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PILING-INDUCED GROUND MOTION: A CASE STUDY INVOLVING HYDROCARBON EXPLOITATION ACTIVITIES IN THE NIGER DELTA

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ABSTRACT

The damage potential of piling-induced vibrations in the humid tropical soils of southeastern Nigeria has been evaluated. The vibration propagation was characterized by a fairly high attenuation coefficient, as evidenced by a rapid decrease of velocity amplitude with distance from source. It was also observed that the bulk of the peak velocity amplitudes measured or calculated fell within the safe limit for structural safety and human tolerance. This therefore implied that the zone of highest damage probability at the case study site did not extend across property line as suspected prior to commencement of piling. In general, the findings and results of this work should enhance the development of environmental impact assessment framework suitable for managing those oil production activities that generate transient-type vibrations.

INTRODUCTION

Ground movements frequently result from piling activities carried out during hydrocarbon exploitation. Depending on the piling features and controlling parameters, such as hammer weight, drop height and impact energy, the ground movements can constitute a nuisance. In particular, they may cause foundation vibrations across property lines, which may lead to structural damage to buildings and constructed facilities.

This paper presents the findings of an investigation into the damage potential of such vibrations in the humid tropical soils of the Niger Delta in southern Nigeria. The investigation was carried out using prototype earth tremors arising from piling operations in a chosen site within the area. The objective of the investigation was to establish a threshold level for the ground movements triggered off in the Niger Delta soils by the piling operation that accompanies oil exploitation. This threshold will be used to develop a model for predicting ground movements and structural vibrations that may be encountered in the course of oil and gas production, a booming industry in the Niger Delta. This has become particularly necessary in view of the growing interest in the environment and the need to forestall damage to third party property.

The results of this investigation will be used to demonstrate the extent to which damage potential is affected or influenced by soil characteristics, particularly the textural and structural characteristics, stress history, drainage condition and degree of saturation, as well as by the load magnitude and duration. By comparing the measured velocity amplitudes with the internationally accepted damage criteria for vibrations, the probable degree of damage to structures and disturbance to human beings expected of the simulated piling-induced ground motion will be established for the study area in particular, and the Niger Delta in general.

PILING OPERATION

Nature of Loading from Piling

The piling activity that generated the ground motions for this investigation involved driving a total of 7 (seven) lengths of pile, each approximately 12 metres long. The average value for the number of blows per metre length of pile penetration varied appreciably, ranging from a low of 103 to a high of 373. The pile driving represented a typical impact loading, similar to blasting. It generated transient-type motions and the effect on structures across property line was carefully monitored by directly measuring ground motion amplitudes in terms of particle velocity and displacement.

The choice of velocity and displacement amplitudes as parameters for quantifying the ground motion was based on the well-known fact that maximum particle velocity is an accepted criterion for evaluating the potential for structural damage induced by vibrations and can be approximately correlated with the Modified Mercalli Intensity in strong ground motion problems. The ground displacement, on the other hand, is known to be directly related to the strains to which structures might be subjected.

NATURE OF NIGER DELTA SOILS IN THE CASE STUDY AREA

<u>Site Location and Description</u>

The project site is located in Omoku, a town approximately 100 Km N. W. of Port Harcourt in the Rain Forest vegetation zone of Southeast Nigeria. The area is virtually flat, featureless and low-lying, with a ground elevation of about 9.5m above mean sea level. The landform here falls under the "Deltas and River Plains" of Southern Nigeria (Ejezie, et al, 1983). More specifically, the site lies within the flood plains of Omoku River, whose valley is at about 400m to the west and 600m to the north of the Well point.

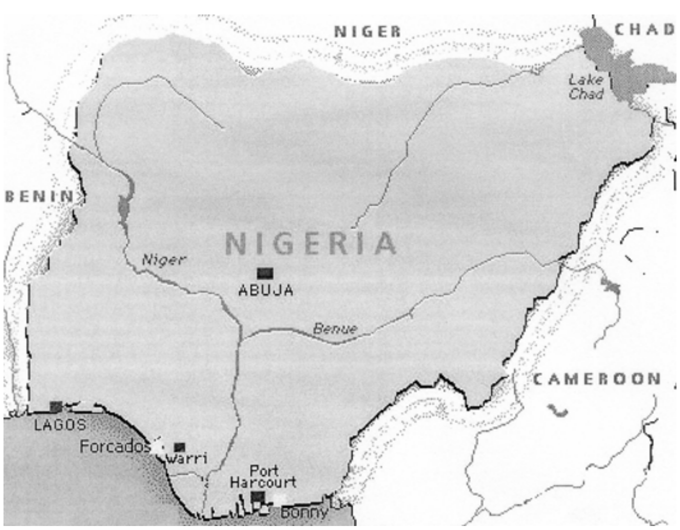


Fig. 1. Map of Nigeria showing the location of Port Harcourt

<u>Site Geology and Subsoil Characterisation</u>

The surface geology is dominated by Quaternary to Recent Deltaic plain deposits, consisting of unconsolidated fine-grained sediments and extending to a depth of about 7.0m. This is underlain by a thick accumulation of "Continental Sands" belonging to the Tertiary "Benin Formation", which is characterised by unconsolidated, medium to coarse-grained, poorly sorted Sands with thin layers of soft grey shale and beds of lignite. This sequence is interrupted, at a depth of about 2380m, by a 100m – thick transition zone consisting of continental Sands as above but with more frequent shale inter-bedding. From a depth of 2480m downward, the succession changes to marine sediments, consisting of Sands and Sandstone with thicker shale inter-beds.

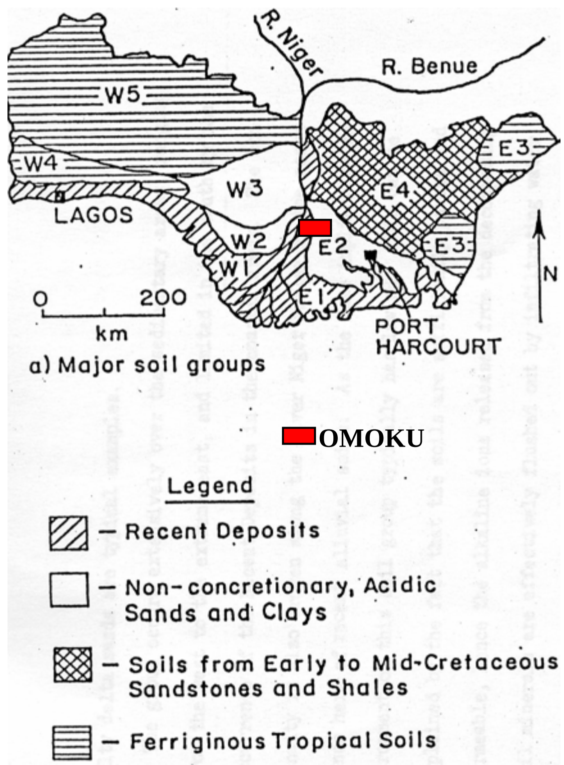


Fig. 2. Major Soil Groups of the Rain Forest Belt of Southern Nigeria including the Study Area (After Ejezie, et al, 1983)

Geotechnical characterisation of the site was carried out, designed to define the soil profile and evaluate the soil properties that would aid the analysis of vibration transmission through the soil. This was achieved using samples from 20m – deep exploratory borings, optimally distributed over the site to yield information that could be regarded as truly representative of the average subsoil conditions in the area. The results show that the water table is at an average depth of 8.3m below the ground surface, with a range of 8.0m - 9.0m.

