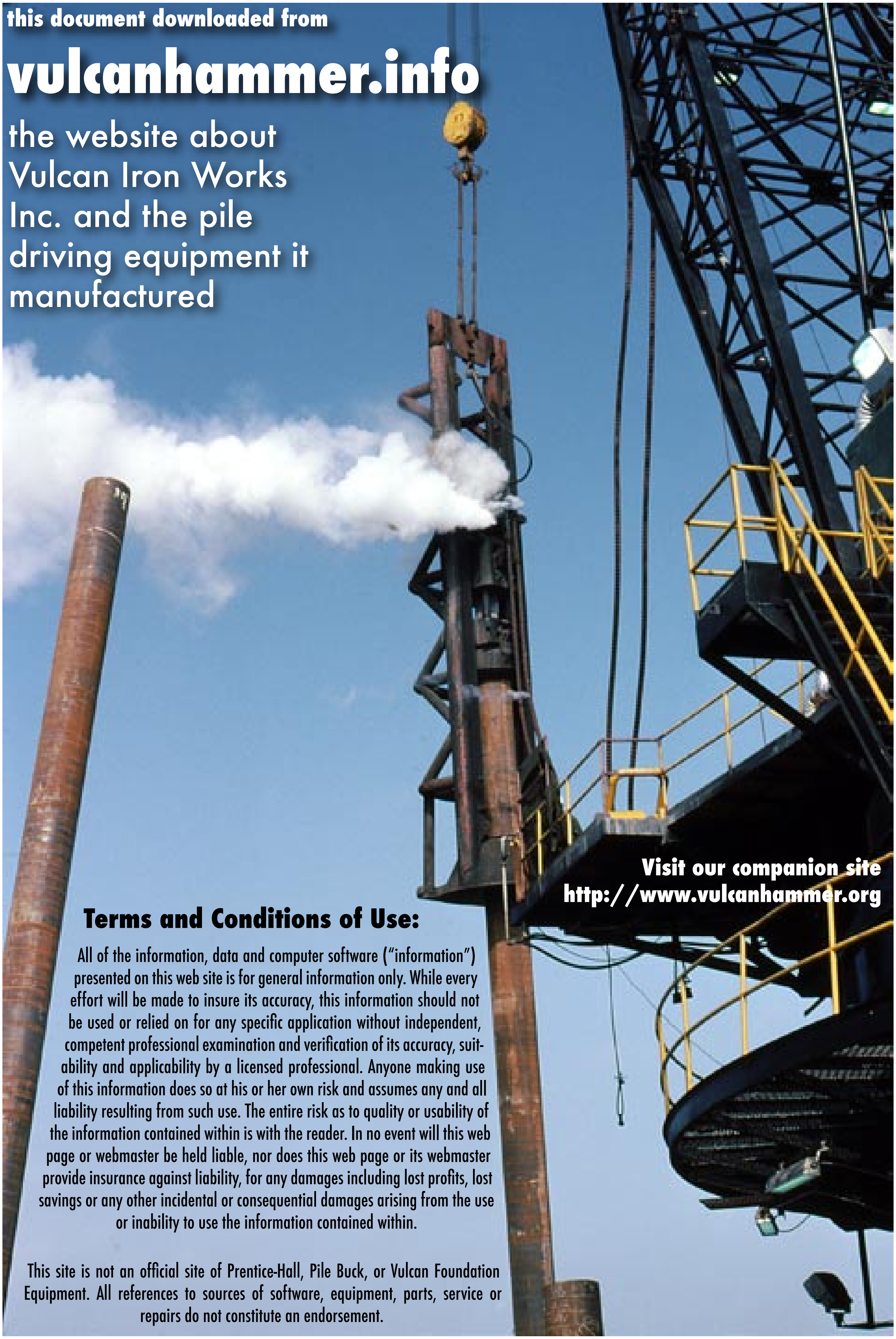


12) in the elastic zone
(17) in the plastic zone
ure at the well base of well type - The
is balanced by receiv-
reaction, Vb, and
the well base from
14. Therefore, the
ire and bearing soil

(21)

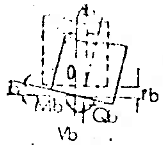
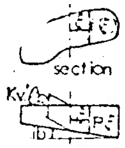


Fig.14 Behavior of well base

when the external
ise of the structure
has occurred

$) \cdot \xi d\xi = e \cdot Vb \quad \}$ (22)



Distribution of reaction

Fig.15 Distribution of vertical reaction at well base

point which is ξ $e^{-\lambda y}$
re
ture at the wel
pe type sheet
well base
n in Eq.

$\cdot C] \cdot i_b, \quad Vb = [k_p] \cdot \ell_b$ (23)

ght angles to leg axis that generates
nt in the direction at right angle to

t right angles to leg axis that generates
leg top or moment that generates at the leg
direction at right angles to leg axis
it rotation angle at the leg top
the soil is uniform and the leg length

$\cdot \beta_i^3, \ [C] = \Sigma C_i = \Sigma 2EI \cdot \beta_i$
each leg

rce, Hi, and bending moment, Mi, of each

(24)

olds when the external forces of V_o M_o and

L_o (25)

solve Eqs.(8) - (12) or Eqs.(13) - (17) by
ntinuous condition, boundary condition and

5TH SOUTHEAST ASIAN CONFERENCE ON SOIL ENGINEERING
BANGKOK, THAILAND
2 – 4 July 1977

MEASUREMENT AND PREDICTION OF VIBRATIONS GENERATED BY DROP HAMMER PILING IN BANGKOK SUBSOILS

R. PETER BRENNER,
Asian Institute of Technology,
Bangkok, Thailand

SUVIT VIRANUVUT,
Asian Institute of Technology,
Bangkok, Thailand

SUMMARY A large number of vibration measurements on the ground surface and on an adjacent building were performed in connection with pile driving activities on a site north of Bangkok. Vibration intensity was expressed in terms of peak particle velocity. A statistical comparison with previously collected data from other sites in the Bangkok area revealed that vibrations generated by driving a pile into one of the bearing strata commonly used for founding piles in this region, i.e. stiff clay or the underlying sand, are not of significantly different magnitude. A previously recommended upper bound for vibrations to be expected could be confirmed. A multiple correlation with penetration data obtained from Dutch cone tests at the site and pile driving records was also attempted but the only variable giving a significant contribution was the cone resistance. The correlation, however, was rather weak.

INTRODUCTION

Vibrations generated by pile driving may be harmful to nearby structures or at least irritating to people living in these buildings, particularly in combination with the associated noise. The effects of vibrations on structures and people have been reviewed in detail by BRENNER & CHITTIKULADILOK (1975). While today modern piling techniques have greatly reduced vibration levels, pile driving by drop hammer is still a widely used practice, mainly in developing regions.

