Available online at www.eccomasproceedia.org
Eccomas Proceedia COMPDYN (2023) 889-899

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COMPDYN 2023
9th ECCOMAS Thematic Conference on
Computational Methods in Structural Dynamics and Earthquake Engineering
M. Papadrakakis, M. Fragiadakis (eds.)
Athens, Greece, 12-14 June 2023

GEOTECHNICAL CONSIDERATIONS AND VIBRATION RESPONSE DURING ROTARY PILING FOR CONSTRUCTING BRIDGES

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Abstract

Vibratory energy is transmitted to surrounding soil (and atmospheric) media during some construction process such as blasting or pile driving in soil. The ground borne vibratory forces may affect the nearby structures and may casue inconvenience to the habitat. Several models to predict the ground borne vibrations due to blasting, impact and hammer type pile driving are available in litearture but not for rotary type. A model of vibration induced by rotary pile driving is proposed. The proposed methodology is validated by vibration monitoring during rotary pile driving for a railway bridge site surrounded by bridges on either side. The results observed during pre-construction and post- construction of the bridge are discussed.

Keywords: Rotary pile, ground borne vibration, vibration attenuation, soil-dynamics modeling, railway bridges.

Peer-review under responsibility of the organizing committee of COMPDYN 2023.
doi: 10.7712/120123.10444.20749

1 INTRODUCTION

During the construction of civil infrastructure, some activities generate vibratory forces in the vicinity of the construction activity such as blasting, pile driving and traffic of vehicles, to name a few. The surrounding structures experience the vibratory forces depending upon the type of inducing vibratory action, soil condition, type of receiving structure (such as masonry, concrete or steel), and the distance of the structure from the source of vibration. Such ground-borne noise and vibration can also influence humans depending on vibration intensity and perception by the receiver.

The energy imparted to the soil by vibrations or impact forces during pile driving is dispersed in the ground. Several types of waves are generated similar to the earthquake and get radiated outwardly in all directions from the source of vibration. There are faster body waves which propagate in the soil as spherical wavefront. When the particle motion is in longitudinal directions, it is called primary waves (P-waves). Other body waves are shear waves (S-waves) in which the particle moves in the transverse direction or perpendicular to the direction of propagation of P-waves. Other waves are surface waves called Rayleigh waves (R-waves) which travel on the surface. These energy waves excite the object coming in the way depending upon the distance to the object before losing their intensity. R-waves are having large energy and diminish slowly inversely in proportion to the distance. Thus, R-waves are more of interest to the interfering buildings/ bridge foundation [1], [2]. Ground-borne vibration is generally expressed as root-mean-square (RMS) of the vibration velocity expressed in terms of vibration decibels (VdB) or velocity as mm/S. Also, the vibration velocity is expressed as peak particle velocity (PPV), defined as the maximum instantaneous peak of the vibration signal in mm per second. The measurement of acceleration is also recorded whose integration yields velocity. The damage to the nearby structures (buildings as well bridges) due to vibratory forces varies based on the intensity and frequency content of the vibration as well as perceived by the structure. Pile driving and other activities have the potential to induce large dynamic forces in the surrounding structures leading to moderate to severe damage or even collapse of the structure similar to the natural hazards causing devastations to bridges [3].

Since guideline for the vibration induced during rotary piling is not available, a model is proposed taking into account the energy imparted by the rotary piling drive. Experimental data are obtained from a railway bridge site surrounded by two parallel bridges on each side of the proposed bridge, whose details are discussed in the present study.

2 PREDICTION OF SURFACE VIBRATIONS DURING PILE DRIVING

The vibration thresholds for the structural condition, when exposed to dynamic forces, are described by some widely accepted standards such as [4] in terms of peak particle velocity which are dependent on the frequency of the structure and are given in Table 1.