Based on the results of laboratory tests and the information from the revealed soil profile, the soil in the upper 7.0m as identified above, is clayey and silty "Lateritic Sand". This "laterite" has low plasticity and exhibits a contractive behaviour, as inferred from the compressibility test, which yielded values of over-consolidation ratio between 1.0 and 4.0.

GROUND MOTION FEATURES AND CHARACTERISTICS IN THE CASE STUDY AREA

General Appraisal of In-Situ Soil Response to Dynamic Loads

As explained earlier, the piling operation under study is a form of dynamic loading on the subsurface materials at the site. Soils are known to respond to this form of loading in a variety of ways, which depend on their textural and structural characteristics, stress history, load magnitude and duration, drainage condition and degree of saturation. These factors are briefly investigated here with particular emphasis on their influence on the behaviour of the soils at the project site under the induced vibrations.

Textural and Structural Characteristics. Cohesive and cohesionless soils behave differently under dynamic loads. For example, loose Sand and Silt have a high potentiality for deformation and rapid build-up of excess pore pressure under saturated and undrained conditions. During dynamic loading this pore pressure increases progressively, eventually leading to total strength loss. Cohesive soils, on the other hand, are relatively impermeable, possess little potential for deformation, and do not generally generate excess pore pressure to a point that results in zero effective stress. Rather, strength deterioration or even an improvement may be observed depending on the stress history (Ejezie, 1984, 1987).

Geotechnical investigations at the project site revealed that the soils were predominantly cohesionless sands except for the upper horizon consisting of very clayey lateritic sand, with appreciable cohesion. It was also found that the lateritic soil possessed a fairly low permeability. On the basis of these it was concluded that while this upper lateritic soil was likely to experience only a gradual excess pore pressure build-up and strength change under dynamic loads, the underlying sand would experience rapid pore pressure development and strength deterioration.

Stress History. Consolidation tests on soil samples from the project site showed that the soils have over-consolidation ratios in the range typifying normally consolidated to lightly over-consolidated soils. In other words, the soils are contractive. They are therefore likely to experience undrained shear strength reduction under dynamic loads, such as the piling operation under study. This inference is based on the general concept that contractive soils usually experience strength reduction while dilative soils experience an increase when subjected to undrained dynamic loading. This behaviour is related to the nature of pore pressure change in these soils during the loading. Usually the contractive soils develop excess positive pore pressure, the build-up of which is attended by strain accumulation and undrained shear strength degradation. On the other hand, heavily over-consolidated soils develop excess negative pore pressures, which lead to strength improvement. They also accumulate strains at a relatively small rate (Sangrey *et al*, 1978; Ejezie, 1984).

Load Magnitude and Duration**.** The vibrations resulting from the piling imparted loads of low magnitude to the surrounding soil through which they were transmitted. As a result, the soil response was elastic and failure did not generally occur. This can be explained using the concept that the response of soils to dynamic loading is limited by the load magnitude. A critical level of loading is known to exist below which pore pressure and strain accumulations attain an equilibrium state after a large number of cycles. The soil then behaves elastically and further changes in strain or pore pressure with increasing number of cycles are recoverable. Hence, theoretically soil failure here is not very likely. Above the critical load however, pore pressure and strain progressively build up as load cycles increase, and soil failure ultimately occurs (Sangrey *et a1*, 1978; Ejezie, 1984, 1987).

It must however be pointed out that although soil failure was not generally observed around the site, the soil within the hole or immediately around the pile shaft experienced much higher load magnitude from the piling hammer. Within this zone therefore there was soil failure or pronounced reduction in soil strength.

Drainage Condition. The influence of this factor is uniquely related to soil type and stress history. For most dynamic loading problems undrained condition is assumed because the loading interval is usually too short to allow significant drainage or pore pressure dissipation. As noted earlier, undrained loading usually causes strength degradation or loss in contractive soils, and an improvement in heavily over-consolidated soils. On the other hand, drained dynamic loading enhances the strength of contractive soils but reduces that of the dilative soils.

In the project under investigation the soils are contractive but predominantly cohesionless. The loads imparted by the vibrations were in fairly rapid successions. Undrained condition was therefore a good approximation. However because of the low cohesion, appreciable drainage most likely occurred between the load cycles. This might have contributed to the observed strength behaviour (non-failure).

Degree of Saturation**.** This governs the pore pressure development and hence, Shear strength behaviour of soils. When completely saturated, a soil under rapidly applied dynamic loads is assumed undrained. When not saturated, drained condition is a better approximation and the response is directly proportional to the degree of saturation (Ejezie, 1984).

Based on the observed water table depth (8.0-9.0m) the soils at the project site were not considered completely saturated. As a result pore pressure response was low, and this resulted in an increase in shear strength and resistance to deformation of the contractive soils.

Dynamic Load Soil Response Models for Case Study Area

These models have been formulated to define the behaviour of soils subjected to dynamic loads in terms of the relationships among the various governing parameters. The existing models fall into three broad categories namely, soil deformation models, pore water pressure response models, and soil strength models (Ejezie, 1984).

The soil deformation models considered to be the most suitable for defining the deformation behaviour of the soils at the case study site are the simplified stress-strain models. These include the "Hyperbolic" and the "Ramberg-Osgood" formulations, which can be adopted to give the relationship between stress and strain in a soil during dynamic loading (Ejezie, 1987). Like other soil deformation models, they uphold that repeated load applications can cause appreciable strains or volume changes in a soil mass. Depending on the load magnitude, it is possible for the strains to increase beyond bounds as the load cycles increase in number.

