50 ksi Steel H-pile Capacity

FINAL REPORT

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16. Abstract
The objective of this study is to re-evaluate the adoption, with the objective of potentially extending the utilization of F_y = 50 ksi for the structural capacity of steel H-piles (AISC HP sections) for bridge foundations. Specific consideration is given to the current capacity equations, $P_n = 0.66A_sF_y$ and $P_r = 0.33A_sF_y$, with the objective of their confirmation or revision; potentially permitting fewer piles for a foundation and an associated cost savings. The impacts of any revisions, particularly upon foundation settlement, are evaluated and recommendations for the revision of DM-4 (as amended by SOL 483-14-04) are provided.

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Executive Summary

The objective of this study is to re-evaluate the adoption, with the objective of potentially extending the utilization of F_y = 50 ksi for the structural capacity of steel H-piles (AISC HP sections) for bridge foundations. Specific consideration is given to the current capacity equations, $P_n = 0.66A_sF_y$ and $P_r = 0.33A_sF_y$, with the objective of their confirmation or revision; potentially permitting fewer piles for a foundation and an associated cost savings. The impacts of any revisions, particularly upon foundation settlement, are evaluated and recommendations for the revision of DM-4 (as amended by SOL 483-14-04) are provided.

The commercially available program GRLWEAP was used to conduct 141 pile driving analyses in a parametric study and an additional 26 benchmark analyses using available field data. For each parametric analysis case considered, representing a pile section, pile length, shaft friction and 'target' capacity, a two-step analytical approach is used. Each analysis begins with trial hammer parameters (type, stroke and energy) and iterates upon these until the target capacity is attained at 240 blows/ft – a value defined as 'refusal'. The objective of each parametric analysis is to achieve the target capacity with the smallest (i.e. least energy) hammer (of those considered) while still providing at least a 0.5 foot working stroke range.

Results indicate that the AASHTO permitted pile capacity of $0.5A_sF_y$ is not technically achievable without the reduction in required over strength permitted using a PDA. Even using a PDA, this capacity may only be achievable for smaller pile sections. The SOL 483-14-04 permitted pile capacity of $0.5(0.66)A_sF_y$ in which F_y = 50 ksi is achievable in cases considered although driving stress in large HP14x117 piles approaches the limit of $0.9A_sF_y$. The theoretical increase in pile capacity realized by accounting for the increase of F_y from 36 to 50 ksi and the revisions to the PennDOT standard results in a theoretical increase in pile capacity of 131% by increasing F_y from 36 to 50 ksi; this increase is achievable for all cases considered. Driving piles to the maximum permitted driving stress of $0.90A_sF_y$ = 45 ksi, resulted in pile capacities ranging from $0.64A_sF_y$ to $0.76A_sF_y$ with smaller pile sections having marginally higher achievable capacities. All HP10x57 piles considered, for instance, could be driven to values exceeding $0.70A_sF_y$ without exceeding driving stress limits.

The benchmark comparisons with available CAPWAP analyses confirmed the method of WEAP analysis to obtain a reasonably accurate driving analysis. Requiring a WEAP analysis to approve the pile hammer and to establish the stroke range at refusal is affirmed as a practical driving analysis methodology to ensure the settlement limit is maintained and the pile is not overstressed during driving.

Analysis of bearing pile settlement indicated that piles having F_y = 50 ksi and design capacities up to the AASHTO-specified capacity of $0.50A_sF_y$ will not exhibit settlements greater than approximately 1 in. at service loads.

A representative cost analysis – normalized on the basis of 100,000 kips driven pile capacity and a number of fundamental assumptions concluded that increasing the design capacity of a pile results in a decrease in cost per driven pile capacity although due to the need for larger hammers and cranes, permitting design capacities greater than 16.5 ksi results in only marginal additional savings.

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1. Literature Review and Enumeration of H-Pile Design Criteria

1.1 Objective

The objective of this study is to re-evaluate the adoption, with the objective of potentially extending the utilization of F_y = 50 ksi for the structural capacity of steel H-piles (AISC HP sections) for bridge foundations. Specific consideration is given to the current capacity equations, $P_n = 0.66A_sF_y$ and $P_r = 0.33A_sF_y$, with the objective of their confirmation or revision; potentially permitting fewer piles for a foundation and an associated cost savings. The impact of any revisions, particularly upon foundation settlement will be evaluated.

1.2 Acronyms and Notation

The following acronyms and notation are used in this report.

WEAP	Wave Equation Analysis of Pile Driving
GRLWEAP	WEAP analysis program distributed by Pile Dynamics Inc.
CAPWAP	Case Pile Wave Analysis Program
PDA	Pile Driving Analyzer

All pile designations are given in standard US terminology omitting the leading “HP”; thus 12x74 indicates a pile having a nominal depth of 12 inches and a weight of 74 lbs/ft.

1.2.1 AASHTO LRFD and DM-4 Editions

Unless otherwise noted, this review refers to the most recent versions of AASHTO LRFD *Bridge Design Specifications* and PennDOT *Pub 15M Design Manual Part 4 (DM-4) Structures*. That is, the 2014 and 2012 versions, respectively. In order to document progression of these standards, some discussion refers to previous versions of AASHTO LRFD and DM-4 *only if these differ from the current versions*. This is indicated when appropriate.

1.3 Introduction

PennDOT *Design Manual* DM-4 (2012) §6.15.1 limits the specified yield strength of steel piles to $F_y \leq 36$ ksi. PennDOT Strike-Off Letter (SOL) 483-13-12 modifies this limit to $F_y \leq 50$ ksi and updates relevant sections of DM-4 and BC-757M and Publication 408 (section 1005) accordingly. SOL 483-13-12 notes “there is no apparent detrimental effect to bridges supported by H-pile foundations” resulting from this change. Subsequently, SOL 483-14-04 was issued to “clarif[y] the current pile design methodology for computing design capacity for H-piles and implements a new design methodology for computing the design capacity of steel pipe piles. This SOL revises DM-4 and replaces certain pages of SOL 483-13-12 which implemented the use of F_y = 50 ksi for steel H-piles for computing the design capacity.”

The primary motivation for making the change from F_y = 36 ksi to F_y = 50 ksi is that the ‘preferred material specification’ (AISC 2011) for H-pile shapes (designated HP) is ASTM A572 (2013) Grade 50 High Strength-Low Alloy Steel. Even if ASTM A36 (2012) steel were specified (availability of HP sections may be limited and therefore such specification would be at an increased cost), there is no upper limit on yield strength. Steel fabrication depends on scrap steel, which includes strength-enhancing elements that are not easily removed; therefore it is difficult for manufacturers to produce structural steel with a yield stress below 50 ksi.

An additional motivation is the expected cost savings, resulting from potentially using smaller sections, that may be realized by increasing the design capacity. The following section addresses potential risks associated with the increase in design capacity from F_y = 36 ksi to F_y = 50 ksi.

1.4 Issues Associated with Adoption of F_y = 50 ksi for H-Pile Design

The following issues have been identified as being potentially impacted by increasing the design strength of H-piles from F_y = 36 ksi to F_y = 50 ksi.

1.4.1 Structural Steel Capacity

Although the higher yield strength improves stability and yield checks, the higher yield strength may adversely affect ductility checks associated with non-compact shapes. As the yield strength increases from 36 to 50 ksi, the flange and web slenderness ratios defining compact and noncompact section limits – both a function of $\sqrt{E/F_y}$ – fall 18% (Table 1).

Table 1 AASHTO LRFD (2007, 2010 & 2014) and DM-4 slenderness limits.

design action	AASHTO/DM-4	plate element	slenderness limit		
			calculation	36 ksi	50 ksi
axial capacity	§6.9.4.2	flanges	$b_f/2t_f \leq 0.56\sqrt{E/F_y}$	15.9	13.5
		web	$(d-2k)/t_w \leq 1.49\sqrt{E/F_y}$	42.3	35.9
strong and weak axis flexure	§A6.3.2 (strong) §6.12.2.2.1 (weak)	compact flange	$b_f/2t_f \leq 0.38\sqrt{E/F_y}$	10.8	9.2
		noncompact flange	$b_f/2t_f \leq 0.83\sqrt{E/F_y}$	23.6	20.0

Of the nine standard HP shapes used by PennDOT (those reported in DM-4 Table 6.15.3.2P), 14x73 and 12x53 become ‘slender for axial load’ when F_y is increased from 36 ksi to 50 ksi. In terms of strong-axis flexure, seven of the nine sections are ‘non-compact for flexure’ at F_y = 50 ksi while only four shapes are non-compact at F_y = 36 ksi. Only 12x84 and 10x57 are compact for both axial and flexural loads for both F_y = 36 ksi and F_y = 50 ksi.

It is worth commenting here that the slender and non-compact shapes are ‘barely’ so. While the slender and non-compact designations trigger more robust calculation of capacity, the actual decrease in capacity over the compact shape calculations is at worst 3.6% for axial and 8% for flexural loads (both for the most slender 14x73 shape).

1.4.2 Pile Welds and Splices

Increased capacity and driving stresses affect the required capacity of pile section welds and splices. In late 2013 PennDOT Standard Drawing BC-757, showing H-pile splices, was revised to require full-penetration welds across the entire section. Previously, full-penetrations welds were required only for the flanges and double-sided splice plates used along the web. There is no known issue with the capacity or integrity of properly executed welds affected by the base material having F_y = 36 ksi or F_y = 50 ksi.

1.4.3 Effect of Corrosion

Corrosion resistance of steel piles is unaffected by strength. The use of higher strength piles may permit smaller pile sections to be used to resist the same load. When considering the effects of corrosion, it is typical to assume section loss of 1/16 in. from all surfaces. Thus a pile having a smaller section area has less ‘reserve’ capacity; that is: the 1/16in. reduction in plate thickness represents a proportionally greater section area for a smaller pile.

1.4.4 Allowable Net Settlement Limit

DM-4 §D10.5.2.2 limits net foundation settlement to 1 inch at service loads. For a bearing pile, this net settlement is the sum of two components: Δ_{tip}, tip displacement and Δ_s, pile shortening. Tip displacement is a function of the bearing rock strata modulus and is independent of the pile steel grade. Pile shortening (Δ_s) is given by the relationship:

$$\Delta_s = (Q_p + \zeta Q_s)L/EA \qquad (1)$$

Where Q_p and Q_s are the loads carried by the pile point and the skin friction, respectively; ζ represents the effect of the friction distribution pattern; L is the length of the pile; A the pile cross-section area; and E is the modulus of elasticity of the pile (E_{steel} = 29000 ksi). The load carried by the pile, P, is marginally less than the sum $Q_p + \zeta Q_s$; for simplicity, however, P is used in the following discussion.

While this relationship is independent of steel grade, the pile yield capacity is affected by steel grade. If pile capacity is increased from 36 ksi to 50 ksi, two implications for design may occur: a) the pile capacity, P, increases for the same pile section; or, b) the pile area, A may be reduced to carry the same value of P. Both cases result in an increase in Δ_s. Assuming a perfectly efficient design (i.e.: 100% utilization of cross section A to resist load P), the increase in Δ_s is equal to the ratio 50/36 = 1.39. Increasing Δ_s, while respecting the same net settlement limit, reduces the allowable tip displacement by a corresponding value. For case a) in which the pile capacity is increased, it must be assumed that Δ_{tip} will also increase at least in proportion to the applied load, P. In this case, the net settlement will increase 39% in going from F_y = 36 ksi to F_y = 50 ksi. For case b) the net settlement will increase, but remain less than 39% as the ratio Δ_{tip}/Δ_s approaches zero.

Related to this, DM-4 §C10.7.3.8.1 cites the findings of Kulhawy et al. (1983) in reporting that a pile driven in soil must displace on the order of 8% of its diameter in order to fully mobilize the tip capacity. Taking this guidance at face value and limiting settlement to 1 in. means that piles having a diameter greater than 12.5 in. cannot fully mobilize their tip capacity without exceeding settlement limits. This hypothetical calculation additionally neglects the effect of pile shortening. Curiously, Kulhawy et al. is not the source of the 8% value; Kulhawy et al. cites Vesic (1977) in this case. It is not entirely clear the basis for the '8% rule-of-thumb' since this value will be affected by soil type and pile length to a degree.

1.4.5 Tip Bearing Capacity

While the pile bearing capacity increases as F_y increases from 36 to 50 ksi, the soil into which the pile is driven and the strata on which it bears is clearly unchanged. Thus it is conceivable that a pile system whose limit state is governed by structural capacity at F_y = 36 ksi is governed by geotechnical bearing capacity at F_y = 50 ksi. This may be particularly the case for "weak rock" conditions.

1.4.6 Driving Stresses and the Need for a Driving Tip

Related to tip bearing capacity, it is equally conceivable that in order to efficiently drive a pile at F_y = 50 ksi, a driving tip is required which may not have been the case for F_y = 36 ksi. The use of the driving tip lowers the capacity of the pile (ϕ decrease from 0.6 to 0.5 (AASHTO) or from 0.45 to 0.35 (DM-4)), reducing the increased pile capacity that may be realized using the higher strength steel. DM-4 §D10.7.8.5 specifically requires driving tips for all point bearing and end bearing piles driven into bedrock, regardless of pile yield strength.

1.4.7 Friction Described as 'Shaft Percentage'

As will be discussed 'shaft percentage' – the portion of bearing pile capacity resisted by friction – is a necessary parameter used in design. The increase in pile strength has no effect on properties affecting friction.

1.5 Literature Review

1.5.1 Compilation of Pile Load Test and Wave Equation Information (Pub 15A)

Pub 15A (1989) is the only parametric study conducted by PennDOT comparing results of wave equation analyses with actual load test data. This study considered different pile hammers, pile sizes, pile lengths, hammer efficiencies and soil damping factors. This 1989 study is believed to be out of date as it used empirical data from actual load tests and maximum compressive stresses were unknown (PennDOT 2012). Pub 15A reports 82 tests. Sheets 1-52 are H-piles; only these are considered in the present study. The test parameters of the 52 H-pile tests reported in Pub 15A are summarized in Appendix A.

Based on Pub 15A, WEAP input parameters were determined (Table 2) and promulgated in DM-4 §D10.7.3.8.4. The DM-4-recommended values have been revised marginally since 1989 as shown in Table 2.

1.5.2 Pile Hammer Analysis Evaluation (PennDOT 2012)

This evaluation was initiated because PennDOT's standard procedure for performing WEAP (using GRLWEAP) did not consistently provide results in agreement with CAPWAP results based on actual PDA input for point and end bearing piles. The objective of the study was to identify parameters within GRLWEAP that may be modified to improve this agreement. The study considered 41 PDA data obtained from 12 projects. The data included four pile shapes (10x57 (n = 13), 12x74 (n = 13), 12x84 (n = 3) and 14x117 (n = 12)) and five hammer types (Pileco D19-42, ICE I-46, Berminghammer B-21, ICE I-30 and ICE I-19). Table 2 summarizes the GRLWEAP parameters recommended based on Pub 15A, those prescribed by DM-4 (2012) §D10.7.3.8.4, and those recommended by PennDOT (2012).

Table 2 GRLWEAP input variables.

GRLWEAP parameter	Pub 15A (1989)	DM-4 (2012)	PennDOT (2012)
shaft quake	0.10 in.	0.10 in.	0.10 in.
toe quake	0.10 in.	0.05 in.	0.05 in.
shaft damping	0.05 sec./ft	0.05 sec./ft	0.05 sec./ft
toe damping	0.20 sec./ft	0.10 sec./ft	0.10 sec./ft
shaft percentage	10%	10%	30%[a]
hammer pressure	100%	100%	100%
[a] recommendation to increase shaft percentage to 20% for one year and evaluate further increase to 30% thereafter.			

The 2012 PennDOT study concluded that although quake and damping parameters affect GRLWEAP output, they do so in an inconsistent manner and only values determined *post priori* improve predictive results of WEAP; thus no change to the presently prescribed values was recommended. Similarly, reducing hammer pressure to 80% had negligible effects on GRLWEAP output.

Increasing the shaft percentage from 10% to 30% was found to reduce the GRLWEAP overestimation of CAPWAP-determined maximum compressive stress. The degree of improvement was greater for larger pile sizes although this reflects the greater overestimation of stress for the larger piles in any event. Table 3 summarises representative results presented by the study based on pile size and hammer type. The data reported in Table 3 was obtained from "Executive Table 1" in PennDOT (2012) and has been updated to correct apparent reporting errors in the original report (revised data provided by PennDOT 3.12.15).

PennDOT (2012) recommend increasing the shaft friction percentage to 30% in GRLWEAP analyses in order to better replicate results observed in the field. Increasing this parameter in GRLWEAP will lead to larger pile hammer stroke values being approved for use and therefore more efficient pile driving operations. The study continues to recommend limiting pile stresses to 32.4 ksi ($0.9F_y$) and 40 ksi ($0.8F_y$) for 36 ksi and 50 ksi piles, respectively. Subsequently, SOL 483-13-12 revised the limiting pile stress to $0.9F_y$.

Table 3 Maximum compressive stress determined in field and by predicted by GRLWEAP for five example cases (PennDOT 2012).

example	**TP-2**	**B3**[1]	**P5**	**TP-12**	**3054**
pile size	12x74	10x57	12x74	14x117	14x117
hammer	ICE I-19	Pileco D19-42	Pileco D19-42	ICE I-30	Pileco D19-42
in situ maximum compressive stress	26.0 ksi	26.2 ksi[1]	24.0 ksi	29.7 ksi[1]	24.9 ksi
GRLWEAP with 10% shaft friction	30.7 ksi	41.6 ksi[1]	31.7 ksi	49.8 ksi[1]	33.8 ksi
GRLWEAP overestimation of *in situ*	18%	59%	32%	68%	36%
GRLWEAP with 30% shaft friction	27.3 ksi	33.4 ksi[1]	26.6 ksi	39.5 ksi[1]	27.4 ksi
GRLWEAP overestimation of *in situ*	10%	27%	11%	33%	10%

[1] revised per PennDOT, 3.12.15.

1.5.3 I-95/I-276 Interchange Pile Testing Program (PTC 2011)

The Pennsylvania Turnpike Commission (PTC) reports a pile testing program undertaken as part of the I-95/I-276 Interchange Project. Of the eight piles tested, six were H-piles: three 12x74 and three 14x89 (Table 4). All piles were reported to be ASTM A-572 Grade 50 steel having a nominal yield strength, F_y = 50 ksi. All piles were driven to absolute refusal, defined as 20 blows per inch in soft or decomposed rock, or dense or hard soil strata.

The objectives of the study were to a) evaluate the capacity of pile drivability into a thick saprolite layer; b) determine the ultimate geotechnical capacity of the piles specifically to determine whether geotechnical or structural capacity controls the design; c) identify if varying saprolite thickness affects the ultimate capacity; and d) monitor ground surface vibration associated with pile driving. Objective b) is the primary concern relative to the present study.

PennDOT and the PTC independently own and designed the interchange structures. PennDOT-owned structures were designed according to DM-4 §6.15.1 which at the time limited the yield strength for steel to be used in structural pile design to 36 ksi. PTC's Design Guidelines allow for the use of 50 ksi. This situation permits a direct comparison of piles designed using the different provisions (Table 4). Although the report focuses on reduced section capacity (accounting for eventual 1/16 in. section loss due to corrosion), only full section capacity, defined as $0.35F_yA$, is presented in Table 4.

PDA monitoring and subsequent CAPWAP analyses were conducted at the end of initial driving (EOID) and at the beginning of restrike (BOR). In all but pile TP-1, BOR capacities are greater than EOID capacities; only the greater value is reported here. In all cases, the PDA-determined driving stresses were below the allowable driving stress of $0.8F_y$ = 40 ksi (Table 4). It is noted that subsequent revision by SOL 483-13-12 increases this limit to $0.9F_y$ = 45 ksi. Both Case Method (Goble et al. 1980) and CAPWAP analyses were performed to determine the pile capacities. The resulting factored (ϕ = 0.65) geotechnical pile capacities all exceed the structural capacities; indicating that regardless of pile yield strength, the structural capacity controls design in these cases.

All piles penetrated a first saprolite layer (SPT > 40 blows for 12 in.) and embedded into a second denser layer (SPT > 50 blows for 6 in.). Piles TP-3 and TP-3B were founded on rock while the others were driven to refusal (20 blows per inch) within the second saprolite layer. A function of the saprolite embedment, with the exception of TP-1, skin friction percentage was predicted (using CAPWAP) to exceed 30% in all cases supporting the primary conclusion of PennDOT (2012).

Table 4 Pile test details and results from PTC Pile Test Program (PTC 2011).

Test	**TP-1**	**TP-1B**	**TP-2**	**TP-2A**	**TP-3**	**TP-3B**
pile size	12x74	14x89	12x74	14x89	12x74	14x89
pile embedment	36.0 ft	33.9 ft	45.0 ft	44.5 ft	57.0 ft	55.0 ft
design structural capacity: F_y = 36 ksi	275 kips	329 kips	275 kips	329 kips	275 kips	329 kips
design structural capacity: F_y = 50 ksi	382 kips	457 kips	382 kips	457 kips	382 kips	457 kips
max. driving stress (from PDA)	38.2 ksi	33.4 ksi	36.1 ksi	37.0 ksi	38.3 ksi	32.7 ksi
factored geotechnical capacity (Case)	477 kips	498 kips	473 kips	551 kips	506 kips	499 kips
factored geotechnical capacity (CAPWAP)	449 kips	472 kips	468 kips	530 kips	474 kips	468 kips

1.5.4 PennDOT (2013b)

PennDOT (2013b) reports a limited evaluation of the geotechnical capacity of H-piles in weak or soft rock – identified as weak shale – in order to assess the implications of the use of 50 ksi (rather than 36 ksi) H-piles. PennDOT considered four data sets from Pub 15A: sheets 18, 23, 27 and 30; these are summarized along with PennDOT's findings in Table 5. For each case, a WEAP analysis (using GRLWEAP) based on 'current methodology' (2013) was followed by a static analysis using actual soil profiles. For F_y = 36 ksi, the piles did not appear to overstress the rock strata nor exceed the service settlement limit of 1 inch in any case (this is expected), although the analyses revealed some issues with the data reported in Pub 15A.

Table 5 Summary of H-pile capacity in weak shale (PennDOT 2013b).

Pub 15A sheet no.	**18**	**23**	**27**	**30**
pile size	12x74	10x57	12x74	12x74
hammer type	ICE-640	LB 520	LB 520	ICE-640
hammer rated energy	40000 ft-lbs	26300 ft-lbs	26300 ft-lbs	40000 ft-lbs
pile embedment	61.0 ft	31.5 ft	33.5 ft	35.5 ft
ultimate capacity from static load test (Pub 15A)	524 kips 24.0 ksi	340 kips 20.2 ksi	290 kips 13.3 ksi	480 kips 22.0 ksi
total settlement at ultimate capacity (Pub 15A)	1.02 in.	0.67 in.	0.45 in.	0.60 in.
WEAP capacity	575 kips 26.4 ksi	376 kips 22.4 ksi	340 kips 15.6 ksi	550 kips 25.2 ksi
static analysis capacity	655 kips 30.0 ksi	376 kips 22.4 ksi	298 kips 13.7 ksi	548 kips 25.1 ksi

The tests reported in sheets 18, 23 and 27 were stopped at displacements of 1 in. or less. This is inadequate to ensure that the bedrock is fully engaged. For example, calibrating the pile tip displacement (0.60 in.) with the static load test results, yield a static modulus of weak shale of only E_s = 1884 ksf, well below the typical minimum value for weak shale of 3000 ksf. This result, like those reported on sheets 23 and 27, indicates a test result dominated by pile friction rather than bearing capacity. PennDOT reports that the 61 ft pile embedment for sheet 18 was too long to adequately assess tip capacity. Additionally, sheets 23 and 27 report smaller hammers were used than would be used for production piles.

The test reported based on sheet 30 therefore provided the basis for most of PennDOT's conclusions. In this case the modulus of the weak shale was computed to be E_s = 10,490 ksf, an appropriately sized hammer was used and the bearing test was carried out to 1.45 in. (resulting in an ultimate capacity of 570 kips (26 ksi).

PennDOT (2013b) concludes that the net settlement limit of 1.0 in. (DM-4 §D10.5.2.2) does not to allow for adequate development of the shaft friction or pile tip bearing resistance to develop the required ultimate geotechnical resistance. This was observed for 36 ksi piles in weak shale. When considering 50 ksi piles, either the pile capacity will increase for the same pile size or the pile size will decrease for the same applied load. In either case, the pile shortening component of settlement will increase. [In theory, the pile shortening component will increase by the ratio of capacity increase for the same pile size or by the inverse of the pile area decrease for the same applied load.] PennDOT recommends revising §D10.5.2.2 to increase the pile foundation settlement limits from 1.0 to 1.5 in. [Based on the limited scope of the study, this recommendation should only be applied to 'weak rock' conditions. It is unlikely to be an issue for stronger rock.] PennDOT additionally cites the '8% rule-of-thumb' as further support for increasing the settlement limit to 1.5 in. Finally, PennDOT (2013b) recommends more refined reporting of geotechnical data for weak rock in order to more accurately assess settlement values.

1.6 H-Pile Design Criteria

1.6.1 Calculation of Factored Axial and Flexural Resistances of Pile Sections – DM-4 Table 6.15.3.2P-1

The following documents the calculations of axial, strong-axis and weak-axis flexural capacities, P_r, M_{rx} and M_{ry}, respectively. All HP section data is that reported in AISC *Steel Construction Manual* (2011). In this discussion gross section properties are assumed. PennDOT additionally considers pile capacity for deteriorated piles having '1/16 inch section loss' (DM-4 Table 6.15.3.2P-2). In the latter case, reduced section properties are used in the design equations. Such reduction due to corrosion is discussed in Section 1.6.7

1.6.2 AASHTO LRFD/DM-4 §6.15.2 – Material Resistance Factors

The material resistance factors (ϕ) used for the calculation of steel H-pile capacity are provided in §6.5.4.2 and are summarized in Table 6.

Different ϕ-values are used for axial resistance than for axial when combined with flexure. This is because the lower values are applied only to sections of the pile "likely to experience damage"; these will not be in regions (along the pile length) where combined loads are critical. The material resistance factors for piles are based on recommendations of Davisson et al. (1983) with modifications to reflect current design philosophy (AASHTO LRFD).

Table 6 Material resistance factors for steel H-piles.