Type	Structure type	Velocity (PPV) as a function of frequency* (mm/s)	
		4-15 Hz	Above 15 Hz
1	Reinforced or framed structures, industrial and heavy commercial buildings.	50 mm/S at or above 4 Hz.	50 mm/S at or above 4Hz.
2	Unreinforced or light-framed structures.	15 mm/S at 4Hz and increases to 20 mm/S at 15 Hz.	20 mm/S at 15Hz and increases to 50 mm/S at and above 40 Hz.

Table 1: Vibration threshold values for cosmetic damage (BS 7385-2)

The values referred in the Table 1 are at the base of the building; for vibration frequencies below 4 Hz, a maximum displacement of the structure should not exceed a value of 0.6 mm. Based on experiments on blast-induced vibrations, researchers [5], [6], [7] and others have observed that the PPV is correlated with the energy of the blast leading to the structural damage; induced PPV is also inversely proportional to the distance of the point of interest (D) from the source of blast and is expressed as Eqn. 1.

$$PPV = CE^{\alpha} \; ; \qquad PPV = kD^{-n} \tag{1}$$

Here, C and α are constants, E is blasting energy, D is the distance from the source, k and n are constants describing the attenuation rate of the vibration velocity. A different relationship for the vibration attenuation in terms of the PPV at a distance D from the reference distance D_0 is given by Bornitz (1931) as Eqn. 2.

$$PPV_D = PPV_0 \left({}^{D}/_{D_0} \right)^{-n} e^{-\propto(D-D_0)} \tag{2}$$

The value of n and α (the attenuation coefficient) is different from the earlier exponents. The value of n varies with the type of dispersive waves such $n = 1$ for the body waves in the ground, $n = 2$ for body waves on the surface and $n = 0.5$ for Rayleigh waves. Based on several researchers' data on pile driving, CDOT [1] proposed vibration velocity due to impact vibratory pile driving as given in Eqn 4 (in equivalent SI units).

$$PPV = PPV_{ref} \left({}^{7.62}/_{D} \right)^{n} \left({}^{E_{eqp}}/_{E_{ref}} \right)^{0.5} \tag{3}$$

Here, PPV_{ref} is 16.51 mm/sec at 7.62 m as the reference point of a pile driver, n is 1.1 for hard soil (compacted sand or dry consolidated clay) and ranges from 1 to 1.4 for different soils, E_{ref} is 48.81 kN-m and E_{eqp} is the rated energy of impact pile driver. The corresponding correlations and attenuated PPV for rotary pile drivers are not available, the relevant model based on energy imparted to soil is proposed as follows. The energy content during rotary piling E_{eqpr} is the function of the diameter of the pile ($2r$), the mass of the rotor (m) and its rpm (ω). Thus, the kinetic energy (KE) imparted is expressed as Eqn. 4 where an additional factor β is introduced to correlate the KE with the impact.

$$E_{eqpr} = 0.5mv^2\beta^2 = 0.5m(r\omega)^2\beta^2 \tag{4}$$

3 FIELD STUDY OF THE BRIDGE

3.1 Description of the bridges

There exist two railway bridges (No. 17) over the river Saraswati near Andul railway station called "Up line" and "Middle & Down lines". The existing piers (and foundations) are made of concrete and the girders are made of steel. The proposed bridge is to be located between these two bridges and is to be founded on RCC piles as shown in Fig. 1. The distance between the existing piers and the proposed location of construction for piles is narrow in the range of 4.0 m from the centre of the pile to the nearest edge of the pier (of the up line). The drilling for the pile is using rotary drill system. This distance is quite less and the vibration getting transmitted to the nearby RC structures may have damaging effects. The diameter of the piles is 1.2 m, the number of pills in a group of six or ten and the depth below the ground level (GL) is 14 m.

