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Predicted, Measured Ground Vibrations Caused by Pile Driving in Sand at the Tuncurry Unloading Wharf N.S.W.

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SUMMARY At the N.S.W. coastal town of Tuncurry it is proposed to replace an old timber wharf with a structure founded on two rows of driven steel tube piles. To assess the effects of pile driving on the adjacent Fisherman's Co-operative Building an initial assessment was carried out using vibration theory and published vibration monitoring data. A pile driving trial was then carried out with monitoring of ground vibrations using a geophone and three accelerometers. The trial generally confirmed the predicted magnitudes of peak particle velocities but recorded higher than expected peak particle accelerations. The peak particle velocities were higher when driving was carried out in a deeper, denser sand layer.

1. INTRODUCTION

The town of Tuncurry is situated beside the channel entrance to Wallis Lake, approximately 300 kilometres north of Sydney. To serve the local fishing fleet, it is proposed to replace an old timber unloading wharf with a reinforced concrete decked structure, founded on two rows of driven steel tube piles. The proposed wharf is adjacent to the Tuncurry Fisherman's Co-operative Building, which is partly supported on shallow strip and pad footings founded on loose sand fill and marine sand underlain by very dense sand. To assess the effects of pile driving on the Co-op structure, a study of the magnitudes and effects of pile driving vibrations was undertaken.

2. EXISTING AND PROPOSED STRUCTURES

The Fisherman's Co-operative Building is of steel framed brick veneer construction. Apart from the south-east corner, where flooring is timber, it has a suspended reinforced floor.

The building superstructure is generally in good condition with only minor cracking. However, some of the concrete floor beams are spalled in places with exposure of tensile reinforcement which has subsequently corroded.

The proposed new construction comprises excavation of the channel bank beneath the existing wharf to a slope of 1.5H:1V and to a depth of 5.5m, and construction of a new wharf founded on driven 406mm diameter closed end steel tube piles, located in rows approximately 2m and 6.5m horizontally from the Co-op Building.

3. SUBSURFACE CONDITIONS

Based on available borehole and test pit data a geotechnical model was developed for the site. This model, showing the range of tidal water levels is superimposed on the proposed bank slope in Figure 1.

4. PILE DRIVING VIBRATION AND DAMAGE

4.1 Wave Propagation Theory Applied to Pile Driving

A description of wave propagation theory applied to pile driving is given by Schwab and Bhatia (1985).

Within a homogeneous isotropic linear elastic (HILE) medium, there can exist both compression and shear waves known as body waves. In saturated soil, compression waves will travel solely through the water portion as the water is essentially incompressible. Since water has no shear strength, the velocity of shear waves is a function of the elastic properties of the soil skeleton.

When body waves arrive at the surface of a HILE halfspace, surface waves known as Raleigh waves are produced.

For an idealised pile driving situation, the tip of the pile can be thought of as the wave generation source. As the pile is driven, spherically expanding body waves travel outwards to the soil surface where they are reflected and/or refracted, leaving Raleigh waves expanding on cylindrical fronts near the surface. As the waves expand outwards, they encompass an increasing volume of soil thereby lowering their energy density though geometric damping. As body waves expand on a spherical front their decay is much more rapid than that of the surface waves.

Vibrations are also attenuated by frictional damping. However, the contribution of the frictional characteristics of the soil is small when compared to the geometric component (Schwab & Bhatia, 1985).

The attenuation of particle velocity and acceleration can be calculated theoretically from kinetic energy equations. The following equation for predicting peak particle velocities was proposed by Attewell & Farmer (1973):

$$V = K \frac{\sqrt{Wo}}{r} \qquad \text{... Equation (1)}$$

Where: V is the peak particle velocity in mm/sec.
Wo is the energy at the source in joules.
r is the distance from the pile tip in metres.
K is an emperical constant usually ranging from 0.25 to 1.5.

Various equations for predicting peak particle acceleration have also been proposed by Attewell & Farmer (1973), Dalmatov et al (1968), Clough & Chameau (1980). These equations can only be used to describe the shape of the attenuation curve once a peak particle acceleration value at some point is known.

