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ABSTRACT

Vibrations generated by driven pile installation often affect adjacent and surrounding buildings in tightly spaced urban environments. These vibrations can lead to complaints from neighboring residents and businesses and/or cause structural damage to adjacent structures, especially older and historic buildings. Pile driving vibrations are of particular interest in the Charleston, South Carolina area since the majority of new structures are founded on driven pile foundations bearing within the underlying Cooper Marl Formation.

By knowing the vibration attenuation relationship of a project area (i.e. the decrease in vibration amplitude with distance), it is possible to develop pile installation plans that minimize discomfort to residents and the risk of damage to adjacent structures. In addition, knowledge of the vibration attenuation relationship for a site can assist in determining the limits for pre-condition surveys to document the existing conditions of adjacent structures prior to pile driving operations.

This paper presents the case histories of vibrations monitored during seven driven pile and one vibratory pile construction projects in the Charleston, South Carolina area. The vibration data was then analyzed to evaluate the energy-attenuation relationships for the individual sites. Comparisons of these analyses were then made in an effort to determine typical energy-attenuation relationships for driven piles within the Charleston, SC area.

INTRODUCTION

Vibrations generated during driven pile installation are often a cause for concern during the project, since these vibrations have the potential to disturb nearby residents and cause damage to adjacent structures. In Charleston, South Carolina, pile driving vibrations are of special concern for several reasons: the tight spacing of buildings, the age and historic significance of many of the city's buildings, and the presence of soft clays and loose sands in the local soil stratigraphy require placement of many of the newer structures under construction on deep foundation systems. Due to cost considerations, many of the deep foundation systems chosen are driven piles.

It has long been understood that vibrations generated by pile driving dissipate with distance (i.e. attenuate). Modeling pile driving vibration attenuation relationships (i.e. the decrease in vibration amplitude with distance) is dependent on a range of factors, with the primary factors being energy delivered to the pile and the surrounding soil types. Previous research (e.g. Wiss, 1980, Woods and Jedele, 1985, Ali et al., 2003) have published attenuation characteristics of certain soil types based on empirical data.

The case histories of vibrations measured at 8 pile installation construction sites in the Charleston area are presented. The attenuation relationships for these sites were analyzed with regards to distance and energy in order to develop typical models for use in future pile driving projects in and around Charleston, SC.

Charleston, South Carolina lies within the Lower Coastal Plain geologic province of the Atlantic Ocean coast. The near surface "overburden" soils consist primarily of Pleistocene deposits of the Quaternary Period. Pleistocene formations generally consist of sand and clay deposits with varying amounts of shells and occasional organics.

Beneath the "overburden" soils lies a highly calcareous soil stratum called the Cooper Group, known locally as the Cooper Marl Formation. The Cooper Marl Formation is a marine deposit of late Eocene to Oligocene Periods that underlies a significant portion of the Charleston Area. The Cooper Marl is typically classified according to the Unified Soil Classification System as a low plasticity sandy silt (ML) or sandy clay (CL). Refer to Klecan et al. (2001) for additional details of the Cooper Marl Formation. Depth to the Cooper Marl Formation varies from approximately 12 to 30 meters (~40 to 100 feet) within the downtown Charleston area. Due to the soft clays and/or loose sands that overlay the Cooper Marl Formation, most deep foundations within the Charleston area are founded within the Cooper Marl. Groundwater in the Charleston area is typically encountered between 3 to 8 feet below the existing ground surface.

VIBRATION ATTENUATION RELATIONSHIPS

In order to determine the vibration attenuation relationships of the Charleston area, previous attenuation research was analyzed. Typically, three vibration attenuation equations are used to model pile driving vibrations with distance: Geometric and material damping (Richart et al, 1970); Pseudo-attenuation with distance (Wiss, 1981); and Pseudo-attenuation with "scaled-distance" (Wiss, 1981). Previous research and analysis by Woods and Jedele (1985) and Woods (1997) has shown that the geometric and material damping model does not fit empirical data well, while the pseudo-attenuation models are in general agreement with previous case history data. Therefore, only the pseudo-attenuation models were used in this paper. The pseudo-attenuation with distance and scaled distance relationships are as follows:

Pseudo-attenuation with distance (Wiss, 1981)

$$PPV = kD^{-n} \tag{1}$$

Where:
PPV = Peak Particle Velocity
D = Distance from source
k = Value of PPV at 1 unit of distance
n = Pseudo-attenuation coefficient

Pseudo-attenuation with "scaled-distance" (Wiss, 1981)

$$PPV = K\left(\frac{D}{\sqrt{E}}\right)^{-N} \tag{2}$$

Where:
PPV = Peak Particle Velocity
D = Distance from source
E = Energy of source
K = Value of PPV at 1 unit of distance
N = Pseudo-attenuation coefficient

Charleston Case Histories

A total of 8 pile construction sites were analyzed. Seven of these sites involved driven pile foundations. One site (i.e. Site 7) did not directly involve driven piles. This project consisted of the installation of earthquake drains (E-Drains™), which is a type of ground modification involving the insertion of perforated PVC tubing wrapped in geotextile fabric using a steel mandrel and a vibratory hammer. However, the process is identical to vibrating a small pile into the ground and was therefore considered to be the equivalent to a pile installation. A description of earthquake drains and their installation process is provided by Ellington and Goughnour (1998). Figure 1 presents the various project sites relative to the Charleston area.

