Ground Vibrations during Installation of Rockfill Columns

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GeoCalgary 2022 October 2-5
Reflection on Resources

ABSTRACT
The installation of rockfill columns involves a vibration process that can propagate vibrations through the ground. Research was done to investigate the vibrations during installations of rockfill columns in a riverbank in Winnipeg, Manitoba. The impact of ground vibrations on a nearby buried aqueduct was also investigated through a calibrated finite element model. Numerical results in terms of peak particle velocity (PPV) were compared to guideline values worldwide, to literature data, and to prediction equations available in the literature. Results show that the PPVs for this research are below the limiting values of four guidelines and agree well with literature data. The prediction equations from the literature underpredicted the values of PPV, so new empirical formulas are proposed.

RÉSUMÉ
L'installation de colonnes en enrochement implique un processus de vibration qui peut propager les vibrations à travers le sol. Des recherches ont été menées pour étudier les vibrations lors de l'installation de colonnes d'enrochement dans une berge à Winnipeg, au Manitoba. L'impact des vibrations du sol sur un aqueduc enterré à proximité a également été étudié à l'aide d'un modèle d'éléments finis calibré. Les résultats numériques en termes de vitesse maximale des particules (PPV) ont été comparés aux valeurs indicatives mondiales, aux données de la littérature et aux équations de prédiction disponibles dans la littérature. Les résultats montrent que les PPV de cette recherche sont inférieures aux valeurs limites de quatre lignes directrices et concordent bien avec les données de la littérature. Les équations de prédiction de la littérature ont sous-estimé les valeurs de PPV, de nouvelles formules empiriques sont donc proposées.

1 INTRODUCTION

Winnipeg is a city located in the Canadian Prairies, in a region once occupied by the large glacial lake Agassiz. Because of the glacial deposits, the city is underlain by a thick layer of soft, weak, high plasticity clay, with silt sediments.

The city has four rivers that, together, cover north, south, east, and west of the city. Even though the presence of the rivers has served as a transportation medium in the past, riverbank instability is a major engineering challenge. These instabilities are due in part to the presence of the thick, soft soil.

Deep-seated slope failures of the riverbanks have been identified, studied, and addressed in various locations in the city for decades (Baracos 1978; Tutkaluk et al 1998), and these continue to occur. Rockfill columns (also known as stone columns or granular inclusions) have been used to stabilize Winnipeg riverbanks since the 1990s (Abdul-Razaq 2007; Thiessen et al. 2011; Bartz et al. 2018).

1.1 Rockfill columns

Rockfill columns are non-rigid structural elements that are used, for example, to stabilize riverbanks. The installation technique varies around the world. In Winnipeg, they consist of pre-bored holes of up to 3 m in diameter, penetrating at least 0.3 m into the stronger layer (till), backfilled with crushed limestone. Typically, about 15-35% of the weak soil is replaced with a material of lower compressibility and higher shear strength (Barksdale and Bachus 1983).

The rockfill is normally densified to increase its stiffness and load-bearing capacity and to reduce settlements (Poorooshasb and Meyerhof 1997; Bouziane, et al. 2020). Densification of rockfill is done through vibration, either using a vibrolance within the stones or by vibratory removal of steel casings from the shaft hole.

1.2 Case study

A riverbank in Winnipeg showed signs of instability. A major 100-year-old aqueduct was crossing this riverbank, which required critical considerations during the design and construction of the stability works.

Rockfill columns were selected as the stabilization method for this area, and two rows of tightly spaced columns were installed in the mid and lower banks. The closest column was located only 4 m from the aqueduct.

The columns had two diameters: a smaller one at the bottom and a larger one at the top portion (Figure 1). This configuration was chosen to ease construction and potentially minimize the vibration impact on the aqueduct. For both portions, the holes were pre-bored, then steel sleeves were pushed through to support the walls. After placing the rockfill, the sleeves were removed through vibratory pulling using a vibrodriver, with the bottom sleeves being pulled out first, and the top sleeves last.

