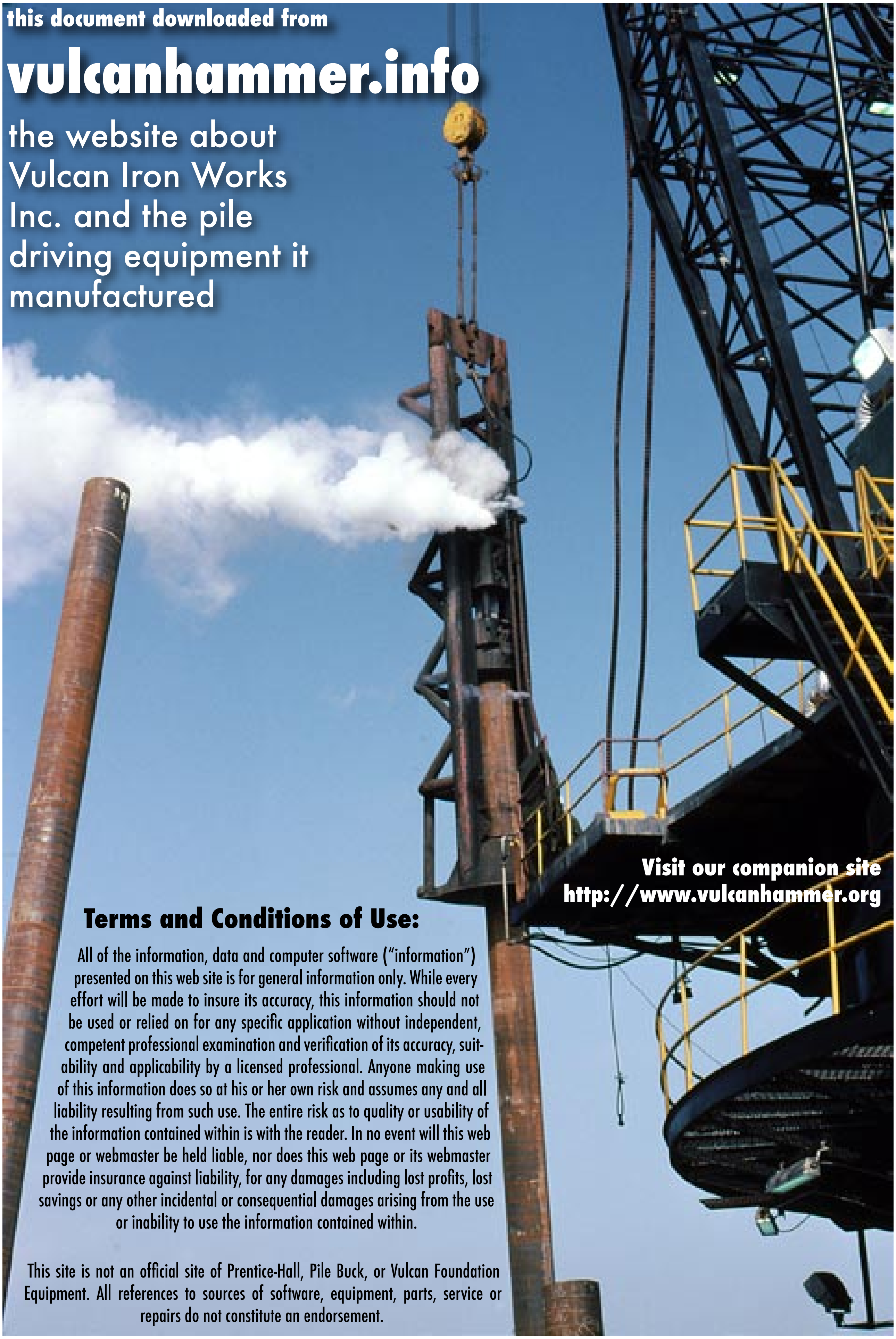

Development of the WSDOT Pile Driving Formula and Its Calibration for Load and Resistance Factor Design (LRFD)

WA-RD 610.1

Final Research Report
March 2005

Washington State
Department of Transportation

Washington State Transportation Commission
Planning and Capital Program Management
in cooperation with:
U.S. DOT – Federal Highway Administration

Final Research Report

Development of the WSDOT Pile Driving Formula and Its Calibration for Load and Resistance Factor Design (LRFD)

by
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Prepared for

Washington State Department of Transportation
And in cooperation with
U.S. Department of Transportation
Federal Highway Administration

March, 2005

TECHNICAL REPORT STANDARD TITLE PAGE

1. REPORT NO.	2. GOVERNMENT ACCESSION NO.	3. RECIPIENT'S CATALOG NO.
WA-RD 610.1		

4. TITLE AND SUBTITLE	5. REPORT DATE
Development of the WSDOT Pile Driving Formula and Its Calibration for Load and Resistance Factor Design (LRFD)	March 2005
	6. PERFORMING ORGANIZATION CODE
7. AUTHOR(S) Tony M. Allen	8. PERFORMING ORGANIZATION REPORT NO.
9. PERFORMING ORGANIZATION NAME AND ADDRESS Washington State Transportation Center (TRAC) University of Washington, Box 354802 University District Building; 1107 NE 45th Street, Suite 535 Seattle, Washington 98105-4631	10. WORK UNIT NO. 11. CONTRACT OR GRANT NO.
12. SPONSORING AGENCY NAME AND ADDRESS Research Office Washington State Department of Transportation Transportation Building, MS 47370 Olympia, Washington 98504-7370 Keith Anderson, Project Manager, 360-709-5405	13. TYPE OF REPORT AND PERIOD COVERED Research report 14. SPONSORING AGENCY CODE

15. SUPPLEMENTARY NOTES

This study was conducted in cooperation with the U.S. Department of Transportation, Federal Highway Administration.

16. ABSTRACT

Prior to 1997, WSDOT used the Engineering News Record (ENR) Formula for driving piling to the design capacity. Washington State Department of Transportation (WSDOT) sponsored research published in 1988 had shown that the ENR formula was quite inaccurate, and that moving toward the Gates Formula would be a substantial improvement (Fragaszy et al. 1988). Hence, in 1996, an in-house study was initiated to update the driving formula used for pile driving acceptance in the WSDOT Standard Specifications.

Recently compiled databases of pile load test results were used as the basis for developing improvements to the Gates Formula to improve pile bearing resistance prediction accuracy. From this empirical analysis, the WSDOT driving formula was derived.

Once the WSDOT driving formula had been developed, the empirical data used for its development were also used to establish statistical parameters that could be used in reliability analyses to determine resistance factors for load and resistance factor design (LRFD). The Monte Carlo method was used to perform the reliability analyses. Other methods of pile resistance prediction were also analyzed, and resistance factors were developed for those methods as well.

Of the driving formulae evaluated, the WSDOT formula produced the most efficient result, with a resistance factor of 0.55 to 0.60. A resistance factor of 0.55 is recommended. Dynamic measurement during pile driving using the pile driving analyzer (PDA), combined with signal matching analysis (e.g., CAPWAP), produced the most efficient result of all the pile resistance prediction methods, with a resistance factor of 0.70 to 0.80.

17. KEY WORDS	18. DISTRIBUTION STATEMENT
Pile foundation, bearing, LRFD, design, calibration	No restrictions. This document is available to the public through the National Technical Information Service, Springfield, VA 22616

19. SECURITY CLASSIF. (of this report)	20. SECURITY CLASSIF. (of this page)	21. NO. OF PAGES	22. PRICE
None	None	45	

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EXECUTIVE SUMMARY

Prior to 1997, WSDOT used the Engineering News Record (ENR) Formula for driving piling to the design capacity. Washington State Department of Transportation (WSDOT) sponsored research published in 1988 had shown that the ENR formula was quite inaccurate and that moving toward the Gates formula would be a substantial improvement (Fragaszy et al. 1988). Hence, in 1996, an in-house study was initiated to update the driving formula used for pile driving acceptance in the WSDOT Standard Specifications.

Included within the scope of this study was an evaluation of whether prediction performance could be improved by making empirical improvements to the Gates Formula. Others, such as the Federal Highway Administration (FHWA), had proposed modifications to the Gates Formula in recent years to deal with recognized deficiencies. Therefore, recently compiled databases of pile load test results were used to verify whether the improvements to the Gates Formula proposed by the FHWA indeed would produce a more accurate pile resistance prediction and to develop any additional necessary improvements. From this empirical analysis, the WSDOT driving formula was derived.

While the effort to develop the WSDOT driving formula started out as an empirical analysis to improve the Gates Formula, so many changes were made that it has in essence become a new driving formula. For example, the square root function of hammer energy was removed (hammer energy is now to the first power), and the $\log_{10}$ function of penetration resistance was replaced with the natural logarithm. In addition, coefficients were added to account for the different hammer types and pile types. The consistency of the WSDOT driving

formula with wave equation predictions was also evaluated to provide the most seamless transition possible to hammer-pile system performance evaluation by the wave equation.

Once the WSDOT driving formula had been developed, the empirical data used for its development were also used to establish statistical parameters that could be used in reliability analyses to determine resistance factors for load and resistance factor design (LRFD). The Monte Carlo method was used to perform the reliability analyses. Other methods of pile resistance prediction were also analyzed, and resistance factors were developed for those methods as well.

Of the driving formulae evaluated, the WSDOT formula produced the most efficient result, with a resistance factor of 0.55 to 0.60. A resistance factor of 0.55 is recommended. Dynamic measurement during pile driving using the pile driving analyzer (PDA), combined with signal matching analysis (e.g., CAPWAP), produced the most efficient result of all the pile resistance prediction methods, with a resistance factor of 0.70 to 0.80.

THE PROBLEM

Prior to 1997, WSDOT used the Engineering News Record (ENR) Formula for driving piling to the design capacity. Washington State Department of Transportation (WSDOT)-sponsored research published in 1988 had shown that the ENR formula was quite inaccurate and that moving toward the Gates formula would be a substantial improvement (Fragaszy, et al. 1988). Hence, in 1996, an in-house study was initiated to update the driving formula used for pile driving acceptance in the WSDOT Standard Specifications.

BACKGROUND

Pile load test data from Paikowsky et al. (1994), later updated with the expanded database also provided by Paikowsky et al. (2004), were used to develop the WSDOT pile driving formula. The WSDOT driving formula, as is true of most driving formulae, was empirically derived. The basic form of the equation has similarities to the Gates Formula. While the Gates Formula proved attractive in previous studies because of the relatively low variability in the predicted resistance relative to the pile load test measured resistance, it tended to over-predict resistance at low driving resistances and under-predict resistance at high driving resistances. To help offset this problem, the Federal Highway Administration (FHWA) proposed a modified Gates Formula (Hannigan et al., 1997). Similarly, the WSDOT pile driving formula was developed to maintain the low prediction variability of the Gates Formula but at the same time minimize its tendency to under- or over-predict the pile nominal resistance.

The WSDOT pile driving formula has the following form:

$$R_n = 6.6 \times F_{eff} \times E \times Ln\left(10N\right) \qquad (1)$$

where:

where:	R_n	=	ultimate bearing resistance, in kips
	F_{eff}	=	hammer efficiency factor
	E	=	developed energy, equal to W times H, in ft-kips
	W	=	weight of ram, in kips
	H	=	vertical drop of hammer or stroke of ram, in feet
	N	=	average penetration resistance in blows per inch for the last 4 inches of driving
	Ln	=	the natural logarithm, in base “e”

In the WSDOT Standard Specifications for Road, Bridge, and Municipal Construction (2004), Section 6-05.3(12), Equation 1 has been simplified to:

$$R_n = F \times E \times Ln\left(10N\right) \qquad (2)$$

where: R_n = ultimate bearing resistance, in tons, and

F = a constant that varies with hammer and pile type

Note that the energy term in the WSDOT formula is intended to represent the actual stroke (single-acting hammers) or equivalent stroke (double-acting hammers) observed during driving multiplied by the ram weight, termed the developed energy. Technically, this is the kinetic energy in the ram at impact for a given blow. If ram velocity is not measured, it may be assumed equal to the potential energy of the ram at the height of the stroke, taken as the ram weight times the stroke. These formulae are not intended to be used with the gross rated energy for the hammer. This issue only affects single-acting (i.e., open ended) diesel hammers and all double-acting hammers. This issue does not affect single-acting air/steam hammers in terms of how these driving formulae are applied in the field.

The data used to develop the current form of the WSDOT formula are provided in Table 1. Most of the data provided in Table 1 were obtained at end of drive (EOD) conditions (i.e., when the pile was first driven to tip elevation), with a limited amount of data provided at beginning of redrive (BOR) conditions (i.e., when the pile is driven a limited distance below the tip elevation achieved during initial driving after an extended period of time, typically several days after the pile was initially driven to tip elevation). Additional BOR data are provided by Paikowsky, et al. (2004). F_{eff} was derived in this formula to be approximately equal to the measured transfer efficiency, defined as the measured transferred energy divided by the estimated developed energy for the hammer.

Table 1. Pile load test database summary (adapted from Paikowsky et al. 2004).

Pile-Case Number	Reference No.	Location	Pile Type	Penetr Depth (ft)	Soil Type Side	Soil Type Tip	Hammer Model	Hammer and Pile Type*	Rated Energy Based on Hammer Model (kip-ft)	Estimated Developed Energy (kip-ft)	Transferred Energy (ft-kips)	Blow Count (BPI)	Measured Static Resist from Load Test (kips)	CAPWAP TEPWAP (kips)	WSDOT Equation (kips)
FWA-EOD	3rd lake	Washington	CEP 48"	24.8	till-gravel	till	Con300	1	90.0	90	44.9	47	1300	295	2010
FWB-EOD	3rd lake	Washington	CEP 48"	109	till-gravel	till	Con300	1	90.0	90	47.2	30	1225		1863
FL3-EOD	Rt.415	Louisiana	PSC24"sq	84.3	silty clay	silty sand	Vul-020	1	60.0	60	14.6	1.67	400	136	613
ST46-EOD	Castletn	New York	CEP 10"	38	silt-sand	silt-sand	VUL-1	1	15.0	15	5.5	2.67	104	82	179
A54-EOD	HICC	Australia	RC10.8"sq	67.6	silty clay	clay	Banut-6	1	34.7	34.7	21.05	3.63	638	383	453
A147-EOD	HICC	Australia	RC10.8"sq	67.6	silty clay	clay	Banut	1	34.7	34.7	19.69	1.95	552	259	374
A3-EOD2	Apalach	Florida	VC 24"sq	90.3	clayey sand	sand	Vul-020	1	60.0	60	18.85	3.42	939	368	769
A25-EOD	Apalach	Florida	VC 24"sq	55.1	clayey sand	sand	Vul-020	1	60.0	60	22.52	4	800	459	803
A16-EOD	Apalach	Florida	PSC18"sq	60.6	sandy clay	sand	Vul-010	1	32.5	32.5	11.52	3.17	308	224	408
A41-EOD	Apalach	Florida	VC 24"sq	52	clay	sand	Vul-020	1	60.0	60	21.94	4.16	530	431	812
A101-EOD	Apalach	Florida	VC 24"sq	61.8	clay	clayey sand	Vul-020	1	60.0	60	20.95	2.91	810	517	734
A133-EOD	Apalach	Florida	VC 24"sq	103.9	clayey sand	sandy clay	Vul-020	1	60.0	60	18.04	5.25	826	311	863
A145-EOD	Apalach	Florida	VC 24"sq	102.9	clayey sand	sand	Vul-020	1	60.0	60	18.67	5.25	940	353	863
CB26-EOD	Choctw	Florida	PSC24"sq	62.5	clayey sand	sand	Vul-020	1	60.0	60	15.53	4.75	965	488	841
CHA1-EOD	Jones Is.	Wisconsin	CEP 12.75"	123.0	sa-si clay	silty sand	Vul-200C	1	50.2	50.2	34.88	17	647	390	936
CHA4-EOD	Jones Is.	Wisconsin	CEP 12.75"	117.0	sa-si clay	silty sand	Vul-200C	1	50.2	50.2	22.99	3.75	504	271	660
CHB2-EOD	Jones Is.	Wisconsin	HP12x63	155.3	sa-si clay	silty sand	Vul-010	1	32.5	32.5	25.3	0.75	315	110	238
CHB3-EOD	Jones Is.	Wisconsin	HP12x63	142.1	sa-si clay	silty sand	Vul-010	1	32.5	32.5	18.8	1	214	105	272
CHC3-EOD	Jones Is.	Wisconsin	CEP14"	155.2	sa-si clay	silty sand	Vul-010	1	32.5	32.5	23.9	1.75	237	110	338
CH4-EOD	Jones Is.	Wisconsin	CEP9.63"	142.5	silty clay		Vul-010	1	32.5	32.5	21.89	0.83	364	150	250
CH39-EOD	Jones Is.	Wisconsin	CEP9.63"	142.0	silty clay	silty clay	Vul-010	1	32.5	32.5	15.6	0.83	656	187	250
CH6-5B-EOD	Jones Is.	Wisconsin	CEP9.63"	144.0	silty clay	silty sand	Vul-010	1	32.5	32.5	13.65	0.5	372		190
CH95B-EOD	Jones Is.	Wisconsin	CEP9.63"	139.0	silty clay	sand & grvl	Vul-010	1	32.5	32.5	31.55	1.17	554	221	290