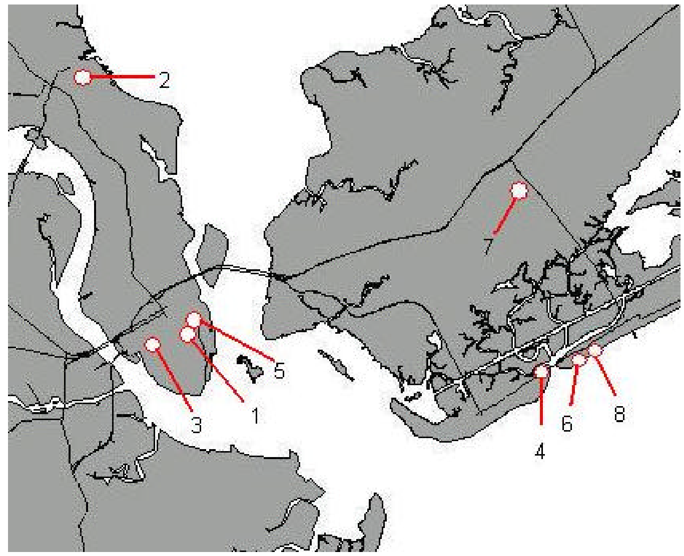

Fig. 1. Location of Case History Project Sites.

Pre-augering (i.e. pre-drilling a hole at the pile location prior to installation of the pile) was performed at all 7 of the driven pile sites. Dynamic pile monitoring using the Pile Driving Analyzer™ was performed on test piles at 4 of the 8 case history sites. This pile monitoring allowed for determination of the energy delivered to the piles and evaluation of hammer performance in addition to pile capacity determination.

A brief description of the various case history sites is presented in the following paragraphs. Table 1 presents a summary of the surface and bearing soils and their associated insitu testing properties. A summary of the various pile types is provided in Table 2. Hammer information for each case history site is presented in Table 3.

SITE 1: A new 3 story library was constructed in downtown Charleston, SC in a neighborhood with many historic buildings. A total of 589 0.3 m (12 inch) square by 29 m (95 feet) Pre-Stressed Concrete (PSC) piles were driven into the Cooper Marl Formation to support the new building. The nearest adjacent structure was 4.6 m (15 ft) feet from a pile location. The piles were pre-augered with a 0.30 m (12 in) diameter auger to a depth of 15.2 m (50 ft).

SITE 2: An addition to an existing single story school and a new stand-alone classroom building in North Charleston, SC were constructed. The existing building addition was founded on 12 HP12x53 piles, while the new classroom building was founded on a total of 94 0.3 m (12 inch) square Pre-Stressed Concrete (PSC) piles. The HP piles were located within 1.8 m (6 ft) of the existing building, while the PSC piles were located 9.1 m (30 ft) from the structure. The HP and PSC piles were driven 15.2 m (50 ft) below the existing ground surface and into the underlying Cooper Marl Formation. The piles were pre-augered with a 0.30 m (12 in) diameter auger to a depth of 10.7 m (35 ft).

SITE 3: A power plant expansion for an existing 5 story medical facility was constructed in downtown Charleston, SC. The foundation consisted of 12 HP12X53 steel piles by 25.9 m (85 ft) long driven into the underlying Cooper Marl Formation. The 5 story modern medical facility and a historic 2 story brick building were located within 1.8 m (6 ft) and 5.6 m (18.5 ft), respectively, from the pile locations. The piles were pre-augered with a 0.20 m (8 in) diameter auger to a depth of 5.2 m (17 ft).

SITE 4: A new residence on Sullivan's Island, SC was supported on 78 0.20m (8 inch) tip diameter timber piles. The piles were driven to 13.7 m (45 ft) below the existing ground surface. The piles were pre-augered with a 0.25 m (10 in) diameter auger to a depth of 3.0 m (10 ft). The nearest structure was 8.5 m (28 ft) from the pile locations.

SITE 5: A new commercial office building is planned for downtown Charleston, SC. A total of six 0.30 m (12 inch) square by 15.2 m (50 feet) Pre-Stressed Concrete (PSC) piles were driven into a dense sand layer above the Cooper Marl formation as part of the test pile program for this project. The piles were pre-augered with a 0.30 m (12 in) diameter auger to a depth of 3.7 m (12 ft).