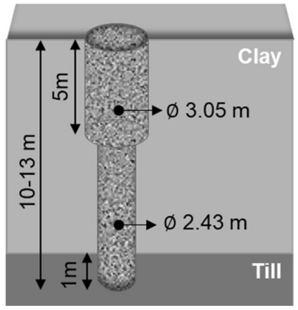

Figure 1. Typical rockfill column dimensions and placement for the case study.

1.3 Ground vibration from rockfill column installation

During the densification of the rockfill, the vibratory waves impact the column stones, but also propagate through the ground. When these waves approach a structure, they may change their intensity through reflections and refractions, but the structure is also impacted. A foundation, for example, may translate, rotate, or settle excessively (Kramer 1996).

When the waves travel through the ground, each soil particle will move in an oscillating manner. The levels of vibration in the ground are usually categorized in terms of peak particle velocity (PPV). PPV is the maximum velocity a single particle will reach during its movement. The vibration from the removal of steel casings or the vibratory installation of piles generates mainly vertical components of vibration, from shear waves (Athanasopoulos and Pelekis 2000; Hamidi et al. 2018).

1.4 Ground vibration and PPV

Considering the number of interrelated variables that lead to ground vibration, the literature has not been able to provide a single relationship to predict the PPV based on the source and ground conditions (Chan et al. 2011). Thus, guidelines worldwide use a limiting value based on field-measured PPV. For instance, since 1960, PPV has been used as the parameter to limit architectural damage from vibratory activities, with a maximum value of about 50 mm/s (Edwards and Northwood 1960). Regarding the protection of buried structures, various guidelines were proposed (Table 1). There seems to be no consensus as to which limiting PPV value should be used.

In the literature, the efforts to predict PPV are based on some empirical relationships that have been developed from limited case studies. As new studies are conducted, more data is added to the original studies, and the empirical formulas are adjusted accordingly.

For vibrated stone columns, the literature currently uses two predictors of PPV with distance from the source: one from Attewell (1995) and the other from Hiller and Crabb (2000). The equations of both predictors are presented later in this paper. The predictor from Hiller and Crabb (2000) is the only one that was developed exclusively for stone columns. Attewell's method was developed for vibratory pile driving, which the literature considers generating vibrations of similar nature to those from the installation of stone columns (Hiller and Crabb 2000; Ramshaw 2002). Both predictors are for PPV measured at the ground surface.

Table 1. Limiting PPV to buried structures.

PPV (mm/s)	Comments	Reference
5	"Safe" Continuous vibration in any structure	Caltrans (2013)
15	Continuous vibration	BS 5228-2 (2014)
30	Transient vibration	BS 5228-2 (2014)
25	Continuous vibration	Eurocode 3 (1998)[1]
40	Transient vibration	Eurocode 3 (1998)[1]
50	From blasting, in Canada	NRC (1960)
76	Tile or concrete drainpipe, sewer and water mains, and pipelines	Wiss (1981)
76-125	From dynamic compaction	FHWA-SA-95-037 (1995)
100	Draining tunnels	Kulkarni (2004)
125	From blasting, measured at the ground surface	USBM RI 9523 (1994)
150	Transport tunnels	Kulkarni (2004)
250	Buried concrete block	AASHTO R 8-96 (2019)
250-500	High pressure pipelines	Wiss (1981)
305	Underground structures	Lauzon, et al. (2011)
915	Tunnel with shotcrete liner	AASHTO R 8-96 (2019)

[1]These values were included in the Appendix C of the Eurocode 3 released in 1998. Newer versions do not include this appendix anymore, although the literature continues to cite them as references for PPV for various kinds of structures.

Additionally, Chan et al. (2011) present data on the PPV versus distance of 19 stone columns installed in the Lower Mainland of British Columbia, Canada. The PPVs were measured at the ground surface. Although the authors did not introduce any predicting equations, their data may be used to expand the knowledge of ground vibration due to the installation stone columns. Their data is also presented later in this paper.

In this paper, the PPV from the installation of one rockfill column in Winnipeg is used to compare with the limiting values shown in Table 1. The PPV at the ground surface is also used with the data from Chan et al. (2011) and compared with the two predictors (from Attewell and Hiller and Crabb), to verify their suitability to this case.