Pile-Case Number	Reference No.	Location	Pile Type	Penetr Depth (ft)	Soil Type Side	Soil Type Tip	Hammer Model	Hammer and Pile Type*	Rated Energy Based on Hammer Model (kip-ft)	Estimated Developed Energy (kip-ft)	Transferred Energy (ft-kips)	Blow Count (BPI)	Measured Static Resist from Load Test (kips)	CAPWAP TEPWAP (kips)	WSDOT Equation (kips)
BB13-EOD	Duval Cnty	Florida	VC 30"sq	92.8	clayey sand	sand	Con300	1	90.0	90	56.68	3.36	988	700	1148
BB24-EOD	Duval Cnty	Florida	VC 30"sq	80.2	sand	clay	Con300	1	90.0	90	64.74	5.75	1094	1300	1324
BC79-EOD	Beaufort	S.Carolina	PSC 24" oct	77.0	si-cl-sand	calcar sand	Vul 320/520	1	60.0	60	30.73	2.97	532		739
BC64-EOD	Beaufort	S.Carolina	PSC 24" oct	61.0	si-cl-sand	calcar sand	Vul 320/520	1	60.0	60	26.81	3.9	1114		798
MB1-EOD	Myrtle Bch	S. Carolina	PSC 16"sq	62.0	sand	silty sand	Con100E	1	50.0	50	24.39	1.75	819	170	519
S1-EOD	Socastee	S. Carolina	OEP 24"	81.5	clayey sand	sandy silt	Vul 512	1	60.0	60	30.19	3.75	586	460	789
S2-EOD	Socastee	S. Carolina	HP14x73	78.0	clayey sand	sandy silt	Vul 512	1	60.0	60	17.21	1.42	318	215	578
DD22-EOD	Orlando	Florida	PSC 14"sq	90.0	clay	sand	Vul 80C	1	24.5	24.48	9.30	5.67	760	255	359
DD23-EOD	Orlando	Florida	CEP 12.75	82.0	clay	sand	Vul 80C	1	24.5	24.48	9.70	2.25	476	153	277
NBTP2-EOD	Newbury	Massachusetts	HP12X74	112.0	si-sa-clay	glacial till	HPSI 1000	1	50.0	50	29.4	3	416	304	617
NBTP3-EOD	Newbury	Massachusetts	HP12X74	108.5	si-sa-clay	silty sand	HPSI 1000	1	50.0	50	32.4	4	448	315	670
NBTP5-EOD	Newbury	Massachusetts	CEP12.75"	111.0	si-sa-clay	glacial till	HPSI 1000	1	50.0	50	25.5	3	400	320	617
DD29-EOD	Orlando	Florida	CEP 12.75"	163	clayey sand	clayey sand	Vul - 80C	1	24.5	24.45	19.5	10.2	737	351	410
QC3-EOD	Queens County	New York	PSC 54" cyl	75.5	sand	dense sand	Conmaco 5300	1	150.0	150	18.8	8	1452	405	2386
NYSP-EOD	SE New York	New York	HP10X42	109.9	silty sand	silty sand w/gr	Vulcan 06	1	19.5	19.5	9.8	13	313	132	345
HFLS3 - EOD	Tampa Bay	Florida	PSC 30" sq	39.60	sa-si-clay	sandy clay	Conmaco C300	1	90.0	90.00	42.0	20.8	1797	1301	1744
HFLS4L - EOD	Tampa Bay	Florida	PSC 30" sq	73.50	cl-si-limestone-sand	limerock	Conmaco C300	1	90.0	90.00	36.0	4.76	857	705	1262
FN2-EOD	I-480	Omaha NE	PSC12"sq	65	silty clay	till	D-30	2	46.4	59.6	12.7	3.5	354	226	403
FN3-EOD	I-480	Omaha NE	PSC14"sq	56	silty clay	till	D-30	2	48.2	59.6	9.9	9.17	374	179	532
FO2-EOD	Cim S-1	Oklahoma	PSC24"oct	63	silty sand	silty sand	DE110	2	86.9	110	18.28	5.08	750	530	833
FO4-EOD	Cim S-2	Oklahoma	RC24"sq	45	sa-si-clay	clayey sand	DE110	2	89.8	110	9.81	11.67	1650	658	1044
FOR1-EOD	Alsea	Oregon	PSC20"sq	125.5	sand & silt	siltstone	D-46-23	2	86.5	107	30.11	9.17	1380	559	955

Pile-Case Number	Reference No.	Location	Pile Type	Penetr Depth (ft)	Soil Type Side	Soil Type Tip	Hammer Model	Hammer and Pile Type*	Rated Energy Based on Hammer Model (kip-ft)	Estimated Developed Energy (kip-ft)	Transferred Energy (ft-kips)	Blow Count (BPI)	Measured Static Resist from Load Test (kips)	CAPWAP TEPWAP (kips)	WSDOT Equation (kips)
FA1-EOD	I-165	Alabama	PSC 18"sq	64	silty sand	silty sand	K-45	2	69.8	92.8	17.53	1.5	345	205	462
FA2-EOD	I-165	Alabama	PSC 18"sq	75	silty sand	silty sand	K-45	2	72.2	92.8	21.22	3.5	535	428	627
FA3-EOD	I-165	Alabama	PSC 24"sq	64	silty sand	silty sand	K-45	2	71.6	92.8	22.79	2.83	614	340	584
FA4-EOD	I-165	Alabama	PSC 24"sq	75	silty sand	silty sand	K-45	2	74.0	92.8	19.06	6.42	773	446	752
FA5-EOD	I-165	Alabama	PSC 36"sq	73	silty sand	silty sand	D-62-22	2	122.4	152.5	37.06	7.67	1074	662	1298
WC3-EOD	White	Florida	PSC24"sq	27.3	ls.-d.sand	dense	Delmag	2	86.6	107	17.5	9.33	610	509	959
WC6-EOD	White	Florida	PSC24"sq	28.3	ls.-d.sand	dense	Delmag	2	84.4	107	17.6	5	495	450	807
ST1-EOD	Site H	Florida	PSC 18"sq	44	-	carb sand	D-36-13	2	64.4	84	33.13	2.42	344	505	501
ST2-EOD	Site P	Florida	PSC 18"sq	40	-	carb sand	D-36-13	2	65.3	84	33.03	3.42	540	616	563
33P4-EOD	Site P	Ontario	PSC 12"sq	54.2	cl-sa-silt	cl-silt-till	B-400	2	35.5	45	24.47	5	500	400	339
33P5-EOD	Site P	Ontario	#14 Timber	28.4	cl-sa-silt	cl-silt-till	B-225	2	23.8	29.3	6.41	10.67	200	143	272
JR17-EOD	James River	Richmond, VA	PSC 24" sq	35.3	cl-si-sand	silty sand	Delmag D-46	2	90.3	107	29.5	27	1230	626	1235
LB3-EOD	Luling Bridge	Kenner, LA	PSC 24" sq	81.5	clay	Sand	Delmag D46-13	2	70.9	96.5	25.17	0.83	398	60.4	366
LB4-EOD	Luling Bridge	Kenner, LA	PSC 30" sq	82	clay	Sand	Delmag D46-13	2	71.9	96.5	23.15	1.17	453	45.4	432
LB5-EOD	Luling Bridge	Kenner, LA	PSC 30" sq	82	clay	Sand	Delmag D46-13	2	73.1	96.5	11.24	1.82	420	59.2	518
LB6-EOD	Luling Bridge	Kenner, LA	PSC 36" cyl	81	clay	Sand	Delmag D46-13	2	72.1	96.5	15.26	1.25	471	90.8	444
LB7-EOD	Luling Bridge	Kenner, LA	PSC 36" cyl	80.7	clay	Sand	Delmag D46-13	2	75.4	96.5	6.76	3.93	488	102.7	676
TW488-EOD	MBTA Project	Massachusetts	PSC 14" sq	76.0	stiff clay	stiff clay	Delmag D30-32	2	53.6	73.66	14.9	0.67	319	82	249
PX7-EOD	Phoenix	Arizona	PSC 16" sq	20	clay & sand	clay	MKT DE 70B	2	51.7	59.5	15	56.3	1123	529	800
TSW/D62/1-EOD	Site 2	Hong Kong	PSC 19.69" cyl	74.48	sa-cl-silt	sandy silt	Delmag D62	2	118.4	152	64.02	3.63	1000	755	1039
QC14-EOD	Queens County	New York	PSC 14" cyl	75.0	sand	dense sand	MKT S-8	2	19.5	23.8	5.8	12	324	279	227
49SB37 -EOD	Clearwater	Florida	PSC 30" sq	23.40	sandy clay	silty limestone	Delmag D62-32	2	50.7	61.49	10.2	14.9	1209	1025	1080
FN1-EOD	I-480	Omaha NE	HP10x42	72	silty clay	till	D-30	3	47.9	59.6	17.3	2.83	300	230	496
FN4-EOD	I-480	Omaha NE	CEP12.75"	66	silty clay	till	D-30	3	47.5	59.6	15.55	2.5	280	244	474

Pile-Case Number	Reference No.	Location	Pile Type	Penetr Depth (ft)	Soil Type Side	Soil Type Tip	Hammer Model	Hammer and Pile Type*	Rated Energy Based on Hammer Model (kip-ft)	Estimated Developed Energy (kip-ft)	Transferred Energy (ft-kips)	Blow Count (BPI)	Measured Static Resist from Load Test (kips)	CAPWAP TEPWAP (kips)	WSDOT Equation (kips)
FIA-EOD	Site 1	Iowa	HP14x89	114.1	clayey sand	sand	K-25	3	41.8	51.5	23.38	3.33	930	367	454
FIB-EOD	Site 1	Iowa	CEP 14"	94.1	clayey sand	sand	K-25	3	43.2	51.5	25.4	5.83	650	511	545
FO1-EOD	Cim S-1	Oklahoma	CEP 26"	60.2	silty sand	silty sand	DE110	3	92.1	110	18.08	5.67	557	496	1154
FO3-EOD	Cim S-2	Oklahoma	HP14x117	63.7	sa-si-clay	clayey sand	DE110	3	98.3	110	16.4	16.67	820	566	1559
FM5-EOD	Site A	Maine	CEP 18"	99	clay & sand	sand	K-45	3	71.1	92.8	27	1.29	420	346	564
FM17-EOD	Site B	Maine	CEP 18"	71.1	till	till	K-45	3	71.5	92.8	39.5	1.42	447	424	589
FM23-EOD	Site B	Maine	CEP 18"	50.7	till	till	K-45	3	71.2	92.8	33.3	1.33	340	323	572
FC1-EOD	Crook	Colorado	CEP12.75"	33.5	sand	sand	KC-25	3	41.9	51.5	15.47	3.5	340	270	462
FC2-EOD	Crook	Colorado	CEP12.75"	26.5	sand	sand	KC-25	3	42.0	51.5	18.07	3.67	376	375	469
FV15-EOD	WRJ	Vermont	HP14x73	75	silt-d.sand	sand gravel	MKT-35B	3	17.3	21	10	4.17	315	194	200
FV10-EOD	WRJ	Vermont	HP14x73	90	silt-d.sand	sand gravel	MKT-35B	3	16.8	21	10.98	2.67	313	159	171
FP5-EOD	Tioga	Penn.	Monotube	23.6	sandy grvl	sandy grvl	D-12	3	19.7	23.6	7.56	5.42	227	210	244
CA1-EOD	Site C-L	O.S. Ont	CEP 9.6"	154.3	si-sa-clay	si-sa-till	B-400	3	40.8	45	20.32	21.33	533	410	679
T1/A-EOD	offshore	Israel	OEP 60"	52.8	clcr sand	sand	D-55	3	106.3	125	44.99	7.37	1984	1775	1418
T2/A-EOD	offshore	Israel	OEP 48"	52.5	clcr sand	sand	D-55	3	103.7	125	60.62	4.83	1470	1252	1247
GZB22-EOD	NWS	Colt Neck	OEP 36"	118	sand-clay	silt-clay	MH72B	3	115.8	135	55.17	8.5	1060	1109	1596
EF62-EOD	Ottawa	Canada	CP 9.625"	62.3	si-sa-clay	till	D30-32	3	62.0	73.7	27.29	6.1	477	522	790
33P1-EOD	Site P	Ontario	HP 12x74	114.4	cl-sa-silt	silty sand	B-400	3	39.4	45	32.67	12	800	439	585
33P2-EOD	Site P	Ontario	CP 12.75"	107.2	cl-sa-silt	silty sand	B-400	3	42.3	45	32.84	39	490	290	783
TRD22-EOD	Site R	Ontario	HP 12x74	20.1	sand	till	D-12	3	21.8	23.6	9.78	30	350	432	386
TRE22-EOD	Site R	Ontario	HP 12x74	25.7	sand	rock	D-22	3	36.9	40.6	15.19	22	570	575	617
TRP5X-EOD	Site R	Ontario	HP 12x53	25.2	sand	rock	D-12	3	22.1	23.6	9.17	38	475	484	408
PX3-EOD	Phoenix	Arizona	HP14X117	50	clay & sand	sa-gr-cobble	MKT DE 70B	3	54.5	59.5	8.9	25.6	1239	554	938
PX4-EOD	Phoenix	Arizona	CEP 14"	22.4	clay & sand	clayey sand	MKT DE 70B	3	57.7	59.5	12.4	65.3	767	508	1160