SITE 6: A new 2 story residence on the Isle of Palms, SC was founded on 55 0.25m (10 inch) tip diameter timber piles. The timber piles were driven within a dense sand layer to a depth of 4.6 m (15 ft) below the existing ground surface. The piles were pre-augered with a 0.20 m (8 in) diameter auger to a depth of 3.7 m (12 ft).

SITE 7: Over 1000 earthquake drains (E-Drains™) were installed as part of a ground improvement program for a single-story retail building in Mount Pleasant, SC. The E-Drains™ were installed to a depth of 9.1 m (30 ft) below the final building pad elevation. No pre-augering was performed for the E-Drains™.

SITE 8: A new 2 story residence on the Isle of Palms, SC was founded on 51 0.20m (8 inch) tip diameter timber piles. The timber piles were driven within a dense sand layer to a depth of 4.6 m (15 ft) below the existing ground surface. The piles were pre-augered with a 0.20 m (8 in) diameter auger to a depth of 3.0 m (10 ft).

Table 1. Site Soil Property Summary

Site	Surface Soil			Bearing Layer		
	Type	**$q_{t\ ave}$ (MPa)**	**N_{ave}**	**Type**	**$q_{t\ ave}$ (MPa)**	**N_{ave}**
1	Sand	NA	12	Marl	NA	18
2	Sand	5	NA	Marl	3	NA
3	Sand	NA	4	Marl	NA	13
4	Sand	4	NA	Inter.[1]	2	NA
5	Sand	3	NA	Sand	25	NA
6	Sand	NA	11	Sand	NA	40
7	Sand	10	13	Sand	25	30
8	Sand	5	NA	Sand	25	NA

NOTES:
1. Interbedded sands, silts, and clays

Table 2. Pile Property Summary

Site	Pile			Pre-auger	
	Type	**L (m)**	**E (m)**	**Ø (m)**	**D (m)**
1	0.30m sq. PSC	29.0	29.0	0.30	15.2
2	0.30m sq. PSC	15.2	15.2	0.30	10.7
	HP12x53	15.2	15.2	0.30	10.7
3	HP12x53	25.9	25.9	0.20	5.2
4	0.20m tip Ø Timber	13.7	13.7	0.30	3.0
5	0.30m sq. PSC	15.2	15.2	0.30	3.7
6	0.25m tip Ø Timber	7.6	4.6	0.20	3.7
7	0.08m Ø E-Drain™	9.1	9.1	NA	NA
8	0.20m tip Ø Timber	7.6	4.6	0.20	3.0

NOTES:
L = Pile length
E = Pile Embedment below the existing ground surface
Ø = Auger Diameter
D = depth of pre-auger
PSC = Pre-Stressed Concrete
E-Drain™ = Earthquake drain

Table 3. Site Hammer Property Summary

Site	Hammer			
	Manufacturer.	**Model**	**Energy[1] (kN-m)**	**EMX[2] (kN-m)**
1	Delmag	D30-23	99.9	25.3
2	Vulcan	06	26.4	11.3
3	Conmaco	C65	26.4	15.5
4	Vulcan	06	26.4	NA
5	ICE	Model75	40.7	12.5
6	Vulcan	06	26.4	NA
7	HMC	51+535	71.6	NA
8	Vulcan	01	20.3	NA

NOTES:
1. Energy = Rated Energy of Hammer
2. EMX = Average Energy delivered to pile (from PDA measurements)

Vibration Data

Vibration monitoring was conducted within and around the project sites by measuring vertical, transverse, and longitudinal ground velocities at selected monitoring points using tri-axial velocity transducers. Monitoring points were located at or near adjacent structures as well as at various intervals from the construction activities to determine attenuation relationships for the different sites. The vibration Peak Vector Sum (PVS) was then calculated from the PPV tri-axial data.

Vibration PVS's were used to examine attenuation relationships since it is the maximum vibration seen at the monitoring location. Therefore, the attenuations from these measurements would present a "worst-case" condition for vibrations and the longest sphere of vibration influence. Previous research investigating vibration attenuation relationships (Ali et. al, 2003) noted that by using the vibration PVS, there is less dependency on equipment setup and wave source type and origination. Ali et al. (2003) also noted that this approach may be overly conservative.

Figures 2 through 9 present the vibration Peak Vector Sums (PVS) vs. distance for the eight case history projects. Figures 10 through 17 present the vibration PVS vs. scaled distance for the eight case history projects. Figure 18 presents the vibration PVS vs. scaled distance for all the presented case history sites. Summaries of the PVS vs. distance and PVS vs. scaled distance best fit lines are presented in Table 4. Note the scaled distance is based on the rated hammer energies.

Table 4. PVS vs. Distance & Scaled Distance Best Fit Line Summary using Equations 1 & 2.