2 METHODOLOGY

The vibration from the installation of one rockfill column of the case study was measured at two different depths, as opposed to the ground surface. The buried measurement was chosen because the focus of the fieldwork was to protect the buried aqueduct. However, to compare the PPV of this project with the literature, it is necessary to obtain the PPV at the ground surface and at various depths, with several radial distances from the column. Thus, a numerical model was developed using finite element analysis, calibrated with the field data from this case study.

The model consists of a 2-D assembly, with the dimensions shown in Figure 2. The transverse velocity was disregarded for the consideration of PPV because the literature has shown that this value is usually very small and a 2-D analysis is adequate (Hamidi, Rooz and Pourjenabi 2018).

Four types of elements were used:

i. plane strain quadrilateral elements with reduced integration and hourglass control for the soil portion;
ii. discrete rigid structure to model the aqueduct;
iii. analytical rigid wire structure to model the sleeves;
iv. infinite elements on the side boundaries to absorb the dynamic waves.

The terminologies of these elements are described in the computer software (Abaqus) used in this study. Mesh size varied from 0.25 m to 0.5 m, with the smaller elements placed between the column and the aqueduct.

The entire removal of the sleeves was modelled, but because of complex mesh distortions, some strategies were necessary. First, the sleeves were already pre-embedded in the soil, similar to the technique used by Ramshaw (2002). Second, the Arbitrary Lagrangian-Eulerian (ALE) adaptive technique was used around the casing, 3 m from the outer portion, to maintain the quality of the mesh even during large deformations. This is similar to the technique used by Ekanayake et al. (2013). Third, a 1.0 mm gap was left between the sleeves and the rockfill (Figure 2), to reduce element distortion during removal as recommended by Henke and Grabe (2006). This gap also eases the process of assigning a 0.2 friction coefficient between the sleeves and the soil/rockfill. To verify the applicability of these strategies, a simpler model was calibrated using the information provided by Ekanayake et al. (2012), detailed in Nobre et al. (2021).

An elasto-plastic model was used for the soil to account for potential small to large strains. The Mohr-Coulomb model is very sensitive to large localized loads; hence, elastic perfectly-plastic materials with Von-Mises yield criterion were used, similar to Ekanayake et al. (2013). The soil model parameters used are shown in Table 2.

Table 2. Material Properties.

Properties	Rockfill	Till	Residual clay	Alluvial clay
ρ (kg/m^3)	1938	1920	1846	1846
E (MPa)	22	120	5	18
ν	0.2	0.3	0.41	0.41
Su (kPa)	-	-	18	20
K_{sat} (m/s)	$1x10^{-1}$	$1x10^{-6}$	$5x10^{-9}$	$1x10^{-7}$

The numerical model simulated the fieldwork. In the field, the sleeves were pulled with a vibrodriver applying a harmonic wave of 28 Hz and an upward pulling force of 667 kN. The removal of the bottom sleeves was completed in 12 minutes, and the top sleeves in 6 minutes. In the numerical model, the same rate of removal was used (12 and 6 minutes), and the pulling force and wave were applied to the node at the top of the sleeves.

The two vibrating monitors present in the field were considered in this case study. The projections of their locations are shown as points A and B in the numerical model (Figure 2). Particle velocities recorded in the field monitors were used to calibrate the numerical model at points A and B. The calibration procedure and results are provided in Nobre et al. (2021).