<table>
<tr><th></th><th>AASHTO (2007, 2010 & 2014)</th><th>DM-4 (2007 & 2012)</th><th>SOL 483-13-12 (2013)</th><th>SOL 483-14-04 (2014) Table 6.15.2-1</th></tr>
<tr><td rowspan="2">For axial resistance of piles in compression and subject to damage due to severe driving conditions where use of a pile tip is necessary</td><td rowspan="2">$\phi_c = 0.50$ with $P_n = A_sF_y$</td><td>$\phi_c = 0.35$ with $P_n = A_sF_y$</td><td colspan="2">$\phi_c = 0.50$ with $P_n = 0.66A_sF_y$</td></tr>
<tr><td colspan="3">DM-4 §D10.7.8.5 requires driving tips for all point bearing and end bearing piles driven into bedrock</td></tr>
<tr><td>For axial resistance of piles in compression under good driving conditions where use of a pile tip is not necessary</td><td>$\phi_c = 0.60$ with $P_n = A_sF_y$</td><td>$\phi_c = 0.45$ with $P_n = A_sF_y$</td><td colspan="2">$\phi_c = 0.60$ with $P_n = 0.66A_sF_y$</td></tr>
<tr><td>For combined axial and flexural resistance of undamaged piles – axial resistance</td><td>$\phi_c = 0.70$ with $P_n = A_sF_y$</td><td>$\phi_c = 0.60$ with $P_n = A_sF_y$</td><td colspan="2">$\phi_c = 0.70$ with $P_n = 0.66A_sF_y$</td></tr>
<tr><td>For combined axial and flexural resistance of undamaged piles – flexural resistance</td><td>$\phi_f = 1.00$</td><td>$\phi_f = 0.85$</td><td colspan="2">$\phi_c = 1.00$ with
$M_{nx} = Z_xF_y$ and $M_{ny} = 1.5S_yF_y$ (compact)
$M_{nx} = S_xF_y$ and $M_{ny} = 1.5S_yF_y$ (noncompact)</td></tr>
<tr><td>For resistance during pile driving[1]</td><td>$\phi = 1.00$</td><td colspan="3">$\phi = 1.00$</td></tr>
<tr><td>For piles bearing on soluble bedrock (limestone, etc.), to provide pile group redundancy and limit the design stress to 9 ksi</td><td>not considered in §6.5.4.2</td><td>$\phi_c = 0.25$ with $P_n = A_sF_y$</td><td>$\phi_c = 0.273$ with $P_n = 0.66A_sF_y$</td><td>$\phi_c = 0.273$ with $P_n = 0.66A_sF_y$</td></tr>
<tr><td>§10.7.8 Drivability Analysis</td><td>0.90</td><td>1.00 (36 ksi)
0.80 (50 ksi)</td><td>$\phi_c = 0.90$</td><td>$\phi_c = 0.90$</td></tr>
<tr><td colspan="5">[1] it is assumed that §10.7.8 supersedes this AASHTO case. Since no mention of this entry is made in DM-4, it is assumed that the AASHTO-prescribed value applies.</td></tr>
</table>

1.6.2.1 PennDOT application of resistance factors

The lower material resistance factors used by PennDOT are reported to be calibrated with load factors to result in the same "factor of safety previously used by the Department" (DM-4 C6.15.2). SOL 483-14-04 modifies C6.15.3P as follows: "The factored compressive resistance for H-piles is established based on historically achievable pile capacities from dynamic testing results and the Compilation of Pile Load Test and Wave Equation Information, Publication 15A, an installed nominal compressive stress of 25.38 ksi, and engineering judgment."

The result of applying the lower material resistance factors (DM-4) and/or the 0.66 factor applied to axial strength (SOLs) is that the reliability associated with PennDOT practice is unknown although it is considerably greater than that used in AASHTO.

Anecdotally, PennDOT appears to prescribe and use $\lambda = 1.0$ in §6.9.4.1 (see section 1.6.3, below) for [noncomposite] H-piles. However, it is noted that DM-4 does *not* modify AASHTO §10.7.3.13.1 which prescribes $\lambda = 0$ for composite piles, although these are not the focus of the present study. SOL 483-14-04, Table 6.15.2-1 clearly prescribes the AASHTO (2010) resistance factors to be applied to be a nominal axial capacity of $0.66A_sF_y$. This results in effective reduction factors – relative to the theoretical capacity of a fully braced section, A_sF_y – lower than the previously prescribed DM-4 values (see Table 7, column 5) and 66% of those prescribed by AASHTO (column 6). Without having the explicit statistical variation associated with both material resistance and applied loads, the resulting reliability may not be calculated. An illustrative example of this calculation based on simple assumptions is provided in Appendix B. The concern with the 'dual factor' approach taken by PennDOT is that it masks statistically anticipated behaviour, promulgates mechanically incorrect design equations and results in misleading measures of reliability. Nonetheless, the Research Team notes that the current PennDOT practice remains conservative and represents no safety concerns.

Table 7 Effective material resistance factors for axial load.

1	2	3	4	5	6
AASHTO (2014)	**DM-4 (2012)**	**DM-4 / AASHTO**	**SOL 483-14-04**	**SOL 483-14-04 / DM-4**	**SOL 483-14-04 / AASHTO**
$0.50A_sF_y$	$0.35A_sF_y$	0.70	$0.50(0.66)A_sF_y = 0.33A_sF_y$	0.94	0.66
$0.60A_sF_y$	$0.45A_sF_y$	0.75	$0.60(0.66)A_sF_y = 0.40A_sF_y$	0.89	0.66
$1.0A_sF_y$	$0.85A_sF_y$	0.85	$1.0(0.66)A_sF_y = 0.66A_sF_y$	0.77	0.66

The 0.66 factor in is applied to axial stress only; no reduction is taken on the flexural stress component for combined loading (see Table 6).

For piles bearing on soluble bedrock, the material resistance factor is calibrated to limit the bearing stress to 9 ksi. Firstly, limiting the steel stress to 9 ksi results in a bearing stress less than this value since friction is neglected. Secondly, limiting the pile steel stress to 9 ksi neglects the greater bearing area that results when a pile tip is used. Regardless of these arguments, the Research Team recommends a clearer statement in DM-4 to address this condition:

- *for piles bearing on soluble bedrock the calculated net bearing stress shall not exceed 9 ksi.*

1.6.3 AASHTO LRFD/DM-4 §6.15.3 – Compressive Resistance of Piles

AASHTO LRFD/DM-4 §6.15.3 refers to §6.9.2.1 for calculation of P_n as follows:

1. Section geometry is checked against compact plate buckling criteria given by Eq. 6.9.4.2-1:

		36 ksi	**50 ksi**
flanges	$b_f/2t_f \le 0.56\sqrt{E/F_y}$	15.9	13.5
web	$(d-2k)/t_w \le 1.49\sqrt{E/F_y}$	42.3	35.9

For HP sections web slenderness is not a concern. The most slender gross section web is an HP 14x73 having a web slenderness of 22.2 while the most slender reduced section web is an HP 12x53 having a slenderness of 30.7.

2. Calculate Axial Capacity per §6.9.4.1.

Although the notation has changed from 2007 to 2010, the calculation of axial capacity is effectively the same: "[the equations given] are equivalent to the equations given in AISC (2010) [and 2005 and 2007] for computing the nominal compressive resistance. The equations are written in a different format…" (AASHTO LRFD §C6.9.4.1.1). However, the change in terminology from the λ factor to the capacity ratio P_e/P_o *appears* to affect the calculated results for fully-supported piles. The ratio P_o/P_e can be shown to be mathematically equivalent to λ; therefore, the interpretation of §6.9.4.1 should remain consistent. That is, $P_o/P_e = \lambda$. The change from a factor 0.66 to 0.658 has a negligible effect (0.3%).

flange slenderness (see step 1)	***AASHTO LRFD (2007)***	***AASHTO LRFD (2010 & 2014)***
	$\lambda = \left(\frac{K\ell}{r\pi}\right)^2 \frac{F_y}{E}$	$P_o = QF_yA_s$ and $P_e = \frac{\pi^2 E}{(K\ell/r)^2} A_s$ $P_o/P_e = \lambda$
Compact	$P_n = 0.66^{\lambda} F_y A_s$	$P_n = 0.658^{Po/Pe} F_y A_s$
Slender	member is designed according to AISC as follows (C6.9.4.1): For $0.56\sqrt{E/F_y} \le b_f/2t_f \le 1.03\sqrt{E/F_y}$ $Q_s = 1.415 - 0.74(b_f/2t_f)\sqrt{F_y/E}$ $P_n = Q_s F_y A_s$	AASHTO LRFD §6.9.4.2.2: For $0.56\sqrt{E/F_y} \le b_f/2t_f \le 1.03\sqrt{E/F_y}$ $Q_s = 1.415 - 0.74(b_f/2t_f)\sqrt{F_y/E}$ $P_n = Q_s F_y A_s$

PennDOT SOL 483-14-04 Table 6.15.2-1 appears to set $P_n = 0.66F_yA_s$ and $Q_s = 1.0$ regardless of slenderness. This is a conservative approach since calculated values of Q_s for conventionally used pile sections (Tables 8 and 9) do not fall below 0.82 (HP 12x53 having F_y = 50 ksi and 1/16 section loss) in any case.

1.6.4 AASHTO LRFD/DM-4 §6.12.2.2.1 – Weak Axis Flexural Resistance of H-Piles

Weak axis flexural resistance, M_{ny} is calculated as follows:

	flange slenderness			$\boldsymbol{M_{ny}}$
		36 ksi	**50 ksi**	
compact flange	$b_f/2t_f \le 0.38\sqrt{E/F_y}$	10.8	9.2	$M_{ny} = M_{py} = 1.5S_yF_y$ (per C6.12.2.2.1)
noncompact flange	$b_f/2t_f \le 0.83\sqrt{E/F_y}$	23.6	20.0	from AASHTO LRFD Eq. 6.12.2.2.1-2: $M_n = Z_yF_y\left[1-\left(1-\frac{S_y}{Z_y}\right)\left(\frac{b_f/2t_f - 0.38\sqrt{E/F_y}}{0.45\sqrt{E/F_y}}\right)\right]$

There is some inconsistency in the AASHTO Equations presented. Where $M_{py} = 1.5S_yF_y$ for HP sections, the equation for non-compact flanges *implies* a calculated reduction (the term in square brackets) to M_{py} calculated as $M_{py} = Z_yF_y$. It is the contention of the research team that, for the sake of continuity, the noncompact equation ***for HP sections*** should be *interpreted* as follows:

$$M_n = 1.5S_yF_y\left[1-\left(1-\frac{S_y}{Z_y}\right)\left(\frac{b_f/2t_f - 0.38\sqrt{E/F_y}}{0.45\sqrt{E/F_y}}\right)\right] \qquad (2)$$

Without this amendment, the effect of increasing F_y on the computed value of M_{ny} may be greater than the increase in F_y itself since $Z_y/S_y > 1.5$ for all HP sections. Changing the Z_yF_y term in Equation 6.12.2.2.1-2 to M_{py} would result in this equation being internally consistent. PennDOT SOL 483-14-04 Table 6.15.2-1 appears to set $M_{ny} = 1.5S_yF_y$ regardless of slenderness for fully braced piles. This may be non-conservative for piles having non-compact flanges.

1.6.5 AASHTO LRFD/DM-4 §A6.3.2 – Strong Axis Flexural Resistance of H-Piles

AASHTO LRFD/DM-4 §6.12.2.2.1 refers to §6.10 for calculation of strong axis flexural resistance, M_{nx}. Calculations for M_n are found in §A6.3.2.

	flange slenderness			**M_{nx}**
		36 ksi	**50 ksi**	
compact flange	$b_f/2t_f \le 0.38\sqrt{E/F_y}$	10.8	9.2	$M_{nx} = M_{px} = Z_x F_y$
noncompact flange	$b_f/2t_f \le 0.83\sqrt{E/F_y}$	23.6	20.0	from AASHTO LRFD Eq. A6.3.2-2: $M_n = Z_x F_y\left[1-\left(1-\frac{0.7S_x}{Z_x}\right)\left(\frac{b_f/2t_f - 0.38\sqrt{E/F_y}}{0.45\sqrt{E/F_y}}\right)\right]$

PennDOT SOL 483-14-04 Table 6.15.2-1 appears to set $M_{rx} = Z_x F_y$ for compact and $S_x F_y$ for non-compact sections. This may be non-conservative for piles having non-compact flanges since the reduction factor in the brackets of AASHTO Eq. A6.3.2-2 is often less than the typical ratio S_x/Z_x. This effect is more significant for those sections having reduced section dimensions.

1.6.6 Reduced Cross Sections Due to Assumed Effect of Corrosion

When vertical H-pile foundations are designed using COMP624P or LPILE, the capacity values given in Tables 6.15.3.2P-1 and 6.15.3.2P-2 may be used. Table 6.15.3.2P-2 provides values for piles assumed to have 1/16 in. section loss resulting from corrosion. The section loss is interpreted as 1/16 in. from all exposed steel and therefore affects geometric parameters as indicated below:

- dimensions that are reduced 1/8 in.: d, t_w, b_f, t_f
- dimensions that are reduced 1/16 in.: k, k_1
- dimension having no change: T

Geometric properties are then calculated using the reduced dimensions. In this study the geometric properties of the gross cross section are those reported in the AISC *Steel Construction Manual* Table 1-4. Calculation of reduced properties neglects the area of the fillets at the web-flange interface.

The loss of section area affects the slenderness of the sections as indicated in Table 8. It has been previously noted that webs are compact for all HP sections regardless of 1/16 in. section loss.

Table 8 Impact 1/16 in. section loss on flange slenderness.

HP	**gross section properties**					**reduced section properties**				
	$b_f/2t_f$	axial		flexure		$b_f/2t_f$	axial		flexure	
		36 ksi	50 ksi	36 ksi	50 ksi		36 ksi	50 ksi	36 ksi	50 ksi
14x117	9.25	compact	compact	compact	***noncompact***	10.85	compact	compact	all are **noncompact**	all are **noncompact**
14x102	10.49	compact	compact	compact	***noncompact***	12.64	compact	compact		
14x89	11.95	compact	compact	***noncompact***	***noncompact***	14.87	compact	***slender***		
14x73	14.44	compact	***slender***	***noncompact***	***noncompact***	19.03	***slender***	***slender***		
12x84	8.97	compact	compact	compact	compact	10.87	compact	compact		
12x74	10.01	compact	compact	compact	***noncompact***	12.46	compact	compact		
12x63	11.77	compact	compact	***noncompact***	***noncompact***	15.38	compact	***slender***		
12x53	13.84	compact	***slender***	***noncompact***	***noncompact***	19.23	***slender***	***slender***		
10x57	9.05	compact	compact	compact	compact	11.48	compact	compact		

Increasing F_y from 36 to 50 ksi results in two HP sections becoming slender for axial load and three additional HP sections becoming noncompact for flexure. When considering reduced sections, two HP sections are slender for F_y = 36 ksi and two additional HP sections at F_y = 50 ksi. Regardless of strength, all sections are noncompact for flexure when the reduced section is considered.

1.6.7 Calculation of Standard HP Section Capacities

Table 9 shows the impact of increasing F_y from 36 ksi to 50 ksi and the impact of the 1/16 in. section reduction on the AASHTO-prescribed (2014) nominal strengths P_n, M_{nx} and M_{rny} of the nine standard pile shapes provided in DM-4 Table 6.15.3.2P-1. The proportional impact is the same, regardless of consistent load case (driving condition, etc.) used (i.e., regardless of ϕ).

HP sections are inherently stocky (compact); thus, although a few sections go from being compact to noncompact for flexure or become slender for axial loads (see Table 8), the effects are marginal. For sections that are compact for both F_y = 36 and 50 ksi, the ratio of capacities between members having these strengths is 50/36 = 1.39. For noncompact or slender shapes, this ratio falls. The lowest value of this ratio for the gross sections considered is 1.27 for an HP 14x73 which is also the least compact of the members considered having $b_f/2t_f$ = 14.44. Similarly, for reduced sections a value of 1.21 is found for an HP 12x53 having $b_f/2t_f$ = 19.23.

Although two sections are classified as slender for axial load, HP 14x73 and HP 12x53, the ‘degree of slenderness’ has little effect on the axial capacity. The value of the reduction factor accounting for slender compression elements, Q_s for these gross sections having F_y = 50 ksi is 0.97 and 0.99, respectively.

Capacity reductions associated with the 1/16 in. section reduction range from to 0.55 to 0.84.

1.6.8 Comparisons of AASHTO and DM-4 Calculated Capacities

As is evident throughout the foregoing discussion and as described in Tables 6 and 7, there are differences between AAASHTO practice and those of PennDOT, as described by DM-4 and subsequent SOLs. These involve the selection of material resistance factor, ϕ, and the use of the 0.66 factor when calculating nominal axial capacity. As a result, the differences in capacity between AASHTO and PennDOT practice, as measured by the ratio of DM-4 to AASHTO- prescribed capacities varies from load case to load case.

Tables 10a and 10b report H-pile capacities (F_y = 50 ksi) reported in revised (per SOL 483-14-04) DM-4 Tables 6.15.3.2P-1 and 6.15.3.2P-2, respectively. These values are compared to those calculated using AASHTO (2014) provisions (i.e., nominal capacities given in Table 9). Table 10 therefore represents current (August 2014) practice. Tables 11a and 11b repeat the comparison for capacities reported in DM-4 (2010) using F_y = 36 ksi. Geometric section properties used in all calculations are those reported in Tables 6.15.3.2P-1 and 6.15.3.2P-2. There is no AASHTO-comparable case to the DM-4 “soluble rock” case which is based on an allowable bearing stress of 9 ksi in any case.

Table 9 Impact of increasing F_y from 36 ksi to 50 ksi and prescribed 1/16 in. section reduction on AASHTO-prescribed (2014) nominal axial and flexural capacities of HP sections.

HP	F_y	$P_n = A_s F_y$			M_{nx} **(see 1.6.5)**			M_{ny} **(see 1.6.4)**		
	ksi	gross kips	reduced kips	P_{nr}/P_{ng}	gross kip-ft	reduced kip-ft	M_{nxr}/M_{nxg}	gross kip-ft	reduced kip-ft	M_{nxr}/M_{nxg}
14x117	36	1238	1034	0.84	582	485	0.83	268	222	0.83
	50	1720	1436	0.84	806	636	0.79	371	292	0.79
	50/36	1.39	1.39		1.38	1.31		1.38	1.32	
14x102	36	1084	878	0.81	507	388	0.76	231	178	0.77
	50	1505	1219	0.81	671	501	0.75	307	231	0.75
	50/36	1.39	1.39		1.32	1.29		1.33	1.30	
14x89	36	940	738	0.79	423	302	0.71	193	139	0.72
	50	1305	983	0.75	550	384	0.70	252	178	0.70
	50/36	1.39	1.33		1.30	1.27		1.31	1.28	
14x73	36	770	524	0.68	317	201	0.63	145	93	0.64
	50	1039	657	0.63	404	244	0.60	186	114	0.61
	50/36	1.35	1.25		1.27	1.21		1.28	1.23	
12x84	36	886	713	0.81	360	288	0.80	156	124	0.80
	50	1230	991	0.81	500	377	0.75	216	163	0.76
	50/36	1.39	1.39		1.39	1.31		1.38	1.31	
12x74	36	785	615	0.78	315	235	0.75	137	102	0.74
	50	1090	854	0.78	424	304	0.72	185	132	0.72
	50/36	1.39	1.39		1.35	1.29		1.35	1.29	
12x63	36	662	492	0.74	257	170	0.66	111	74	0.67
	50	920	644	0.70	335	215	0.64	145	94	0.65
	50/36	1.39	1.31		1.30	1.26		1.31	1.27	
12x53	36	558	356	0.64	202	117	0.58	87	51	0.59
	50	767	446	0.58	259	142	0.55	112	63	0.56
	50/36	1.37	1.25		1.28	1.21		1.29	1.24	
10x57	36	605	462	0.76	200	149	0.75	89	66	0.75
	50	840	642	0.76	277	195	0.70	123	87	0.70
	50/36	1.39	1.39		1.39	1.31		1.38	1.32	

Table 10a Comparison of DM-4 (as revised by SOL 483-14-04) and AASHTO factored capacities for gross section properties and F_y = 50 ksi.

	axial resistance, P_{rSTR}							**combined axial and flexural resistance**								
	severe driving conditions			**good driving conditions**			**soluble rock**	**P_r (kips)**			**M_{rx} (kip-ft)**			**M_{ry} (kip-ft)**		
	DM-4	AASHTO	DM-4/AASHTO	DM-4	AASHTO	DM-4/AASHTO	DM-4	DM-4	AASHTO	DM-4/AASHTO	DM-4	AASHTO	DM-4/AASHTO	DM-4	AASHTO	DM-4/AASHTO
P_n or M_n (compact)	$0.66A_sF_y$	A_sF_y		$0.66A_sF_y$	A_sF_y		$0.66A_sF_y$	$0.66A_sF_y$	A_sF_y		Z_xF_y	Z_xF_y		$1.5S_yF_y$	$1.5S_yF_y$	
P_n or M_n (noncompact)		$Q_sA_sF_y$			$Q_sA_sF_y$				$Q_sA_sF_y$		S_xF_y	Eq. A6.3.2-2			Eq. 6.12.2.2.1-2	
ϕ	0.50	0.50		0.60	0.60		0.273	0.70	0.70		1.00	1.00		1.00	1.00	
14x117	568	860	0.66	681	1032	0.66	310	795	1204	0.66	711	806	0.88	372	380	0.98
14x102	497	752	0.66	596	902	0.66	271	695	1053	0.66	619	671	0.92	321	315	**1.02**
14x89	431	652	0.66	517	782	0.66	235	603	913	0.66	538	550	0.98	277	257	**1.08**
14x73	353	520	0.68	424	624	0.68	193	494	728	0.68	439	404	**1.09**	224	189	**1.19**
12x84	406	615	0.66	487	738	0.66	222	568	861	0.66	492	500	0.98	216	216	1.00
12x74	360	545	0.66	432	654	0.66	196	504	763	0.66	385	424	0.91	190	189	**1.01**
12x63	304	460	0.66	364	552	0.66	166	425	644	0.66	324	335	0.97	158	148	**1.07**
12x53	256	384	0.67	307	461	0.67	140	358	538	0.67	272	259	**1.05**	132	114	**1.16**
10x57	277	420	0.66	333	504	0.66	151	388	588	0.66	273	277	0.99	123	123	1.00

Table 10b Comparison of DM-4 (as revised by SOL 483-14-04) and AASHTO factored capacities for reduced section properties and F_y = 50 ksi.

	axial resistance, P_{rSTR}							**combined axial and flexural resistance**								
	severe driving conditions			**good driving conditions**			**soluble rock**	**P_r (kips)**			**M_{rx} (kip-ft)**			**M_{ry} (kip-ft)**		
	DM-4	AASHTO	DM-4/AASHTO	DM-4	AASHTO	DM-4/AASHTO	DM-4	DM-4	AASHTO	DM-4/AASHTO	DM-4	AASHTO	DM-4/AASHTO	DM-4	AASHTO	DM-4/AASHTO
P_n or M_n (compact)	$0.66A_sF_y$	A_sF_y		$0.66A_sF_y$	A_sF_y		$0.66A_sF_y$	$0.66A_sF_y$	A_sF_y		Z_xF_y	Z_xF_y		$1.5S_yF_y$	$1.5S_yF_y$	
P_n or M_n (noncompact)		$Q_sA_sF_y$			$Q_sA_sF_y$				$Q_sA_sF_y$		S_xF_y	Eq. A6.3.2-2			Eq. 6.12.2.2.1-2	
ϕ	0.50	0.50		0.60	0.60		0.273	0.70	0.70		1.00	1.00		1.00	1.00	
14x117	474	718	0.66	569	862	0.66	259	664	1005	0.66	603	636	0.95	309	298	**1.04**
14x102	402	610	0.66	483	732	0.66	220	563	854	0.66	512	501	**1.02**	260	235	**1.11**
14x89	338	492	0.69	406	590	0.69	185	474	689	0.69	430	384	**1.12**	217	180	**1.21**
14x73	261	328	0.80	313	394	0.80	143	366	459	0.80	332	244	**1.36**	166	116	**1.43**
12x84	327	496	0.66	392	595	0.66	178	458	694	0.66	357	377	0.95	173	167	**1.04**
12x74	282	427	0.66	338	512	0.66	154	395	598	0.66	308	304	**1.01**	148	135	**1.10**
12x63	225	322	0.70	271	386	0.70	123	316	451	0.70	246	215	**1.14**	117	95	**1.23**
12x53	178	223	0.80	214	268	0.80	97	250	312	0.80	195	142	**1.37**	92	63	**1.46**
10x57	212	321	0.66	254	385	0.66	116	297	449	0.66	189	195	0.97	94	88	**1.07**

Table 11a Comparison of DM-4 (2010) and AASHTO factored capacities for gross section properties and F_y = 36 ksi.

	axial resistance, P_{rSTR}				**combined axial and flexural resistance**								
	severe driving conditions			**soluble rock**	**P_r (kips)**			**M_{rx} (kip-ft)**			**M_{ry} (kip-ft)**		
	DM-4	AASHTO	DM-4/AASHTO	DM-4	DM-4	AASHTO	DM-4/AASHTO	DM-4	AASHTO	DM-4/AASHTO	DM-4	AASHTO	DM-4/AASHTO
P_n or M_n (compact)	A_sF_y	A_sF_y		A_sF_y	A_sF_y	A_sF_y		Z_xF_y	Z_xF_y		$1.5S_yF_y$	$1.5S_yF_y$	
P_n or M_n (noncompact)		$Q_sA_SF_y$				$Q_sA_SF_y$		S_xF_y	Eq. A6.3.2-2			Eq. 6.12.2.2.1-2	
ϕ	0.35	0.50		0.25	0.60	0.70		0.85	1.00		0.85	1.00	
14x117	434	619	0.70	310	743	867	0.86	495	582	0.85	228	268	0.85
14x102	378	540	0.70	270	648	756	0.86	431	507	0.85	197	231	0.85
14x89	329	470	0.70	235	564	658	0.86	334	423	0.79	169	197	0.86
14x73	270	385	0.70	193	462	539	0.86	273	317	0.86	137	148	0.94
12x84	310	443	0.70	221	531	620	0.86	306	360	0.85	132	156	0.85
12x74	275	392	0.70	196	471	549	0.86	268	315	0.85	116	137	0.85
12x63	232	331	0.70	166	397	463	0.86	202	257	0.79	97	113	0.86
12x53	195	279	0.70	140	335	391	0.86	170	202	0.84	81	89	0.91
10x57	212	302	0.70	151	363	423	0.86	170	200	0.85	75	89	0.84
10x42	156	223	0.70	112	268	312	0.86	111	140	0.79	54	63	0.86

Table 11a Comparison of DM-4 (2010) and AASHTO factored capacities for reduced section properties and F_y = 36 ksi.

	axial resistance, P_{rSTR}				**combined axial and flexural resistance**								
	severe driving conditions			**soluble rock**	**P_r (kips)**			**M_{rx} (kip-ft)**			**M_{ry} (kip-ft)**		
	DM-4	AASHTO	DM-4/AASHTO	DM-4	DM-4	AASHTO	DM-4/AASHTO	DM-4	AASHTO	DM-4/AASHTO	DM-4	AASHTO	DM-4/AASHTO
P_n or M_n (compact)	A_sF_y	A_sF_y		A_sF_y	A_sF_y	A_sF_y		Z_xF_y	Z_xF_y		$1.5S_yF_y$	$1.5S_yF_y$	
P_n or M_n (noncompact)		$Q_sA_SF_y$				$Q_sA_SF_y$		S_xF_y	Eq. A6.3.2-2			Eq. 6.12.2.2.1-2	
ϕ	0.35	0.50		0.25	0.60	0.70		0.85	1.00		0.85	1.00	
14x117	362	517	0.70	259	620	724	0.86	369	485	0.76	189	226	0.84
14x102	307	439	0.70	219	527	615	0.86	313	388	0.81	159	181	0.88
14x89	258	369	0.70	185	443	517	0.86	263	302	0.87	133	141	0.94
14x73	199	262	0.76	142	342	367	0.93	203	201	**1.01**	101	94	**1.07**
12x84	250	357	0.70	178	428	500	0.86	219	288	0.76	106	127	0.84
12x74	215	306	0.70	153	368	428	0.86	188	235	0.80	90	103	0.87
12x63	172	246	0.70	123	295	344	0.86	151	170	0.89	72	75	0.96
12x53	136	178	0.76	97	234	249	0.94	119	117	**1.02**	56	52	**1.08**
10x57	162	231	0.70	116	277	323	0.86	116	149	0.78	57	67	0.85
10x42	107	150	0.71	77	183	210	0.87	77	82	0.94	37	37	1.00

A number of observations are made when comparing DM-4 and AASHTO prescribed capacities.