Pile-Case Number	Reference No.	Location	Pile Type	Penetr Depth (ft)	Soil Type Side	Soil Type Tip	Hammer Model	Hammer and Pile Type*	Rated Energy Based on Hammer Model (kip-ft)	Estimated Developed Energy (kip-ft)	Transferred Energy (ft-kips)	Blow Count (BPI)	Measured Static Resist from Load Test (kips)	CAPWAP TEPWAP (kips)	WSDOT Equation (kips)
TSW/D62/2-EOD	Site 3	Hong Kong	HP12X120?	97.44	sa-cl-silt	sandy silt	Delmag D62	3	125.3	152	67.04	4.38	1011	1091	1469
OD1J-EOD	Site 1	Oakland, CA	OEP 24"	27.8	silty sand	silty clayey sand	Delmag D62-22	3	118.3	152	70.90	1.667	1691	786	1032
OD2P-EOD	Site 2	Oakland, CA	OEP 24"	40.0	silty sand	silty sandy clay	Delmag D62-22	3	114.1	152	53.60	0.917	655	350	784
OD2T-EOD	Site 2	Oakland, CA	CEP 24"	35.0	silty sand	silty sand & clay	Delmag D46-32	3	92.0	113.00	56.60	3.58	745	817	1022
OD3H-EOD	Site 3	Oakland, CA	OEP 42"	100.5	stiff clay	clay w/ sa-si-gr	Delmag D62-22	3	114.1	152	65.90	0.917	1124	324	784
OD4L-EOD	Site 4	Oakland, CA	CEP 24"	64.0	sandy clay	silty sandy clay	Delmag D62-22	3	117.1	152	66.90	1.417	959	504	963
OD4P-EOD	Site 4	Oakland, CA	CEP 24"	56.0	silty clay	silty sandy clay	Delmag D30-32	3	58.2	73.66	29.60	2.167	684	273	555
OD4T-EOD	Site 4	Oakland, CA	CEP 24"	60.0	sandy clay	silty sandy clay	Delmag D30-32	3	56.9	73.66	28.30	1.5	740	301	478
OD4W-EOD	Site 4	Oakland, CA	CEP 24"	60.0	sandy clay	silty sandy clay	Delmag D46-32	3	86.4	113.00	52.60	1.25	903	397	677
FMN2-EOD	Rt. 18	Minnesota	HP14x73	96	sa-si-clay	fat clay	ICE-90S	6	78.4	90	28.29	1.83	740	342	707
FMI1-EOD	Rt. 115	Missouri	CEP 14"	83	sand-gravel	sand	ICE-640	4	33.7	40.6	11	3	310	285	265
FMI2-EOD	Rt. 115	Missouri	CEP 14"	61	sand-gravel	sand	ICE-640	4	32.7	40.6	11.66	1.42	160	184	201
FKG-EOD	Rt.27	Kentucky	PSC14"sq	34.7	soft clay	dense	LB-520	4	23.7	26.3	8.31	23.25	465	288	299
GZA3-EOD	Civic	Prov. RI	CEP13.38"	125.5	silt-sand	gr-sa-silt	ICE-640	4	36.4	40.6	16.12	20	480	365	445
GZA5-EOD	Civic	Prov. RI	CEP 9.75"	93.8	silt-sand	till-shale	ICE-640	4	34.7	40.6	17.36	6	296	293	328
GZA6-EOD	Civic	Prov. RI	CEP 9.75"	156	silt-sand	gr-sa-silt	ICE-640	4	36.0	40.6	13.4	15	326	275	416
GZBBC-EOD	Civic	Prov. RI	CEP 10"	99.5	silt-sand	silt	ICE-640	4	36.4	40.6	17.67	20	530	413	445
GZBP2-EOD	Civic	Prov. RI	CEP13.38"	106	silt-sand	gr-sa-silt	ICE-640	4	36.4	40.6	9.57	20	320	317	445
GZB6-EOD	Civic	Prov. RI	CEP13.38"	92.3	silt-sand	si-sa-till	ICE-640	4	35.5	40.6	15.91	11	390	341	386
GZZ5-EOD	Deer Is.	Boston MA	CEP 14"	87	till-clay	till	ICE1070	4	61.1	72.6	28.73	4.2	440	214	528

Pile-Case Number	Reference No.	Location	Pile Type	Penetr Depth (ft)	Soil Type Side	Soil Type Tip	Hammer Model	Hammer and Pile Type*	Rated Energy Based on Hammer Model (kip-ft)	Estimated Developed Energy (kip-ft)	Transferred Energy (ft-kips)	Blow Count (BPI)	Measured Static Resist from Load Test (kips)	CAPWAP TEPWAP (kips)	WSDOT Equation (kips)
GZO5-EOD	Deer Is.	Boston MA	CEP 14"	54	till-clay	till	ICE1070	4	61.1	72.6	23.71	4.2	486	205	528
GZCC5-EOD	Deer Is.	Boston MA	CEP 14"	80	till-clay	till	ICE1070	4	61.8	72.6	34.05	5.4	490	492	569
GZL2-EOD	Deer Is.	Boston MA	CEP 14"	83	till-clay	till	ICE1070	4	63.0	72.6	25.81	9	660	267	655
GZP14-EOD	Deer Is.	Boston MA	CEP 14"	60.5	till-clay	till	ICE1070	4	61.6	72.6	25.68	5	420	305	556
GZP11-EOD	Deer Is.	Boston MA	CEP 14"	56.5	till-clay	till	ICE1070	4	61.7	72.6	16.13	5.3	386	239	566
GZP12-EOD	Deer Is.	Boston MA	CEP 14"	69	till-clay	till	ICE1070	4	63.9	72.6	34.64	12.6	560	520	714
GF19-EOD	Site 1	Pgh. PA	HP10x42	49.5	grvl-snd-slt	shale	LB-520	4	23.6	26.3	9.4	20	397	398	289
GF110-EOD	Site 1	Pgh. PA	HP12x74	49.7	grvl-snd-slt	shale	LB-520	4	24.3	26.3	10.16	44	550	457	342
GF222-EOD	Site 2	Pgh. PA	HP12x74	61.1	grvl-snd-slt	shale	ICE-640	4	36.4	40.6	16.6	20	570	512	445
GF224-EOD	Site 2	Pgh. PA	Monotube	29.6	grvl-snd-slt	grvl-snd-slt	ICE-640	4	34.4	40.6	21	5	463	419	311
GF312-EOD	Site 3	Pgh. PA	HP12x74	28.2	snd-grvl-shl	shale	LB-520	4	23.5	26.3	6.86	18	310	405	282
GF313-EOD	Site 3	Pgh. PA	HP10x57	31.5	snd-grvl-shl	claystone	LB-520	4	23.6	26.3	10.05	20	330	446	289
GF412-EOD	Site 4	Pgh. PA	HP12x74	33.6	grvl-snd-slt	claystone	LB-520	4	24.2	26.3	8.49	39	272	455	334
GF413-EOD	Site 4	Pgh. PA	HP10x57	34.6	grvl-snd-slt	claystone	LB-520	4	24.2	26.3	9.07	39	300	428	334
GF414-EOD	Site 4	Pgh. PA	HP10x57	34.7	grvl-snd-slt	claystone	ICE-640	4	37.7	40.6	16.47	48	390	524	538
GF415-EOD	Site 4	Pgh. PA	HP12x74	34.1	grvl-snd-slt	claystone	ICE-640	4	36.9	40.6	12.25	28	500	561	480
PO19-EOD	Port Orng	Florida	PSC18"sq	17.2	sand	dense sand	ICE-640	4	35.3	40.6	7.37	9.17	265	245	368
DI221-EOD	Deer Island	Massachusetts	PSC 14" sq	63.0	sa-si-clay	fine sand & silt	ICE 640	4	35.1	40.6	17.1	8	351		355
TSW/HHK9/1-EOD	Site 2	Hong Kong	OEP 60"		sa-cl-silt	sandy silt	Junttan HHk9	5	78.1	78.1	61.14	3.91	1021	857	945
TSW/HHK9/2-EOD	Site 3	Hong Kong	OEP 48"		sa-cl-silt	sandy silt	Junttan HHk9	5	78.1	78.1	72.94	3.63	1055	947	926
TSW/HHK9/1-BOR	Site 2	Hong Kong	PSC 19.69" cyl		sa-cl-silt	sandy silt	Junttan HHk9	5	78.1	78.1	57.97	11.55	1021	1091	1224
TSW/HHK9/2-BOR	Site 3	Hong Kong	HP12X120?		sa-cl-silt	sandy silt	Junttan HHk9	5	78.1	78.1	64.20	6.51	1055	978	1076

Pile-Case Number	Reference No.	Location	Pile Type	Penetr Depth (ft)	Soil Type Side	Soil Type Tip	Hammer Model	Hammer and Pile Type*	Rated Energy Based on Hammer Model (kip-ft)	Estimated Developed Energy (kip-ft)	Transferred Energy (ft-kips)	Blow Count (BPI)	Measured Static Resist from Load Test (kips)	CAPWAP TEPWAP (kips)	WSDOT Equation (kips)
D2-BOR1	Delft	Holland	PSC 19.69" cyl		clay-sand	clay	IHC	5	28.03	28.0	8.46	3.58	124	147	331
D3-BORb	Delft	Holland	HP12X120?		clay-sand	sand	IHC	5	28.03	28.0	14.55	2.76	223	156	307
D5-BORb	Delft	Holland	PSC 9.7"sq		clay-sand	sand	IHC	5	28.03	28.0	15.65	4.23	228	296	346
CA5-BOR2	Site A	N.Y. Ont	PSC 9.7"sq		fill-sand	sand	49kdrop	6	54.2	54.2	31.44	11	480	489	471
CHB3-BOR3	Jones Is.	Wisconsin	PSC 9.7"sq		sa-si clay	silty sand	8tndrp	6	48	48.0	28.3	2.4	214	335	282
CHC3-BORL	Jones Is.	Wisconsin	CEP11.73"		sa-si clay	silty sand	8tndrp	6	48	48.0	38.3	1.5	237	390	240

*Hammer/pile type combinations are as follows:
1 = air/steam hammers with all piles
2 = open ended diesel hammers with concrete or timber piles
3 = open ended diesel hammers with steel piles
4 = closed ended diesel hammers
5 = hydraulic hammers
6 = drop hammers.

DATABASE ANALYSIS AND WSDOT PILE DRIVING FORMULA DEVELOPMENT

The observed stroke for the single-acting diesel and the double-acting hammers was not reported in the available database (see Table 1). Using the rated energy in the WSDOT formula (and other driving formulae as well) would result in a higher predicted nominal driving resistance than would typically be the case in practice, since in practice the developed energy would normally be used, at least for those hammer types in which the stroke is affected by the driving resistance. This could cause the calibration of the WSDOT driving formula (i.e., the determination of the resistance factor φ to be used discussed later in this report) to be overly conservative relative to practice in the field. Therefore, the likely observed stroke for these types of hammers had to be estimated so that the developed energy could be calculated for each case history in the database.

Because the wave equation produces an estimate of the driving resistance–stroke relationship, the wave equation (GRLWEAP 2003) was used to make this estimate. Combinations of hammer model (Delmag, Kobe, and MKT open-ended diesel, and ICE closed and open-ended diesel)—with rated energies ranging from 40 to 150 ft-kips, pile length ranging from 60 to 120 ft, and pile types including steel pipe and precast concrete—were used to assess the stroke–driving resistance relationship. An upper bound approach, which included increasing the stroke predicted by the wave equation by 1 ft and also establishing the stroke–driving resistance relationship near the upper end of the plotted wave equation results, was used to establish this relationship to make sure that the calibration remained conservative. Examples of these results are provided in figures 1 and 2. The stroke ratio in the figure is defined as the predicted stroke divided by the maximum possible stroke for the hammer. This stroke ratio multiplied by the rated energy for the hammer would be approximately equal to the developed energy for the hammer.

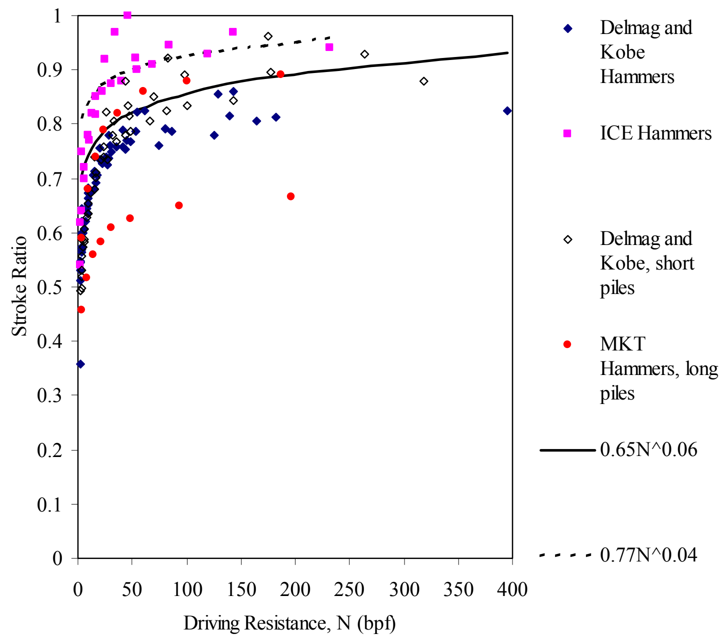

Figure 1. Stroke–driving resistance relationship for open-ended diesel hammers and steel piles based on wave equation predictions.

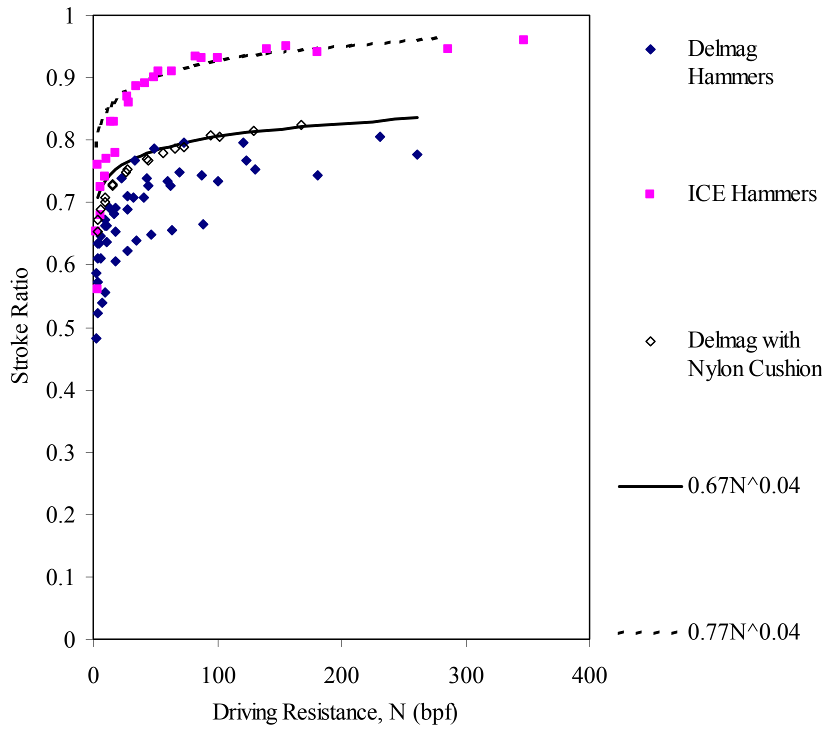

Figure 2. Stroke–driving resistance relationship for open-ended diesel hammers and concrete piles based on wave equation predictions.

