small pile deformation of about 2.1 mm (0.083 inch) per hammer blow recorded by PDA. This could lead to the underestimation of pile resistance by Capwap. Unfortunately, a static load test was not performed to validate these observations.

Discussion and recommendations

Table 2 summarises the pile performance acceptances for the two case studies described in this paper and the three case studies described by Ng *et al.* (2015) using either Weap or Capwap as the construction control method. A pile performance is accepted when the factored pile resistance is greater than the required factored load.

The two test piles PI3P1 and A1P1 of the Burns South Road project with embedded pile lengths of 11.9 m (39 ft) and 22 m (72 ft) were driven in silty sand and terminated in sandstone bedrock. The test pile A2P1 of the Casper Street project with an embedded pile length of 10.1 m (33 ft) was driven in silty-to-gravelly sand and terminated in unweathered sandstone bedrock. The test pile A2P1 of the Torrington Street project with an embedded pile length of 30.5 m (100 ft) was driven in well-graded sand followed by poorly graded sand and well-graded gravel and eventually terminated in weathered claystone bedrock. All four test piles were HP 360 × 109 (HP 14 × 73), installed in Wyoming, USA.

The results summarised in Table 2 indicate that five out of six test piles did not satisfy the LRFD strength limit state when Weap was used as the only construction control method at the EOD event. Even if restrike tests were performed at the BOR event, four test piles still did not satisfy the LRFD strength limit state. The performance of these production piles was considered following the performance of the test pile at the same bridge structural location. For example, the performance of the five production piles at the Owl Creek project based on either Weap or Capwap was not acceptable since the test pile did not satisfy the LRFD

strength limit state. For a total of 68 production piles at their respective test pile locations, 45 piles or 66% were considered unacceptable when Weap was used as the only construction control method at the BOR. When PDA/Capwap was used as the construction control method at the EOD, 49 piles (72%) were considered unacceptable. However, the number of unacceptable piles reduced to 14 (21%) at the BOR condition. It is also important to recognise that the dynamic test using PDA with Capwap analysis is not a proof load test and does not provide unique resistance estimation (Ng and Sritharan, 2013). Alternatively, a static load test could be conducted to verify the pile performance. Unfortunately, it has not been a local practice to conduct a static load test. This discussion clearly illustrates the current limitations and challenges for driven piles on rock. A combination of limited geomaterial properties, absence of a pragmatic static analysis method, unknown pile–geomaterial interaction and no proof load test led to high uncertainty in pile resistance estimations and performance verifications.

An immediate need for research on piles driven on rock or very stiff geomaterials is plain. Research should produce outcomes described as follows

- a database of rock properties and load tests on piles driven on rock or very stiff geomaterials
- a static analysis method to estimate the geotechnical resistance of a pile on rock
- a guideline for evaluating the structural pile capacity considering the different surrounding soil confinements and pile toe conditions
- adequate design and construction control recommendations in the Aashto's LRFD bridge design specifications for piles driven on rock.

To initiate the resistance estimation of piles driven on rock or IGM, the pile–geomaterial interaction in terms of pile bracing from its surrounding geomaterial was evaluated at two assumed pile toe fixities: fixed and pinned. Matching the nominal resistance estimated from Capwap at the EOD to its structural compressive capacity calculated using Equations 1–3, the required per cent pile bracing, the ratio of braced length to total embedded pile length in percentage, was determined for each test pile for both pile toe conditions. A relationship of per cent bracing and embedded pile length was established in Figure 10 for steel H-piles driven in soil overlying rock or IGM. The rationale of matching the resistances assumes that the pile resistance will be governed by its structural strength, although it is not always the case, while the geotechnical resistance will be indirectly accounted for in terms of per cent pile bracing. It is believed that this approach will improve the current pile resistance estimation and alleviate the discrepancy between estimated and measured resistances. Recognising that limited data were available in this study, the following observations provide a basis for future investigations and should be further validated when more pile data become available.

Figure 10. A relationship of per cent bracing and embedded pile

- The per cent bracing increased with increasing embedded pile length. A logarithmic relationship can be established between the per cent bracing and the embedded pile length with reasonably good coefficient of determination (R^2).
- The required average per cent bracing based on a pinned-toe support was larger than that based on a fixed-toe support. However, the difference decreases with increasing embedded pile length.
- Based on the currently available test results, the maximum possible per cent bracing increased from about 65% for a fixed-toe support to 80% for a pinned-toe support.
- When the regression fit curves were extrapolated backwards, the per cent bracing approached zero at embedded lengths of about 1.2 m (4 ft) and 6.4 m (21 ft) for the pinned-toe and fixed-toe supports, respectively. This observation suggests that the per cent bracing becomes insignificant in the resistance estimation when the embedded pile length is short. However, additional pile data and further analysis are needed to validate this observation and improve the correlation. One of the possible analyses could be to perform a laterally loaded pile analysis to determine the point of fixity of the pile which could minimise the uncertainty associated with its structural behaviour and better describe the pile restraints.

Conclusions

Motivated by the challenges faced by WYDOT on steel H-piles driven on rock and IGM, detailed pile analyses of two case studies along with three case studies conducted by Ng *et al.* (2015) were used to illustrate the limitations of current pile design and construction control procedures. It is envisioned that this paper will facilitate research on piles driven on rock. This study drew the following conclusions.

- Current static analysis methods provided inconsistent and potentially conservative geotechnical resistance estimations of a driven pile on rock.
- Construction control using Weap produced a higher uncertainty than that based on Capwap. This finding agreed

with the relatively lower resistance factors for Weap than those for Capwap promulgated in the *AASHTO LRFD Bridge Design Specifications* (Aashto, 2014).

