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SYNOPSIS: Vibrations due to pile installation have long been a concern to owners, contractors, and engineers. Specifically, what levels of vibrations can be assumed for a given pile, hammer, and subsurface conditions and how can these levels be predicted in advance of construction so an assessment of nearby structures can be made? This paper presents the results of vibration monitoring at several sites where various piles and pile hammers have been used, and recommends a conservative method of predicting peak particle velocity at the ground surface near pile installations. Where sensitive structures are involved, a response spectrum analysis is recommended.

INTRODUCTION

Pile installation causes vibrations to occur in the soil due to wave propagation. In urban areas, the level of vibration can be a concern for nearby structures. All parties involved, namely contractors, owners of the constructed project, owners of nearby structures, and engineers, would like to limit or eliminate the risk of damage for obvious reasons. No one wants to be in litigation determining who was negligent if adjacent structures are damaged.

Problems associated with vibrations, although not entirely avoidable, can at least be anticipated and planned for before construction. Rubin (1978) suggests various avenues that can be taken during the preconstruction phase of a project to minimize the risk of legal action. First and foremost is the recognition on the part of the design engineer that installation of his foundation design could impose unwanted vibrations on adjacent structures. Unfortunately, in some cases, this is not recognized until construction begins or until damage of nearby structures takes place and the affected property owner raises a red flag.

On most projects where pile foundations are to be installed, the contract specifications call out the pile to be used and include a minimum hammer energy. Even where this is not the case, most specifications require the contractor to submit equipment lists during the bid phase. Knowing the hammer and pile type allows an estimate of peak particle velocity (PPV) to be made. This can be used by the design engineer to assess the potential for damage to nearby structures by applying previously developed empirical correlations relating PPV with structural damage. Where sensitive structures or equipment are involved, it may be necessary to perform more sophisticated analyses to determine damage potential.

This paper will present the results from various cases where PPVs were measured during pile installation. These velocities were correlated with predicted velocities to evaluate potential damage to adjacent structures. In addition, a case history will be presented where structure response due to pile installation was measured for use in a response spectrum analysis for a sensitive structure.

BACKGROUND

Permanent damage to structures from pile driving can occur in two ways: 1) transient vibration displacement due to impulse loading or noise and 2) permanent structure displacement due to densification. A third type of damage can occur that is not permanent but can be very costly, i.e., equipment or instrument shutdown. For power plants, this can be a significant item depending on how widespread a shutdown or equipment "trip" is.

Generally, damage due to pile driving is related to PPV, without regard to the response of the structure or equipment that might be affected. For well-engineered, large, reinforced concrete structures, using a PPV threshold criterion is usually adequate. This is mainly because, in the case of grassroots projects, there are no other structures nearby, and, in the case of existing structures, they are either founded on competent material or on deep foundations. However, for sensitive structures and equipment, using a PPV criterion with no provision for frequency can be unwise and may prove to be very costly.

The PPV threshold criterion the authors use is the same criterion that has been developed and used in the blasting industry for several years, i.e., limiting the PPV to less than 2 inches per second (ips). Many other such criteria are in use today. A good summary of these is presented in Theissen and Wood (1982).

In the authors' experience, vibration damage is rarely a concern unless piles are being installed immediately adjacent to an existing structure. Once the distance between pile installation and the structure exceeds about 10 feet, measured PPVs are typically less than 2 ips. Nevertheless, vibration monitoring is always required for documentation purposes. Further, preconstruction surveys, walkdowns, and inspections are usually made for nearby structures to document pre-pile installation conditions. Generally, this procedure is adequate for the majority of structures.

Several investigators, including Dowding (1977), Medearis (1977), Naik (1979), and Siskind et al. (1980), have proposed a response spectrum type of analysis that relates the structural or equipment response to the imposed vibrations. Analysis of this type has been performed for several years in the nuclear industry. It is more rational since it takes into account not only the response of the ground but also the response of the structure being affected. Although for most types of projects a PPV threshold criterion is adequate, a response spectrum type of analysis may be warranted for sensitive structures or pieces of equipment.

PEAK PARTICLE VELOCITY DETERMINATION

The theory of vibration and subsequent wave propagation will not be discussed here, but can be found elsewhere in the literature. Heckman and Hagerty (1978), Wiss (1981), and Thiessen and Wood (1982) provide good summaries with respect to pile driving.