Estimating the "actual" stroke in this way does introduce some uncertainties. The conservative approach taken to estimate the "actual" stroke using the wave equation should offset these uncertainties. However, the nominal pile bearing resistances have been estimated by using both the estimated developed energy and the rated energy, and compared to the measured pile bearing resistances.

The data provided in Table 1 were arranged by hammer and pile type to facilitate development of F_{eff} for the WSDOT driving formula. From this database, the average measured transfer efficiency, as defined above, for the various combinations of hammer type and pile type are summarized in Table 2.

Table 2. Average transfer efficiencies for various hammer and pile type combinations.

Hammer and Pile Type Combination	Average Measured Transfer Efficiency (relative to Developed Energy)	F_{eff} Used for WSDOT Driving Formula	"F" Used in WSDOT Driving Formula, per WSDOT Standard Specifications Section 6-05.3(12) ($F = 6.6 \times F_{eff}/2$)
Air/Steam hammers, all piles	0.49	0.55	1.8
Open ended diesel hammers with concrete or timber piles	0.30	0.37	1.2
Open ended diesel hammers with steel piles	0.48	0.47	1.6
Closed ended diesel hammers	0.41	0.35	1.2

While an attempt was made to have F_{eff} be approximately equal to the transfer efficiency as defined above, this was not fully achievable while also ensuring that the average ratio of the measured to predicted values of pile bearing resistance for each hammer and pile type combination be as close to 1.0 as possible. Therefore, the average measured values of transfer efficiency were used only as a starting point. The values of F_{eff} and "F" used in the final WSDOT driving formula that provided the best prediction of pile bearing resistance relative to the measured pile load test bearing resistance values for each case history are also provided in Table 2. The values of F_{eff} are within 0.07 of the average measured transfer efficiency.

Note that the data for hydraulic and drop hammers are not provided in Table 2 (they are provided in Table 1, however). As can be observed from Table 1, the available data for these two types of hammers are extremely limited, and most of the data are for beginning of redrive (BOR) conditions. However, these very limited data appear to indicate that "F" should be approximately 1.9 for hydraulic hammers (F_{eff} = 0.58) and approximately 0.9 for drop hammers (F_{eff} = 0.28) to provide a reasonable bearing resistance prediction, given that the N values (i.e., driving resistance) reflect that some soil setup has already taken place.

For the purpose of comparison, pile resistance predictions were also generated using Engineering News Record (ENR) and FHWA Gates formulae. The ENR formula was originally developed as an allowable stress design method, and contained within the formula a factor of safety of 6 (Peck, et al., 1974). The ENR equation as reported by Peck, et al. is specifically as follows:

$$R_a = \frac{W_H H}{FS(s+C)} \tag{3}$$

where: R_a = allowable (working) pile resistance measured during driving

W_H = weight of the hammer ram, expressed in the same units as R_a

H = height of fall of the ram (i.e., its stroke), expressed in the same units as s and C

s = pile permanent set, (IN)

C = energy loss per hammer blow, (0.1 IN for all hammers except drop hammers, and 1.0 IN for drop hammers)

FS = factor of safety, recommended as 6.0.

Note that $W_H H = E$, the developed hammer energy as defined previously.

For load and resistance factor design (LRFD), built in safety factors must be removed so that a nominal resistance is calculated. Therefore, if the safety factor is removed,

$$R_n = \frac{W_H H}{(s+C)} \tag{4}$$

where: R_n = nominal pile resistance measured during driving.

The FHWA attempted to modify the original Gates Formula to address deficiencies in its prediction accuracy. The FHWA Gates formula is as follows (Hannigan, et al., 1997):

$$R_n = 1.75\sqrt{E}\,\log_{10}(10N) - 100 \tag{5}$$

where: R_n = nominal pile resistance measured during pile driving (kips)

E = developed hammer energy. This is the kinetic energy in the ram at impact for a given blow. If ram velocity is not measured, it may be assumed equal to the potential energy of the ram at the height of the stroke, taken as the ram weight times the stroke (FT-LBS)

N = Number of hammer blows for 1 IN of pile permanent set (Blows/IN)

Plots of predicted versus measured pile nominal resistances using the WSDOT, FHWA Modified Gates, and ENR formulae are provided in figures 3 through 7. The plots are shown for predictions using the developed hammer energy and predictions using the rated hammer energy. For the various combinations of hammer and pile type, the WSDOT formula provides a better visual match of measured to predicted values than the FHWA Modified Gates and the ENR formulae. Note that the FHWA Modified Gates formula tends to over-predict bearing resistance for all diesel hammers at bearing resistances of less than 700 kips and significantly under-predicts resistance for all diesel hammers at resistances of 1000 kips or more. However, for steam hammers, the FHWA Modified Gates formula consistently under-predicts bearing resistance at all values of bearing resistance.

As shown in Figure 7, the ENR formula significantly over-predicts bearing resistance in most cases, and the degree of scatter in the data is visually greater than is the case for the WSDOT and FHWA Modified gates formulae. Note that to keep the axis in Figure 7 the same as for the four previous figures, a significant number of data in which the predicted resistance was greater than 3000 kips are not shown. Also note that because the ENR formula was derived as an allowable stress design method, a factor of safety of 6.0 was built into the formula. The factor of safety was removed from the ENR formula to produce the plot of nominal resistance in Figure 7.

When the plots in which developed energy is used to estimate the pile bearing resistance are compared to the plots in which rated energy is used to estimate pile bearing resistance, it appears that the bearing resistance prediction is less conservative and the scatter is slightly greater, though the differences are minimal.

Figures 3 and 4 suggest that the WSDOT driving formula remains reasonably accurate up to nominal bearing resistances of approximately 1200 kips. For diesel hammers, it is possible that this limit could be stretched up to approximately 1500 kips, although the available data become rather sparse at this high a bearing resistance.

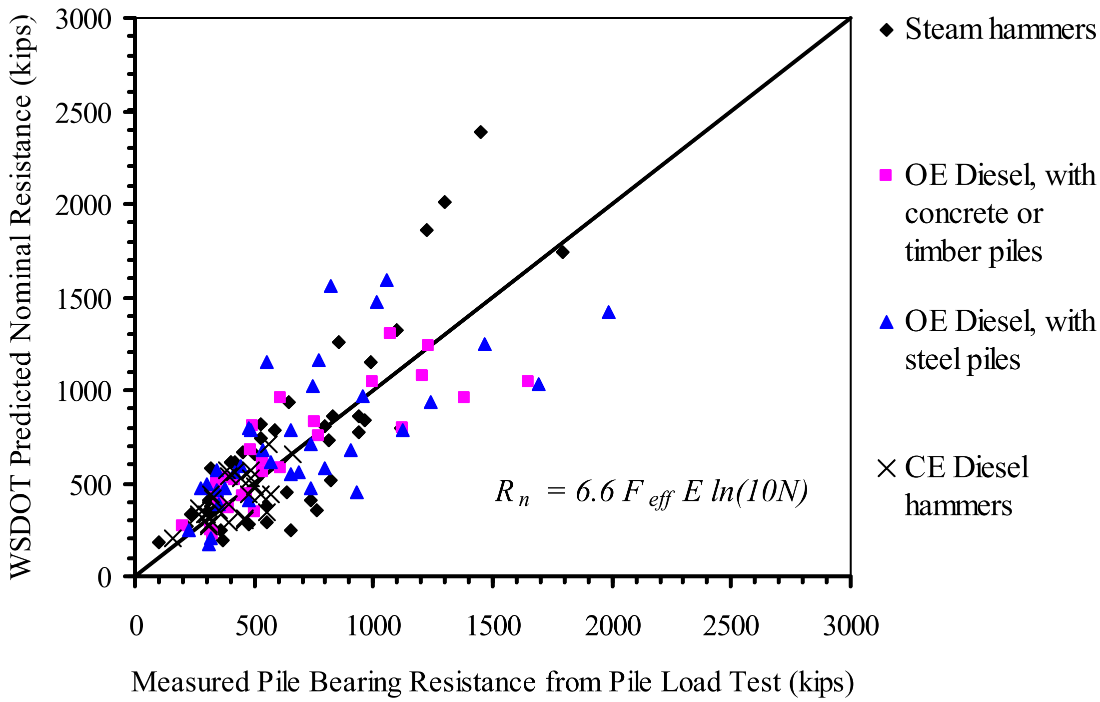

Figure 3. Predicted nominal versus measured pile bearing resistance for the WSDOT pile driving formula, based on developed energy.

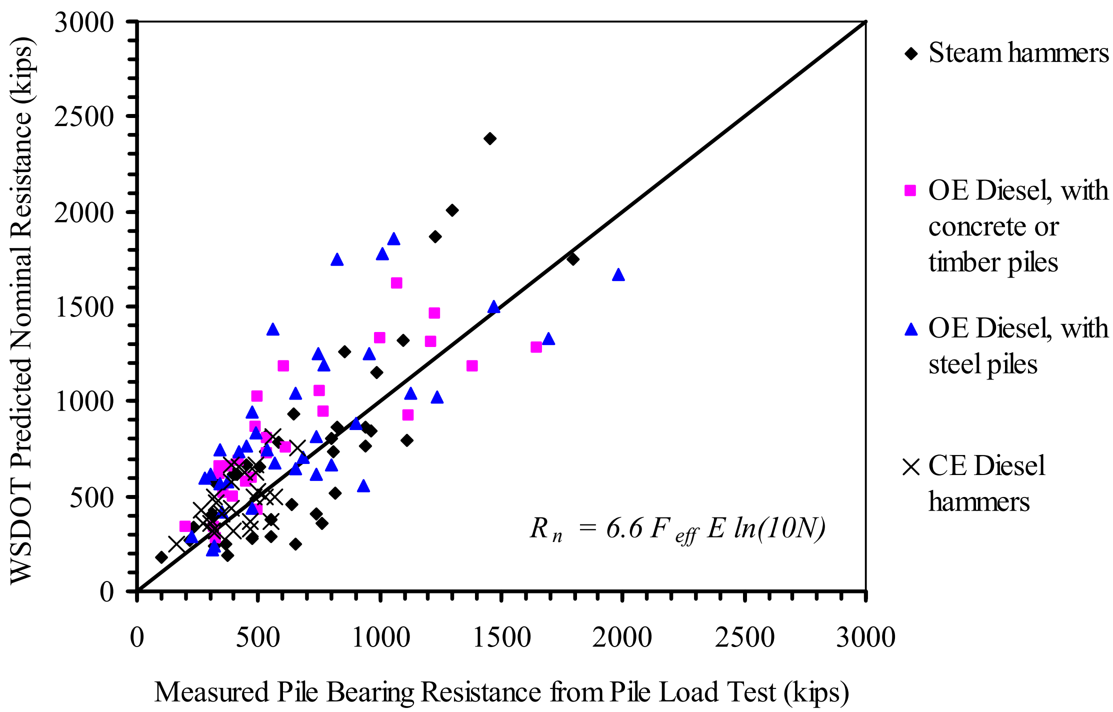

Figure 4. Predicted nominal versus measured pile bearing resistance for the WSDOT pile driving formula, based on rated energy.

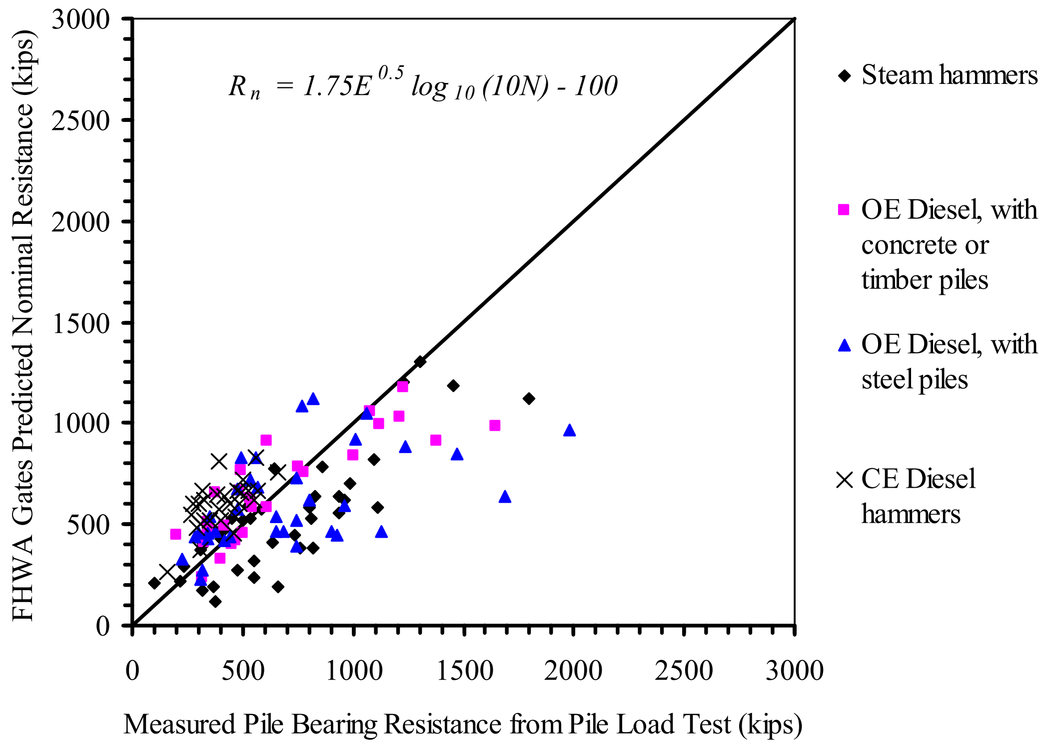

Figure 5. Predicted nominal versus measured pile bearing resistance for the FHWA Modified Gates driving formula, based on developed energy.

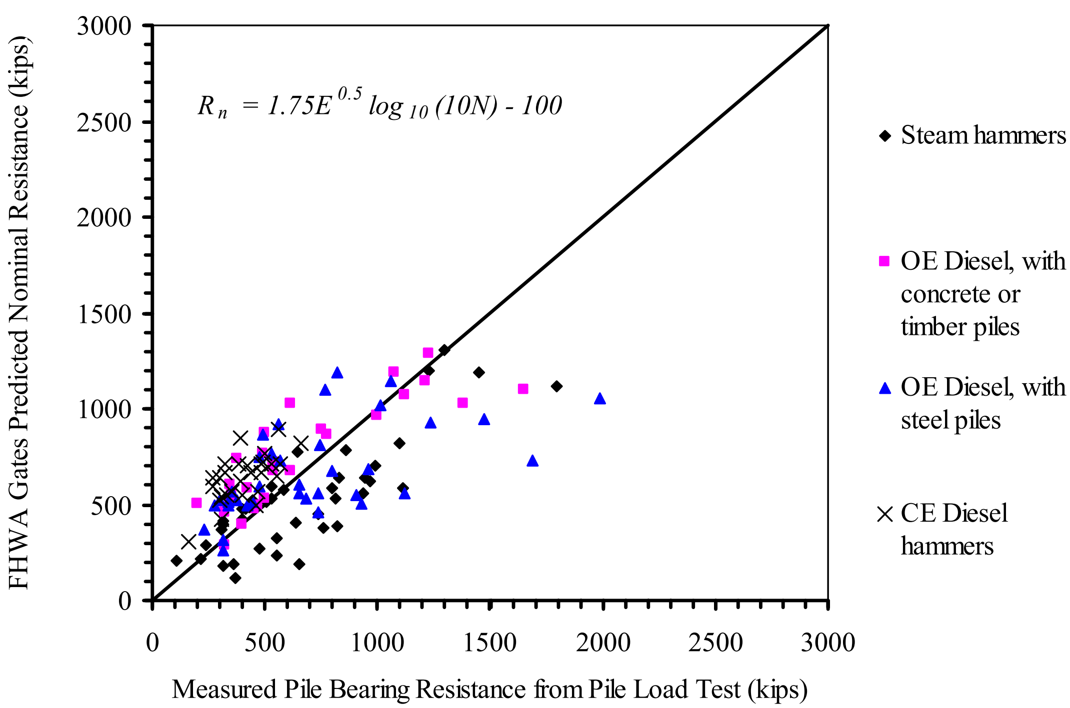

Figure 6. Predicted nominal versus measured pile bearing resistance for the FHWA Modified Gates driving formula, based on rated energy.