Although numerous measurements on vibrations caused by pile driving have certainly been carried out in the past, only few case histories have been published. D'APPOLONIA (1971) has summarized some of the available information and additional data has been provided by ATTEWELL & FARMER (1973). In Fig. 1 and Table I an attempt has been made to collect all pertinent information available to the authors. Vibrations are usually plotted as peak particle velocity, v, versus the scaled driving energy, i.e. the square root of

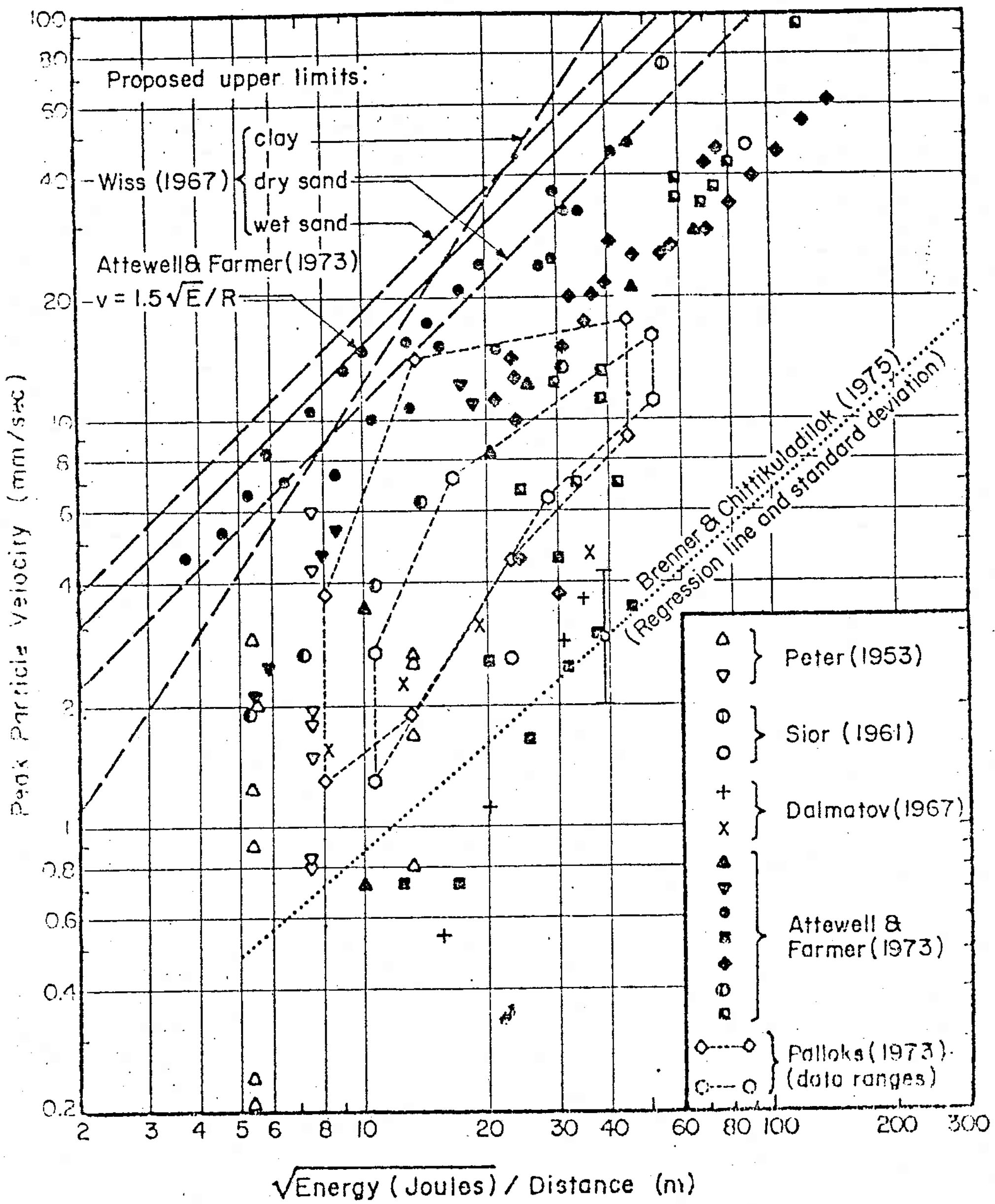
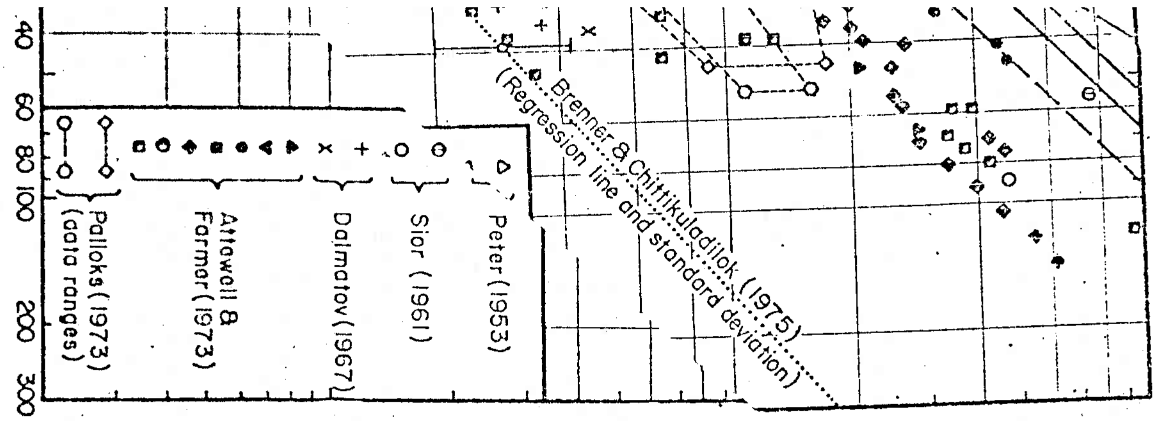


Fig. 1 - Available information on ground motions expressed as peak particle velocities caused by pile driving (see also Table I)

TABLE I

SUMMARY OF AVAILABLE INFORMATION ON VIBRATION MEASUREMENTS IN CONNECTION WITH PILE DRIVING

Symbol (refer to Fig. 1)	Author (Year)	Soil Type	Hammer type (Weight, kN)	Pile type and dimensions (m)	Mass hammer / Mass pile	Measurement range (m)	Driving energy (Joule)	Vibration component recorded	Vibration measuring device and natural frequency (Hz)
△	Peter (1953)	1.8m humus 1.1m clayey sand followed by sandy loam	Single acting Express steam (25500)	cast-in-place concrete ϕ 0.55x3.00	1.47	20	21680	vertical (V) horizontal (H_1, H_2) $H_1 \perp H_2$	3-component displacement seismograph (3.7, 3.6, 2.9)
▽			Demag quick stroke steam (29040)		1.67	20	11615		
△		3.5m fill	Single acting Express steam	[illegible]	0.55	20	21680		electrodynamic vibration pickup