Peak particle velocity can best be expressed in terms of a scaled-energy propagation equation. Wiss (1981) has proposed the following equation for use:

$$V = K\left(\frac{D}{\sqrt{E}}\right)^{-n} \qquad (1)$$

where, in typical units:

- V = peak particle velocity in inches/second (ips)
- K = intercept (ips)
- D = distance (ft)
- E = hammer energy (ft-lb)
- n = attenuation rate, and

$\frac{D}{\sqrt{E}}$ = scaled distance

Values of n and K vary. Usually, n ranges between 1 and 2 (Wiss, 1981) while K can range from 0.05 to about 0.3 (Heckman and Hagerty, 1978). For typical onshore pile projects, with rated hammer energies in the range of 30,000 ft-lb and pile capacities less than about 100 tons, computed PPV values at close distances from a driven pile (10-to-20-foot range) using equation (1) and the range of n and K given above could be very high. The authors have used this equation for predicting PPV for various projects that have used different pile types in varying soil conditions. As will be presented, PPV measurements for these projects will be used to refine values of n and K.

MONITORING DATABASE

The authors have monitored and collected vibration data at various project sites over the past 14 years. Information on pile type, the hammer used, and general subsurface conditions for each of the projects is given in Table 1. All of the sites were for power plant structures, except for site 1, which was for a U.S. Government installation. Project sites 2 and 7 were "grassroots" projects, while projects 3 through 6 were at existing facilities. Although project 1 was a grassroots project, there was a sensitive structure nearby. Project 1 is the subject of the case study that will be discussed later.

The data for sites 2 through 7 were collected with seismographs capable of measuring PPV in three mutually perpendicular directions. The resultant PPV was then computed from the square root of the vector sum. Data acquisition for site 1 was made using both accelerometers and velocity pickups. In all cases, velocity data were collected continuously with pile penetration. In this study, however, only the peak value is reported.

Figure 1 summarizes the results in terms of PPV versus distance from the pile. The results show a distinct relationship between distance from the pile being driven and the measured PPV. A similar relationship can be seen in Figure 2, which is a plot of PPV versus scaled distance.

In both Figures 1 and 2, the solid line indicates the relationship using equation (1) and an expected energy of 10,000 ft-lb, which is based on measurements made (GRL 1988) of measured transferred energy to a pile from a hammer. (Typically, the transferred energy is about 30 to 40 percent of the hammer rated energy. An average assumed rated energy of 30,000 ft-lb thus results in a transferred energy of approximately 10,000 ft-lb.) Assuming n = 1, the value of K for the line is about 0.1. This correlation gives a reasonable prediction of PPV for the data presented. The differences in pile type/size and soil conditions probably account for much of the variation in PPV shown in Figures 1 and 2. The hammer energy is accounted for in the scaled distance, but its effect is muted (to the power

Table 1. Site and Pile Summary

Site Number	Pile Type	Impedance: lb sec/in	Driven Length: ft	Hammer	Rated Energy: ft–lbs	General Soil Conditions
1	14–inch square precast concrete	5,875	80	ICE 640	40,000	Loose to dense sands and silty sands with shell, GWT at 4 ft.
2	Raymond Step Taper; 12 ft shells, 000BR	7,000	78	Vulcan 80c	24,450	Fill to 7 ft, soft clayey silt and peat to 30 ft, medium clayey sand to 100 ft, GWT at 7 ft.
3	Sheet pile PZ–27	1,770	30	Delmag D–15	27,000	Medium to dense sands, GWT at ground surface.
4	Raymond Step Taper; 12 ft shells, 000BR	7,000	40	Vulcan 80c	24,450	Fill to 12 ft, soft silts and clay to 24 ft, dense to medium dense sand to 45 ft, underlain by rock, GWT within 2 ft of ground surface.
5	Closed–end Pipe 10.75"x0.219"	1,078	30	Vulcan 06	19,500	Loose to medium sand to 21 ft, soft clay to 27 ft, very dense sand to 37 ft, GWT at ground surface.
6	H–Pile 14 x 117	5,370	30	Vulcan 06	19,500	Thirty feet of medium dense to dense sand over rock GWT at 10 ft.
7	Raymond Step Taper; 12 ft shells, 0BR	7,000	80	Vulcan 06	19,500	Loose sand to 15 ft, soft clayey silt to 60 ft, underlain by medium dense to dense sand, GWT at 10 ft.

one half). The distance from the pile being driven is the most influential factor.