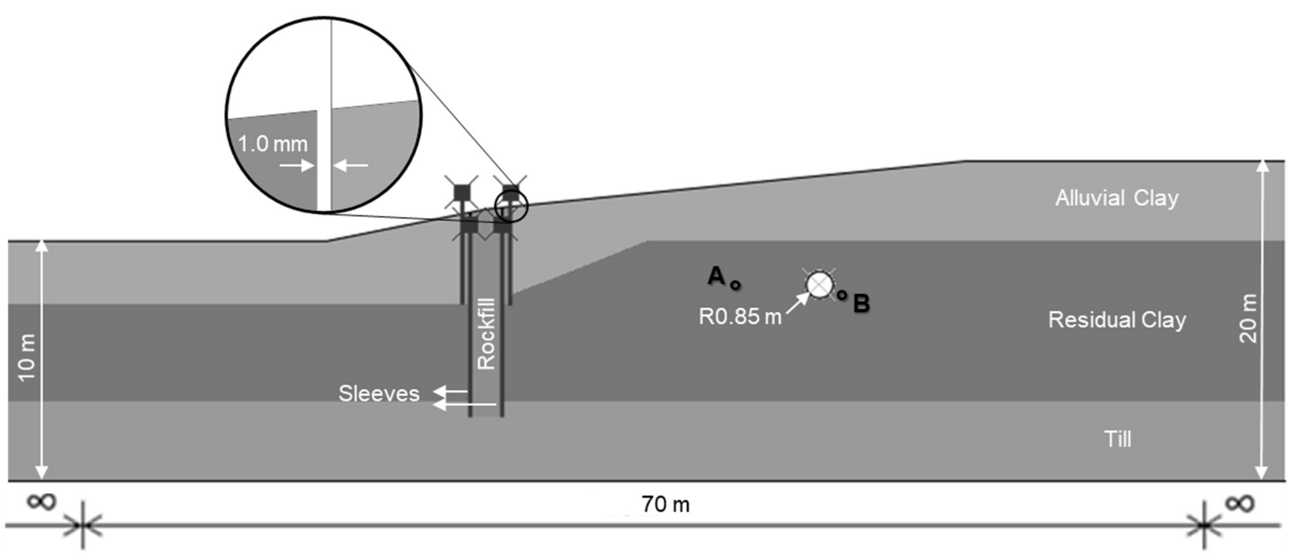

Figure 2. Model assembly.

3 PPV WITH DISTANCE FROM THE COLUMN

With the calibrated numerical model, it was possible to extract the PPVs at other nodes in the model, besides points A and B (Figure 2). Figure 3 shows the locations of the nodes used to obtain the PPV versus distance and at several depths. For each node, the data were manually analyzed and only the highest particle velocity (PPV) was selected. This was a very time-demanding process, which forced the authors to select only the discrete nodes shown in Figure 3.

The intervals between nodes were of approximately one column diameter (3.0 m), to a maximum distance of ten diameters from the edge of the rockfill column. Depth-wise, the selected nodes were located at each of the three soil layers modelled (till, residual clay, and alluvial clay), at the ground surface, and at the transition between layers.

3.1 PPV in various depths and limiting PPV

The numerical response of the PPV with distance and depth is shown in Figure 4. Limiting values of PPV for buried structures based on various guidelines are also indicated.

An overall analysis of Figure 4 indicates that there is a tendency for the PPV to decay with the distance, followed by an increase around and after the location of the aqueduct. This is consistent with the literature: waves tend to attenuate with distance, but they may increase due to reflection and refraction interactions of the waves when they strike boundaries between different materials (Kramer 1996).

Figure 4 also shows that the PPV at the greater depths was the highest, especially closer to the vibration source (column edge). These results, at least for the till layer, are compatible with the parametric studies done by Ramshaw