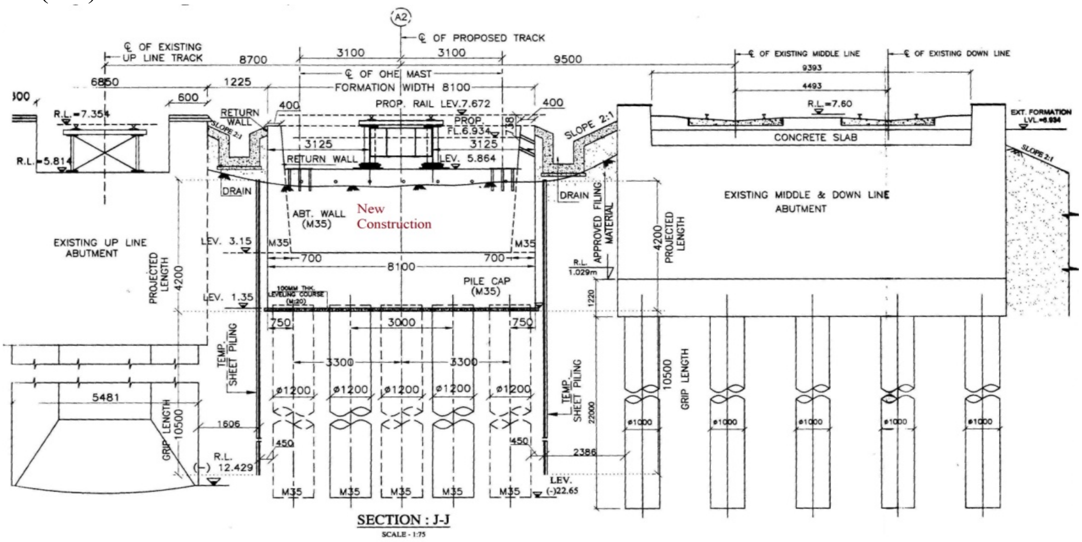

Fig. 1: Sectional view of the bridges (existing and new construction)

3.2 Soil characteristics

The soil characteristics are studied which are adopted from the soil report [7]. Four boreholes (BH) up to 45 m depth were used and Standard Penetration Test (SPT) values were obtained apart from soil properties using standards [8], [9] and [10]. The soil is primarily grey silty clay. The standing water (during Jan. /Feb.) on boreholes was observed at 0.80 m to 1.20 m below the ground level. The shear wave velocity is assessed and is shown in Fig. 2 for BH-1 at the Andul site. The soil is clayey-silt as shown in Table 2 and the remaining constituent is fine and medium sand. Representative values of other properties of interest are available.

Bore Hole	Type of Soil (representative value in percentage)	Bulk Density (gm/CC)	Chemical Analysis	Water Table below GL (m)
BH1	Clayey (33)-Silt (58)	1.700	pH: 7.3 to 7.9. Chlorides: <0.053%	7.75
BH2	Clayey (30)-Silt (58)	1.725	pH: 7.6 to 7.7. Chlorides: <0.052%	4.50
BH3	Clayey (24)-Silt (72)	1.689	pH: 6.9 to 7.8. Chlorides: <0.057%	5.10
BH4	Clayey (22)-Silt (60)	1.870	pH: 6.7 to 7.4. Chlorides: 0.050%	8.55

Table 2: Soil properties at the site

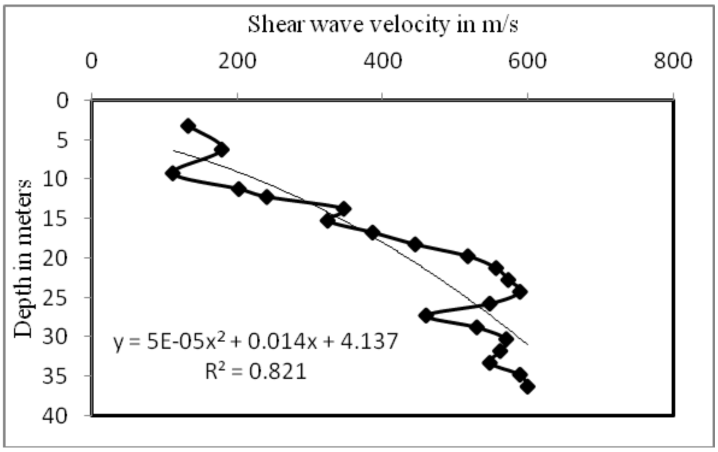

Fig.2. Shear wave velocity vs. depth (BH-1) at Andul site

3.3 Drilling for piles

The drilling for piles is a rotatory type and a view is depicted in Fig. 3. The process of piling using a vibratory hammer induces lower forces compared to the vibratory impacts. The rotary bored pile machine has a rig height of 8m, rig weight of 100 kg, rig length of 28 m to 36 m, dia of the ring as 1200 mm, and works at 1000 rpm. Vibrations from transportation activities and during construction process have a frequency range of 10– 30 Hz and is usually centred around 15 Hz [11].