Using the above relationship results in predicted PPV being less than 2 ips at distances greater than 10 feet from the pile driven. Thus, structural damage is usually not a concern. However, architectural or cosmetic damage may be significant. This damage, usually impacting nearby property owners, is generally the damage that results in litigation. For this reason, it is not sufficient at many locations to simply ensure that a PPV threshold criterion is not exceeded.

It should be noted that Heckman and Hagerty (1978) proposed a relationship between K and pile impedance, which is the product of the pile area times the pile density times the sonic velocity of the pile material. As the impedance increases, K decreases. Although the relationship is generally reasonable, the authors could not confirm the same relationship with the data presented.

CASE STUDY

The case study presented here was undertaken as part of a preproduction pile load test program for a government facility. The program was conducted not only to determine allowable design loads, but also because vibration and settlement were a concern for a nearby sensitive installation. The following paragraphs only discuss measurements taken during the preproduction program.

Subsurface Conditions

The site is located in the coastal plain of the eastern United States. The sediments are of Pleistocene age and generally consist of alternating layers of fine sand and silty fine sand, both with varying amounts of shell fragments. The sands range in denseness from loose to very dense. Figures 3 and 4 present the results of some representative standard penetration test (SPT) N-values, and three cone penetration test (CPT) tip resistances measured at the site. The dense layer between about elevation -60 and -90 feet is the bearing stratum for the piles. Based on the subsurface information, with the exception of the few dense layers, the majority of the sands have a relative density near 50 percent.

Pile Type and Installation Equipment

The piles installed were precast, prestressed concrete, 14 inches on a side. Design compressive strength of the concrete was 5,000 pounds per square inch (psi) with an initial prestress of 800 psi. Each pile had four 9/16-inch, 7-wire, low-relaxation prestressing strands, and all piles were spirally reinforced and cast in 80-foot lengths.

Pile installation was accomplished with an ICE 640 closed-ended diesel pile hammer, which has a rated energy of 40,000 ft-lb. The hammer cushion consisted of a 2-inch-thick nylon disk and a 1/2-inch aluminum disk, with an area of about 380 square inches, an assumed elastic modulus of 175 kips per square inch (ksi), and a coefficient of restitution of 0.92. The pile cushion generally consisted of seven sheets of 3/4-inch plywood. Typically, the plywood cushion would compress to about 3 inches thick during driving. New cushions were used for each pile.