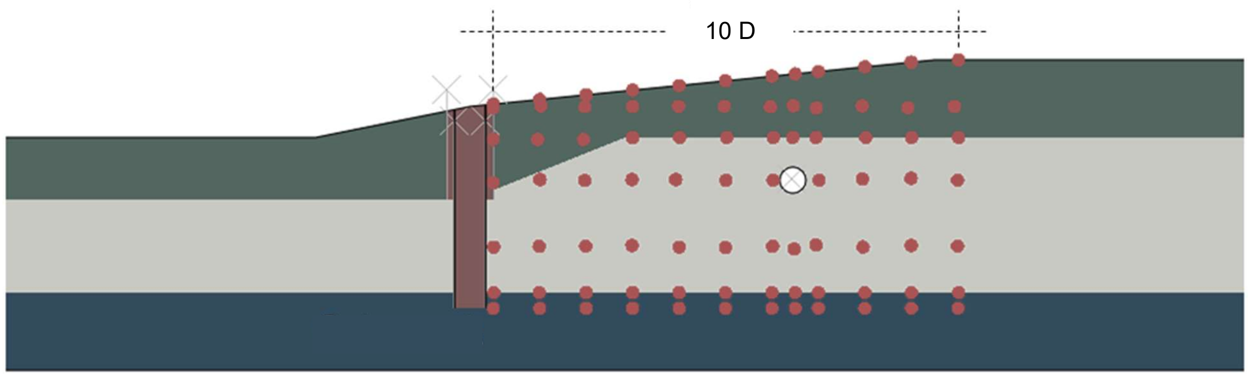

Figure 3. Locations for PPV with distance.

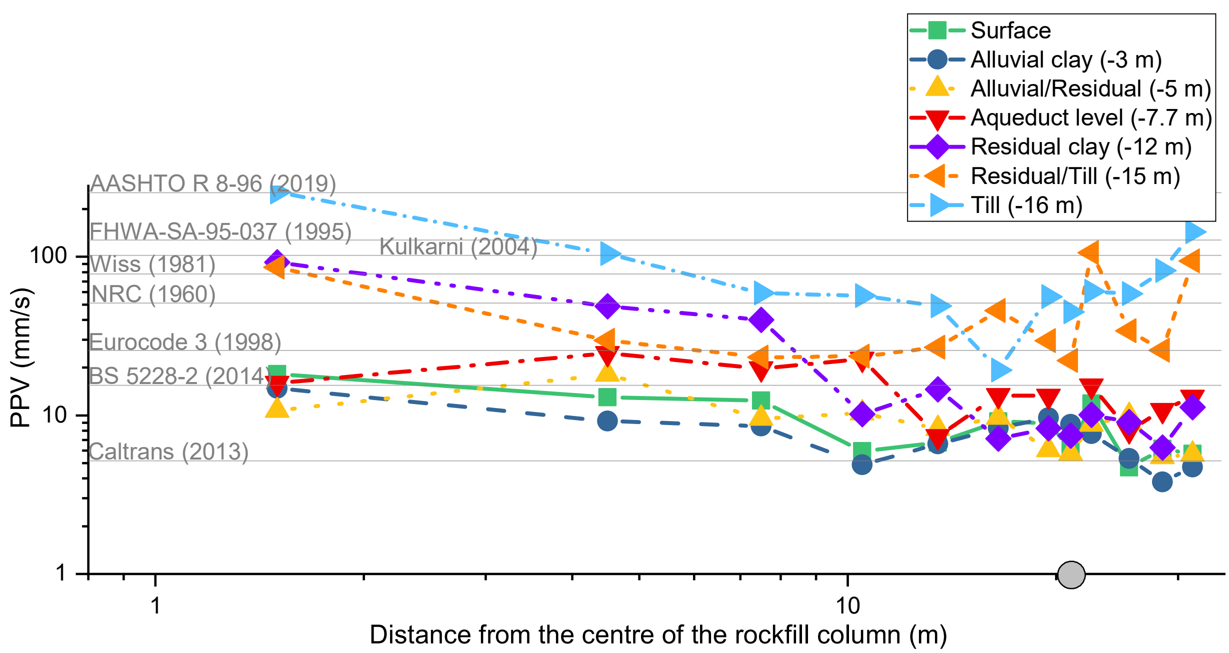

Figure 4. PPV with distance and depth – from the numerical model.

(2002) and Ekanayake (2014). According to their studies, higher PPV occurs closer to the source for the soils with higher elasticity modulus, meaning stiffer soils. Another reason for the higher PPV at the bottom three depths analyzed is that all these PPVs were a result of the removal of the lower sleeves of the column. The lower sleeves were located from the bottom of the rockfill column (at the depth of -16 m from the ground surface, in the till) to the depth of -9 m from the ground surface, exactly within the region where the lower three analysis' layers are selected.

In Figure 4, the PPV values at the aqueduct level were below the 25 mm/s limit recommended by Eurocode 3 (1998). However, they are above the target limit of 10 mm/s agreed by the design engineer and the client for the fieldwork. Note that if the client had agreed to use the limiting value proposed in the AASHTO R 8-96 (2019) of 250 mm/s, all PPVs from the numerical model would be below this recommended value.