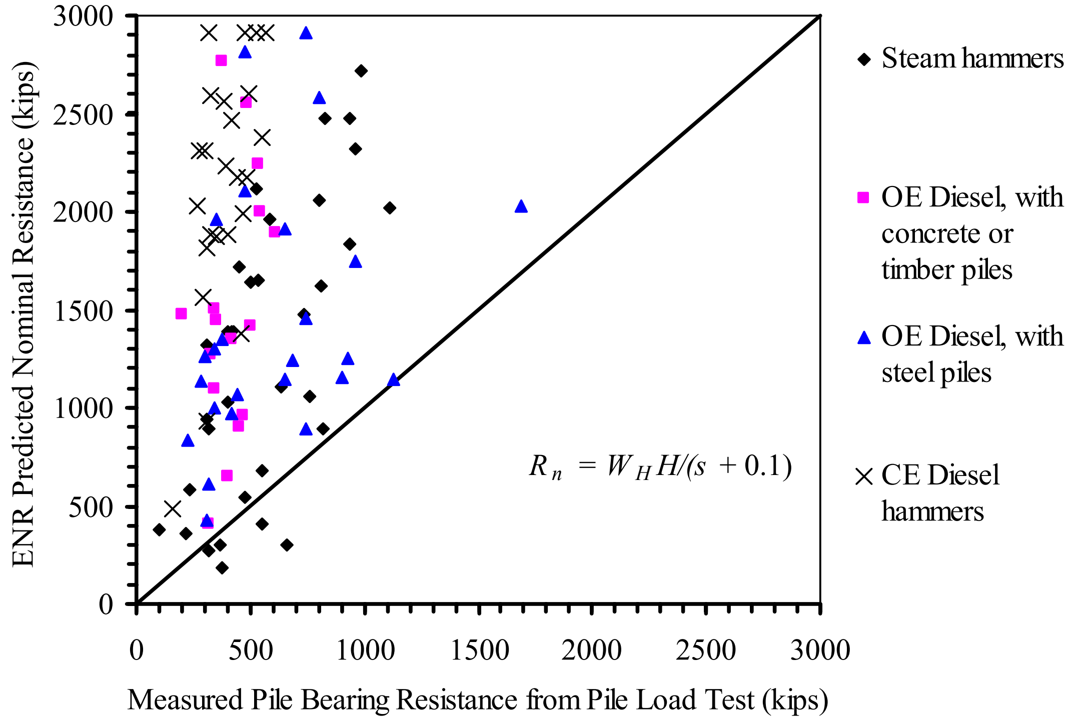

Figure 7. Predicted nominal versus measured pile bearing resistance for the ENR driving formula, based on developed energy.

The WSDOT pile driving formula, as well as other pile driving formulae, was calibrated to N values obtained at the end of driving (EOD). Because the pile nominal resistance obtained from pile load tests is typically obtained days, if not weeks, after the pile has been driven, the gain in pile resistance that typically occurs with time (i.e., soil setup) is, in effect, correlated to the EOD N value through the driving formula. That is, the driving formula assumes that an "average" amount of setup will occur after EOD when the pile nominal resistance is determined from the formula. On the basis of the available database (the EOD data in Table 1 and BOR data from Paikowsky et al. 2004), and utilizing the available CAPWAP/TEPWAP data obtained at EOD and at BOR for specific sites, the average amount of pile resistance setup inherent in the WSDOT pile driving formula prediction is approximately 30 to 70 percent. The observed setup based on EOD and BOR data pairs at the available sites in the database is summarized for various database subgroups in **Table 3**. Note that five of the sites reported in the database had an unusual amount of setup, most likely because of high plasticity clay along the sides of the pile. These five sites were excluded from some of the groupings so that a truer average could be obtained for the overall grouping.

Soil setup was also estimated by using the driving resistance N and the WSDOT pile driving formula. Note that the driving formula did not indicate as much setup as did the CAPWAP/TEPWAP measurements, indicating that increased driving resistance is not the only contributor to the indication of soil setup.

Table 3. Soil setup observed for the case histories reported by Paikowsky et al. (2004).

Database Subgroup	**Setup Factor Based on CAPWAP/TEPWAP Measurements (BOR Resistance/EOD Resistance)**	**Setup Factor Based on Driving Resistance, N, Using WSDOT Formula (BOR Resistance/EOD Resistance)**
Side resistance derived in general from sands, silty sands, or tills	1.30	1.11
Side resistance derived in general from sandy silts and clays	1.72	1.43
Side resistance derived in general from high plasticity soft to medium clays	6.29	2.03
All concrete and timber piles, excluding high plasticity clay sites (5 sites)	1.64	1.26
All steel piles (no high plasticity clay sites)	1.45	1.28
All steam hammer data (no high plasticity clay sites)	1.84	1.44
All open ended diesel hammer/steel pile data (no high plasticity clay sites)	1.30	1.22
All open ended diesel hammer/concrete pile data, excluding high plasticity clay sites (5 sites)	1.42	1.11
All Closed ended diesel hammer (no high plasticity clay sites) – note data where a direct comparison between EOD and BOR resistance were very limited for this category	1.11	1.09

An additional check on the development of the WSDOT pile driving formula was conducted. Because the Wave Equation is typically used to assess the acceptability of the contractor's pile-hammer system for piles with nominal resistances of 300 tons or more per the WSDOT Standard Specifications (2004), the WSDOT driving formula should produce a final driving criterion that is consistent with the pile drivability analysis conducted to approve the hammer system for the project using the Wave Equation. With regard to the relationship between hammer acceptance and the driving criteria, the following two scenarios would be undesirable:

- To allow the contractor to use a hammer that would not be capable of driving the pile to the bearing determined by the WSDOT driving formula, and
- To force the contractor to select an overly robust pile-hammer system that would result in a very low driving resistance to obtain the bearing determined by the WSDOT formula.

When the wave equation is used to approve the contractor's pile-hammer system, it is preferable that dynamic measurements with signal matching be used to develop the pile resistance acceptance criteria. However, this is not always practical in terms of cost or potential time delays, especially for smaller projects. Therefore, in many cases, the driving formula would still need to be used.

It must first be recognized that the Wave Equation is a theoretical approach to estimating pile resistance and drivability and has not been empirically adjusted to full-scale pile load test results. Because of this, the Wave Equation does not inherently account for soil setup. The Wave Equation must be run for the selected hammer/pile combination, and then the nominal resistance values that correspond to the driving resistance values (*N*) output by the wave equation must be increased by the estimated setup factor "after-the-fact." Because of this, it is unrealistic to expect that the WSDOT driving formula will closely match the Wave Equation results for the same size hammer. The wave equation can also take into account many variables that a driving formula is simply incapable of directly addressing. All that can be hoped for is that overall, the WSDOT driving formula will provide an approximate match to the Wave Equation results for the same size hammer, once soil setup is taken into account.

To this end, Wave Equation analyses were conducted with GRLWEAP (1996) and compared to the bearing resistance predicted by the WSDOT driving formula. For the Wave Equation analyses, a range of situations regarding the pile length (60 to 120 ft), diameter (12 to 24 inches), cross-sectional area (0.250- to 0.438-inch pipe pile walls), and skin friction distribution (triangular, 20 to 80 percent of the resistance) were used for each hammer evaluated. Steel piles were primarily evaluated, since steel piles are by far the most common in WSDOT practice. The standard input as described in the WSDOT Standard Specifications for Construction (2004), Section 6-05.3(9)A, was used for these analyses, as well as the standard hammer input and standard soil quake and damping parameters recommended by the program.

Sample results are shown in figures 8 through 10. In each figure, “least conservative” in the Wave Equation analyses refers to 18-in. diameter, 60-ft length, 0.438-in. wall, 80 percent skin friction, and steel pipe piles, and “most conservative” refers to 18-in. diameter, 120-ft length, 0.375-in. wall, 20 percent skin friction, and steel pipe piles. A setup factor of 1.3 was used in all the Wave Equation analyses, which is representative of a silty sand typical in Washington for pile foundation situations.

On the basis of these figures and similar analyses that were conducted, the WSDOT driving formula tends to be a little less conservative than the wave equation regarding the driving resistance, N, needed to obtain a given nominal bearing resistance, R_n. However, only a nominal amount of soil setup was applied to the wave equation results. Had a soil setup factor of 1.5 been used, which would have been more consistent with the database used to derive the WSDOT formula, the WSDOT driving formula would have been more conservative than the wave equation results. This highlights the point that assumptions regarding soil setup are critical to a comparison between a driving formula and the Wave Equation. If the Wave Equation is used for hammer approval, but the WSDOT driving formula is specified for pile bearing verification, the contractor should expect that the hammer could be oversized to drive the pile to the specified bearing resistance, if soil setup is not considered in the Wave Equation analysis. From a geotechnical design standpoint, this situation is more desirable than the case in which the hammer pile system is undersized to achieve the desired bearing resistance and maximum anticipated driving resistance to reach the minimum penetration specified.

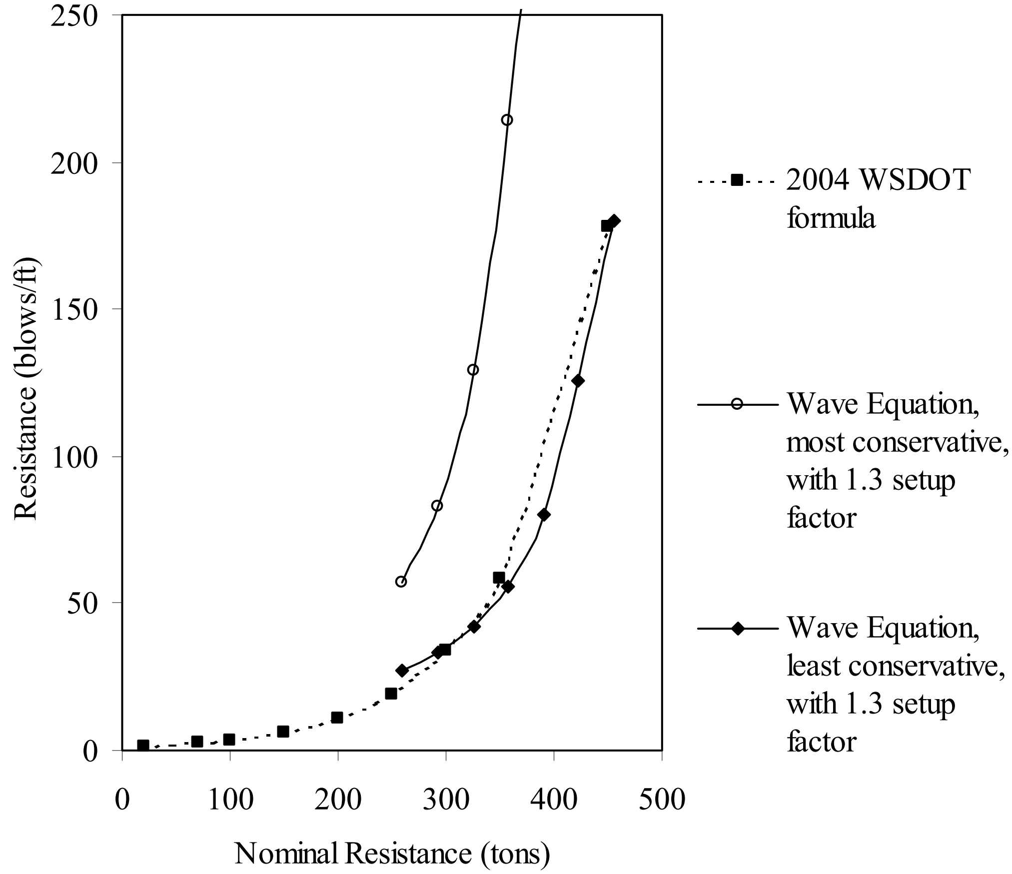

Figure 8. Comparison of Wave Equation and WSDOT driving formula for 18-inch diameter steel piles using a steam hammer with a rated energy of 25 ft-tons.

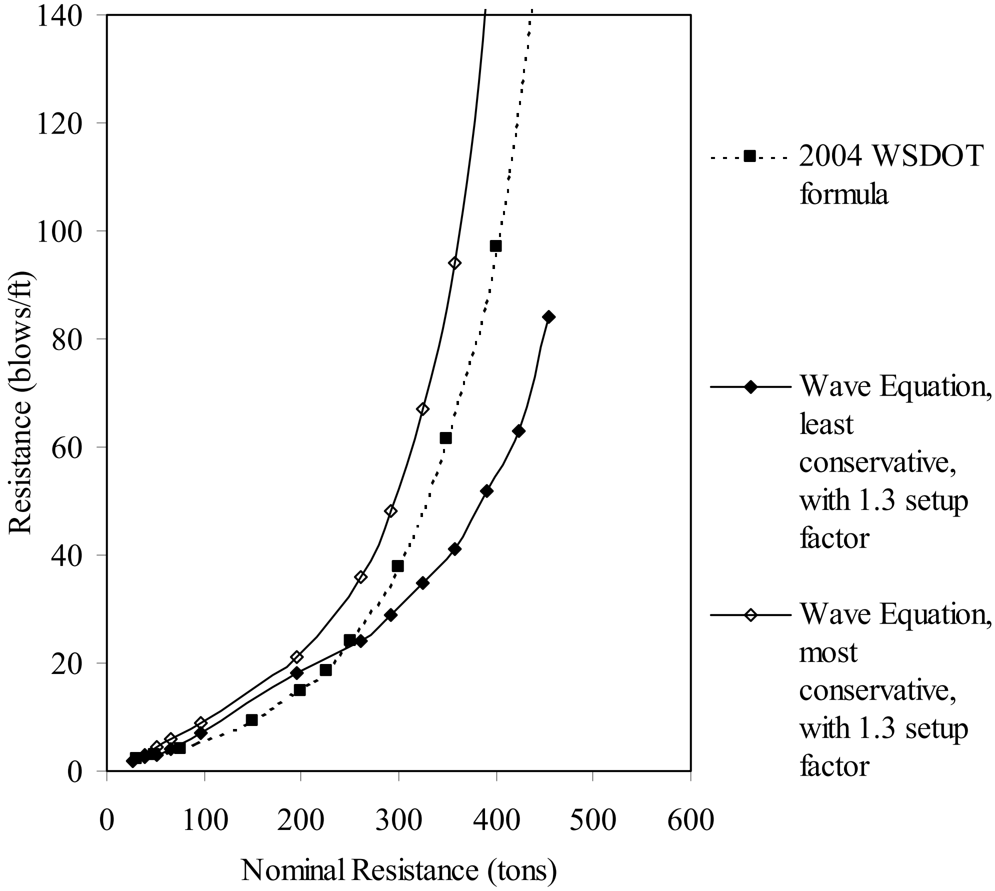

Figure 9. Comparison of Wave Equation and WSDOT driving formula for 18-inch diameter steel piles using an open-ended diesel hammer with a rated energy of 27.5 ft-tons.