1.6.8.1 Current (DM-4 as modified by SOL 483-14-04) practice (Table 10)

1. DM-4 does not differentiate between compact and slender axial capacity. As a result the ratio of DM-4 prescribed capacity to that of AASHTO (2014) (DM-4/AASHTO in Table 10) varies from 0.66 (for all compact sections) to as high as 0.80 for corroded slender sections HP14x73 and HP12x53. Thus a uniform additional 'factor of safety' or 'reliability' over and above AASHTO-prescribed capacities is not achieved.
2. The simplification of using Z_xF_y and S_xF_y for capacities of compact and noncompact sections when determining strong axis flexure (M_{rx}) result in non-conservative capacities for noncompact members (noted in red text in Table 10). This results from the ratio S_x/Z_x being greater than the reduction coefficient given by AASHTO LRFD Eq. A6.3.2-2 (see 1.6.5). This effect is particularly pronounced when reduced sections (Table 10b) are considered. For the most slender members (HP14x73 and HP12x53) the overestimation of strong axis flexural capacity made by DM-4 is as high as 37%.
3. The simplification of using $1.5S_yF_y$ regardless of slenderness when determining weak axis flexure (M_{ry}) result in non-conservative capacities for noncompact members. This results from the ratio Z_y/S_y being greater than 1.5 for all HP shapes. This effect is particularly pronounced when reduced sections (Table 10b) are considered. For the most slender members (HP14x73 and HP12x53) the overestimation of weak axis flexural capacity made by DM-4 is as high as 46%.

1.6.8.2 Previous DM-4 (2010) practice (Table 11)

1. The ratios of DM-4 prescribed capacity to that of AASHTO (2007, 2010 or 2014) (DM-4/AASHTO in Table 11) essentially follow the ratio of prescribed material resistance factors (i.e., 0.35/0.50 = 0.70 and 0.60/0.70 = 0.86). Because DM-4 does not differentiate between compact and slender axial capacity, small variations to this ratio result.
2. The simplifications used by DM-4 in calculating flexural capacity and the resulting differences with AASHTO are the same as noted above (items 2 and 3 in Section 1.6.8.1). However the application of $\phi = 0.85$ prescribed by DM-4 results in reduced DM-4/AASHTO capacity ratios. Only those sections that are noncompact for F_y = 36 ksi (HP14x73 and HP12x53) have ratios greater than unity and only for the reduced sections (Table 11b).

1.6.8.3 Comparison of pre- and post-SOL 483-14-04 DM-4 capacities

Table 9 provides the ratios of capacities attributed to increasing F_y from 36 to 50 ksi. As described in section 1.6.7, this value is 1.39 for compact sections and falls to as low as 1.21 for noncompact sections having reduced sections.

Table 12 summarizes the normalized pile capacity prescribed by DM-4. The ratios presented are the capacities post-SOL 483-14-04 (i.e., current capacity) to the pre-SOL capacity (i.e., 2010). In order to normalize for increasing F_y from 36 to 50 ksi, the pre-SOL capacity is multiplied by the 50/36 ratio reported in Table 9. In this manner both the increased yield strength and changes to section slenderness (Table 8) are accounted for.

As illustrated in Table 12, the normalized axial capacity has fallen (i.e., DM-4/SOL > 1.0) with the adoption of SOL 483-14-04. For combined axial and flexural loads, the current (SOL 483-14-04) normalized axial capacity is approximately 77% that of DM-4 (2010). This is an indication that the SOL provisions are not permitting the increase in pile yield strength to be effectively utilized to resist axial load. The normalized moment capacity, on the other hand has increased substantially (i.e., DM-4/SOL < 1.0): by as much as 35% for the noncompact sections having reduced sections.

In either case, the combination of changes made as part of SOL 483-14-04 has resulted in changes to the cross section capacity utilization, that is, the structural efficiency, of the HP sections used.

Table 12a Comparison of pre-and post-SOL 483-14-04 gross section capacities normalized for steel yield strength.

	axial resistance, P_{rSTR} severe driving conditions				combined axial and flexural resistance											
					P_r (kips)				M_{rx} (kip-ft)				M_{ry} (kipft)			
	DM-4 (2010)	SOL 483-14-04	50/36	DM-4/SOL	DM-4 (2010)	SOL 483-14-04	50/36	DM-4/SOL	DM-4 (2010)	SOL 483-14-04	50/36	DM-4/SOL	DM-4 (2010)	SOL 483-14-04	50/36	DM-4/SOL
F_y (ksi)	36	50			36	50			36	50			36	50		
Table ref:	11	10	9		11	10	9		11	10	9		11	10	9	
14x117	434	568	1.39	1.06[1]	743	795	1.39	1.30	495	711	1.38	0.96	228	372	1.38	0.85
14x102	378	497	1.39	1.06	648	695	1.39	1.30	431	619	1.32	0.92	197	321	1.33	0.82
14x89	329	431	1.39	1.06	564	603	1.39	1.30	334	538	1.30	0.81	169	277	1.31	0.80
14x73	270	353	1.35	1.03	462	494	1.35	1.26	273	439	1.27	0.79	137	224	1.28	0.78
12x84	310	406	1.39	1.06	531	568	1.39	1.30	306	492	1.39	0.86	132	216	1.38	0.84
12x74	275	360	1.39	1.06	471	504	1.39	1.30	268	385	1.35	0.94	116	190	1.35	0.82
12x63	232	304	1.39	1.06	397	425	1.39	1.30	202	324	1.30	0.81	97	158	1.31	0.80
12x53	195	256	1.37	1.04	335	358	1.37	1.28	170	272	1.28	0.80	81	132	1.29	0.79
10x57	212	277	1.39	1.06	363	388	1.39	1.30	170	273	1.39	0.87	75	123	1.38	0.84

[1] example calculation: (434x1.39)/568 = 1.06

Table 12b Comparison of pre-and post-SOL 483-14-04 reduced section capacities normalized for steel yield strength.

	axial resistance, P_{rSTR} severe driving conditions				combined axial and flexural resistance											
					P_r (kips)				M_{rx} (kip-ft)				M_{ry} (kipft)			
	DM-4 (2010)	SOL 483-14-04	50/36	DM-4/SOL	DM-4 (2010)	SOL 483-14-04	50/36	DM-4/SOL	DM-4 (2010)	SOL 483-14-04	50/36	DM-4/SOL	DM-4 (2010)	SOL 483-14-04	50/36	DM-4/SOL
F_y (ksi)	36	50			36	50			36	50			36	50		
Table ref:	11	10	9		11	10	9		11	10	9		11	10	9	
14x117	362	474	1.39	1.06	620	664	1.39	1.30	369	603	1.31	0.80	189	309	1.32	0.81
14x102	307	402	1.39	1.06	527	563	1.39	1.30	313	512	1.29	0.79	159	260	1.30	0.80
14x89	258	338	1.33	1.02	443	474	1.33	1.24	263	430	1.27	0.78	133	217	1.28	0.78
14x73	199	261	1.25	0.95	342	366	1.25	1.17	203	332	1.21	0.74	101	166	1.23	0.75
12x84	250	327	1.39	1.06	428	458	1.39	1.30	219	357	1.31	0.80	106	173	1.31	0.80
12x74	215	282	1.39	1.06	368	395	1.39	1.29	188	308	1.29	0.79	90	148	1.29	0.78
12x63	172	225	1.31	1.00	295	316	1.31	1.22	151	246	1.26	0.77	72	117	1.27	0.78
12x53	136	178	1.25	0.96	234	250	1.25	1.17	119	195	1.21	0.74	56	92	1.24	0.75
10x57	162	212	1.39	1.06	277	297	1.39	1.30	116	189	1.31	0.80	57	94	1.32	0.80

1.7 Geotechnical Resistance of Driven Piles

The geotechnical resistance of a driven pile is the ultimate capacity of the supporting soil and/or rock layers for carrying the design load. For an end-bearing pile, the ultimate geotechnical capacity is the sum of the tip bearing resistance and skin friction of the pile. Adequate geotechnical data is required to ensure an accurate estimation of the geotechnical resistance of a pile. According to AASHTO LRFD §10.4.2, an extensive subsurface exploration of the soil deposits and/or rock formations is needed to analyze foundation stability and settlement. A subsurface study should contain information about the: present geotechnical formation(s), location and thickness of soil and rock units, engineering properties of soil and rock units (such as unit weight, shear strength and compressibility), groundwater conditions, ground surface topography, and local considerations (such as liquefiable, expansive or dispersive soil deposits, underground voids from solutions weathering or mining activity or slope instability). Static analysis methods are empirical and semi empirical and are used to determine the ultimate axial capacity of a single pile and pile group. Static analysis methods use the soil strength and compressibility properties to determine pile capacity and performance from which the number of piles and pile lengths may be determined.

There are different static analysis methods introduced for determining the nominal bearing resistance of piles. PennDOT DM-4 §10.7.3.8.6 prescribes the use of methods such as: α – method, β – method, λ – method, Nordlund/ Thurman method, and the SPT or CPT methods. Hannigan et al. (2006) classifies the use of different static analysis methods for cohesionless and cohesive soils.

1.7.1 AASHTO LRFD/DM-4 §10.7.3.8.6 – Pile Bearing Resistance

The factored pile bearing resistance, $R_{R,}$ is the sum of the pile tip and side resistances:

$$R_R = \phi R_n = \phi_{stat} R_p + \phi_{stat} R_s \qquad (3)$$

Where R_p = pile tip resistance: $R_p = q_p A_p$

R_s = pile side resistance: $R_s = q_s A_s$

In which q_p and q_s are the unit tip and side resistances, respectively; and A_p and A_s are the area of the pile tip and the surface area of the pile side, respectively. ϕ_{stat} is the resistance factor for the bearing resistance of a single pile specified in DM-4 Table 10.5.5.2.3-1 (repeated here as Table 13).

Table 13 Resistance factors for single driven piles, ϕ_{stat} (DM-4 Table 10.5.5.2.3-1).

clay or mixed soils	α-method	$\phi_{stat} = 0.70$
	β-method	$\phi_{stat} = 0.50$
	λ-method (Vijayvergiya & Focht 1972)	$\phi_{stat} = 0.55$
sandy soils	Nordlund/Thurman Method	$\phi_{stat} = 0.50$
	SPT-method	$\phi_{stat} = 0.45$
	CPT-method	$\phi_{stat} = 0.55$

1.7.2 Static analysis methods for determining nominal bearing resistance of piles in cohesionless soils

The nominal bearing capacity of piles in cohesionless soils depends on the relative density of the soil. The driving forces increase the relative density of the soil around the pile-soil interface and, as a result, the bearing capacity of the pile increases. The type of pile has an impact on the relative density of soil: piles with large displacement (precast concrete piles) increase the relative density of cohesionless material more than low displacement steel H-piles (Hannigan et al. 2006; i.e., *FHWA NHI-05-042*). The different static methods promulgated by DM-4 for cohesionless soils are summarized below and in Table 14.

β – Method – This method is used to calculate the bearing resistance of piles in cohesionless, cohesive, and layered soils. This is an effective-stress based method which is developed to model the long term drained shear strength conditions.

λ – Method **-** This method estimates the undrained skin friction considering the length of a pile incorporating both the effective overburden stress and the undrained shear strength of the soil. This method relates the unit skin resistance to short term passive earth pressure.

Nordlund Method – This method is based on field observations and considers the shape of pile taper and its soil displacement in calculation the shaft resistance. The method also accounts for the differences in soil-pile coefficient of friction for different pile materials. The method is based on the results of several load test programs in cohesionless soils. The piles used to develop the method's design curves had widths in the range of 10-20 inches. The Nordlund method tends to overpredict pile capacity for piles with widths greater than 24 inches.

Meyerhof Method – This method is based on standard penetration test (SPT) data

Nottingham and Schmertmann Method – This method is based on the Cone Penetration Test (CPT) data and it is used for both cohesive and cohesionless soils.

Table 14 Summary of static analysis methods for piles in cohesionless soils.

Method	AASHTO (2010) NHI-05-042 (2006)	Tip resistance	Side resistance	Parameters
β – Method	10.7.3.8.6c 9.7.1.3	$q_p = N_p\sigma'_p$	$q_s = \beta\sigma'_v$	N_p = tip bearing capacity coefficient σ'_p = effective overburden pressure at the pile tip β = an empirical coefficient σ'_v = vertical effective stress
λ - method	10.7.3.8.6d	-	$q_s = \lambda(\sigma'_v + 2S_u)$	S_u = undrained shear strength $(\sigma'_v + 2S_u)$ = passive lateral earth pressure λ = an empirical coefficient
Nordlund/ Thurman method	10.7.3.8.6f 9.7.1.1c	$q_p = \alpha_t N'_q\sigma'_v \leq q_L$	$q_s = K_\delta C_F \sigma'_v \dfrac{\sin(\delta+\omega)}{\cos\omega}$	α_t = coefficient N'_q = bearing capacity factor q_L = limiting unit tip resistance K_δ = coefficient of lateral earth pressure at mid-point of soil layer C_F = correction factor for K_δ when $\delta \neq \phi_f$ ω = angle of pile taper from vertical δ = friction angle between pile and soil
Meyerhof Method (ksf units)	10.7.3.8.6g 9.7.1.1a	$q_p = \dfrac{0.8(N1_{60})D_b}{D} \leq q_L$	$q_s = \dfrac{\overline{N1}_{60}}{50}$	$N1_{60}$ = representative SPT blow count near pile tip D = pile width or diameter D_b = depth of penetration in bearing strata q_L = limiting tip resistance taken as $8N1_{60}$ for sands and $6N1_{60}$ for nonplastic silt (ksf)
Nottingham Method	10.7.3.8.6g 9.7.1.7b	$q_p = 0.5(q_{c1}+q_{c2})$	$R_s = K_{s,c}\left[\sum_{i=1}^{N1}\left(\dfrac{L_i}{8D_i}\right)f_{si}a_{si}h_i + \sum_{i=1}^{N2} f_{si}a_{si}h_i\right]$	q_{c1} = average static cone tip resistance over a distance yD below the pile tip q_{c2} = average static cone tip resistance over a distance $8D$ above the pile tip $K_{s,c}$ = correction factor for clays and sands L_i = depth to middle of length interval i D_i = pile width or diameter f_{si} = unit sleeve friction resistance from CPT a_{si} = pile perimeter h_i = length interval N_1 = intervals between ground surface $8D$ below ground surface. N_2 = intervals between $8D$ below ground surface and pile tip

1.7.3 Static analysis methods for determining nominal bearing resistance of piles in cohesive soils

The bearing resistance of a pile in cohesive soil is the sum of the tip resistance and skin friction (or shaft resistance). However the shaft resistance of piles driven in cohesive soils is frequently as much as 80 to 90% of the total capacity. The pile design load should be supported by soil resistance developed only in soil layers that contribute to long term load support. The soil resistance from soils subjected to scour, or from soil layers about soft compressible soils should not be considered (Hannigan et al. 2006). The different static methods promulgated by DM-4 for cohesive soils are summarized below and in Table 15.

α – Method - This is a total stress method used to calculate the ultimate capacity of undrained cohesive soil using the shear strength of the soil. This method assumes that the shaft resistance is independent of the effective overburden pressure.

β – Method – This method is used to calculate the bearing resistance of piles in cohesionless, cohesive, and layered soils. This is an effective-stress based method which is developed to model the long term drained shear strength conditions.

Nottingham and Schmertmann Method – This method is based on the Cone Penetration Test (CPT) data and it is used for both cohesive and cohesionless soils.

Table 15 Summary of static analysis methods for piles in cohesive soils.

Method	**AASHTO LRFD (2010) NHI-05-042 (2006)**	**Pile tip resistance**	**Pile side resistance**	**Parameters**
α – method	10.7.3.8.6b 9.7.1.2a	$q_p = 9S_u$	$q_s = \alpha S_u$	α = adhesion factor applied to S_u S_u = undrained shear strength
β – Method	10.7.3.8.6c 9.7.1.3a	$q_p = N_p\sigma'_p$	$q_s = \beta\sigma'_v$	N_p = tip bearing capacity coefficient σ'_p = effective overburden pressure at the pile tip β = an empirical coefficient σ'_v = vertical effective stress
Nottingham-Method	10.7.3.8.6g 9.7.1.7b	$q_p = 0.5(q_{c1}+q_{c2})$	$R_s = \alpha' f_{si} A_s$	q_{c1} = average static cone tip resistance over a distance yD below the pile tip q_{c2} = average static cone tip resistance over a distance $8D$ above the pile tip α' = Ratio of pile shaft resistance to cone sleeve friction A_s = pile-soil surface area over f_{si} depth interval

1.7.4 Nominal bearing capacity of piles on rock

Pile foundations on rock are designed to support large loads. The determination of load capacity of driven piles on rock should be made on the basis of driving observations, local experience and load tests. Except for soft weathered rock, the structural capacity of the pile will generally be lower than the capacity of rock to support loads for toe bearing piles on rock of fair to excellent quality; therefore the allowable design stress for the pile material will govern the pile capacity in many cases (Hannigan et al. 2006).

According to AASHTO LRFD/DM-4 §10.7.3.2.2, piles supported on soft rock should be designed similar to piles supported on soils and the bearing resistance should be estimated as described in §10.7.3.8 or by geotechnical analysis to determine the limiting resistance as either the structural resistance or the geotechnical resistance (see above). Revision to DM-4 C10.7.3.2.2 defines “soft or weak rock” as rock having uniaxial compressive strength less the 500tsf (6.95ksi).

1.8 Dynamic Analysis of Piles (WEAP)

Wave equation analysis of pile driving (WEAP) is a numerical method for assessing the driving behaviour of driven piles. WEAP predicts the pile capacity versus blow count relationship, the so called bearing graph, and pile driving stress. A WEAP model represents the pile driving hammer and its accessories (ram, cap, and cap block) and the pile, as a series of lumped masses and springs in a one-dimensional analysis. The soil response for each pile segment is modelled as being viscoelastic-plastic. The conceptual one-dimensional WEAP model is shown in Figure 1.

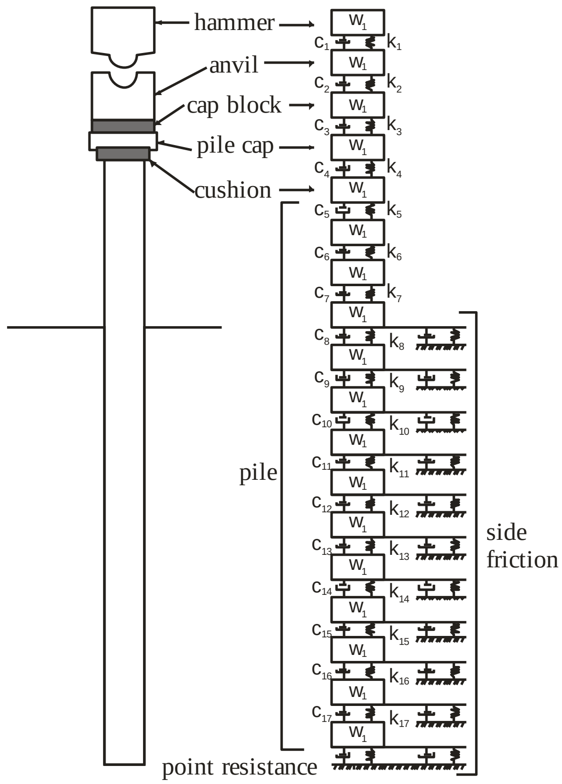


Figure 1 Conceptual representation of WEAP analysis.

WEAP analyses are often used to conduct a drivability analysis to select the parameters for safe pile installation, including recommendations on cushion stiffness, hammer stroke and other driving system parameters that optimize blow counts and pile stresses during pile driving. For a single hammer blow, a WEAP analysis may (Lowery 1993):

1. predict the driving stresses induced in the pile
2. determine the resulting motion of the pile
3. determine the resistance to penetration afforded by the soil

With this information, the following engineering questions may be addressed (Lowery 1993):

1. can the given hammer drive the pile to the required depth?
2. what rate of penetration will be provided; i.e.: how long will it take to drive the pile?
3. what is the maximum penetration that may be achieved?
4. will excessive stresses occur in the pile during driving?

The primary objective of a dynamic analysis, relevant to this study, is to determine whether a pile is overstressed when driven to a capacity equal to the factored axial resistance increased by a resistance factor. For this purpose, the present study will used GRLWEAP software (PDI 2010) and focused on the computation of the bearing graphs, and to a lesser extent the drivability analysis, in addressing these issues. The WEAP analysis requires as input information regarding the hammer, pile, and soil column.

The steps involved in a WEAP analysis are the following (FHWA 2003):

1. determine the pile length.
2. determine the distribution and magnitude of side friction.
3. determine damping factors: Case or Smith skin damping, skin quake for soils and rocks.
4. hammer selection, helmet and cushion properties.
5. permissible driving stress.
6. compute ultimate capacity and maximum driving stress.

In an analysis, the major engineering effort lies in steps (2) and (3). These steps require the incorporation and interpretation of geotechnical information. Not only is it necessary to calculate the static resistance and its distribution; but also additional dynamic soil resistance parameters, damping and quake, both at the shaft and toe must be estimated.

For step (2), the piles considered in this study were be primarily end-bearing but some shaft skin friction was be present during driving. The skin friction can be quickly computed using the FHWA computer program DRIVEN 1.2, a 32 bit Windows program, which supports the application for H piles. DRIVEN provides the pile bearing capacity with depth in terms of the contributions from skin friction and end bearing. It also facilitates the creation of an input file for GRLWEAP for drivability analysis by computing the friction loss/gain factor. DRIVEN program follows the methods and equations presented by Nordlund (1963), Thurman (1964), Meyerhof (1976), Cheney and Chassie (1982), Tomlinson (1986), and Hannigan, et.al. (1997).

Illinois DOT (2009) suggests the use of the IDOT static method with pile-type correction factors as being more accurate than DRIVEN, and suggests that the WSDOT formula be used to replace the FHWA formula. For the objectives of the present study, the issue of accuracy is not an issue since our interest is the likelihood of pile overstress for a given capacity requirement. FHWA NHI-04-041 (Hartle et al. 2003) provides a detailed example showing how to go from computed static capacity to drivability analysis.

For step (3), in the application for bearing graphs, average shaft damping values are used. GRLWEAP-suggested quake and damping values, along with those used by PennDOT (2012) and those recommended by DM-4 §10.7.3.8.4bP are listed in Tables 16 and 17. The present study will address these values as input parameters and identify the sensitivity of resulting pile stresses to each.

Table 16 Recommended quake values for impact driven piles.

	soil type	**pile type or size**	**quake (in.)**		
			GRLWEAP	**PennDOT (2012)**	**DM-4 (2010)**
shaft quake	all soil types	all pile types	0.10	0.10	0.10
toe quake	all soil types, soft rock	non-displacement piles	0.10	-	0.10
	very dense or hard soils	displacement piles having diameter or width D	D/120	-	D/120
	soils that are not dense or hard		D/60	-	D/60
	hard rock	all pile types	0.04	0.05	0.05

Table 17 Recommended damping values for impact driven piles.

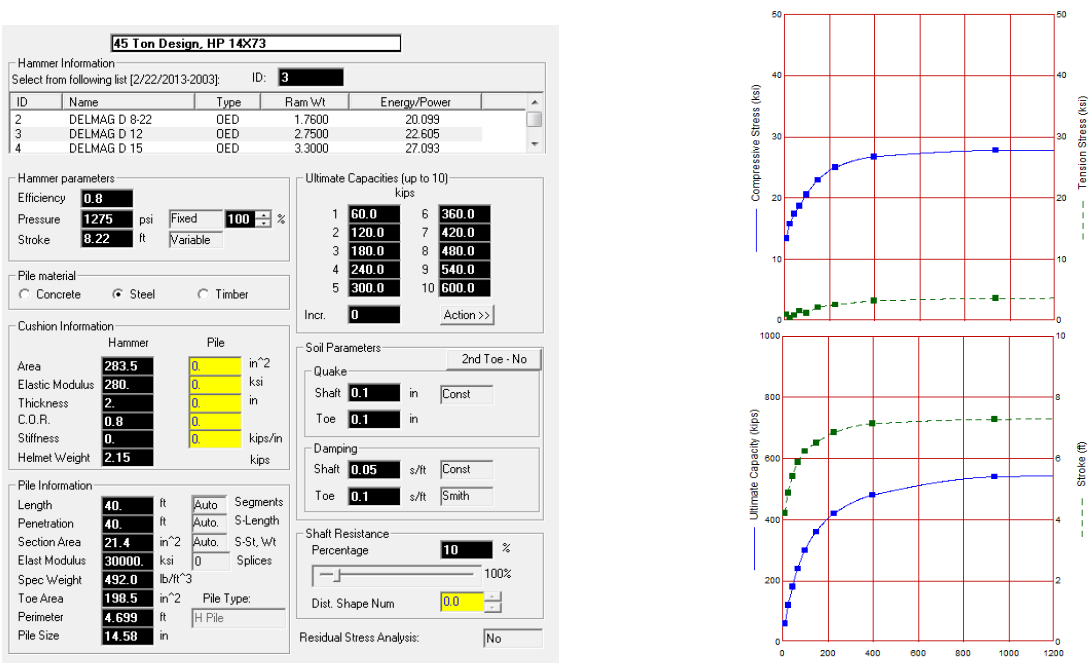
	soil type	damping factor (s/ft)		
		GRLWEAP	PennDOT (2012)	DM-4 (2010)
shaft damping	non-cohesive soils	0.05	0.05	0.05
	cohesive soils	0.20	-	0.20
	rock (end or point bearing piles)	-	-	0.05
toe damping	cohesive or non-cohesive soils	0.15	-	0.15
	rock (end or point bearing piles)	0.15[1]	0.10	0.10

[1] implied by term "all soil types"

In the present study, typically-available/used hammers were used (step 4). These are listed in Section 3.2.2 (Table 19). The allowable driving stress for the H-pile has been addressed in the preceding sections.