Pore pressure response models generally incorporate the fundamental concept that cyclic loading causes a build-up of pore-water pressure, which could result in soil failure or non-failure depending on the soil characteristics, loading and drainage condition. For the soils at the project site the "critical state limiting pore pressure response model" is deemed ample. This model is capable of predicting pore pressures in all types of soils under different magnitudes of dynamic loads and stress histories. Its application assumes complete saturation of the soil, which is not the case at the present case study site. However. this is not likely to introduce any appreciable error.

The soil strength model appropriate for the soil in the case study site is the "post cyclic loading peak strength model". This model generally describes the end effects of dynamic loading on the shear strength of soils. These effects depend on the stress history, water content, drainage condition during loading, and magnitude, duration and type of the applied load. These factors have been duly considered as they apply to the project site before concluding that the model is a suitable formulation for the strength behaviour of the soils. This model can predict the soil strength immediately after the dynamic loading (undrained), a long time after (drained), and at intermediate states of the load cycle (partial drainage).

Vibration Monitoring Operation

The vibration monitoring was designed to measure directly the round motion amplitudes at various points around the site. The measurements were extended across property lines and expanded radially outwards with respect to the source and along the four cardinal axes - East, West, North and South. The monitoring stations were located at 50m intervals along these axes. The parameters measured were the particle velocity and displacement. Maximum values of these parameters were recorded regardless of where they occurred during the measurement. At each monitoring station the measurements were generally taken in three mutually perpendicular directions - vertical, radial to source projected on a horizontal plane, and transverse to source also projected on a horizontal plane. The only exceptions though, were those stations not located within or near residential buildings or other structures that could permit easy measurement of the radial and transverse components, such as stations along the Western axis. In these cases only the vertical component was measured.

Two portable seismographs (vibration meters), the 308M vibration/noise level meter and the TK80 vibration meter, were used to monitor the vibrations. Both instruments had the capability of measuring velocity and displacement directly. The maximum velocity readings at each station were vectorially added to obtain the peak particle velocity. Frequencies were computed from the velocity and displacement readings by assuming that the motion was simple harmonic.

This assumption allowed the use of the following relationship in the calculations,

$$U = V/2\pi f \quad (1)$$

$$\text{Or } V = 2\pi f U, \quad (2)$$

$$\Rightarrow f = V/2\pi U \quad (3)$$

In these expressions, U = displacement, V = velocity, and f = frequency.

The following peculiar trends were observed in the readings recorded during the field vibration measurements (presented in the appendix) and they may be explained in terms of specific features encountered during the monitoring exercise, such as:

a) The vertical component of the vibration was generally less in magnitude than the horizontal (radial and transverse) components. This might be due (in part) to the fact that there was a general increase in vibration amplitude from the foundation level up the walls to the roof. The vertical component was always measured at the foundation level (on the floor) while the radial and transverse components were measured higher up on the walls.

b) Vibration amplitudes decreased as the radial distance away from the source increased. This trend was however occasionally distorted where, along a particular monitoring axis, relatively more rigid or stable structures such as concrete buildings were encountered closer to the source than relatively less rigid ones such as mud-houses with bamboo-reinforced walls.

c) Vibration amplitudes were generally greater in the less-rigid structures than in the more rigid ones. Hence, the former category was observed to be more susceptible to damage than the latter.

d) The nearest property line to the source was at a distance of 100m. Hence the bulk of the measurements was concentrated in the region beyond 100m from the piling point.

ANALYSIS OF VIBRATION DATA

Vibration Propagation and Attenuation in Soil.

The data from the monitoring phase of this project were analysed in terms of ground motion – the nature of its propagation and attenuation in the soils around the site, and its effects on structures and human beings. The analysis focused on two main parameters: particle velocity and displacement. This is because, as mentioned earlier, the maximum particle velocity is an accepted criterion for evaluating the potential for structural damage induced by vibrations. In strong ground motion problems it can be approximately correlated with Modified Mercalli Intensity. The critical level of the velocity depends on the frequency characteristics of the structures, frequency of ground motion, nature of the overburden soil, and capability of the structures to withstand dynamic stress. The ground displacement, on the other hand, is directly related to the strains to which structures might be subjected.

The stated objective of this project has been achieved through a rigorous analysis of the piling vibration data. The variation of velocity with distance away from the piling point was ascertained by plotting the velocity readings against the corresponding distances. The plots were made on log-log coordinates based on the vibration propagation law:

$V = K D^n$ (Bureau of Mines, 1971) (4)

Where:

V = particle velocity,

D = distance (monitor station to source, in hundreds),

K = intercept, velocity at D = 1.0 (scaled in hundreds of meters)

n = exponent.

The data were grouped into vertical, radial and transverse components along the East, West, North and South – monitoring axes and plotted. This was designed to complement an analysis of variance performed on the data to determine if significant differences existed in the amplitudes and attenuation of the velocity components along the different axes and also to see if different components could be pooled or combined either for one axis or for all the axes. It was observed that each velocity component showed some significant uniqueness in magnitude and its pattern of variation with distance from source, which was fairly similar along the different axes. Hence it was confirmed that different components could not generally be combined whereas similar components along the different axes could. These deductions agreed with the Geotechnical investigation results, which portrayed the site as possessing a simple geology and the subsoil as homogenous, and further revealed that it was anisotropic. Consequently, the vertical velocity components along the four axes were combined and plotted, and so also were the- radial and transverse components. Finally the peak velocities computed by taking the vector sum of the maximum velocities at each monitor station were combined and plotted. These are shown in Figs. 3 - 6.

The values of K and n were determined for each set of plotted data by statistical analysis using the method of least squares. The values for K represent the average velocity amplitudes along the property lines (D=1.0), while n approximates the rate of attenuation of the velocity with distance from the source. The values of K obtained for the various velocity components are given in Table 1 below.

Based on the results of the foregoing analysis, contours have been developed for velocity amplitudes with increasing distance from the source. This gave a clear picture of the zonation of damage probabilities around the project site as illustrated in Figs. 7 to 10. The graphs and the contours reveal that the vibration died out rapidly with increasing distance away from the piling point. This implied that the effect on structures and human beings across property lines could not spread over an extensive area.

Table 1: Computed values of the particle velocity intercept, K at D = 1.0 (property line, 100m from source) for the various sets of velocity data.