Site	Rated Energy (kN-m)	# of pts	Pseudo-Attenuation Factors			
			k	K	*n* & N	R^2
1	99.9	43	22.44	2.98	0.611	0.5633
2	26.4	30	29.4	3.2	0.841	0.751
3	26.4	8	23.3	5.2	0.708	0.370
4	26.4	106	30.3	3.2	0.851	0.407
5	40.7	12	17.8	4.3	0.496	0.361
6	26.4	55	150.0	10.0	1.028	0.803
7	71.6	9	75.5	4.3	0.913	0.811
8	20.3	17	68.1	8.9	0.810	0.702
All	N/A	280	NA	4.4	0.972	0.563

NOTES:
R^2 = Coefficient of Determination

As shown in Figs. 2 through 17, clear vibration attenuation relationships exist for each case history site, although significant scatter can be seen for several projects. The extent of this scatter is shown in the coefficient of determination (R^2) calculations for the vibration attenuation best fit lines (see Table 4). A clear vibration attenuation relationship is also observed when examining the PVS vs. scaled distance for all the data for the eight projects (see Fig. 18).

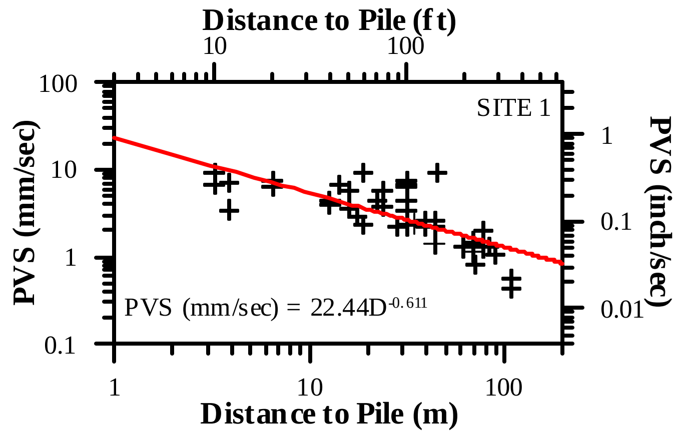

Fig. 2. PVS vs. Distance for Site 1.

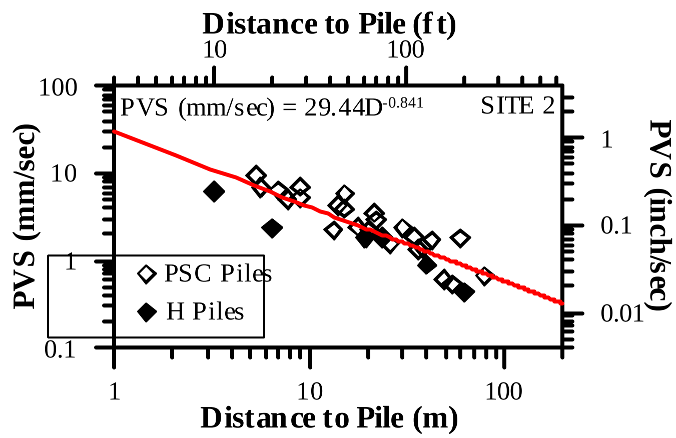

Fig.3. PVS vs. Distance for Site 2.

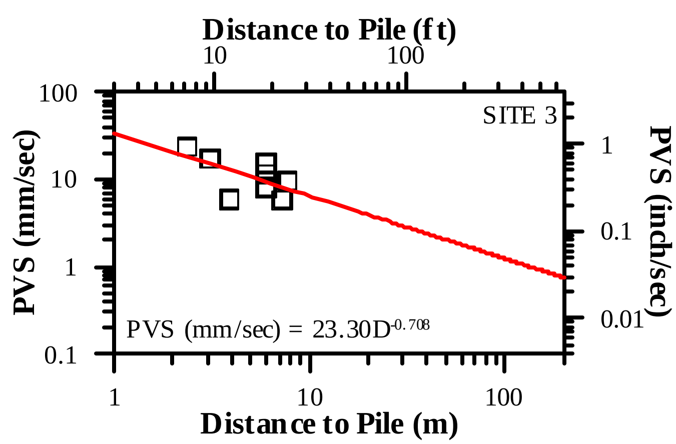

Fig.4. PVS vs. Distance for Site 3.

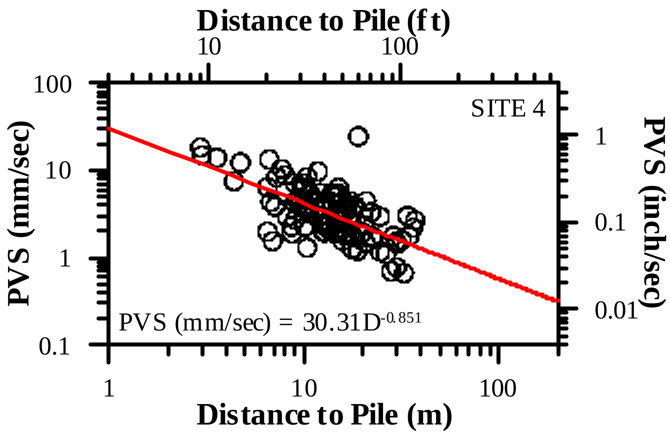

Fig. 5. PVS vs. Distance for Site 4.