...round motion ...pressed
...s caused by pile driving

TABLE I

SUMMARY OF AVAILABLE INFORMATION ON VIBRATION MEASUREMENTS IN CONNECTION WITH PILE DRIVING

Symbol refer to Fig. 1)	Author (Year)	Soil type	Hammer type (Weight, kN)	Pile type and dimensions (m)	Mass hammer / Mass pile	Measurement range (m)	Driving energy (Joule)	Vibration component recorded	Vibration measuring device and natural frequency (Hz)
△	Peter (1953)	1.8m humus 1.1m clayey sand followed by sandy loam	Single acting Express steam (25500)	cast-in-place concrete Ø 0.55x3.00	1.47	20	21680	vertical (V) horizontal (H_1, H_2), $H_1 \perp H_2$	3-component displacement seismograph (3.7, 3.6, 2.9)
▽			Demag quick stroke steam (29040)		1.67	20	11615		
△		3.5m fill 3.2m peat then gravel with silty sand	Single acting Express steam (25500)	cast-in-place concrete Ø 0.55x8.00	0.55	20	21680	V, H_1, H_2	electrodynamic vibration pickup GM 5520 (Philips)
▽			Demag quick stroke steam (29040)		0.62	20	11615		
▽		gravel	Demag quick stroke steam (29040)	cast-in-place concrete Ø 0.55x10.00	0.50	8.2	11615	V, H_1, H_2	3-component displacement seismograph (5.6, 7.7, 7.7)
⦶	Sior (1961)	cohesive	Steam	sheet-pile		4.0	49050	V	
○		cohesive	Steam	cast-in-place length: 3.2m		4.0 ; 15.0	117720	V	
+	Dalmatov (1967)	sat. sand	drop hammer	sheet-pile length: 8.0m		2.8 to 6.2	[illegible]10	V, H_1, H_2	3-component displacement seismograph at 2.4m depth
×		0-2m: building waste >2m: sat. loam	drop hammer	hollow reinforced concrete Ø 0.60x24.0		4.8 to 20.6	0-5m: 29430 >5m: 58860		3-component displacement seismograph at 3.25m depth
▲	Attewell & Farmer (1973)	fine sand	wet vibroflotation			1 to 4.5	1350/cycle	V	Cambridge vibrograph
▼		uncompacted fill	dry vibroflotation			1 to 6.0	2700/cycle	V	
●		stiff silty clay on firm laminated clay		sheet piles		5 to 40	2400 to 60700	V	Velocity pickup
■		coarse sand on stiff clay base	Shell boring			1.5 to 3.0	1050 to 10500	V	Cambridge vibrograph
◆		layered medium sand and silt		H-piles		1.5 to 7.5	9100 to 36500	V	
☾		laminated clay	Diesel hammer			5 to 40	20000	V	Velocity pickup
◘		layered coarse sand and gravel		concrete mandrel		3 to 15	127000 to 212000	V	Cambridge vibrograph
◇	Palloks (1973)	medium sand, dense	Diesel Delmag D12 (12263) (Piston)	prestressed concrete 0.34x0.34x15.0	0.28	4 to 22	30656	V	electrodynamic vibration pickup or piezoelectric accelerometer
○			Diesel Delmag D22 (21582) (Piston)		0.50	5 to 22	53955	V	
....	Brenner & Chittikuladilok (1975)	clayey sand	drop hammer (68670)	prestressed concrete 0.43x0.43x25.0	0.62	1 to 20	68670	vertical (V) radial (R) tangential (T)	velocity pickup (14)

the energy, E, delivered by the hammer (weight of hammer times drop height) divided by the horizontal distance, R, from the pile axis to the point of interest. (Note that WISS (1967) used the distance from the pile tip to the location of interest on the ground surface). All data points shown in Fig. 1 represent the vertical component of the particle velocity, except those of PETER (1953) and DALMATOV et al (1967) which give the resultant of the vertical and two horizontal components. It can be seen that the upper limit proposed by ATTEWELL & FARMER (1973) successfully bounds all data plotted in Fig. 1.

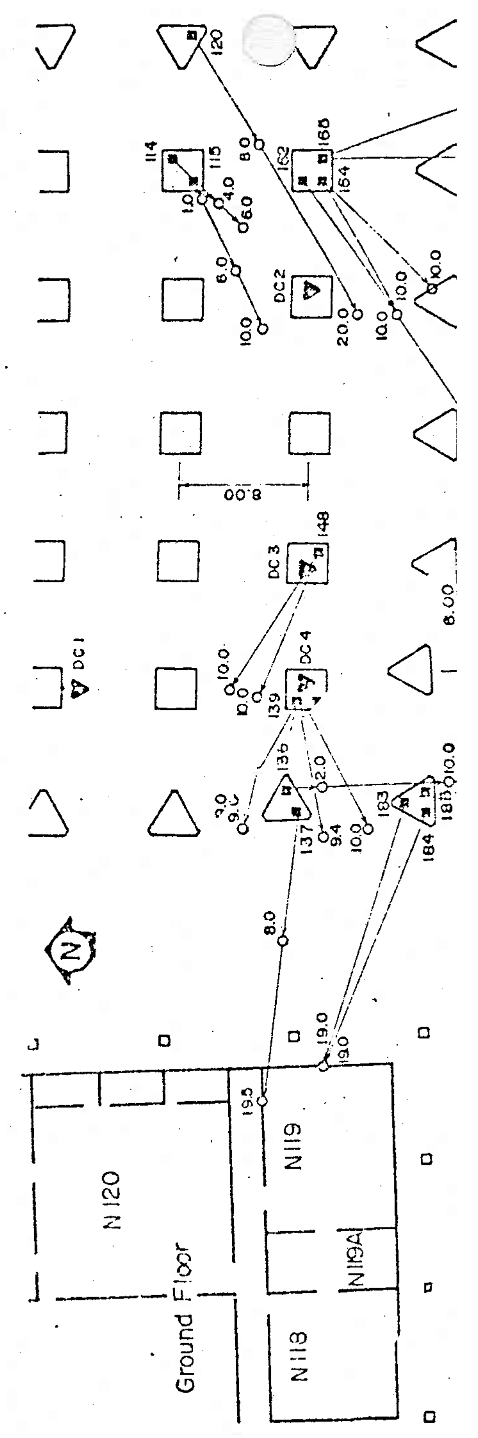
In the Bangkok area, a fair amount of data has been collected by BRENNER & CHITTIKULADILOK (1975). Due to the soil conditions in this area, the upper limit of expected peak particle velocities is considerably lower. The subsurface conditions around Bangkok consist of a thick deposit of recent marine clay. This deposit, generally designated as Bangkok Clay, can be divided into an upper section of about 9 to 15 m of soft clay and a lower section of stiff clay followed by sand. The groundwater table is usually about 1 m below the ground surface. Details on the geotechnical properties of Bangkok clay may be found, for example, in MOH et al (1969). Due to these soil conditions most structures in the Bangkok area are founded on piles and driving these piles by drop hammer is commonly employed.

BRENNER & CHITTIKULADILOK (1975) measured vibrations from pile driving on two sites, one of them south and the other one north of the City of Bangkok. In addition, a tentative prediction of vibration levels based on limited data from Dutch cone soundings was also presented. In the present study, measurements were carried out on the campus of the Asian Institute of Technology (AIT) in Klong Luang District (approximately 40 km north of Bangkok) during the installation of piles for the foundation of a Regional Computer Center (RCC). In addition to recording vibrations on the ground and on an adjacent building, several Dutch cone soundings were performed and it was tried to correlate measured vibrations with a number of variables which were thought to influence vibration levels. A statistical comparison with previous measurements from other sites was also attempted.