Velocity Component	**Velocity Intercept, k**
Vertical	2.16
Radial	3.07
Transverse	2.60
Peak	3.15

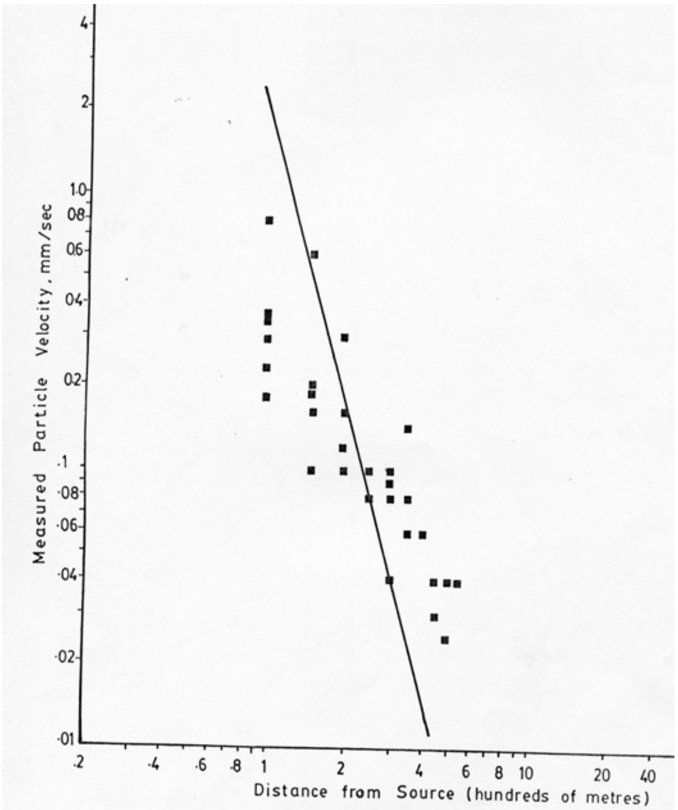
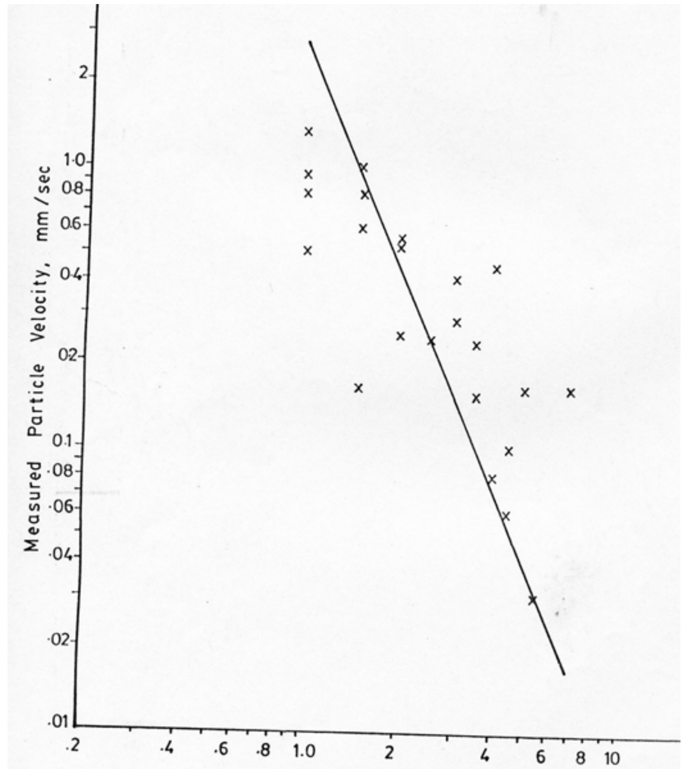


Fig. 3. Vertical Component of Particle Velocity versus Distance from Source



Distance from Source (Hundreds of metres)