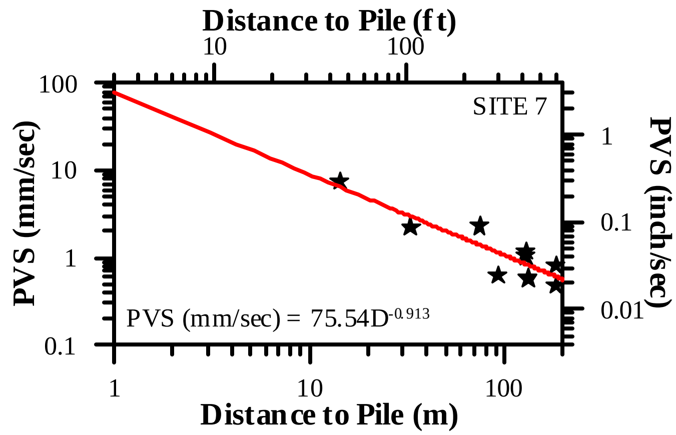

Fig. 8. PVS vs. Distance for Site 7.

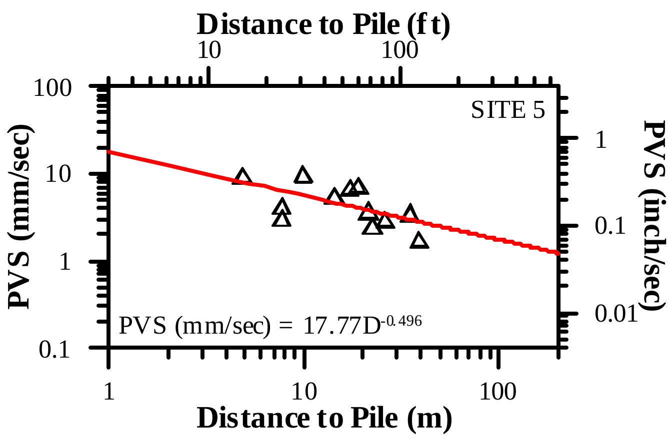

Fig 6. PVS vs. Distance for Site 5.

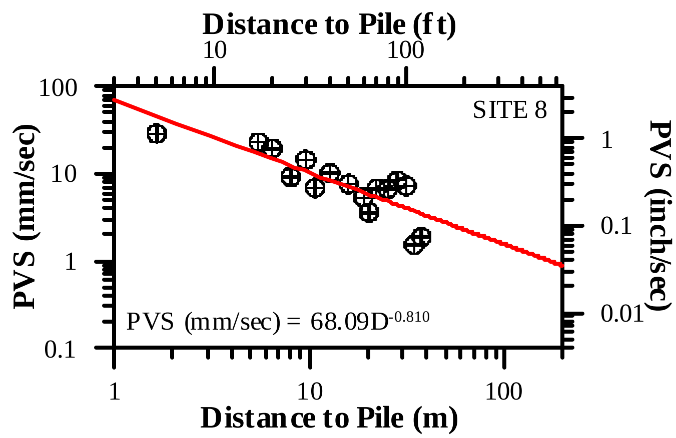

Fig. 9. PVS vs. Distance for Site 8.

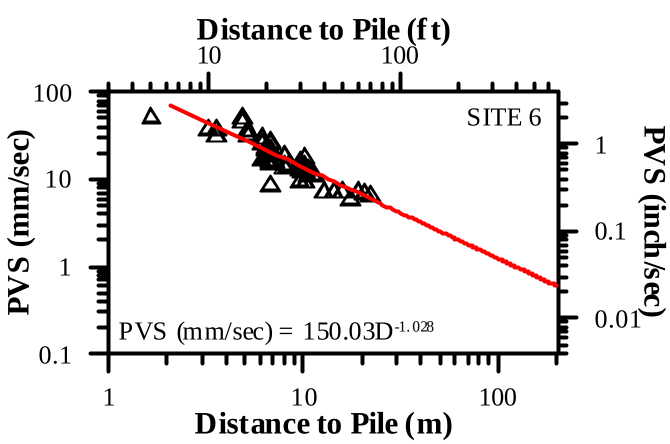

Fig.7. PVS vs. Distance for Site 6.

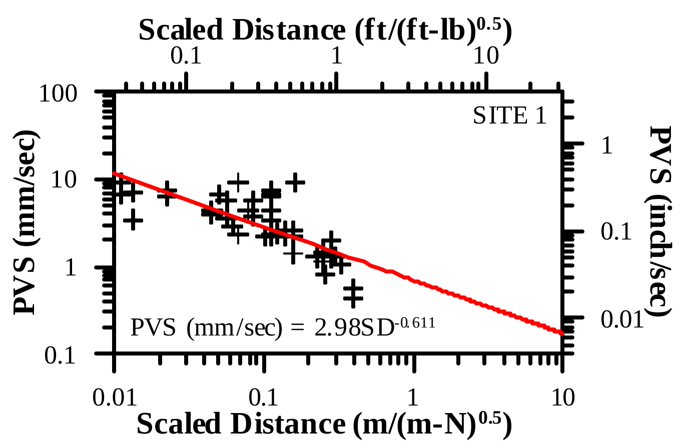

Fig. 10. PVS vs. Scaled Distance for Site 1.