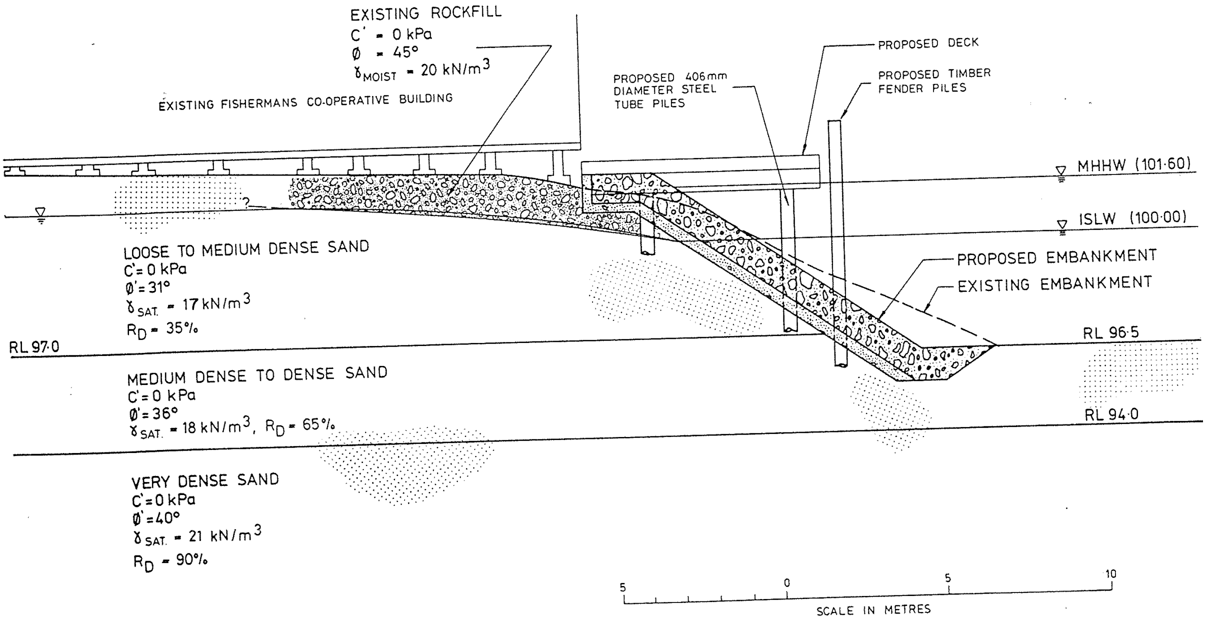

FIGURE 1 GEOTECHNICAL MODEL

4.2 Causes of Vibration Damage

Vibration damage may be categorised as:

(1) Direct Vibration Damage - Vibration waves propagating through the elements of a building exposing one or more of those elements to ultimate tensile or shear stresses, or

(2) Indirect Vibration Damage - Vibration waves travelling through the foundation and causing the foundation to displace in a manner detrimental to the building.

Vibrations within the structural elements can sometimes be amplified by resonance.

For vibrations propagating through the elements of a building, the criteria for a "safe" level of vibrations is defined in terms of a maximum peak particle velocity, sometimes coupled with a frequency range.

For limiting the displacement of foundation soils by vibrations, the criteria for safe levels of vibration are much less well defined. However, vibrocompaction, generation of excess pore pressures, liquefaction and reductions in slope stability are generally best estimated using the peak particle acceleration.

Holmberg et al (1984) point out that some vibration damage may best be termed accelerated ageing. They suggest that it is relatively uncommon to find damage generated directly by vibrations in buildings previously undamaged. The differentiation between direct vibration damage and accelerated ageing is important as the pre-existing condition of a building, in particular the static stresses already influencing the building elements, can have a major influence on the minimum vibration levels at which damage occurs. It is important that any distress such as cracks in plaster be documented prior to the commencement of the vibrations, not only to provide a pre-construction record, but to judge the susceptibility of the building to vibration damage and to choose appropriate criteria.

4.3 Damage Criteria

Vibration damage criteria applied in different parts of the world to vibrations caused by pile driving are summarised in Table 1.

For the Co-op Building, the client indicated that minor damage to the building would probably be acceptable. A maximum peak particle velocity of 15mm/sec was considered an appropriate vibration limit.

Damage criteria applied to vibrations in foundations causing foundation displacement depend on such factors as in situ stress and density state of the soil, groundwater conditions and the buildings tolerance to differential settlement. As a general guide, Holmberg et al (1984) suggest that a small increase in the density of sand (presumably very loose) can occur at acceleration levels as low as 0.1g to 0.2g.