3.2 PPV at the ground surface and predictions

The methods of predictions available in the literature and used in this case study are presented in Table 3. In this table, PPV_G is the PPV at the ground surface; W is the source's energy in Joules; r is the radial distance between the vibrodriver and the point of interest at the ground surface, in metres; and k and n are constants determined empirically. Note that $W = \frac{P}{f}$, which is the energy of the vibrodriver per cycle. In this case study, for the vibrodriver used in the field, and simulated in the numerical model, the power of the vibrodriver (P) is 563 kW, and the frequency (f) is 28 Hz.

Figure 5 shows the values of the PPVs at the ground surface obtained in the numerical model of this paper, with the data of Chan et al. (2011) for the installation of stone columns. Figure 5 also includes the curves of the prediction equations from the literature (from Table 3).

In Figure 5, the PPVs of this case study follow the overall trend shown in Chan et al. (2011). Comparing the numerical modelling results with the prediction curves, that of Attewell (1995) fitted the average PPV values better than Hiller and Crabb (2000). Note that Attewell's equation was developed for vibratory pile driving.

Table 3. Predicting equations of PPV.

Purpose and Reference	Equation	Comments
PPV for vibratory pile driving (Attewell 1995)	$\mathrm{PPV}_G = k\left(\frac{\sqrt{W}}{r}\right)^n$	For high level of confidence that predicted values will not be exceeded: k=18; n=1
PPV for vibrated stone columns (Hiller and Crabb 2000)	$\mathrm{PPV}_G = \frac{k}{r^{1.4}}$	This equation is valid for $8 \leq r \leq 100$ m. For confidence of 95%: k=95

It is important to understand that the predictions of Attewell (1995) and Hiller and Crabb (2000) were developed to not be exceeded by 5% of the PPV values at the ground surface. Figure 5 shows that Hiller and Crabb was exceeded by about 90% of the PPV values, and Attewell was exceeded by about 40%. Therefore, none of them predicted the PPVs for stone columns appropriately.

For the case specific of the data shown (Chan et al. 2011; and the PPVs from the numerical results of the rockfill column of this study), the authors of this paper propose that PPV at the ground surface may be empirically approximated with the equation shown in Table 4. For PPV at the ground surface to not be exceeded by 5%, 50% and 95% of the values, the coefficients to be used in the equation are also shown in Table 4. The data with the fitted curves (5%, 50% and 95%) are shown in Figure 6.

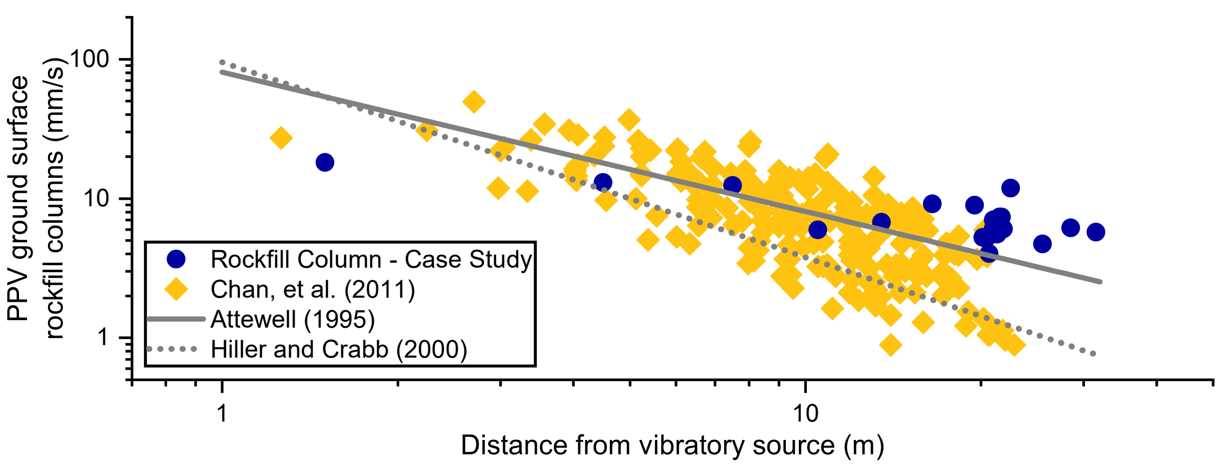

Figure 5. PPV at the ground surface, data from the literature, and predictors from the literature for the installation of stone columns.