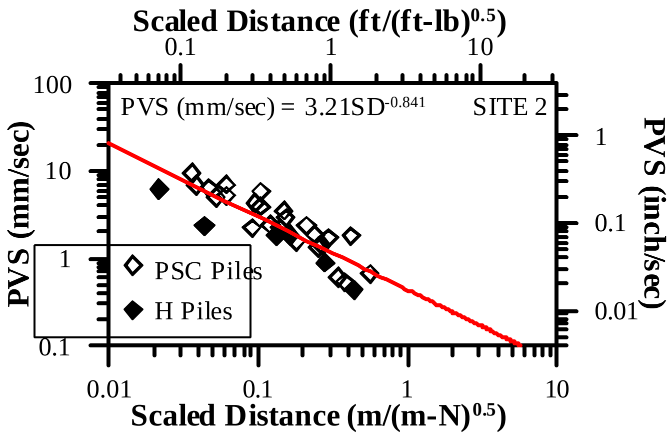

Fig. 11. PVS vs. Scaled Distance for Site 2.

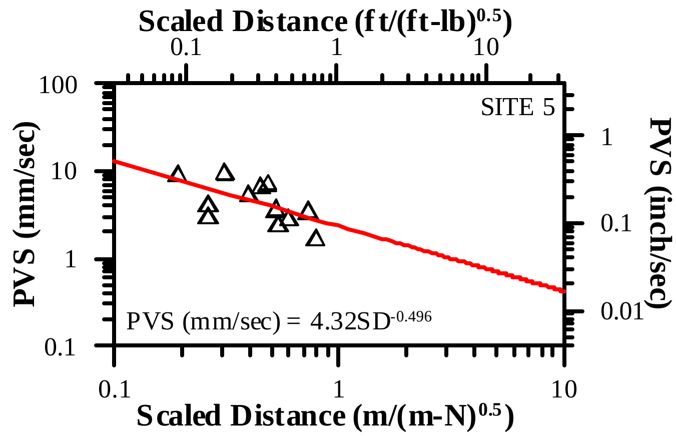

Fig. 14. PVS vs. Scaled Distance for Site 5.

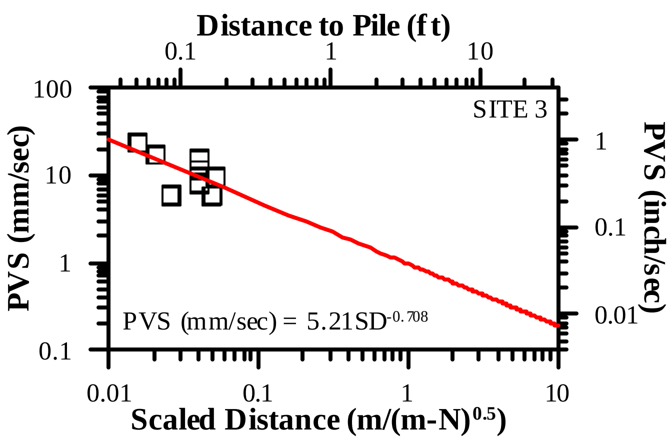

Fig. 12. PVS vs. Scaled Distance for Site 3.

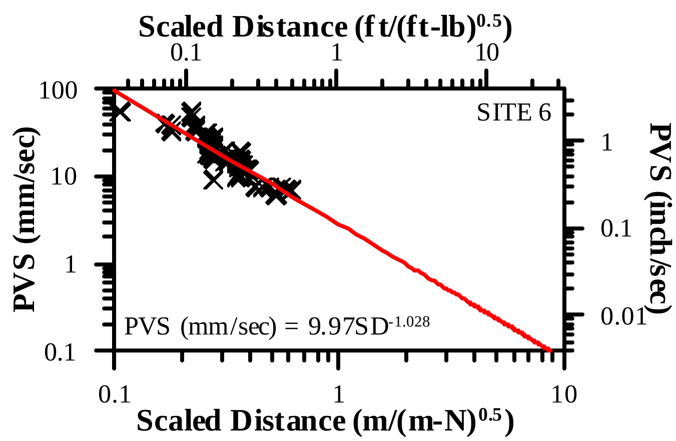

Fig. 15. PVS vs. Scaled Distance for Site 6.