Fig. 3: The view of the site showing the location for new construction between two rail bridges

4 INSTRUMENTATION SCHEME

The data of vibrations emanating from the construction activity for a short distance is scanty in literature [5], [12], [13], [14] and others. Accordingly, an instrumentation scheme has been devised consisting of accelerometers as well as velocity sensors due to their effective data sensing and recording. Several sensors were placed on nearby structures (piers) and on the ground soil to measure vibration levels caused by the drilling process. Also, the effects of the ongoing source of the movement of trains were simultaneously recorded. The more dominating locations (A, B, C, and D) are marked in Fig 4, also the mark of the pile under drilling and placement of sensors on the ground.

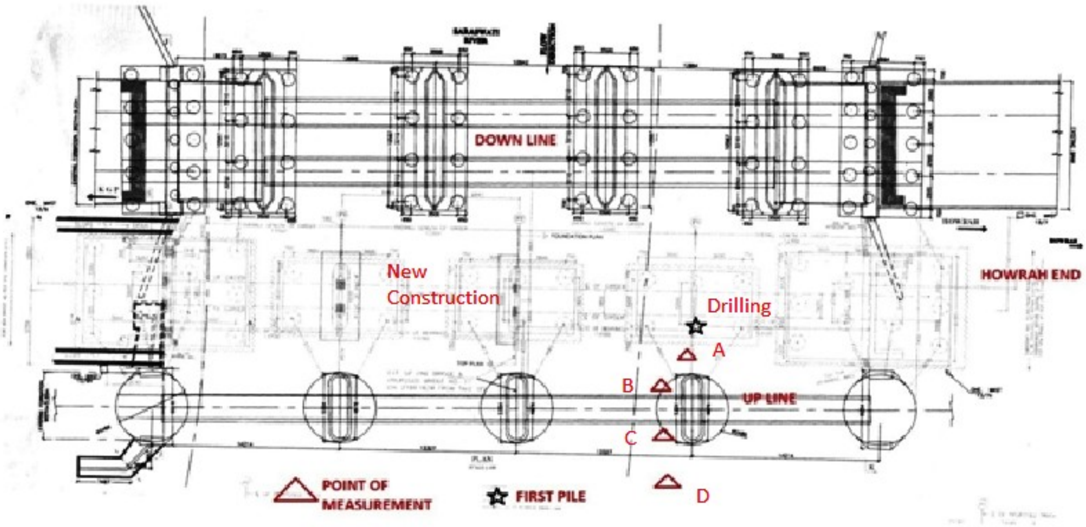

Fig. 4: Plan view of the bridges with locations of interest of measurements

The drilling location of pile number 2 for pier P1 as proposed is critical to transmitting vibration to the nearby pier, i.e. P1 of the up-line. The dominating locations for vibration measurements relative to the location of drilling are marked as A at 1.5 m from the surface of the pile, B at 4m, C at 8 m and D at 12 m. Two types of accelerometers namely uniaxial and triaxial are used to measure acceleration response in all three directions, respectively. These are small size, small weight sensors and are suitable for the test having a sensitivity of nearly 1000 mV/g and a frequency range of 1 Hz to 10 kHz. The Dynamic Data Acquisition System (DDAQ) is of make Samurai of M/s Sinus Co., Germany. Also, another DDAQ employed is a 16-channel system of OROS, France and is called Mobi-Pack due to its compactness. These are suitable for field studies and can resist reasonably harsh environments (mud, oil, rain) without impacting the instrument. There is a 24-bit sigma-delta ADC for each input and the highest sampling rate is 102.4 kHz with suitable anti-aliasing filters. The highest sampling rate is used in this experiment. Some views of the instrumentation along with sensors at the site are given in Fig. 5.

