A PROPOSED MODEL FOR GROUND VIBRATION INDUCED BY A STATNAMIC TEST

Chew Soon Hoe [i)] and Eng Zi Xun [ii)]

i) Assistant Professor, Department of Civil and Environmental Engineering, National University of Singapore, 1 Engineering Drive 2, #07-03, Block E1A, Singapore 117576

ii) General Manager, Geonamics (M) Sdn. Bhd., No 6, Lot 25, Jalan Udang Harimau 1, Medan Niaga Kepong, 51200, Kuala Lumpur, Malaysia.

ABSTRACT

Statnamic Load Test (STN) is an economical alternative to a Static Load Test (SLT) for the determination of bearing capacity of piles. The shock impulse from STN test of about 150-250ms will induce vibration in the ground (in terms of Peak Particle Velocity, PPV). This vibration will then be damped and attenuated radially with distance from the test pile location. Lack of a reliable method in assessing the risk of ground vibration over distance limits the application of STN, especially if there is sensitive structures nearby. Middendrop (2011) and Chew et al. (2012) showed that PPV is proportional to the test load based on the compilation of a few sets of actual measured data, with test load up to about 16 MN, in Europe and Malaysia/Singapore respectively. However, over the last 10 years, there are many more number of STN tests conducted in Malaysia and Singapore with ground vibration measured, and at much higher test load level. It was observed that the maximum vibration induced by a STN test at much higher test load did not increase proportionally, but is seems to be capped at a maximum threshold value. It is noted that the piles in Malaysia and Singapore usually are terminated at competence soil (SPT'N >100) or hard rock. The data also suggest that the pile penetration length seems to have significant influence on the ground vibration. Massarch & Fellenius (2008) proposed a model to predict ground vibration induced by a hard driving of pre-cast pile, taking into account the effect of pile length. This paper aims to present a modified model to predict the ground vibration induced by a STN test at a higher test load (till 40 MN), with pile length effect included. A comparison between predicted and field measured PPV from a number of STN project sites is presented in this paper. The comparison shows a very good agreement between the measured and predicted PPV.

Keywords: Statnamic load test, vibration monitoring, Peak Particle Velocity, ground vibration

1 INTRODUCTION

Statnamic Load Test (STN) loading mechanism is based on launching a reaction mass using fast-expanding and high-pressurized gas within a piston cylinder assembly as shown in Fig. 1. The reaction of this will generate a designed equivalent downwards force (impulse) acting onto the pile head. This shock impulse is having a duration of about 150-250 ms, and will induce a vibration (in terms of Peak Particle Velocity, PPV) in the adjacent soils. This vibration will be damped and attenuated radially with distance from the test pile location. This ground vibration may have some adverse effect onto the nearby buildings or installations, especially when the buildings contain some vibration sensitive equipment or facility.

The ability to predict this vibration with certain accuracy is important for the designer. While it was shown that STN is a good alternative to the conventional proof load test using static test method (Hölscher et al., 2012; Chew et al., 2019), lack of a reliable method in assessing the risk of ground vibration over distance limits the application of STN, especially if there are sensitive structures nearby. This paper seeks to provide a reliable method to predict the ground vibration with a reasonable accuracy.

1.1 Review on Compiled Measured Data on Ground Vibration in Statnamic Load Test (STN Test)

Middendrop (2011) and Chew et al. (2012) compiled actual measured data on ground vibration due to STN tests, in Europe and Malaysia/Singapore respectively. The test load reported is from <2 MN to up to 16 MN. It was shown that PPV is proportional to the test load

applied (as shown in Fig. 2). It was also highlighted that the vibration attenuation effect can be affected by soil profile adjacent to the test pile; however, no specific adjustment factor for different soil profile was proposed in these works.

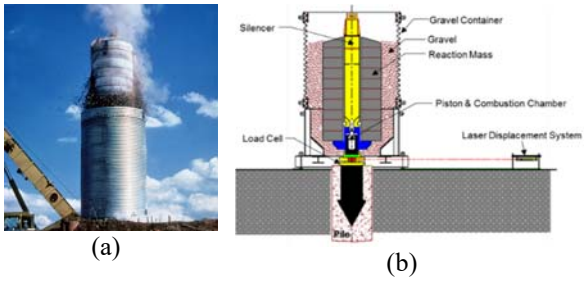

Fig. 1. Typical Statnamic Load Test: (a) Launching of reaction mass; (b) Schematic diagram with the key components shown.