EXPERIMENTAL INVESTIGATIONS

Site Description

A portion of the foundation plan of the building to be constructed and its relative position to the adjacent existing building can be seen in Fig. 2. Three and four piles were driven for the triangular and square pile caps, respectively. The precast concrete piles were 0.35 m square and 18 m long. They were driven by a drop hammer of mass 4 Mg and with drop heights of 0.30, 0.40, 0.60 and 0.90 m, depending on the resistance to penetration. A typical soil profile at this site can be seen in Fig. 3a. The boundary between the stiff clay and the sand is not sharp, however, with pockets of sand occurring in the clay and vice versa. The stiff clay contains a high percentage of sand.



er times drop height)
xis to the point of
om the pile tip to the
points shown in Fig. 1
ity, except those of
resultant of the ver-
hat the upper limit
ds all data plotted in

un collected by BRENNER
in this area, the upper
ably lower. The sub-
eposit of recent marine
lay, can be divided
and a lower section of
usually about 1 m
properties of Bangkok
Due to these soil con-
d on piles and driving

ions from pile driving
h of the City of
ition levels based on
ited. In the present
the Asian Institute of
y 40 km north of
idation of a Regional
itions on the ground and
were performed and it
ber of variables which
tical comparison with
pted.

to be constructed and
ng can be seen in Fig. 2.
nd square pile caps,
square and 18 m long.
th drop heights of 0.30,
penetration. A typical
e boundary between the
ockets of sand occurring
a high percentage of

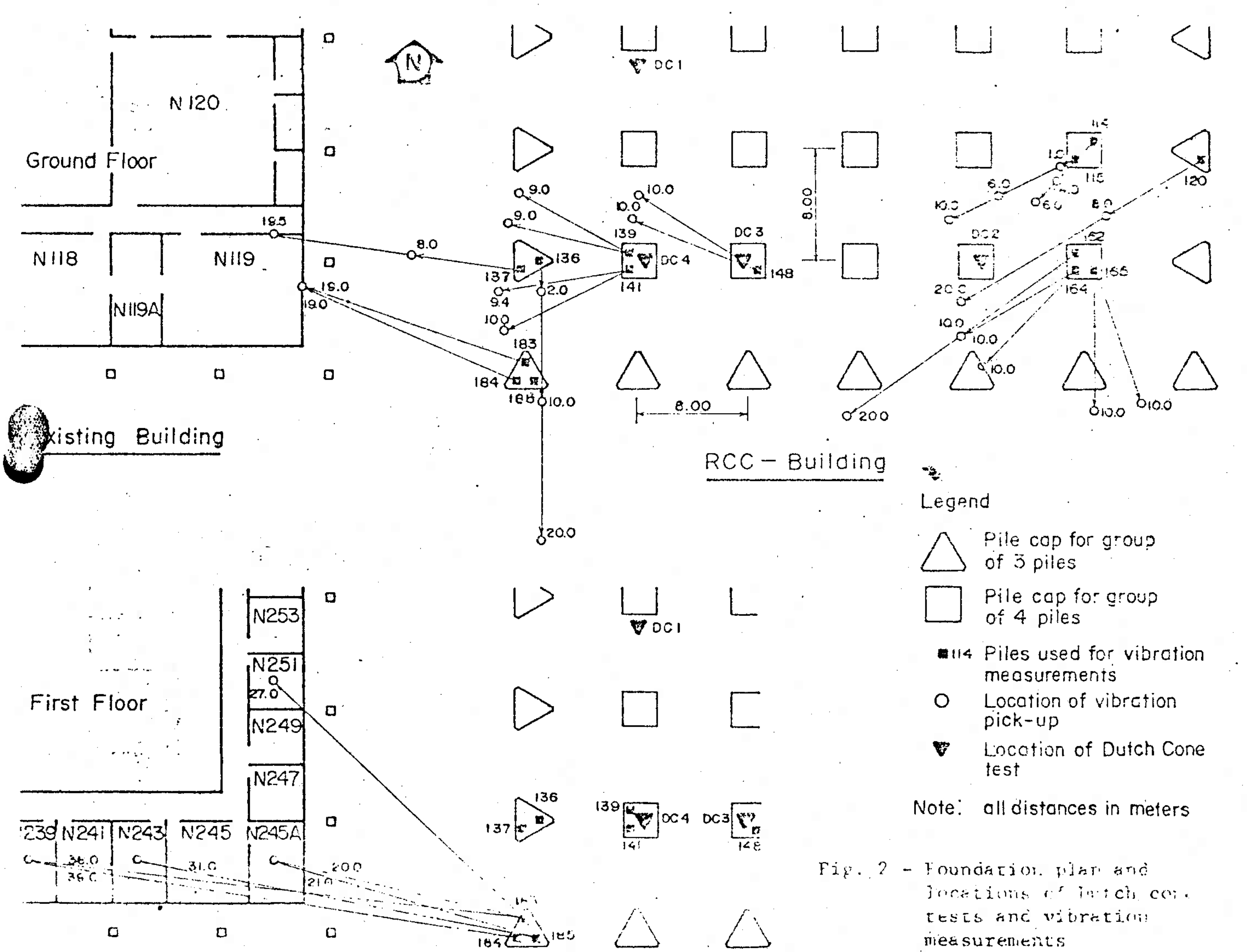


Fig. 2 – Foundation plan and locations of Dutch cone tests and vibration measurements

Measurements

A total of four Dutch cone tests were performed at selected locations where piles would be driven (see locations marked by DC in Fig. 2). The Dutch cone apparatus had a 2.5 t capacity and was of the friction jacket type. During pile driving numerous vibration measurements were performed for piles being installed in the vicinity of the Dutch cone soundings. These measurements were made on the ground for various depths of pile embedment at distances ranging from one to 20 m from the pile. Only the vertical component of vibration was recorded, since previous investigations have shown that the horizontal components in the radial and tangential direction were usually small, i.e. about 30%, and never exceeded 80% of the vertical component. Thus, the resultant velocity, v_{tot}, which is the vectorial sum of the three components, is not larger than 1.5 times the vertical component. On the adjacent building which was a two-story reinforced concrete structure, also supported on piles, vibration records were taken on floors and on a wall. In all cases vibrations were sensed by means of velocity pickups and recorded on an oscilloscope with a Polaroid camera.

During driving, a record of blow counts per 0.30 m (1 foot) of pile penetration was also kept. The rather uncontrollable way in which cushioning material was applied between pile and helmet and between helmet and hammer resulted in a variation of the energy transmitted to the pile at constant height of drop. This variation could not be quantified but will constitute a portion of the variability observed in the vibration records.

RESULTS AND DISCUSSION

Dutch Cone Soundings and Driving Records

Values of cone resistance and local friction obtained from Dutch cone soundings are shown in Figs. 3b and 3c. Sounding DC1 could only be carried out to a depth of 14.2 m which is less than the final pile tip elevation, probably because of the presence of a pocket of dense sand. Also within the depth range of about 12 to 14 m, the values scatter considerably. In this depth range, sounding DC4 probably penetrated stiff clay while others passed through sand. In Fig. 4 blow counts for each 0.30 m of pile penetration have been plotted for the ten piles which were used in the vibration measurements. Again data scatter increases notably once the pile tip has to penetrate the sand stratum.