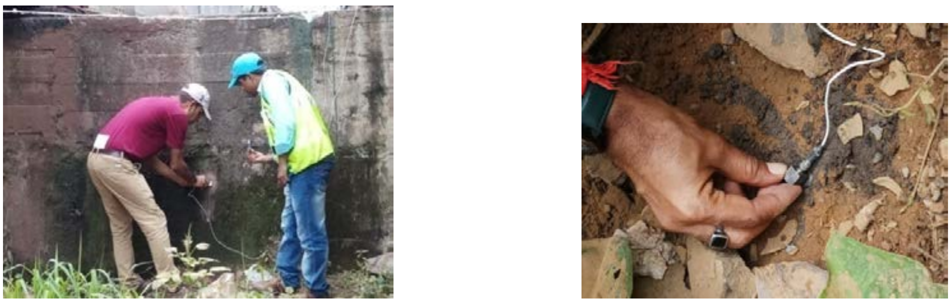

Fig.5: Placement of sensor- a) on the pier, b) on the ground

5 RESULTS AND DISCUSSIONS

5.1 During the construction stage

The longitudinal direction of the bridge i.e. along the span length of the bridge is X-direction, across the span is Y-direction and the vertical direction is Z-direction for reporting the measurements. The frequency response observed during the drilling operation was 20 Hz to 200 Hz and with the dominating frequency of 31.5 Hz when 1/3rd octave analysis was carried out. The observed representative vibration levels are provided in Table 3. Here, AdB is the acceleration shown in decibels (dB).

Depth 4m	dB (x)	dB (Y)	dB (z)
H=1m	88.2	95.3	99.8
H=4m	66.52	79.17	75.62
Depth 6m	dB (x) peak	dB (Y)	dB (z)
H=1m	77.9	90.9	89.1

Table 3: Vibration levels as measured at various depths

The spectrum of the existing pier under the passage of the train is shown in Fig. 6a and the natural frequency of the pier is 5.8 Hz. The acceleration octave response of the pile driver at **a** piling depth of 4 m and at a horizontal distance of 4m on the ground surface (vibration acceleration in m/S^2 vs. frequency in Hz; three colours represent x, y and z directions) is given in Fig. 6b. Based on observations during driving motion the maximum vibration levels of accelerations are deduced in terms of peak particle velocity (PPV) and are reported in Table 4. The maximum level of AdB at the site has been recorded as 111.3 dB under influence of the passage of the train and the equivalent PPV at the site (mm/s) is 26.71 mm/s.

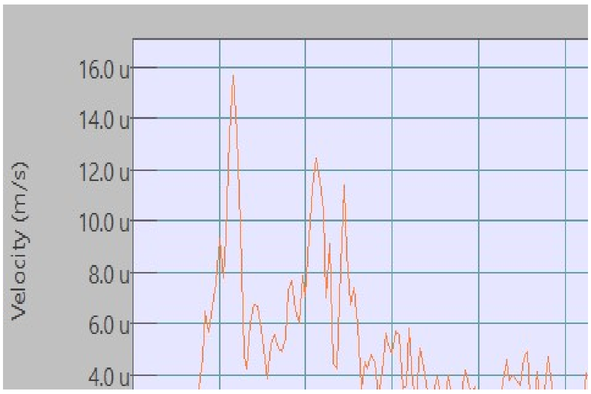

Fig.6a: Frequency spectrum of the existing pier (up line)