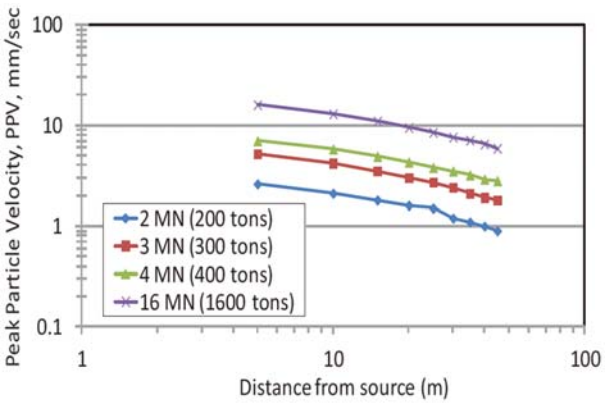

Fig. 2. Predictive curves of PPV vs Distance plot for Statnamic Tests at various Loads up to 16 MN by Chew et al. (2012)

Over the last 10 years, there were a lot more STN tests conducted in Malaysia/Singapore with much larger test load, up to 60 MN. Some of them are monitored with vibration measurement. The analysis of these data seems to indicate that the vibration did not increase with test load, but instead capped at certain value. Fig. 3 shows a compilation of PPV over horizontal distance measured by the authors for these projects. Fig. 3 also included the comparison with the trend lines proposed by Chew et al. (2012) and Middendrop (2011).

It is clear that while the vibration for those test load <10 MN seems to be slightly lower than those with test load >10 MN, the vibrations level at varies horizontal distance seems fall within a band irrespective of magnitude of test load, for the case when the test load is >10 MN. These test data showed that the vibration level at higher test load seems to be capped by a saturated value, with respect to the horizontal distance.

It should be noted that most, if not all, of the piles in Malaysia and Singapore are designed such that the pile toe is well socket into stiff soil (with SPT'N>100) or into competent rock layer. They tend to have a high skin friction component at the bottom part of the pile, and high end bearing component (Chew et. al., 2019). It was also observed that those piles socketed well into competent rock layer may induce different ground vibration from those piles that have no or very small length of rock penetration. This suggests that influence of the penetration depth of the pile into rock layer shall be considered in the prediction of ground vibration.

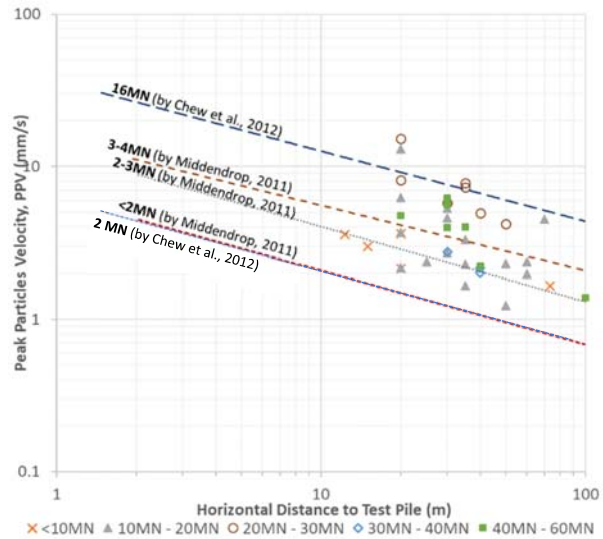

Fig. 3. Measured PPV in Statnamic Load Test with higher test load compared to the previous studies by Chew et al. (2012) and Middendrop (2011).

1.2 Prediction of Ground Vibration in Pile Driving using Energy Concept

Free field vibration due to pile driving action was calculated using wave energy concept by several researchers in the past. Attewell & Farmer (1973) and Head & Jardine (1992) discussed two sources of energy transfer for transmission of ground vibration from pile driving: i.e. via the pile toe and the pile shaft (as shown in Fig. 4). At the pile toe, the displacement of soil generates both compressional P-waves and shear S-waves that propagate outward from the pile toe in a spherical wave form in all directions. The reflection and refraction of these body wave forms at the ground surface gave rise to the surface waves (R-wave, and L-wave). For the pile shaft part, the downward motion of the pile shaft, interacting with the surrounding soil, causing polarized shear wave to propagate outwards in a near cylindrical wave front. However, due to the short distance between the vibration measurement points and the pile location, the various wave trains arrived at the measurement points in a superimposed group, rather than separate energy parcels. Hence, the combined resultant PPV will be a more practical means of evaluating the ground vibration from energy consideration.