5. PREDICTED GROUND VIBRATIONS

5.1 Predicted Peak Particle Velocities

Peak particle velocities predicted from equation 1 using different assumed K values are plotted on Figure 2. Predictions are summarised in Table II.

5.2 Predicted Peak Particle Accelerations

Peak particle accelerations were predicted after Clough & Chameau (1980), who published data from field measurements of vibration levels for hard driving of sheet piles using vibratory driving equipment (see Figure 3). Estimates are given in Table II.

5.3 Predicted Settlements

Published experimental and field observed relationships for sand settlement as a function of density and peak particle acceleration of vibration are presented on Figure 4. Predicted settlements of the upper loose to medium dense sand layer due to driving of the steel tube piles are summarised in Table III.

TABLE I - VIBRATION DAMAGE CRITERIA

REFERENCE	MAXIMUM RECOMMENDED PEAK PARTICLE VELOCITY (mm/sec)	TYPE OF STRUCTURE	COMMENTS, FREQUENCY RANGE
German Institute (1983)	8*	Sensitive Buildings	10 to 50 Hz
Esrig & Ciancia (1981)	13	Historic Buildings	30 ft soil over bedrock
Swiss Association (1978)	3*	Historic Buildings	10 to 30 Hz
	3- 5*		
	12*	Building in steel or reinforced concrete	10 to 30 Hz
	12-18*		
Holmberg et al (1984)	5**	Any, normal circumstances	For frequencies produced by piling
Konon & Schuring (1983)	6	Sensitive older buildings	1 to 10 Hz
	13	Sensitive older buildings	10 to 100 Hz

* Resultant of vertical, longitudinal and lateral peak particle velocities
** Vertical component of peak particle velocity