1.8.1 Sample GRLWEAP Analysis

Figure 2a shows a sample input to generate bearing graph for a H14X73 pile. Figure 2b shows the resulting bearing graph (capacity vs. blow count) and maximum pile stresses predicted.



a) input user interface b) bearing and pile stress graphs

Figure 2 GRLWEAP example input and output.

1.9 Summary of Literature Review

This study addresses the potential impacts of increasing the design strength of H-piles from F_y = 36 ksi to F_y = 50 ksi. As a baseline for comparison, AASHTO (2007, 2010 and 2014) design capacities for fully-braced H-piles subject to severe driving conditions where use of a pile tip is necessary are given as: $\phi_c P_n = \phi_c A_g F_y = 0.50 A_g(50) = A_g(25\ ksi)$. By contrast, prior to 2014, PennDOT (DM-4 2007. 2012) H-pile capacity was based on F_y = 36 ksi and given as: $\phi_c P_n = \phi_c A_g F_y = 0.35 A_g(36) = A_g(12.6\ ksi)$ despite H-pile being essentially only available with F_y = 50 ksi. When permitting the use of F_y = 50 ksi in design, PennDOT SOL 483-14-04 prescribes: $\phi_c P_n = \phi_c\, 0.66 A_g F_y = 0.50(0.66) A_g(50) = A_g(16.5\ ksi)$.

Most available literature identifies the steel pile capacity, rather than the geotechnical capacity, as being the dominant limit state for bearing pile design. This conclusion may reflect the degree of conservatism inherent in geotechnical capacity calculations. In this case, the following issues have been identified as being potentially impacted by increasing the design strength of steel H-piles from F_y = 36 ksi to F_y = 50 ksi.

Structural steel ductility and local stability (section slenderness) – Although the higher yield strength improves stability and yield checks, the higher yield strength may adversely affect ductility checks associated with non-compact shapes. As the yield strength increases from 36 to 50 ksi, the flange and web slenderness ratios defining compact and noncompact section limits fall 18% (Table 1).

Effect of corrosion – The use of higher strength piles may permit smaller pile sections to be used to resist the same load. A pile having a smaller initial section area has less 'reserve' capacity to account for section loss due to corrosion.

Allowable net settlement limit – A more heavily loaded pile and/or a smaller pile carrying the same load will exhibit greater deformation effectively reducing the 'allowable' pile settlement for the same gross settlement limit.

Tip bearing capacity – A more heavily loaded pile and/or a smaller pile carrying the same load exerts greater bearing stresses on both the bearing strata and at the pile cap.

Driving sresses and the need for a driving tip – Related to increased tip bearing capacity, it is conceivable that in order to efficiently drive a pile at F_y = 50 ksi, a driving tip is required which may not have been the case for F_y = 36 ksi. The required (DM4 §10.7.8.5P) use of the driving tip lowers the capacity of the end/point bearing H-piles (ϕ decrease from 0.6 to 0.5) due to "severe driving conditions", reducing the increased pile capacity that may be realized using the higher strength steel.

Friction described as 'shaft percentage' – An increase in pile strength has no effect on properties affecting friction. Thus the shaft percentage may be different for higher driving stresses. Prior to the 2012 PennDOT study, 10% shaft friction was typically used for the WEAP analysis of end and point bearing piles.

Methodology of performing WEAP analysis to determine stroke range and refusal – An increase in pile capacity has the effect of increasing the pile hammer size or requiring more energy by increasing the stroke to drive the pile to the increased ultimate driving stress at refusal.

2. Survey of Regional Practice

A survey of state DOT H-pile design criteria was carried out at two levels. An initial, high-level review of state DOT design manuals/guides (equivalent to *DM*-4) was conducted to assess the proliferation of the use of F_y = 50 ksi in determining H-pile structural capacity as distinct from allowable driving stresses and geotechnical capacity. Following this, a more detailed survey of practice was distributed to five agencies: Pennsylvania, Ohio, New York and Delaware DOTs and the Pennsylvania Turnpike Commission (PTC). The four DOTs responded; the PTC did not.

2.1 National Review

A review of state DOT Bridge Design Manuals (BDM), or equivalent, was conducted. This was conducted using internet resources. Some states do not have their entire BDM available on line, a few provide it only to registered users and, in a number of cases, it is unclear if the available version is the most recent. Nonetheless, a survey of H-pile provisions was made based on available information. Information was found for 49 state DOTs (no data could be found for MD) as summarized in Table 18 and presented in detail in Appendix C. In Appendix C, two queries are made: 1) is steel having F_y = 50 ksi permitted in determining the structural capacity of H-piles (independent of geotechnical considerations)? and 2) regardless of design requirements, is steel having F_y = 50 ksi specified for H-piles? In interpreting either query, silence on an issue is taken to imply deferral to AASHTO LRFD (which permits the use of H-piles designed for F_y = 50 ksi). With regard to the second query, reference to ASTM A572 material is interpreted as addressing HP sections since A572 is typically only applicable to HP shapes (see AISC *Steel Construction Handbook* Table 2-4). In many cases steel having F_y = 50 ksi is specified although design may be limited to F_y = 36 ksi. Because of the significant variation in the way BDMs are presented, the data assembled in Appendix C should be considered representative and not necessarily complete. A summary of observations includes:

a) 31/49 states for which information was found permit, either explicitly or by implication, H-piles having $F_{y\,=}$ 50 ksi for determining structural capacity.
b) Some states specify F_y = 36 ksi be used to determine structural capacity but permit F_y = 50 ksi under the conditions noted; for example:
   - GA permits F_y = 50 ksi “if called for by the BFI.”
   - NH permits F_y = 50 ksi “with special provision.”
   - MO permits F_y = 50 ksi in cases where “structural analysis or drivability analysis requires ASTM A709 (Grade 50) steel”.
c) 35/49 states for which information was found permit or specify the use of H-piles having $F_{y\,=}$ 50 ksi for construction regardless of design capacity permitted.
d) Six states report an allowable stress design (ASD) method for maximum stress at the pile tip. This value is taken as 9 ksi – apparently nominally based on $0.25F_y$ with F_y = 36 ksi (LA, MI, NC, and WA) or 13 ksi, based on 1.45 x $0.25F_y$ (AL). ND also takes an ASD approach but with a 12.5 ksi limit, specifically identifying this value as being calculated as $0.25F_y$ with F_y = 50 ksi.
e) Other states (e.g., KY) cite a $0.25F_y$ limit (allowing F_y = 50 ksi) although an LRFD approach is followed.
f) CA limits “working [service] stress” to $0.25F_y$ (allowing F_y = 50 ksi) although this is based on the reduced cross section area (determined based on 1/16 in. section loss all around). CA limits driving stress to the AASHTO-prescribed value of $0.9F_y$ based on gross section area.
g) ID provides some relevant commentary on the use of F_y = 50 ksi: “For economy 36 ksi steel should be specified in most cases because the large ultimate loads that are possible with 50 ksi steel are very difficult to achieve and verify with standard pile driving equipment.” ID imposes a driving stress limit of $0.75F_y$.
h) WI takes a hybrid approach specifying F_y = 50 ksi material strength, basing design capacity on F_y = 36 ksi but permitting F_y = 50 ksi when considering driving stress limits.

Many states (e.g., GA, MA, NV, RI, VA, VT) specifically defer to AASHTO LRFD. Those that are silent on an issue (e.g., AZ, CT) also *de facto* defer to AASHTO which permits the use of F_y = 50 ksi for H-pile design.

A number of states cite limiting values of factored design capacity lower than AASHTO-prescribed values: $0.25F_y$ (items d-f, above) and $0.33F_y$ (e.g., LA). In these cases, the factors, 0.25 and 0.33 are given as revised values of AASHTO ϕ_c factors. Only PA explicitly maintains the AASHTO ϕ_c factors and provides an additional reduction to the pile gross section structural capacity (i.e., $\phi_c 0.66A_sF_y$).

Table 18 Summary of H-pile provisions available online (may not be current).

Number of states	**F_y permitted in determining structural capacity**	**Basis for determining P_r**	**Resulting design stress**	**Notes**
	ksi		ksi	
12	36	$0.25F_y$	9	
3	50	$0.18F_y$	9	
3	50	$0.25F_y$	12.5	
1	50	$0.27F_y$	13	
1	50	$0.28F_y$	14	
1	50	$0.29F_y$	14.5	
1	50	$0.33F_y$	16.5	current PennDOT SOL 483-14-04
1	50	$0.55F_y$	27.5	
14	50	$0.50F_y$	25	current AASHTO LRFD
12	no specification found; assumed to be current AASHTO requirement			

2.2 Regional Survey Results

A copy of the regional survey and cover letter distributed and a transcription of all responses received is provided in Appendix D.

In summary, all four responding jurisdictions report permitting design with F_y = 50 ksi; PennDOT for only about one year and Delaware, for perhaps as long as twenty years. The respondents report a reasonable number of projects designed with F_y = 50 ksi – approximately 140 altogether – with no difficulties or poor performance reported. All respondents report requiring WEAP analyses be conducted although the use of PDA or CAPWAP analyses is less consistent and seemingly reserved for test piles only.

Of the regionally surveyed states, only PennDOT applies an additional reduction beyond that required by AASHTO in determining pile axial capacity (i.e., $\phi_c 0.66A_pF_y$). NYSDOT, DelDOT and OHDOT all report using the AASHTO-prescribed design capacity. Only PennDOT has a specific 'weak rock' bearing requirement. DelDOT requires predrilling through weak layers.

PennDOT and OHDOT cite savings based on using fewer or smaller piles having F_y = 50 ksi (as compared to F_y = 36 ksi). NYSDOT and DelDOT, on the other hand, cite more efficient driving resulting from the higher resulting driving stress limits. OHDOT specifically noted that "commonly available pile driving hammers in Ohio are the limiting factor as far as the geotechnical resistance is concerned."

3. Parametric Study Methods and Matrix

The primary result of a WEAP (Wave Equation Analysis of Pile Driving) analysis is a bearing graph which represents the relationship between blow count and pile capacity. It is an efficient tool used to control pile driving. The accuracy of a bearing graph depends on the parameters related to the dynamic hammer – pile –soil system (Figure 1, Section 1.8). Input parameters are based on experience and therefore often they cannot perfectly imitate an actual situation. For instance, often the blow count is inaccurate or the soil resistance changes with time which, in the end, will result in a bearing graph with inaccurate data. Therefore knowledge of fundamental characteristics of the mechanics and dynamic interaction of all components involved in the pile driving process is required. The parameters investigated in this study are selected based on the effects that they have on the pile capacity and drivability. The input parameters are categorized as parameters related to: the driving system (hammer); the pile section; and the soil.

The hammer impact on the top of the pile generates an elastic compression wave causing strain (deformation) in the pile and motion of the pile into the soil. The length and initial intensity of the stress wave in the pile depend on: ram weight; hammer stroke; hammer efficiency; hammer and pile cushion stiffness and coefficient of restitution (COR); and helmet weight. Pile physical and mechanical properties also play important roles in pile drivability. The blow count may be twice as high for heavier and stiffer piles.

Damping and quake factors are the two important parameters related to the characteristics of the soil as described in Section 1.8. Damping is analogous to friction which must be overcome when driving a pile and quake quantifies the degree of rebound caused by the soil. Damping effects are more critical in cohesive soils for which higher skin (shaft) damping values are used (see Table 17). For this reason the blow counts required to achieve a desired pile capacity in cohesive soils are higher than those in cohesionless soils (Hussein , Bixler, & Rausche, 2003). On the other hand, soil stiffness is inversely proportional to the quake. The quake factor only varies substantially at the pile toe and is primarily as a function of the volume of soil displaced. Higher bearing capacities with lower blow counts are attained for soils with lower toe quake compared to soils with higher values of toe quake (AASHTO LRFD 2010).

3.1 Methodology

In this study, the commercially available program GRLWEAP (PDI 2010) is used for all analysis (see Section 1.8). For each case considered, a two-step analytical approach is used. Each case represents a pile section, pile length, shaft friction and 'target' capacity as described subsequently. Each analysis begins with trial hammer parameters (type, stroke and energy) and iterates upon these until the target capacity is attained at 240 blows/ft – a value defined as 'refusal'. The objective of each analysis is to achieve the target capacity with the smallest (i.e. least energy) hammer (of the five considered; described in Section 3.2.2, below) while still providing at least a 0.5 foot working stroke range. All results are reported with 'one decimal precision'; that is, 0.1 ksi, 0.1 ft and 0.1 kip-ft precision. Capacity is reported to the nearest kip. Each analysis progresses as follows:

Case 1: The pile is driven using a constant hammer stroke analysis such that the following capacities are attained at 240 blows/ft refusal:

a) A_sF_y, representing twice the AASHTO LRFD (2010) design capacity for severe driving conditions; i.e., 2 x $0.5A_sF_y$;
b) $0.66A_sF_y$, representing twice the current DM-4 (SOL 483-14-04) design capacity for severe driving conditions; i.e., 2 x $0.5(0.66A_sF_y)$; and,
c) $0.50A_sF_y$. This case provides 'historic' perspective for 36 ksi piles. That is, twice the design capacity = 2 x $0.35A_s$(36 ksi) = 25.2 ksi ≈ $0.5A_s$(50 ksi).

For each specified capacity the resulting driving stress and hammer parameters (type, stroke, energy) are recorded. The factor 2 in each case represents the required ultimate capacity to which a pile must driven when PDA is not used in the field. In instances where PDA is used to monitor the driving operation, this factor is permitted to be reduced to 1.54 (DM-4 Table 10.5.5.2.3-1). These analysis cases are referred to as 1a, 1b and 1c.

Case 2: In order to assess maximum potential pile capacity, a fourth case, using the same hammer as used in Case 1a (or Case 1b, or both) in which the pile is driven using a constant hammer stroke analysis such that the driving stress is $0.9F_y$ = 45 ksi at 240 blows/ft refusal is conducted. Resulting pile capacity and hammer parameters are recorded. These are referred to as Cases 2a and 2b.

Case 3: Using the same hammer as used in Case 1a, the pile is driven using a constant hammer stroke analysis such that the capacity is $0.66A_sF_y$ ***or*** the driving stress reaches the PennDOT-prescribed lower limit of 25 ksi at 240 blows/ft refusal (DM-4 C6.15.3P). This represents the minimum PennDOT-acceptable capacity to which the pile/hammer case may be driven. From this case, the following data is recorded: [minimum] stroke, pile capacity at refusal and hammer energy. If the difference in required stroke between cases 2 and 3 does not exceed 0.5 ft, a different hammer will be selected and cases 2 and 3 repeated.

Case 4: Using the same hammer as used in Case 1a, the pile is driven using a constant hammer stroke analysis such that the driving stress is 25 ksi at 240 blows/ft refusal (DM-4 C6.15.3P). From this case, the following data is recorded: [minimum] stroke, pile capacity at refusal and hammer energy.

3.2 Parameter Selection

The parameters considered in this study are described briefly in the following paragraphs.

3.2.1 Pile section

Three pile sections are selected:

HP 14x117 is a representative heavy section which is compact for axial load at F_y = 50 ksi. Benchmark data available is available from PennDOT (2012).

HP 12x74 is a representative medium section and is the most common shape used in PA. Benchmark data is available in Publication 15A, PennDOT (2012), PTC (2011) and recent 50 ksi pile driving records.

HP 10x57 is a compact section having capacity at F_y = 50 ksi suitable to 'replace' 36 ksi HP12x74 piles; theoretically affecting a weight savings of 17 lbs/ft or 23%.

3.2.2 Hammer types, weights and cushions

Hammer types for inclusion in the analyses were recommended by PennDOT as those readily available to PA contractors; these are summarized in Table 19 and are shown from smallest to largest from left to right across the table. Hammer parameters used in the GRLWEAP analysis are also shown.

Table 19 Hammer parameters used in this study.

Hammer:		**ICE I-12v2**	**Pileco D19-42**	**ICE I-30v2**	**ICE I-36v2**	**ICE I-46v2**
GRLWEAP ID		1501	852	1504	1505	1506
ram weight	kips	2.82	4.01	6.61	7.94	10.14
maximum stroke	ft	11.45	12.6	12.6	13.1	13.1
rated stroke	ft	10.5	10.6	11.5	11.8	11.8
ram diameter	in.	11.8	12.6	16.5	19.7	19.7
efficiency		0.80	0.80	0.80	0.80	0.80
energy/power	kip-ft	29.6	42.5	76.0	93.7	119.8
fuel setting	psi	1450	1520	1570	1510	1560
Cushion area	in^2	398	398	398	491	491
Cushion modulus	ksi	175	285	175	175	175
Cushion thickness	in.	2.0	2.0	2.0	4.0	4.0
COR		0.91	0.80	0.91	0.91	0.91

3.3.3 Soil types above rock

The ground is composed of layers of soils that support the pile by friction and the bedrock at the bottom that support the pile by bearing. Only hard rock will be considered as bearing strata as this is the most severe driving condition. Non-cohesive soil is considered since this has smaller skin damping, and therefore also represents a more severe driving scenario. On the other hand, cohesive soils are critical when considering minimum hammer stroke requirements.

3.2.4 Shaft friction

PennDOT (2012) recommended using 20% shaft friction, potentially increasing this to 30%. Values of both 20 and 30% are used in the present study.

3.2.5 Pile length

Representative embedded pile lengths of 20, 50 and 80 feet are considered in the analyses. Each pile has 4 feet added to its embedded length facilitate driving.

Table 20 represents a matrix of 126 base scenarios (i.e.: 3 pile shapes x 3 pile lengths x 2 shaft friction values x 7 analysis cases). Additional sensitivity analyses addressing toe damping, toe quake and skin damping are also made on a subset of these base scenarios considering only HP 12x74 piles having an embedded length of 50 ft. Benchmark analyses will also be made as described in Section 3.3.

Table 20 Analysis parameter matrix.

Parameter	**units**	**values considered in analyses**
pile Section		HP 10x57, HP 12x74 and HP 14x117
embedded length	ft	20, 50 and 80 ft
pile length	ft	24, 54 and 84 ft
hammer		smallest hammer of those listed in Table 19 that achieves target capacity at 240 blows/ft
toe damping	sec/ft	0.10 (rock); 0.15 (soil; 50 ft long HP 12 x 74 only)
toe quake	in.	0.05 (hard rock); 0.10 (soft rock; 50 ft long HP 12 x 74 only)
skin damping	sec/ft	0.05 (non-cohesive soil); 0.20 (cohesive soil; 50 ft long HP 12 x 74 only)
skin quake	in.	0.10
shaft friction	%	20 and 30

3.3 Benchmark scenarios

Within the proposed parameter scenario matrix it is necessary to establish some benchmark tests – tests for which the PDA/CAPWAP data is available – against which a comparison of analytical data may be made. This helps to validate the GRLWEAP analyses conducted and permits a refined assessment of shaft friction parameters.

The five analyses presented by PennDOT (2012), summarized in Table 3, are essentially contained within the proposed analytic matrix and are used as benchmarks; these will require additional analysis runs to match pile lengths and hammer types. Similarly, three analyses presented in PTC (2011), summarized in Table 4, are appropriate benchmark candidate data, although these were driven using an ICE-19v2, requiring additional individual analyses to be conducted.

Due to differences in hammers, a direct comparison with Publication 15A data is generally not possible since the hammers reported in Pub. 15A are smaller than those used in the present study. Nonetheless, the data presented on Pub. 15A sheet 18, albeit using an ICE 640 hammer (similar to a Pileco D19-42), may prove an appropriate benchmark case.

Finally, PennDOT has provided some recent pile driving analyses from which benchmarks for 50 ksi design capacity may be obtained. Although having very short pile lengths of 13.5 and 16.5 feet, respectively, TP-439 and TP-440 reported for Abutment 2 of Structure S-33234A (Grindstone Bridge) on SR 4002 are suitable benchmarks for HP 12x74 driven using a Pileco D19-42 hammer.

A summary of benchmark analysis cases, along with their results, is provided in Table 23 at the end of Chapter 4. Each benchmark test is analysed as indicated and the result compared with the available PDA/CAPWAP data. Values of toe damping (0.10), toe quake (0.05), skin damping (0.05) and skin quake (0.10) are the same as those used in the parametric study. As in PennDOT (2012), shaft friction is varied to assess the effect of this parameter and determine the value most closely approximating available data.

4. Results of Parametric Study and Discussion

This chapter reports the findings of the parametric and benchmark studies described in Chapter 3. The results of the WEAP analyses of all cases are provided in Appendix E. An illustrative example of one series (HP12x74 having L = 54 ft) of output from parametric analysis is provided in Table 21. The shaded entries in each row represent the ‘target’ values for each analysis as described in Section 3.1

Table 21 Results of WEAP analysis for HP12x74.

Case	Case	Hammer	At 240 blows/ft refusal…					
			Pile capacity			Driving stress	Stroke	Energy
			$1/A_sF_y$	ksi	kips	ksi	ft	kip-ft
1a	HP12x74 (A_s = 21.8 in^2) L = 54 ft SF = 0.20 TD = 0.10; TQ = 0.05 SD = 0.05; SQ = 0.10	ICE 1-36v2	1.00	50.0	1090	62.0	11.81	52.00
1b		Pileco D19-42	0.66	33.0	719	39.0	9.75	23.60
1c		ICE 1-12v2	0.50	25.0	545	30.8	8.66	12.90
2a		ICE 1-36v2	0.73	36.5	800	44.8	8.20	29.90
2b		Pileco D19-42	0.70	35.0	763	41.5	10.60	26.50
3		ICE 1-36v2	0.66	33.0	719	39.5	7.11	23.40
4		ICE 1-36v2	0.43	21.6	466	25.2	4.90	11.30

Based only on the case shown in Table 21, the following is observed:

1. The AASHTO-permitted capacity of the HP12x74 considered, A_sF_y, cannot be reached without significantly exceeding the driving stress limit of $0.9F_y$ = 45 ksi. A driving stress of 62 ksi was predicted. [from case 1a]
2. The SOL 483-14-04-permitted capacity of the HP12x74 considered, $0.66A_sF_y$, can be reached without exceeding the driving stress limit of $0.9F_y$ = 45 ksi. A driving stress of 39 ksi was predicted. [case 1b]
3. The maximum capacity that can be achieved, respecting the driving stress limit of $0.9F_y$ = 45 ksi is $0.73A_sF_y$ = 800 kips (using an ICE I-36v2) or $0.70A_sF_y$ = 763 kips (using an Pileco D19-42) [cases 2a and 2b]. In the latter case, the maximum stroke of the hammer limited the driving capacity and resulted in a maximum driving stress of only 41.5 ksi.
4. Requiring a minimum driving stress of 25 ksi at refusal results in a ‘minimum’ capacity of $0.43A_sF_y$ = 466 kips [case 4].
5. For the ICE I-36v2 hammer, the limits reported in observations 3 and 4 are found over a stroke range of 4.9 to 8.2 feet (range = 3.3 ft) [case 2a – case 4].
6. The ICE-I-36v2 hammer is shown to satisfactorily drive the pile – meeting, or exceeding a pile capacity of $0.66A_sF_y$ while respecting upper [case 2a] and lower [case 3] driving stress limits and having a stroke between these limits of 1.09 ft (exceeding 0.5 ft) [case 2a – case 3].
7. The Pileco D19-42 hammer is shown to satisfactorily drive the pile – meeting, or exceeding a pile capacity of $0.66A_sF_y$ while respecting upper [case 2b] and lower [case 1b] driving stress limits and having a stroke between these limits of 0.85 ft (exceeding 0.5 ft) [case 2b – case 1b].
8. The change from DM-4 to SOL 483-14-04 provisions resulted in the following [comparing cases 1c and 1b]:
   a) A pile capacity increase of 32% (545 to 719 kips).
   b) A larger hammer (ICE I-12v2 to Pileco D19-42) being required to drive the pile (energy increase of 77%).
   c) A resulting 20.5% increase in pile driving stress (31 to 39 ksi).

Results from all analyses are presented in Appendix E and synthesized in the follow section.

4.1 Observations from GRLWEAP analyses

4.1.1 Driving stress

Figure 3 shows the driving stress results for the three HP sections investigated in the parametric study. Each group of six data points are arranged by pile shape and target capacities at refusal (i.e., results of cases 1a through 1c) and are presented in the order indicated in the box at the lower right corner of the Figure. The dashed lines in the plot show the upper and lower limits for the driving stress; that is, $0.9F_y$ = 45 ksi and 25 ksi, respectively.

The data shown in Figure 3 clearly shows that the driving stress will always exceed ultimate stress. Therefore piles cannot achieve an ultimate stress of A_sF_y when driving stress is limited to $0.9A_sF_y$. However, this does not imply that a design capacity of $0.5A_sF_y$ cannot be achieved, it only requires PDA to accompany driving – in which case the target ultimate capacity is only $0.5A_sF_y/0.65 = 0.77A_sF_y$ rather than $0.5A_sF_y/0.50 = A_sF_y$ when no PDA is used. By the same token, the minimum permissible driving stress of 25 ksi = $0.5A_sF_y$ implies that the minimum drivable ultimate capacity is only somewhat lower than this.

Due to hammer limitations, it was not possible to find a hammer suitable to drive HP14x117 piles having a length greater than 50 ft to a capacity of A_sF_y. Using the largest available hammer (ICE I-46v2), these cases were driven to a capacity between $0.81A_sF_y$ and $0.84A_sF_y$ as indicated in Figure 3 by the solid triangles.

For a given pile section, driving stress decreases with increased pile length. The driving stress also decreases when shaft friction is increased from 0.20 to 0.30. This effect is clearly more pronounced for longer piles and is somewhat more pronounced for larger pile sections due to their greater perimeter dimension.

Due to the need for larger hammers, driving stress increases with pile section. For all piles (except HP14x117 having L = 20 and shaft friction = 0.20) driving stress for piles driven to a capacity of $0.66A_sF_y$ remained below the $0.90A_sF_y$ = 45 ksi limit.