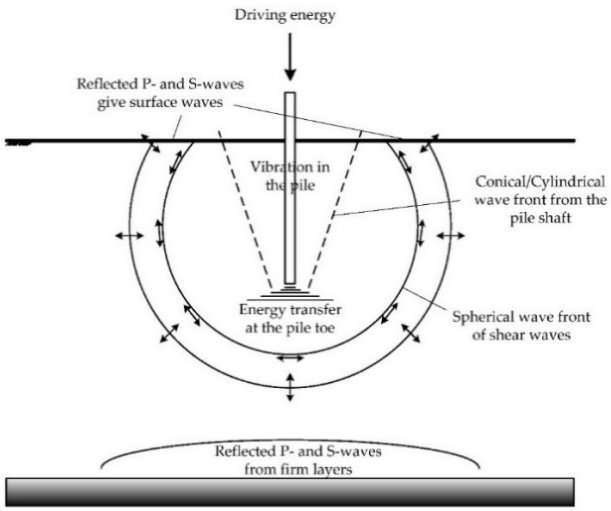

Fig. 4. A schematic diagram of different wave types that can be generated at pile driving (After Deckner, 2013).

Based on energy consideration, Attewell and Farmer (1973) developed a simple equation for the ground vibration attenuation with distance, as shown in Eq. (1):

$$v = k\,\frac{\sqrt{W}}{r} \tag{1}$$

where v is vibration velocity expressed in resultant PPV, r is the horizontal distance from the energy source (i.e. the pile), W is the energy per blow at source (J/blow), k is an empirical vibration factor (m/s√J) account for the pile-soil interaction. It was suggested that k=0.5 for soft soil, while k=1 for stiff soil or rock. Luk et al. (1990) commented that this empirical model gives a conservative upper bound in estimating the ground vibration. Massarch & Fellenius (2008) further highlighted that this empirical model did not adequately include the influence of the type and strength or stiffness of the surrounding soils. In practices, often the horizontal distance on the ground from pile to the measurement point is used to estimate the PPV. This is a "conservative" approach as it disregards the depth of pile penetration, which is the actual location of the major source of vibration.

Massarch & Fellenius (2008) proposed a revised model by taking the folliwng factors into consideration for prediction of ground vibration due to pile drivig: (i) influence of pile penetration depth, (ii) responses of dynamic pile-soil interaction, (iii) responses of dynamic resistance at pile shaft and pile toe, and (iv) energy propagation in elastic medium. Two extreme cases of pile driving were considered: soft and hard pile driving process. Energy imparted into the pile during the STN test can be seen as similar to the case of hard driving. Hence, Massarch & Fellenius (2008) proposed model for the hard driving case will be further modifed to suit the situation of STN test and will be presented in the next section.

2 PROPOSED MODIFIED MODEL FOR VIBRATION PREDICTION FOR STATNAMICS LOAD TEST

In pile driving situation, three main types of waves will be induced by the impact action onto the pile head: (i) Spherical waves (P-wave), (ii) cylindrical waves (S-wave) and (iii) surface waves (R-wave). The intensity of each wave vary with respect to the dynamic soil resistance along the pile shaft and at the pile toe. For easy driving condition, cylindrical waves emit from pile shaft attenuate along the shaft into surrounding soil and dominates the intensity of vibration. For hard driving condition, spherical waves emit from pile toe dominates the intensity of vibration and attenuate towards the ground surface. Surface waves are caused by refraction of P- and S-waves at the ground surface at the critical distance.

Due to local practice in Malaysia and Singapore always terminates the pile toe in competence soil or rock layer, a STN pile test is closer to a hard driving condition than a soft driving condition. Thus, the proposed model is modified from the hard driving of pile, which is dominated by a spherical wave (P-wave) as discussed in Massarch & Fellenius (2008). Fig. 5 illustrates the vibration emitted from pile toe in STN test.