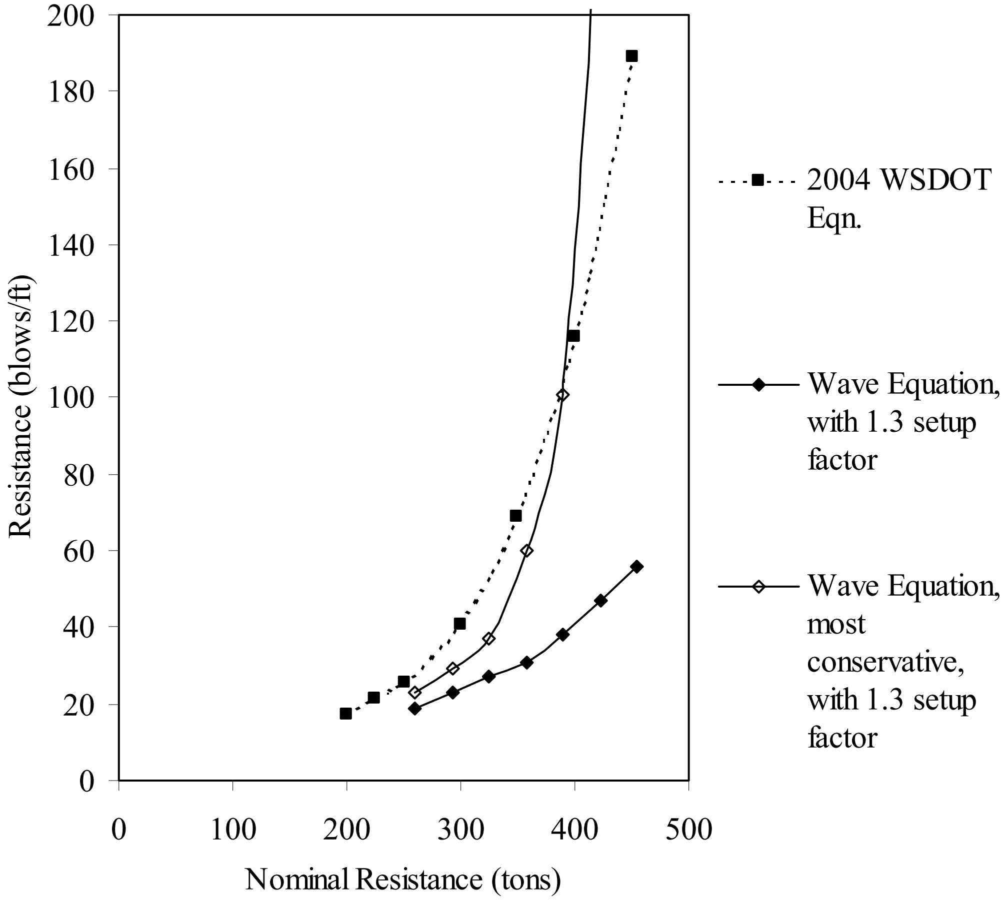

Figure 10. Comparison of Wave Equation and WSDOT driving formula for 18-inch diameter steel piles using a closed ended diesel hammer with a rated energy of 36 ft-tons.

If soil setup (or relaxation) is an issue or highly uncertain, or if relatively high nominal resistance piles are needed (i.e., nominal values of greater than 1200 kips), dynamic measurements with signal matching analysis should be conducted. Based on the data provided in Table 1, plots of predicted versus measured pile nominal resistances, when dynamic measurements with signal matching analysis (e.g., CAPWAP) are used to estimate pile bearing resistance, are provided in figures 11 through 13.

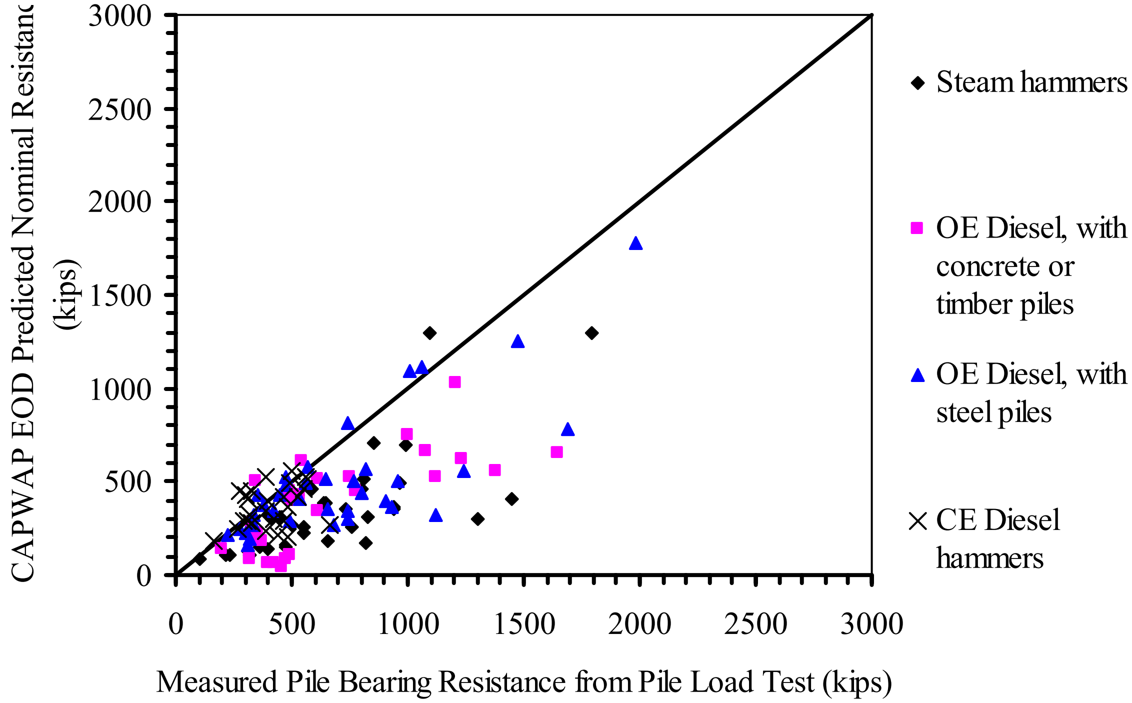

Figure 11. Predicted nominal versus measured pile bearing resistance for CAPWAP/TEPWAP results at EOD.

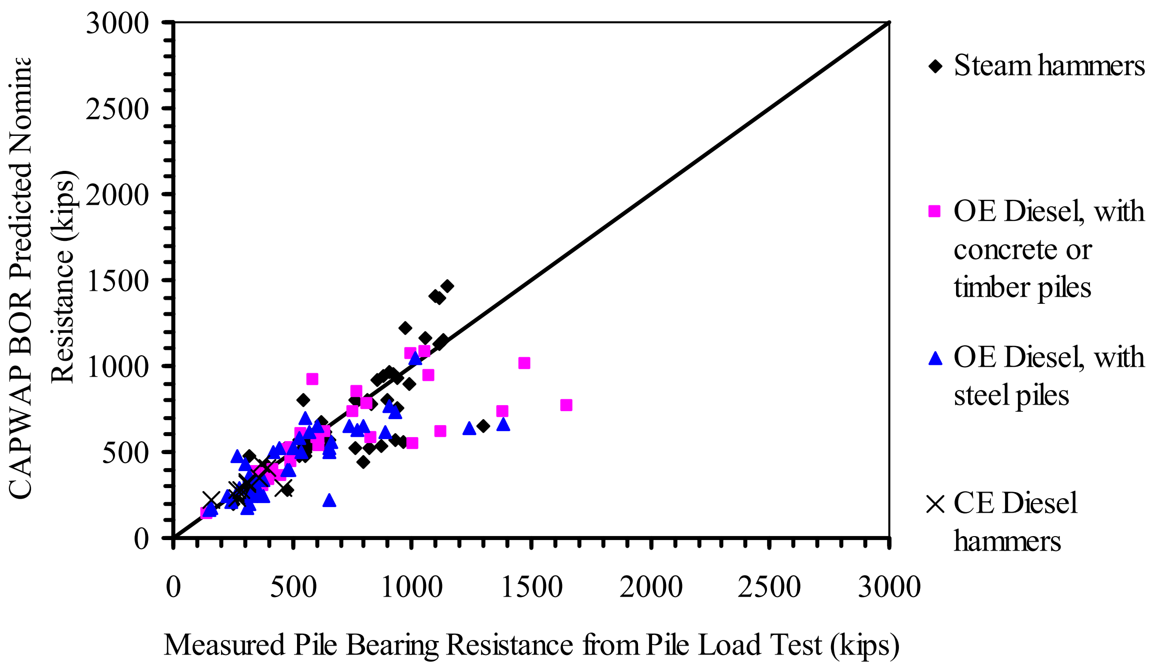

Figure 12. Predicted nominal versus measured pile bearing resistance for CAPWAP/TEPWAP results at BOR (the data used to produce this figure are in Paikowsky et al. 2004).

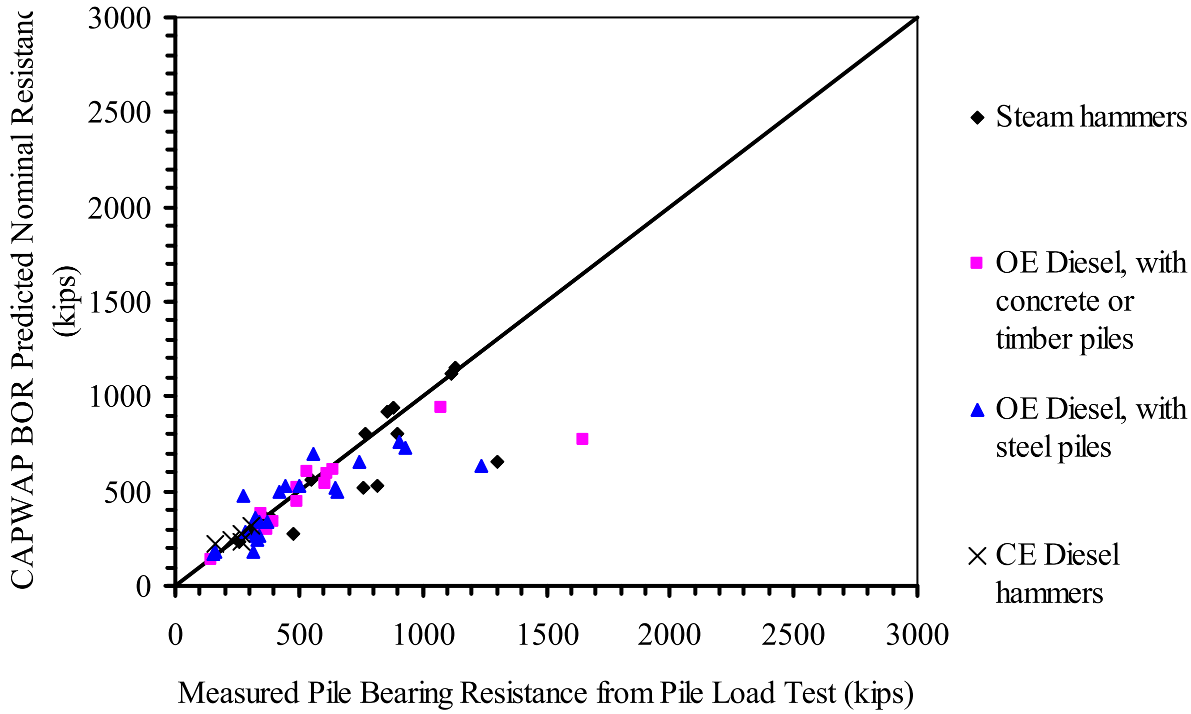

Figure 13. Predicted nominal versus measured pile bearing resistance for CAPWAP/TEPWAP results at BOR, but only for final driving resistances of 8 blows per inch or less (the data used to produce this figure are in Paikowsky et al. 2004).

As can be observed from these figures, the CAPWAP/TEPWAP method provides an overly conservative estimate of the pile bearing resistance if the analysis is conducted at EOD conditions. This approach is still consistently conservative if it is used at BOR conditions, but it is most accurate if it is used to estimate resistance at BOR when the driving resistance, *N*, is 8 blows/inch or less. Since this method has no built-in soil setup, this method works best if the pile is allowed to set up before a final bearing resistance is determined. Therefore, it is recommended that this method be used primarily at BOR, unless it is known that soil setup (as well as relaxation) will not be an issue.

STATISTICAL ANALYSIS AND LRFD CALIBRATION

A key aspect of Load and Resistance Factor (LRFD) foundation design is the selection of load and resistance factors to account for uncertainty in the design. The uncertainty in the driving formula, or other pile bearing resistance verification method, must be taken into account during foundation design, as the uncertainty in the pile bearing resistance verification method controls the pile foundation design reliability (Allen, 2005). Reliability theory can be used to calibrate load and resistance factors so that a consistent level of reliability is obtained. A complete description of the calibration process for estimating load and resistance factors using reliability theory is provided by Allen et al. (in press). Furthermore, important background regarding the development of the current resistance factors for foundation design is provided by Allen (2005).

Using the procedures provided by Allen et al. (in press) and the database provided in Table 1, a statistical analysis of the ratio of measured to predicted bearing resistance values (i.e., the bias) was conducted. To characterize the pile bearing resistance data, the bias (X) values were plotted against the inverse of the standard normal cumulative distribution function, CDF (i.e., the standard normal variable or variate, or z), for each data point. This was accomplished by sorting the bias values in the data set from lowest to highest, calculating the probability associated with each bias value in the cumulative distribution as $i/(n+1)$, and then calculating z in Excel as:

$$z = \text{NORMSINV}(i/(n+1)) \qquad (6)$$

where: i = the rank of each data point as sorted, and

n	=	the total number of points in the data set.

Standard normal variable plots were used to determine the pile bearing resistance CDFs and their characteristics. This type of plot is essentially the equivalent of plotting the bias values on normal probability paper. An important property of a CDF plot is that data that are normally distributed plot as a straight line with a slope equal to $1/\sigma$, where σ is the standard deviation, and the horizontal (bias) axis intercept is equal to the mean, μ_s. However, lognormally distributed data plot as a curve. Note that a lognormally distributed dataset can be made to plot as a straight line by plotting the natural logarithm of each data point.

Figures 14 and 15 provide the CDFs for the WSDOT formula based on developed and rated energy, respectively. These two figures show that the theoretical lognormal CDFs for these datasets provide a much better fit than do the theoretical normal CDFs.