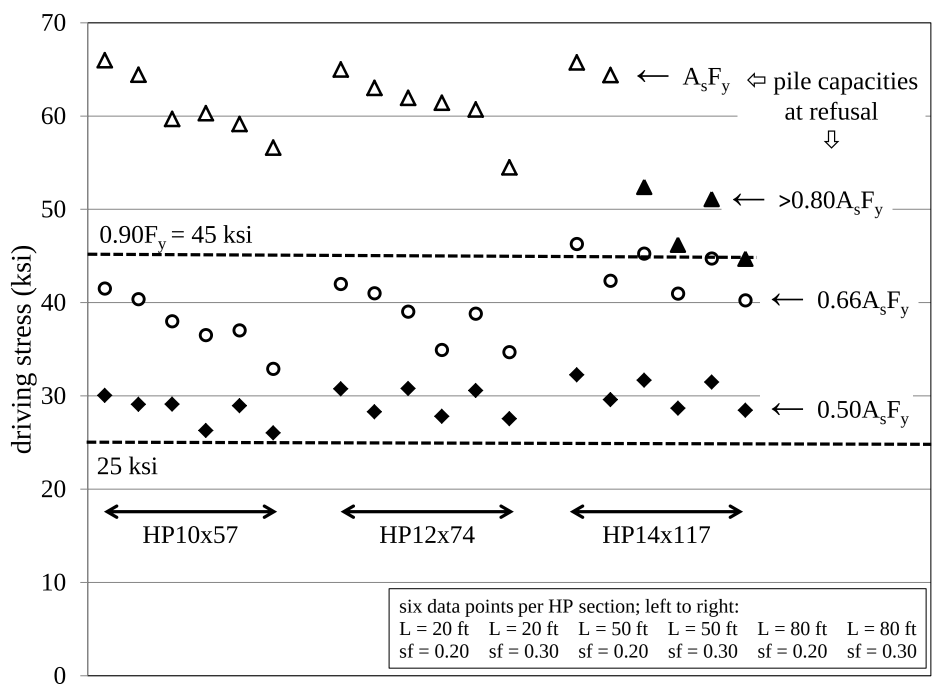


Figure 3 Driving stress distribution.

4.1.2 Attainable pile ultimate capacity

Figure 4 shows the range of attainable pile ultimate capacities for the HP sections considered. Using the minimum permitted driving stress of 25 ksi, all pile capacities ultimately fell between $0.40A_sF_y$ and $0.5A_sF_y$ [case 4]

Driving piles to the maximum permitted driving stress of $0.90A_sF_y$ = 45 ksi, resulted in pile capacities ranging from $0.64A_sF_y$ to $0.76A_sF_y$ [case 2b] All HP10x57 piles exceeded $0.70A_sF_y$ and the achievable capacity falls with increasing pile size.

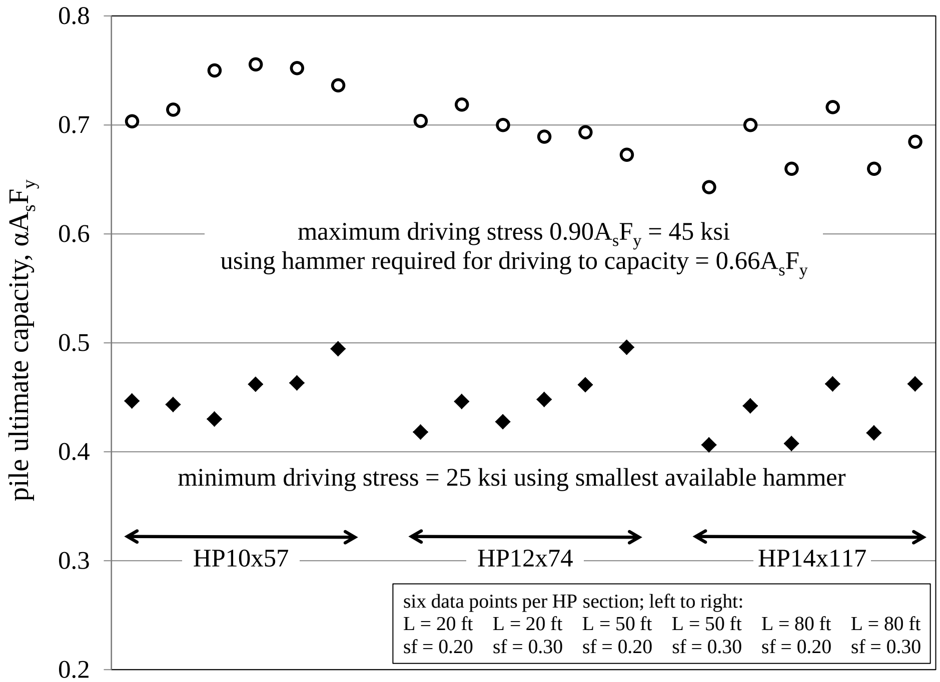


Figure 4 Pile ultimate capacity range.

4.1.3 Range of hammer stroke

Figure 5 shows the hammer stroke range available to drive the piles considered to a capacity of $0.66A_sF_y$ [case 2b – case 1b]. All HP10x57 and HP12x74 piles had available hammer strokes exceeding the minimum range of 0.5 ft. The stroke range decreased with increasing pile size. Only HP14x117 piles having shaft friction = 0.30 exhibited inadequate hammer stroke ranges less than 0.5 ft.

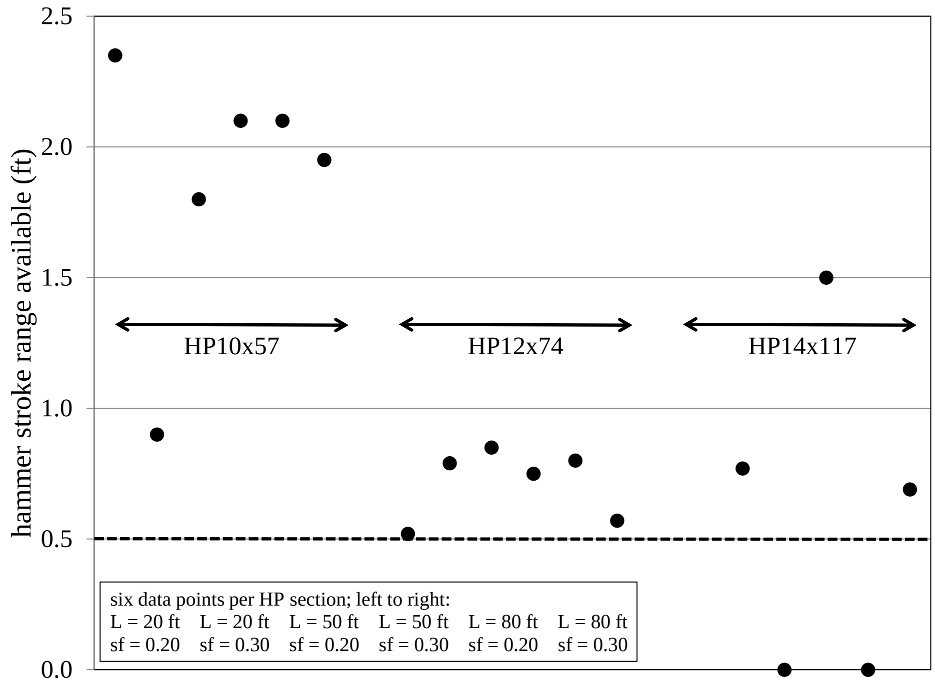


Figure 5 Hammer range available to attain ultimate capacity = $0.66A_sF_y$.

4.2 Varying Driving Parameters

The histogram representation of ultimate pile capacity and driving stress variations obtained by varying selected parameters are shown in Figures 6 and Figure 7, respectively. Four cases are shown for each pile target capacity (horizontal axis):

1. The first case is control case from the primary analysis described above.
2. The second increased shaft damping from 0.05 to 0.20 sec/ft leaving all other parameters the same as the control.
3. The third increases toe damping from 0.10 to 0.15 sec/ft and toe quake from 0.05 in. to 0.10 in. leaving all other parameters the same as the control.
4. The fourth increases shaft damping and toe damping and quake as in the previous cases.

As can be seen in Figure 6, varying the parameters as indicated has little effect on pile ultimate capacity at a given target capacity. Due to reduced driving stresses, piles having shaft damping increased to 0.20 achieved marginally higher ultimate capacities. Driving stresses are also not significantly affected although increasing toe damping and quake values increases driving stresses marginally (Figure 7).

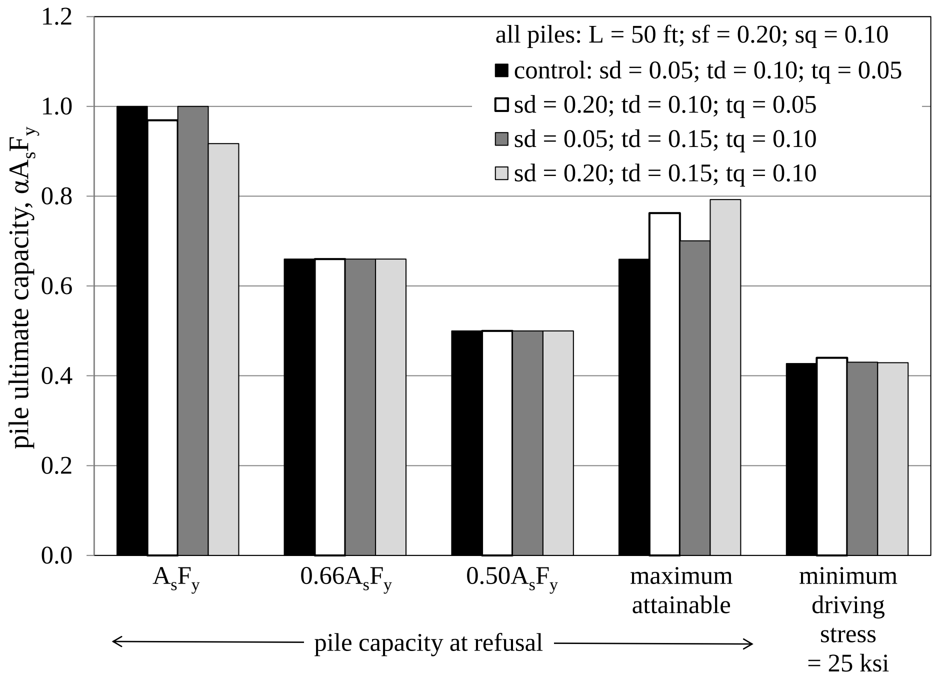


Figure 6 Effect of GRLWEAP parameters on pile ultimate capacity.

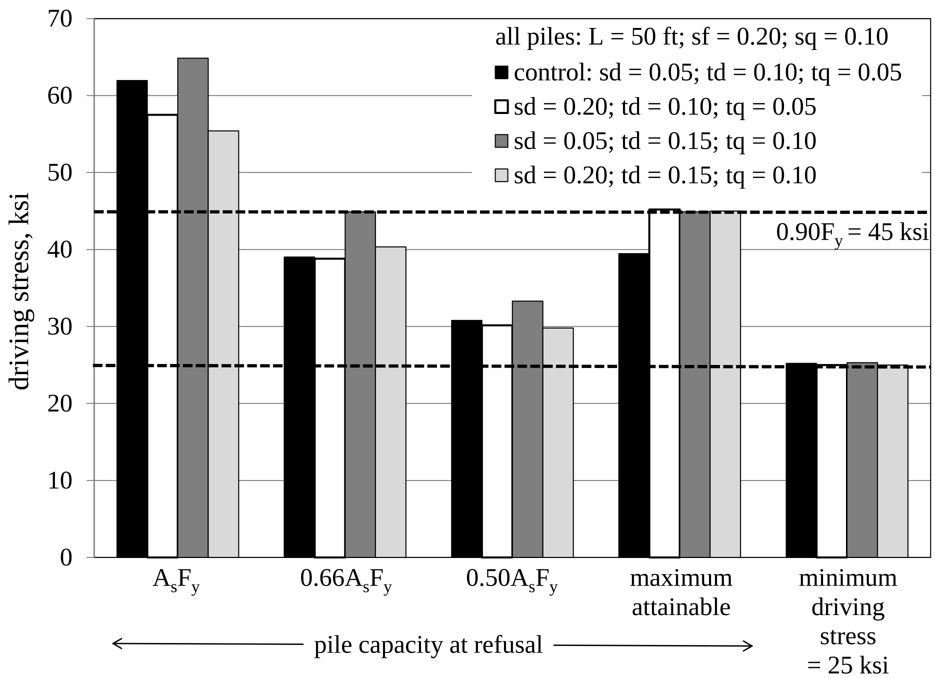


Figure 7 Effect of GRLWEAP parameters on driving stress.

4.3 Ratio of Driving Stress to Ultimate Stress

Table 22 summarizes and compares the ratio of predicted driving stress to ultimate stress for all analyses conducted. Only data with 20% shaft friction are included.

As described in Section 4.1.1, smaller pile sections require relatively lower driving stresses and therefore allow greater relative capacities to be achieved. Longer pile sections also result in proportionally lower driving stresses.

Given the relatively consistent COV values, these ratios may be used as a rule of thumb for estimating driving stress based on required pile ultimate capacity. Using average values reported in Table 22 and a driving stress limit of $0.9F_y$, HP10x57 sections may be driven to a capacity of $0.76A_sF_y$ (0.9/1.19 = 0.75) while HP14x117 sections would be limited to $0.70A_sF_y$ (0.9/1.29). Using the 'high' value of the ratios, these capacity limits become $0.68A_sF_y$ and 0.65 A_sF_y, respectively.

Table 22 Ratio of predicted driving stress to target ultimate stress.

	All shapes considered				**HP10x57**	**HP12x74**	**HP14x117**
Pile length	All	20 ft	50 ft	80 ft	All	All	All
Average	1.23	1.27	1.22	1.19	1.19	1.21	1.29
COV	0.06	0.05	0.05	0.06	0.05	0.05	0.04
Low	1.07	1.12	1.15	1.07	1.07	1.10	1.20
High	1.40	1.40	1.37	1.36	1.32	1.30	1.40

4.4 Conclusions Based on Present Code Provisions

The following conclusions relevant to AASHTO and PennDOT practice are made:

1. The AASHTO permitted pile capacity of $0.5A_sF_y$ is not technically achievable without the reduction in required over strength permitted using a PDA (as discussed in Section 4.1.1). Even using a PDA, this capacity may only be achievable for smaller pile sections [case 1a].
2. The SOL 483-14-04 permitted pile capacity of $0.5(0.66)A_sF_y$ in which F_y = 50 ksi is achievable in cases considered although driving stress in the large HP14x117 piles approaches the limit of $0.9A_sF_y$ [case 1b]
3. The theoretical increase in pile capacity realized by accounting for the increase of F_y from 36 to 50 ksi and the revisions to the PennDOT standard is a factor of 1.31 (i.e.: from (0.35 x 36 ksi)A_s to (0.50 x 0.66 x 50 ksi)A_s). This theoretical increase is achievable for all cases considered [compare cases 1c and 1b].
4. For pile sections lighter than HP12x74 having F_y = 50 ksi, the previously (DM-4) prescribed value of $\varphi_c = 0.35$ is achievable.

4.5 Benchmark Scenarios: GRLWEAP Analyses Results

Results of GRLWEAP analyses conducted for benchmark scenarios described in Section 3.3 are shown in Table 23.

The GRLWEAP analyses for the first five benchmark runs, TP-2, B3, P5, TP-12, and 3054 (PennDOT 2012) were conducted over four cases of shaft friction (10%, 20%, 30% and 40%). The remainder of the runs considered only one case of shaft friction (as determined from source documentation) as shown in Table 23. The ratio of predicted driving stress (using GRLWEAP) to that observed in the field (CAPWAP) is provided in each case. In general, it would be preferable that the predicted driving stress exceed the observed value for a conservative design to result.

For the most recent data available from PTC (2011; that is TP-1, TP-2 and TP-3) and from 2014 field data (TP 439 and TP 440), the WEAP data generally agrees very well with the field data. The driving stress predicted for the Sheet 18 (*Pub 15A* 1989) is notably lower than indicated in *Publication* 15A, however, as noted in Section 1.5.1, the data from this document is limited in the context of the present work. Additionally, the hammer and cushion data used in the generation of Sheet 18 are unknown and were assumed in the present analysis.

The benchmarking of the PennDOT (2012) data is inconsistent. Predictions for TP-2(F) and 3054 are quite good and indicate that a shaft friction of 20% is appropriate for predicting driving stresses. B3 and P5 predictions vary to a greater extent and indicate a shaft friction of 30% is appropriate. TP-12 overestimates observed field data. The field-reported driving stress for TP-12, an HP14x117 driven to $0.83A_sF_y$, is given as only 29.7 ksi which seems too low for this case resulting in the large over-prediction by the present analysis. Shaft friction values of 20% are most predictive while remaining marginally conservative (i.e. Predicted/Field ration > 1.0).

Table 23 Results of benchmark analyses comparing CAPWAP and predicted driving stresses.

ID	**HP**	**F_y**	**A_sF_y**	**Pile length**	**Embedded length**	**Bearing rock and RQD**	**Hammer (hammer energy)**	**Capacity at 240 blows/ft[1]**			**Shaft friction**	**Predicted driving stress**	**Predicted Field**
								Predicted Stress	**Predicted Capacity**	**Field driving (CAPWAP) stress**			
		ksi	kips	ft	ft			1/ A_sF_y	kips	ksi	%	ksi	
TP-2(F)[a]	12x74	36	784	40.3	9.1	Siltstone RQD = 10%	ICE I-19v2 (42,200 ft-lbs)	0.57	447	26.0	0.10	27.5	1.06
											0.20	26.1	1.01
											0.30	24.9	0.96
											0.40	24.2	0.93
B3[a]	10x57	36	605	60.0	44.1	Limestone RQD = 36%	Pileco D19-42 (25,600 ft-lbs)	0.73	440	26.2	0.10	32.1	1.23
											0.20	29.2	1.11
											0.30	27.6	1.05
											0.40	26.3	1.00
P5[a]	12x74	36	784	70.0	38.0	Soft Schist RQD = 8%	Pileco D19-42 (30,400 ft-lbs)	0.65	509	24.0	0.10	29.6	1.23
											0.20	27.1	1.13
											0.30	24.4	1.02
											0.40	22.1	0.92
TP-12[a]	14x117	36	1238	65.3	61	Siltstone RQD = 20%	ICE I-30v2 (64,680 ft-lbs)	0.83	1024	29.7	0.10	44.1	1.48
											0.20	40.3	1.36
											0.30	36.4	1.23
											0.40	33.0	1.11
3054[a]	14x117	36	1238	100.6	90.1	Sandstone RQD = 70%	Pileco D19-42 (36,000 ft-lbs)	0.56	696	24.9	0.10	27.2	1.09
											0.20	24.9	1.00
											0.30	22.5	0.90
											0.40	20.1	0.81
TP-1[b]	12x74	50	1090	60.0	36.0	Saprolite	ICE I-19v2 (42,200 ft-lbs)	0.63	691	38.2	0.20	39.8	1.04
TP-2[b]	12x74		1090	65.0	45.0			0.60	651	36.1	0.20	37.1	1.03
TP-3[b]	12x74		1090	65.0	57.0			0.61	660	38.3	0.20	37.6	0.98
Sheet 18[c]	12x74	36	784	70.0	60.0	Grey Shale	ICE-640 (28,800 ft-lbs)	0.33	262	21	0.20	14.1	0.67
TP-439[d]	12x74	50	1090	17.6	13.5	Siltsone RQD = 25%	Pileco D19-42 (31,300 ft-lbs)	0.61	662	36.4	0.30	39.0	1.07
TP-440[d]	12x74		1090	25.3	16.5		Pileco D19-42 (37,000 ft-lbs)	0.81	883	54.2	0.30	49.9	0.92

[1] value reported by source
[a] PennDOT (2012)
[b] PTC (2011)
[c] *Publication 15A* (1989)
[d] SR 4005 Abutment 2 (Foundation Testing Services 07.30.2014)

5. Estimation of Pile Settlement

The total settlement (Δ) of a bearing pile under service loads may be estimated from Equation 4. For piles bearing on rock, settlement associated with shaft friction may be neglected (DM-4 §10.7.1.6.2).

$$\Delta = \Delta_s + \Delta_{tip} \tag{4}$$

In which:

Pile shortening (Δ_s) is determined from fundamental mechanics as (see discussion in Section 1.4.3):

$$\Delta_s = (Q_p + \xi Q_s)L/E_s A_s \tag{5}$$

Settlement due to the load at the pile tip (Δ_{tip}) may be estimated from Eq. 6 which considers a point load on an elastic half space (Vesic (1977) as reported by Das (2010)):

$$\Delta_{tip} = 0.85(Q_p/A_s)d(1-\mu^2)/E \tag{6}$$

Where: Q_p is the load carried by the pile point (i.e.: $Q_p = P - Q_s$);
Q_s is the load carried by pile skin friction (i.e.: $Q_s = sP$, where s = shaft percentage)
ξ represents the effect of the friction distribution pattern (ξ = 0.5 for uniform or parabolic distributions and ξ = 0.67 for triangular);
L is the length of the pile;
A_s the pile bearing area, this value may be varied to consider A_{pile}, A_{tip} or A_{box};
E_s is the modulus of elasticity of the pile (E_s = 29000 ksi);
d is the depth (or diameter) dimension of the pile;
μ is the Poisson ratio of the rock; and,
E is the modulus of elasticity of the rock beneath the pile point.

Empirical values of μ and E are selected within an appropriate range for bearing piles including those founded on soluble rock. Values of μ range from 0.2 to 0.3 and those of E, from 300 – 3000 ksi.

As our interest lies in maximum settlement, some parameters may be selected to produce the greatest settlement:

Minimum shaft percentage produces greatest settlement:	$s = 0.20$
Triangular friction distribution produces greatest settlement:	$\xi = 0.67$
Smallest cross section area produces greatest settlement:	$A_s = A_{pile}$
Additionally, maximizing L produces greatest settlement:	

If $P = \phi_c A_s F_y$, it can be shown that Eq. 5 is independent of the pile section (i.e., A_s) and may be written as:

$$\Delta_s = \phi_c F_y[1-s(1-\xi)]L/E_s \tag{7}$$

Setting parameters to maximize settlement: $\Delta_s = 0.934\phi_c F_y L/E_s$ (8)

Similarly, Eq 6 can be written as being independent of A_s (provided $A_s = A_{pile}$), although not entirely independent of the pile section selected (d):

$$\Delta_{tip} = 0.85\phi_c F_y(1-s)d(1-\mu^2)/E \tag{9}$$

Setting parameters to maximize settlement: $\Delta_{tip} = 0.68\phi_c F_y d(1-\mu^2)/E$ (10)

Figure 8 shows the calculation of maximized Δ_s (Eq. 8) for piles having L = 80 ft and F_y = 36 and 50 ksi. The calculations are shown for varying values of ϕ_c. Since this calculation is proportional to only F_y, maximum settlements are predicted to increase 39% due to the theoretical 39% increase in yield capacity from 36 ksi to 50 ksi and therefore pile section area utilization (i.e., $A_s F_y$).

Also shown in Figure 8 are ranges of estimated values of Δ_{tip} based on very soft (υ = 0.2 and E = 300 ksi) and very stiff (υ = 0.3 and E = 3000 ksi) bearing conditions. The ranges capture the effect of pile dimension d with the top of the range corresponding to d = 14 in. and the lower bound to d = 10 in. (Eq. 10). As seen in Figure 8, even under very soft bearing conditions, the pile shortening term, Δ_s dominates settlement for bearing piles.

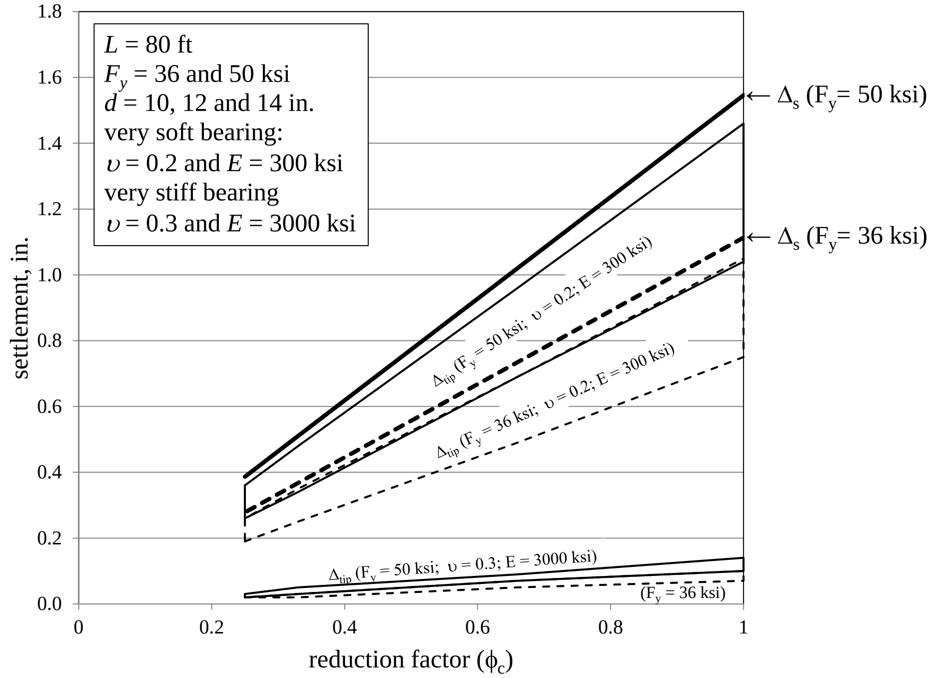


Figure 8 Range of shortening (Δ_s) and tip (Δ_{tip}) displacement components of pile settlement.

Figure 9 adds the shortening and tip displacement terms together (Eq 4) considering the upper and lower bounds of the calculations for piles having L = 80 ft. If, for example, the worst cases for F_y = 36 and 50 ksi are considered (i.e. d = 14 in.), the following results:

DM-4: estimated settlement based on $0.35A_sF_y$ with F_y = 36 ksi is about 0.75 inches.

SOL 483-14-04: estimated settlement based on $0.50(0.66)A_sF_y$ with F_y = 50 ksi is about 1.0 inch. [an increase of (50/36)(0.33/0.35) = 1.31 or 31%]

AASHTO: estimated settlement based on $0.50A_sF_y$ with F_y = 50 ksi is 1.5 in.

Based on such fundamental estimates, it is clear that settlement is directly proportional to section utilisation in terms of both F_y and ϕ_c. Additionally it is inversely proportional to E. The dominant Δ_s component is proportional to pile length, L.