2.1 Influence of Pile Penetration Depth

Since the pile toe is the main energy source in this case, in the proposed model to predict the ground vibration from STN test, the distance from energy source to measurement point should take into account the exact distance rather than horizontal distance. The influence of the pile penetration depth (i.e. the location of the energy source) is thus included by estimating the incidence angle at the ground surface towards the pile toe (see Fig. 5).

2.2 Responses of Dynamic Pile-Soil Interaction

In STN test, the pile will be "driven" into the ground under an imparted reaction force after launching the reaction mass. As a result, the "dynamic" velocity-dependent soil resistance will be activated and subsequently induced the vibrations. As compared to the original Massarch & Fellenius (2008) model, the pile hammer efficiency and energy ratio are simplified to a unity as the imparted force in STN can always be accurately estimated according to the calibrated weight of propellant and the reaction mass. Also there is only one "blow" for STN test. Similarly, the energy of the pile per "blow", W_o, can be computed based on the work done of the pile in STN according to Eq.(2).

$$W_o = \max\,[F_i\ x\ d] \tag{2}$$

where F_i is the impacting force at the pile top and d is the displacement at the pile top.

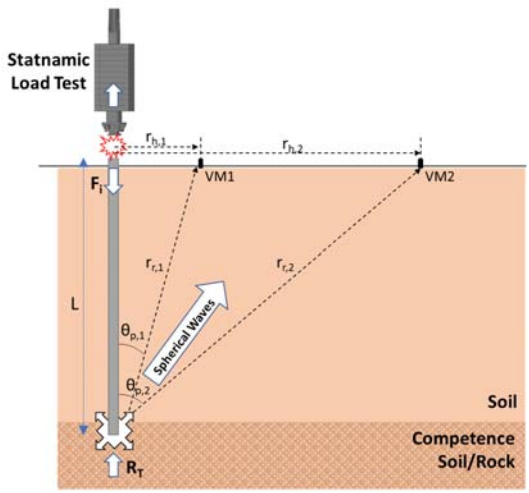

Fig. 5. Schematic diagram of wave propagation induced at pile toe in Statnamic Load Test (modified after Massarch & Fellenius, 2008).

2.3 Vibration Transmission Efficacy at Pile Toe

The dynamic force at the pile toe (i.e. soil resistance), R_T, which is the source of spherical waves emitted from the pile toe. By assuming pile top and pile toe move at same velocity, the vibration transmission efficacy can be deduced by dividing the dynamic resistance at the pile toe by the impact force at pile top, F_i, as shown in equation below:

$$E_T = \frac{R_T}{F_i} \tag{3a}$$

The pile top impact force F_i can be expressed in terms of pile impedance Z_{pile} ($Z_{pile} = A_{pile}\, c_{pile}\, \rho_{pile}$) and P-wave velocity in pile material $v_{p,\,pile}$. Whereas the dynamics soil resitance at the pile toe R_T can be expressed as soil impedance with the same corss section area as the pile, Z_{soil} ($Z_{soil} = A_{pile}\, c_{soil}\, \rho_{soil}$) times pile impact velocity v_o. Note that during the force impacting onto the pile, only part of the impact kineic energy will be converted into the P-wave travelled in the pile material, hence, it is assumed that $v_{p,pile} = 1/2\, v_o$.

During the STN load application, the pile movement is at pseudo-static condition, and the soil/rock stiffness will change with time and hence, an additional empirical factor, R_R, is introduced to account for this effect. Hence, the vibration transmission efficacy can be expressed as :

$$E_T = 2\, R_R \frac{c_{soil}}{c_{pile}} \frac{\rho_{soil}}{\rho_{pile}} \tag{3b}$$

where R_R is the empirical factor that takes into account the changes of soil/rock stiffness changes during STN duration, especially soil compaction or disturbance at the pile toe; c_{soil} is the velocity of P-wave in the soil colcum (m/s); c_{pile} is the velocity of P-wave in the pile (m/s); ρ_{soil} is the density of soil (kg/m^3); and ρ_{pile} is the density of pile (kg/m^3).