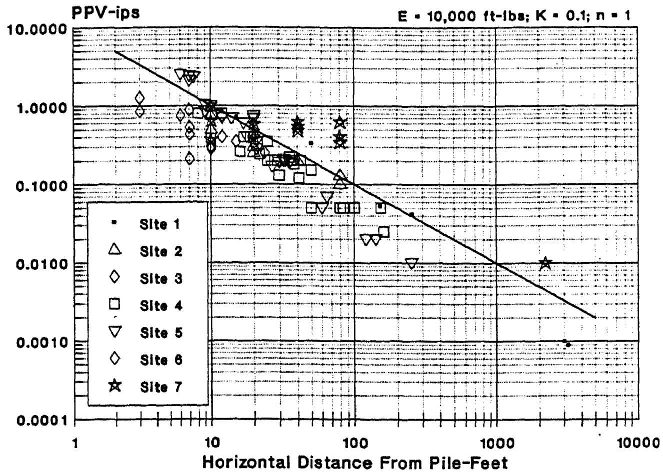


Figure 1. Velocity vs Distance

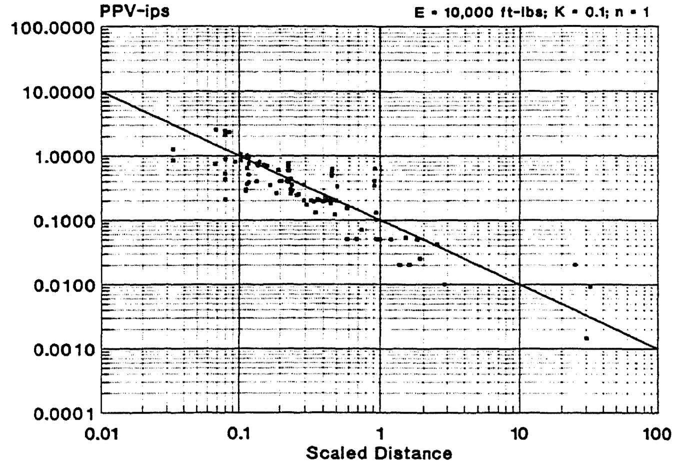


Figure 2. PPV vs Scaled Distance

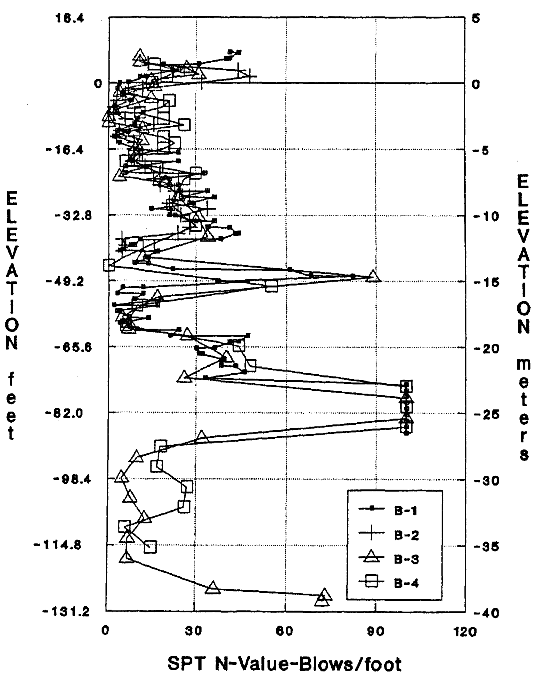
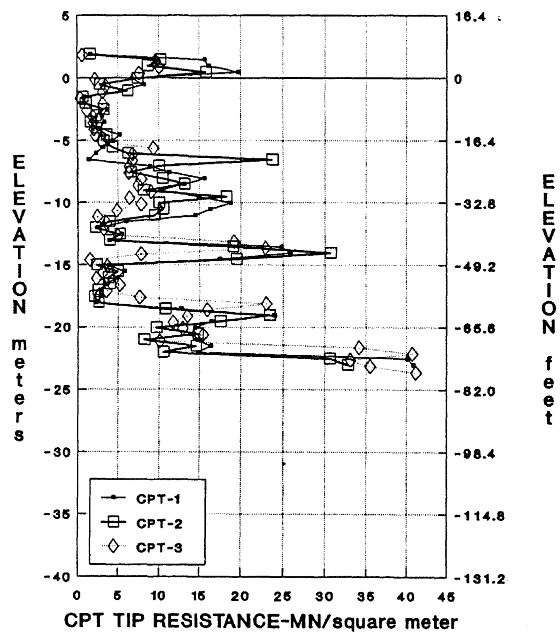


Figure 3. SPT N-values



1 MN/square meter = 10.44 TSF

Figure 4. CPT Tip Resistance

Settlement Monitoring

Settlement monitoring was performed around individual piles as well as for a five-pile group during the preproduction load test program. Monitoring was also conducted near a set of existing railroad tracks, located about 150 feet from the nearest pile being driven.

The results are shown on Figure 5. As noted, movements ranged from about 1/2 inch of heave to nearly 3 inches of settlement. The solid line shown represents the average of all movements measured, while the dashed line represents the limits of maximum measured ground movement. The amount of measured heave was about 20 percent of the absolute value of measured settlement within about 10 feet of the pile. At distances greater than about 20 feet, the amounts of heave and settlement were similar. No movement took place at distances beyond about the length of the piles being driven, which supports earlier findings by Dowding (1991).

Vibration Monitoring

Vibration measurements were made with both accelerometers and velocity pickups. Acceleration data are used to compute the structure response, while velocity is generally a better indication of the energy transmitted to the ground.

The equipment used consisted of three PCB Model 393C high sensitivity accelerometers, two Mark Products Model L4 geophones with 1 Hz suspension, and a TEAC Model MR-30 seven-channel FM magnetic tape recorder. The response of the accelerometers was from 0.25 to 500 Hz; the response of the velocity pickups was from 1 to 200 Hz.