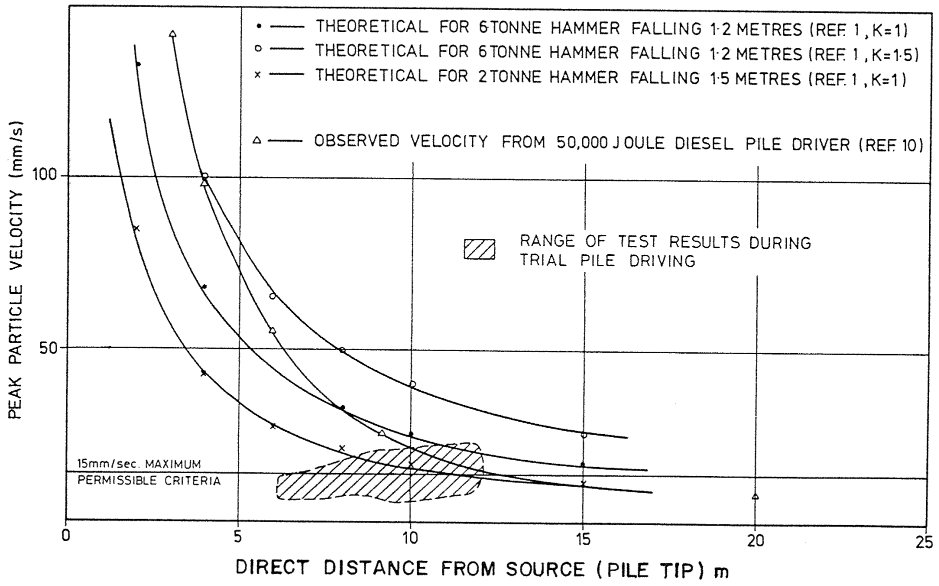

FIGURE 2 PREDICTED PEAK PARTICLE VELOCITIES

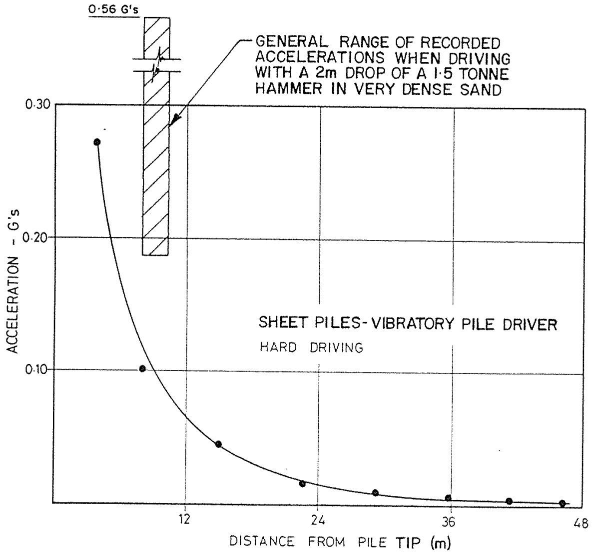

FIGURE 3 PREDICTED PEAK PARTICLE ACCELERATION

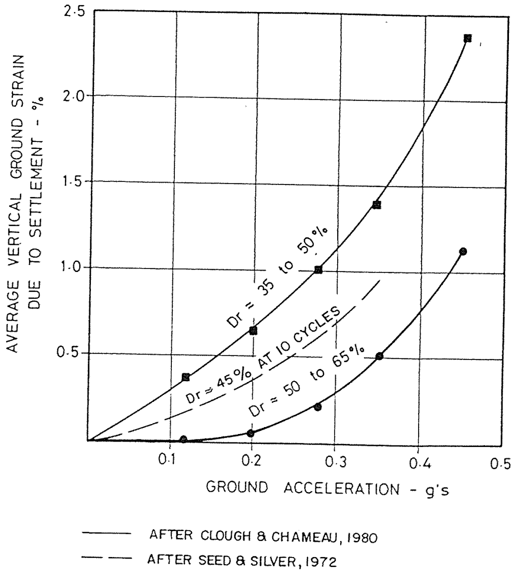

FIGURE 4 GROUND SETTLEMENT VERSUS GROUND ACCELERATION

TABLE II - THEORETICAL ESTIMATES OF PEAK PARTICLE VELOCITY

PILE TYPE	PROBABLE DRIVING CONFIGURATION	DISTANCE OF PILE TIP FROM CO-OP BUILDING FOOTINGS (m)	ESTIMATED PEAK PARTICLE VELOCITY* (mm/sec)	ESTIMATED PARTICLE ACCELERATION
406mm Dia. Steel Tube	6 tonne hammer 1.2m drop	2.5 to 8 (First row)	< 17 to 100	> 0.25G
406 Dia. Steel Tube	6 tonne hammer 1.2m drop	7.5 to 11 (Second row)	< 12 to 35	0.11G

* Assumes K = 0.5 to 1.0

TABLE III ESTIMATED GROUND SETTLEMENTS

DISTANCE OF PILE TIP FROM BUILDING	SETTLEMENT AT NEAREST FOOTING	SETTLEMENT AT SECOND NEAREST FOOTING	ANGULAR DISTORTION	MAXIMUM ALLOWABLE ANGULAR DISTORTION (after Polshin & Tokar (1957)
2.5m (first row)	55mm	40mm	1/130	1/400
7.5m	17.5mm	12.5mm	1/400	1/400