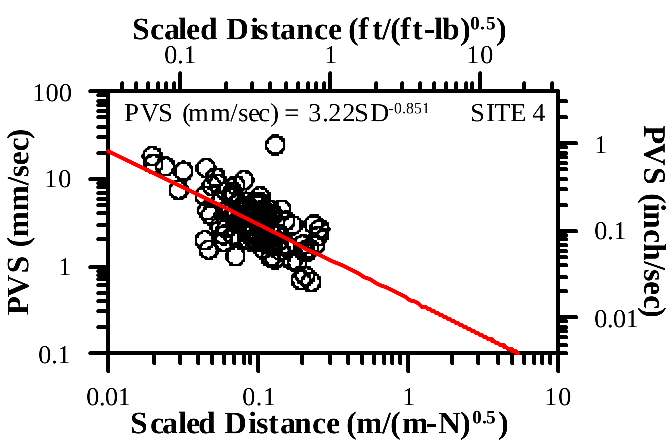

Fig. 13. PVS vs. Scaled Distance for Site 4.

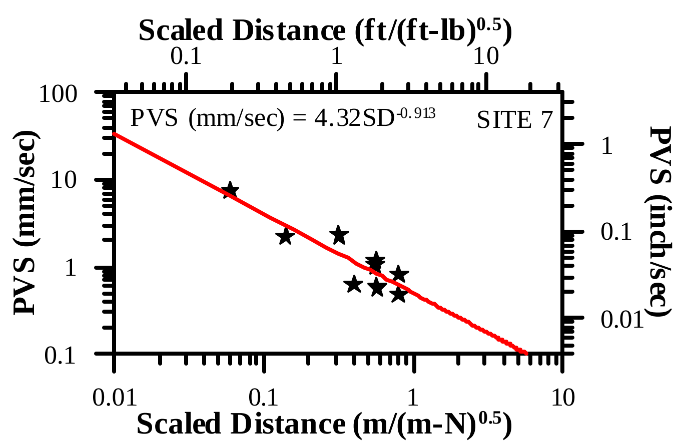

Fig. 16. PVS vs. Scaled Distance for Site 7.

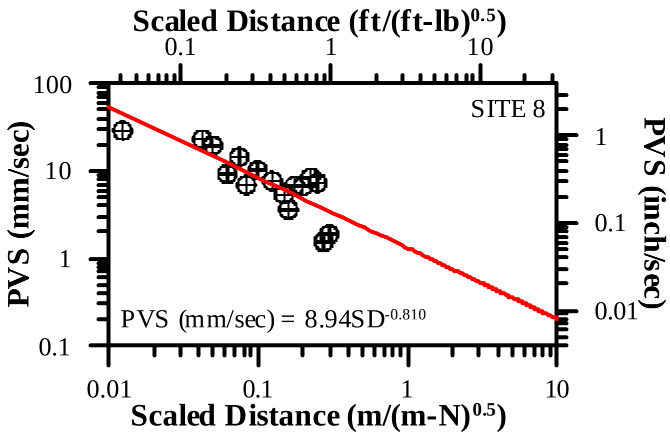

Fig. 17. PVS vs. Scaled Distance for Site 8.

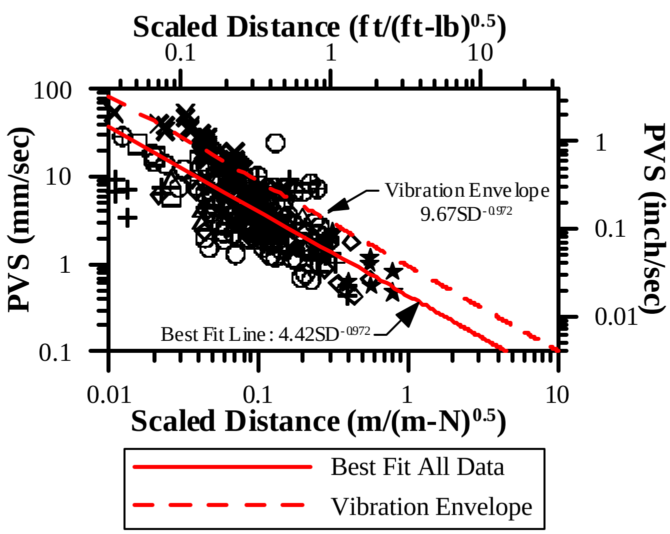

Fig. 18. PVS vs. Scaled Distance for all case histories and Charleston area vibration envelope.

Examination of the vibration data best fit lines shows slight variation between the attenuation rates for the eight sites and the combined total data. This variation is to be expected, as attenuation rates are site specific and are affected by variations in soils, groundwater levels, and other factors (Wiss, 1981). The limited number of data within several of the projects may also account for some of the attenuation rate variation. A comparison was made to determine if the attenuation rates (i.e. *n* or N values) for the Charleston sites were similar to those previously published in the technical literature. This comparison is presented in Table 5. As shown in Table 5, the attenuation rates compare favorably with the rate for sands presented by Wiss (1981) and Ali et al. (2003). However, the Charleston site attenuation rates are significantly lower than those presented by Woods and Jedele (1985) and Brenner and Chittikuladiok (1999) for most sands and surface sands, respectively.