Measurements were taken with the five transducers on the ground surface at various distances from the pile being driven, and on the floor slab of the sensitive structure, which was located about 3,200 feet from the pile-driving operation. The accelerometers were attached to a single mounting block, which was placed on the ground surface or on the concrete floor slab. The velocity pickups were placed individually on the ground surface or floor slab. The data were previewed through an oscilloscope to aid in adjusting the gain for each channel.

In addition to monitoring the response due to pile driving, response due to a locomotive was measured. The locomotive was used to transport equipment to and from the sensitive structure. The tracks were approximately 50 feet from the structure foundation and approximately 150 feet from the monitoring point.

Analysis of Results

A typical acceleration time-history plot is shown on Figure 6 for both a pile being driven and a locomotive passing nearby. Both are shown for damping ratios of 1/2 percent. The peak velocities at various distances from the pile driving are shown in Figure 1, site 1.

Assuming simple harmonic motion, particle velocity and acceleration are related by the following expression:

$$V = \frac{a}{2\pi f} \qquad (2)$$

where: V = peak particle velocity
a = acceleration
f = frequency

Using the peak velocities at distances of 150, 250, and 3,000 feet from the pile driving, the computed frequency of the forcing function ranges from 23 to 28 Hz, measured on the ground. The computed frequency measured on the floor slab is about 9 Hz. Table 2 summarizes these results.

Table 2. Measured Velocities, Accelerations, and Computed Frequencies

Monitoring Point	Distance (ft)	Velocity (ips)	Acc. (g)	Frequency (Hz)
Floor	3,200	0.0009	0.00014	9
Ground	3,000	0.0015	0.0006	25
Ground	250	0.042	0.016	23
Ground	150	0.053	0.024	28

The measured velocities are in good agreement with predicted velocities using the earlier reported values of E = 10,000 ft-lb, K = 0.1, and n = 1.

In reviewing the acceleration response, the pile driving resulted in higher amplitudes at frequencies less than about 10 to 15 Hz. At higher frequencies, the locomotive gave higher amplitudes.

At this point, a description of the dynamic analysis used to analyze the structure and equipment inside is appropriate. The government agency performed the dynamic analysis, but, because the structure and equipment housed inside were classified, the results could not be released. Typically, the analysis would consist of modeling the structure and its components in a structural analysis computer program, inputting the motion generated, and determining if the resulting response was within acceptable limits.

CONCLUSIONS

Peak particle velocity measurements from seven sites with varying soil conditions, and different piles and hammers used for installation have been presented. The results confirm previous methods for estimating particle velocities. However, the authors would recommend the use of hammer transferred energy rather than rated hammer energy in the predictive equation. Values of K = 0.1 and n = 1 are reasonable for a first approximation of PPV.