Fig. 5. Transverse Component of Particle Velocity versus Distance from Source

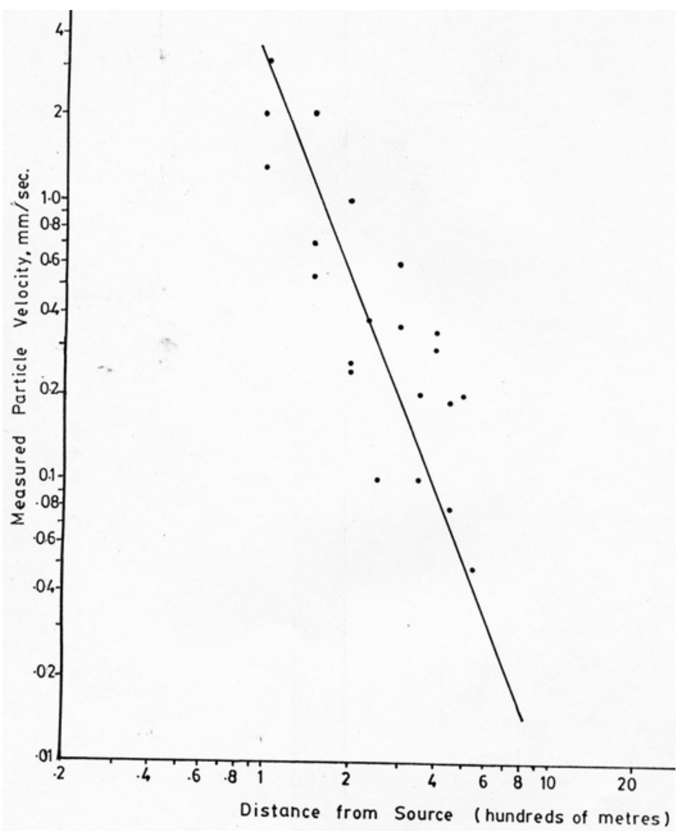


Fig. 4. Radial Component of Particle Velocity versus Distance from Source

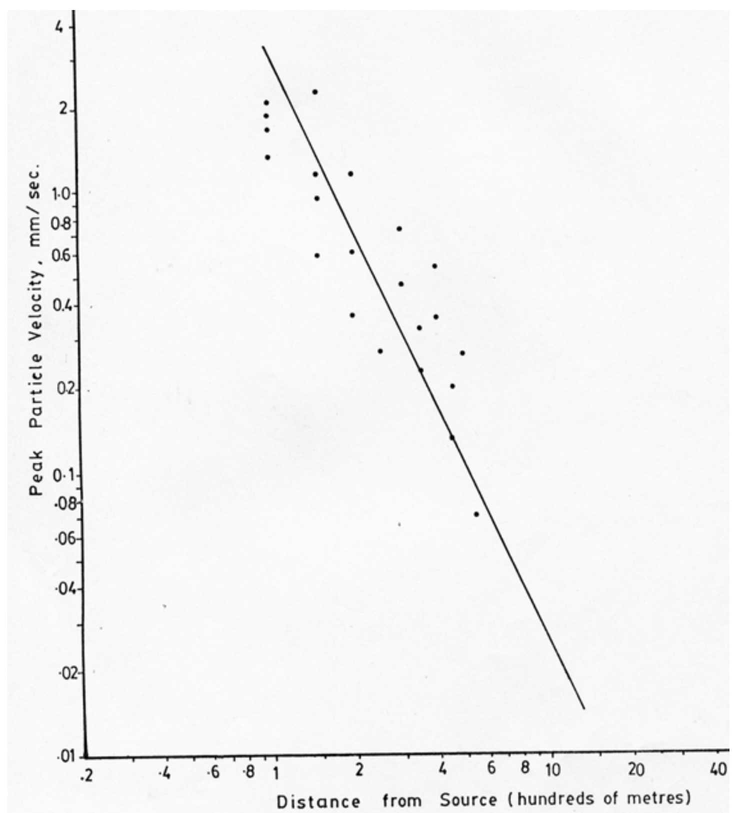


Fig. 6. Peak Particle Velocity versus Distance from Source

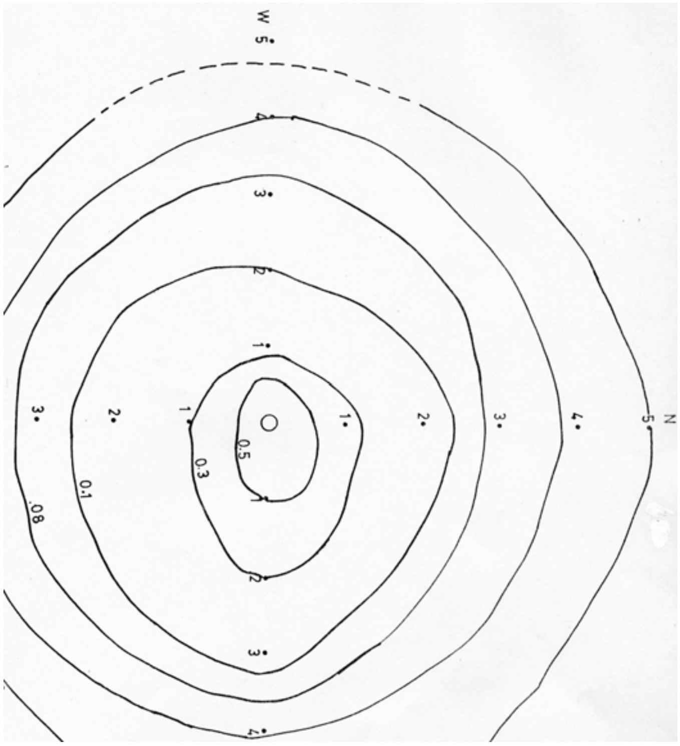


Fig. 7. Contours of Vertical components of velocity, in mm/sec, measured around the Site during Piling. (Whole Numbers along Monitor Axes indicate Distances in hundreds of metres from Source).

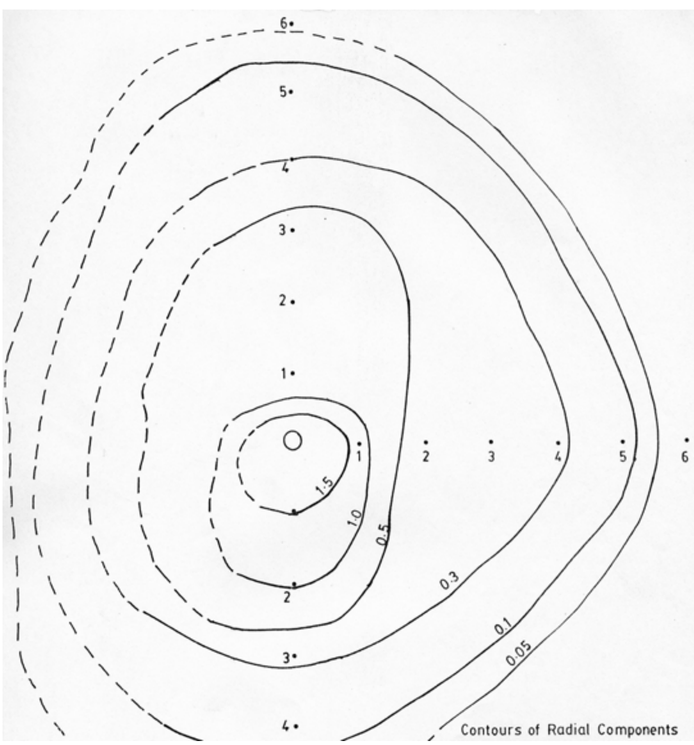


Fig. 8. Contours of Radial components of velocity, in mm/sec, measured around the Site during Piling. (Whole Numbers along Monitor Axes indicate Distances in hundreds of metres from Source

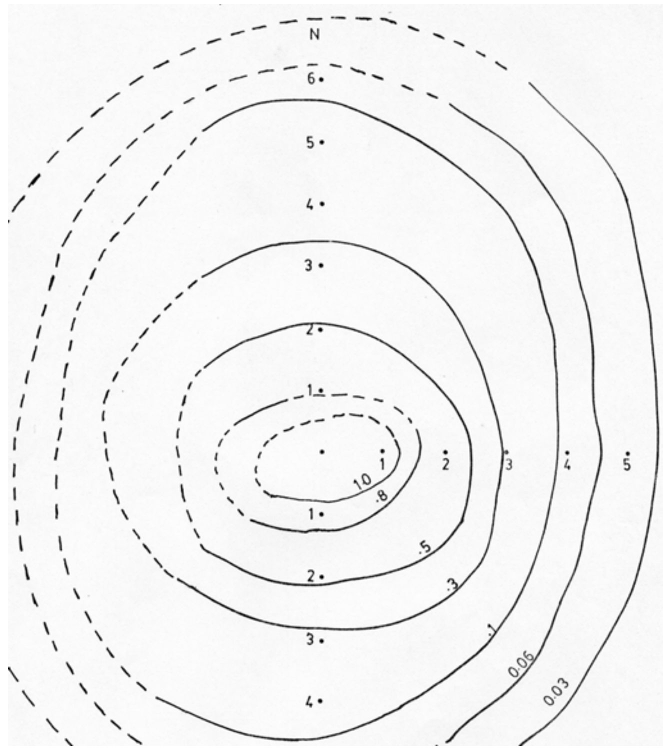


Fig. 9. Contours of Peak velocity, in mm/sec, measured around the Site during Piling. (Whole Numbers along Monitor Axes indicate Distances in hundreds of metres from Source).