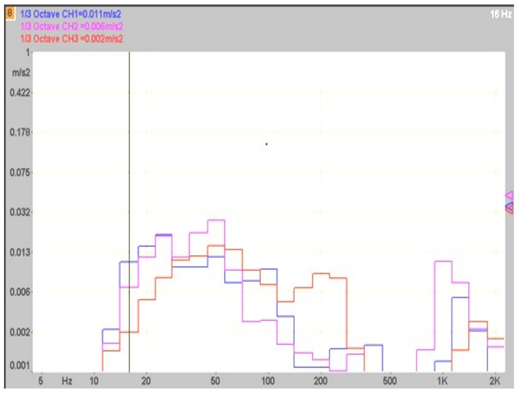

Fig.6b: Octave Response

Location	Vibration level (acceleration)- Drilling only (AdB)				Max Vibration level- Drilling with the passage of the train-	
	X-Direction	Y-Direction	Z-Direction	Eq. PPV Max (mm/S)	Vibration level(AdB)	Eq. PPV Max(mm/S)
A (1.5)	100.0	106.4	101.7	25.53	111.3	26.71
B (4)	90.2	98.2	93.1	23.55	102.6	24.22
C (8)	88.6	90.0	89.1	21.59	94.1	22.50
D (12)	75.3	80.9	83.9	19.90	84.5	20.27

Table 4: Maximum Vibration measured at the site during construction

Considering the frequency contents by 1/3rd octave analysis and equivalent peak particle velocity (PPV) has been deduced. The allowable PPV (mm/s) is 50 mm/s and eq. AdB level is

120.0 dB [3] and [12] for cosmetic damage to the structure. The maximum observed vibration level at the site has been 111.3 AdB or eq. PPV of 26.71 mm/s which is below the threshold PPV of 50 mm/s for concrete structure for a range of frequency near 31.5 Hz of excitation and structure itself at 5.8 Hz. Since the structural vibration frequency (5.8 Hz) is far from the excitation frequency (31.5 Hz) as per soil dynamic considerations [15] and the vibration level (26.71 mm/s) is lower than the threshold of vibration (50 mm/s), the structures namely the piers are safe due to drilling combined with the passage of the train. As per IRS-IRBM [9], Section 505 "Imposition of speed restriction-Group II-10. Observations of any excessive vibration in part of the bridge structure", it may be noted that the train speed was restricted to 30 kmph, which has been maintained during the drilling operation near the site. Also, other piles in the vicinity of the present study are at a farther distance of 3m than those worked in the study. Therefore, it was safe to continue the construction operation of drilling at a lower speed of passage of trains as mentioned above.

5.2 Validation proposed rotary driving impact

The method as proposed in Eqn. 4 for rotary pile driving, the vibration response obtained in the present study described in the previous section is utilised. The vibration attenuation observed in the study is shown in Fig. 7.

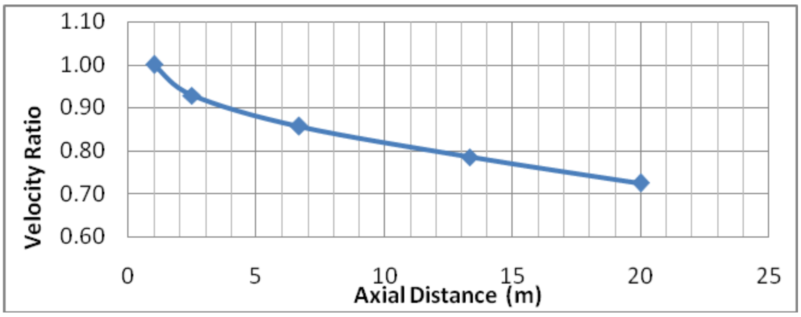

Fig.7: Observed vibration attenuation

In the present study, $r = 0.6\ m$ (diameter of the pile, $2r = 1.2\ m$), the mass m is 100 kg (10.91 N/m.S^2), the rpm, $\omega = 1000$ rpm $(= 2\pi.\frac{1000}{60}$ radian per sec) thus, E_{eqpr} is 19.96 β^2 kN-m. Also, the correspondence of the function, beta as given in Eqn. 4 is validated as shown in Fig. 8.