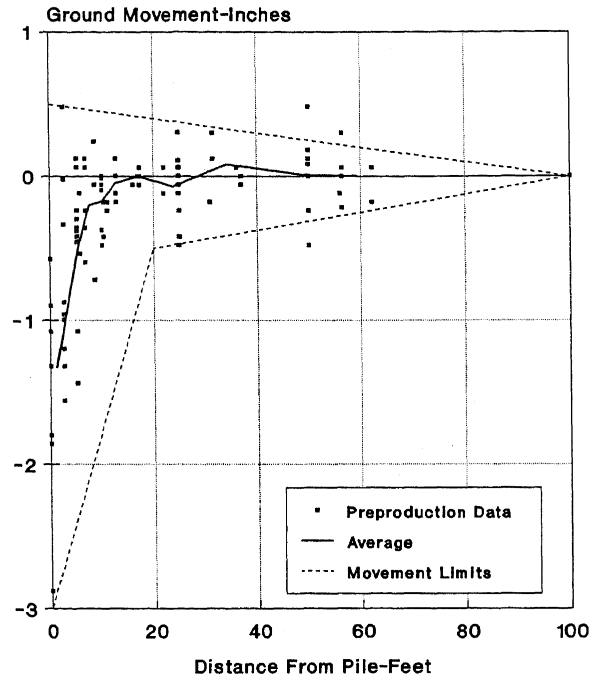


Figure 5. Ground Movement vs Distance

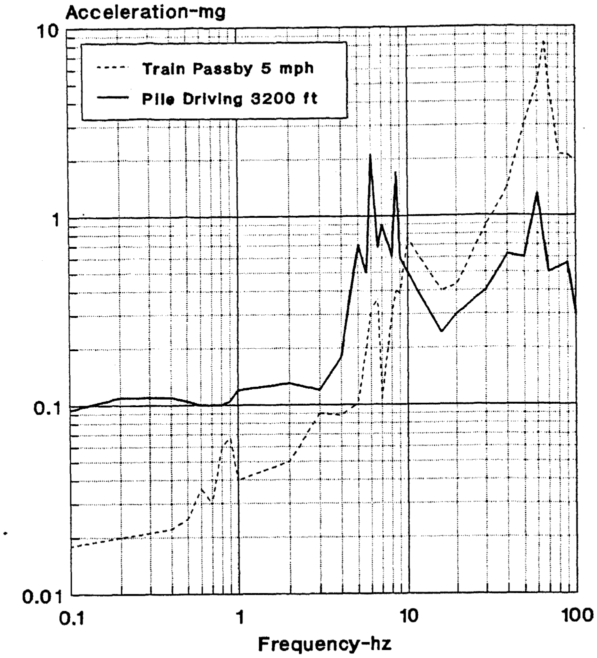


Figure 6. Structure Response

Based on the data presented, at scaled distances greater than 0.1 (distances of 10 feet with transferred hammer energies of 10,000 ft-lb), PPV is less than the commonly accepted threshold limit of 2 ips.

It is recommended that preconstruction walk-downs, inspections, and examinations be made of adjacent structures. These may include photographs, settlement surveys, and room-by-room inspections with tape-recorded transcripts. During pile installation, PPV measurements should be made, settlement surveys taken, and routine inspections of nearby structures made, where possible. In short, a detailed record should be made before and during construction.

If warranted, a more sophisticated response spectrum analysis could be performed. Actual measured response would have to be recorded, generally during a preproduction pile installation/load test program. In the authors' experience, this type of analysis is the exception rather than the rule, but is recommended where sensitive (physically or politically) structures are involved.

The case study confirmed previous results that indicate settlement occurs due to pile driving within loose sands at distances of up to the pile length. When a number of piles are to be driven for a particular project, the cumulative settlement could be significant. Measured values of heave were about 20 percent of the absolute value of measured settlement near a pile, while at distances greater than about 20 feet, values of heave and settlement were nearly equal.

REFERENCES

Dowding, C. H. (1977), "Ground-Structure Response to Blasting Vibration," Proceedings of the 14th Annual Meeting, Society of Engineering Sciences, pp. 1085-1093.

Dowding, C. H. (1991), "Permanent Displacement and Pile Driving Vibrations," Deep Foundations Institute 16th Annual Members Conference, Chicago, IL, pp. 67-84.

Goble Rausche Likins and Associates (GRL) (1988), Case Files.

Heckman, W. S. and D. J. Hagerty (1978), "Vibrations Associated With Pile Driving," Journal of the Construction Division, ASCE, Vol. 4, No. CO4, pp. 385-394.

Medeáris, K. (1977), "The Development of Rational Damage Criteria for Low-Rise Structures Subjected to Blasting Vibrations," Proceedings of the 18th Symposium on Rock Mechanics, pp. 1A2-1 to 1A2-6.

Naik, T. R. (1979), "Predictions of Damage to Low-Rise Buildings Due to Ground Vibrations Created by Blasting," American Concrete Institute SP 60-11, pp. 249-264.

Rubin, R. A. (1978), "What To Do If You Think a Pile Claim Is Coming," Journal of the Construction Division, ASCE, Vol. 4 CO4, pp. 503-514.

Siskind, D. E., M. S. Stagg, J. W. Kopp, and C. H. Dowding (1980), Structure Response and Damage Produced by Ground Vibration from Surface Mine Blasting, U.S. Bureau of Mines, Report of Investigation, RI 8507.

Theissen, J. R. and W. C. Wood (1982), "Vibrations in Structures Adjacent to Pile Driving," Conference Proceedings Geo-Pile '82, San Francisco, CA.

Wiss, J. F. (1981), "Construction Vibrations: State-of-the-Art," Journal of the Geotechnical Division, ASCE, Vol. 107, No. GT2, pp. 167-181.