Ground Vibrations

Measured values of vertical ground motion in terms of peak particle velocity (i.e. one-half the max. peak-to-peak velocity evaluated from a record) are shown in Figs. 5 and 6. In Fig. 5 vibration has been plotted versus depth of pile embedment for various distances from the pile. A great number of measurements were made at a distance of 10 m for purpose of correlation with cone resistance. At any distance the velocities appear to vary in a similar pattern as the cone resistances, namely an increase within the zone ranging from about 12 to 15 m and [illegible]equent decrease at greater depths.

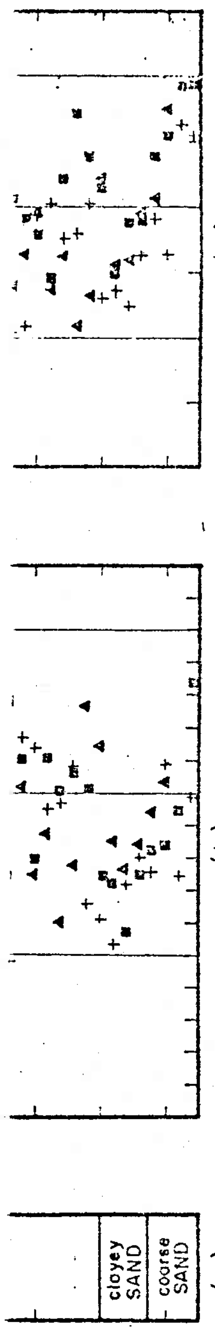


Fig. 3 - Results of four Dutch cone tests: (a) soil profile, (b) cone resistance, (c) local friction

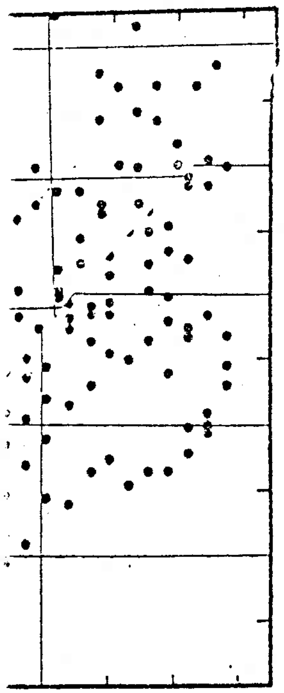


Fig. 4 - Blow counts for piles no. 114, 115, 120, 136, 139, 141, 148, 162, 164, 165

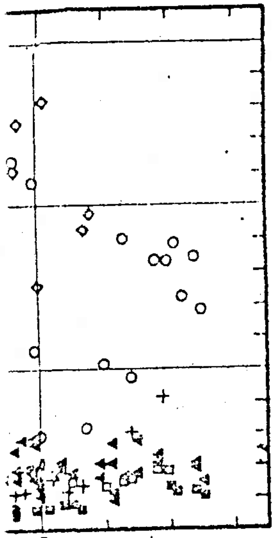


Fig. 5 - Peak particle velocities (vertical component) measured on ground surface versus depth of pile embedment for various distances from pile

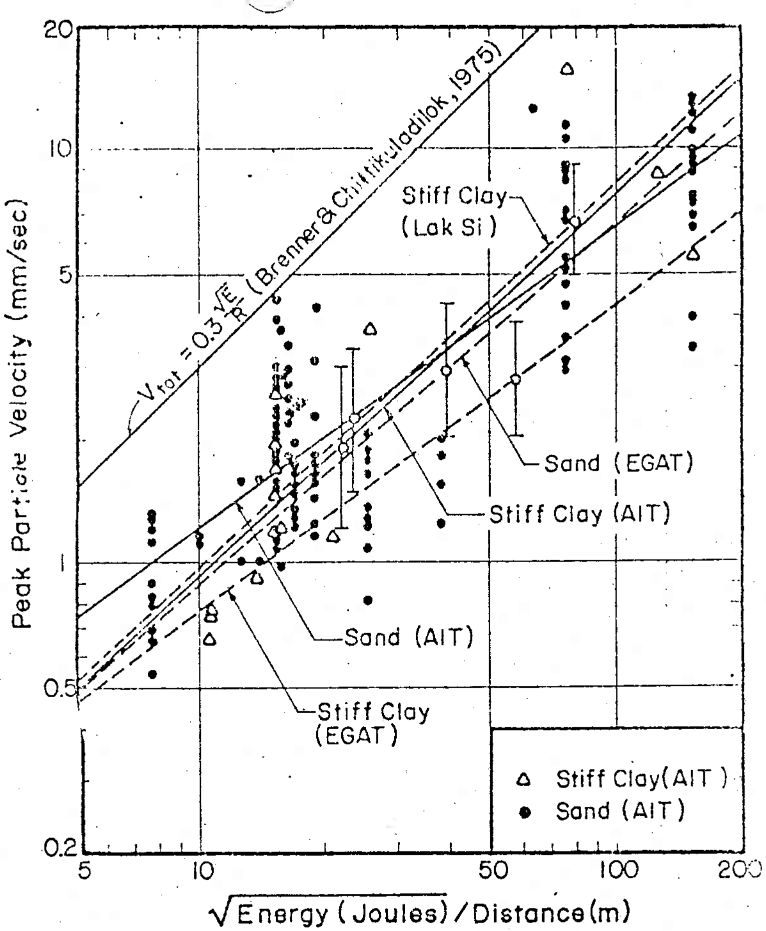


Fig. 6 - Attenuation of vertical ground motion at surface and comparison with regression lines from other sites

Fig. 6 shows a plot of vertical ground motion versus scaled energy measured when the pile tip passed through the stiff clay and through the sand. Regression lines and corresponding standard errors of estimate (indicated by a vertical line through the mean value of each regression line) for the two sets of data are also given (full lines). In addition, regression lines from an earlier investigation (BRENNER & CHITTIKULADILOK, 1975) have been incorporated in this figure (dotted lines) for purpose of comparison. These data were obtained from two sites, one called 'EGAT', at Phrapradaeng, approx. 20 km southeast of Bangkok and the other called 'Lak Si', about 16 km north of Bangkok. It can be seen that all these regression lines fall within a rather narrow band when compared with their standard errors of estimate. A previously proposed upper limit of total vibration in the Bang area (based on data from sand at EGAT) is also given in this figure, namely

$$v_{tot} = 0.3\,\frac{\sqrt{E}}{R} \qquad (1)$$

It appears that the type of layer penetrated by the pile, the location of the site and also the size of the pile have little influence on the magnitude of vibration on the ground surface. All data collected have therefore been lumped together and an overall regression was determined. This regression line and its 95% confidence belt for a single observation of vertical vibration are given in Fig. 7. Even if the upper confidence limit is multiplied