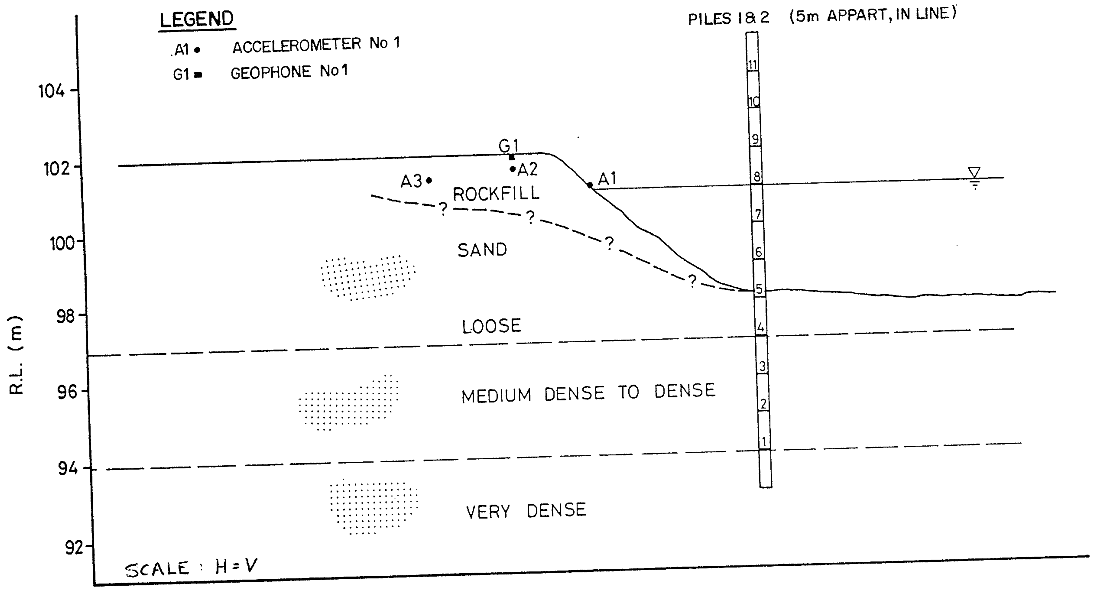

FIGURE 5 PILE DRIVING TRIAL

5.4 Slope Stability

Dynamic slope stability analyses were carried out using Sarma's Method, (Sarma, 1979). Assuming a peak particle acceleration at the centre of mass of the slope of 0.15G's the slope during pile driving had a factor of safety of just greater than 1.1 which was considered acceptable.

Insufficient data on sand in situ properties was available for a quantifiable assessment of liquefaction potential. However, experience with pile driving at other locations along the existing bank indicated that liquefaction was not likely to be a problem.

6. MONITORED GROUND VIBRATIONS

6.1 Pile Driving Trial

The pile driving trial involved driving two 355mm x 375mm "H" piles using a 1.5 tonne hammer with various drop heights. Vibrations were measured using 3 Bruel and Kjaer 4369 accelerometers with amplifier and cassette recorder and a triaxial geophone with a Sinco S4 peak vibration monitor. These were installed at horizontal distances ranging from 4.6m to 8.4m from the piles. The accelerometers were affixed to cobbles in the sand fill in backfilled trenches approximately 0.5m deep. The geophone was buried beneath about 0.2m of sand at the crest of the slope.

The locations of the trial piles in relation to vibration monitors are shown on Figure 5.

6.2 Recorded Peak Particle Velocities

The peak particle velocities recorded during the trial are plotted on Figures 2 and 6. Pile driving within the loose and medium dense sand recorded velocities of generally less than 15mm/sec for all distances. Once driving proceeded into the very dense sand the velocities increased considerably, with values as high as 22.7mm/sec being recorded. Even with low energies, such as a 1m drop of the 1.5 tonne hammer, velocities above 16mm/sec were recorded at the closest monitoring location.

The highest peak particle velocities were recorded on the accelerometer furthest from the source. This could have been due to the geometry of the slope and the presence of the shallow layers of rock fill and looser sand which resulted in a more efficient transmission of vibrations from the pile tip to the furthest monitor. Different degrees of fixity of the accelerometers to the ground could also have been a contributing factor.

In summary, the trial indicated that for the 70 kJ driving energy required for the proposed steel tube piles, peak particle velocities in the range 17 mm/sec to 22 mm/sec would occur at the Co-op Building footings while piles in the second row were being driven. These values were within the predicted range of 12 mm/sec to 35 mm/sec, but above the maximum recommended limit of 15 mm/sec.

6.3 Recorded Peak Particle Accelerations

Peak vertical particle accelerations when driving in the very dense sand ranged between 0.19g and 0.56g, when driving with a 2m drop of the 1.5 tonne hammer. These values were much higher than those predicted using the records of Clough & Chameau (1980), as shown on Figure 5.

7. CONCLUSIONS

The principal conclusion reached from the pile driving trial was that the vibration levels due to driving both rows of proposed steel tube piles would exceed the recommended maximum levels with regard to both direct and indirect vibration damage. An alternate method of construction using groutcrete piles, including a contiguous pile wall to solve the slope stability problem was proposed, but reservations concerning the durability of the piles in a tidal environment were expressed by a major groutcrete piling contractor and this alternative was rejected. It is most probable that the problem will be solved by relocating the new wharf.

With regard to prediction of peak particle velocities, the major factor found to affect velocity levels during the trial was the density of the sand into which the pile was being driven. Even though the predictions were approximately in line with recorded velocities it is interesting to note that this factor received very little attention during the theoretical study.