Table 4. Predicting equation of PPV for the data studied.

Equation	Probability of PPVs greater than curve	k	n
	95%	55.0	1.6
$\mathrm{PPV}_G = \frac{k}{r^n}$	50%	63.6	1.0
	5%	75.5	0.6

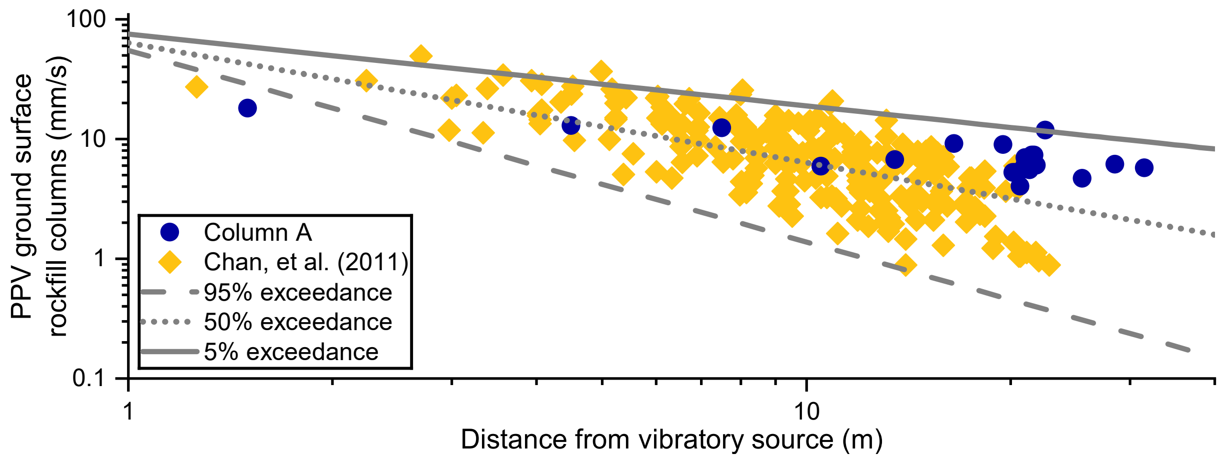

Figure 6. PPV at the ground surface, data from the literature, and predictors from this study for the installation of stone columns.

4 CONCLUSIONS

Rockfill columns were installed in the field in Winnipeg, Manitoba and ground vibration generated during installation was monitored. The ground vibrations in terms of particle velocities were used to calibrate a finite element model, where values of Peak Particle Velocity (PPV) were obtained at various locations in the ground.

The results show that stiffer soils have higher values of PPV and the presence of a buried aqueduct in the region also affected the ground vibrations. There is a tendency for the PPV to reduce with increasing distance from the centre of the rockfill column, but there is an increase in values around the aqueduct. This is consistent with the literature that shows that waves tend to dissipate with the distance but change forms when interacting with the boundary of different materials (Kramer 1996).

The literature is not consistent about the limiting values of PPV to minimize the risk of damaging buried structures. The PPVs obtained in the numerical analysis show that all vibrations would pass the guideline based on AASHTO R 8-96 (2019) of 250 mm/s. This value is 25 times greater than the PPV agreed with the client for the field installation of this rockfill column.

The PPVs at the ground surface show a good agreement with the data provided by Chan et al. (2011) for the installation of stone columns. However, the PPVs of those authors and this research are greater than some of predicting equations available in the literature. A new predicting equation is proposed in this paper.

ACKNOWLEDGEMENTS

The authors would like to thank the City of Winnipeg and KGS Group for the technical support, and the University of Manitoba for the academic and financial support.

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