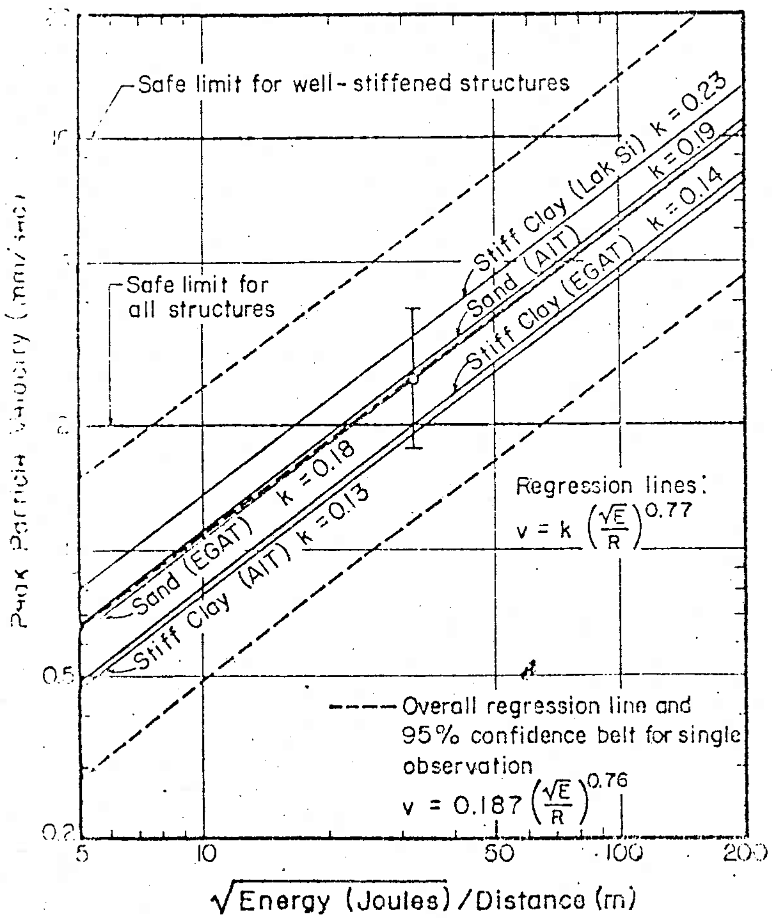


Fig. 7 - Overall regression for data from three sites and result of multiple regression taking into account the different origin of the data

by 1.5 to appr mate total v
$v_{tot} = 0.3\sqrt{E}/$ or values of
posed upper limit for v_{tot} rer
spectrum of different strata a

A more refined analysis,
my variables (see for example
different origin of the data.
layer or location. The regres
then the form:

$$Y = \beta_o +$$

where Y and X_1 are the logarit
are dummy variables used as fo

X_2	X_3
1	0
0	1
0	0
0	0
1	1

This results in a family of pa
tion in intercept from the one
the difference in stratum chara
the origin of the data is, howe
seen from Fig. 7 that the two
lines for sand almost coincide.
to behave differently. This da
pletely representative as it wa
tances from the pile. In Fig.
data block lies to the far righ
($\sqrt{E}/R$)-range considered. It sh
top of the stiff clay and of th
the AIT-site, but this increase
creased stiffness or density of
vibration levels of similar mag

Also shown in Fig. 7 are t
German Standard DIN 4150 (1971)
type indicated should occur.

Building Vibrations

Particle velocities measure
ing are shown in Fig. 8. All va
lowest threshold of damage given
applies to old structures in a p
could be clearly felt by people

by 1.5 to approximate total vibration, it still falls below the line $v_{tot} = 0.3\sqrt{E}/R$ for values of $\sqrt{E}/R$ greater than 25. Thus the previously proposed upper limit for v_{tot} remains a reasonable bound even with this enlarged spectrum of different strata and locations.

A more refined analysis, however, can be accomplished by the use of dummy variables (see for example DRAPER & SMITH, 1966) which can account for the different origin of the data. In this case "origin" would mean either soil layer or location. The regression model for the prediction of vibration takes then the form:

$$Y = \beta_o + \beta_1 X_1 + \beta_2 X_2 + \beta_3 X_3 + \beta_4 X_4 \qquad (2)$$

where Y and X_1 are the logarithms of v and $\sqrt{E}/R$, respectively, and X_2, X_3, X_4 are dummy variables used as follows:

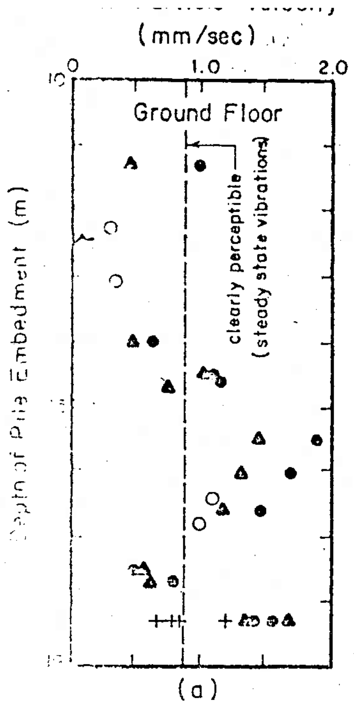
X_2	X_3	X_4	data block
1	0	0	for stiff clay at EGAT
0	1	0	for sand at EGAT
0	0	1	for stiff clay at Lak Si
0	0	0	for sand at AIT
1	1	0	for stiff clay at AIT

This results in a family of parallel lines shown in Fig. 7, with the deviation in intercept from the one for the AIT sand stratum giving an estimate of the difference in stratum characteristic and location. This distinction of the origin of the data is, however, statistically not significant. It can be seen from Fig. 7 that the two lines for stiff clay at EGAT and AIT and the two lines for sand almost coincide. Only the line for stiff clay at Lak Si appears to behave differently. This data block, however, may not be considered completely representative as it was obtained over a rather narrow range of distances from the pile. In Fig. 6, it can be observed that the mean of this data block lies to the far right, rather than in the central region of the ($\sqrt{E}/R$)-range considered. It should be kept in mind that at the EGAT-site the top of the stiff clay and of the sand stratum was about 6 to 9m deeper than at the AIT-site, but this increased depth appears to be compensated by the increased stiffness or density of the clay or sand, respectively, thus producing vibration levels of similar magnitude at the ground surface.

Also shown in Fig. 7 are two limiting levels of v_{tot}, recommended by the German Standard DIN 4150 (1971), below which no damage to structures of the type indicated should occur.