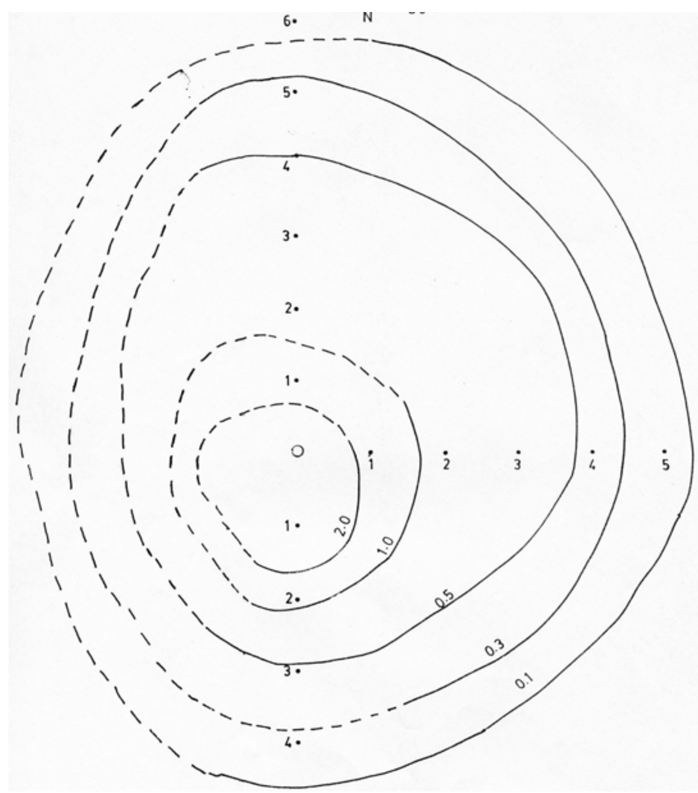


Fig. 10. Contours of Peak velocity, in mm/sec, measured around the Site during Piling. (Whole Numbers along Monitor Axes indicate Distances in hundreds of metres from Source).

MOTION AMPLITUDE VERSUS RESPONSE CRITERIA

Effects of the Vibrations on Structures and Human Beings

Tolerable vibration amplitude decreases as frequency increases. This forms the basis for determining the probable degree of damage to structures and/or disturbance to human beings caused by vibrations from a given source. Since damage is more closely related to particle velocity than to displacement, data on velocity amplitude and frequency were used in this project to establish structural and human response criteria for the monitored vibration to portray the likely reactions of structural facilities and human beings in the vicinity of the piling to the resulting vibrations.

The safe vibration criterion was based upon a consideration of individual velocity components because seismic motion is a vector quantity. Log-log plots were therefore developed for the vertical, radial and transverse components of velocity versus the corresponding frequencies. The peak velocities were also plotted against the maximum frequencies. These individual plots were then combined to reflect all contributions to the vibratory motion experienced at the project site as shown in Fig. 11.

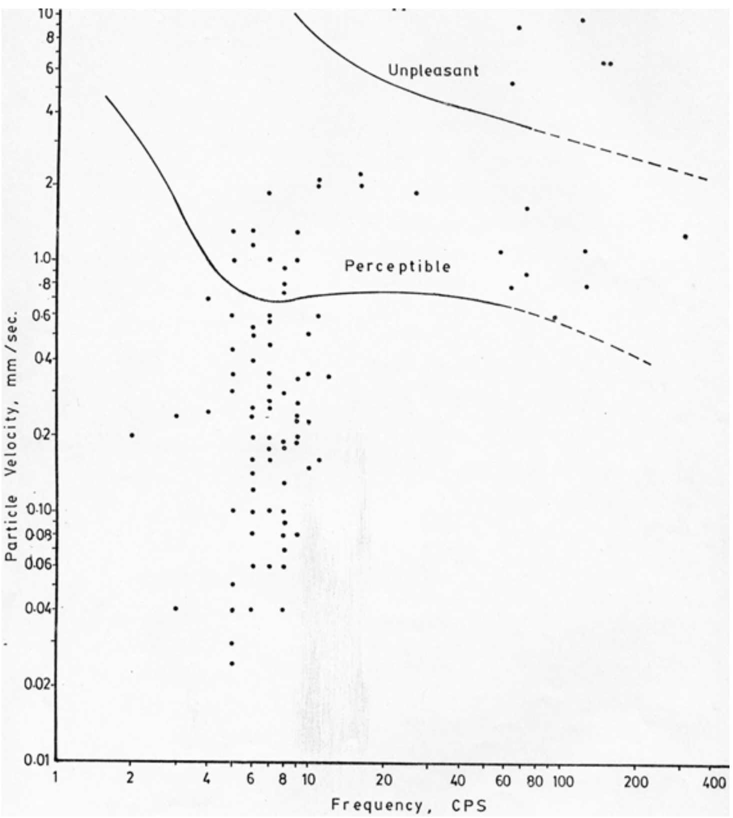


Fig. 11. Relationship between Velocity Amplitude and Frequency

Damage criteria established for vibrations are usually probability-types. Any safe criterion is not a value, below which damage will not occur and above which it must occur. Rather it is a vibration level (in terms of particle velocity), which, if exceeded by the vertical, radial, or transverse components would indicate that there is a reasonable probability that damage will occur. Many structures can experience vibration amplitudes higher than this level without suffering damage.

In terms of human tolerance of vibrations, a lot of subjectivity exists and this introduces appreciable flexibility in establishing human response criteria. For example, some people may consider vibration levels that are completely safe for structures annoying and very uncomfortable. In general, the subjective response of the human body to vibratory motion is categorized into three levels namely, perceptible, unpleasant, and intolerable corresponding respectively to low, medium high and high velocity amplitudes. This scheme was first proposed by Goldman (1948) and has since been developed and adopted for application in a wide range of vibration problems.

Two widely used and internationally accepted damage criteria for vibrations include those developed by:
(i) the Bureau of Mines of the United States Department of the Interior (1971), and
(ii) the United States Department of the Navy (1982).

These are presented in Figs. 12 to 15.