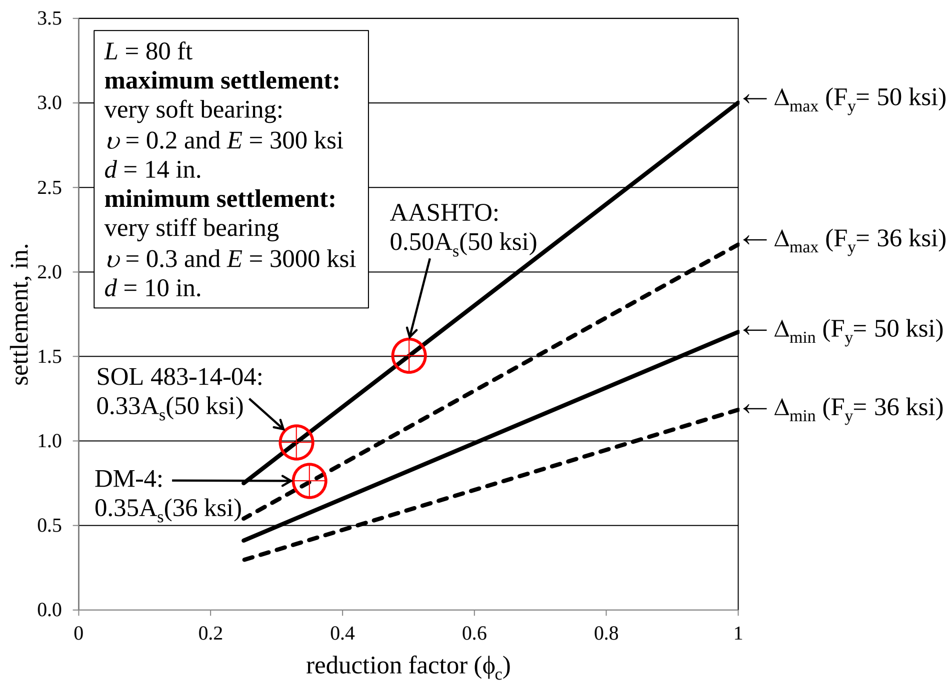


Figure 9 Pile settlement estimates for 80 foot long pile.

5.1 DM-4 §D10.5.2.2 net foundation settlement limit

DM-4 §D10.5.2.2 limits net foundation settlement to 1 inch at service loads. DM-4 C6.15.3.2P *implies* a ratio of service to design stresses of 0.25/0.35 = 0.71, although this is a gross approximation. Nonetheless, it provides an approximate basis for assessing the settlement predictions noted above and in Figure 9.

For long (L = 80 ft) bearing piles in soft rock bearing conditions (υ = 0.2 and E = 300 ksi) having parameters selected to maximize settlement (s = 0.20, ξ = 0.67, $A_s = A_{pile}$ and d = 14 in.) the analysis shown in Figure 8 and summarized in Figure 9 indicates that piles having F_y = 50 ksi and design capacities up to the AASHTO-specified capacity of $0.50A_sF_y$ will not exhibit settlements greater than approximately 1 in. at service loads. Most cases will exhibit considerably less settlement. For this reason, based on this analysis, no change to the 1 in. settlement limit to accommodate HP piles having F_y = 50 ksi is considered necessary.

6. Estimation of Cost

In order to assess potential cost savings realized by utilizing F_y = 50 ksi in place of F_y = 36 ksi for the design of steel H-piles, a rudimentary cost comparison was made. Costs were normalized on the basis of ***driven pile capacity***. To do this, the cost of driving 100,000 kips worth of pile capacity was established using the capacities determined from the GRLWEAP analyses presented in Chapter 4. The following assumptions were made based on estimates provided by PennDOT and, in some cases, extrapolated to different hammers.

1. Cost of steel HP sections was taken as $0.50 per pound (*ENR* 2014 Q4 Cost Report).
2. Pile hammer rental, mobilization/demobilization and driving costs are those given in Table 24.
3. Support crane rental and mobilization/demobilization costs are those given in Table 24.
4. Production rates are 4500 linear feet of driven pile per month per hammer.
5. The costs do not include on site moves.

It is acknowledged that these estimates may vary considerably and the cost data presented should be used to compare driving cases and not as absolute values.

Table 24 Bases for cost estimates.

Hammer	hammer monthly rental	hammer mob/demob.	required crane capacity	crane monthly rental	crane mob/demob.	driving cost
Pilco D19-42	$8,098	$1,500	90 ton	$12,000	$10,000	$7.02 / lf
ICE I-12v2[1]	$7,049	$1,500	70 ton	$10,000	$6,000	$5.46 / lf
ICE I-30v2	$9,116	$1,500	140 ton	$16,000	$20,000	$10.35 / lf
ICE I-36v2	$11,200	$1,500	140 ton	$16,000	$20,000	$10.82 / lf
ICE I-46v2[2]	$15,368	$1,500	140 ton	$16,000	$20,000	$11.76 / lf

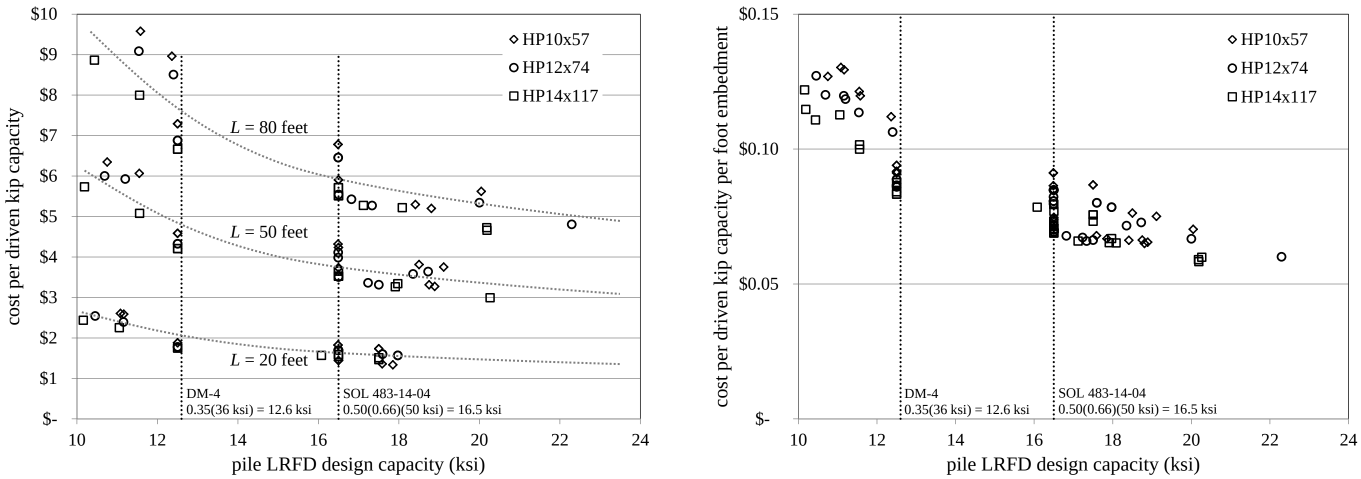
[1] assumed same as Pilco D12 costs provided by PennDOT
[2] extrapolated from ICE I-36 costs

An example calculation for the cost of driving 100,000 kips capacity of HP12x74 using a Pilco D19-42 hammer to drive to 50 ft embedded depth to a capacity of 719 kips (see case 1b on Table 21) is as follows:

piles required	100,000/719 kips	140 piles
total pile length	140 x 50 ft	7000 lf
hammer months required	7000/4500 lf/mo	1.56 months
cost of steel	74 lb/ft x 7000 ft x $0.50	$259,000
cost of hammer and crane rental	($8098 + 12,000) x 1.56 mo.	$31,353
cost of hammer and crane mobilization/demobilization	$1500 + $10,000	$11,500
driving cost	7000 lf x $7.02/lf	$49,140
Total cost to drive 100,000 kips capacity		$350,993
cost per driven kip	$350.993/100000 kip	$3.51
cost per driven kip per foot embedment	$3.51/50 ft	$0.07
cost per pile	$350,993/140 piles	$2507

Figure 10a shows the cost per driven kip capacity for all cases in the parametric study that satisfied driving stress limits (i.e. 25 ksi < driving stress < $0.9F_y$ = 45 ksi). The data is shown in terms of the pile design capacity. The design capacity from DM-4 with F_y = 36 ksi is 0.35(36) = 12.6 ksi and the SOL483-14-04 capacity with F_y = 5 ksi is 0.33(0.66)(50) = 16.5 ksi as shown on Figure 10. The data is clustered around three trend lines for L = 20, 50 and 80 ft, respectively reflecting the dominance of the steel material cost to the overall cost (see example above). The same trend is evident when the cost data is further normalized by pile length (cost per driven kip capacity per foot embedment) as shown in Figure 10b.

As shown in Figure 10, increasing the design pile capacity 31% from 12.6 ksi to 16.5 ksi results in a cost savings of approximately 20% which is independent of pile size and length.



a) estimated cost by pile length

b) estimated cost normalized by pile length

Figure 10 Estimated pile cost per driven kip capacity.

To better understand how this savings is realized, Figure 11 shows a breakdown of the cost per driven kip capacity for the case of an HP12x74 driven to L = 50 ft. The case shown is for the shaft friction = 0.20. It can be seen that the cost of steel is essentially linear and falls in proportion to the increase in steel yield strength used for design; that is the cost of steel falls 31%. The driving requirements to achieve the increased pile capacity increase the hammer costs and unit driving costs resulting in a net savings of 19% for the case shown. Also shown on Figure 11 is the cost per pile. While the cost per pile increases from $2353 to $2507 (6.5%), the number of piles required falls from 184 to 140 (31%) as the design stress increases from 12.6 ksi to 16.5 ksi resulting in the 19% net savings.

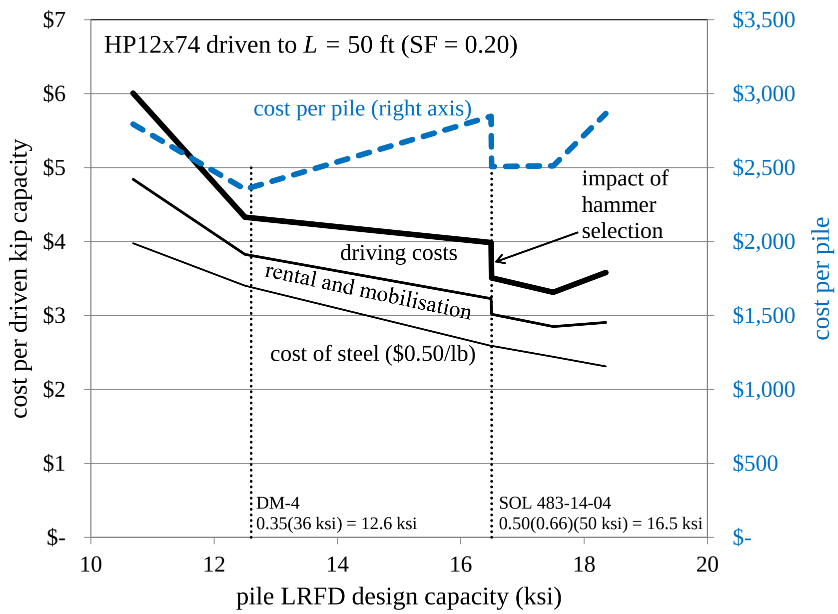


Figure 11 Estimated cost breakdown for HP12x74 having L = 50 ft.

The impact of appropriate hammer selection is clearly seen in Figure 11. Both a Pilco D19-42 (analysis #58; see Appendix E) and an ICE-36v2 (analysis #62) are able to drive the HP12x74 to a design capacity of 16.5 ksi. Based on the cost assumptions given in Table 24, the savings by using the Pilco D19-42 are clearly evident, reducing the cost per driven kip from $3.99 to $3.51, or the per pile cost from $2847 to $2507.

Finally, it is also seen in Figures 10 and 11 that permitting design capacities greater than 16.5 ksi results in only marginal additional savings.

7. Discussion and Recommendations

7.1 Observations and Conclusions from Parametric Study

The following conclusions relevant to AASHTO and PennDOT practice are made:

- The AASHTO permitted pile capacity of $0.5A_sF_y$ is not technically achievable without the reduction in required over strength permitted using a PDA (as discussed in Section 4.1.1). Even using a PDA, this capacity may only be achievable for smaller pile sections. Responses from the state surveys (Section 2) report that "the large ultimate loads that are possible with 50 ksi steel are very difficult to achieve and verify with standard pile driving equipment" (ID) and reinforce that with higher strength piles "the factored axial geotechnical resistance frequently governs design. This is particularly apparent for end-bearing piles on poor-quality and/or soft bedrock and for friction piles." (ME).
- The SOL 483-14-04 permitted pile capacity of $0.5(0.66)A_sF_y$ in which F_y = 50 ksi is achievable in cases considered although driving stress in the large HP14x117 piles approaches the limit of $0.9A_sF_y$.
- The theoretical increase in pile capacity realized by accounting for the increase of F_y from 36 to 50 ksi and the revisions to the PennDOT standard is a factor of 1.31 (i.e.: from (0.35 x 36 ksi)A_s to (0.50 x 0.66 x 50 ksi)A_s). This theoretical increase is achievable for all cases considered.
- Driving piles to the maximum permitted driving stress of $0.90A_sF_y$ = 45 ksi, resulted in pile capacities ranging from $0.64A_sF_y$ to $0.76A_sF_y$ with smaller pile sections having marginally higher achievable capacities. All HP10x57 piles considered, for instance, could be driven to values exceeding $0.70A_sF_y$ without exceeding driving stress limits.

Based on the GRLWEAP analyses conducted it is concluded that:

- As confirmed by the benchmark comparison with available CAPWAP analyses (Section 4.5), the current methodology of performing a WEAP analysis, including the various parameters used, is adequate to obtain a reasonably accurate driving analysis.
- Shaft friction values of 20% are most predictive while remaining marginally conservative.
- Requiring a WEAP analysis to approve the pile hammer and to establish the stroke range at refusal is affirmed as a practical driving analysis methodology to ensure the settlement limit is maintained and the pile is not overstressed during driving.
- Utilizing PDA for test piles to confirm WEAP analysis results is prudent practice. Additionally, since the required pile ultimate capacity may be reduced from 2 to 1.54 times the design capacity when PDA is used, PennDOT may consider the benefits of permitting PDA as a means of enhancing driven pile capacity; Further study of this approach is suggested.

7.2 Observations and Conclusion from Estimation of Pile Settlement

The analysis shown in Section 5 indicates that piles having F_y = 50 ksi and design capacities up to the AASHTO-specified capacity of $0.50A_sF_y$ will not exhibit settlements greater than approximately 1 in. at service loads. Most cases will exhibit considerably less settlement. Based on this analysis:

- No change to the 1 in. settlement limit to accommodate HP piles having Fy = 50 ksi is considered necessary.

7.3 Estimation of Cost

A representative cost analysis – normalized on the basis of 100,000 kips driven pile capacity and a number of fundamental assumptions – was conducted as reported in Section 6. It is concluded that:

- Increasing the design capacity of a pile results in a decrease in cost per driven pile capacity although due to the need for larger hammers and cranes, permitting design capacities greater than 16.5 ksi results in only marginal additional savings.

7.4 Construction Practice

While beyond the scope of this study, two current PennDOT pile construction practices that seem prudent when designing and driving end bearing piles to higher stress are:

- Requiring driving tips for end bearing piles (DM-4 §10.7.8.5P)
- Requiring full-section full-penetration welds when splicing piles. (Standard Drawing BC-757)

7.5 Capacities of Braced and Unbraced Piles

In lieu of reference to AASHTO §6.9.4.1, SOL 483-14-04 calculates the nominal compressive resistance of ***fully braced*** piles as:

$$P_n = 0.66F_yA_s \qquad \text{SOL 483-14-04 Eq. 6.15.3-1}$$

The AASHTO §6.5.4.2-specified material resistance factor, $\phi_c = 0.50$ is then applied resulting in the factored resistance: $P_r = \phi_c P_n = 0.50(0.66)F_yA_s$.

What about piles having some unbraced length kL? Above-grade examples are shown in Figure 12. Below grade, embedded piles are assumed to be fully braced (AASHTO LRFD §10.7.3.13.1); this is a reasonable assumption assuming the presence of a pile cap and no loss of soil through mechanisms such as scour.

a) Union Pacific Railway over Feather Creek, Yuba County CA (www.bphod.com)

b) *Connection details for PBES: Case Study 3* (www.fhwa.dot.gov/bridge/prefab/if09010/appd.cfm)

Figure 12 Examples of HP sections having an unsupported length.

In this case, SOL 483-14-04 Table 6.15.2-1 directs the designer to AASHTO §6.9.4.1 which defines the nominal compressive resistance of a steel HP member as follows (we will assume compact sections for this discussion; thus Q = 1):

$$P_n = 0.658^{Po/Pe}F_yA_s \qquad \text{AASHTO 6.9.4.1.1-1}$$

in which $P_o = QF_yA_s$

and $$P_e = \frac{\pi^2 E}{(K\ell/r)^2}A_s \qquad \text{AASHTO 6.9.4.1.2-1}$$

Table 6.15.2-1 modifies Eq. 6.9.4.1.1-1 as follows, limiting the calculated capacity to $0.66A_sF_y$ as follows:

$$P_n = 0.658^{Po/Pe}F_yA_s \le 0.66A_sF_y \qquad \text{AASHTO 6.9.4.1.1-1 modified by SOL 483-14-04}$$

It is noted that SOL 483-14-04 Eq. 6.15.3-1 is the mathematically equivalent to AASHTO Eq. 6.9.4.1.1-1 with $P_o/P_e = 1$.

The impact of using guidance of SOL 483 14-04 Table 6.15.2-1 is illustrated in the following example of the same axially loaded pile driven into the same soil having different extension lengths (*kL*). For the sake of comparison Q = 1.0, K = 1.0 and "severe" driving conditions are assumed. These assumptions affect resulting capacities but not the overstrength ratios discussed. Consider the following four scenarios whose calculated capacities are given in Table 25:

Scenario A: a "fully braced" (i.e. fully embedded) HP 12x74 pile.

Scenario B: an HP 12x74 driven in identical soil as Scenario A but having an extension of 10 feet above the ground level (see Figure 12b, for example). This scenario represents a case of $P_o/P_e < 1$

Scenario C: an HP 12x74 driven in identical soil as Scenario A but having an extension of 18.4 feet above the ground level (see Figure 12a, for example). This scenario represents the case of $P_o/P_e = 1$.

Scenario D: an HP 12x74 driven in identical soil as Scenario A but having an extension of 22 feet above the ground level (see Figure 12a, for example). This scenario represents a case of $P_o/P_e > 1$

All four scenarios have the same 'unknowns' associated with their embedment, most notably, their effective depth of fixity; thus similar reliabilities should be expected.

Table 25 Calculation of unsupported length corresponding to case in which $P_e = P_o$.

		Scenario	**A**	**B**	**C**	**D**
	kL	ft	0	10	18.4	22
	$P_o = QA_sF_y$	kips	-	1090	1090	1090
	$P_e = \dfrac{\pi^2 E}{(kL/r)^2} A_s$	kips	-	3695	1090	763
	P_o/P_e		0.00	0.30	1.00	1.43
AASHTO[1]	$P_n = 0.658^{Po/Pe} F_y A_s$	kips	1090	961	717	599
AASHTO[1]	$\phi_c = 0.50$	kips	545	480	358	300
Recommendations B and D (see below)	$\phi_c = 0.33$	kips	363	320	239	200
SOL 483 14-04	$P_n = 0.658^{Po/Pe} F_y A_s \leq 0.66 A_s F_y$	kips	719	719	717	599
SOL 483 14-04	$\phi_c = 0.50$	kips	360	360	358	300
Overstrength provided						
AASHTO/SOL 483 14-04			1.50	1.33	1.00	1.00
Recommendations/SOL 483 14-04			1.00	0.89	0.66	0.66
Recommendations/AASHTO			1.50	1.50	1.50	1.50

[1] not modified by DM-4 or SOL 483-14-04

As can be seen in from the AASHTO/SOL 483 overstrength ratio shown, the present SOL 483 14-04 results in a varying degree of overstrength when compared to AASHTO and no overstrength is present at larger values of *kL*.

Recommendations B and D (see below; essentially using AASHTO Eq. 6.9.4.1.1-1 with $\phi_c = 0.33$), result in a uniform overstrength when compared to AASHTO. This result ***achieves a uniform overstrength compared to the nominal AASHTO-prescribed capacity***. Furthermore, the Recommendations may be said to be 'calibrated' to the present AASHTO-prescribed capacity for 'fully supported' piles (and therefore uses the same reliability characteristics; i.e. 1/1.5 = 0.66) and then calculates capacities that make sense from the perspective of fundamental mechanics; that is, capacity falls as *kL* increases.

While mathematically identical (i.e.: $\phi_c 0.66 A_s F_y = 0.33 A_s F_y$ when $\phi_c = 0.50$) the *mechanical and statistical meanings* of the 0.66 and ϕ_c are, in the opinion of the research team, incorrectly applied leading to a variation the overstrength when compared to the AASHTO LRFD-prescribed practice. As determined from the Survey of State practice (Chapter 2), PennDOT is the only jurisdiction that takes this approach.

7.6 Recommendation for Revision to DM-4

The following recommendations are made for the revision of DM-4 (as amended by SOL 483-14-04):

It is recommend that the nominal compressive resistance of braced piles be calculated as $P_n = F_y A_s$ (i.e., per AASHTO 2014 Eq. 6.9.4.1.1-1) and a value of ϕ_c be adopted that addresses the approach to bearing pile design in Pennsylvania practice which considers factors such as the strength of rock, potential for pile damage and hammer energy, etc.. Adopting such a single ϕ_c factor approach would allow most of §6.15.2 and much of 6.15.3 to be deleted from DM-4 thereby better aligning DM-4 with AASHTO, simplifying design and mitigating opportunities for confusion and/or error.

Adopting the value $\phi_c = 0.33$ for severe driving conditions is mathematically equivalent to the present SOL 483-14-04 provisions while allowing the noted clauses to be deleted. Limiting driving stress to $0.9F_y = 45$ ksi as recommended by AASHTO and SOL 483-14-04 is believed to be appropriate.

Recommendations for revision to DM-4

Item	DM-4 amended by SOL 483-14-04	Proposed revision	Rationale
A	*5.5.4.2.1 Conventional Construction* • for axial resistance of the concrete portion and the steel portion of concrete filled steel pipe piles bearing on soluble bedrock in compression .... $\varphi = 0.273$ **5.13.4.1 General** The following shall supplement A5.13.4.1. Piles shall be designed as structural members capable of safely supporting all imposed loads. A pile group composed…	delete second bullet ***only*** (shown) **5.13.4.1 General** The following shall supplement A5.13.4.1. Piles shall be designed as structural members capable of safely supporting all imposed loads. For concrete filled steel pipe piles bearing on soluble bedrock (limestone, etc.), the calculated net bearing stress shall not exceed 9 ksi. A pile group composed …	see item C
B	**6.5.4.2 Resistance Factors** The following shall supplement the pile resistance factors in A6.5.4.2. • for axial resistance of piles bearing on soluble bedrock ....$\varphi_c = 0.273$ • for axial resistance of concrete filled pipe piles, see D5.5.4.2	**6.5.4.2 Resistance Factors** The following shall replace the pile resistance factors in A6.5.4.2. • for axial resistance of piles in compression and subject to damage due to severe driving conditions where use of a pile tip is necessary .... $\varphi_c = 0.33$ • for axial resistance of piles in compression under good driving conditions where use of a pile tip is not necessary...................... $\varphi_c = 0.40$ • for axial resistance of steel portion of concrete filled pipe piles in compression........... $\varphi_c = 0.33$ • for combined axial and flexural resistance of undamaged piles: axial resistance ……….................... $\varphi_c = 0.80$ axial resistance (H-piles) .... $\varphi_c = 0.46$ flexural resistance ………….............. $\varphi_f = 1.00$	The impact of the proposed revisions on the calculation of P_r are *mathematically* identical to the SOL 483-14-04. Reversion to the φ_c values given in DM-4 (2012) – i.e.: (in order shown) 0.35, 0.45, 0.35 – would represent only a marginal difference and is not believed to affect design in a meaningful way. That is, withdrawing SOL 483-14-04 is sufficient. see item C for rationale for deletion of third bullet

C	**6.15.2 Structural Resistance** The following shall supplement A6.15.2. For piles bearing on soluble bedrock (limestone, etc.), the φ factor of 0.273 shall be applied to the axial capacity of the pile to provide pile group redundancy and limit the design stress to 9 ksi. Table 6.15.2-1 – Pile Resistance References	**6.15.2 Structural Resistance** The following shall supplement A6.15.2. For piles bearing on soluble bedrock (limestone, etc.), the calculated net bearing stress shall not exceed 9 ksi. delete Table 6.15.2-1 in its entirety	As discussed in Section 1.6.2, this proposed revision is more concise than the present wording and captures the true intent of the provision – to limit the bearing stress on the rock, not the pile. Table is not required with proposed revision B
D	**6.15.3 Compressive Resistance** The following shall supplement A6.15.3. The design of steel piles shall follow A6.9, except as specified herein. For braced H-piles and braced unfilled steel pipe piles, the nominal compressive resistance shall be taken as: P_n=0.66$F_y$$A_s$ (6.15.3-1) where the and the slender element reduction factor, Q, shall be 1.0. For H-piles and unfilled steel pipe piles with unbraced lengths, the nominal compressive resistance shall be established in accordance with A6.9.4. For unbraced unfilled steel pipe piles, the values of P_n computed from A6.9.4 shall not exceed P_n=0.66$F_y$$A_s$. For H-piles, the computed values of P_r, the factored resistance, shall not exceed those established in Tables D6.15.3.2P-1 and D6.15.3.2P-2. For concrete filled steel pipe piles, see D5.13.4.7.1P for the factored resistance. **6.15.3.1 Axial Compression** The following shall replace A 6.15.3.1. For piles under axial load, the factored resistance of piles in compression, P_r, shall be taken as specified in A6.9.2.1 using the resistance factor, φ_c, specified in A6.5.4.2 except as specified herein.	**6.15.3 Compressive Resistance** The following shall supplement A6.15.3. The design of steel piles shall follow A6.9, except as specified herein. **6.15.3.1 Axial Compression** The following shall replace A 6.15.3.1. For piles under axial load, the factored resistance of piles in compression, P_r, shall be taken as specified in A6.9.2.1 using the resistance factor, φ_c, specified in D6.5.4.2. **6.15.3.2 Combined Axial Compression and Flexure** The following shall replace A 6.15.3.2. Piles subjected to axial load and flexure shall be designed in accordance with A6.9.2.2 using the resistance factors, φ_c and φ_f, specified in D6.5.4.2. Vertical H-pile foundations designed using COM624P or LPILE per D10.7.3.12.2P may use the values given in Tables 6.15.3.2P-1 and 6.15.3.2P-2. where: D = Depth of the pile (in.) Area = Area of the pile (in.^{2}) I_x, I_y = Moment of inertia about their respective axis (in.^{4}) P_{rSTR} = Factored axial resistance (kips) P_r = Factored axial resistance for combined	Consistent with Item B, the proposed revisions essentially return to DM-4 (2012) wording which is more concise. Revised Tables 6.15.3.2P-1 and 6.15.3.2P-2 are provided following this table of revisions.