Building Vibrations

Particle velocities measured at several locations in the adjacent building are shown in Fig. 8. All values were less than 2 mm/sec, which is the lowest threshold of damage given by any of the damage criteria and which applies to old structures in a poor state of repair. Vibrations, however, could be clearly felt by peopl[illegible]ng in the building and could be classified



Symbol	Pile No.	Location of Measurement	Distance (m)
○	137	Corridor (Wall)	19.5
●	184	N 119 (Floor)	19
▲	184	N 119 (Wall)	19
+	183	N 119 (Floor)	19
◇	183	N 245A (Floor)	20
▽	183	N 239 (Floor)	38
×	183	N 251 (Floor)	27
◆	185	N 245A (Floor)	21
▼	185	N 239 (Floor)	39
■	185	N 243 (Floor)	31

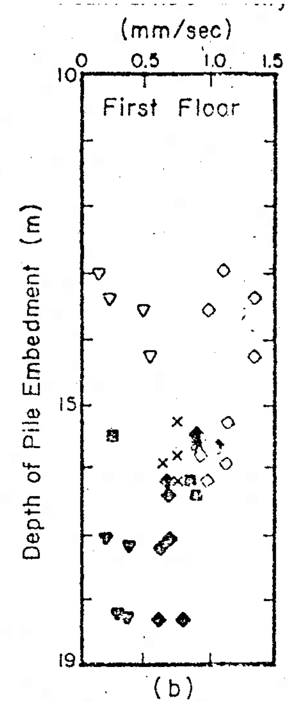


Fig. 8 - Vertical vibration observed on floors and wall of adjacent building as a function of pile embedment

as "clearly perceptible". In Fig. 8a this limit is indicated for steady-state vibrations. For transient vibrations, such as pile driving, this limit is usually higher, i.e., beyond 2 mm/sec (see WISS & PARMELEE, 1974). In this case, the effect of the vibrations on people was of minor importance compared to the high noise level produced by the impact of the pile hammer.

It was also observed that the vibrations, as weak as they were, could impair the performance of sensitive tests. Thus for stress-controlled triaxial tests carried out in room N119, the deformation dial gages responded to each hammer blow. Also during a pile load test carried out some 150 m away from the pile driving site, the deflection dial gages vibrated slightly in response to each blow, while vibration was imperceptible to people. Thus whenever piles are driven in the Bangkok area closer than about 25 m to a building, equipment sensitive to floor vibration cannot be operated.

Vibration Prediction

It was tried to correlate measured vertical particle velocities on the ground at 10 m distance from a pile being driven with several quantities which were thought to influence vibration. Thus for various depths and for

been carried out, veloc
frictio..., cumulative
pile pe...tration E_p (a
the penetration (in m)

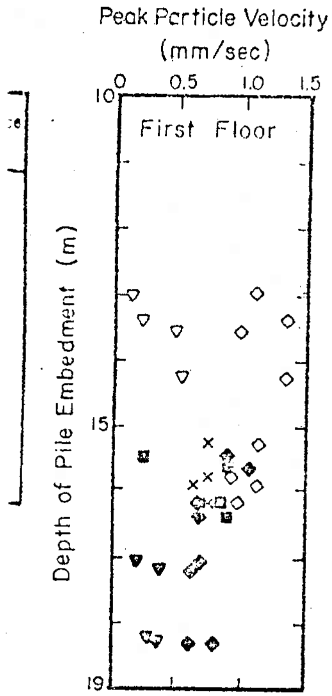
v = β

Both arithmetic and log
a stepwise regression t
tion. In such a procec
sponse enters first int
whose partial correlati
regression each variabl
previously incorporated
respect to their contri
the model. A detailed
in DRAPER & SMITH (1966

The stepwise regre
in a linear model was t
city at 10 m from the p
double logarithmic scal
gression is significant
slope of the regression
tative prediction from
is also shown in Fig.
appears to be too close
was about three times a
stiff clay and sand str
the AIT-site and for th
quired to generate the
stratum must affect the
needed, however, to per
tum position.

Based on the obser
with earlier measuremen
may be drawn:

(1) For practical use,
by driving a pile
any site in the Ba
total vibration, i
the pile tip is pe
appear rather low
various soil condi



(b)

d wall of adjacent

icated for steady-
le driving, this limit
RMELEE, 1974). In this
or importance compared
ile hammer.

s they were, could
ress-controlled tria-
l gages responded to
out some 150 m away
brated slightly in
e to people. Thus
n about 25 m to a
be operated.

e velocities on the
everal quantities
ious depths and for

piles which were at or very close to locations where Dutch cone soundings had been carried out, velocity was correlated with cone resistance, q_c, local friction q_f, cumulative or total friction q_{tf}, and the energy per meter of pile penetration E_p (a quantity obtained by dividing the driving energy by the penetration (in m) per blow), i.e.

$$v = \beta_0 + \beta_1 q_c + \beta_2 q_f + \beta_3 q_{tf} + \beta_4 E_p \qquad (3)$$

Both arithmetic and logarithmic scales were considered for the variables and a stepwise regression technique employed to find the "best" regression equation. In such a procedure the variable most highly correlated with the response enters first into the regression equation followed by the variables whose partial correlation with the response are highest. At every step of regression each variable entering the regression equation and all variables previously incorporated into the model are checked by partial F-tests with respect to their contribution, regardless of their actual point of entry into the model. A detailed description of this method may be found, for example, in DRAPER & SMITH (1966).

The stepwise regression revealed that the only variable to be retained in a linear model was the cone resistance, q_c. In Fig. 9, the particle velocity at 10 m from the pile has been plotted versus the cone resistance in a double logarithmic scale together with the regression line. Although the regression is significant, the coefficient of correlation is only 0.616. The slope of the regression line is almost identical with a previously given tentative prediction from the EGAT-site (BRENNER & CHITTIKULADILOK, 1975) which is also shown in Fig. 9. The relative position of the two lines, however, appears to be too close considering the fact that the energy at the EGAT-site was about three times as high as at the AIT-site. On the other hand, the stiff clay and sand stratum at the EGAT-site were considerably deeper than at the AIT-site and for the same penetration resistance more energy will be required to generate the same surface vibration. Thus the depth of the soil stratum must affect the intercept of the regression line. More data would be needed, however, to permit a quantitative assessment of the influence of stratum position.

CONCLUSIONS

Based on the observations carried out at the AIT-site and the comparison with earlier measurements in the Bangkok area, the following major conclusions may be drawn:

(1) For practical use, no distinction is necessary between vibrations caused by driving a pile through stiff clay or through the sand underneath at any site in the Bangkok area. A previously recommended upper bound for total vibration, i.e. $v_{tot} = 0.3\sqrt{E}/R$, may be considered as safe when the pile tip is penetrating these strata, although this upper bound may appear rather low when compared with other vibration data collected for various soil conditions and piling arrangements.