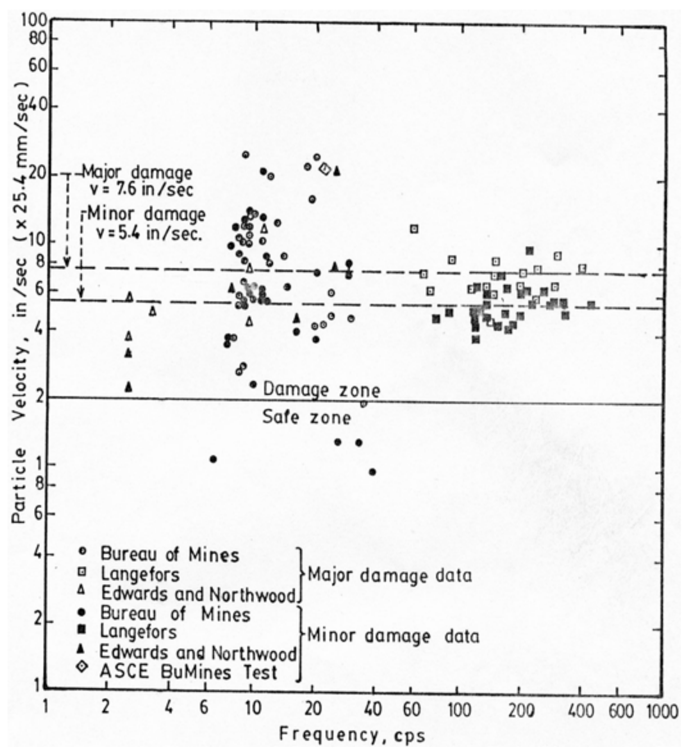


Fig. 12. Bureau of Mines recommended Vibration Criteria

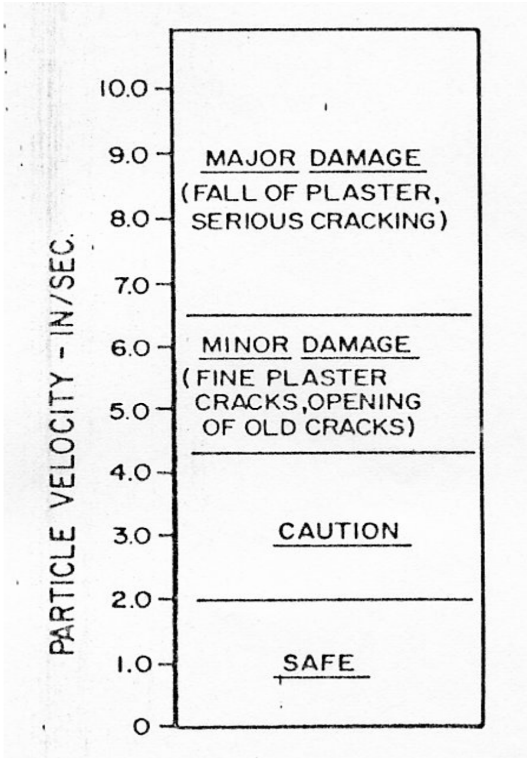


Fig. 13. Guideline for Assessing Damage Potential of Blasting Vibrations on Residential Structures founded on Dense Soil or Rock (After US Department of the Navy, 1982)

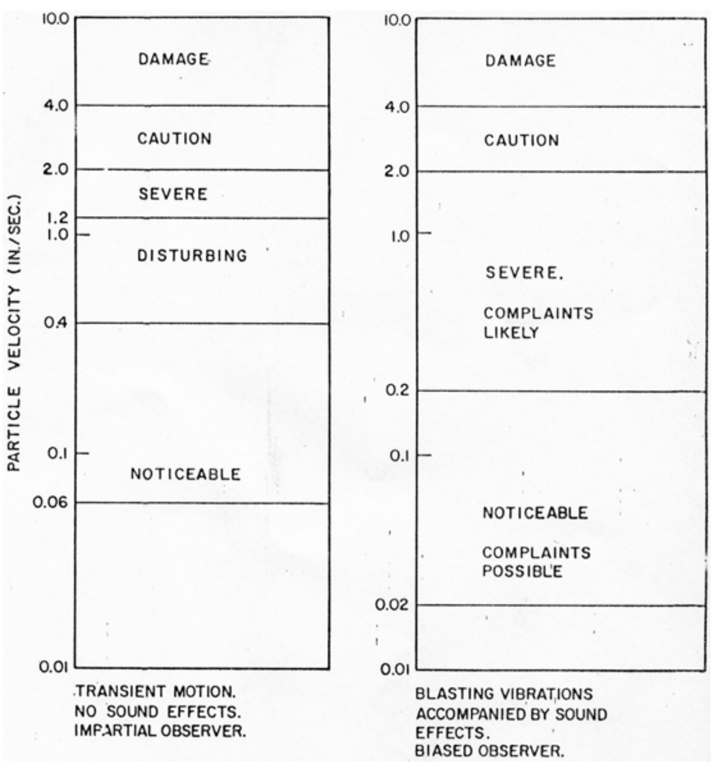


Fig. 14. Guide for Predicting Human response to Vibrations and blasting effects (After US Department of the Navy, 1982)

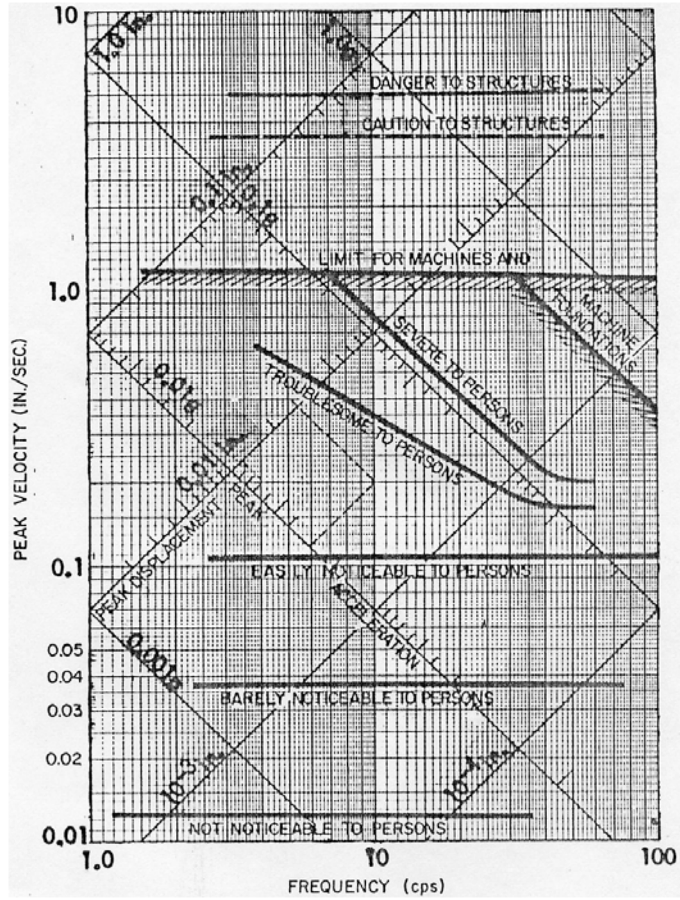


Fig. 15. Allowable Amplitude for Vertical Vibrations (After US Department of the Navy, 1981)

The Bureau of mines criteria stipulate a velocity amplitude of 2.0 in/sec (50.8 mm/sec) as the threshold vibration level below which structures are considered safe and above which structural damage is likely. Within the damage zone, two levels of damage are also identified namely, the minor and major damage levels. The exact boundary between the two is about 6.5 in /sec. (165 mm /sec).