- (c) Apparent pile set-up was observed 1 d after the EOD, and most pile performances were satisfied during the restrrike tests. Thus, it is recognised as a good practice by most agencies to include a restrrike test in a pile construction control program.
- (d) Piles driven in overburdened soil and into rock were neither fully unbraced nor braced. To facilitate the resistance estimation of piles driven into rock materials by assuming that the pile's structural strength governs the design while indirectly accounting for the geotechnical strength in terms of pile per cent bracing, a logarithmic relationship between per cent bracing and embedded pile length was established. Pile per cent bracing increases with increasing embedded pile lengths. Based on this study, the maximum per cent bracing increases from about 65% for a fixed-toe support to 80% for a pinned-toe support. Per cent bracing for a fixed support is always lower than that for a pinned support. It is important to note that these observations were concluded based on limited pile data. However, these serve as a basis for future investigations and should be further validated when more pile data become available.
- (e) The yield strength of the steel pile was not fully mobilised due to the constraint of its geotechnical resistance. However, there is a possibility that the pile resistance was not fully mobilised during the dynamic testing as indicated by a relatively small pile deformation recorded by PDA. This could lead to the underestimation of pile resistance by Capwap. Unfortunately, a static load test was not performed to verify the test pile performance.

Acknowledgements

The authors would like to thank WYDOT for sponsoring the project.

REFERENCES

- Aashto (American Association of State Highway and Transportation Officials) (2012) M 145: Standard specification for classification of soils and soil–aggregate mixtures for highway construction purposes. Aashto, Washington, DC, USA.
- Aashto (2014) *AASHTO LRFD Bridge Design Specifications*, 7th edn. Aashto, Washington, DC, USA.
- ASTM (2011) D 2487: Standard practice for classification of soils for engineering purposes. ASTM International, West Conshohocken, PA, USA.
- Burland JB (1973) Shaft friction of piles in clay. *Ground Engineering* **6**(3): 30–42.
- Hannigan PJ, Goble GG, Likins GE and Rausche F (2006) *Design and Construction of Driven Pile Foundations – Volume I*. Federal Highway Administration, Washington, DC, USA, FHWA-NHI-05-042.
- Mathias D and Cribbs M (1998) *DRIVEN 1.0: a Microsoft Windows Based Program for Determining Ultimate Static Pile Capacity*. Federal Highway Administration, Washington, DC, USA.
- Meyerhof G (1976) Bearing capacity and settlement of pile foundations. *Journal of the Geotechnical Engineering Division, ASCE* **102**(3): 195–228.
- Ng KW and Sritharan S (2013) Improving dynamic soil parameters and advancing the pile signal matching technique. *Computers and Geotechnics* **54**: 166–174, <http://dx.doi.org/10.1016/j.compgeo.2013.07.007>.
- Ng KW, Roling M, AbdelSalam SS, Suleiman MT and Sritharan S (2013) Pile setup in cohesive soil. I: experimental investigation. *Journal of Geotechnical and Geoenvironmental Engineering* **139**(2): 199–209, [http://dx.doi.org/10.1061/\(ASCE\)GT.1943-5606.0000751](http://dx.doi.org/10.1061/(ASCE)GT.1943-5606.0000751).
- Ng KW, Yasrobi SY and Sullivan AT (2015) Current limitations and challenges of driven piles in rock as demonstrated using three case studies. In *IFCEE 2015* (Iskander M, Suleiman MT, Anderson JB and Laefer DF (eds)). American Society of Civil Engineers, Reston, VA, USA, Geotechnical Special Publication no. 256, pp. 500–517.
- Nordlund RL (1963) Bearing capacity of piles in cohesionless soils. *Journal of Soil Mechanics and Foundations Division, ASCE* **89**(3): 1–36.
- Nottingham L and Schmertmann J (1975) *An Investigation of Pile Capacity Design Procedures*. Department of Civil Engineering, University of Florida, Gainesville, FL, USA, Report D629.
- O'Neill MW and Reese LC (1999) *Drilled Shafts: Construction Procedures and Design Methods*. Federal Highway Administration, Washington, DC, USA, Publication No. FHWA-IF-99-025.
- Serrano A and Olalla C (2002) Ultimate bearing capacity at the tip of a pile in rock – part 1: theory. *International Journal of Rock Mechanics & Mining Sciences* **39**(7): 833–846, [http://dx.doi.org/10.1016/S1365-1609\(02\)00052-7](http://dx.doi.org/10.1016/S1365-1609(02)00052-7).
- Thompson CD and Thompson DE (1985) Real and apparent relaxation of driven piles. *Journal of Geotechnical Engineering* **111**(2): 225–237, [http://dx.doi.org/10.1061/\(ASCE\)0733-9410\(1985\)111:3A2\(225\)](http://dx.doi.org/10.1061/(ASCE)0733-9410(1985)111:3A2(225)).
- Tomlinson MJ (1980) *Foundation Design and Construction*, 4th edn. Pitman Publishing, London, UK.
- Tschebotarioff GP (1973) *Foundations, Retaining, and Earth Structures*, 2nd edn. McGraw-Hill, New York, NY, USA.
- Vijayvergiya VN and Focht Jr JA (1972) A new way to predict capacity of piles in clay. *Proceedings of the 4th Annual Offshore Technology Conference, Houston, TX, USA*.
- WYDOT (Wyoming Department of Transportation) (2012) *Standard Specifications for Road and Bridge Construction*, 2010 edn. WYDOT, Cheyenne, WY, USA.
- WYDOT (2014) *Construction Manual*. WYDOT, Cheyenne, WY, USA.

How can you contribute?

To discuss this paper, please submit up to 500 words to the editor at journals@ice.org.uk. Your contribution will be forwarded to the author(s) for a reply and, if considered appropriate by the editorial board, it will be published as a discussion in a future issue of the journal.