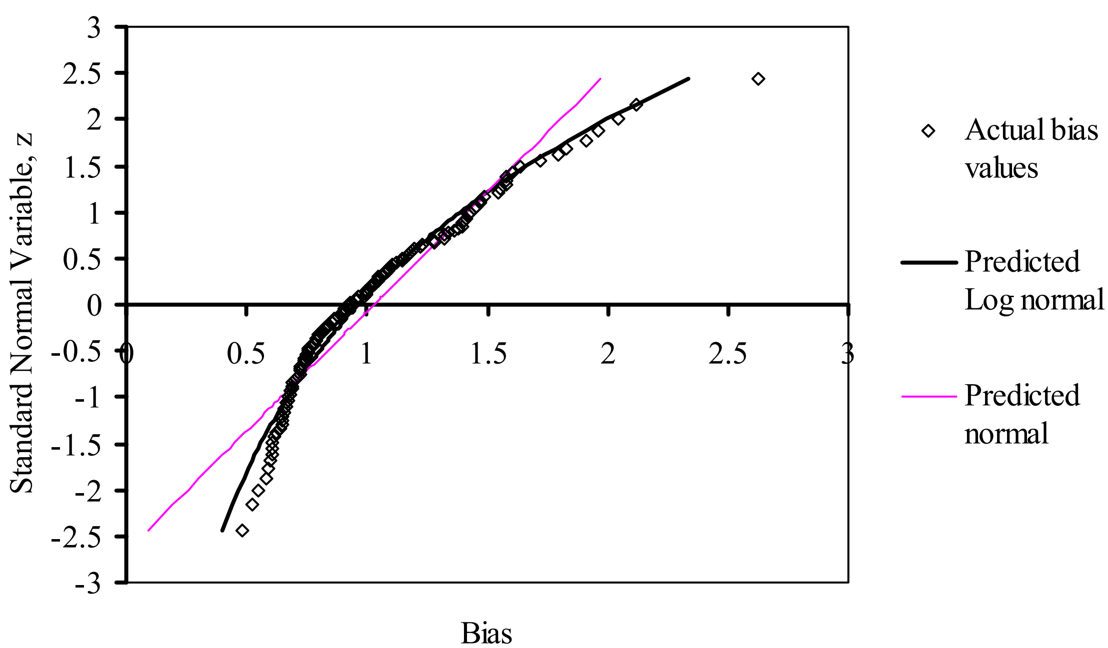

Figure 14. CDF for WSDOT pile driving formula bearing resistance bias values, in which the estimated developed hammer energy is used to predict nominal pile bearing resistance.

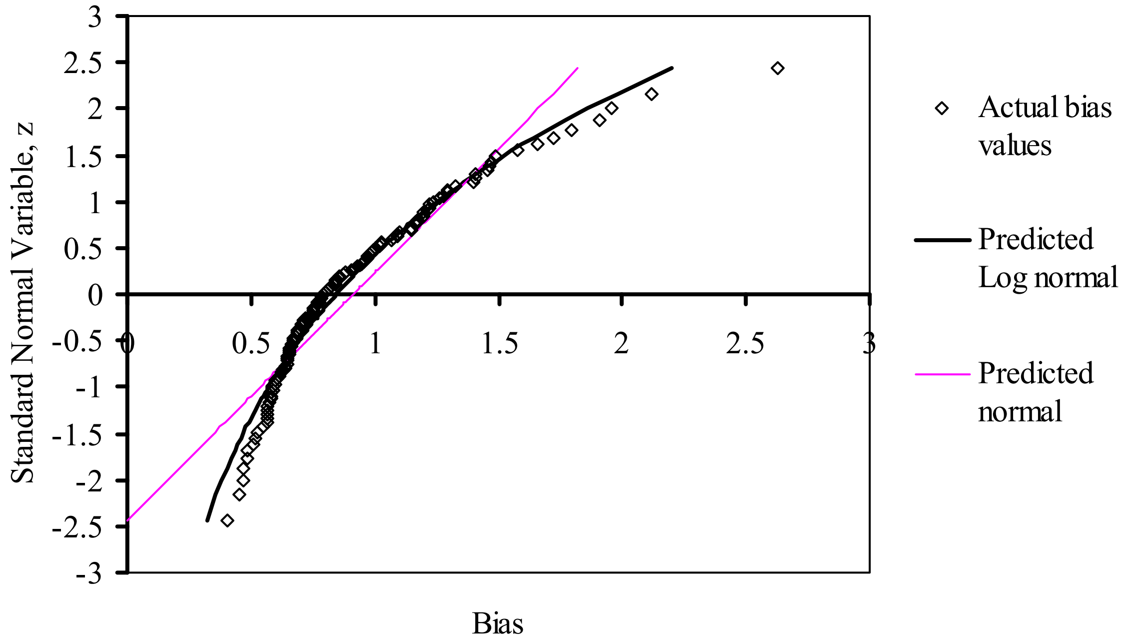

Figure 15. CDF for WSDOT pile driving formula bearing resistance bias values, in which the rated hammer energy is used to predict nominal pile bearing resistance.

For resistance factor calibration purposes, when reliability theory is used, the lower tail of the resistance CDF is critical to the accuracy of the calibration. The upper tail really has no influence on the end result of the calibration. The opposite is true of the load CDF, primarily because, by design, the resistance is made to be greater than the load to provide a safe design. Also note that for the lower tail, CDFs that are located to the left of the data points in the tail region are more conservative for reliability analysis than a CDF that fits exactly on the data in the tail. Again, for the load distribution, the opposite is true, in that CDFs located to the right of the actual data are more conservative for reliability analysis.

Basic load and resistance factor design (LRFD) is summarized in Equation 7:

$$\sum \gamma_i Q_{ni} \leq \varphi R_n \tag{7}$$

where: γ_i = a load factor applicable to a specific load type, Qn_i; the summation of $\gamma_i Qn_i$ terms is the total factored load for the load group applicable to the limit state being considered;

φ = the resistance factor; and

R_n = the nominal unfactored (design) resistance available (either ultimate or the resistance available at a given deformation).

Equation 7 is the design equation, but it can serve as the basis for the development of a limit state equation that can be used for calibration purposes. If there is only one load component, Q_n, then Equation 7 can be shown as:

$$\varphi_R R_n - \gamma_Q Q_n \geq 0 \tag{8}$$

The limit state equation that corresponds to Equation 8 is as follows:

$$g = R - Q \geq 0 \tag{9}$$

where g is a random variable representing the safety margin, R is a random variable representing resistance, Q is a random variable representing load, R_n is the nominal (design) resistance value, Q_n is the nominal (design) load value, and φ_R and γ_Q are resistance and load factors, respectively.

This concept of equations 7 through 9 and the influence of the distribution tails on the calibration are illustrated in Figure 16, which shows conceptual plots of load and resistance distributions, as well as the distribution of the safety margin, g, that results from the load and resistance distributions. The magnitudes of the load and resistance factors used in the design equation are established to yield the desired reliability index, β, which can be related to the probability of failure, P_f. As can be observed from this figure, it is the overlap in the load and resistance distributions that influences the reliability index and the probability of failure, and the opposite tails of the distributions have no influence on P_f.

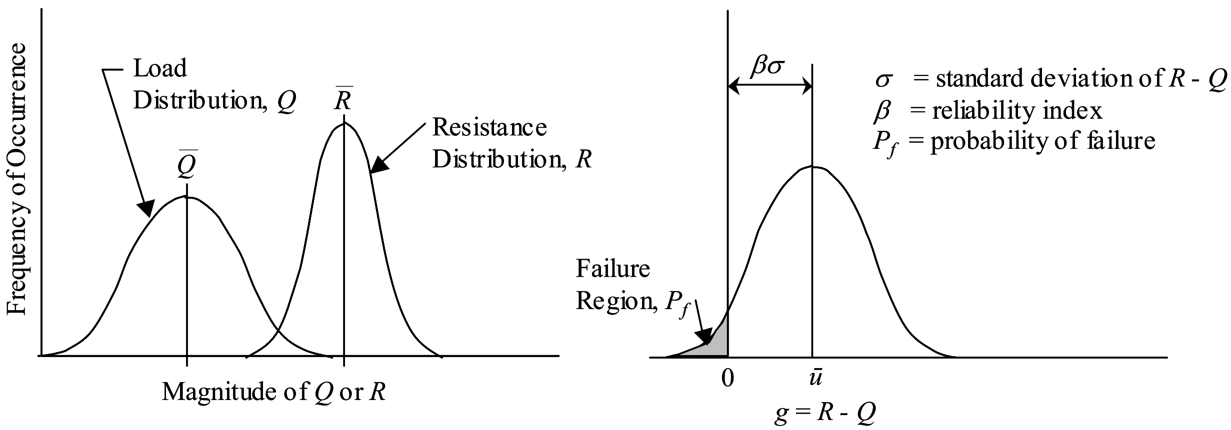

Figure 16. Probability of failure and reliability index (adapted from Withiam et al. 1998).

This concept leads to the practice of making sure that the statistical parameters selected result in the best fit possible in the tail region, termed here as the "best fit to tail." For the data shown in figures 14 and 15, the theoretical distribution that best fits the tail region is illustrated in figures 17 and 18.

Similar analyses were conducted for the FHWA Modified Gates and ENR formulae, and for the CAPWAP/TEPWAP bearing resistance predictions. The statistical parameters obtained from these analyses are summarized in Table 4. Note that these statistical analyses excluded the hydraulic and drop hammer data because of the paucity of data for those two hammer types. No outlier data points were removed from any of the datasets analyzed to produce the statistics shown in the table. Also note that only normal distribution statistics are presented in the table.

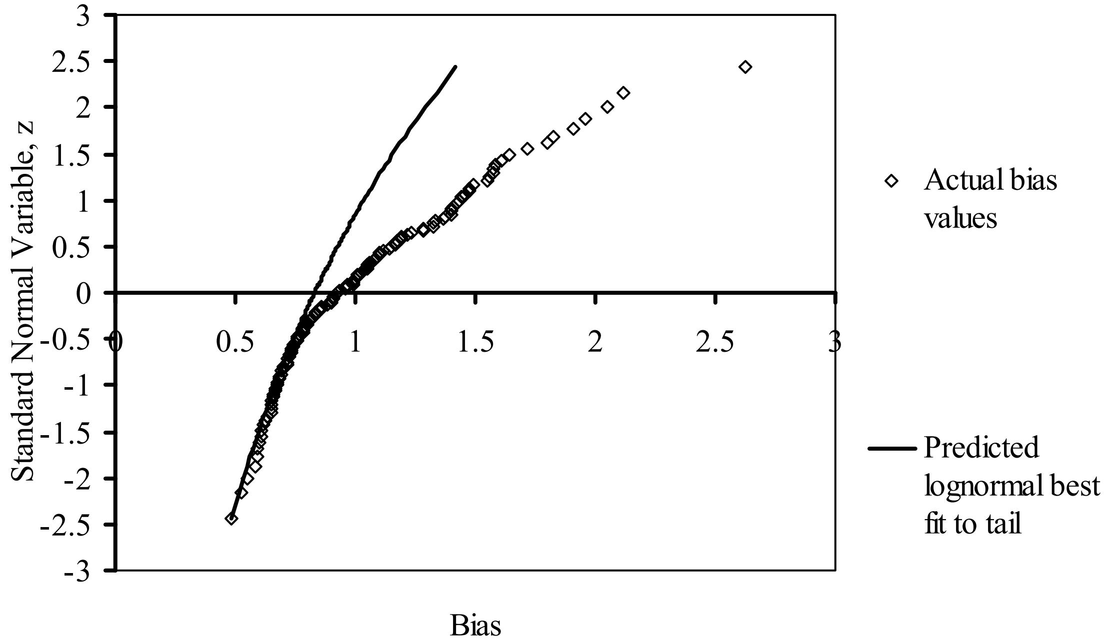

Figure 17. Best fit to tail CDF for WSDOT pile driving formula bearing resistance bias values, in which the estimated developed hammer energy is used to predict nominal pile bearing resistance.

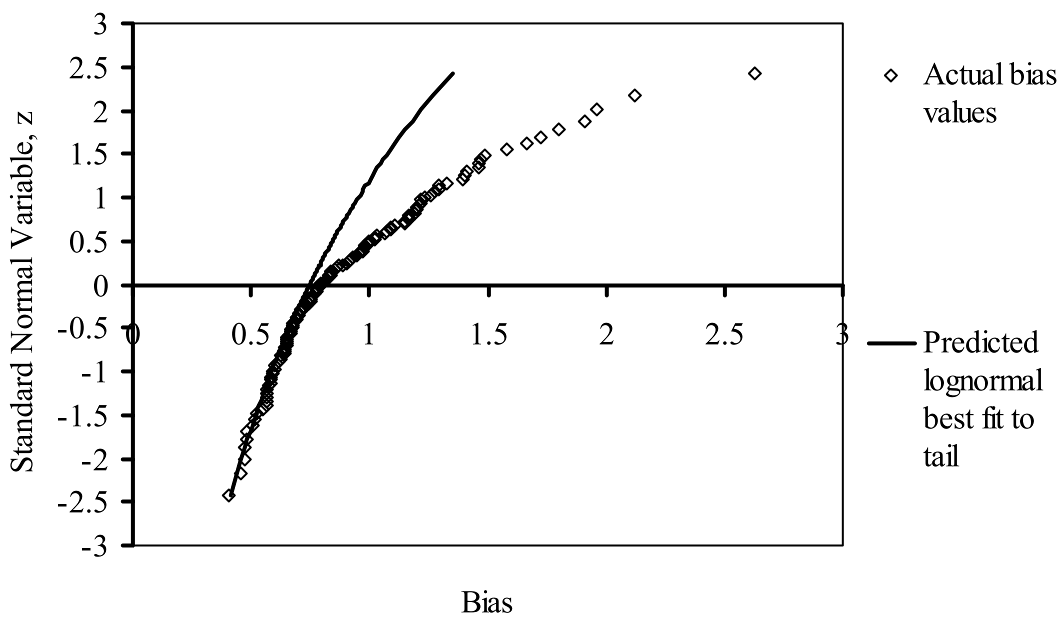

Figure 18. Best fit to tail CDF for WSDOT pile driving formula bearing resistance bias values, in which the rated hammer energy is used to predict nominal pile bearing resistance.

Table 4. Summary of resistance statistics used for calibration of resistance factors.