The Department of the Navy adopts similar guidelines for assessing the potential for damage induced by vibrations to residential structures. In this case particle velocities from 0 to 2.0 in /sec (0-50.8 mm /sec) represent the safe zone, 2.0 to 4: 3 in /sec (50.8-109.2 mm /sec) the zone of caution, 4.3 to 6.5 in/sec (109.2-165 mm /sec) the minor damage zone involving fine plaster cracks and opening of old cracks, while velocity amplitudes greater than 6.5 in/sec (165 mm/sec) represent the major damage level involving fall of plaster and serious cracking. Additionally, the Navy criteria incorporate specifications for predicting human response to vibrations. These can be summarized as follows (Hendron, 1976):

< 0.02 in/sec (<0.5 mm/sec): Not easily noticeable to persons;
0.02-0.2 in /sec (0.5-5.0 mm/sec): Noticeable to persons, and complaints possible,
0.2-1.2 in /sec (5.0-30mm/sec): Disturbing, and complaints likely
1.2-2.0 in/sec (30 -50.8 mm/sec): severe.

> 2.0 in/sec (>50.8 mm/sec): Damage likely.
It should be noted however that the limits in the above criteria could shift up or down depending on various factors. For example, if there are no sound effects and the observer is impartial, velocity amplitude of up to 0.06 in/sec (1.5 mm/sec) is needed for the vibration to be noticeable. On the other hand, with a biased observer of vertical vibrations accompanied by sound effects, particle velocity amplitude as low as 0.013 in/sec (0.3mm/sec) may be enough to consider the vibration noticeable. Furthermore, the velocity amplitude required for a particular human response to a given vibration decreases appreciably with increase in frequency.

The plots developed for the measured velocity amplitudes against frequency for this case study, as presented in Fig. 11, disclose the following information:

a) The bulk of the observed frequencies generally ranged from 5 to 30 cps (excepting few values that fall below or above these limits). This is in close agreement with the findings of the Bureau of Mines (1971) that predominant frequencies generated by vibrations from impact loading are commonly in the range from 6 to 40 cps.
b) The velocity amplitudes recorded across property lines during the piling operation were predominantly in the range from 0.03 to 3.0 mm/sec. (Very few measurements gave values in the range, 5 - 15 mm/sec).

On the basis of the above information it is inferred that the peak particle velocities measured fell within the safe zone. The accompanying vibrations were therefore unlikely to cause structural damage in the area. Furthermore, an appreciable percentage of the velocity amplitudes for the piling operation fell within the zone where "vibrations were noticeable and complaints possible." Only very few fell within the "disturbing" zone. Besides, the pile driving was not a continuous process. The longest time taken to drive one pile was about 90 minutes - the first lasted about 10 minutes, the second 20 minutes while the 3rd took 45 minutes. The interval between successive pilings was about 3-4 hours.

It is therefore evident from the above considerations that the velocity amplitudes were within the limits of human tolerance when viewed objectively and without bias. However there is a high probability of complaints against inconvenience from occupants of residential structures located at less than 200m from the piling point. This is owing primarily to sound effects and bias, which are likely to be prominent factors in their response to the vibrations. A greater percentage of these complaints are likely to come from residents of non-rigid buildings such as those of bamboo-reinforced earth.

SUMMARY AND CONCLUSION

Summary

The vibration-monitoring program has enabled accurate determination of the amplitudes of the vibrations, which were transmitted across the property lines from the piling point. The data were used to assess the environmental impact of the vibrations vis-à-vis the acceptable levels for human tolerance and structural safety. The piling operation imparted impact loads that generated transient-type motions. These were transmitted to structural foundations through the overburden soil. Hence the propagation and attenuation of the vibrations depended on the soil characteristics among other factors.

Geotechnical investigations revealed that the soils underlying the site consisted of very clayey, medium dense, lateritic sand to a depth of about 7m followed by a thick accumulation of medium-coarse sand with occasional gravels and traces of silt. The subsurface was fairly homogenous throughout the project area. These soil characteristics were corroborated by the observed nature of the vibration propagation - typified by a fairly high attenuation coefficient as reflected in the rapid decrease of velocity amplitude with distance from the source.

Analyses of the vibration data revealed that different velocity components could not be pooled even along the same monitoring axis for attenuation assessment. However, the site homogeneity permitted the pooling of similar components from different axes. The analysis results also showed that the zone of highest damage probability did not extend across the property line. This deduction was based on a comparison between the measured velocity amplitudes and the internationally accepted damage criteria for vibrations. These criteria show that the dividing line between the zones of structural damage and safety coincides with constant velocity amplitude of 50.8 mm/sec. The bulk of the velocity measurements obtained fell below 3.0 mm/sec.

Human response to vibrations is generally subjective and depends on such factors as personal bias and sound effects. Usually, where these two are present, velocity amplitude as low as 0.3 mm/sec is enough to make the vibration noticeable. Also, the frequency of the vibration affects the observed response to it. For example, the velocity amplitude required for a particular human response to a given vibration decreases with increase in frequency.

Conclusion

Based on the result of the vibration data analyses and the foregoing discussion, the following conclusion could be drawn:

(i) Damage to residential structures from ground-borne vibrations correlates more closely with particle velocity than with any other parameter.

(ii) All the peak velocities measured or calculated in the project area were within the safe zone (less than 50.8 rpm/sec.). Therefore structural damage was very unlikely.

(iii) The characteristics of the soils at the site are typically associated with fairly high attenuation coefficient. Hence velocity amplitudes decreased rapidly with increasing distance from the source. Therefore the effect of the vibrations could not be felt over a very large area (vertical component of velocity was not noticeable at about 500m from the source during piling).

(iv) An appreciable percentage of the velocity amplitudes from the piling fell above the limit for "Noticeable vibrations by human beings". This was however predominantly confined within the zone of "possible complaints / noticeable vibrations".

(v) The vibration levels experienced throughout the monitoring operation were within the limits of human tolerance specified by internationally accepted standards (if viewed objectively and without bias).

(vi) Owing to sound effects, and particularly bias which is likely to be a major factor, there is a fairly high probability of complaints from owners of buildings and other structures sited less than 200m from the piling point.

Recommendations

(a) Damage criteria are probabilistic specifications. Therefore all site-specific variables must always be incorporated in the analyses to establish them.

(b) For every job likely to trigger off ground vibrations, adequate soil exploration and vibration monitoring should be conducted to establish the probable level of response from both structures and humans.

(c) Extrapolation of vibration response data from one area to another should be discouraged, except where adequate correlation has been established among the controlling factors based on thorough subsurface material characterization and dynamic load response analysis for the soils.

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