Table 5. Attenuation Rate Comparison (after Ali et. al, 2003).

Reference	**Site**	**Soil Type**	***n***
Current Paper	1-8	Sand	0.496-1.03
	All	Sand	0.972
Ali et al (2003)		Sands	0.88-1.02
Brenner and Chittikuladiok (1999)		Surface Sands	1.5
		Sand fill, over soft clays	0.8 – 1.0
Wiss (1981)		Sands	1.0
Woods and Jedele (1985)		Dense compacted sands (15<N<50)	1.1
		Most sands (5<N<15)	1.5

An examination of the scaled distance attenuation relationships using both the rated energy of the pile driving hammer and the measured energy delivered to the pile was also conducted. Figure 19 presents the PVS vs. scaled distance using the measured energy for Site 2. As shown in Fig. 19, the use of measured energy shifts the best fit line to encompass a larger zone of influence without changing the rate of attenuation (i.e. N value). This result was to be expected due to the following:

- The energy delivered to a pile from an impact hammer is less than the rated hammer energy due to various energy losses.
- The inverse relationship between PPV and rated hammer energy as shown in equation 2 (i.e. a decrease in rated energy equates to a greater PPV for a given distance)

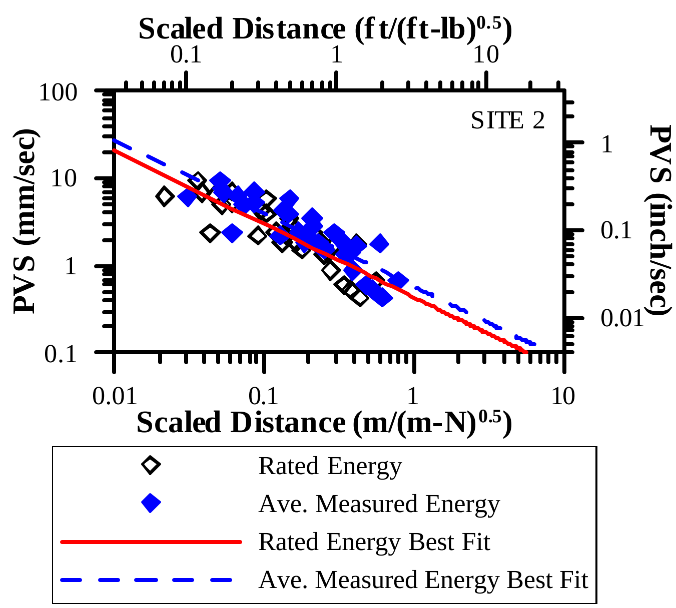

Fig. 19. PVS vs. Rated & Measured Energy Scaled Distance for Site 2.

In order to develop a vibration envelope for future projects in the Charleston area where the attenuation characteristics are not known, a parallel line offset from the combined data best fit attenuation line was calculated to encompass the majority of vibration PVS vs. scaled distance data. This vibration envelope is presented in Fig. 18. Scaled distance was chosen to develop the vibration envelope since the rated energy of the hammer provides a means of normalizing the data. The similar attenuation rates for all eight sites and the combined data suggests that this envelope would be suitable for other sites within the Charleston area. It should be noted that this proposed vibration envelope may be conservative since it is based on PVS data.

This vibration envelope can serve as a valuable guide to develop future vibration monitoring and pre-condition survey plans in the Charleston area where site specific attenuation relationships are not known. An example of the use of the vibration envelope is as follows: A rated hammer energy of 26.4 kN-m (19.5 kip-ft), which is common to the Charleston area, would most likely produce maximum vibrations of 8.4 mm/sec (0.33 inches/sec) at a distance of 15.2 m (50 ft). This vibration level is below the minimum suggested vibration criteria for historic and older buildings in the Charleston area developed by Hajduk et al. (2004).

CONCLUSIONS

Vibration attenuation relationships for 8 case histories in the Charleston, SC area are presented. Analysis of these relationships showed that the attenuation rates are similar to each other and the combined data. Comparison of these attenuation relationships to others presented in the technical literature for similar soil types showed excellent concurrence with some and poor agreement for other case histories.

An examination of scaled distance relationships using both the rated and measured hammer energies was also conducted using dynamic pile measurement from four of the eight case histories. This examination showed that the use of measured energy delivered to the pile shifts the best fit attenuation line to encompass a larger zone of influence without changing the rate of attenuation.

Finally, a vibration attenuation envelope for the Charleston area is presented to aid in estimating driven pile vibrations for future projects. This vibration envelope was developed from the combined data of all 8 case histories. The similar attenuation rates for all eight sites and the combined data suggests this envelope would be appropriate for other sites within the Charleston area. An example is provided calculating the predicted maximum vibrations at a given distance using a rated hammer energy commonly used in Charleston. As shown in this paper, this vibration envelope can be a valuable tool for estimating vibrations for future driven pile projects in the Charleston area where site specific attenuation relationships are not known.

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