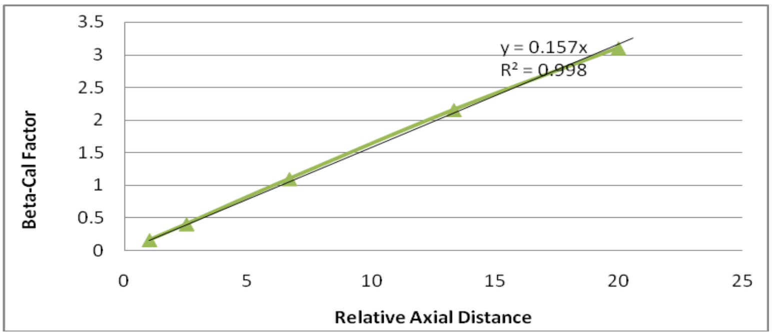

Fig. 8: Correspondence with the factor beta

5.3 During post-construction stage

Although the piers of the existing bridges are not damaged during the drilling process as per visual inspection, the vibration monitoring was carried out on the old pier as well as the new pier (P1). The time series of vibration (velocity) of the upline pier top of P1 is shown in Fig. 6a. The Octave plot corresponding to acceleration in three directions of the upline pier top of P1 and the spectrum of acceleration at the old pier top of P1 in three directions were also obtained.

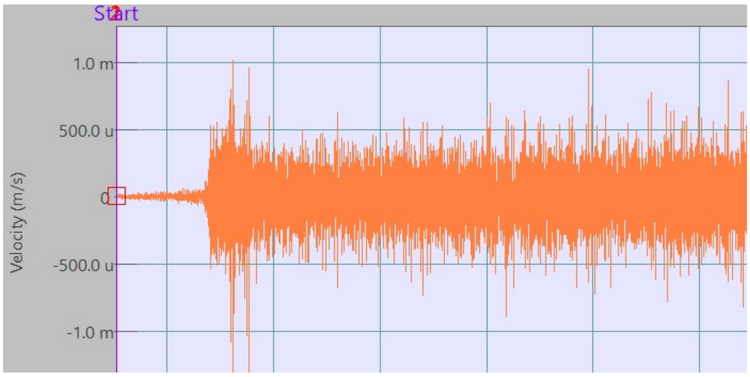

Fig. 9: Time series vertical velocity Up line pier top of P1 {ch7Fileser33/goodstrain1708}

6 CONCLUSIONS

- Systematic vibration monitoring using accelerometers and velocity sensors was carried out at the bridge site during the construction and post-construction stages.
- A model for predicting vibration energy from rotary pile driving is proposed and validated using field study.
- The frequency response observed during the drilling operation was 20 Hz to 200 Hz and with a dominating frequency of 31.5 Hz when 1/3rd octave analysis was carried out.
- The maximum level of acceleration (AdB) at the site has been recorded as 106.4 dB under drilling operation only and the eq. PPV, peak particle velocity at the site (mm/s) is 25.53 mm/s.
- The allowable PPV (mm/s) is 50 mm/s and eq. AdB level is 120.0 dB (BS7385-2) for cosmetic damage to the structure. The maximum observed vibration level at the site of 111.3 AdB or eq. PPV of 26.71 mm/s under combined drilling and train movements is below the threshold PPV of 50 mm/s for concrete structure for a range of frequency near 31.5 Hz of excitation and structure itself at 5.8 Hz. The structures namely the piers are safe due to drilling even during the passage of the train. Therefore, it was safe to continue the construction operation of drilling.
- The vibration measurements at the old and newly constructed piers were carried out after the construction of the bridge and found some similarities between the two structures. The old bridges are found to be intact based on the vibration monitoring (consistency of values in three directions) post-construction.
- The study has culminated successful construction of the foundations, piers and superstructure of the planned bridge between existing railway bridges without affecting the structure in its vicinity due to pile driving operations.

ACKNOWLEDGEMENTS

This work is accomplished under a project, "Impact on Vibration Levels on Railway Bridge between Andul-Sankrail during Construction Process", sponsored by South Eastern Railway (SER), Indian Railways, Kolkata, India.

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