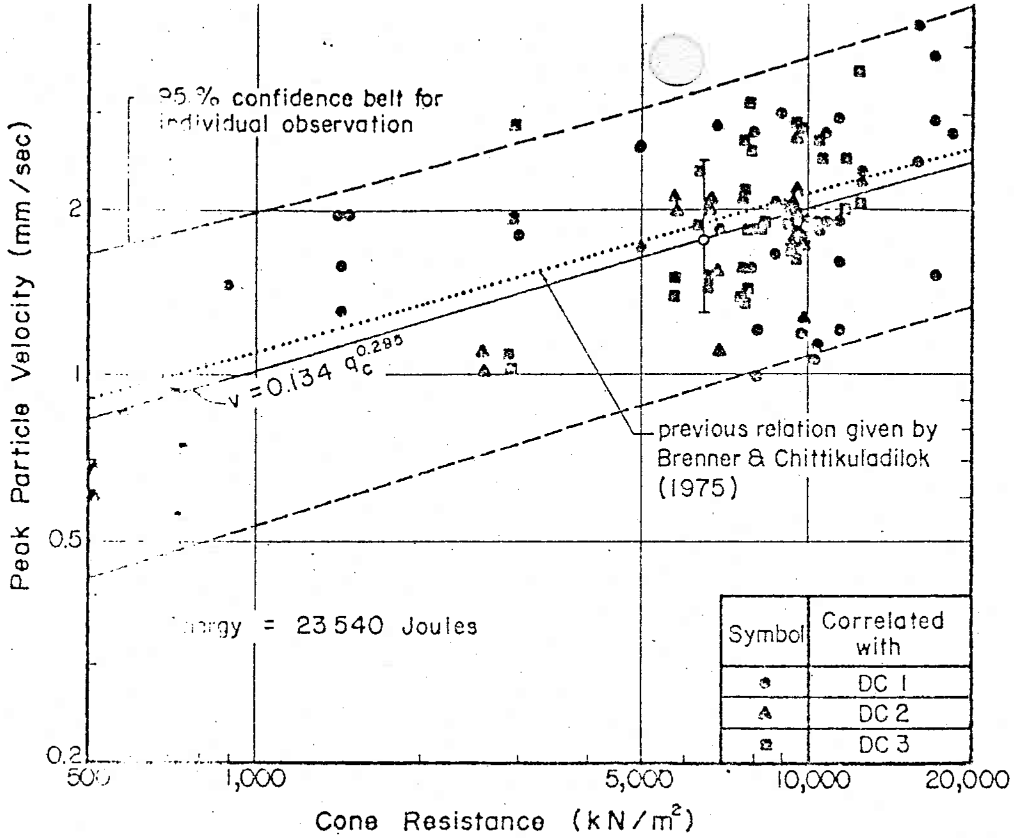


Fig. 2 - Relationship between peak particle velocity (vertical component) at 10 m distance from pile and cone resistance

(2) Driving concrete piles by drop hammer in Bangkok subsoil at distances of 20 m or more from existing buildings will not cause any kind of damage due to vibrations although these vibrations might be clearly perceptible to people. Vibration sensitive equipment, however, may usually not be operated during driving.

(3) Correlating surface ground motions with penetration data, such as Dutch cone soundings and pile driving records, is of limited success. A correlation between particle velocity and Dutch cone resistance can be established and enables the prediction at a site, however, the vibration intensity for layers of equal cone resistance depends also on the depth of the layer.

The author re indebted
tistical analys of the data

ATTEWELL, P.B. and FARMER, I.W
from Pile Driving", Groun

BRENNER, R.P. and CHITTIKULADI
Driving in the Bangkok Ar
pp. 167-197.

DALMATOV, B.I., ERSHOV, V.A. a
Foundation Settlement in
Symposium on Wave Propaga
Albuquerque, New Mexico,

D'APPOLONIA, D.J. (1971), "Ef
Structures", Proceedings,
Mechanics and Foundation

DIN (German Standards) 4150 (19

DRAPER, N. and SMITH, H. (1966
& Sons, Inc.

MOH, Z.C., NELSON, J.D. and BR
Behaviour of Bangkok Clay
Soil Mechanics and Founda
225-230.

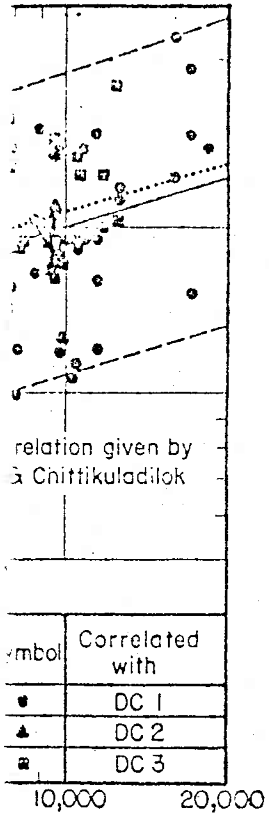
PALLOKS, W. (1973), "The Propa
on Structures", Proceedin
Foundation Engineering, M

PETER, U.G. (1953), "Measureme
Cast-in-Place Concrete Pi
ings, Third Int. Conferen
Zürich, Vol. 2, pp. 80-83.

SIOR, G. (1961), "Die Schadens
Bautechnik, Vol. 38, No.

WISS, J.F. (1967), "Damage Ef
Research Record, No. 155,

WISS, J.F. and PARMELEE, R.A.
tions", J. of the Structu



ity (vertical
cone

soil at distances of
any kind of damage
clearly perceptible
may usually not be

data, such as Dutch
ted success. A corre-
stance can be esta-
er, the vibration
ds also on the depth

ACKNOWLEDGEMENTS

The authors are indebted to Mr. G.S.N. Adikari who assisted in the statistical analysis of the data and made valuable suggestions.

REFERENCES

ATTEWELL, P.B. and FARMER, I.W. (1973), "Attenuation of Ground Vibrations from Pile Driving", Ground Engineering, Vol. 6, No. 4, pp. 26-29.

BRENNER, R.P. and CHITTIKULADILOK, Boonchai (1975), "Vibration from Pile Driving in the Bangkok Area", Geotechnical Engineering, Vol. 6, No. 2, pp. 167-197.

DALMATOV, B.I., ERSHOV, V.A. and KOVALEVSKY, E.D. (1967), "Some Case of Foundation Settlement in Driving Sheeting and Piles", Proceedings, Int. Symposium on Wave Propagation and Dynamic Properties of Earth Materials, Albuquerque, New Mexico, pp. 607-613.

D'APPOLONIA, D.J. (1971), "Effects of Foundation Construction on Nearby Structures", Proceedings, Fourth Pan-American Conference on Soil Mechanics and Foundation Engineering, Puerto Rico, Vol. 1, pp. 189-236.

DIN (German Standards) 4150 (1971), Erschütterungen im Bauwesen (draft).

DRAPER, N. and SMITH, H. (1966), Applied Regression Analysis, John Wiley & Sons, Inc.

MOH, Z.C., NELSON, J.D. and BRAND, E.W. (1969), "Strength and Deformation Behaviour of Bangkok Clay", Proceedings, Seventh Int. Conference on Soil Mechanics and Foundation Engineering, Mexico City, Vol. 1, pp. 225-230.

PALLOKS, W. (1973), "The Propagation of Ramming Vibrations and Their Effects on Structures", Proceedings, Eighth Int. Conference on Soil Mechanics and Foundation Engineering, Moscow, Vol. 4.3, pp. 454-455.

PETER, U.G. (1953), "Measurements of Vibrations Originating in Constructing Cast-in-Place Concrete Piles According to the Express System", Proceedings, Third Int. Conference on Soil Mechanics and Foundation Engineering, Zürich, Vol. 2, pp. 80-83.

SIOR, G. (1961), "Die Schadenswirkungen von Rammerschütterungen", Die Bautechnik, Vol. 38, No. 6, pp. 181-185.

WISS, J.F. (1967), "Damage Effects of Pile Driving Vibrations", Highway Research Record, No. 155, pp. 14-20.

WISS, J.F. and PARMELEE, R.A. (1974), "Human Perception of Transient Vibrations", J. of the Structural D[illegible], ASCE, Vol. 100, pp. 773-787.