	6.15.3.2 Combined Axial Compression and Flexure The following shall replace A 6.15.3.2. Piles subjected to axial load and flexure shall be designed in accordance with A6.9.2.2 using the resistance factors, φ_c and φ_f, specified in A6.5.4.2. Vertical H-pile foundations designed using COM624P or LPILE per D10.7.3.12.2P may use the values given in Tables 6.15.3.2P-1 and 6.15.3.2P-2. where: D = Depth of the pile (in.) Area = Area of the pile (in.^{2}) I_x, I_y = Moment of inertia about their respective axis (in.^{4}) P_{rSTR} = Factored axial resistance (kips) P_r = Factored axial resistance for combined axial and flexural resistance (kips) M_{rx}, M_{ry} = Factored flexural resistance of the vertical pile in the x-axis and y-axis, respectively (kip-ft.)	axial and flexural resistance (kips) M_{rx}, M_{ry} = Factored flexural resistance of the vertical pile in the x-axis and y-axis, respectively (kip-ft.)	
E	AASHTO Eq. 6.12.2.2.1-2: $M_n = \left[1-\left(1-\frac{S_y}{Z_y}\right)\left(\frac{\lambda_f-\lambda_{pf}}{0.45\sqrt{E/F_y}}\right)\right]F_y Z_y$	AASHTO Eq. 6.12.2.2.1-2: $M_n = \left[1-\left(1-\frac{S_y}{Z_y}\right)\left(\frac{\lambda_f-\lambda_{pf}}{0.45\sqrt{E/F_y}}\right)\right]M_p$	As discussed in Section 1.6.4, the rationale for this change is that M_{py} is defined by AASHTO LRFD to be $1.5S_yF_y$ for HP sections in C6.12.2.2.1. The intent of Eq. 6.12.2.2.1-2 is to modify the plastic moment capacity. For some HP sections $Z_y/S_y > 1.5$ (when applying a uniform section loss of 1/16 in., more HP sections are affected), thus without the proposed revision, it is possible for a pile having noncompact flanges to have a capacity greater than a pile with compact flanges. This is mechanically inconsistent.

The following are proposed revisions to Tables 6.15.3.2P-1 and -2 reflecting all proposed recommendations.

Table 6.15.3.2P-1 – H-Pile Properties, Factored Axial and Flexural Resistances with Full Pile Section. F_y = 50 ksi

Section	Depth, D	Area, A_g	I_x	I_y	P_{rSTR}			Factored Combined Axial and Flexural Resistance		
					severe	good	soluble	P_r	M_{nx}	M_{ny}[1]
					$0.33A_gF_y$	$0.40A_gF_y$	A_g(9 ksi)	$0.46A_gF_y$	A6.12.2.2.1	
	(in.)	(in^2)	(in^4)	(in^4)	(kips)	(kips)	(kips)	(kips)	(kip-ft)	(kip-ft)
14x117	14.21	34.4	1220	443	568	688	310	791	806	371
14x102	14.01	30	1050	380	495	600	270	690	671	307
14x89	13.83	26.1	904	326	431	522	235	600	550	252
14x73	13.61	21.4	729	261	353	428	193	492	404	186
12x84	12.28	24.6	650	213	406	492	221	566	500	216
12x74	12.13	21.8	569	186	360	436	196	501	424	185
12x63	11.94	18.4	472	153	304	368	166	423	335	145
12x53	11.78	15.5	393	127	256	310	140	357	259	112
10x57	9.99	16.8	294	101	277	336	151	386	277	123

[1] M_{ny} is based on Recommendation E

Table 6.15.3.2P-2 – H-Pile Properties, Factored Axial and Flexural Resistance with 1/16" Section Loss. F_y = 50 ksi

Section	Depth, D	Area, A_g^*	I_x	I_y	P_{rSTR}			Factored Combined Axial and Flexural Resistance		
					severe	good	soluble	P_r	M_{nx}	M_{ny}[1]
					$0.33A_g^*F_y$	$0.40A_g^*F_y$	A_g^*(9 ksi)	$0.46A_g^*F_y$	A6.12.2.2.1	
	(in.)	(in^2)	(in^4)	(in^4)	(kips)	(kips)	(kips)	(kips)	(kip-ft)	(kip-ft)
14x117	14.09	28.73	1019	365	474	575	259	661	636	292
14x102	13.89	24.39	853	305	402	488	220	561	501	231
14x89	13.71	20.51	708	253	338	410	185	472	384	178
14x73	13.49	15.83	537	192	261	317	142	364	244	114
12x84	12.16	19.81	521	168	327	396	178	456	377	163
12x74	12.01	17.02	443	143	281	340	153	391	304	132
12x63	11.82	13.66	349	112	225	273	123	314	215	94
12x53	11.66	10.81	272	87.5	178	216	97	249	142	63
10x57	9.87	12.84	224	75.6	212	257	116	295	195	87

[1] M_{ny} is based on Recommendation E

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Appendix A - test parameters of the 52 H-pile tests reported in Pub 15A

#	H-pile Section	General Soil Conditions	Material at Pile Tip	Ultimate Capacity (tons)	Stress Corresponding to Ultimate Capacity (ksi)	Gross settlement at ultimate capacity (in.)
1	10 x 42	Fine-grained and Mixed strata	Decomposed mica-schist	172	27.7	1.05
2	14 x 89	Mixed-strata	Decomposed Mica-Schist	240	18.4	0.90
3	10 x 42	Mixed-strata	Stiff Sandy Clay	>180	>29.0	-
4	14 x 73	Mixed-strata	Decomposed Mica-Schist	>190	>17.8	-
5	14 x 73	Mixed-strata	Dense fine Sand	>190	>17.8	-
6	14 x 73	Mixed-strata	Dense fine Sand	>150	>14.0	-
7	14 x 73	Mixed-strata	Dense fine Silty Sand	>190	>17.8	-
8	14 x 73	Fine-grained	Boring did not extend to tip	>190	>17.8	-
9	14 x 73	Mixed-strata	Sandy clay	>190	>17.8	-
10	10 x 42	No soil information available		>200	>32.3	-
11	14 x 89	Mixed-strata	Decomposed Mica-Schist	241	18.5	0.98
12	10 x 57	Mixed-strata	Shale W/Sandy Laminae	145	17.4	0.60
13	12 x 74	Mixed-strata	Shale W/Sandy Laminae	256	23.5	0.78
14	10 x 42	Mixed-strata	Shale W/Sandy Laminae	144	23.2	0.72
15	12 x 74	Mixed-strata	Shale W/Sandy Laminae	238	21.8	0.73
16	10 x 57	Mixed-strata	Shale W/Sandy Laminae	146	17.5	0.62
17	12 x 74	Coarse-grained	Soft Shale	291	26.7	0.90
18	12 x 74	Coarse-grained	Soft Shale	262	24.0	0.88
19	10 x 57	Fine-grained	Very soft Shale (Redbeds)	79	9.5	0.35
20	12 x 74	Fine-grained	Very soft Shale (Redbeds)	97	8.9	0.36
21	10 x 42	Fine-grained	Very soft Shale (Redbeds)	106	17.1	0.47
22	12 x 74	Fine-grained	Very soft Shale (Redbeds)	156	14.3	0.43
23	10 x 57	Fine-grained	Very soft Shale (Redbeds)	168	20.1	0.42
24	10 x 57	Coarse-grained	Soft Shale W/Clay Seams	85	10.2	0.44
25	12 x 74	Coarse-grained	Soft Shale W/Clay Seams	89	8.2	0.44
26	10 x 42	Coarse-grained	Soft Shale W/Clay Seams	147	23.7	0.58
27	12 x 74	Coarse-grained	Soft Shale W/Clay Seams	122	11.2	0.45
28	10 x 57	Coarse-grained	Soft Shale W/Clay Seams	153	18.3	0.52
29	10 x 57	Coarse-grained	Soft Shale W/Clay Seams	181	21.7	0.55
30	12 x 74	Coarse-grained	Soft Shale W/Clay Seams	240	22.0	0.60
31	12 x 53	Mixed-strata	Stiff Silty Clay	120	15.5	0.78
32	12 x 53	Mixed-strata	Stiff Silty Clay	126	16.3	0.80
33	12 x 53	Mixed-strata	Stiff Silty Sand	>145	>18.7	-
34	12 x 53	Mixed-strata	Stiff Silty Clay	111	14.3	0.73
35	12 x 53	Mixed-strata	Stiff Silty Clay	105	13.5	0.68
36	12 x 53	Mixed-strata	Stiff Silty Clay	120	15.5	0.74
37	12 x 53	Mixed-strata	Stiff Silty Clay	112	14.5	0.78
38	12 x 53	Mixed-strata	Stiff Silty Clay	85	11.0	0.63

#	H-pile Section	General Soil Conditions	Material at Pile Tip	Ultimate Capacity (tons)	Stress Corresponding to Ultimate Capacity (ksi)	Gross settlement at ultimate capacity (in.)
39	12 x 53	Mixed-strata	Stiff Silty Clay	107	13.8	0.69
40	12 x 53	Mixed-strata	Compact Clay	>145	>18.7	-
41	12 x 53	Mixed-strata	Stiff Silty Clay	91	11.7	0.63
42	12 x 53	Mixed-strata	Stiff Silty Clay	61	7.9	0.51
43	10 x 42	Mixed-strata	Stiff Silty Clay	39	6.3	0.45
44	10 x 42	Mixed-strata	Stiff Silty Clay	93	15.0	0.83
45	10 x 42	Mixed-strata	Stiff Silty Clay	>110	>17.7	-
46	12 x 53	Mixed-strata	Stiff Silty Clay	70	9.0	0.52
47	12 x 53	Mixed-strata	Stiff Silty Clay	76	9.8	0.58
48	12 x 53	Mixed-strata	Stiff Clay	39	5.0	0.40
49	12 x 53	Mixed-strata	Stiff Silty Clay	75	9.7	0.54
50	12 x 53	Mixed-strata	Compact Clay	>140	>18.1	-
51	12 x 53	Mixed-strata	Stiff Clay	140	18.1	0.70
52	14 x 89	Mixed-strata	Silty Shale	300	37.3	0.70

Appendix B – Illustrative examples of reliability calculation

The PennDOT risk management policy as outlined in DM-4, is based on local conditions, experience and judgment. The following illustrative statistical analysis does not reflect the true construction and geologic conditions within Pennsylvania.

The following are illustrative examples of the effects of using reduced material resistance factors as prescribed in DM-4. The calculations, admittedly, use simplifications since appropriate input data is not available. The examples are intended to illustrate trends rather than specific values. Detailed derivations of the LRFD basis for reliability, material resistance factors and load factors are provided in *Load and Resistance Factor Design (LRFD) for Highway Bridge Substructures* (NHI 2001).

As a rule of thumb, a [geotechnical] reliability coefficient of β = 2.5 corresponds to an appropriate reliability for a 50-100 year structure life, while β = 3.5 corresponds to 200-500 years (NHI 2001 and Reese and O'Neil 1988). For example, β = 3.5 is used in conventional seismic design in which the return period for the design earthquake is 472 years (10% probability of exceedance in 50 years).

Reliability is based on the probability of failure; that is the probability that the effects of load (Q) exceed the available resistance (R); in general: $P[R>Q]$. For lognormal distribution of R and Q and β values between 2 and 6 (NHI 2001):

$$P[R>Q] \approx 460e^{-4.3\beta} \qquad \text{(B1)}$$

Where β is the reliability index described as $\beta = \frac{mean\ (R-Q)}{standard\ deviation\ (R-Q)} = \frac{\overline{R-Q}}{\sigma_{R-Q}}$

For most structural engineering applications, a value $\beta = 3.5$ is reasonably assumed equating to a probability of failure of $P[R>Q] = 0.000134$ or 1 in 7471. Due to the acknowledged conservative nature of geotechnical assumptions, a lower index of $\beta = 2.5$ (P[R>Q] = 0.00986 or 1 in 101) is sometimes assumed when R represents a geotechnical capacity. Regardless of the choice of β, the following calculations remain valid; only the values change.

From the statistical distribution of R obtained from material tests, we define the mean and coefficient of variation of the material capacity, $\bar{R}$ and COV_R. The resulting material resistance factor is then approximated as:

$$\phi \approx \frac{\bar{R}}{R_n} e^{-0.55\beta COV_R} \qquad \text{(B2)}$$

In which R_n is the nominal capacity of the material (F_y, say).

For the sake of illustration, we assume values of $\bar{R}$ and COV_R for structural steel to be $1.05R_n$ and 0.08. From these values, with no other factors considered and a target $\beta = 3.5$, we obtain the typical value of ϕ = 0.9 for steel:

$$\phi = \frac{1.05R_n}{R_n} e^{-0.55(3.5)(0.08)} = 0.90 \qquad \text{(B2a)}$$

In order to assess what is ***implied*** by the AASHTO factors for piles, we will consider the case of axial capacity in severe conditions ($\phi = 0.5$) and a target value of $\beta = 3.5$. The pile capacity itself ($\bar{R}$) does not change therefore the effectively implied variation accounting for other factors identified in §6.15.2 may be found to be $COV_R = 0.385$. That is:

$$\phi = \frac{1.05R_n}{R_n} e^{-0.55(3.5)(0.385)} = 0.50 \qquad \text{(B2b)}$$

Using $COV_R = 0.385$, we can do one of two things using the PennDOT value $\phi = 0.33$:

1. Calculate the implied COV_R for PennDOT practice leaving $\beta = 3.5$: $COV_{R,PA} = 0.60$; that is:

$$\phi = \frac{1.05R_n}{R_n} e^{-0.55(3.5)(0.60)} = 0.33 \quad \text{(B2c)}$$

This is an increase of 56% over AASHTO-assumed ***variation***.

2. Calculate the implied increase in reliability index leaving $COV_R = 0.385$: $\beta = 5.46$; that is:

$$\phi = \frac{1.05R_n}{R_n} e^{-0.55(5.46)(0.385)} = 0.33 \quad \text{(B2d)}$$

This implies of probability of failure of $P[R>Q] \approx 3 \times 10^{-8}$ or 1 in 34 million!

Although reality lies somewhere between these cases having some combination of β x $COV_R = 2.1$, neither result, in the opinion of the Research Team, may be rationalized.

Repeating the same calculations with $\beta = 2.5$ leads to:

Equation B2b: implied $COV_R = 0.52$

Equation B2c: implied $COV_{R,PA} = 0.84$; a 61% increase

Equation B2d: implied $\beta = 4.04$ ($P[R>Q] = 0.000013$ or 1 in 76,000)

This Appendix demonstrates that the linear reduction of the material resistance factor implies a highly nonlinear effect on resulting reliability. From the perspective of risk and reliability, it is beyond the scope of this project to propose a new value of ϕ_c. Thus the recommendation of $\phi_c = 0.33$ is simply an acceptance of PennDOT's perception of risk.

Appendix C – Summary of State H-pile Provisions Available Online

(these may not be current)

State	Permit F_y = 50 ksi in determining structural capacity?	Specify F_y = 50 ksi material	Notes
Alabama	yes	yes	allowable stress = 13 ksi based on 1.45 x 0.25F_y which implies F_y = 36 ksi
Alaska	yes	yes	cites ASTM A709 Grade 50T3
Arizona	yes	yes	cites ASTM A719 Grade 50 although silent on design capacity "H-piles are generally classified as friction piles"
Arkansas	no	permitted, see note	"Steel H-piles shall conform to AASHTO M270 Grade 36 or greater"
California	yes	yes	0.25F_y limit on *reduced* section (1/16" section loss all around); 0.9F_y driving stress on gross section.
Colorado	unknown	unknown	"All projects with piling shall require a minimum 26,000ft-lb hammer; therefore, no piling should be used with a section area less than an HP12X53." In reference to superstructure components: "ASTM A36 should be used for member and components where higher yield strength steel would not appreciably reduce the required section."
Connecticut	yes	yes	cites ASTM A719 Grade 50 although silent on design capacity "Piles end bearing on bedrock or dense hardpan typically are steel H-piles."
Delaware	yes, see note	yes	DelDOT Bridge Design Manual implies F_y = 36 ksi but is reported to be out of date. (see App. F)
Florida	no	no	"Miscellaneous… shapes… shall conform to ASTM A709, Grade 36. … Use ASTM A 709 HPS 50W or HPS 70W for steel substructure elements <u>excluding piles</u>."
Georgia	yes, see note	yes	"use 36 ksi steel H-pile in design unless the BFI calls for 50 ksi piles"; design explicitly refers to AASHTO 10.7
Hawaii	no	no	ASTM A36 is cited
Idaho	yes, see note	permitted, see note	0.75F_y driving stress limit; "For economy 36 ksi steel should be specified in most cases because the large ultimate loads that are possible with 50 ksi steel are very difficult to achieve and verify with standard pile driving equipment"
Illinois	yes, see note	yes	Maximum nominal structural resistance is limited to 0.54F_yA_s
Indiana	yes, see note	yes	"minimum" F_y = 50 ksi specified. A "maximum nominal soil resistance", defined as 27.5 ksi = 0.55A_sF_y (F_y = 50 ksi), is prescribed in addition to structural resistance and bearing capacity.
Iowa	yes, see note	yes	Maximum nominal structural resistance is limited to 0.725A_sF_y.
Kansas	yes	yes	"Unless specified otherwise, provide steel that complies with ASTM A 709 Grade 50 or ASTM A 572 Grade 50"
Kentucky	yes	yes	0.25F_y limit
Louisiana	no	no	0.25F_y to 0.33F_y limit; F_y = 36 ksi is implied by values given in Design Manual; AASHTO M270, Grade 36 is cited. Additionally, "pile loads shall be within 75% of the structural capacity of the piles"
Maine	yes, see note	yes	"minimum" F_y = 50 ksi "Experience in using 50 ksi steel for H-Pile foundations has shown that the factored axial geotechnical resistance frequently governs design. This is particularly apparent for end-bearing piles on poor-quality and/or soft bedrock and for friction piles."

State	Permit F_y = 50 ksi in determining structural capacity?	Specify F_y = 50 ksi material	Notes
Maryland	data not found		
Massachusetts	yes	yes	refers exclusively to AASHTO LRFD
Michigan	no	yes	allowable stress = 9 ksi; AASHTO M270 Grade 50 required.
Minnesota	yes	yes	ASTM A 572, Grade 50 is cited
Mississippi	yes	yes	"The minimum structural steel grade shall be 50 ksi. No structural bridge components shall be specified as grade 36 ksi."
Missouri	yes, see note	permitted, see note	"Steel piling shall be ASTM A709 (Grade 36) unless structural analysis or drivability analysis requires ASTM A709 (Grade 50) steel."; otherwise refers exclusively to AASHTO
Montana	no	no	"Furnish structural steel piles meeting requirements of ASTM A36"
Nebraska	no	no	""H" pile and other pile shall meet the requirements in ASTM A6."
Nevada	yes	yes	refers to AASHTO LRFD
New Hampshire	no, see note	no, see note	"Steel H-piles should be based on a yield strength Fy of 36,000 psi (250 MPa). If a yield strength of 50,000 psi (345 MPa) is needed a special provision is required."
New Jersey	yes, see note	yes	"Material for steel H-piles shall conform to AAASHTO M270 Grade 50"
New Mexico	yes	yes	ASTM A 572, Grade 50 is cited
New York	yes	yes	(see App. F)
North Carolina	no	no	allowable stress ≈ 9 ksi
North Dakota	yes	yes	$0.25F_y$ limit; effectively: ASD with 12.5 ksi
Ohio	yes	yes	(see App. F)
Oklahoma	yes	yes	cites AASHTO M270 (ASTM A572 Grade 50)
Oregon	yes, see note	permitted, see note	"Structural steel for steel piling, metal sign structures and other incidental structures should conform to ASTM A36, ASTM A572 or ASTM A588." Based on design tables provided, maximum nominal structural resistance is limited to approximately $0.6A_sF_y$
Pennsylvania	yes	yes	
Rhode Island	yes	yes	refers exclusively to AASHTO LRFD
South Carolina	uncertain	uncertain	"Grade 36 is typically used for steel piles. Grade 36 steel is becoming less used and thus less available at time. There is a little or no cost difference between Grade 50 and Grade 36."
South Dakota	no	no	cites ASTM A36
Tennessee	no	no	"Structural steel piles shall be rolled steel sections and shall be meeting the requirements of ASTM A 36."
Texas	no, see note	yes	"For H-piling, furnish steel that meets ASTM A572 Grade 50 or ASTM A588." Texas specifies maximum bearing capacity by pile depth (i.e.: HP10, HP12) without citing pile weight. Values are low.
Utah	no	no	cites ASTM A36
Vermont	yes	yes	refers exclusively to AASHTO LRFD
Virginia	yes, see note	yes	cites ASTM A36, A572 or A992; factors same as AASHTO LRFD
Washington	no	no	allowable stress = 9 ksi
West Virginia	yes, see note	permitted, see note	F_y = 36 ksi specified but F_y = 50 ksi is permitted
Wisconsin	no	yes	required F_y = 50 ksi although F_y = 36 ksi is used for design and F_y = 50 ksi is permitted for driveability analysis
Wyoming	yes	permitted, see note	cites ASTM A709 Grade 36 or 50

Appendix D – Regional Survey Instrument

University of Pittsburgh Letterhead

1 August 2014 ***electronic transmission via PennDOT***

RE: USE OF 50 ksi STEEL AS DESIGN BASIS FOR H-PILES – STATE OF PRACTICE SURVEY

The Structural Engineering and Mechanics Group in the Department of Civil Engineering at the University of Pittsburgh, funded by the Pennsylvania Department of Transportation (PennDOT), is conducting a study aimed at establishing the state of practice for the use of steel H-piles (HP sections) having a yield strength of 50 ksi, rather than 36 ksi.

Your assistance is requested in completing the attached survey. This survey has been approved by PennDOT and the University of Pittsburgh Institutional Review Board.

The survey responses will be tabulated and all identifying remarks stricken prior to any publication of results. In this way, presentation of the responses will be anonymous to all but myself, the graduate student assisting with this project and the project oversight committee. Nonetheless, we ask that you provide your contact information so that we may "check off" your organization's response and provide you a copy of the survey results.

We ask that you complete the survey, preferably electronically (the survey is provided in MSWord and Adobe PDF format for your convenience), and return it before **September 15, 2014** to:

Dr. Kent A. Harries, P.Eng.
kharries@pitt.edu
fax: 412.624.0135

Thank you for your assistance with this survey. Please feel free to contact me at any time.

Sincerely,

Signature

Kent A. Harries, Ph.D., P.Eng.
Associate Professor

State-of-Practice Survey
STEEL H-PILES

Survey completed by:

Name: ______________________ Title: ______________________

Jurisdiction: ______________________ email: ______________________

Address: ______________________ Telephone: ______________________

____

PLEASE RETURN COMPLETED SURVEY <u>BEFORE September 15, 2014</u> TO:

Kent A. Harries
University of Pittsburgh
Civil and Environmental Engineering
742 Benedum Hall
Pittsburgh PA 15261
fax: (412) 624-0135
kharries@pitt.edu

Introduction

In responding to this survey, please consider current practice (since 2012) in your jurisdiction only.

The survey asks about H-piles ***designed*** using a nominal yield strength of F_y = 50 ksi. We recognize that the 'preferred material specification' for H-pile shapes (designated HP) is ASTM A572 Grade 50 High Strength-Low Alloy Steel and that the actual yield strength will exceed 50 ksi. Nonetheless, in many cases, permitted capacity for ***design*** is 36 ksi.

The purpose of this research study is to assess the current state of practice associated with the use of steel H-piles having yield strength of 50 ksi. If you are willing to participate, you will be asked to provide your professional contact information and direct responses to the survey questioned asked. There are no foreseeable risks associated with this project, nor are there any direct benefits to you. Your participation is voluntary. All surveys will be kept in confidence and responses will be stripped of remarks identifying an individual, organization or jurisdiction. This study is being conducted by Dr. Kent A. Harries, who can be reached at 412.624.9873 or kharries@pitt.edu, if you have any questions.

1. Does your jurisdiction permit the use of F_y = 50 ksi for the ***design*** of steel H-piles?

Yes ☐ No ☐

If **No** to Question 1, please respond to Question 2 and your survey is complete.

If **Yes** to Question 1, please go to Question 3

2. Is there any reason for ***not*** permitting the use of F_y = 50 ksi for the ***design*** of steel H-piles?

Thank you for your time in completing this survey, please return Survey to kharries@pitt.edu

3. When did your jurisdiction adopt the use of F_y = 50 ksi for the ***design*** of steel H-piles?

4. Is F_y = 50 ksi used in ***design*** of steel H-piles regularly?

Yes ☐ No ☐

If **No** to Question 4, please continue to Question 5

If **Yes** to Question 4, please go to Question 6

5. Under what conditions is F_y = 50 ksi used for the ***design*** of steel H-piles?

6. Approximately how many H-pile projects, ***designed*** using F_y = 50 ksi for the structural capacity of the piles have been installed in your jurisdiction?

7. Approximately what proportion of piles installed since 2010 were ***designed*** with F_y = 50 ksi?

8. What are the most common 50 ksi H-pile sections used?

9. What is/was the most common 36 ksi H-pile sections used?

10. Does your jurisdiction realise any advantages to using F_y = 50 ksi in the ***design*** of steel H-piles (e.g.: fewer piles, smaller piles, more efficient driving, etc.)?

11. Has your jurisdiction had any difficulties or issues arising in the ***design*** of steel H-piles having F_y = 50 ksi? (e.g.: geotechnical capacity, increased settlement, etc.)

12. Has your jurisdiction had any difficulties or issues arising in the driving of steel H-piles ***designed*** with F_y = 50 ksi?

13. How long have H-piles ***designed*** with F_y = 50 ksi been in-service, and has your jurisdiction had any difficulties or issues arising in the overall performance of these piles?

14. Has the adoption of steel H-piles having F_y = 50 ksi required changes to related standards (pile cap design or geotechnical limitations, may be examples)?

15. Does your jurisdiction have any limitations on the use F_y = 50 ksi for the ***design*** of steel H-piles?

16. How is the compressive structural capacity of H-piles ***designed*** with F_y = 50 ksi determined? (For example, $P_o = A_p$ (50 ksi), or $P_o = 0.25A_p$(50 ksi), etc.)

17. Does your jurisdiction perform WEAP analyses in order to formally approve pile hammers as submitted by contractors?

18. Is PDA and/or CAPWAP required for test piles and/or production piles?

19. Does your jurisdiction consider pile settlement in "weak rock" as limiting 50 ksi H-pile capacity? If so, how do you define "weak rock".