It appears that velocities were primarily dependent on pile set: When the set was large (in loose sand) the driving energy was presumably converted to frictional energy as the soil below the pile tip failed. In the dense sand, the energy is mostly transferred to the soil as an elastic shock which radiated outward in a form roughly predicted by the vibration theory. The higher velocities produced by driving in the dense sand were readily transmitted to the monitoring instruments through the loose sand layers.

Based on the trial, best fit K values for use in equation (1) were found to be 0.2 to 0.4 for driving in the loose sand, and 0.6 to 1.2 for driving in the very dense sand.

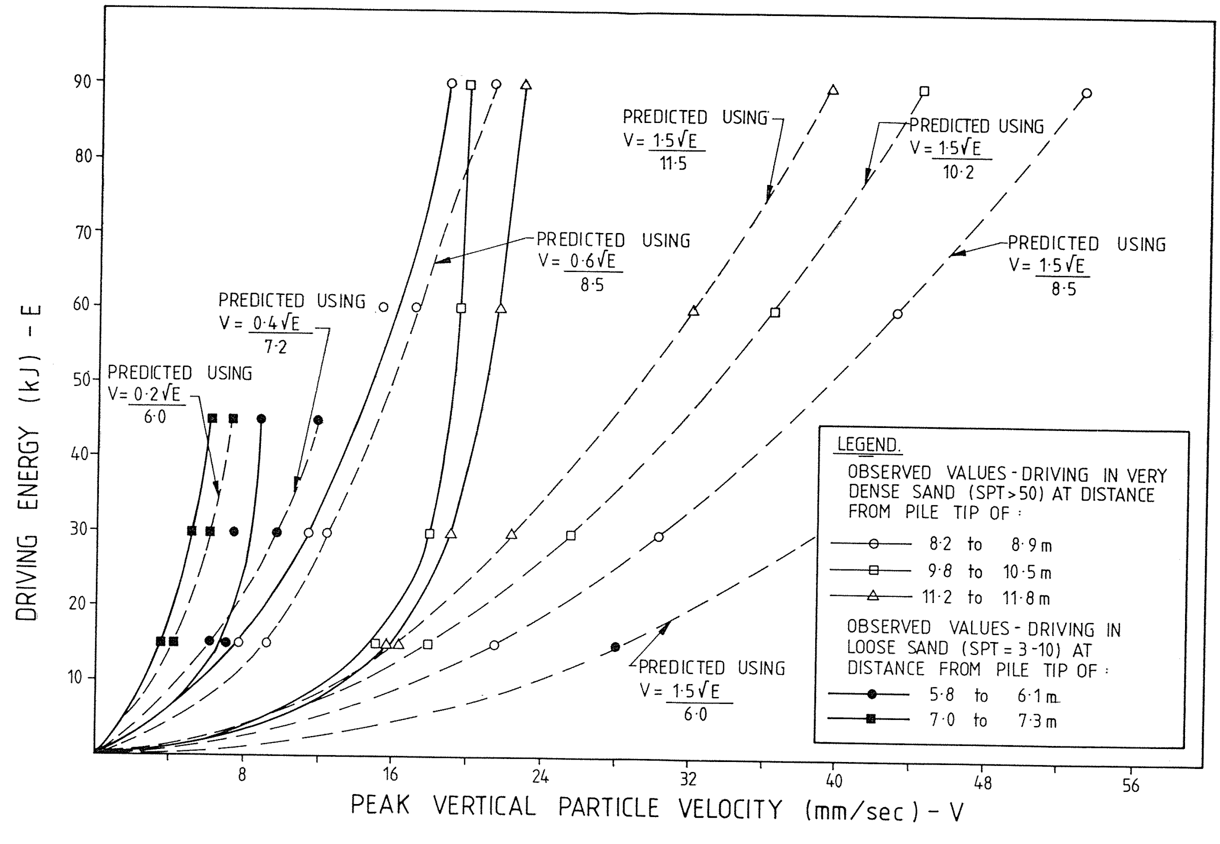

FIGURE 6 PREDICTED AND OBSERVED PEAK PARTICLE VELOCITIES

With regard to peak particle acceleration, underprediction based on published data was probably due to differences between vibrating and impact pile driving as well as differences in energy levels.

8. ACKNOWLEDGEMENTS

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