2.4 Energy Propagation in Elastic Medium

To quantify the energy transmission through an elastic medium, a material coefficient, k_s (comparable to k value in Eq.1), can be deduced by Eq.(4).

$$k_s = \frac{1}{(2\pi\rho\lambda)^{0.5}} \tag{4}$$

where ρ is the density of soil or rock (kg/m^3).
λ is the wavelength (m).

Based on elastic theory, the P-wave velocity in soil, c_{soil} also depends on the wavelength, λ, of the propagating wave, which can be determined from Eq.(5) with frequency of vibration, f.

$$\lambda = \frac{c_{soil}}{f} \tag{5}$$

At the ground surface, the emitted spherical wave from the pile toe would be reflected or refracted when it encounters a free surface. The reflection or refraction of waves depend on the angle of incidence, θ. By following Massarch & Fellenius (2008) model in analyzing the reflection of P-wave, an amplification factor, F_v, in the vertical direction could be adopted in the calculation of ground vibration. The amplification effect due to vertical reflection of vertical vibration amplitudes at the ground surface is accounted by F_V, considering also the angle of incidence of the emitted wave at the ground surface, θ_p.

$$F_v = 2\frac{\cos\theta_p \cos 2\theta_s}{s^2 \sin 2\theta_p \sin 2\theta_s + cos^2 2\theta_s} \tag{6}$$

where θ_p is the angle of incidence of spherical wave (P-wave) at ground surface.
θ_s is the angle of incidence of cylindrical wave (S-wave) at ground surface.
s is the ratio of sinus for angles of incidence of the P-wave and S-wave (see Eq.7).

$$s = \frac{sin\,\theta_s}{sin\,\theta_p} = \sqrt{\frac{1-2v}{2(1-v)}} \tag{7}$$

where v is the Poisson ratio of soil.

Finally, the vertical component of PPV velocity, PPT_v, due to spherical P-waves emitted from the pile toe, at a radial distance to the observation point, r_r, can be determined by Eq.(8), being modified from Eq.(1) with all the adjustment factors. This proposed model predicts the ground vibration in vertical direction due to STN test. It will be used to compare with the field measured results in the next section.

$$PPV_v = k_s F_v E_T \frac{\sqrt{W_o}}{r_r} \cos\theta_p \quad (8)$$

3 FIELD MEASURED RESULTS, PREDICTION AND DISCUSSION

This section summarizes the vibration measurement of a number of projects involved testing large diameter bored piles using STN test. The vibration of ground is monitored at various distance. The details of each test piles, include the maximum test load, pile geometry, and the key soil profile are summarized and discussed. It is followed by the derivation of the input parameters for the proposed model, as well as the predicted peak particle velocity PPV.

3.1 The Ground Vibration in Statnamic Load Tests

A total of 10 numbers of bored piles conducted with STN are selected for this study. These are with diameter (D) of 1.0 to 2.0m, with pile penetration length ranged from 12 to 36m. Note that four numbers of piles terminated at competence soil without penetrating into rock layer. The rest of the piles are terminated at varies length into the rock layer. The details of the pile information are tabulated in Table 1.

Table 1. The Dimension & Geometry of Test Piles

Project	Max. Test Load (kN)	Pile Diameter (m)	Total Pile Length (m)	Penetration Length in Soil (m)	Penetration Length in Rock (m)
Sungai Penchala	38220	1.8	12.6	9.4	3.2
SUKE	42700	1.5	27.4	20.4	7.0
TRX	33250	1.5	21.4	19.9	1.5
DASH #1	53820	2.0	37.8	32.0	5.8
AKASA	51030	1.8	24.9	23.9	1.0
ACE	15350	1.2	29.1	29.1	-
	15090	1.2	26.6	26.6	-
	24050	1.5	34.4	34.4	-
	24580	1.5	35.5	35.5	-
DASH #2	11000	1.0	18.9	16.4	2.5

3.2 Derivation of the Input Parameters of the Proposed Model

In this study, the test piles are dicied into two groups: (a) Group 1 for piles toe terminated well into competent rock layer, such that "Rock penetration length" >2D, and (b) Group 2 for pile toe terminated in soil layer or only shallow depth into rock layer, i.e. "Rock penetration length" = 0 to <2D, where D = pile diameter.