Pile Capacity Prediction Method (all EOD, using developed energy)	Normal Distribution Parameters - All data				Parameters for Best Fit to Tail		
	n	Mean of Bias Values, λ	COV of Bias Values	Actual Distribution Type	Mean of Bias Values, λ	COV of Bias Values	Distribution Type
WSDOT Formula (developed energy)	131	1.03	0.377	Lognormal	0.850	0.224	Lognormal
WSDOT Formula (rated energy)	131	0.913	0.410	Lognormal	0.770	0.247	Lognormal
WSDOT Formula (developed and rated energy, steam hammers only, with maximum nominal resistance of 1200 kips)	34	1.08	0.458	Lognormal	0.790	0.215	Lognormal
FHWA Modified Gates Formula (estimated developed energy)	131	1.10	0.485	Lognormal	0.970	0.356	Lognormal
FHWA Modified Gates Formula (rated energy)	131	1.03	0.506	Lognormal	0.930	0.376	Lognormal
ENR with FS of 6 removed (estimated developed energy)	131	0.370	0.870	Lognormal	0.280	0.464	Lognormal
ENR with FS of 6 removed (rated energy)	131	0.332	0.949	Lognormal	0.230	0.435	Lognormal
CAPWAP (EOD all data)	126	1.87	0.701	Lognormal	1.54	0.390	Lognormal
CAPWAP (EOD with N < 8 bpi)	83	2.05	0.725	Lognormal	1.50	0.313	Lognormal
CAPWAP (BOR all data)	145	1.19	0.334	Lognormal	1.10	0.245	Lognormal
CAPWAP (BOR with N < 8 bpi)	56	1.13	0.270	Lognormal	1.03	0.204	Lognormal

If the distribution is actually lognormal, the lognormal parameters can be calculated theoretically using the following equations from Benjamin and Cornell (1970):

$$\mu_{ln} = LN(\mu_s) - 0.5\sigma_{ln}^2 \tag{10}$$

$$\sigma_{ln} = (LN((\sigma/\mu_s)^2 + 1))^{0.5} \tag{11}$$

Note that LN is the natural logarithm (base e). From these parameters, the theoretical normal (Equation 12) and lognormal (Equation 13) distribution of the bias as a function of z can be calculated as follows:

$$\text{Bias} = X = \lambda + \sigma z \tag{12}$$

$$\text{Bias} = X = \text{EXP}(\mu_{ln} + \sigma_{ln} z) \tag{13}$$

Table 4 illustrates that the variability in each of these methods is significantly greater at higher bias values than is the case at lower bias values. The "best fit to tail" statistics represent the variability for low bias values, since the lower tail contains all of the low bias values. Where the bias is less than 1.0 (i.e., where the measured resistance is less than the predicted resistance, which is non-conservative), the WSDOT formula is significantly more consistent and therefore reliable, based on these statistics, than the other driving formulae. This gives the WSDOT formula the advantage regarding the magnitude of the resistance factor needed relative to the other methods.

The reliability of the design is dependent on both the load and resistance factors used, and the statistical parameters associated with those factors. While calibration can be conducted to determine the magnitude of both the load and resistance factors, for this study the load factors were held constant, and the magnitude of the resistance factor that yielded the desired level of reliability determined. The load factors recommended in the current AASHTO specifications (AASHTO 2004) were used for the calibrations conducted as a part of this study. These load factors are provided in Table 5. The purpose, therefore, of these calibrations was to determine the resistance factor needed to achieve the target β value (i.e., desired level of reliability), assuming that the load factors shown in Table 5 are used.

The load statistics needed for the reliability analysis are provided in Table 5. These load statistics were developed and reported by Nowak (1999). Only the summary statistics are provided here. Dead load and live load are the typical load components applicable for a pile foundation design. For foundation design, it can be assumed that the live load transmitted to the pile top includes dynamic load allowance (AASHTO 2004). The live load statistics provided below assume that the live load includes dynamic load allowance. The dead load statistics assume that the primary source of dead load is from cast-in-place concrete structure members. Because the statistics and load factors for dead load and live load are different, the calibration results will depend on the ratio of dead load to live load. Because of this, dead load to live load ratios ranging from 2 to 5, which are typical for bridges and similar structures, were investigated.

Table 5. Load statistics used for the calibration of resistance factors (from Nowak 1999).

Load Type	**Mean of Bias Values**	**COV of Bias Values**	**Distribution Type**	**Load Factor Used**
Dead load	1.05	0.10	Normal	1.25
Live load	1.15	0.18	Normal	1.75

Allen et al. (in press) and Allen (2005) discussed the determination of the appropriate P_f and β to use for the reliability analysis. Based this work and work by others (e.g., Paikowsky et al. 2004), the reliability of the pile group is typically much greater than that of the individual pile, considering the redundancy inherent in pile foundations, and considering that the pile bearing resistance required for all piles in the group is typically based on the most heavily loaded pile. In general, for structural design, a target reliability index, β_τ, of 3.5 (an approximate P_f of 1 in 5,000) is used. For the pile group, this β_τ can be achieved if the reliability index of the individual pile is 2.3 (an approximate P_f of 1 in 100), provided that the group size is greater than four piles. Paikowsky, et al. (2004) indicated that for pile groups consisting of four piles or less, a β of 3.0 (an approximate P_f of 1 in 1000) should be targeted to address the lack of redundancy.

Monte Carlo simulation, as described by Allen et al. (in press), was used to perform the reliability analysis to estimate β and the resistance factor needed to achieve the target value of β (i.e., either 2.3 or 3.0). The load factors currently prescribed in the AASHTO LRFD Design Specifications (AASHTO 2004) provided in Table 5**,** in combination with the resistance factors and CDFs summarized in Table 4**,** were used in this analysis. The simulation was carried out by generating 10,000 values of load and resistance using a random number generator, and by subtracting the random load from the random resistance values to obtain 10,000 values of the margin of safety, *g*.

An example of the Monte Carlo results, in this case for the WSDOT formula using the estimated developed hammer energy, assuming a resistance factor of 0.60, is provided in Figure 19. The β value obtained is equal to the negative of the intercept of the safety margin curve (*g*) with the standard normal variable axis.

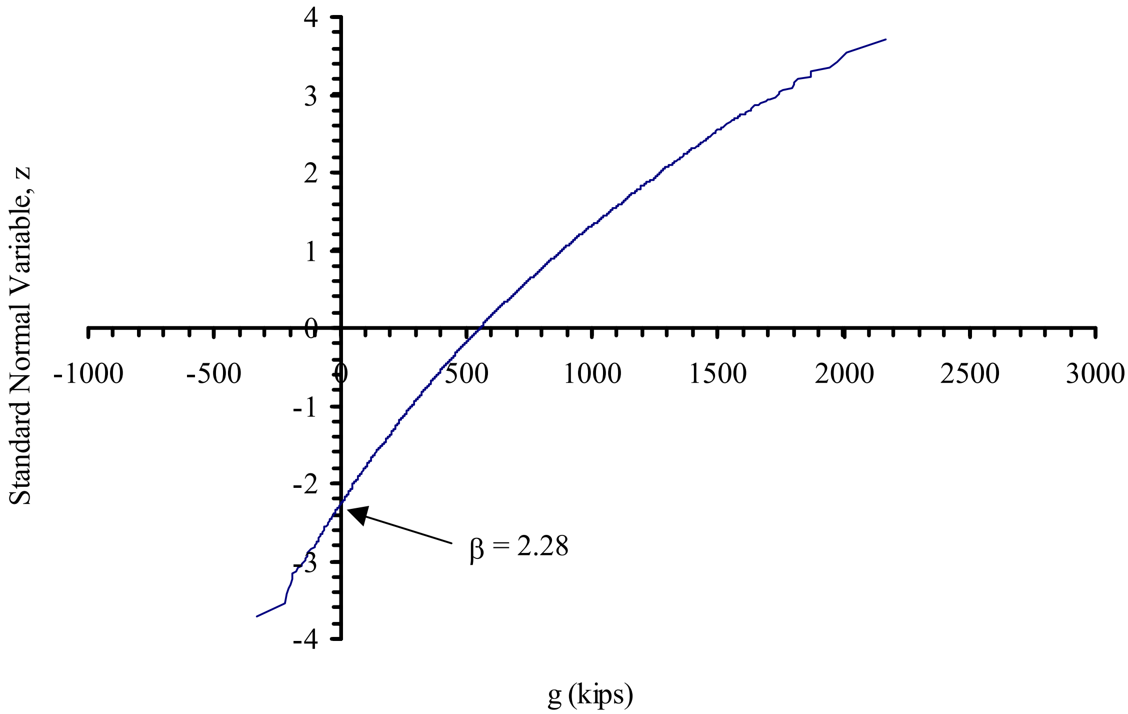

Figure 19. Monte Carlo simulation results for the WSDOT formula, using the estimated developed energy, a dead load to live load ratio of 3, and a resistance factor of 0.60.

Similar analyses were conducted for various combinations of dead and live load, and for each of the driving formulas and CAPWAP/TEPWAP analyses results.

CALIBRATION RESULTS

The calibration results are summarized in Table 6. The relative degree of conservatism for each formula/method can be assessed by dividing the resistance factor by the bias for the dataset (third column in Table 6). In general, it is desirable to keep the degree of conservatism in the design method as low as possible. Therefore, the lower the relative conservatism ratio (see Table 6), the more cost effective the design method is capable of being. As shown in the table, the WSDOT formula is the least conservative method of the driving formulae, and the CAPWAP/TEPWAP method is the least conservative method overall if used at BOR.

Table 6 also shows that there is a significant difference in the resistance factor required for small pile groups that lack redundancy. In general, the resistance factor required for small pile groups (i.e., less than five piles in the group) is approximately 80 percent of the resistance factor required for larger pile groups, using the target β values discussed previously.

Resistance factors were determined for the WSDOT formula for the case in which estimated developed hammer energy was used, and for the case in which the rated energy was used. As discussed previously, the developed hammer energy was not available for the case histories in the database, necessitating an approximate but conservative estimate of the developed energy used in these analyses. Therefore, the rated hammer energy, as well as only hammer cases in which the rated and developed energy were identical, were also analyzed. These analyses resulted in resistance factors ranging from 0.50 to 0.57, respectively, in addition to the resistance factor of 0.60 obtained when all the data related to the developed hammer energy were considered.

The data provided in Table 6 also show that the magnitude of the resistance factors is not strongly affected by the DL/LL ratio. This is likely due to the fact that the uncertainty in the loads is much less than the uncertainty in the resistance. This finding is consistent with the findings by others (Barker, et al., 1991; Allen 2005). Therefore, it is feasible to recommend one resistance factor that is independent of the DL/LL ratio.

Table 6 indicates that a resistance factor of 0.45 could be used for the FHWA Gates formula and 0.71 for the CAPWAP method at BOR for larger (redundant) pile groups. These are slightly higher than what is recommended in Paikowsky, et al. (2004) and Allen (2005). The difference is the result of differences in how well the CDF is fitted to the tail of the data, as Paikowsky, et al. (2004) just use a general lognormal fit to the entire data set, whereas the lower tail region was fit more accurately in the present study (see figures 17 and 18 as examples).

Table 6. Summary of resistance factors obtained from the Monte Carlo simulations.

Pile Resistance Prediction Method	**β= 2.3**				**β= 3.0**
	DL/LL = 2 φ	**DL/LL = 3 φ**	**DL/LL = 5 φ**	**DL/LL = 3 Relative Conservatism Ratio, φ/λ**	**DL/LL = 3 φ**
WSDOT Formula (developed energy)	0.61	0.60	0.59	0.58	0.50
WSDOT Formula (rated energy)	0.52	0.50	0.50	0.56	0.41
WSDOT Formula (developed and rated energy, steam hammers only)	--	0.57	--	0.53	--
FHWA Modified Gates Formula (estimated developed energy)	--	0.51	--	0.46	0.40
FHWA Modified Gates Formula (rated energy)	--	0.46	--	0.45	0.37
ENR with FS of 6 removed (estimated developed energy)	--	0.11	--	0.30	0.08
ENR with FS of 6 removed (rated energy)	--	0.10	--	0.30	0.075
CAPWAP (EOD all data)	--	0.73	--	0.39	0.56
CAPWAP (EOD with N < 8 bpi)	--	0.83	--	0.41	0.66
CAPWAP (BOR all data)	--	0.71	--	0.60	0.59
CAPWAP (BOR with N < 8 bpi)	--	0.75	--	0.66	0.62

RECOMMENDATIONS

In general, the resistance factors provided in the AASHTO LRFD specifications (AASHTO 2004) are rounded to the nearest 0.05. Based on the analyses summarized in Table 6, a resistance factor of 0.55 is recommended for the WSDOT Pile Driving Formula for larger (redundant) pile group foundations. Note that the DL/LL ratio has only a minor effect on the resistance factor required, and a φ of 0.55 appears to be applicable to most DL/LL combinations that would be encountered in practice. For smaller pile groups (i.e., four piles or less), a resistance factor of 0.45 is recommended so that a higher target β of 3.0 is achieved.

In addition to these recommended values, resistance factors for other pile bearing resistance field verification methods are presented in Table 7. Note that while a resistance factor is provided for the ENR formula in Table 6, it is extremely low, which reflects the exceptional degree of uncertainty in that particular formula. A recommended resistance factor for the ENR formula is not provided in Table 7 because of the high degree of uncertainty in the predicted pile resistance using that formula.

Table 7. Recommended resistance factors for pile foundations.

Pile Resistance Prediction Method	β= 2.3 Resistance Factor φ	β= 3.0 Resistance Factor φ
WSDOT Formula (developed energy)	0.55	0.45
FHWA Modified Gates Formula (estimated developed energy)	0.45	0.40
CAPWAP (EOD with N < 8 bpi)	0.75	0.65
CAPWAP (BOR with N < 8 bpi)	0.70	0.60

REFERENCES

AASHTO, 2004, *LRFD Bridge Design Specifications*, American Association of State Highway and Transportation Officials, Third Edition, Washington, D.C., USA.

Allen, T. M., 2005, *Development of Geotechnical Resistance Factors and Downdrag Load Factors for LRFD Foundation Strength Limit State Design*, Publication No. FHWA-NHI-05-052, Federal Highway Administration, Washington, DC, 41 pp.

Allen, T. M., Nowak, A. S., and Bathurst, R. J., in press, *Calibration to Determine Load and Resistance Factors for Geotechnical and Structural Design*, Transportation Research Circular ___, Transportation Research Board, Washington, DC., 2005.

Barker, R. M., Duncan, J. M., Rojiani, K. B., Ooi, P. S. K., Tan, C. K. and Kim. S. G., 1991, *Manuals for the Design of Bridge Foundations,* NCHRP Report 343, TRB, National Research Council, Washington, DC.

Benjamin, J. R., and Cornell, C. A., 1970, *Probability, Statistics, and Decision for Civil Engineers*, New York: McGraw-Hill, pp.

Fragaszy, R. J., Higgins, J. D., and Argo, D. E., 1988, *Comparison of Methods for Estimating Pile Capacity*, WSDOT Research Report WA-RD 163.1, 62 pp.

GRL Engineers, Inc., 2003, Wave Equation Analysis of Pile Driving (GRLWEAP 2003).

GRL Engineers, Inc., 1996, Wave Equation Analysis of Pile Driving (GRLWEAP 1996).

Hannigan, P., Goble, G., Thendean, G., Likins, G., and Rausche, F., 1997, *Design and Construction of Driven Pile Foundations*, NHI Course Nos. 13221 and 13222, Workshop Manual, Volume 1.

Nowak, A. S., 1999, *Calibration of LRFD Bridge Design Code*, NCHRP Report 368, Transportation Research Board, Washington, DC

Paikowsky, S. G., Regan, J., and McDonnell, J., 1994, *A Simplified Field Method for Capacity Evaluation of Driven Piles*, FHWA Report No. FHWA-RD-94-042, Washington, DC.

Paikowsky, S. G., Kuo, C., Baecher, G., Ayyub, B., Stenersen, K, O'Malley, K., Chernauskas, L., and O'Neill, M., 2004, *Load and Resistance Factor Design (LRFD) for Deep Foundations*, NCHRP Report 507, Transportation Research Board, Washington, DC.

Peck, R. B., Hansen, W. E., and Thornburn, T. H. (1974). *Foundation Engineering.* Second Edition, John Wiley and Son, Inc., New York, 514 pp.

Washington State Department of Transportation, 2004, *Standard Specifications for Road, Bridge, and Municipal Construction*, Olympia, WA.

Withiam, J. L., Voytko, E. P., Barker, R. M., Duncan, J. M., Kelly, B. C., Musser, S. C., and Elias, V., 1998, *Load and Resistance Factor Design (LRFD) for Highway Bridge Substructures*, FHWA HI-98-032.