Thank you for your time in completing this survey, please return Survey to kharries@pitt.edu

Individual Survey Responses

Question		PennDOT	Ohio DOT	New York State DOT	Delaware DOT
1	Does your jurisdiction permit the use of F_y = 50 ksi for the ***design*** of steel H-piles?	yes	yes	yes	yes
3	When did your jurisdiction adopt the use of F_y = 50 ksi for the ***design*** of steel H-piles?	11.27.2013	2010	approximately 2004	We are not aware of when the department began allowing use of 50 ksi steel piles, or if the use of 50 ksi steel H-Piles was ever forbidden.
4	Is F_y = 50 ksi used in ***design*** of steel H-piles regularly?	yes	yes	yes	yes
5	Under what conditions is F_y = 50 ksi used for the ***design*** of steel H-piles?	[did not respond]	[did not respond]	[did not respond]	[did not respond]
6	Approximately how many H-pile projects, ***designed*** using F_y = 50 ksi for the structural capacity of the piles have been installed in your jurisdiction?	Approximately 10 projects out of 80 in 2014	over 100	From May 2012 to July 2014 there were 20 bridges had H-piles with 50 ksi Yield installed.	Since 2005, I would say that approximately 10 bridge locations have incorporate use of F_y = 50 ksi steel piles.
7	Approximately what proportion of piles installed since 2010 were ***designed*** with F_y = 50 ksi?	Approximately 10 out of 605	100%	From May 2012 to July 2014 the proportion was 36%.	For steel piles, 100% of the piles to our knowledge were designed with F_y = 50 ksi.
8	What are the most common 50 ksi H-pile sections used?	HP12x74	HP10x42, HP12x53 and HP14x73	HP10x42, HP10x57,HP 12x53 and HP12x84.	HP10 and HP12
9	What is/was the most common 36 ksi H-pile sections used?	HP12x74	none	HP10x42, HP10x57 and HP12x53.	[responded] na
10	Does your jurisdiction realise any advantages to using F_y = 50 ksi in the ***design*** of steel H-piles?	Yes, it is estimated that the Department will save approximately $1.0m annually in steel cost savings.	Fewer and smaller sized piles.	Yes, more efficient driving.	Yes, in most instances where we incorporate use of H-Piles, we tend to drive to the bedrock. This is where we realize advantages of using a F_y = 50ksi steel H-Piles, especially because the rock bed tends to consist of gneiss rock. I would say that we see more benefits to use of 50 ksi steel piles for driving conditions rather than to resist design loads (Strength I, etc.).
11	Has your jurisdiction had any difficulties or issues arising in the ***design*** of steel H-piles having F_y = 50 ksi?	no	None. Commonly available pile driving hammers in Ohio are the limiting factor as far as the geotechnical resistance is concerned.	no	We have not encountered any issues specifically because we used F_y = 50 ksi steel H-piles.
12	Has your jurisdiction had any difficulties or issues arising in the driving of steel H-piles ***designed*** with F_y = 50 ksi?	no	no	no	Not to our knowledge.
13	How long have H-piles ***designed*** with F_y = 50 ksi been in-service, and has your jurisdiction had any difficulties or issues arising in the overall performance of these piles?	In-service less than 1-year, no issues thus far.	since 2010, none	Approximately 10 years and no we have not had any difficulties or issues in performance.	To our knowledge, we have not used 36 ksi piles for a long time; I would say that we have been using 50 ksi piles for at least the past 20 years. And also to our knowledge, we have never

Question		PennDOT	Ohio DOT	New York State DOT	Delaware DOT
					had any issues with the overall performance of the piles themselves, and any issues we have would be as result to geotechnical conditions (boulders, etc.). But we have mostly countered this issue by adding a contingency predrilling item for driving.
14	Has the adoption of steel H-piles having F_y = 50 ksi required changes to related standards?	Not necessarily "required", but made all splice welds full-penetration or required the use of a "splicer"	[did not respond]	no	Our design procedure remains the same for other structural elements regardless of what F_y -value we use for our H-Piles.
15	Does your jurisdiction have any limitations on the use F_y = 50 ksi for the ***design*** of steel H-piles?	no	no	no	no
16	How is the compressive structural capacity of H-piles ***designed*** with F_y = 50 ksi determined?	$P_r = \varphi_c\, 0.66A_p(50\text{ ksi})$ $\varphi_c = 0.50$ [see Task 1 report]	Factored resistance is the only consideration i.e. severe vs good driving conditions [i.e., AASHTO]	The structural strength limit is determined using the AASHTO LRFD Code.	The design structural capacity of the pile (Strength I) would be calculated in accordance to Chapter 6 of the most current AASHTO LRFD Bridge Specifications.
17	Does your jurisdiction perform WEAP analyses in order to formally approve pile hammers as submitted by contractors?	yes	yes	yes	We do not perform our own WEAP analysis. But it is required in our specifications that it is the contractor's responsibility to perform WEAP analysis and submit the report for the department's approval.
18	Is PDA and/or CAPWAP required for test piles and/or production piles?	No, but generally PDA is only performed for test piles.	yes	Only on a few projects a year.	We require PDA and CAPWAP for all test piles. These test piles often are also used as a production pile.
19	Does your jurisdiction consider pile settlement in "weak rock" as limiting 50 ksi H-pile capacity? If so, how do you define "weak rock".	*C10.7.3.2.2* The following shall supplement AC10.7.3.2.2. Soft and weak rock may be considered rock with uniaxial compressive strength less than 500 tsf.	OhioDOT has a refusal criteria for piles driven to bedrock which is 20 blows/inch after several inches penetration into weak rock or 20 blows for less than an inch of penetration after contacting sound rock. Settlement is not a consideration when the piles are driven to the refusal criteria unless relaxation is anticipated.	no	We will evaluate the bearing capacity of the rock using RQD test results and the type of rock encountered in accordance to Chapter 10 in the AASHTO LRFD Bridge Specifications and Chapter 4 in the Standard Specifications for Highway Bridges. As for the settlement, we tend not to consider it due to heavy presence of gneiss rock, which is classified as being a "very hard, sound rock". We would likely classify a rock layer as being "weak" regardless of rock type if the RQD values are low. It is possible that we do not consider settlements due to our tendency to predrill through the weak layer(s) until we reach the layer that contains a minimum of 70% RQD.

APPENDIX E – GRLWEAP RESULTS

Table E1 Parametric study results.

ID		HP	As	Stress at refusal	Capacity at refusal	Length	Friction	Toe Damping	Toe Quake	Skin Damping	Smallest Hammer	Driving stress at refusal	Cal. stroke	Energy
BASE			in^2	$1/A_sFy$	kips	ft	%					ksi	ft	k-ft
1	1a	10x57	16.8	1.00	840	20	0.20	0.10	0.05	0.05	ICE I-30v2	66	8.86	23.80
2	1b	10x57	16.8	0.66	554	20	0.20	0.10	0.05	0.05	ICE I-12v2	42	9.05	11.40
3	1c	10x57	16.8	0.50	420	20	0.20	0.10	0.05	0.05	ICE I-12v2	30	6.70	7.20
4	2a	10x57	16.8	0.70	588	20	0.20	0.10	0.05	0.05	ICE I-30v2	45	5.95	12.4
5	2b	10x57	16.8	0.70	591	20	0.20	0.10	0.05	0.05	ICE I-12v2	45	10.00	13.00
6	3	10x57	16.8	0.66	554	20	0.20	0.10	0.05	0.05	ICE I-30v2	43	5.59	11.10
7	4	10x57	16.8	0.45	375	20	0.20	0.10	0.05	0.05	ICE I-30v2	25	4.20	5.70
8	1a	10x57	16.8	1.00	840	20	0.30	0.10	0.05	0.05	ICE I-30v2	64	8.93	24.10
9	1b	10x57	16.8	0.66	554	20	0.30	0.10	0.05	0.05	ICE I-12v2	40	9.10	11.60
10	1c	10x57	16.8	0.50	420	20	0.30	0.10	0.05	0.05	ICE I-12v2	29	6.70	7.20
11	2a	10x57	16.8	0.70	588	20	0.30	0.10	0.05	0.05	ICE I-30v2	45	5.95	11.80
12	2b	10x57	16.8	0.71	600	20	0.30	0.10	0.05	0.05	ICE I-12v2	45	10.30	13.80
13	3	10x57	16.8	0.66	554	20	0.30	0.10	0.05	0.05	ICE I-30v2	41	5.60	11.10
14	4	10x57	16.8	0.44	372	20	0.30	0.10	0.05	0.05	ICE I-30v2	25	4.20	5.70
15	1a	10x57	16.8	1.00	840	50	0.20	0.10	0.05	0.05	ICE I-30v2	59.7	11.40	42.90
16	1b	10x57	16.8	0.66	554	50	0.20	0.10	0.05	0.05	PilcoD1942	38.0	8.30	18.80
17	1c	10x57	16.8	0.50	420	50	0.20	0.10	0.05	0.05	ICE I-12v2	29.1	7.40	10.60
18	2a	10x57	16.8	0.74	618	50	0.20	0.10	0.05	0.05	ICE I-30v2	45.0	7.90	25.10
19	2b	10x57	16.8	0.75	627	50	0.20	0.10	0.05	0.05	PilcoD1942	44.6	10.10	25.00
20	3	10x57	16.8	0.66	554	50	0.20	0.10	0.05	0.05	ICE I-30v2	39.2	6.70	19.20
21	4	10x57	16.8	0.43	363	50	0.20	0.10	0.05	0.05	ICE I-30v2	24.7	4.50	8.50
22	1a	10x57	16.8	1.00	840	50	0.30	0.10	0.05	0.05	ICE I-36v2	60.3	9.77	43.50
23	1b	10x57	16.8	0.66	554	50	0.30	0.10	0.05	0.05	PilcoD1942	36.5	8.40	19.50
24	1c	10x57	16.8	0.50	420	50	0.30	0.10	0.05	0.05	ICE I-12v2	26.3	7.51	10.80
25	2a	10x57	16.8	0.76	642	50	0.30	0.10	0.05	0.05	ICE I-36v2	45.0	7.15	26.70
26	2b	10x57	16.8	0.76	635	50	0.30	0.10	0.05	0.05	PilcoD1942	43.4	10.50	26.60
27	3	10x57	16.8	0.66	554	50	0.30	0.10	0.05	0.05	ICE I-36v2	37.6	6.025	19.10

ID		HP	As	Stress at refusal	Capacity at refusal	Length	Friction	Toe Damping	Toe Quake	Skin Damping	Smallest Hammer	Driving stress at refusal	Cal. stroke	Energy
BASE			in^2	$1/A_sFy$	kips	ft	%					ksi	ft	k-ft
28	4	10x57	16.8	0.46	388	50	0.30	0.10	0.05	0.05	ICE I-36v2	25	4.50	9.60
29	1a	10x57	16.8	1.00	840	80	0.20	0.10	0.05	0.05	ICE I-36v2	59.2	10.70	58.00
30	1b	10x57	16.8	0.66	554	80	0.20	0.10	0.05	0.05	PilcoD1942	37.0	8.50	22.90
31	1c	10x57	16.8	0.50	420	80	0.20	0.10	0.05	0.05	ICE I-12v2	29.0	7.50	12.00
32	2a	10x57	16.8	0.80	672	80	0.20	0.10	0.05	0.05	ICE I-36v2	46.2	8.01	37.00
33	2b	10x57	16.8	0.75	632	80	0.20	0.10	0.05	0.05	PilcoD1942	43.3	10.60	31.20
34	3	10x57	16.8	0.66	554	80	0.20	0.10	0.05	0.05	ICE I-36v2	37.3	6.41	25.70
35	4	10x57	16.8	0.46	389	80	0.20	0.10	0.05	0.05	ICE I-36v2	24.8	4.68	13.10
36	1a	10x57	16.8	1.00	840	80	0.30	0.10	0.05	0.05	ICE I-36v2	56.6	11.05	61.30
37	1b	10x57	16.8	0.66	554	80	0.30	0.10	0.05	0.05	PilcoD1942	32.9	8.65	23.60
38	1c	10x57	16.8	0.50	420	80	0.30	0.10	0.05	0.05	ICE I-12v2	26.0	7.64	12.20
39	2a	10x57	16.8	0.80	674	80	0.30	0.10	0.05	0.05	ICE I-36v2	45.0	8.25	39.90
40	2b	10x57	16.8	0.74	619	80	0.30	0.10	0.05	0.05	PilcoD1942	37.8	10.60	31.50
41	3	10x57	16.8	0.66	554	80	0.30	0.10	0.05	0.05	ICE I-36v2	35.4	6.49	26.70
42	4	10x57	16.8	0.49	415	80	0.30	0.10	0.05	0.05	ICE I-36v2	26	5.00	16.00
43	1a	12x74	21.8	1.00	1090	20	0.20	0.10	0.05	0.05	ICE I-30v2	65	10.95	30.80
44	1b	12x74	21.8	0.66	719	20	0.20	0.10	0.05	0.05	ICE I-30v2	42	6.68	14.70
45	1c	12x74	21.8	0.50	545	20	0.20	0.10	0.05	0.05	ICE I-12v2	31	8.20	9.50
46	2a	12x74	21.8	0.70	767	20	0.20	0.10	0.05	0.05	ICE I-30v2	45	7.20	16.60
47	2b	12x74	21.8	0.70	767	20	0.20	0.10	0.05	0.05	ICE I-30v2	45	7.20	16.60
48	3	12x74	21.8	0.66	719	20	0.20	0.10	0.05	0.05	ICE I-30v2	42	6.68	14.70
49	4	12x74	21.8	0.42	456	20	0.20	0.10	0.05	0.05	ICE I-30v2	25	4.60	7.10
50	1a	12x74	21.8	1.00	1090	20	0.30	0.10	0.05	0.05	ICE I-30v2	63	11.05	31.50
51	1b	12x74	21.8	0.66	719	20	0.30	0.10	0.05	0.05	ICE I-30v2	41	6.71	15.00
52	1c	12x74	21.8	0.50	545	20	0.30	0.10	0.05	0.05	ICE I-12v2	28	8.27	9.70
53	2a	12x74	21.8	0.72	784	20	0.30	0.10	0.05	0.05	ICE I-30v2	45	7.50	17.90
54	2b	12x74	21.8	0.72	784	20	0.30	0.10	0.05	0.05	ICE I-30v2	45	7.50	17.90
55	3	12x74	21.8	0.66	719	20	0.30	0.10	0.05	0.05	ICE I-30v2	41	6.70	14.90
56	4	12x74	21.8	0.45	486	20	0.30	0.10	0.05	0.05	ICE I-30v2	26	4.80	7.90
57	1a	12x74	21.8	1.00	1090	50	0.20	0.10	0.05	0.05	ICE I-36v2	62	11.81	52.00

ID		HP	As	Stress at refusal	Capacity at refusal	Length	Friction	Toe Damping	Toe Quake	Skin Damping	Smallest Hammer	Driving stress at refusal	Cal. stroke	Energy
BASE			in^2	$1/A_sFy$	kips	ft	%					ksi	ft	k-ft
58	1b	12x74	21.8	0.66	719	50	0.20	0.10	0.05	0.05	PilcoD1942	39	9.75	23.60
59	1c	12x74	21.8	0.50	545	50	0.20	0.10	0.05	0.05	ICE I-12v2	31	8.66	12.90
60	2a	12x74	21.8	0.73	800	50	0.20	0.10	0.05	0.05	ICE I-36v2	45	8.20	29.90
61	2b	12x74	21.8	0.70	763	50	0.20	0.10	0.05	0.05	PilcoD1942	42	10.60	26.50
62	3	12x74	21.8	0.66	719	50	0.20	0.10	0.05	0.05	ICE I-36v2	39	7.11	23.40
63	4	12x74	21.8	0.43	466	50	0.20	0.10	0.05	0.05	ICE I-36v2	25	4.90	11.30
64	1a	12x74	21.8	1	1090	50	0.30	0.1	0.05	0.05	ICE I-46v2	61.43	10.27	54.9
65	1b	12x74	21.8	0.66	719	50	0.30	0.10	0.05	0.05	PilcoD1942	35	9.85	24.10
66	1c	12x74	21.8	0.50	545	50	0.30	0.10	0.05	0.05	ICE I-12v2	28	8.78	13.10
67	2a	12x74	21.8	0.75	816	50	0.30	0.10	0.05	0.05	ICE I-46v2	45	7.40	33.30
68	2b	12x74	21.8	0.69	751	50	0.30	0.10	0.05	0.05	PilcoD1942	37	10.60	26.60
69	3	12x74	21.8	0.66	719	50	0.30	0.10	0.05	0.05	ICE I-46v2	38	6.30	24.70
70	4	12x74	21.8	0.45	488	50	0.30	0.10	0.05	0.05	ICE I-46v2	25	4.60	12.30
71	1a	12x74	21.8	1	1090	80	0.20	0.1	0.05	0.05	ICE I-46v2	60.7	11.24	73
72	1b	12x74	21.8	0.66	719	80	0.20	0.10	0.05	0.05	PilcoD1942	39	9.80	26.70
73	1c	12x74	21.8	0.50	545	80	0.20	0.10	0.05	0.05	ICE I-12v2	31	8.75	14.00
74	2a	12x74	21.8	0.80	872	80	0.20	0.10	0.05	0.05	ICE I-46v2	45	8.40	47.60
75	2b	12x74	21.8	0.69	756	80	0.20	0.10	0.05	0.05	PilcoD1942	41	10.60	29.70
76	3	12x74	21.8	0.66	719	80	0.20	0.10	0.05	0.05	ICE I-46v2	37	6.70	33.00
77	4	12x74	21.8	0.46	503	80	0.20	0.10	0.05	0.05	ICE I-46v2	25	4.80	16.30
78	1a	12x74	21.8	1.00	1090	80	0.30	0.10	0.05	0.05	ICE I-46v2	55	11.61	77.50
79	1b	12x74	21.8	0.66	719	80	0.30	0.10	0.05	0.05	PilcoD1942	35	10.03	27.50
80	1c	12x74	21.8	0.50	545	80	0.30	0.10	0.05	0.05	ICE I-12v2	28	8.92	14.30
81	2a	12x74	21.8	0.89	972	80	0.30	0.10	0.05	0.05	ICE I-46v2	45	9.00	54.10
82	2b	12x74	21.8	0.67	733	80	0.30	0.10	0.05	0.05	PilcoD1942	36	10.60	29.60
83	3	12x74	21.8	0.66	719	80	0.30	0.10	0.05	0.05	ICE I-46v2	34	6.79	34.00
84	4	12x74	21.8	0.50	541	80	0.30	0.10	0.05	0.05	ICE I-46v2	25	5.20	20.00
85	1a	14x117	34.4	1.00	1720	20	0.20	0.10	0.05	0.05	ICE I-46v2	66	11.45	47.60
86	1b	14x117	34.4	0.66	1135	20	0.20	0.10	0.05	0.05	ICE I-30v2	46	9.75	25.40
87	1c	14x117	34.4	0.50	860	20	0.20	0.10	0.05	0.05	PilcoD1942	32	8.55	15.40

ID		HP	As	Stress at refusal	Capacity at refusal	Length	Friction	Toe Damping	Toe Quake	Skin Damping	Smallest Hammer	Driving stress at refusal	Cal. stroke	Energy
BASE			in^2	$1/A_sFy$	kips	ft	%					ksi	ft	k-ft
88	2a	14x117	34.4	0.70	1204	20	0.20	0.10	0.05	0.05	ICE I-46v2	45	7.46	24.90
89	2b	14x117	34.4	0.64	1106	20	0.20	0.10	0.05	0.05	ICE I-30v2	45	9.50	24.60
90	3	14x117	34.4	0.66	1135	20	0.20	0.10	0.05	0.05	ICE I-46v2	42	7.03	22.60
91	4	14x117	34.4	0.41	699	20	0.20	0.10	0.05	0.05	ICE I-46v2	25	5.00	10.50
92	1a	14x117	34.4	1.00	1720	20	0.30	0.10	0.05	0.05	ICE I-46v2	64	11.60	48.70
93	1b	14x117	34.4	0.66	1135	20	0.30	0.10	0.05	0.05	ICE I-30v2	42	9.81	25.70
94	1c	14x117	34.4	0.50	860	20	0.30	0.10	0.05	0.05	PilcoD1942	30	8.59	15.50
95	2a	14x117	34.4	0.70	1204	20	0.30	0.10	0.05	0.05	ICE I-46v2	45	7.54	25.60
96	2b	14x117	34.4	0.70	1204	20	0.30	0.10	0.05	0.05	ICE I-30v2	45	10.58	28.60
97	3	14x117	34.4	0.66	1135	20	0.30	0.10	0.05	0.05	ICE I-46v2	41	7.10	23.10
98	4	14x117	34.4	0.44	760	20	0.30	0.10	0.05	0.05	ICE I-46v2	25	5.10	12.30
99	1a	14x117	34.4	0.84	1444	50	0.20	0.10	0.05	0.05	ICE I-46v2	52	11.81	62.20
100	1b	14x117	34.4	0.66	1135	50	0.20	0.10	0.05	0.05	ICE I-30v2	45	10.30	33.70
101	1c	14x117	34.4	0.50	860	50	0.20	0.10	0.05	0.05	PilcoD1942	32	8.81	19.30
102	2a	14x117	34.4	0.72	1236	50	0.20	0.10	0.05	0.05	ICE I-46v2	45	9.60	46.40
103	2b	14x117	34.4	0.66	1135	50	0.20	0.10	0.05	0.05	ICE I-30v2	45	10.30	33.70
104	3	14x117	34.4	0.66	1135	50	0.20	0.10	0.05	0.05	ICE I-46v2	41	8.48	38.60
105	4	14x117	34.4	0.41	701	50	0.20	0.10	0.05	0.05	ICE I-46v2	25	5.20	15.90
106	1a	14x117	34.4	0.83	1420	50	0.30	0.10	0.05	0.05	ICE I-46v2	46	11.81	62.30
107	1b	14x117	34.4	0.66	1135	50	0.30	0.10	0.05	0.05	ICE I-30v2	41	10.50	34.60
108	1c	14x117	34.4	0.50	860	50	0.30	0.10	0.05	0.05	PilcoD1942	29	8.95	19.70
109	2a	14x117	34.4	0.81	1394	50	0.30	0.10	0.05	0.05	ICE I-46v2	45	11.40	59.40
110	2b	14x117	34.4	0.72	1232	50	0.30	0.10	0.05	0.05	ICE I-30v2	45	12.00	41.50
111	3	14x117	34.4	0.66	1135	50	0.30	0.10	0.05	0.05	ICE I-46v2	37	8.58	39.20
112	4	14x117	34.4	0.46	795	50	0.30	0.10	0.05	0.05	ICE I-46v2	25	5.80	20.00
113	1a	14x117	34.4	0.82	1407	80	0.20	0.10	0.05	0.05	ICE I-46v2	51	11.81	72.90
114	1b	14x117	34.4	0.66	1135	80	0.20	0.10	0.05	0.05	ICE I-30v2	45	10.54	38.70
115	1c	14x117	34.4	0.50	860	80	0.20	0.10	0.05	0.05	PilcoD1942	31	8.91	20.60
116	2a	14x117	34.4	0.72	1244	80	0.20	0.10	0.05	0.05	ICE I-46v2	45	10.00	57.60
117	2b	14x117	34.4	0.66	1135	80	0.20	0.10	0.05	0.05	ICE I-30v2	45	10.54	38.70

ID		HP	As	Stress at refusal	Capacity at refusal	Length	Friction	Toe Damping	Toe Quake	Skin Damping	Smallest Hammer	Driving stress at refusal	Cal. stroke	Energy
BASE			in^2	$1/A_sFy$	kips	ft	%					ksi	ft	k-ft
118	3	14x117	34.4	0.66	1135	80	0.20	0.10	0.05	0.05	ICE I-46v2	41	8.68	46.20
119	4	14x117	34.4	0.42	718	80	0.20	0.10	0.05	0.05	ICE I-46v2	25	5.40	19.50
120	1a	14x117	34.4	0.81	1389	80	0.30	0.10	0.05	0.05	ICE I-46v2	45	11.81	72.80
121	1b	14x117	34.4	0.66	1135	80	0.30	0.10	0.05	0.05	ICE I-30v2	40	10.81	39.70
122	1c	14x117	34.4	0.50	860	80	0.30	0.10	0.05	0.05	PilcoD1942	28	9.10	21.10
123	2a	14x117	34.4	0.81	1389	80	0.30	0.10	0.05	0.05	ICE I-46v2	45	11.81	72.80
124	2b	14x117	34.4	0.68	1178	80	0.30	0.10	0.05	0.05	ICE I-30v2	42	11.50	43.60
125	3	14x117	34.4	0.66	1135	80	0.30	0.10	0.05	0.05	ICE I-46v2	36	8.84	47.30
126	4	14x117	34.4	0.46	795	80	0.30	0.10	0.05	0.05	ICE I-46v2	25	5.93	23.80

Table E2 Sensitivity analyses results.

ID	HP	As	Stress at refusal	Capacity at refusal	Length	Friction	Toe Damping	Toe Quake	Skin Damping	Smallest Hammer	Driving stress at refusal	Cal. stroke	Energy
SENSITIVITY		in^2	$1/A_sFy$	kips	ft	%					ksi	ft	k-ft
127	12x74	21.8	0.97	1056	50	0.20	0.10	0.05	0.20	ICE I-46v2	58	11.81	67.90
128	12x74	21.8	0.66	719	50	0.20	0.10	0.05	0.20	ICE I-30v2	39	9.00	29.60
129	12x74	21.8	0.50	545	50	0.20	0.10	0.05	0.20	ICE I-12v2	30	10.07	15.90
130	12x74	21.8	0.76	831	50	0.20	0.10	0.05	0.20	ICE I-46v2	45	8.49	41.70
131	12x74	21.8	0.44	480	50	0.20	0.10	0.05	0.20	ICE I-46v2	25	4.90	14.90
132	12x74	21.8	1.00	1090	50	0.20	0.15	0.10	0.05	ICE I-46v2	65	11.34	65.30
133	12x74	21.8	0.66	719	50	0.20	0.15	0.10	0.05	ICE I-30v2	45	8.92	29.70
134	12x74	21.8	0.50	545	50	0.20	0.15	0.10	0.05	ICE I-12v2	33	10.30	16.90
135	12x74	21.8	0.70	764	50	0.20	0.15	0.10	0.05	ICE I-46v2	45	7.37	34.00
136	12x74	21.8	0.43	469	50	0.20	0.15	0.10	0.05	ICE I-46v2	25	4.69	13.30
137	12x74	21.8	0.92	1000	50	0.20	0.15	0.10	0.20	ICE I-46v2	55	11.81	69.10
138	12x74	21.8	0.66	719	50	0.20	0.15	0.10	0.20	ICE I-30v2	40	10.25	36.70
139	12x74	21.8	0.50	545	50	0.20	0.15	0.10	0.20	PilcoD1942	30	8.84	21.10
140	12x74	21.8	0.79	864	50	0.20	0.15	0.10	0.20	ICE I-46v2	45	9.40	49.70
141	12x74	21.8	0.43	468	50	0.20	0.15	0.10	0.20	ICE I-46v2	25	4.88	14.60