The density and velocity of P-wave in the competence soil is assumed to be 1900 kg/m^3 and 1450 m/s respectvely. This is corresponding to soil profile where ground water table are near to ground surface which is the case in Singapore and Malaysia. For Group 2 cases, where "Rock penetration length" <2D, the P-wave is also assumed to be dominated by the adjacent competence soils rather than the rock layer. For Group 1 cases, i.e. with relatively long rock penetration length (>2D), the density of rock and velocity of P-wave are adopted as 2200 kg/m^3 and 3000 m/s respectively.

The peak ground vibration can be computed using the proposed model with the input parameters summarised in Table 2, based on the equaitions discussed in previous section. In addition, for Group 1 cases, i.e. piles with a relatively long rock penetration length (>2D), the empirical factor, R_R, account for disturbance of soil/rock properties at the pile toe, can be taken as not significant and thus assumed to be 1. For Group 2 cases, R_R will be conservatively adopt a value of 2, to take into account the compaction of soil near the pile toe.

Table 2 – The Key Input Parameters of the Proposed Model

Projects	k_s (√[m^2/kg])	F_v	R_R	E_T	r_r (m)	θ_p (°)
Sungai Penchala	0.0010	0.63	2	1.15	41.94	72.50
	0.0009	0.74	2	1.15	32.54	67.20
SUKE	0.0007	1.02	1	1.38	48.48	55.59
TRX	0.0009	0.86	2	1.15	45.36	61.85
DASH #1	0.0004	1.46	1	1.38	48.26	38.44
	0.0004	1.46	1	1.38	48.26	38.44
AKASA	0.0008	0.56	2	1.15	103.1	76.02
	0.0006	1.15	2	1.15	38.99	50.31
ACE #1	0.0007	1.01	2	2.30	66.68	64.13
	0.0007	1.44	2	1.15	41.79	45.87
ACE #2	0.0006	1.38	2	1.15	40.09	48.44
	0.0007	0.96	2	2.30	65.63	66.09
ACE #3	0.0006	1.54	2	1.15	45.64	41.09
	0.0007	1.22	2	2.30	60.69	55.47
ACE #4	0.0007	1.75	2	1.15	40.75	29.40
	0.0007	1.38	2	1.15	53.48	48.41
DASH #2	0.0005	1.25	1	1.38	27.52	46.62
	0.0004	1.09	1	1.38	31.34	52.91

3.3 Field Measured Ground Vibration

Majority of the test piles with STN conducted were instrumented with vibration measurement at two locations at varies horizontal distances from the test pile. Table 3 tabulates the dominant frequency and the measured PPV in the vertical direction in each project site. The dominant frequency generally ranged between 6Hz to 23Hz and the PPV in vertical direction ranged between 1.5m/s to 5.5m/s.

3.4 Comparison between Measured and Predicted Ground Vibration

Based on the project and pile information for 10 numbers of project sites, a total 18 locations of ground vibration are predicted and compared to the measured PPV according to the dominant frequency of vibration. Fig. 6 and Table 3 plots and tabulates the predicted PPV

determined according to the proposed model versus the measured PPV. It can be seen that the proposed model agrees very well with the measured PPV data. The linear best fit line betwene the predicted vs measured PPV is having a gradient of 0.990, with a R^2 error of 0.8588. Furthermore, it can be seen also that almost all data points fall well within the 95%-tiles lines on both sides of the best-fit line.

Table 3 – The Field Measurement nad Predicted of Ground Vibration in Vertical Direction

Projects	Distance, r_h (m)	Dominant Frequency , f (Hz)	Measured Vertical PPV (mm/s)	Predicted Vertical PPV (mm/s)
Sungai Penchala	40	17	1.51	1.71
	30	13	1.75	1.46
SUKE	40	23	1.94	3.29
TRX	40	13	1.94	2.02
DASH #1	30	7.0	5.43	4.22
	30	7.9	3.86	4.49
AKASA	100	11.0	0.51	0.31
	30	6.9	2.54	3.51
ACE #1	60	8.3	1.81	0.59
	30	9	3.17	2.25
ACE #2	30	6.4	1.11	1.55
	60	8.8	1.68	0.47
ACE #3	30	6.3	2.32	2.52
	50	8.1	3.48	1.28
ACE #4	20	8.0	3.19	4.48
	40	9.1	3.16	2.19
DASH #2	20	8.5	3.52	2.00
	25	6.6	1.52	1.18
		Standard Deviation	1.775	1.325
		Correlation Coefficient	0.990	
		For 95% Prediction Interval	1.20 mm/s	

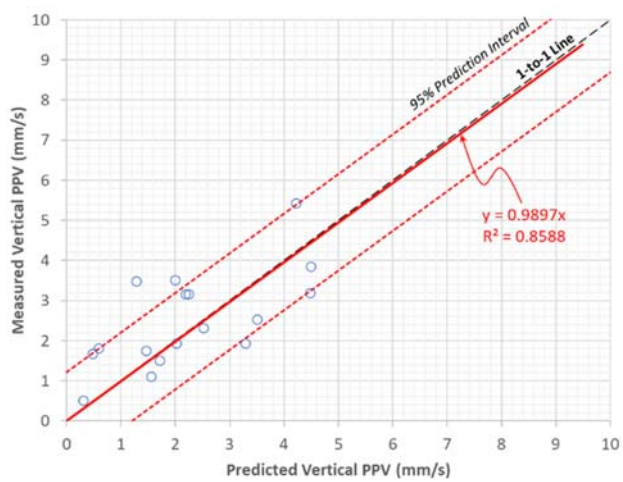

Fig. 6. The comparison of the measured and predicted PPV in varies Statnamic Load Test.

4 CONCLUSIONS

A new model was sucessfully developed for the prediction of vertical component of peak particle velocity PPV of Statnamics pile load test on pile well socket into competent soil/rock, based on the modification from Massarch & Fellenius (2008)'s Model for pile driving. The proposed model makes some assumptions on energy tramssmission in soils, and generalization of soil/rock properties for the ease of computation. Despite of the simplification, the proposed model shows very good comparison with the field measured data. Most importantly, this model is able to predict acurately the PPV for STN test with test load of 11 MN to 53 MN, which was not able to be done using the previous predictive curves that show continous increase of PPV with load. This model predicts the PPV taking into account not ony the test load and horizontal distance, but also the soil-pile interaction, pile penetration length, as well as soil type surrounding the pile.

REFERENCES

1) Attenwell, P.B. and Farmer, I.W. (1973). "Attenuation of Ground Vibrations from Piles Driving". In *Ground Engineering*, Vol. 6, No. 4, pp. 26-29.

2) Chew, S.H., Chuah L.S., Seow, L.L., and Tan, H.H. (2012). "Vibration Effect due to Statnamic Pile Load Test". In *9th International Conference on Testing and Design Methods for Deep Foundations, 18-20 September 2012, Kanazawa, Japan*, pp. 273-280.

3) Chew, S.H., Chuah L.S., Tan, H.H.,and Eng, Z.X. (2019). "A Comparison Study for Static and Statnamic Load Tests on Two Instrumented Piles in Southeast Asia". *Stress Wave Theory and Testing Methods for Deep Foundations: 10th International Conference, ASTM STP1611*, P. Bullock, G. Verbeek, S. Paikowsky, and D. Tara, Eds., ASTM International, West Conshohocken, PA, 2019, pp. 308–323.

4) Deckner, F. (2013). "Ground vibrations due to pile and sheet pile driving: influencing factors, predictions and measurements". (*Doctoral dissertation*, KTH Royal Institute of Technology).

5) Head. J. M. and Jardine, F.M. (1992). "Ground-borne Vibrations Arising from Piling". *Technical Note 142, Construction Industry Research & Information Association (CIRIA)*, London, UK.

6) Hölscher, P., Brassinga, H., Brown, M., Middendorp, P., Profittlich, M., & van Tol, F. A. (2011). "Rapid load testing on piles: Interpretation guidelines" (Vol. 230). CRC Press.

7) Luk, S.T., Ko. J.M., and Cheng, C.Y. (1990). "Ground Vibrations Induced by Driving Concrete Piles". *Proceedings of the Conference on Vibration and Noise, Sydney*, pp. 245-249.

8) Massarsch, K. Rainer, and Bengt H. Fellenius (2008). "Ground vibrations induced by impact pile driving." *6th International Conference on Case Histories in Geotechnical Engineering. Arlington, VA, USA*.

9) Middendorp, P (2011). 'Private communication'.